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PROCEEDINGS OF THE GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM

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FOREWORD

The complex nature of man encompasses a desire for the gifts of peace, the will to wage war, and magnanimity toward his fellow in misfortune. As institutions and governments of man reflect these traits, the Office of Foreign Disaster Assistance of the Agency for International Development acts to carry out the deep-seated humanitarian concern that the people of the United States have for the unfortunate victims of natural and manmade calamities in other nations.

In the case of natural disasters, it is now widely recognized that mitigative and preparatory steps taken before the event are the key to the ability of a region or a community to survive and restore its normal functions within a short time after the disaster. The Office of Foreign Disaster Assistance recognized this fact and provided funds to the Geological Survey of the United States to conduct a training course on geologic and hydrologic hazards reduction for scientists and officials from other countries. Those countries invited to participate either have experienced or may be subject to the ravages of floods, earthquakes, volcanic eruptions, and landslides. This document is a report of that course.

The purpose of the course was to provide a vehicle for the diffusion of knowledge and experience gained in the United States through hazard reduction programs and in response to actual events. The course addressed basic questions: what are the physical effects of these events, how can these effects be avoided, and how can these effects be reduced if not avoided. We believe that the training course advanced the complex process of information gathering, research, and analysis that will answer these questions for each participant.

We hope that this effort ultimately will lead to actions that reduce loss and suffering in hazard-prone regions and increase the spirit of goodwill between nations.

John R. Filson
Chief, Office of Earthquakes,
Volcanoes, and Engineering

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March 5-30, 1984

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INTEGRATION, ANALYSIS, AND EVALUATION OF HAZARD DATA

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INTRODUCTION

Estimation of seismic hazard requires careful integration, analyses, and evaluation of all useful available data. A wide range of geological and geophysical data are potentially useful in earthquake hazard assessment but very commonly many kinds of useful data are not available. For example, careful geologic investigations aimed at the discovery of Holocene or younger fault breaks have been undertaken in only a few areas of the world. Earthquake catalogs are frequently quite incomplete except for large shocks and the geotechnical properties of the shallow (<500 meters) materials underlying a region or site are generally not well known. However, a hazard evaluation is always possible, even with limited data. Perhaps the most important factor in developing a meaningful hazard evaluation is the skillful use of the existing data.

HAZARD DATA

The geological and geophysical data available for hazard estimation must be carefully reviewed. Emphasis here is on the word available. The type and quality of the information will control the reliability of the hazard evaluation. The following is a listing of data that, ideally, are needed for hazard evaluation.

1. Seismicity

A. Earthquake Catalog - A reliable earthquake catalog containing the hypocenter, size, and description of the distribution of shaking (usually given in terms of intensity) of the earthquakes. Ideally, additional data such as the focal mechanism, evidence of surface rupture (or lack of it), seismic moment and/or magnitude should also be available.

B. Geological evidence of earthquake activity (fault slip data).

2. Seismotectonic features

Careful geologic mapping, particularly of Quaternary geology, with particular emphasis on historic and Holocene faulting, should be assembled if available. Division of areas into seismotectonic provinces is desirable (see references in paper by Thenhaus). Many types of geophysical data such as high resolution seismic reflection surveys, gravity and magnetic maps, and seismic refraction surveys may be useful in delineating seismotectonic features or zones.

3. Seismic wave attenuation

The important sources of seismic wave attenuation data are intensity observations and instrumental strong motion records.

4. Site response

Site response information can be obtained from intensity data. It may also be obtained on a limited basis from: (a) strong motion records; (b) special instrumental site response studies; and (c) from investigation of the geotechnical properties of materials at shallow depths (generally less than 500 meters) underlying the site or area of interest.

5. Potential for ground failure

Potential for soil liquefaction and or landsliding should be collected.

HAZARD ANALYSIS

Two important questions need to be addressed at the onset of a hazard analysis.

1. What are the objectives of the analysis? It is very important to identify the purpose of the analysis, because different objectives require different types of input data and varying levels of analysis and sophistication in the results. For example, a hazard analysis aimed at improving the lateral force requirements of a building code may be quite different than an analysis to be used for disaster mitigation or for nuclear power plant siting.

2. Acceptable risk and its relation to hazard analysis. Hazard analysis generally has associated with it the implicit idea that the hazard analysis may be used to aid in the determination of the acceptable risk. Acceptable risk is the amount or level of loss that society, a group, an individual, etc., is willing to sustain without taking some additional action to mitigate the risk. For example, an individual might continue to live in a highly seismic region as long as his home is not heavily damaged by earthquakes (acceptable risk), or he may seek professional engineering advice to strengthen his home if is has been seriously damaged as a result of an earthquake (unacceptable risk). Risks are often evaluated as acceptable or unacceptable as a result of the effects of one earthquake, several earthquakes over a short period of time, average losses over a long period of time, or as a result of seismic hazard and risk assessments based on geophysical and geological data.

   It seems clearly desirable to form an opinion concerning acceptable risk based upon scientific and engineering evaluation of the available data before unacceptable losses are experienced. In this way unacceptable risks can be mitigated through improved building codes, disaster mitigation practices, land use planning, etc.

3. Parameters for Estimation of Ground Motion

   A. Seismicity
      1) Earthquake size
         a) Magnitude
            $M = \log_{10} A+G$ (r,h,f, wave types) where M is magnitude, A is amplitude of the ground motion at a distance r produced by a

particular type of wave with frequency f. h is the focal depth and G takes into account the attenuation of the wave being considered as well as the source spectral amplitude. Various magnitude scales used (M_L Ms, m_b, etc).

b) Intensity

Description of effects classified according to intensity of shaking (various scales, i.e., Modified Mercalli, MSK, Japanese, etc).

c) Moment

$M_o=\mu\Delta$ d where M_o=moment; μ is the rigidity modulus of the crust, Δ is the areas of the ruptured fault surface, and d is the average displacement of the fault surface.

2) Spatial distribution of earthquakes
   a) epicentral maps with indications of depths
   b) "energy" maps

3) Rate of occurrence of earthquakes
   a) log N(M) = a-bM (Figure 1)

   where N(M) = number of earthquakes occurring within a region in a given time period with magnitude (M) greater than or equal to M. a and b are constants to be determined. N(M) is sometimes defined as the number of earthquakes in a given magnitude range, rather than the cumulative number of earthquakes.

   b) Determination of a and b
   c) Problems of catalog incompleteness (Figure 2)
   d) Incorporation of geologic evidence of earthquake activity (Figure 3)
   e) M - I (intensity) relationships (Figure 4)

4) Mode of faulting

Effect on attenuation

5) Fault rupture length (Figure 5)

6) Determination of the upper bound magnitude

B. Seismotectonics

Delineation of seismic source zones (Figure 6)

C. Attenuation

Determination of (Figures 7, 8, and 9) an appropriate ground motion parameter (acceleration, velocity, displacement, intensity, response spectrum)

D. Site Response

E. Ground failure potential

F. Types of Hazard Analysis (Figures 10 and 11, Table 1)

1) Deterministic Model
   - a) Single event (Figure 12 and 13)
   - b) Composite events
   - c) Maximum ground motion from an ensemble of events

2) Probabilistic Model
   - a) Poisson Model (Figures 14, 15, and 16)
     - Assumptions
     - Applications
   - b) Time dependent models
     - Assumptions
     - Applications

GLOSSARY OF TERMS FOR PROBABILISTIC SEISMIC-RISK AND HAZARD ANALYSIS

ACCEPTABLE RISK - a probability of social or economic consequences due to earthquakes that is low enough (for example in comparison with other natural or manmade risks) to be judged by appropriate authorities to represent a realistic basis for determining design requirements for engineered structures, or for taking certain social or economic actions.

ACTIVE FAULT - a fault that on the basis of historical, seismological, or geological evidence has a high probability of producing an earthquake. (Alternate: a fault that may produce an earthquake within a specified exposure time, given the assumptions adopted for a specific seismic-risk analysis.)

ATTENUATION LAW - a description of the behavior of a characteristic of earthquake ground motion as a function of the distance from the source of energy.

B-VALUE - a parameter indicating the relative frequency of occurrence of earthquakes of different sizes. It is the slope of a straight line indicating absolute or relative frequency (plotted logarithmically) versus earthquake magnitude or meizoseismal Modified Mercalli intensity. (The B-value indicates the slope of the Gutenberg-Richter recurrence relationship.)

COEFFICIENT OF VARIATION — the ratio of standard deviation to the mean.

DAMAGE - any economic loss or destruction caused by earthquakes.

DESIGN ACCELERATION - a specification of the ground acceleration at a site, terms of a single value such as the peak or rms; used for the earthquake-resistant design of a structure (or as a base for deriving a design spectrum). See "Design Time History."

DESIGN EARTHQUAKE - a specification of the seismic ground motion at a site; used for the earthquake-resistant design of a structure.

DESIGN EVENT, DESIGN SEISMIC EVENT - a specification of one or more earthquake source parameters, and of the location of energy release with respect to the site of interest; used for the earthquake-resistant design of a structure.

DESIGN SPECTRUM - a set of curves for design purposes that gives acceleration velocity, or displacement (usually absolute acceleration, relative velocity, and relative displacement of the vibrating mass) as a function of period of vibration and damping.

DESIGN TIME HISTORY - the variation with time of ground motion (e.g., ground acceleration or velocity or displacement) at a site; used for the earthquake-resistant design of a structure. See "Design Acceleration."

DURATION - a qualitative or quantitative description of the length of time during which ground motion at a site shows certain characteristics (perceptibility, violent shaking, etc.).

EARTHQUAKE - a sudden motion or vibration in the earth caused by the abrupt release of energy in the earth's lithosphere. The wave motion may range from violent at some locations to imperecptible at others.

ELEMENTS AT RISK - population, properties, economic activities, including public services etc., at risk in a given area.

EXCEEDENCE PROBABILITY - the probability that a specified level of ground motion or specified social or economic consequences of earthquakes, will be exceeded at the site or in a region during a specified exposure time.

EXPECTED - mean, average.

EXPECTED GROUND MOTION - the mean value of one or more characteristics of ground motion at a site for a single earthquake. (Mean ground motion.)

EXPOSURE - the potential economic loss to all or certain subset of structures as a result of one or more earthquakes in an area. This term usually refers to the insured value of structures carried by one or more insurers. See "Value at Risk."

EXPOSURE TIME - the time period of interest for seismic-risk calculations, seismic-hazard calculations, or design of structures. For structures, the exposure time is often chosen to be equal to the design lifetime of the structure.

GEOLOGIC HAZARD - a geologic process (e.g., landsliding, liquefaction soils, active faulting) that during an earthquake or other natural event may produce adverse effects in structures.

INTENSITY - a qualitative or quantitative measure of the severity of seismic ground motion at a specific site (e.g., Modified Mercalli intensity, Rossi-Forel intensity, Housner Spectral intensity, Arias intensity, peak acceleration, etc.).

LOSS - any adverse economic or social consequence caused by one or more earthquakes.

MAXIMUM - the largest value attained by a variable during a specified exposure time. See "Peak Value."

MAXIMUM CREDIBLE
MAXIMUM EXPECTABLE
MAXIMUM EXPECTED
MAXIMUM PROBABLE

These terms are used to specify the largest value of a variable, for example, the magnitude of an earthquake, that might reasonably be expected to occur. In the Committee's view, these are misleading terms and their use is discourage. (The U.S. Geological Survey and some individuals and companies define the maximum credible earthquake as "the largest earthquake that can be reasonably expected to occur." The Bureau of

Reclamation, the First Interagency Working Group (Sept. 1978) defined the maximum credible earthquake as "the earthquake that would cause the most severe vibratory ground motion capable of being produced at the site under the current known tectonic framework." It is an event that can be supported by all known geologic and seismologic data. The maximum expectable or expected earthquake is defined by USGS as "the largest earthquake that can be reasonably expected to occur." The maximum probable earthquake is sometimes defined as the worst historic earthquake. Alternatively, it is defined as the 100-year-return-period earthquake, or an earthquake that probabilistic determination of recurrence will take place during the life of the structure.)

MAXIMUM POSSIBLE - the largest value possible for a variable. This follows from an explicit assumption that larger values are not possible, or implicitly from assumptions that related variables or functions are limited in range. The maximum possible value may be expressed deterministically or probabilistically.

MEAN RECURRENCE INTERVAL, AVERAGE RECURRENCE INTERVAL - the average time between earthquakes or faulting events with specific characteristics (e.g., magnitude $\geq$ 6) in a specified region or in a specified fault zone.

MEAN RETURN PERIOD - the average time between occurrences of ground motion with specific characteristics (e.g., peak horizontal acceleration $\geq$ 0.1 g) at a site. (Equal to the inverse of the annual probability of exceedance.)

MEAN SQUARE - expected value of the square of the random variable. (Mean square minus square of the mean gives the variance of random variable.)

PEAK VALUE - the largest value of a time-dependent variable during an earthquake.

RESPONSE SPECTRUM - a set of curves calculated from an earthquake accelerogram that gives values of peak response of a damped linear oscillator, as a function of its period of vibration and damping.

ROOT MEAN SQUARE (rms) - square root of the mean square value of a random variable.

SEISMIC-ACTIVITY RATE - the mean number per unit time of earthquakes with specific characteristics (e.g., magnitude $\geq$ 6) originating on a selected fault or in a selected area.

SEISMIC-DESIGN-LOAD EFFECTS - the actions (axial forces, shears, or bending moments) and deformations induced in a structural system due to a specified representation (time history, response spectrum, or base shear) of seismic design ground motion.

SEISMIIC-DESIGN LOADING - the prescribed representation (time history, response spectrum, or equivalent static base shear) of seismic ground motion to be used for the design of a structure.

SEISMIC-DESIGN ZONE - seismic zone.

SEISMIC EVENT - the abrupt release of energy in the earth's lithosphere, causing an earthquake.

SEISMIC HAZARD - any physical phenomenon (e.g., ground shaking, ground failure) associated with an earthquake that may produce adverse effects on human activities.

SEISMIC RISK - the probability that social or economic consequences of earthquakes will equal or exceed specified values at a site, at several sites, or in an area, during a specified exposure time.

SEISMIC-RISK ZONE - an obsolete term. See "Seismic Zone."

SEISMIC-SOURCE ZONE - an obsolete term. See "Seismogenic Zone" and "Seismotectonic Zone."

SEISMIC ZONE - a generally large area within which seismic-design requirements for structures are constant.

SEISMIC ZONING, SEISMIC ZONATION - the process of determining seismic hazard at many sites for the purpose of delineating seismic zones.

SEISMIC MICROZONE - a generally small area within which seismic-design requirements for structures are uniform. Seismic microzones may show relative ground motion amplification due to local soil conditions without specifying the absolute levels of motion or seismic hazard.

SEISMIC MICROZONING, SEISMIC MICROZONATION - the process of determining absolute or relative seismic hazard at many sites, accounting for the effects of geologic and topographic amplification of motion and of seismic microzones. Alternatively, microzonation is a process for identifying detailed geological, seismological, hydrological, and geotechnical site characteristics in a specific region and incorporating them into land-use planning and the design of safe structures in order to reduce damage to human life and property resulting from earthquakes.

SEISMOGENIC ZONE, SEISMOGENIC PROVINCE - a planar representation of a three-dimensional domain in the earth's lithosphere in which earthquakes are inferred to be of a similar tectonic origin. A seismogenic zone may represent a fault in the earth's lithosphere. See "Seismotectonic Zone."

SEISMOGENIC ZONING - the process of delineating regions having nearly homogeneous tectonic and geologic character, for the purpose of drawing seismogenic zones. The specific procedures used depend on the assumptions and mathematical models used in the seismic-risk analysis or seismic-hazard analysis.

SEISMOTECTONIC ZONE, SEISMOTECTONIC PROVINCE - a seismogenic zone in which the tectonic processes causing earthquakes have been identified. These zones are usually fault zones.

SOURCE VARIABLE - a variable that describes a physical characteristic (e.g., magnitude, stress drop, seismic moment, displacement) of the source of energy release causing an earthquake.

STANDARD DEVIATION - the square root of the variance of a random variable.

UPPER BOUND - see "Maximum Possible."

VALUE AT RISK - the potential economic loss (whether insured or not) to all or certain subset of structures as a result of one or more earthquakes in an area. See "Exposure."

VARIANCE - the mean squared deviation of a random variable from its average value.

VULNERABILITY - the degree of loss to a given element at risk, or set of such elements, resulting from an earthquake of a given magnitude or intensity, which is usually expressed on a scale from 0 (no damage) to 10 (total loss).

TABLE 1

SUMMARY OF EARTHQUAKE RECURRENCE FORMULAS *

AREA	RISK ZONE	EARTHQUAKES PER 100 YEARS PER 100,000 KM.^{2}			
		V	V1	V11	V111
1. 2. California, Nevada	3,2	300	84.6	23.8	6.72
4. Montana, Idaho, Utah, Arizona	3,2	64.4	17.7	4.89	1.35
3. Puget Sound, Washington	3,2	68.0	16.3	3.92	0.94
8. Mississippi Valley, St. Lawrence Valley	3,2,1	24.2	7.65	2.42	0.76
7. Nebraska, Kansas, Oklahoma	2,1	13.0	4.20	1.35	0.45
5. Wyoming, Colorado, New Mexico	3,2,1	32.8	6.85	1.42	0.31
6. Oklahoma, North Texas	2,1	13.3	3.73	1.07	0.30
9. East Coast	3,2,1	12.8	3.39	0.88	0.23

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* See Figure 10 for location of zone.

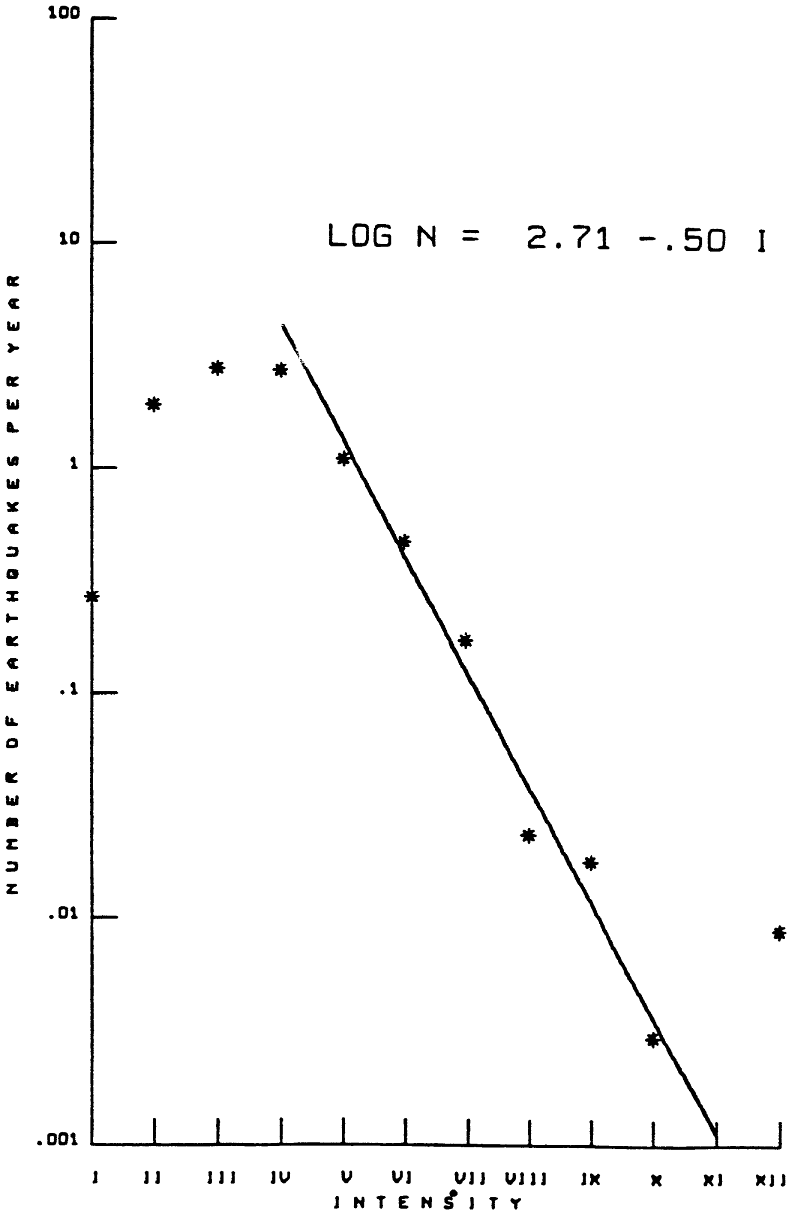


Figure 1. Typical earthquake recurrence curve illustrating incompleteness in the data (Area 8A or 8B or both).

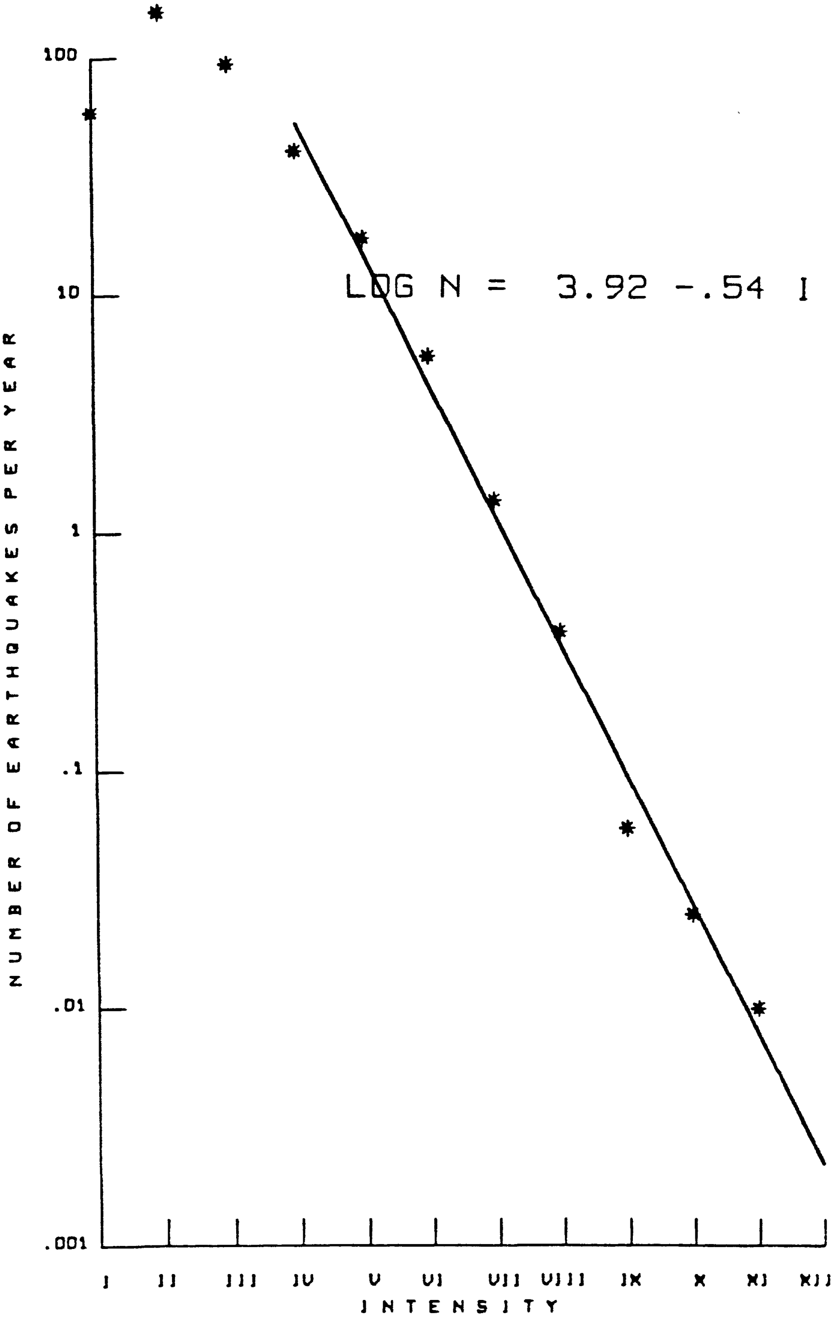
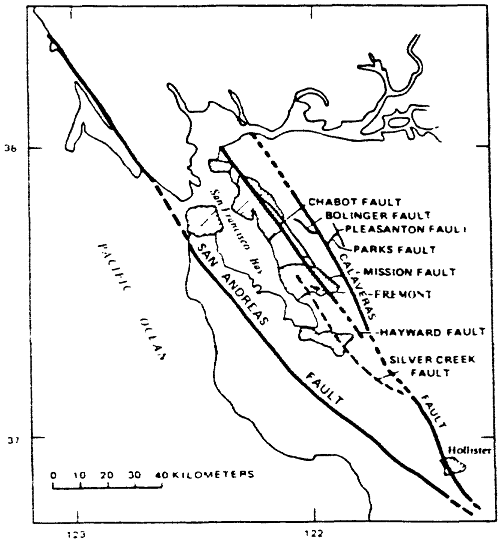


Figure 2. Typical earthquake occurence curve illustrating data from a highly seismic area (Area 1 of Figure 10).

—*Classification of fault activity*

[From Cluff and others (1972)]

	Active	Potentially active	Uncertain activity	Inactive
Historical	Surface faulting and associated strong earthquakes Tectonic fault creep or geodetic indications of movement	No reliable report of historic surface faulting		No historic activity
Geologic	Generally young deposits have been displaced or cut by faulting Fresh geomorphic features characteristic of active fault zones present along fault trace Physical ground-water barriers in geologically young deposits	Geomorphic features characteristic of active fault zones subdued, eroded, and discontinuous Faults are not known to cut or displace the most recent alluvial deposits, but may be found in older alluvial deposits Water barrier may be found in older materials Geologic setting in which the geomorphic relation to active or potentially active faults suggests similar levels of activity	Available information does not satisfy enough criteria to establish fault activity If the fault is near the site, additional studies are necessary	Geomorphic features characteristic of active fault zones are not present, and geologic evidence is available to indicate that the fault has not moved in the recent past
Seismological	Earthquake epicenters are assigned to individual faults with a high degree of confidence	Alinement of some earthquake epicenters along fault trace, but locations have a low degree of confidence		Not recognized as a source of earthquakes



—Principal faults in the vicinity of Fremont, Calif (modified from Cluff and others, 1972)

—*Classification of selected faults relative to Fremont, Calif*

[From Cluff and others (1972)]

Fault	Classification of activity	Criteria
Hayward . .	Active	Historical faulting
Mission . . .	Potentially active	Geologic setting and microearthquake epicenters
Silver Creek	Tentatively active	No geomorphic features, no ground water barrier, no earthquake epicenters
Chabot . . .	Potentially active	Geologic setting
Calaveras . . .	Active	Geomorphic features, historical faulting, and strong earthquakes
Pleasanton .	Active	Documented tectonic fault creep
Bolinger	Potentially active near junction with Calaveras Tentatively inactive away from Calaveras junction	Geologic setting, no characteristic geomorphic features
Parks	Potentially active	Ground-water barrier in Tertiary-Quaternary gravels
San Andreas	Active	Geomorphic features, historic faulting, and strong earthquakes

Figure 3. Classification of Fault Activity (After Hays, 1980).

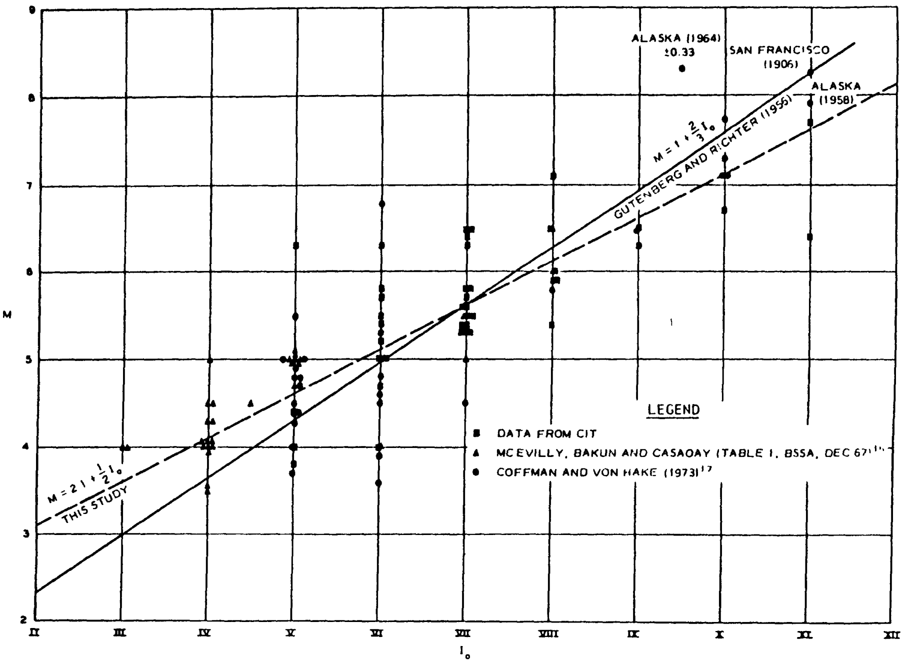


Figure 4. Relation between earthquake magnitude and intensity in Western United States.

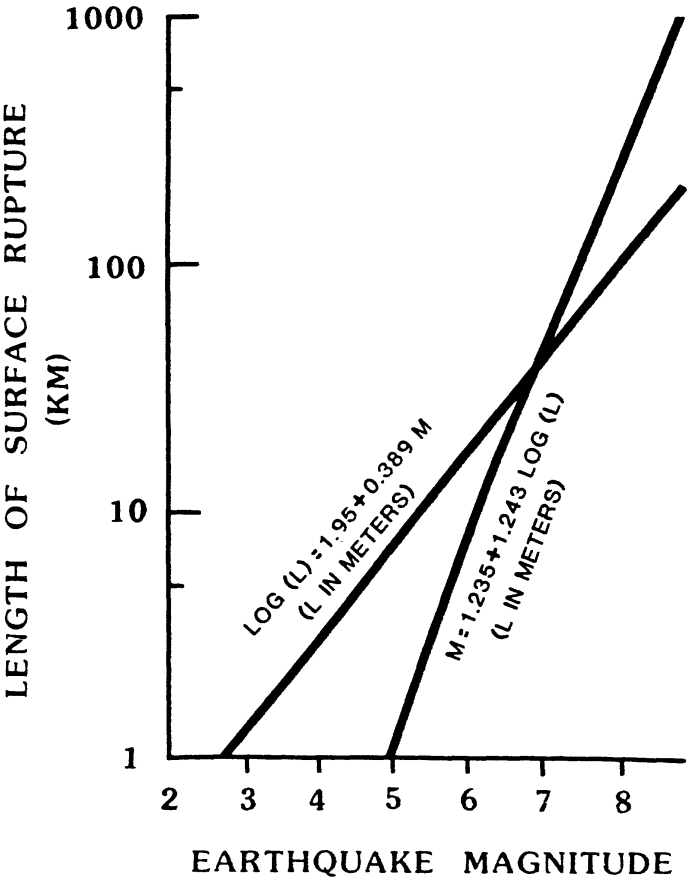


Figure 5. Rupture-magnitude relationships (after Mark, 1977).

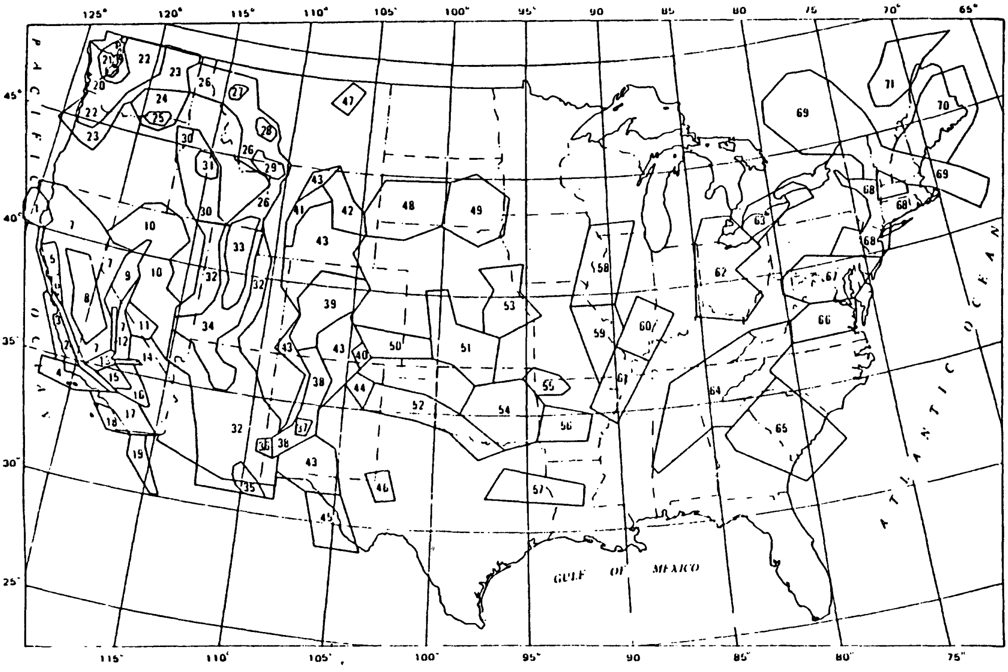


Figure 6. Seismic source zones within the conterminous United States (from Algermissen and Perkins, 1976).

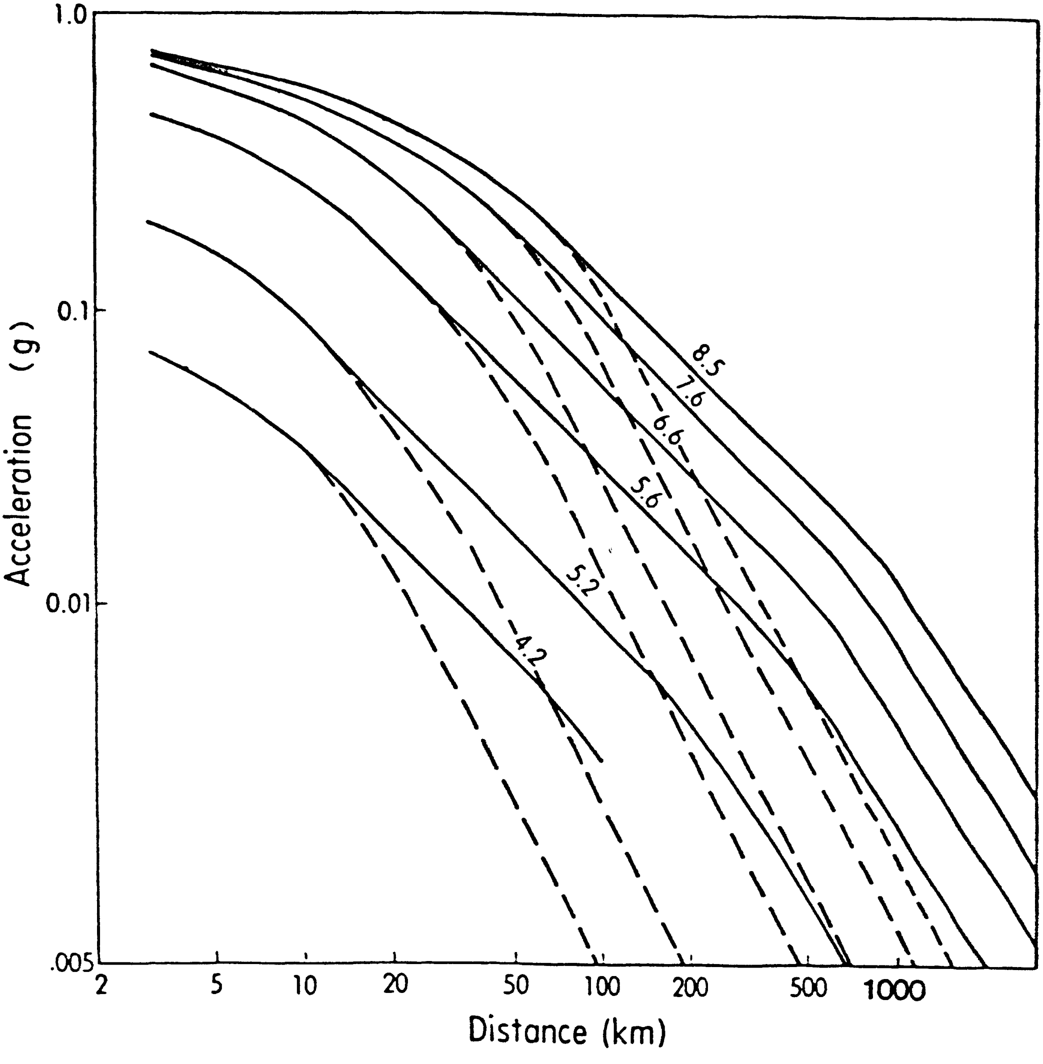


Figure 7. Acceleration attenuation curves (Algermissen and Perkins, 1976). The solid lines are curves used for the eastern region of the United States. The dashed lines together with solid lines at close distances are the attenuation curves used for the western region and are taken from Schnabel and Seed (1973).

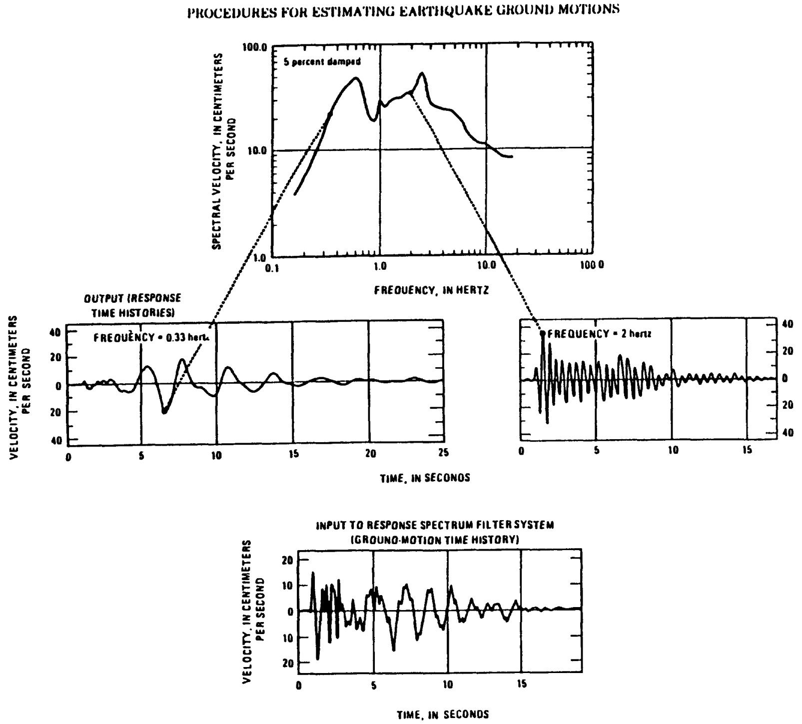


Figure 8. The narrow-band-pass filtering involved in deriving a response spectrum. (After Hays, 1980).

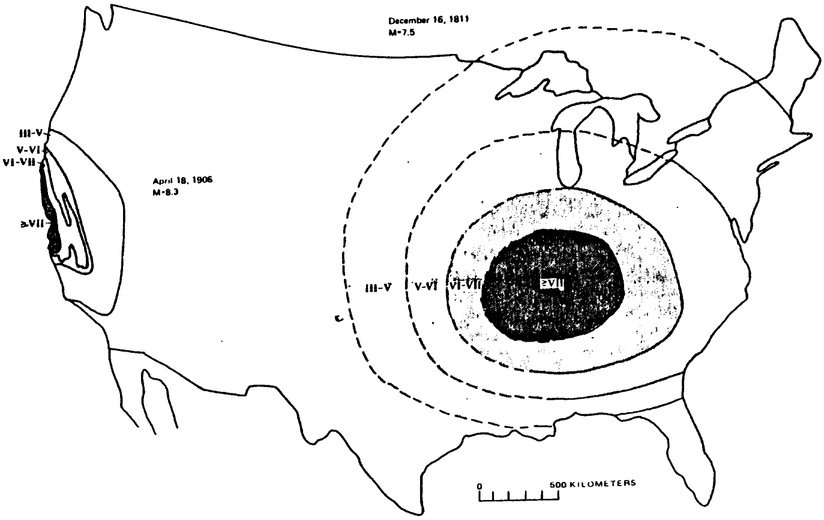


Figure 9. Isoseismal contours for 1906 San Francisco and 1811 New Madrid earthquakes (modified from Nuttli, 1973).

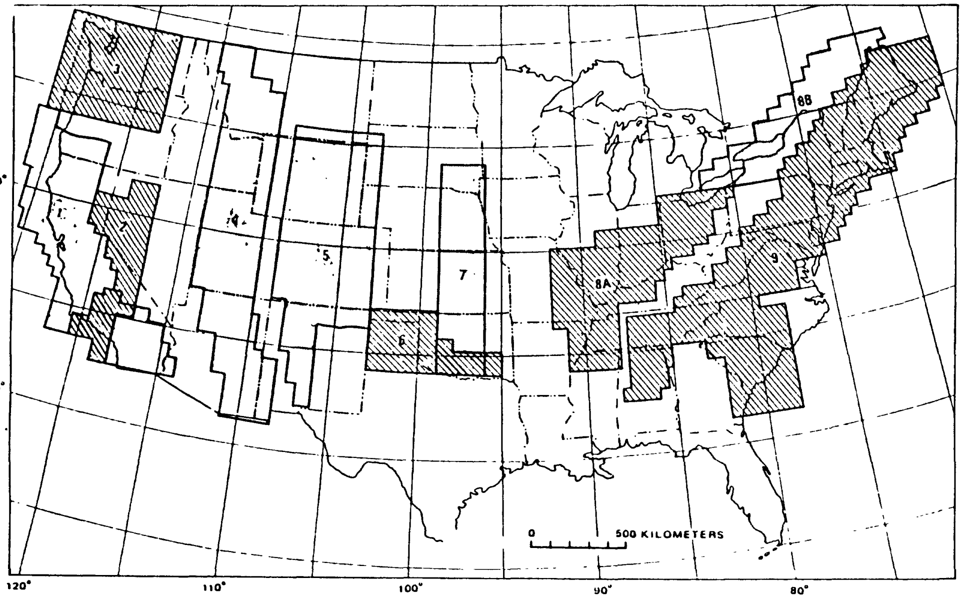


Figure 10. Geographic areas of the conterminous United States where regional seismicity studies have been made (modified from Algermissen, 1969). Numbered areas are described in Table 1. Shading depicts boundaries of regions.

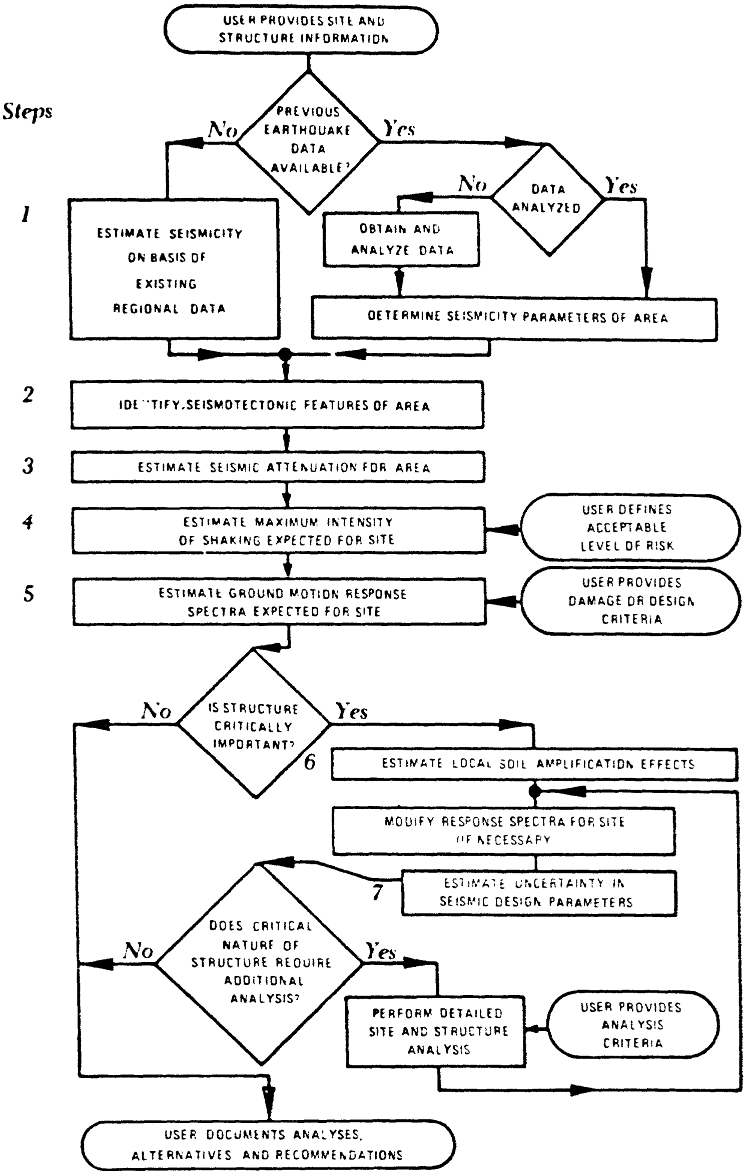


Figure 11. Steps in estimating ground motion for design of earthquake-resistant structures.

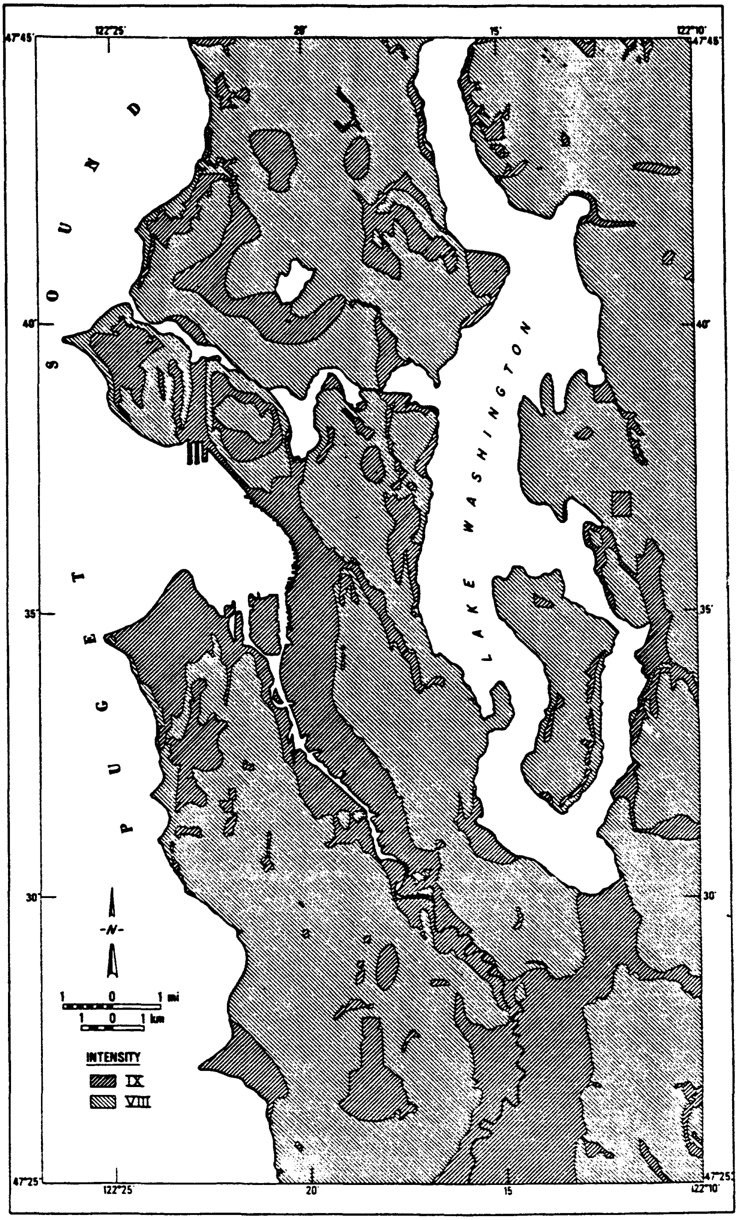


Figure 12. Map showing estimated Modified Mercalli intensity distribution in Seattle for a projected earthquake of magnitude 7.5 located at 47°30' N, 122°12' W, 50 km deep, and example of hazard assessment for a single, hypothesized earthquake.

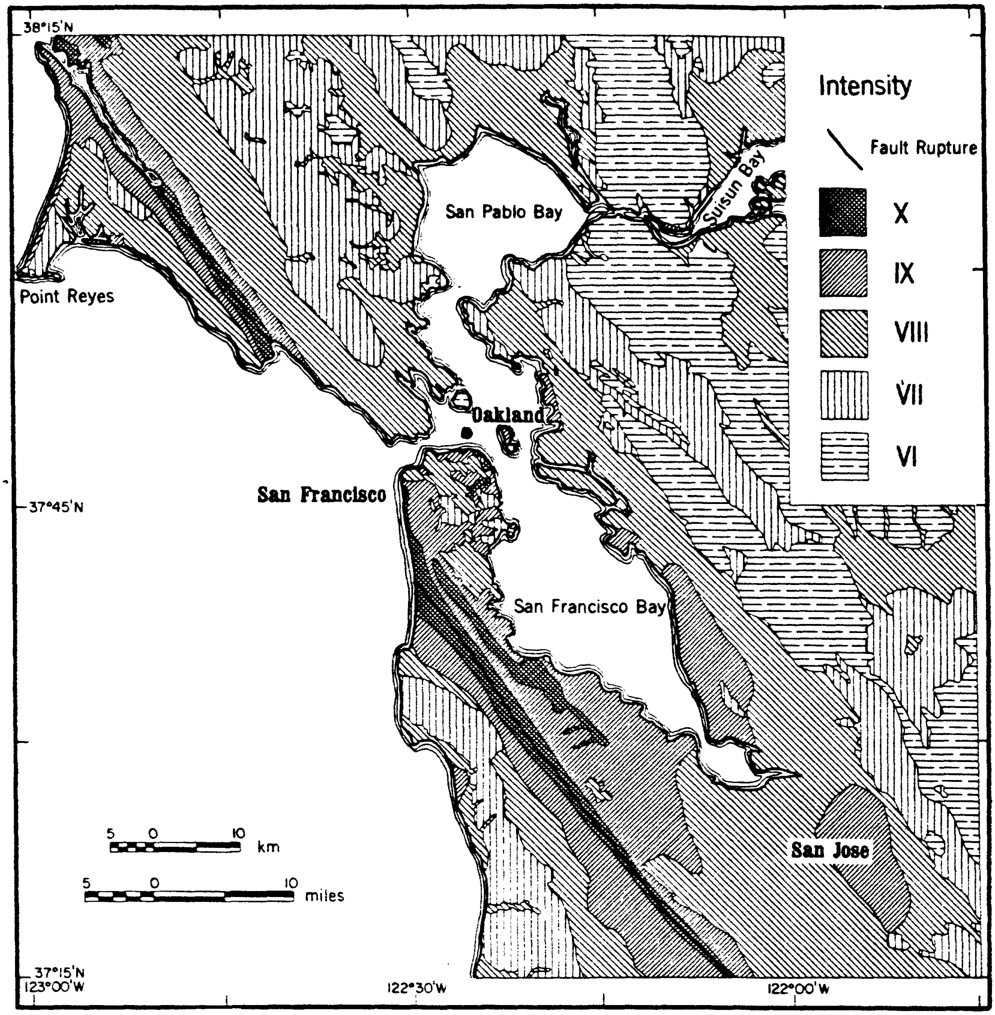


Figure 13. Intensity distribution from a magnitude 8.3 earthquake on the San Andreas fault.

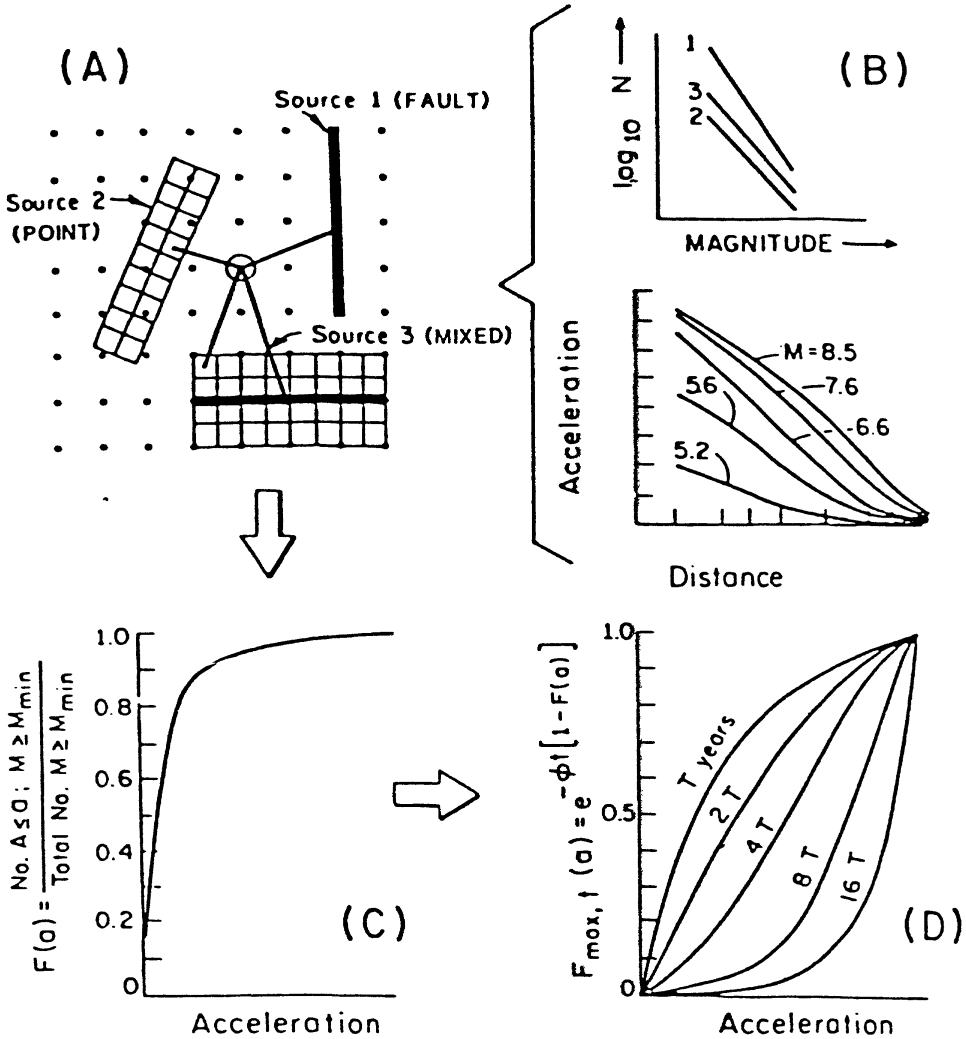
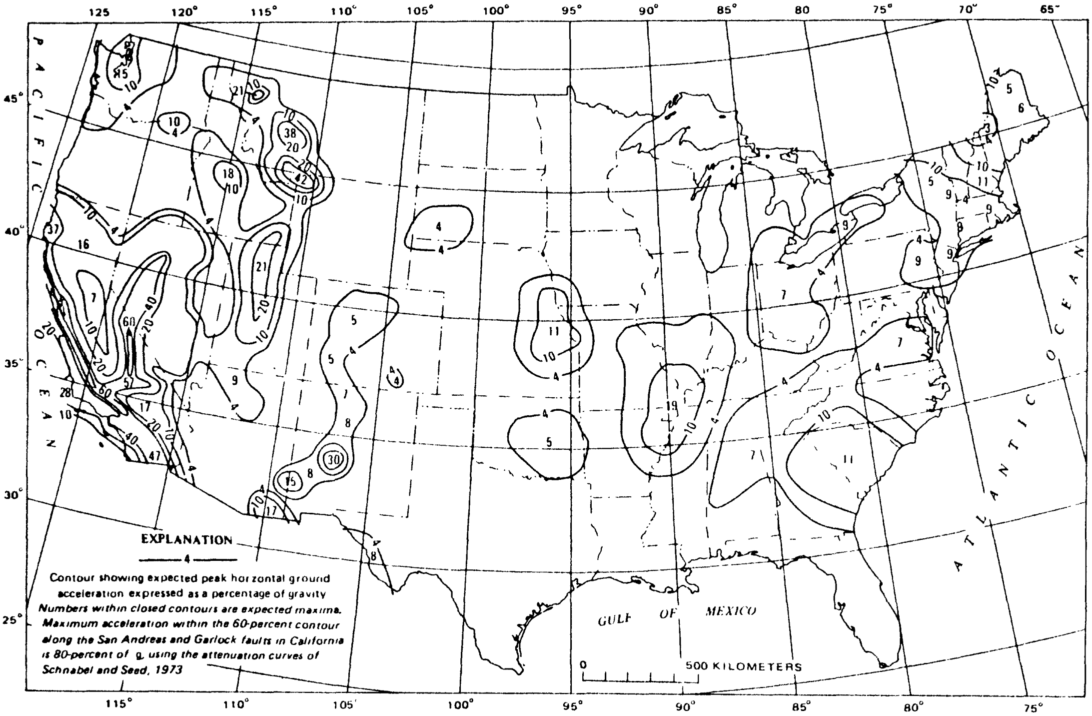


Figure 14. Schematic diagram of a typical probabalistic ground motion calculation. a) Typical source areas and grid of points at which the hazard is to be computed. b) Statistical analysis of seismicity data and typical attenuation curves. c) Cumulative conditional probability distibution of acceleration. d) The extreme probability $F_{max, t}$ (a) for various accelerations and exposure times (T).



Contour showing expected peak horizontal ground acceleration expressed as a percentage of gravity *Numbers within closed contours are expected maxima. Maximum acceleration within the 60-percent contour along the San Andreas and Garlock faults in California is 80-percent of g, using the attenuation curves of Schnabel and Seed, 1973*

Figure 15. Levels of peak hoizonal ground acceleration expected at rock sites in the United States (from Algermissen and Perkins, 1976). The contoured acceleration values represent the 90-percent probability level; in other words, there is a 10-percent chance that these values will be exceeded within a 50-year period.

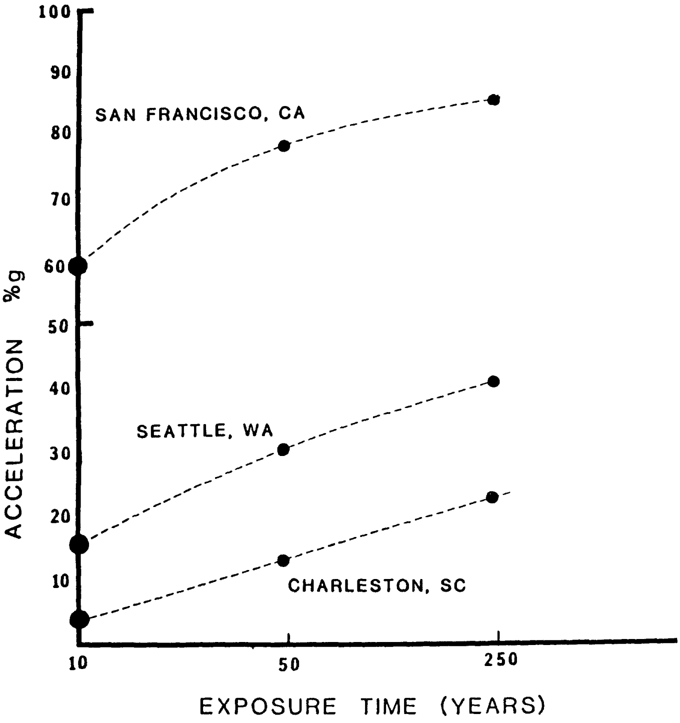


Figure 16. Comparison of hazard at different geographical locations.

THE SELECTION AND DEVELOPMENT OF STRONG MOTION SCALING RELATIONSHIPS FOR SEISMIC HAZARD STUDIES

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ABSTRACT

Research on strong ground motion characteristics conducted in the last ten years (1974-1984) form the basis for a detailed discussion of important factors that should be considered when selecting or developing strong-motion scaling relations for use in earthquake engineering and seismic hazard studies. While emphasis is placed on the empirical prediction of ground motion parameters, there is a brief discussion of procedures that can be used when sufficient strong-motion data are not available with which to perform an adequate statistical analysis. The discussion is followed by a tabulated summary of selected strong-motion scaling reltions that have been proposed and developed in the last ten years to acquaint the reader with the types of relationships that are currently available.

INTRODUCTION

Studies concerned with the evaluation of seismic hazards associated with ground shaking require the prediction of strong ground motion from earthquakes that pose a potential threat to the facility. In order to make such a prediction, one must know certain fundamental characteristics of the earthquake, or source of the seismic waves, the medium through which the waves propagate, the local geology of the site, and the structures comprising the facility. If sufficient numbers of strong-motion recordings having the required characteristics are available, then it is straightforward to select an ensemble of these recordings for use in the evaluation or design of the facility (Fallgren _et al_., 1974; Jennings and Guzman, 1975; Guzman and

Jennings, 1976; Bernreuter, 1981a; Kimball, 1983; Campbell, 1984a; Heaton et al., 1984). Estimates of strong motion using this approach are currently referred to as site-specific.

For most applications, site-specific procedures are not feasible due to an insufficient number of recordings having the appropriate characteristics. This is especially true for probabilistic analyses where a wide range of earthquake sizes and locations are hypothesized, or where near-source estimates of ground motion are required. In such cases, a predictive model is needed. Such a model, referred to as a scaling or attenuation relationship, is expressed as a mathematical function relating a strong-motion parameter to parameters of the earthquake, propagation medium, local site geology, and structure (Figure 1). While less common, the term scaling relationship is used throughout this paper in lieu of the term attenuation relationship. This latter term applies only to the distance attenuation properties of the relation and omits any description of other important scaling properties, such as those related to earthquake magnitude or site effects.

The remainder of this paper will be concerned with those factors that should be considered in the selection or development of a strong-motion scaling relation for use in both deterministic and probabilistic seismic hazard studies. The discussion is divided into five elements: (1) the selection of parameters, (2) the selection of a data base, (3) the selection of a model or functional equation, (4) selection of an analysis procedure, and (5) evaluation of the relationship. Emphasis is placed on relationships derived from ground-motion recordings, however, there is a brief discussion of procedures that may be used when sufficient strong-motion data are not available. Following this is a summary of selected scaling relationships for peak acceleration, peak velocity, and other simple indices of strong ground motion that have been developed in the last 10 years. Other general discussions on this subject may be found in Idriss (1978) and Boore and Joyner (1982).

PARAMETER SELECTION

In statistical terminology, the parameter to be predicted, in this case a strong-motion parameter, is referred to as a dependent variable. The parameters used to predict this variable are referred to as independent variables. There are two important factors to be considered in the selection of a parameter as an independent variable. First, the parameter should be reliable. That is, it should be characteristic of the earthquake, propagation medium (path), site, or structure it is meant to represent, and its estimation from existing data should be reasonably accurate and precise. Second, since the scaling relationship will be used to predict strong ground motion for future hypothesized events, the parameter should be predictable. That is, it should be easily estimated from known seismotectonic characteristics of the region under study.

Concerning the selection of a dependent variable, one should choose a strong-motion parameter (or parameters) that best relates to the purpose of the prediction, whether it be for zoning, planning, or design. While all would agree that the parameter selected should be representative of the seismic performance or damageability of the structure under consideration, there remains considerable controversy as to what parameters best relate to these effects. This stems from a poor understanding of what characteristics of ground motion cause damage in specific structures, a topic currently the subject of two important workshops (Applied Technology Council, 1984; Earthquake Engineering Research Institute, 1984). It has become increasingly clear to the earthquake engineering community that peak acceleration alone is not adequate to characterize the seismic performance of structures (Sharpe, 1982; Campbell and Murphy, 1983; Kennedy, et al., 1984), although this has been known for sometime by experienced structural dynamicists (e.g., Housner, 1971). The remainder of this section will present a discussion of the various parameters that may be used to represent dependent and independent variables and factors that should be considered in selecting specific parameters to be used for the prediction of strong ground motion.

Strong-Motion Parameters

One must first decide what strong-motion parameter is to be predicted. Peak ground acceleration is most commonly used; however, as discussed above, it has come under much criticism for its lack of correlation with observed structural performance during past earthquakes. This has led several investigators to study a number of other parameters, including peak velocity, response spectra, and Fourier spectra; as well as several energy related parameters such as Arias intensity, r.m.s. acceleration, power spectral density, and spectrum intensity. A list of references for publications relating to these strong-motion parameters may be found in the Bibliography.

The acceleration time history is probably the most comprehensive description of ground motion one could use in earthquake engineering applications. It has the potential for incorporating all the salient features of ground motion, in both the time and frequency domains, and can be used in elastic and inelastic analyses of all types of structures. However, such time-domain analyses are extremely expensive and time consuming to perform and are not feasible for most engineering applications. The response spectrum is probably the most complete description of ground motion that is easily used by design engineers (Sharpe, 1982), but these data are not as readily available nor as complete as, say, peak acceleration. They also require the development of several scaling relationships, one for each structural period and damping of interest.

Because strong gound motions are usually recorded on three orthogonal components, one must decide which component(s), horizontal or vertical, are to be predicted. In addition, treatment of strong-motion parameters from the two horizontal components can include the use of (1) the largest of the two components, (2) both components, (3) the mean of both components, or (4) the vectoral combination of both components. The use of both horizontal components results in a prediction representing a random selection of components and is found to give median predictions identical to those using the mean of the two horizontal components (Campbell, 1982a). While the use of either a random component or the mean component is preferred over the use of the largest component from an engineering point of view (Donovan, 1982a), the writer prefers the mean or vectoral component over the random component for two reasons. First, there is larger scatter associated with the random component as compared to the mean component (Campbell, 1982a) and, second, because of the strong correlation between the two horizontal components, the use of both components as independent data points will artificially increase the statistical significance of the resulting analyses by increasing the number of degrees-of-freedom.

Earthquake Parameters

The parameter most commonly used to characterize earthquake size in strong-motion scaling relationships is earthquake magnitude. This comes in part as a result of magnitude being the only source parameter routinely reported by seismographic networks. However, other source parameters used in the past have included source dimensions (Ts'ao, 1980; Bernreuter, 1981b), seismic moment or moment magnitude (Hanks, 1979; McGuire and Hanks, 1980; Hanks and McGuire, 1981; Joyner and Boore, 1981, 1982), and stress drop (Hanks and Johnson, 1976; Hanks, 1979; Ts'ao, 1980; Bernreuter, 1981b; McGuire and Hanks, 1980).

While stress drop is an important source parameter from a theoretical point of view, in practice its estimation is associated with a large degree of uncertainty. This, coupled with the results of several studies that suggest that localized stress drop may be relatively independent of other measures of earthquake size (Hanks and McGuire, 1981; Aki, 1982; Papageorgiou and Aki, 1983) and that static stress drop does not correlate with r.m.s. acceleration (Hanks and McGuire, 1981), would indicate that stress-drop parameters are not very reliable. Seismic moment, or its equivalent moment magnitude (Hanks and Kanamori, 1979), is preferred by some investigators (e.g., Boore and Joyner, 1982) because it corresponds to a well-defined physical property of the source. While this may be true, its use is currently hindered by its poor reliability. Only recently has routine calculations of seismic moment become available. For many past earthquakes, as well as most smaller events, seismic moment is unavailable or only crudely estimated. As a result, Joyner and

Boore (1981) were forced to use local magnitude (M_L) in place of moment magnitude for several earthquakes in their data set.

Earthquake magnitude, although routinely reported and universally used as a measure of earthquake size, is not without its limitations. The variety of magnitude scales that exist can lead to confusion in comparing various predictions. There is also a clear tendency for all scales, except moment magnitude, to reach a limiting value (saturate) as the size of the earthquake increases (Figure 2). Because most magnitude scales are based on the peak amplitude of an instrumental recording, one might expect a good correlation between magnitude and a ground-motion parameter of similar frequency. For example, Boore (1980) found a strong correlation between peak velocity and peak amplitude of a Wood-Anderson seismograph, suggesting a direct relationship between peak velocity and M_L. Extending this logic, short-period estimates of ground motion, such as peak acceleration or short-period spectral estimates, might be expected to correlate best with short-period estimates of magnitude such as m_b (body-wave magnitude) or M_L (local magnitude), and long-period estimates of ground motion might be expected to correlate best with M_s (surface-wave magnitude) or M (moment magnitude). It should be pointed out, however, that complications in this logic arise because of the broad-band frequency characteristics of strong ground motion and those instruments used to record them as compared to the narrow-band response of instruments used to compute magnitude (Boore and Joyner, 1982), and because of the stochastic nature of large, extended ruptures (McGuire and Hanks, 1980; Hanks and McGuire, 1981).

A critical element in the choice of a magnitude scale involves the specification of the magnitude of a future hypothetical earthquake. Because of limitations of most magnitude scales, magnitudes are usually specified in terms of one or more different scales. For instance, surface-wave magnitudes are not reliably determined for magnitudes of about 6 M_s and below and, because of saturation, M_L and m_b become relatively independent of earthquake size for magnitudes near 7. Therefore, magnitudes are generally specified in terms of m_b or M_L for smaller earthquakes and M_s for larger earthquakes. This dual use of magnitude scales is consistent with the interpretation of the

Richter magnitude scale by Nuttli (1979), who suggests that the widely used Richter scale represents M_L for magnitudes less than about 6 and M_s for larger earthquakes. A similar generic scale has been used in relationships among earthquake source dimensions and magnitude (Slemmons, 1977), which form the basis for estimating maximum magnitudes for many faults. Therefore, if the scaling relation is to represent a wide range of magnitudes, it may be desirable to use a dual magnitude scale to be consistent with the application of the relationship (e.g., Campbell, 1981a). For regions outside the Western United States, it is probably more appropriate to replace M_L by m_b, a standard worldwide measure of magnitude, or some similar regional magnitude scale (e.g., m_{bLg} in the Eastern United States). Whatever scale is used, it is important to clearly state the choice and be consistent in its use.

Chung and Bernreuter (1981) and Nuttli and Herrmann (1982) have observed regional differences in magnitude determinations for m_b that should also be considered in the development and application of scaling relationships. They found that the determination of m_b is strongly affected by regional variations in the Q structure (attenuation characteristics), composition, and physical state within the earth. For example, because of differences in attenuation properties between the Western and Eastern United States, a regional m_b magnitude bias exists, which, depending on where the earthquake occurs and where the ground motion is recorded, can lead to magnitudes as much as one-third unit larger in the Eastern United States. Chung and Bernreuter (1981) also point out that when using regional catalogs to obtain magnitudes, it is often necessary to determine how the reported magnitudes were determined. This may also be true for more universal scales. For example, a significant change in the m_b scale occurred in the early 1960's when the World-Wide Standard Seismograph Network (WWSSN) was established. This change in instrumentation had a significant effect on estimated magnitudes and the saturation level of the m_b scale (e.g., compare m_b and m_B in Figure 2). The older, longer period instruments recorded larger magnitudes than can be recorded with the WWSSN instruments.

Another earthquake source parameter found to be related to strong ground motion is fault or focal mechanism. Campbell (1983), in his empirical analysis of near-source ground motion, found that reverse and reverse-oblique mechanisms are associated with ground motions approximately 30-40 percent larger than strike-slip mechanisms. Young (1980a) attributes this difference to regional differences in stress drop. This is consistent with theoretical analyses of McGarr (1982), who suggests that the highest levels of peak acceleration are associated with reverse faults, the next highest with strike-slip faults, and the lowest with normal faults, and Anderson and Luco (1983), who together with McGarr find differences in strong motion based on differences in tectonic stress as well as differences due to the geometry of the fault plane. Other source effects found to influence strong ground motion are source directivity (Boatwright and Boore, 1982; Singh, 1983) and passage of the rupture front (Luco and Anderson, 1984). The latter effect is especially significant for sites located near the fault.

Propagation parameters

These parameters characterize the effects of wave scattering, geometrical attenuation, and anelastic attenuation of ground motion as it travels from the source to the site. The independent variable universally used to characterize these parameters is distance. The attenuation parameters themselves are usually determined from the data. Exceptions to this will be discussed later. Because earthquake rupture can extend over tens to hundreds of kilometers, a number of distance measures have come into use (Figure 3). The measure actually used should depend on the specific application. For sites located several source dimensions from the earthquake, there is little difference between distance measures. However, for shorter distances, the difference between measures becomes significant. In the near-source region, where predictions are of gratest concern, the use of epicentral or hypocentral distance (M1 and M2 in Figure 3) leads to considerably greater scatter in estimates of strong ground motion than the use of distance measures representing closest distance to the fault (M4 and M5). Schnabel and Seed (1973) first recognized the importance of using a fault distance measure for sites near the rupture, and most recent studies have adopted such a measure (see Table 2). Notable exceptions are the relationships of Trifunac (1976b), McGuire (1978b), and Hanks and McGuire (1981), which utilize epicentral or hypocentral distance. Most of these investigators, however, acknowledge that their relationships should not be used at near-fault distances, that is, within several source dimensions of the rupture zone.

Some investigators have argued that using a fault distance measure can lead to biased predictions, especially if the strong-motion stations used in the analysis have a nonrandom distribution around the fault and if the strong motions come from a localized source (or sources) on the fault (Shakal and Bernreuter, 1981; Toro, 1981). This latter concept of a localized source is represented by distance measure M3 in Figure 3. While the distribution of stations are rarely random for a particular earthquake, the randomization introduced by considering recordings from an ensemble of earthquakes should help to reduce this possible bias. In any case, Boore and Joyner (1982) point out that the placement of recording instruments from which the data are obtained and the placement of structures (or sites) for which predictions are to be made are comparable sampling processes from a statistical point of view.

If the strong motions are radiated from small areas of the fault rupture surface (referred to as asperities), then a fault distance measure would tend to underestimate the actual distance to these localized sources. This is not, however, a serious limitation in practice. While it may be possible to identify these asperities for some past earthquakes, it is virtually impossible to anticipate their locations during future events. Thus, such a distance measure is unpredictable. Because of this, most earthquake senarios, whether for probabilistic or deterministic applications, use the closest approach of the fault, tectonic structure, or earthquake rupture as the representative distance from a hypothesized earthquake. This is completely consistent with the definition of the closest distance measures M4 and M5 in Figure 3 and justifies their use in scaling relationships used to predict strong ground motions from such events. If, however, an analysis hypothesizes earthquake sources to be equally distributed along a fault or within an area, with no accommodation of source rupture, then epicentral distance, hypocentral distance, or distance to the energy center would be the more appropriate measure to use. In this case, scaling relationships in terms of fault distance will indeed underestimate the true ground motions.

Site Parameters

Traditionally, site parameters have been related to simple geologic descriptions of the recording stations. The simplest and most common characterization has been the classification of sites as soil or rock. More sophisticated classifications have been based on surface geology, depth of deposits, and seismic velocity. A summary of classification schemes used in the last 10 years is presented in Table 1. The diversity of site classifications used in the past attests to the complex and poorly understood relationship between strong ground motion and site characteristics. While the classifications in Table 1 may be used as a guide in establishing site parameters for strong-motion scaling relations, they should not be adopted without careful consideration of several factors. One such factor is the complex relationship between site and structure effects. Crouse (1978) suggests that effects attributed to the free-field response of the recording site in the past may actually reflect a modification of the ground motion by the structure housing the instrument. This was confirmed by Campbell (1983) who found that factors such as fault mechanism, site topography, soil depth, instrument embedment, and structure size, if not properly accounted for in the development of strong-motion scaling relations, can significantly influence the quantification of site effects.

Campbell (1981a, 1983), Chiaruttini and Siro (1981), and Fraccioli (1981) have recently observed a large amplification (as much as a factor of two) in accelerations associated with shallow soil deposits for sites located near the source of small to moderate earthquakes. The classification of these shallow sites as rock, a common practice in the past, can significantly increase estimates of short-period components of strong ground motion for rock if enough of these sites are included in an analysis. One should also be aware of the possible effects of site topography. The significant influence of topography was first documented for the Pacoima Dam recording of the 1971 San Fernando earthquake by Boore (1973) and Mickey _et al_. (1973). Campbell (1983) finds that the majority of rock recording sites in the United States are situated in areas of steep topography, suggesting that this may have influenced the relationship between rock and soil response found in past analyses. Instrument embedment is another factor to be considered (Campbell, 1983, 1984b). Its influence can be significant, especially for near-source recordings of small earthquakes (Campbell, 1979).

The site classification scheme selected should be compatible with the strong-motion parameter being predicted. Different site characteristics will influence each strong-motion parameter differently, and their effects will vary depending on the distance of the recording from the source and the size of the earthquake. These differences relate to differences in the frequency content of the ground motion. For example, while shallow soils have been observed to amplify accelerations at sites located relatively near the source, peak velocities are found to be virtually unaffected by such shallow depths (Campbell, 1983). The depth of the sediments (i.e., the depth to basement rock) is a parameter that correlates only with moderate- to long-period components of strong ground motion (Trifunac and Lee, 1978a).

Structure Parameters

If free-field predictions of strong ground motion are desired, then parameters characterizing the effect of the structure in which the recording was obtained may be required. These effects have been usually neglected or confused with the effects of site response in the past. However, recent empirical studies (Figures 4, 5, and 6) have indicated that ground motions can be significantly affected by the size and embedment of a building (Crouse, 1978; Boore *et al*., 1980; McCann and Boore, 1982; Campbell, 1983, 1984b), confirming the results of theoretical soil-structure interaction analyses. Boore *et al*. (1980) classified structures into large buildings (greater than two stories in height) and small buildings or shelters and found significant differences in peak accelerations recorded during the 1971 San Fernando earthquake. This formed the basis for excluding large buildings in their subsequent analyses (Joyner and Boore, 1981, 1982). Campbell (1979, 1981a, 1982a, 1984b) found differences in peak accelerations between embedded and ground-level buildings, and Campbell (1983, 1984b) gives evidence showing systematic differences in peak acceleration between buildings of different height. As with site parameters, these effects will vary depending on the strong-motion parameter investigated, distance to the source, and size of the earthquake.

Mickey et al. (1973) and Riemer et al. (1973) document the effects of the response of Pacoima Dam on the abutment instrument recording during the 1971 San Fernando earthquake. This suggests that the responses of dams may also have to be considered in the development of strong-motion scaling relations. In fact, Joyner and Boore (1981, 1982) removed recordings on the abutments and toes of dams for this reason. Bycroft (1978), Crouse (1983), McNeill (1983) and Campbell (1983, 1984b) indicate that so-called free-field recordings can be amplified substantially by small instrument shelters, especially if they are founded on very soft soils. Recordings obtained at the Differential Array in El Centro, California during the 1979 Imperial Valley earthquake are evidence of this potentially important effect (Figure 7).

DATA SELECTION

Once the dependent and independent parameters have been selected, a data base must be chosen. Selection criteria should be established to insure that minimum standards of quality and consistency are met. If this is not done biases will be introduced into the analyses, resulting in increased scatter in the predictions. Significant bias and scatter can be largely avoided if records are selected to represent (1) tectonic provinces of similar attenuation and source characteristics, (2) recording instruments of similar response characteristics, (3) consistent and accurate record processing techniques, and (4) consistent definitions of strong-motion, earthquake, path, site, and structure parameters. Data should be selected to represent the range of parameters for which predictions are to be made. Inclusion of data outside this range can also result in increased bias and scatter in the preditions. Another potential source of bias arises when independent variables are highly correlated. This results in biased estimates of coefficients during regression analysis. Scatter plots (Figure 8) or correlation analyses may be used to identify any significant correlations that may exist. A modification of the selection criteria may be required if significant biases are found.

Consistency among the data may be obtained by either excluding those records that do not meet the recording characteristics to be predicted or by including parameters that adequately account for these characteristics. The first technique is used when undesirable recordings make up a relatively small percentage of the total data set or when there is sufficient data having the appropriate characteristics for a statistically stable analysis. The most common application of this technique has been the selection of data based on site characteristics (Schnabel and Seed, 1973; Seed et al., 1976a,b; Donovan and Bornstein, 1978; Fraccioli, 1978; Sadigh et al., 1978; Boore et al. 1980; Seed and Idriss, 1982; Idriss, 1983); however, others have used this procedure to segregate data by magnitude (Seed et al., 1976b; Sadigh et al., 1978; Boore et al., 1980; Bolt and Abrahamson, 1982) and structure size (Boore et al., 1980; Campbell, 1982a, 1983; Joyner and Boore, 1981, 1982). The second technique is used when a parameter represents an independent variable required for the prediction, such as magnitude or distance, or excluding the undesirable data would leave too few data for a stable statistical analysis.

Data should not be removed from the data base when they represent a random characteristic of the earthquake, path, site, or structure. A random characteristic is one that cannot be reliably predicted in the future. For example, the azimuthal variations in ground motion due to source radiation patterns and directivity (directional focusing) require a knowledge of the location and direction of rupture, characteristics generally not known in advance. The scatter represented by these data reflect a true random uncertainty in the prediction of a strong-motion parameter. Such random uncertainty can be appropriately accounted for in both probabilistic and deterministic analyses. Inclusion of data that represent a systematic characteristic of the earthquake, path, site, or structure will lead to a biased (higher) estimate of the uncertainty in the strong-motion parameter. This bias is extremely critical when uncertainty is treated as random scatter in probabilistic analyses for which predictions are made for small probability levels (long return periods).

MODEL SELECTION

The next step in the development of a strong-motion scaling relationship is the selection of a mathematical function or model relating the independent and dependent variables. The functional form of the model will depend, in general, on the use of the relationship and the data base selected. If the data base represents a relative uniform subset of data, then a function having only a few parameters would be appropriate. If predictions are to be restricted to a range of parameters well-represented by the data (e.g., near the centroid of the magnitude-distance space defined in Figure 8), then a relatively simple empirical model would be justified. However, if the scaling relationship is to be extrapolated much beyond the centroid of the data, then it is important that the model have a physical basis for such an extrapolation to be meaningful.

The physical basis of scaling relationships used in the past have been restricted only to the most fundamental principles of seismology and geophysics. However, this has given little information on what form the function should take at distances close to the fault where details of the rupture process become important. Modeling this process can help to define the form of the function in this critical region (e.g., McGarr *et al.*, 1981; Hadley *et al.*, 1982; Scholz, 1982; Gusev, 1983).

The general form chosen by most investigators in the past is

$$Y = b_1 \; f_1(M) \; f_2(R) \; f_3(M,R) \; f_4(P_i) \; \varepsilon \tag{1}$$

in which Y is the strong-motion parameter (dependent variable); $f_1(M)$ is a function of the magnitude scale M; $f_2(R)$ is a function of the distance measure R; $f_3(M,R)$ is a joint function of M and R; $f_4(P_i)$ is a function representing parameters of the earthquake, path, site, or structure; and ε is a random variable representing the uncertainty in Y.

In its most common form, the function $f_1(M)$ is an exponential function of magnitude,

$$f_1(M) = e^{b_2 M} \tag{2}$$

which comes from the basic definition of magnitude as a logarithmic measure of ground motion amplitude (Richter, 1958). However, others have used the exponential of a quadratic of magnitude and the reciprocal of magnitude to represent this function.

The most common form for $f_2(R)$ is

$$f_2(R) = e^{b_4 R} \; [R+b_5]^{-b_3} \tag{3a}$$

where the term in brackets accounts for attenuation due to geometrical spreading (b_3 representing the geometrical attenuation rate) and the exponential of R accounts for anelastic attenuation, that is, material damping and scattering (b_4 representing the coefficient of anelastic attenuation). Both of these functions come from basic principles of wave propagation in elastic media. The coefficient b_5 is used by some investigators to limit the value of Y at zero distance, a property referred to as saturation (in this case saturation with distance). This is especially necessary when a distance measure, such as epicentral distance or distance to the fault trace, is used, which can take on values of zero. An alternate expression commonly used in place of equation (3a) is

$$f_2(R) = e^{b_4 R} \; \left[\sqrt{R^2 + b_5^2}\,\right]^{-b_3} \tag{3b}$$

where the term in brackets is consistent with the definition of hypocentral distance. Some investigators (e.g. Bolt and Abrahamson, 1982; Brillinger and Preisler, 1984) have used more complicated expressions to account for the distance saturation properties of strong ground motion, but some of these have no physical basis. Variations in distance scaling characteristics due to differences in functional models is demonstrated in Figure 9.

The function $f_3(M,R)$ is used to account for differences in magnitude scaling with distance. In its most common form, this function is given by the expression

$$f_3(M,R) = \left[R + b_6 e^{b_7 M}\right]^{-b_3} \tag{4}$$

which simply replaces b_5 in equation (3a) with an exponential function of magnitude. A similar function has been used in place of b_5 in equation (3b) by some investigators. For negative values of b_7, as is generally the case, this function reduces the amount of magnitude scaling at short distances, another form of saturation (in this case saturation with magnitude). Magnitude saturation of peak acceleration near the fault is a property proposed on both empirical and physical grounds (e.g., Campbell, 1981a; Chung and Bernreuter, 1981; McGarr, 1982; Campbell and Niazi, 1982; Hadley et al., 1982; Gusev, 1983; Munguia and Brune, 1984; Joyner, 1984; see also Campbell, 1981a for a list of earlier references). An alternate expression for this function involves replacing b_3 in equation (3a) by a linear function of magnitude, b_3M, though this appears to have no physical basis.

The function $f_4(P_i)$ is usually represented by an expression of the form

$$f_4(P_i) = \Pi e^{b_i P_i} \tag{5}$$

While somewhat arbitrary, this expression agrees with empirical evidence suggesting that most source and site effects are multiplicative. The most common parameter included in this expression is that related to geologic classifications of the site; however, parameters relating to characteristics of the earthquake, path, site, or structure have been included in this way (see Table 2). Although not commonly done, one could add functions of magnitude and distance to equation (5) if P_i is found to correlate with these parameters.

The random variable ε is usually assumed to be lognormally distributed, although this is not a requirement in most analysis techniques. Some justification comes from the exponential form of the functions used in equation (1), and the property that the product of lognormally distributed variables is itself lognormally distributed. An *a posteriori* empirical justification in support of a lognormal distribution for ε comes from statistical tests on the observed scatter about the predicted values of Y (Esteva, 1970; Donovan, 1973; Donovan and Bornstein, 1978; McGuire, 1978a; Campbell, 1981a), but this may be biased by the assumed functional form of the relationship.

SELECTION OF ANALYSIS PROCEDURE

Having selected a model, one must choose a procedure for determining the unknown coefficients (the b's) in equations (1) through (5). Such a procedure is referred to as regression analysis. Because of the apparent lognormal (or near lognormal) distribution for Y, the strong-motion parameter to be predicted, regressions are usually done on the logarithm of Y, giving the model

$$y = \ln b_1 + \ln[f_1(M)] + \ln[f_2(R)] + \ln[f_3(M,R)] + \ln[f_4(P_i)] + \varepsilon' \qquad (6)$$

where $y=\ln Y$ and $\varepsilon'=\ln \varepsilon$. ε' is a random variable with a mean of zero and a standard deviation of σ. The term σ is referred to as the standard error of estimate of y. If ε is lognormally distributed, then ε' will have a normal (Gaussian) distribution. This, however is not a necessary requirement for regression analyses. It is required in order to make certain statistical statements about the results, as will be discussed in the next section.

Brillinger and Preisler (1984) present statistical procedures for determining the optimal functions (transformations) for Y, M, R and P_i in equation (1) from strong-motion data. This powerful technique would eliminate the need for *a priori* assumptions regarding the form of the model. For example, using the data base of Joyner and Boore (1981), Brillinger and Preisler found that the optimal transformation of Y was $Y^{1/3}$ not ln Y.

However, one must be extremely cautious in adopting such statistically determined functions, since they may or may not conform with known physical characteristics of ground motion. While the discussions that appear in the remainder of this paper are based on the transformation y=ln Y, they are equally applicable to any other transformation of Y. One need only replace y by the appropriate transformation in the equations that follow.

Regressions in the past have been performed exclusively using a least-squares procedure. This procedure minimizes the sum square error

$$\sum_{i=1}^{n} w_i (\hat{y} - y_i)^2 \tag{7}$$

in which $\hat{y}$ is the predicted value of y, y_i is the i^{th} observed value of y, and w_i is the weight assigned to y_i (in the case of weighted regressions). However, other procedures, such as the least absolute sum criterion or maximum liklihood technique, may be used to minimize the influence of outlying observations on the results. If equation (6) (or some alternate model used in place of this model) is linear with respect to the coefficients to be determined, then standard linear least-squares procedures can be used. If not, then nonlinear procedures (e.g., Gallant, 1975; More et al., 1980; SAS Institute, 1980) must be used. If the model is linear and the coeffients are normally distributed, then a t-test may be used to establish the statistical significance of the coefficients. Any coefficient not meeting the required significance level (say a 90 percent probability of not being zero) should then be removed from the model. Stepwise regression procedures are useful for this purpose, especially if it is not known in advance which parameters are important. If the model is nonlinear, the distributions for the nonlinear coefficients must be developed empirically using Monte Carlo simulation (Gallant, 1975; Campbell, 1981a; Boore and Joyner, 1982).

Biased estimates of the coefficients will be obtained if the data are not distributed evenly among the parameters, for example, if magnitude and distance are statistically correlated, or if the data are dominated by many recordings from a few earthquakes. Attempts at reducing this bias have included: restricting the data sample to no more than a certain number of recordings from a given earthquake and a given site (McGuire, 1978a,b); use of weighted regression procedures to equalize the impact of recordings from individual earthquakes (Campbell, 1981a,b, 1982a,b, 1983); use of a two-step regression procedure to separate the estimation of the distance and magnitude scaling coefficients (Joyner and Boore, 1981, 1982); regression on Y rather than on ln Y to increase the impact of the larger values of Y (Bolt and Abrahamson, 1982); and use of a random effects model to separate the uncertainties associated with between earthquake and within earthquake variation (Brillinger and Preisler, 1984). None of these techniques are completely satisfactory. For example, restricting the data base throws out potentially significant data; the use of weighting techniques gives large weight to potentially less reliable data, such as single recordings; the use of a two-step regression precludes optimizing the overall fit, resulting in larger standard errors; regressing on Y rather than on ln Y gives large weight to large values of Y, emphasizing the extremes of the data; and the random effects model is not currently supported by standard statistical packages and, therefore, is not readily available.

The use of a weighted regression is probably the most reasonable of the procedures currently available because it utilizes all the data while optimizing the overall fit, and may be used with standard statistical packages. To demonstrate this procedure, the weighting scheme proposed by Campbell (1981a, 1982a) is presented. The range of distances used in his analysis (0-50 km) was divided into nine intervals within which each earthquake received equal weight. A relative weighting factor of $1/n_{ij}$ was used, where n_{ij} is the total number of recordings for the $i^{\underline{th}}$ earthquake within the $j^{\underline{th}}$ distance interval. The weights were then normalized so that their sum was equivalent to the total number of recordings used in the analyses, n, giving a weight for each recording of

$$w_{ij} = \frac{n\Sigma(1/n_{ij})}{n_{ij}} \tag{8}$$

This last step assured that the statistics of the analyses would represent the correct number of degrees-of-freedom. By basing the weights on nine distance intervals, the scheme was believed to balance important information on distance attenuation characteristics offered by well-recorded earthquakes with important information on magnitude scaling characteristics offered by earthquakes having only a few recordings.

Uncertainty in the predicted value of y is best presented in terms of confidence intervals. For nonlinear models, this interval must be developed by Monte Carlo simulation. However, for linear models analytical procedures are available. In this case, the α-percent confidence interval for the mean of k future observations given a specific set of model coefficients is given by the expression

$$\hat{y} \pm t_{n-p-1,1-\frac{\alpha}{2}} \left[\frac{\sigma^2}{k} + \sigma^2(\hat{y})\right]^{1/2} \qquad (9)$$

where $t_{n-p-1,1-\frac{\alpha}{2}}$ is the absolute value of the t-statistic associated with an exceedance probability α and n-p-1 degrees of freedom (this statistic is tabulated in most statistic books), n is the number of recordings used in the analysis, p is the number of coefficents in the model, σ is the standard error of estimate of the regression, and $\sigma(\hat{y})$ is the standard deviation of the mean prediction of $\hat{y}$. The variance of $\hat{y}$ in matrix notation is given by

$$\sigma^2(\hat{y}) = \sigma^2(X'CX) \qquad (10)$$

in which X is a vector containing specified values of the model parameters (e.g., M or ln R), X' is the transpose of X, and C is the covariance matrix of the model coefficients (b_i's).

The interval given by equation (9) represents the bounds within which the mean of k observations of y will fall (1- α)•100 percent of the time. The most significant application of equation (9) from the standpoint of design is in estimating the confidence interval of a single observation, for which k=1. For example, let the value of a strong-motion parameter for a hypothetical design earthquake of specified magnitude and distance from the facility represent that value expected to be exceeded, say, 16 percent of the time (the 84th percentile). In this case k=1, since we are concerned with a single occurrence of the hypthesized event, and $\alpha = 0.32$, since for the value to be exceeded 16 percent of the time it must fall outside the 68 percent confidence interval of $\hat{y}$. The usual, though inappropriate, application of equation (9) is to estimate the α -percentile value of $\hat{y}$ by the expression

$$\hat{y} + z_{\alpha}\sigma \tag{11}$$

where z_{α} is the standard normal variable associated with a cumulative probability of α. This involves two assumptions: first, $t_{n-p-1,1-\frac{\alpha}{2}}$ is assumed to be equal to z_{α}, valid only for a large number of degrees-of-freedom (say $n-p-1 \geq 30$); second, $\sigma(\hat{y})$ is assumed to be zero, thus neglecting any uncertainty in the mean prediction of $\hat{y}$. This second assumption is only approximately true for predictions near the centroid of the data. For extrapolations of the model as is common in design applications, the uncertainty associated with the mean of $\hat{y}$ can be significant, making equation (11) an inappropriate representation of equation (9).

ADEQUACY OF THE MODEL

The adequacy of the model is best assessed from an analysis of residuals. A residual is simply the difference between the observed and predicted values of y. Before analysis, it may be convenient to normalize the residuals to have a mean of zero and a standard deviation of unity. By the very nature of the regression analysis, the residuals will have a mean near zero. If a weighted regression is used, then it is also necessary to weight the residuals. Letting n equal the total number of observations used in the regression, the normalized weighted residual (NWR) for the ith observation may be computed from the expression

$$NWR_i = \frac{[w_i(y_i - \hat{y})] - MWR}{\sigma} \tag{12}$$

where,

$$MWR = \frac{1}{n} \sum_{i=1}^{n} w_i(y_i - \hat{y})$$

$$\sum_{i=1}^{n} w_i = n$$

In these expressions, w is the weight of the observation and MWR is the mean weighted residual. For unweighted analyses one simply substitutes $w_i=1$ in the above expressions.

The first step in the analysis is to plot the NWR's (hereafter referred to simply as the residuals) versus the predicted value of y and the independent variables. Such a plot is shown in Figure 10. If no trend in the residuals are observed in these plots, then the model can be considered adequate. A trend would indicate an inadequacy in the model to predict the data and would require modifying the functional form. Figure 11 gives an example of residuals that exhibit such trends.

The analytical computation of confidence intervals for $\hat{y}$ require that the residuals have a normal distribution (Monte Carlo simulation could be used to establish confidence intervals for any type of distribution for the residuals). A qualitative assessment of normality may be obtained by inspecting a histogram of the residuals, like the one appearing in the inset of Figure 12. It should resemble the standard bell-shaped curve of the normal distribution. A Kolmogorov-Smirnoff or Chi-square test may be used to statistically test the hypothesis that the distribution is normal. A graphical procedure closely related to the Kolmogorov-Smirnoff test involves making a normal probability plot, a plot of the normal score or estimate of the standard normal variable, versus the normalized residual. If this plot (Figure 12) represents a straight line, then the residuals can be considered normally distributed. Although this latter technique requires judgment on the part of the investigator, it does allow a more rigorous assessment than is possible from inspection of a histogram alone.

OTHER CONSIDERATIONS

In many regions of the world, strong-motion recordings may be unavailable or extremely limited. For these regions, the development of strong-motion scaling relations cannot rely on empirical procedures alone. One of the most common practices in such regions has been the prediction of ground motion from intensity, a qualitative measure of the severity of ground motion (e.g., see the description of the Modified Mercalli intensity scale in Richter, 1958). This approach requires relationships between strong-motion parameters and intensity, such as those offered by Trifunac and Brady (1975), Murphy and O'Brien (1977), Ambraseys (1978), Trifunac (1979), and Chiaruttini and Siro (1981). These can either be used in conjunction with an intensity scaling relationship (Howell and Schulz, 1975; Gupta and Nuttli, 1976; Anderson, 1979; Chandra, 1979) or site-specific estimates of intensity to establish estimates of strong ground motion. McGuire (1977), Cornell et al. (1979), and Bernreuter *et al.* (1984) describe the procedures and assumptions required for such an approach, and specific applications may be found in Nuttli and Herrmann (1978), Battis (1981), Bernreuter (1981a), and Hasegawa *et al.* (1981).

Theoretical earthquake models can also be used to predict ground motion in regions where strong-motion recordings are limited. However, at present, such models are not commonly used for engineering applications due to their relative complexity and unknown reliability. These models fall into three basic categories. The first type uses kinematic and dynamic models of the fault rupture process to generate deterministic predictions of ground motion. Swanger et al. (1980, 1981) and Aki (1982) describe the characteristics of this type of model. The second category of theoretical models uses stochastic simulation of ground motions based on simple seismological source models (sometimes in conjunction with random vibration theory) to produce random predictions of strong ground motion. Most recent examples of this type of model are found in Hadley et al. (1981), Boore (1983a), Gusev (1983), and Joyner (1984). The third type of model uses simple seismological source models to deterministically predict strong ground motions. Because of its simplicity, this type of model has been most widely used to generate scaling relationships (e.g., Campbell and Duke, 1974a; Hanks and Johnson, 1976; Midorikawa and Kabayashi, 1978; Hanks, 1979; Ang and Mohammadi, 1981; Bernreuter, 1981b; Hanks and McGuire, 1981; McGarr, 1981; Scholz, 1982). The reader is referred to Boatwright (1982), Boore (1983b), Luco and Anderson (1984), and Earthquake Engineering Research Institute (1984) for a comprehensive review and compilation of recent work on theoretical earthquake modelling.

Even if sufficient data are available with which to develop a strong-motin scaling relationship, it may still be desirable to constrain some of the coefficients of the model. This is particularly useful when specific coefficients are highly correlated with one another and neither can be determined accurately, or when the data are not distributed well enough to give robust estimates of some coefficients. Some investigators have simply preferred to use constraints in the development of their relationships to be consistent with well-established seismological principles.

The most common seismological constraints used in past studies are those related to geometrical attenuation (Schnabel and Seed, 1973; Nuttli, 1979; Nuttli and Herrmann, 1978, 1984; Joyner and Boore, 1981, 1982) and magnitude scaling (Trifunac, 1976a,b; Espinosa, 1979, 1980; Boore, 1980). Others have used regional and teleseismic data to constrain anelastic attenuation rates (Nuttli, 1979; Campbell, 1981b, 1982b; Nuttli and Herrmann, 1984); while others have constrained coeffcients based on strong-motion recordings of nuclear explosions (Blume, 1977; Orphal and Lahoud, 1974) or based on the results of other empirical studies (Eguchi, 1980; Battis, 1981; Campbell, 1981a; Hasegawa et al., 1981). The most common and least supported empirical constraint used to develop strong-motion scaling relations has involved the coefficient b_5 in equations (3a)-(3b). Typically values of 20 to 25 km have been assumed for this coefficient in order to control the amplitudes of strong-motion parameters at small distances (McGuire, 1974; Blume, 1977; Donovan and Bornstein, 1978; Fraccioli, 1978; Sadigh et al., 1978; Battis, 1981; Idriss, 1983). Recently a value of $b_5 > 0$ has been justified statistically by Campbell (1981a, 1982a) and Boore and Joyner (1982).

REVIEW OF EXISTING RELATIONSHIPS

There have been a vast number of strong-motion scaling relations that have been proposed throughout the years. Table 2 contains a summary of some of the more significant relationships proposed within the last decade (i.e., from 1974 to 1984). The 10-year criterion is used to limit the number of relationships tabulated to those commonly used in practice. Scaling relationships published before 1974 have generally been revised or have become obsolete due to the rapid advancement in the field of engineering seismology.

To further limit the number of scaling relationships summarized in Table 2, the compilation has been restricted to those relationships that (1) predict peak acceleration, peak velocity, or some other single index of ground motion such as Arias intensity, r.m.s. acceleration, etc. (peak displacement is excluded for reasons specified below); (2) are available in the open literature, that is, in professional journals or conference proceedings; and (3) are based at least in part on strong-motion data. The restriction to single indices is required to eliminate spectral values from the listing due to their large number of parameters (one for each period and damping). Peak displacements are not included because of their generally poor accuracy resulting from errors in the record processing procedures used to integrate and filter the accelerograms and from long-period noise inherent to the records themselves (Trifunac and Lee, 1978b; Sunder and Connor, 1982). The restriction to relationships published in the open literature limits the compilation to those relations generally available to engineers and seismologists and, thus, have had the opportunity of being subjected to peer review and acceptance. That is not to say that those relationships omitted from tabulation are not of equal or even greater value, only that such relationships are not widely known and have not had as much opportunity for peer review. References to many of these are included in the Bibliography. The restriction to relationships based on strong-motion data merely requires that the relationships have at least some empirical basis, eliminating the large number of theoretical models that have been proposed recently. A more complete listing of strong motion scaling relations for the last 10 years, including those available in reports, appears in the Bibliography. Additional compilations and bibliographies are available in Donovan (1973), Idriss (1978), Eguchi and Wiggins (1979), Young (1980a,b), Boore and Joyner (1982), and Boore (1983b).

The summary provided in Table 2 is organized into five categories: (1) a reference, (2) a definition of parameters, (3) a statement of applicability, (4) a description of the strong-motion parameter predicted, and (5) the model. The table is provided only as a summary of those relationships that have been developed in the past 10 years. Specific relationships should not be used without careful consideration of the application for which they are intended. The guidelines for developing scaling relations presented earlier in this paper can serve as a framework to be used in evaluating these existing relationships for specific applications.

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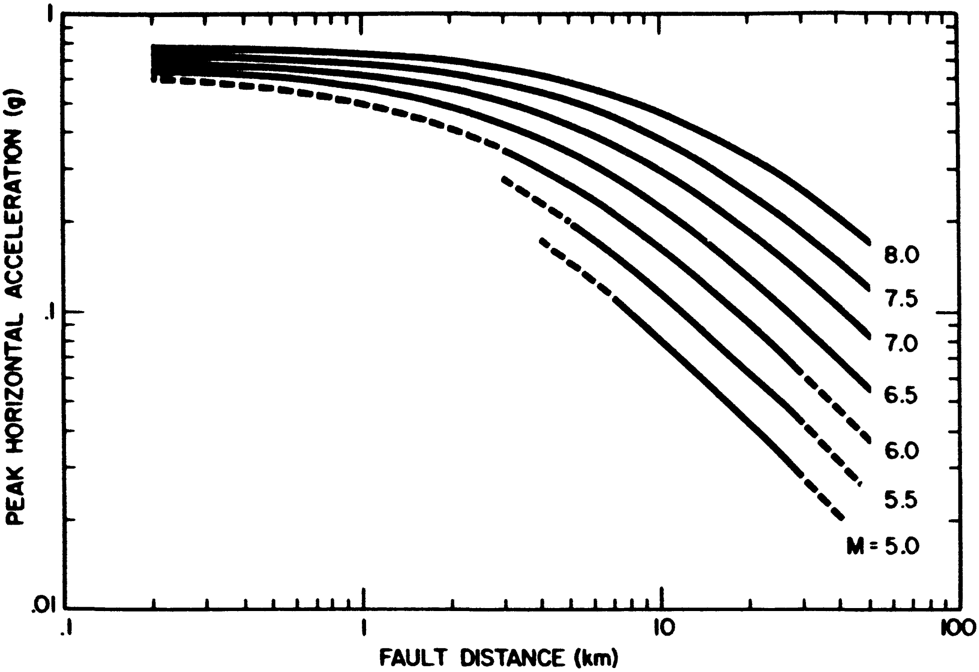


Figure 1. Scaling relationship of peak horizontal acceleration (Campbell, 1981a)

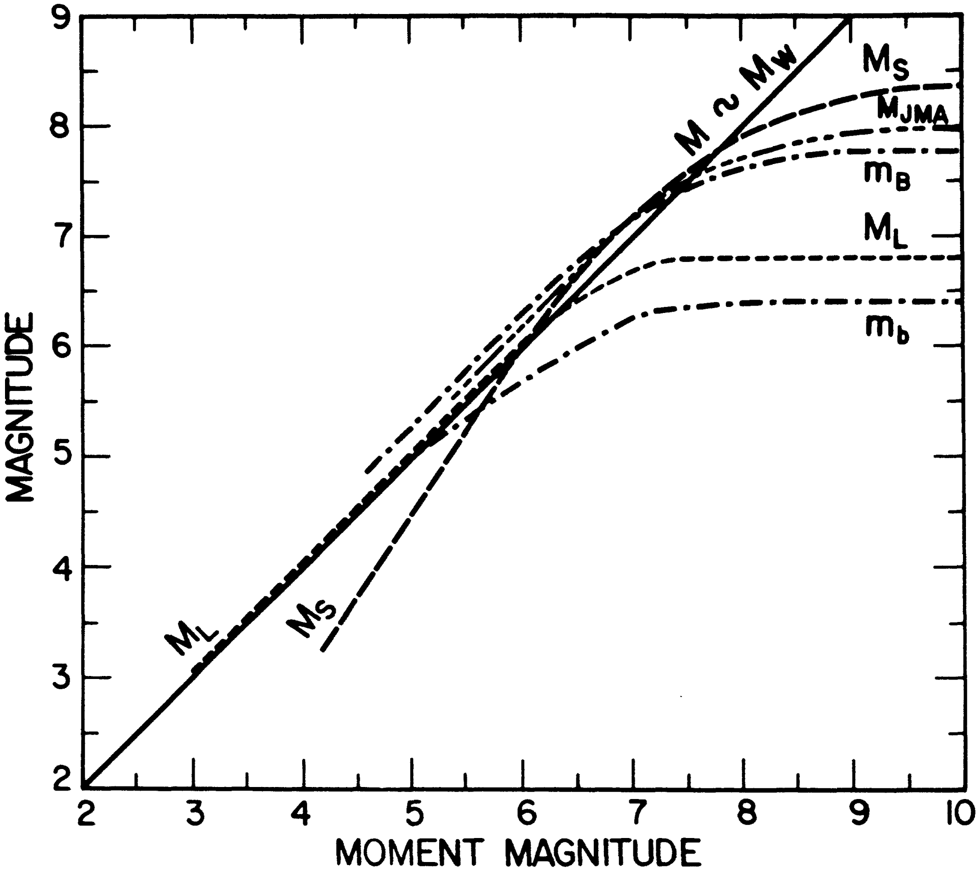
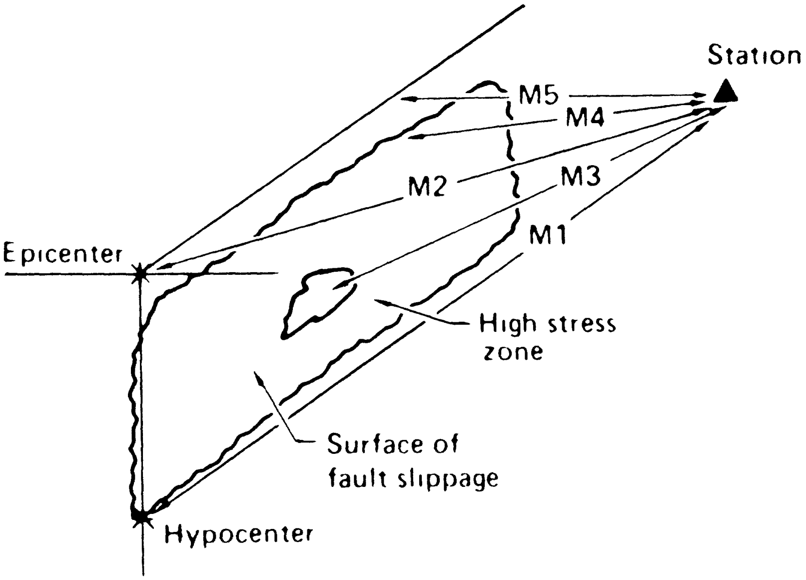


Figure 2. Relationship among various magnitude scales showing saturation with respect to moment magnitude (Heaton et al, 1984): M_S (surface wave); M_{JMA} (Japan Meteorological Agency); m_B (long-period body wave); M_L (local); m_b (short-period body wave); M_w (moment magnitude)



Distance Measures (from recording station)

MI - Hypocentral
M2 - Epicentral
M3 - Dist. to energetic zone
M4 - Dist. to slipped fault
M5 - Dist. to surface projection of fault

Figure 3. Schematic diagram showing various measures used to characterize source-to-site distance.

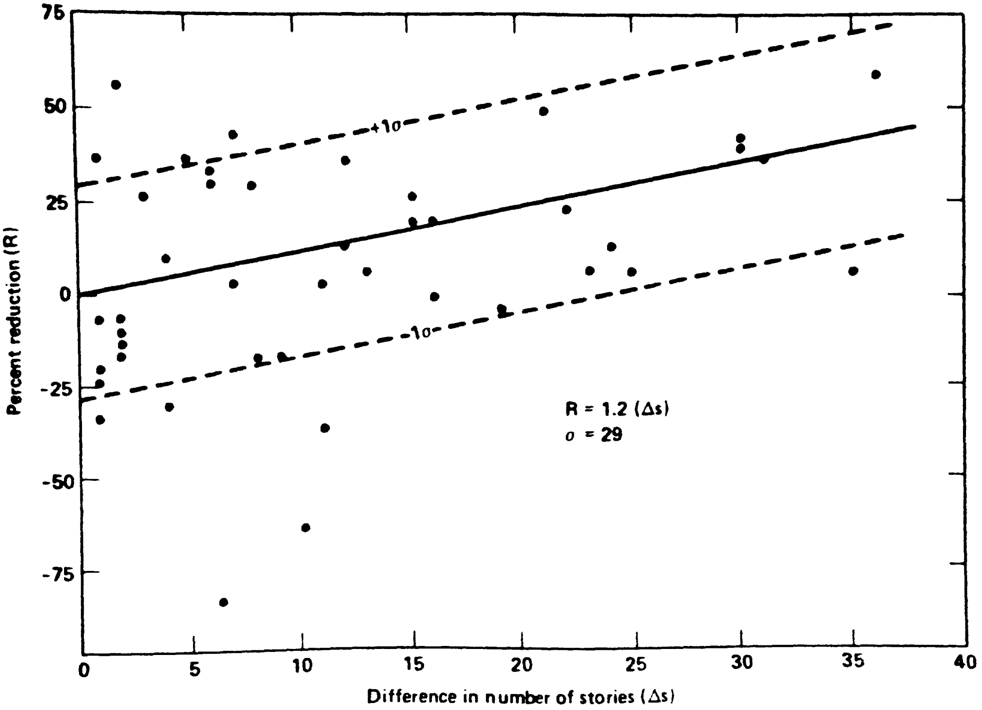


Figure 4. Reduction in peak horizontal acceleration due to differences in assumed functional relationships (Bolt and Abrahamson, 1982).

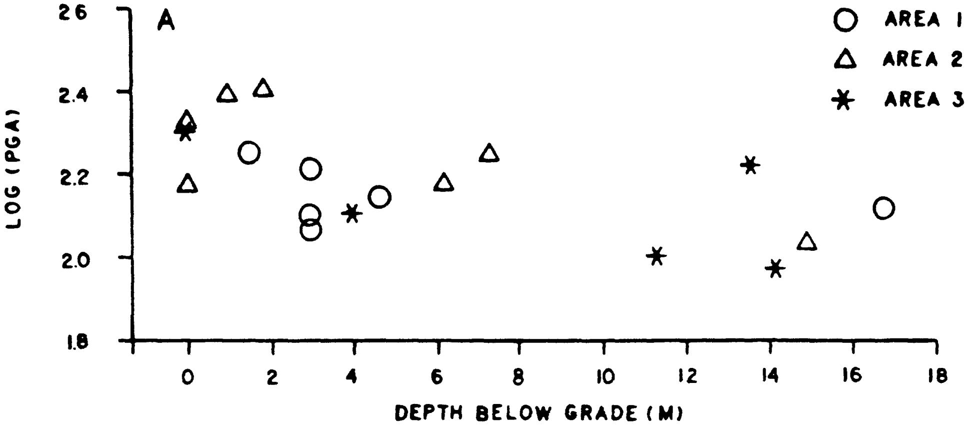


Figure 5. Relation between log acceleration (log PGA) and instrument depth showing reduction in acceleration with increasing depth of burial (McCann and Boore, 1983).

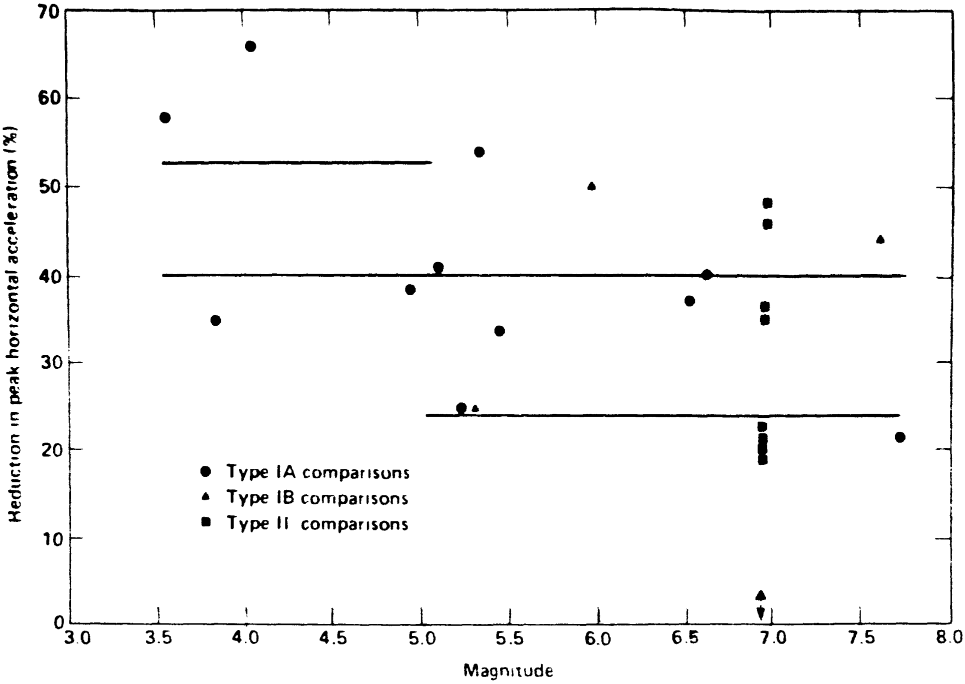


Figure 6. Reduction in peak horizontal acceleration due to building embedment as a function of magnitude. Symbols represent actual case studies. Solid lines represent the result of regression analyses (Campbell, 1983).

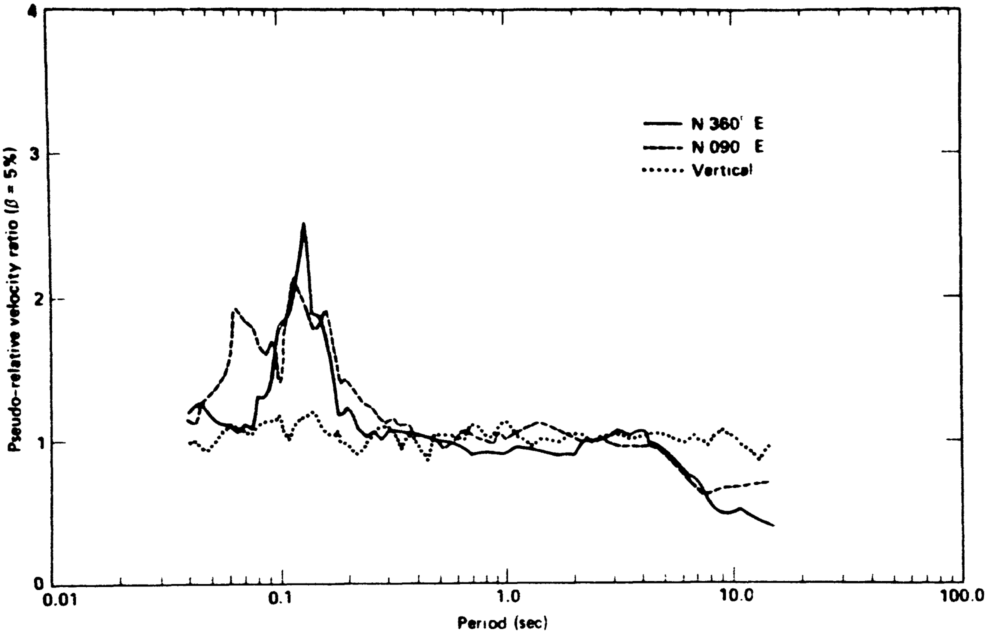


Figure 7. Ratio of 5%-damped pseudo velocity spectra between Dogwood Road instrument shed and differential array No. 1 free field site recorded during the 15 October 1979 Imperial Valley earthquake.

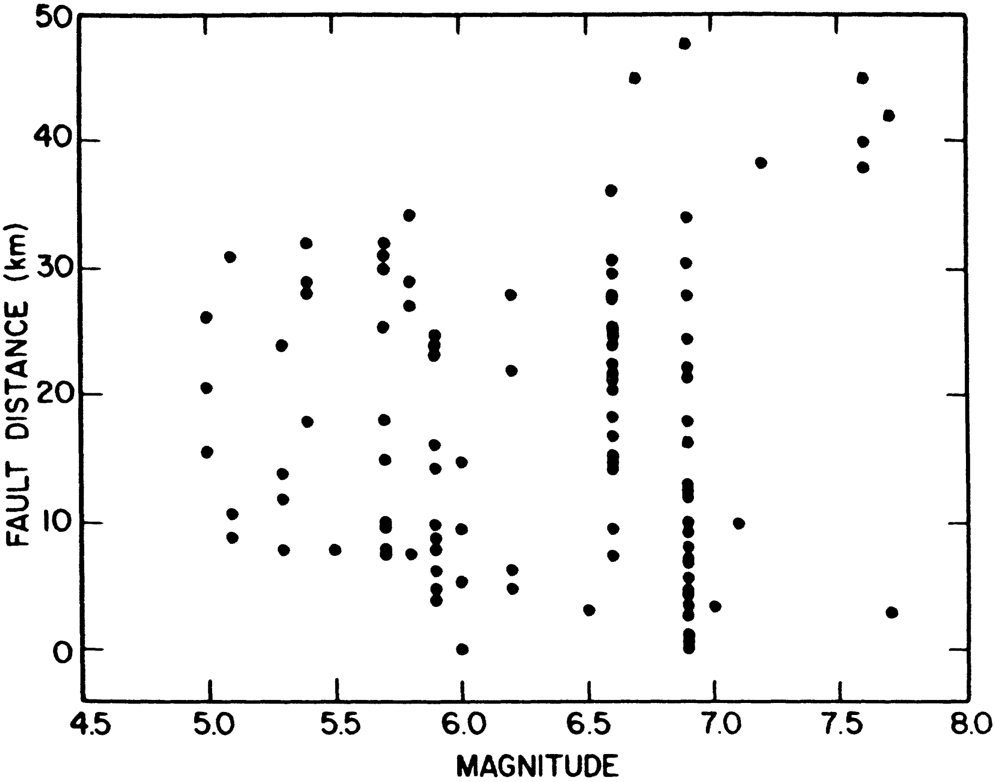


Figure 8. Example of magnitude-distance scatter plot of strong-motion recordings used in regression analysis (Campbell, 1981a).

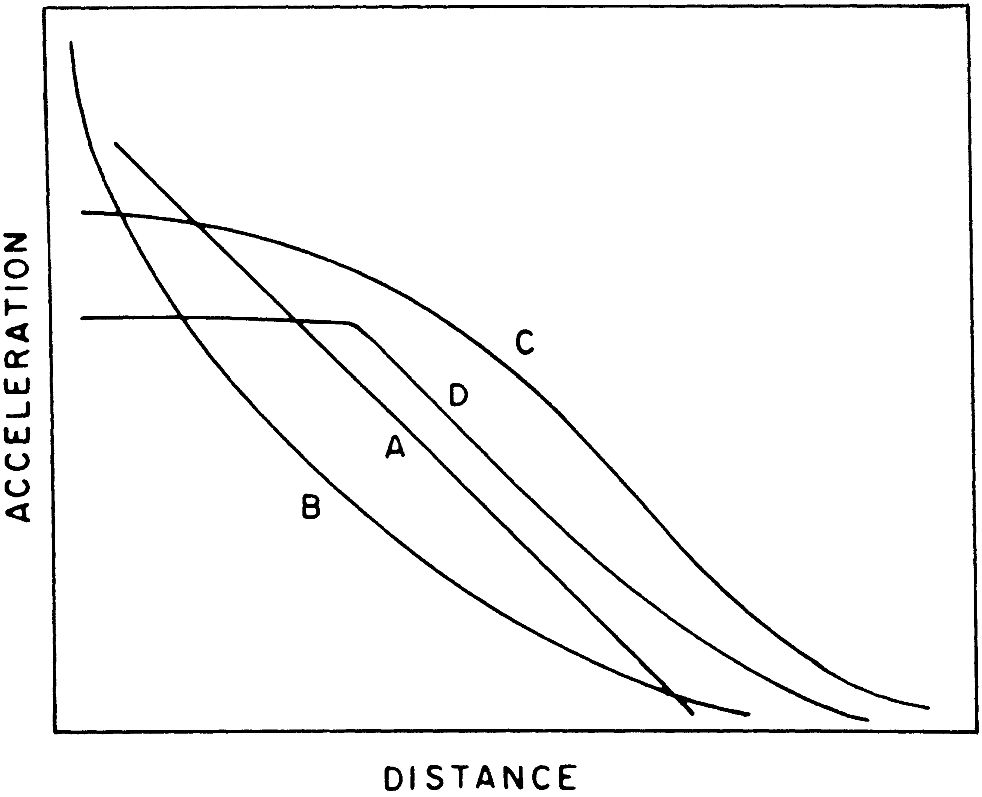


Figure 9. Schematic diagram showing differences in attenuation of acceleration due to differences in assumed functional relationships (Bolt and Abrahamson, 1982).

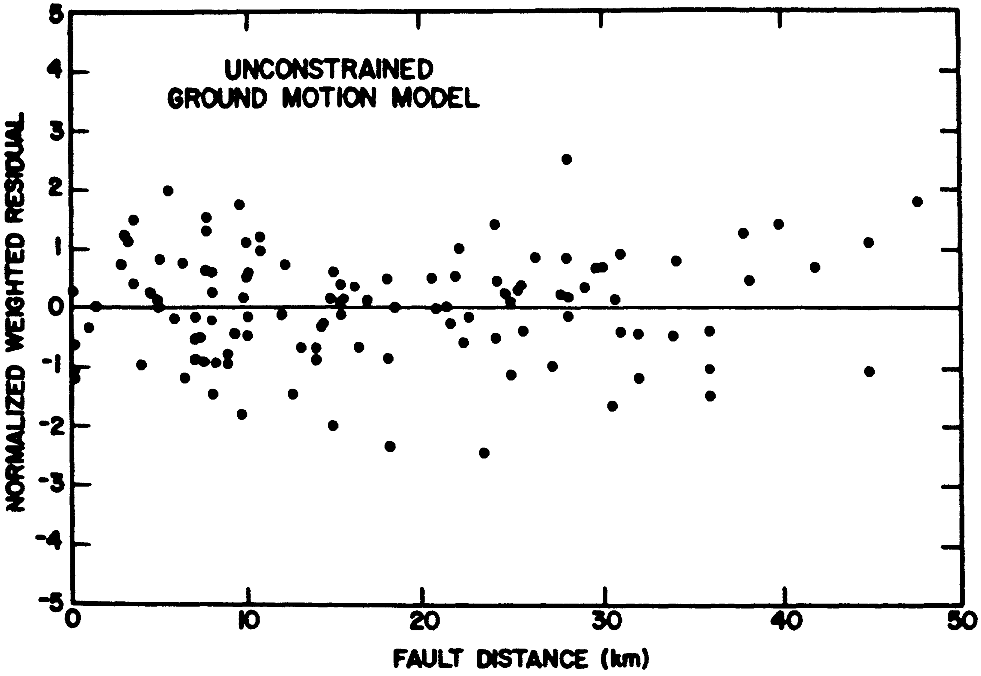


Figure 10. Plot of normalized weighted residuals versus fault distance (Campbell, 1981a).

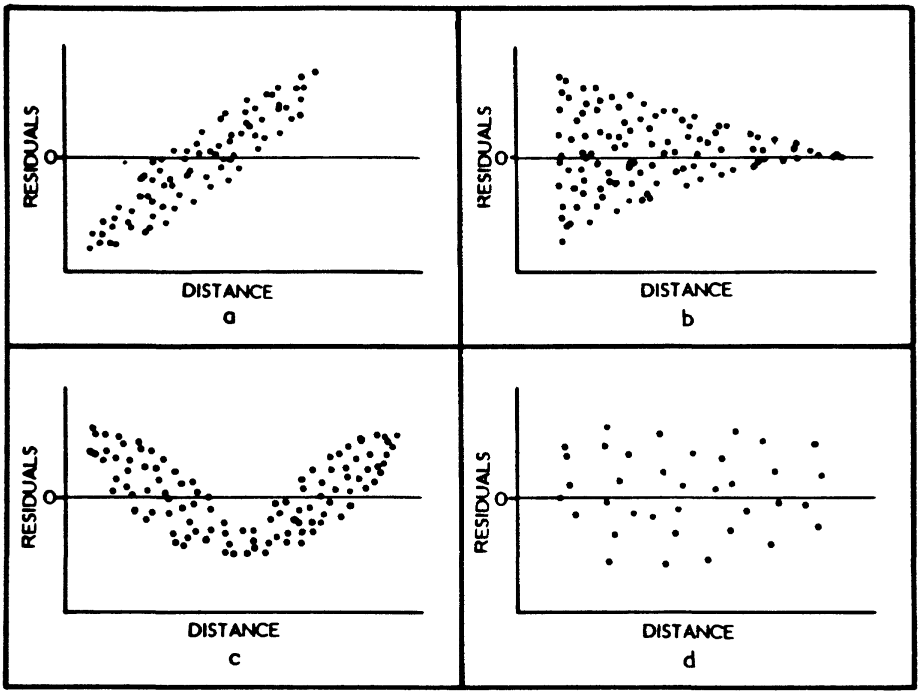


Figure 11. Examples of possible residual trends derived from regression analysis. Plots a through c indicate a deficiency in the assumed functional relationship between the strong-motion parameter and distance. Plot d indicates that the functional relation is adequate.

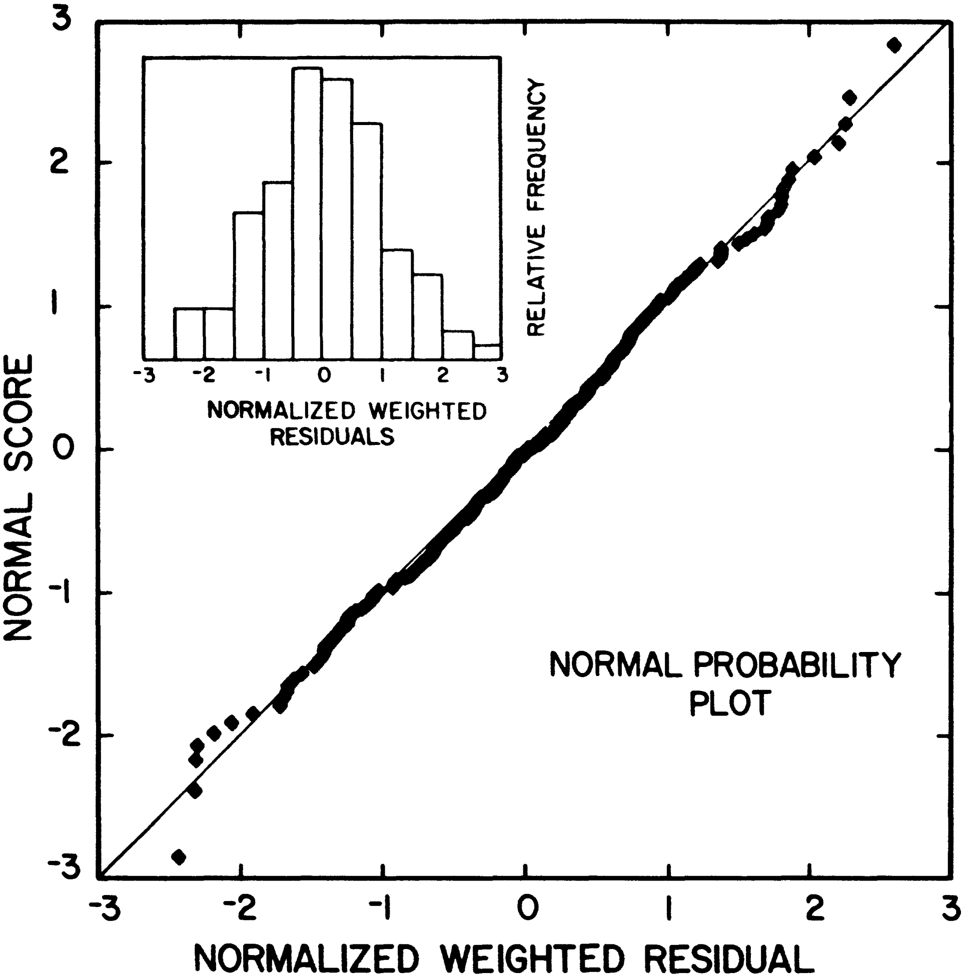


Figure 12. Normal probability plot and histogram of residuals (Campbell, 1981a).

TABLE 1

SUMMARY OF RECENT SITE CLASSIFICATION SCHEMES

Classification	Description	Reference
Soil Rock	Undefined Undefined	Newmark and Hall (1982)
Soil Rock	Alluvium and soft deposits >10 m deep Basement rock, sedimentary rock, and soil ≤10 m deep	McGuire (1978a,b)
Soil Rock	Alluvium, soil, mud, fill, and glacial deposits >4-5 m deep Basement rock, sedimentary rock, and soil ≤4-5 m deep	Boore *et al.* (1980), Joyner and Boore (1981,1982)
Soft Intermediate Hard	Soft alluvium Sedimentary rock Basement or crystalline rock	Trifunac and Brady (1975), Trifunac (1976a,b), McGuire (1977), Murphy and O'Brien (1977)
Shallow alluvium Deep alluvium Sedimentary rock Crystalline rock	Alluvium 7-20 m deep Alluvium >20 m deep Sedimentary rock and alluvium <7 m deep Igneous rock, metamorphic rock, and alluvium <7 m deep	Duke *et al.* (1972), Campbell and Duke (1974a,b)
Rock Alluvium Alluvium (<9 m) Alluvium (9-60 m)	Basement and sedimentary rock Alluvium of unspecified thickness Alluvium <9 m deep Alluvium 9-60 m deep	Mohraz (1976)
Rock Stiff soils Deep cohesionless soils Soft soils	Shale-like and harder rock with V_s >760 m/s Stiff clay, sand, or gravel <45 m deep Generally cohesionless soils >75 m deep Soft to medium-stiff clays with sand and gravel Generally cohesionless soils >75 m deep Soft to medium-stiff clays with sand and gravel	Schnabel and Seed (1973), Seed *et al.* (1976a,b), Applied Technology Council (1978), Donovan and Bornstein (1978), Fraccioli (1978), Sadigh *et al.* (1978), Werner *et al.* (1979), Ts'ao (1981), Seed and Idriss (1982), Idriss (1983)
Very dense and stiff soils Intermediate soils Very loose soils	——— ——— ———	Hayashi *et al.* (1971)
Rock-like Thin alluvium Thick alluvium	Hard rock and stiff soils Alluvium ≤20 m deep Alluvium >20 m deep	Chiaruttini and Siro (1981)
Rock or very stiff soils Shallow alluvium Deep alluvium	Rock and alluvium <5 m deep Alluvium 5-20 m deep Alluvium >20 m deep	Fraccioli (1981)
Group-1 Group-2 Group-3	Tertiary age or older rock or diluvium <10 m deep Diluvium ≥10 m deep, alluvium <10 m deep, or alluvium <25 m deep including soft layer <5 m deep Other than the above, usually soft alluvium or reclaimed land	Okubo *et al.* (1983)
Hard rock Soft rock Pleistocene deposits Recent alluvium Shallow soils Soft soils	Crystalline, hard metasedimentary, and hard volcanic rock Sedimentary, soft metasedimentary, and soft volcanic rock Pleistocene age soils >10 m deep Holocene age soils >10 m deep Soils ≤10 m deep Extremely soft or loose soils	Campbell (1981a,b;1982a,b;1983)
Depth of sediments	Depth to the top of basement rock	Trifunac and Lee (1978a)
Impedance, ρV_s	ρ = specific gravity of deposit V_s = shear-wave velocity of deposit (m/s) ρV_s = 610 m/s (soil) ρV_s = 3,660 m/s (hard rock)	Blume (1977)
Shear-wave velocity, V_s	Average shear-wave velocity over a depth of one-fourth wavelength of the period of interest (soils only)	Joyner *et al.* (1983)

TABLE 2

SUMMARY OF SELECTED STRONG-MOTION SCALING RELATIONS (1974-1984)

[Note: Definition of symbols appears in Appendix]

Reference	Parameters	Applicability	Y	Model		$\sigma_{\ln Y}$
Campbell and Duke (1974a)	$M = M_s$ $R = R_e$	California; $M = 4.5\text{–}8.5$; $R = 15\text{–}160$	AI_v	$Y = 18.8\ e^{M(0.33M-1.47)}\ R^{-3}\ S$ $S = 1.80$ (basement rock) 3.63 (sedimentary rock) 3.74 (alluvium ≤ 60 ft) 5.12 (alluvium >60 ft)	——	——
Campbell and Duke (1974b)	—do——	—do——	—do——	$Y = 21.6\ e^{M(0.33M-1.47)}\ R^{-3.79}\ S$ $S = 0.58\ R^{0.46}$ (basement rock) $1.02\ R^{0.51}$ (sedimentary rock) $0.37\ R^{0.81}$ (alluvium ≤ 60 ft) $0.65\ R^{0.74}$ (alluvium >60 ft)	——	——
Ophal and Lahoud (1974)	$M = M_R$ $R = R_h$	California; $M = 4.1\text{–}7.0$; $R = 15\text{–}350$	PHA_1	$Y = 0.066\ e^{0.4M}\ R^{-1.39}$	——	0.69
			PHV_1	$Y = 0.726\ e^{0.52M}\ R^{-1.34}$	——	0.64
Seed et al. (1976b)	$M = M_L$ $R = R_{ze}$	Western U.S.; $M = 6.5$; $R = 20\text{–}350$	PHA_b	$Y = aR^{-c}$	(graphical)	——
			PHV_b	$Y = aR^{-c}$	(graphical)	——
Trifunac (1976b)	$M = M_R$ $R = R_e$	Western U.S.; $M = 3.0\text{–}7.7$; $R = 20\text{–}200$	PHA_b, PVA	$Y = Y_o A_o e^{2.3M}$ $Y_o = 6.182\times10^{-10}\ e^{-0.138S}\ e^{-0.762V}\ F(M)$ $F(M) = e^{4.12M}\ e^{-0.428[M^2-(M-7.5)^2]}$ $= e^{4.12M}\ e^{-0.428M^2}$ 2.023×10^4	—— $(M \ge 7.5)$ $(7.5 > M \ge 4.8)$ $(M \le 4.8)$	0.73
			PHV_b, PVV	$Y = Y_o A_o e^{2.3M}$ $Y_o = 4.395\times10^{-9}\ e^{-0.309S}\ e^{-0.792V}\ F(M)$ $F(M) = e^{4.74M}\ e^{-0.463[M^2-(M-7.61)^2]}$ $e^{4.74M}\ e^{-0.463M^2}$ 1.856×10^5 $S = 0$ (alluvium) 1 (intermediate rock) 2 (basement rock) $V = 0$ (horizontal components) 1 (vertical components)	—— $(M \ge 7.61)$ $(7.61 \ge M \ge 5.12)$ $(M \le 5.12)$	0.92
Blume (1977)	$M = M_R$ $R = R_h$	Western U.S.;	PHA_b	$Y = 0.318\ e^{1.03M}\ (29)^{1.14S}\ (R+25)^{-1.14S}$	$(M \le 6.5)$	0.93
				$Y = 26.0\ e^{0.432M}\ (29)^{1.22S}\ (R+25)^{-1.22S}$ $S = 0.5\ \log_{10}\ (\rho V_s)$	$(M > 6.5)$	0.59
Milne (1977)	$M = M_L$, M_s $R = R_h$	Western N.A.; $M = 3.0\text{–}7.7$; $R = 20\text{–}200$; Soil and rock	PHA_1	$Y = 0.04\ e^{0.99M}\ R^{-1.39}$	——	——
Ambraseys (1978)	$M = m_b$, M_L $R = R_h$	Europe; $M = 2.7\text{–}7.1$; $R = 1\text{–}480$	PHA_1	$Y = 0.0288\ e^{1.45M}\ R^{-1.1}$	$(R = 1\text{–}480)$	——
				$Y = 0.0131\ e^{1.46M}\ R^{-0.92}$	$(R \le 30)$	——
Donovan and Bornstein (1978)	$M = M_R$ $R = R_{ec}$	California; $M = 5.0\text{–}7.7$; $R = 5\text{–}320$ Rock and stiff soil	PHA_b	$Y = A\ e^{BM}\ (R+25)^C$ $A = 2.198\times10^3\ R^{-2.1}$ $B = 0.046 + 0.445\ \log_{10} R$ $C = -2.515 + 0.486\ \log_{10} R$	——	Graphical
Fraccioli (1978)	$M = M_s$ $R = R_h$	Circum-pacific; $M = 4.9\text{–}8.0$; $R = 15\text{–}360$; Soft soil; $H \le 100$	PHA_b	$Y = 0.111\ e^{0.265M}\ (R+25)^{-0.80}$	——	0.54
			PHV_b	$Y = 1.480\ e^{0.282M}\ (R+25)^{-0.425}$	——	0.58

TABLE 2

SUMMARY OF SELECTED STRONG-MOTION SCALING RELATIONS (1974-1984)--continued

Reference	Parameters	Applicability	Y	Model		$\sigma_{\ln Y}$
McGuire (1978b)	$M = M_R$	California;	PHA_b	$Y = 0.0306\ e^{0.89M}\ R^{-1.17}\ e^{-0.20S}$	——	0.62
	$R = R_h$	$M = 4.5-7.7$;	PHV_b	$Y = 0.3680\ e^{1.07M}\ R^{-0.96}\ e^{0.07S}$	——	0.64
		$R = 10-200$		$S = 0$ (rock)		
				1 (soil)		
Nuttli and Herrmann (1978)	$M = m_b$	Central U.S.;	PHA_v	$Y = 4.459X10^{-4}\ e^{1.20M}$	$(R \leq 15)$	——
	$R = R_e$	$M = 4.0-6.5$;		$Y = 7.061X10^{-3}\ e^{1.20M}\ R^{-1.20}$	$(300 \geq R \geq 15)$	——
		$R = 25-300$	PHV_v	$Y = 0.0012\ e^{2.03M}\ R^{-1.0}$	$(R \geq 15)$	——
Sadigh *et al.* (1978)	$M = M_L$	California;	PHA_b	$Y = 69\ (R+20)^{-1.6}$	——	0.45
	$R = R_{ze}$	$M = 6.5$;	PVA	$Y = 218\ (R+20)^{-2.0}$	——	0.35
		$R = 10-220$;	PHV_b	$Y = 2887\ (R+20)^{-1.3}$	——	0.41
		Deep soil	PVV	$Y = 2095\ (R+20)^{-1.4}$	——	0.47
Espinosa (1979)	$M = M_L$	Western U.S.;	PHV_v	$Y = 6.16x10^{-4}\ e^{2.3M}\ R^{-1.35}$	——	——
	$R = R_h$	$M = 4.0-7.2$;				
		$R = 5-300$				
Fraccioli and Agalbato (1979)	$M = M_L$	Friuli, Italy;	PHA_b	$Y = 0.0397\ e^{0.58M}\ R^{-0.79}$	——	0.58
	$R = R_h$	$M = 3.7-6.3$;	PVA	$Y = 0.0245\ e^{0.55M}\ R^{-0.78}$	——	0.58
		$R = 5-190$;	PHV_b	$Y = 0.1700\ e^{0.99M}\ R^{-0.74}$	——	0.58
		Freefield				
Nuttli (1979)	$M = m_b$	Western U.S.;	PHA_3	$Y = a\ e^{1.15M}\ R^{-0.83}\ e^{-\gamma R}$	(graphical)	——
	$R = R_e$	Central U.S.;	PHV_3	$Y = a\ e^{2.30M}\ R^{-0.83}\ e^{-\gamma R}$	(graphical)	——
		$M = 4.0-7.0$;				
		$R \geq 10$				
Boore (1980)	$M = M_L$	California;	PHV_1	$Y = 0.77\ A_o\ e^{2.3(M-3)}$	$(R>15-40)$	——
	$R = R_f,\ R_e$	$M = 5.5-7.5$;		Graphical	$(R<15-40)$	——
		$R \geq 10$				
Boore *et al.* (1980)	$M = M_L$	Western N.A.;	PHA_1	$Y = 1.58\ R^{-0.9}$	$(M_L=5.0-5.7,\ R=5-30)$	Graphical
	$R = R_f$	Soil and rock		$Y = 10.0\ R^{-1.2}$	$(M_L=6.0-6.4,\ R=15-55)$	
				$Y = 398\ R^{-2.0}$	$(M_L,M_S=7.1-7.6,\ R=40-150)$	
			PHV_1	$Y = 251\ R^{-1.2}$	$(M_L=5.3-5.7,\ R=5-30)$	
				$Y = 40.0\ R^{-0.6}$	$(M_L=6.4,\ R=40-150)$	
Eguchi (1980)	$M = M_R$	Western U.S.;	PHA_b	$Y = 0.069\ e^{0.85M}\ R^{-1.44}$	——	0.64
	$R = R_h$	$M = 3.5-7.7$;	PHV_b	$Y = 1.130\ e^{1.04M}\ R^{-1.19}$	——	0.55
		$R = 10-400$;				
		Soil and rock				
Espinosa (1980)	$M = M_L$	Western U.S.;	PHA_1	$Y = 5.235x10^{-7}\ e^{2.3M}\ R^{-0.06}$	$(R \leq 10)$	——
	$R = R_h$	$M = 4.0-7.5$;		$Y = 1.776x10^{-5}\ e^{2.3M}\ R^{-1.59}$	$(10 \leq R \leq 60)$	——
		$R = 5-300$		$Y = 4.153x10^{-3}\ e^{2.3M}\ R^{-2.93}$	$(60 \leq R \leq 300)$	——
Campbell (1981a)	$M = M_L\ (M<6)$	Worldwide;	PHA_m	$Y = 0.0159\ e^{0.868M}\ [R + C(M)]^{-1.09}$	——	0.37
	$= M_S\ (M \geq 6)$	$M = 5.0-7.7$;		$C(M) = 0.0606\ e^{0.7M}$		
	$R = R_f$	$R \leq 50$;				
		Soil and rock				
Campbell (1981b)	$M = M_L\ (M<6)$	Western U.S.;	PHA_m	$Y = 0.0142\ e^{0.790M}\ [R + C(M)]^{-0.862}\ e^{-\gamma R}$	(R_f)	0.41
	$= M_S\ (M \geq 6)$	Central U.S.;		$C(M) = 0.0268\ e^{0.778M}$		
	$R = R_f,\ R_e$	$M = 5.0-7.7$;		$Y = 0.0823\ e^{0.922M}\ (R + 25.7)^{-1.27}\ e^{-\gamma R}$	(R_e)	0.55
		$R \geq 0$;				
		Soil and rock				
Chiaruttini and Siro (1981)	$M = M_L$	Italy;	PHA_1	$Y = 1.10\ e^{0.94M}\ R^{-0.99}\ e^{0.55A}\ e^{0.53B}$		0.60
	$R = R_h$	Alpide Region;		$A = 0$ (Friuli, Italy)		
		$M = 3.5-6.6$;		1 (Ancona, Italy)		
		$R = 4-60$		$B = 0$ (Friuli, Italy)		
				1 (Alpide Region)		

TABLE 2

SUMMARY OF SELECTED STRONG-MOTION SCALING RELATIONS (1974-1984)--Continued

Reference	Parameters	Applicability	Y	Model		$\sigma_{\ln Y}$
Hanks and McGuire (1981)	$R = R_h$	California; $M = 4.0–6.5$; $R = 10–100$;	$RMSA_b$	$Y = 0.119\ R^{-1}\ \sqrt{\frac{fmax}{fo}}$	—	—
			PHA_b	$Y = 0.119\ R^{-1}\ \sqrt{\frac{2fmax}{fo}\ \ln\left(\frac{2fmax}{fo}\right)}$	—	—
Hasegawa *et al.* (1981)	$M = M_L$ (W) $= m_b$ (E) $R = R_h$	Canada; $M = 4.0–7.0$; $R = 10–200$; Soil and rock	PHA_b	$Y = 1.02\text{x}10^{-2}\ e^{1.3M}\ R^{-1.5}$	(W. Canada)	—
				$Y = 3.47\text{x}10^{-3}\ e^{1.3M}\ R^{-1.1}$	(E. Canada)	—
			PHV_b	$Y = 4.00\text{x}10^{-4}\ e^{2.3M}\ R^{-1.3}$	(W. Canada)	—
				$Y = 1.80\text{x}10^{-4}\ e^{2.3M}\ R^{-1.0}$	(E. Canada)	—
Joyner and Boore (1981)	$M = M$ $R = R_s$	Western N.A.; $M = 5.0–7.0$; $R = 0–370$; Small structures; Soil and rock	PHA_1	$Y = 0.095\ e^{0.573M}\ D^{-1}\ e^{-0.0059D}$ $D = (R^2 + 7.3^2)^{1/2}$		0.60
			PHV_1	$Y = 0.85\ e^{1.13M}\ D^{-1}\ e^{-0.0059D}\ e^{0.39S}$ $D = (R^2 + 4.0^2)^{1/2}$ S = 0 (rock) 1 (soil)		0.51
Bolt and Abrahamson (1982)	$M = M$ $R = R_s$	Western N.A.; $R = 0.370$; Small structures; Soil and rock	PHA_1	$Y = 1.20\ [(R+23)^2 + 1]^{0.033}\ e^{-0.066(R+23)}$	(M=5.0–5.9)	0.06g (σ_y)
				$Y = 1.20\ [(R+25)^2 + 1]^{0.042}\ e^{-0.044(R+25)}$	(M=6.0–6.9)	0.10g (σ_y)
				$Y = 0.24\ [(R+15)^2 + 1]^{0.100}\ e^{-0.022(R+15)}$	(M=7.0–7.7)	0.05g (σ_y)
Campbell (1982)	$M = M_L\ (M<6)$ $= M_s\ (M\geq 6)$ $R = R_f$	Worldwide; $M = 5.0–7.7$; $R \geq 0$	PHA_m	$Y = 0.0137\ e^{0.777M}\ [R + C(M)]^{-0.797}\ e^{-\gamma R}$ $C(M) = 0.012\ e^{0.898M}$ $\gamma = \frac{0.898\ T^{\eta}}{Q_o T}$	—	0.41
Joyner and Boore (1982)	$M = M$ $R = R_s$	Western N.A.; $M = 5.3–7.7$; $R = 0–110$; Small structures; Soil and rock	PHA_1	$Y = 3.09\ e^{0.53(M-6)}\ D^{-1}\ e^{-0.0062D}$ $D = (R^2 + 8.0^2)^{1/2}$	—	0.64
			PHV_1	$Y = 148\ e^{1.13(M-6)}\ D^{-1}\ e^{-0.0060D}\ e^{0.39S}$ $D = (R^2 + 4.0^2)^{1/2}$ S = 0 (rock) 1 (soil)		
Seed and Idriss (1982)	$M = M_R$ $R = R_s$	Western U.S.; $M = 5.0–8.5$; $R = 2–160$ Rock	PHA_b	Graphical	—	0.45
Campbell (1983)	$M = M_L\ (M<6)$ $= M_S\ (M\geq 6)$ $R = R_f$	Worldwide; $M = 5.0–7.7$; $R \leq 50$ Small structures; Soil and rock	PHA_m	$Y = 0.0591\ e^{0.827M}\ [R + C(M)]^{-1.39}\ e^{0.34F}\ e^{-0.12E}$ $C(M) = 0.177\ e^{0.595M}$		0.38
			PVA	$Y = 0.279\ e^{0.687M}\ (R + 9.15)^{-1.68}\ e^{0.35F}\ e^{-0.31E}$		0.41
			PHV_m	$Y = 0.283\ e^{1.01M}\ [R + C(M)]^{-0.919}$ $C(M) = 0.00681\ e^{0.88M}$ F = 0 (strike-slip faults) 1 (reverse faults) E = 0 (ground level) 1 (embedded)		0.52
Idriss (1983)	$M = M_S,\ M_L$ $R = R_s$	Western U.S.; $M = 5.1–7.7$; $R = 3–100$; Stiff soil	PHA_m	$Y = a\ e^{bM}\ (R+20)^{-c}$	(Graphical)	Graphical
Okubo *et. al.* (1983)	$M = M_J$ $R = R_e$	Japan; $M = 5.0–7.9$; $R = 5–500$; Freefield; $H < 60$	PHA_b	$Y = 1.095\ e^{0.51M}\ (R+30)^{-1.25}$	(Rock)	0.50
				$Y = 0.232\ e^{0.71M}\ (R+30)^{-1.20}$	(Diluvium)	0.55
				$Y = 0.411\ e^{0.60M}\ (R+30)^{-1.21}$	(Alluvium)	0.58
			PHV_b	$Y = 23.86\ e^{0.63M}\ (R+30)^{-1.28}$	(Rock)	0.51
				$Y = 2.666\ e^{0.97M}\ (R+30)^{-1.18}$	(Diluvium)	0.56
				$Y = 5.136\ e^{0.95M}\ (R+30)^{-1.26}$	(Alluvium)	0.58

APPENDIX

GLOSSARY OF TERMS USED IN TABLE 2

Heading	Term	Definition
Parameters	m_b	Body-wave magnitude
	M	Magnitude (generic)
	$\mathbf{M}$	Moment magnitude
	M_J	JMA magnitude
	M_L	Local magnitude
	M_R	Richter magnitude
	M_S	Surface-wave magnitude
	R	Distance (generic, km)
	R_e	Epicentral distance
	R_{ec}	Distance to center of energy release
	R_f	Closest distance to fault rupture
	R_h	Hypocentral distance
	R_s	Closest horizontal distance to zone of energy release or surface projection of fault rupture
	R_{ze}	Closest distance to zone of energy release
Applicability	H	Focal depth (km)
	M	Magnitude
	R	Distance (km)
Y (Parameters)	Y	Strong-motion parameter (generic)
	AI	Arias intensity (cm/sec)
	PHA	Peak horizontal acceleration (g)
	PHV	Peak horizontal velocity (cm/sec)
	PVA	Peak vertical acceleration (g)
	PVV	Peak vertical velocity (cm/sec)
	RMSA	Root-mean-square acceleration (cm/sec)

APPENDIX (continued)

Heading	Term	Definition
Y (subscripts)	b	Use of both horizontal components
	1	Use of maximum horizontal component
	m	Use of mean of horizontal components
	v	Use of maximum vectorial combination of horizontal components
	3	Use of third largest peak
Model	A,B	Region variables
	A_o	Attenuation factor (Richter, 1958)
	E	Instrument embedment variable
	f_o	Spectral corner frequency (Hz)
	f_{max}	Maximum spectral frequency (Hz)
	F	Fault mechanism variable
	H	Focal depth (km)
	M	Magnitude
	Q_o	Specific attenuation factor (1 Hz)
	R	Distance (km)
	S	Site classification variable
	T	Predominant period (sec)
	V	Component variable
	V_s	Shear-wave velocity (ft/sec)
	η	Scattering parameter
	γ	Coefficient of anelastic attenuation (km^{-1})
	ρ	Specific gravity
$\sigma_{\ln Y}$	$\sigma_{\ln Y}$	Standard error of ln Y

THE ISSUES SURROUNDING THE EFFECTS OF GEOLOGIC CONDITIONS ON THE INTENSITY OF GROUND SHAKING

By

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INTRODUCTION

This paper will outline the issues related to site response effects and expand some of these topics based on data principally from the Los Angeles region. In some cases we will review and reevaluate old and new data from the perspectives provided by comparison of these data sets. In spite of fairly strong evidence showing the significance of site conditions on the level of ground shaking, questions continue to be raised about either the importance of site effects or whether they can be accurately predicted. We outline many of these questions and address some of them in the following discussion.

THE ISSUES

A. What do we know about site response?

  a. What is the range and nature of site effects in the time and frequency domain?

    i.) What are the effects of site conditions on peak ground motion parameters?

    ii.) Is the influence of site effects minimal in peak acceleration data sets?

    iii.) If site effects do not significantly contribute to peak acceleration variability, are these effects important to consider in design?

    - iv.) What are the effects of site conditions on the ground motion spectrum? How important is site resonance and under what conditions does it occur?
    - v.) What is the evidence for non-linear soil behavior?
    - vi.) When non-linear soil behavior occurs, how important is it, and is its effect on the site transfer function significant?
    - vii.) Will all soils in the zone of non-linearity have the same level of ground shaking?
  - b. How great is the variability in site effects?
    - i.) Are site effects at a given location reproducible within useful statistical bounds?
    - ii.) Is site response too variable geographically to accurately predict without unreasonable quantities of geotechnical information?
    - iii.) Is the rate of geographical variability in site response a function of the period band of interest?
    - iv.) Do surface topography and lateral subsurface heterogeneity produce important changes in site response, particularly as epicentral distance or source-receiver azimuth changes?
- B. Can site response effects be predicted?
  - a. What theoretical methods exist?
    - i.) Are linear or non-linear methods most appropriate?
    - ii.) How well do theoretical models predict observed site response?
  - b. What empirical methods exist?
    - i.) Does non-linear soil behavior invalidate empirical methods of prediction that have been based on low-level ground motions?
    - ii.) Do low-level, ground-motion measurements of site response significantly overestimate site effects during strong shaking?
    - iii.) How well do empirical models predict observed site response?
  - c. Are prediction methods available that will work in any region and that reconcile to some degree the diverse site response observations from many regions into a unified picture?

C. Do we need to perform additional experiments?

  a. Are the geologic and ground motion data bases adequate to study site response effects?

    i.) Are they of high quality, sufficient quantity, and appropriately collected?

    ii.) Has soil-structure interaction significantly modified the strong motion data base?

  b. What experiments might be performed to resolve the issues?

    i.) Are instrument arrays in place that would provide data to further test the importance of non-linear soil behavior?

    ii.) Are instrument arrays in place that would provide data to test various prediction schemes?

    iii.) What geotechnical data are needed, and how will the data be collected?

    iv.) How can existing data be further analyzed to study site effects?

D. How can site response predictions and data best be utilized?

  a. Should design ground-motions be modified for site response?

    i.) Are the currently employed mean spectral shapes adequate to describe changes in site response as a function of geologic conditions?

    ii.) Do we know how to modify design coefficients or synthethic seismograms to account for site response in a realistic way?

    iii.) How will imbedment modify the design motion?

    iv.) How will soil-structure interaction modify the design motion?

  b. How should site response predictions be used for urban planning, disaster planning, and loss estimation?

    i.) Are site response maps the most useful form of input to these studies?

THE EXISTENCE OF SITE SHAKING EFFECTS

A reasonable starting point is to address a more basic question than those listed above. That is, is site response an important phenomena that significantly affects the level of strong shaking at ground periods of engineering interest. This question has been frequently addressed in the literature with responses ranging from "very significant" to "not important in this region" to "important, but impossible to predict." The preponderance of data, however, including damage statistics, intensities, strong motion measurements, and low-level ground-motion measurements indicates that site response effects are significant. The Japanese and Russians have examined numerous case studies showing that the geographic distribution of damage and intensity is in some way related to the thickness and/or other physical properties of the near-surface sediments (Kanai, 1952; Ooba, 1957; Minakami and Sakuma, 1948; Medvedev, 1962). Using recordings of low-level ground motions produced by distant nuclear explosions, Borcherdt and Gibbs (1976) found a high correlation between geographic changes in intensities in the 1906 San Francisco earthquake and geographic changes in mean spectral amplification. This correlation exists up to the highest shaking levels that occurred in that earthquake (Modified Mercalli $\geq$ X). Site effects have recently been isolated in the strong motion data from the 1979 Imperial Valley earthquake (Mueller and others, 1982) and are also observed in the strong motion data recorded during the Fruili and Ancona, Italy earthquakes (Chiaruttini and Siro, 1981). Analysis of the San Fernando earthquake data set by Rogers and others (1983a) shows that site effects occur at the 28 strong-motion sites they studied. For these sites mean spectral amplification on soil ranged as high as a factor of 5. Data at other Los Angeles sites, employing recordings of Nevada Test Site nuclear explosions, indicated that at some alluvium sites mean spectral amplifications as large as 11 are observed. Many other examples of ground motion studies could be cited that demonstrate the importance of geologic conditions on strong ground shaking.

A DISCUSSION OF THE IMPORTANCE OF NON-LINEAR SOIL BEHAVIOR

Although laboratory data suggest that soils behave in a non-linear fashion when strain levels exceed 10^{-5} (Seed and Idriss, 1970) or 10^{-4} (Turner and Stokoe, 1982), field data have been collected suggesting that high- and low-amplitude soil response are similar, perhaps for strains up to 10^{-3}. Rogers and others (1983a) compare site shaking effects measured using distant nuclear explosions (strains near 10^{-5}) with site effects at the same locations measured with data from the 1971 San Fernando earthquake (strains near 10^{-3}) and find that the two data sets are equivalent within the expected variability of the statistics. Rogers and Hays (1978) using strong ground motion recordings of nuclear explosions on the Nevada Test Site, find that site response measured at strains of 10^{-3} and 10^{-4} are approximately equal. Joyner and others (1981) find that site response can be predicted using linear theory for strains up to 10^{-4}. Several recently collected strong motion data sets also indicate that if non-linear soil behavior is a factor limiting the magnitude of ground motions on soils, the limiting values may still be quite high (Mueller and others, 1982; Chiaruttini and Siro, 1981). The Borcherdt and Gibbs (1976) comparison of intensity increments and mean spectral ratio, cited above, may also be evidence that nonlinear soil behavior is of secondary importance.

The effect of non-linear soil response may also be restricted to a small area surrounding the causative fault (Hays and Algermissen, 1982). For instance, a magnitude 7-7.5 earthquake develops velocities on soil sites exceeding 100 cm/s at distances less than 7-13 km (50 percentile, Joyner and Boore, 1981). For soil sites with 200 m/s shear velocities, strains of 5×10^{-3} will be developed within this zone. Based on the observations discussed above this strain level may still be below the level of significant non-linear behavior. Because damaging motions on soils IMM > VI occur to distances of 60-100 km (50 percentile) for a 30 km rupture, the area of damage susceptible to non-linear soil response is about 2-9% of the total area of damage. In cases where the fault passes through developed regions, however, the zone of non-linear behavior may be the zone of greatest life loss, although a high percentage (as much as 90%) of the economic loss occurs outside this zone (Algermissen and others, 1972). In other cases where the fault passes outside the urban zone or the earthquake is too small to induce non-linear soil response, this behavior will not be a factor at all. The 1967, $M = 6.4$, Caracas, Venezuela earthquake is a good example that demonstrated strong site effects withing a city that experienced numerous building failures and several hundred lives lost for an earthquake 50 km away (Seed and others, 1972). The 1977 Romania earthquake ($M = 7.2$; ISC, 1977) was centered about 120 km from Bucharest, where 1387 people were killed. Bucharest is underlain by alluvium, some of Holocene age (Carte des Sols, 1964), that we infer had a role in the damaging ground motions that occurred there, but because of the distance (120 km) from the earthquake fault, the alluvium is unlikely to have behaved nonlinerarly (the peak velocities in Bucharest are estimated to be about 50 cm/s (Skinner, 1977)).

PREDICTION OF SITE RESPONSE EFFECTS

Prediction of site response has frequently been attempted by several empirical techniques. In one empirical method, low-strain measurements of small earthquakes or distant nuclear explosions are obtained over a region at rock and soil sites in patterns dense enough to essentially measure relative site-response (Hays and Algermissen, 1982; Murphy and Hewlett, 1975). Another empirical technique compares limited measurements of site-response, relating response to known geological data, and predicting relative shaking effects, over a broad region or to a specific site, based on mapped geologic units having distinctive seismic behavior. This technique, using Nevada Test Site nuclear explosions, was first examined by Borcherdt (1970) using recordings in the San Francisco region and by Murphy and others (1970) in the Las Vegas region. These studies demonstrated that mean spectral amplification from nuclear explosions is correlated with changes in earthquake shaking intensity and that, in cases where geologic structure and shear velocities are well known, the measured site response can be predicted from theory with fair success. Borcherdt and Gibbs (1976) also established a correlation between the age of surficial deposits and mean spectral amplification that enabled them to produce a regional map showing the predicted relative geographic changes in shaking intensity. These studies in addition to studies by Rogers and others (1983a), Mueller and others (1982), and King (1982) suggest that in many cases differences in wave types, angles of incidence, and source azimuth have a second order importance relative to site response and do not mask the underlying character of the site shaking effect.

In this study we extend the technique developed by Borcherdt and Gibbs (1976) to the Los Angeles region and recast the technique to include the effects of geologic structure and nearsurface site properties. In order to determine relations between local geologic factors and shaking amplification, nineteen nuclear explosions were recorded at 98 sites throughout the Los

Angeles region (Figure 1). Sites for the study were chosen to obtain as complete a sample of underlying geologic conditions and as broad a geographic coverage as possible. Because the seismic source lies between 400 and 450 km from the recording sites, effects of azimuthal variations in the energy radiated by the nuclear source and major portions of the crustal propagation paths are similar for all sites. Estimation of each site's response characteristics over the period band 0.2 to 10 seconds was accomplished by computing Fourier spectra and alluvium-to-crystalline rock spectral ratios (Rogers and others, 1980). The site CIT, underlain by crystalline rock, was occupied for every recorded nuclear explosion and served as the base rock site.

In the case of ground motions from distant nuclear explosions, the effects of site conditions predominate on the recorded time histories. For example, Figure 2 shows time histories recorded simultaneously at eight sites from a single Nevada Test Site nuclear explosion. The example illustrates several effects local site conditions commonly have on the recorded time histories from distant sources of shaking. For instance, maximum amplitudes of motion recorded on the alluvial sites are several times larger than those recorded on the sedimentary or the crystalline rock sites. The degree of amplification occurring in the long-period peak amplitudes of these records is greatest at sites underlain by the thickest sediments. Comparison of all three components (Rogers and others, 1980) shows that the amplification of horizontal ground motions is commonly larger than the amplification of the vertical motion; in the following discussion, however, only the horizontal components of ground motion will be emphasized because of their predominant importance in structural engineering.

Rogers and others (1980) have shown that the spectral ratios are very reproducible from one nuclear explosion to the next. The geometric means and geometric standard deviations were computed for 9 sites, showing that the geometric standard deviation averages 1.38 and is approximately independent of frequency. By comparison, this dispersion of spectral ratios is lower than that associated with the empirical prediction of root-mean-square acceleration on rock (Hanks and McGuire, 1981).

The amplitude spectral ratios computed for the simultaneous recordings shown in Figure 2 are presented in Figure 3, where station CIT has been used as the reference station. The ratios show that the effects of site conditions relative to those at CIT are strongly frequency dependent, with amplification occurring for many of the sites over most of the frequency band for which a good signal-to-noise ratio exists (Rogers and others, 1980). Horizontal amplification factors in the range 2-7 are apparent for the lower-frequency ground motions (<1 Hz) for those sites on thick sections of alluvium, with lower amplifications being apparent at these frequencies for the site underlain by a thin section of alluvium. Considerable amplification at intermediate frequencies (1-2 Hz) and at higher frequencies (2-5 Hz) is readily apparent at several of the sites, especially FS4, where a predominant ground resonant frequency is observed. Note that resonance is not a factor for the thick alluvium sites, which display relatively flat spectra across the entire observed frequency range. The spectral ratios for the GOC site suggest that the response of the two crystalline rock sites (GOC and CIT) is very similar for the lower frequencies, but the intermediate and higher frequency motions recorded at GOC are larger than those recorded at CIT. Site 3838, located on sedimentary rock, shows a uniformly higher response compared to CIT over most of the frequency band.

COMPARISON OF GROUND RESPONSE WITH GEOLOGIC FACTORS

Geotechnical data describing underlying site conditions and spectral ratios, reduced to mean spectral response values over several period bands, were assembled into a data matrix to facilitate study of the relations between response and the geotechnical attributes (Table 1). These parameters were chosen to characterize the recording sites because either the parameters have some direct application in a theoretical model of site response or the parameters have been reported to have some influence on ground shaking in past studies. Thus, parameters such as percent silt-clay, percent saturation, and depth-to-water table have been reported to influence site response, while shear velocity (or void ratio, which strongly influences the shear modulus), Holocene thickness, Quaternary thickness, and depth-to-basement are all parameters that might be used directly in a model of site response. Most of these data are obtainable from geologic maps, well logs, and city files

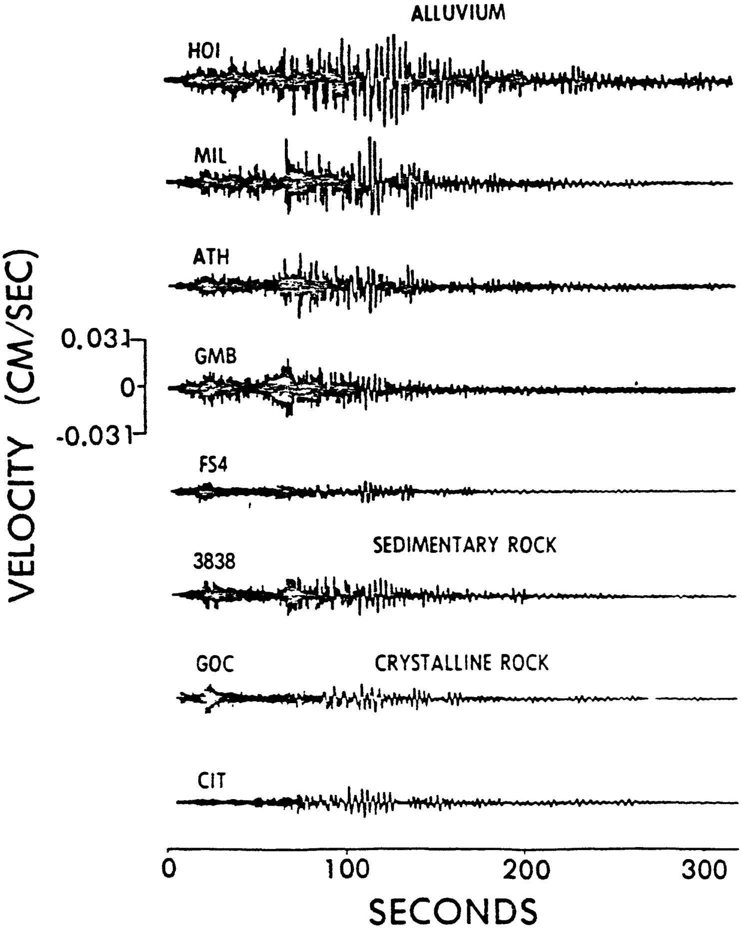

Figure 2.—Radial component time histories of shaking from a distant underground nuclear explosion in Nevada recorded simultaneously at eight sites, in the Los Angeles region, grouped according to type of geologic materials immediately beneath earth recording station. The ground motions are clearly amplified at locations underlain by alluvium compared to locations underlain by rock. The degree of amplification also appears to be related to the thickness of underlying alluvium (HOI: 300 m; 372 m: ATH: 372 m GMB: 120 m; FS4; 15 m).

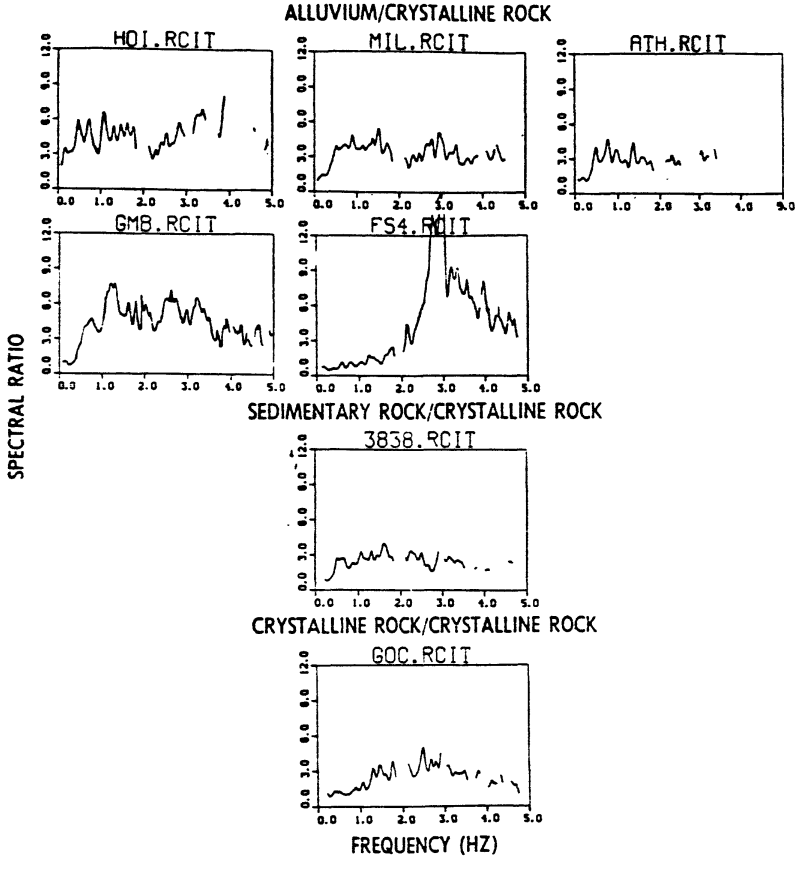

Figure 3.—Spectral ratios of the radial components of ground motion recorded on sites of various geology (see Figure 3) relative to the crystalline rock site (CIT). These diagrams show that spectral site amplifications range from about 1 to greater than 12, with the highest ratios occurring at sites underlain by alluvium. Site FS4 displays a resonant frequency between 2 and 3 HZ, whereas other sites have more uniform amplification, averaging as much as 6 in some frequency bands.

Table 1. List of geotechnical parameters compiled for each station and the percentage of stations for which each parameter was available. The spectral response parameters were available for all stations.

Spectral Response Date	Geologic Data	
Short-Period Band	Type	Percent Reported
0.2 - 0.3 s		
0.3 - 0.5	Mean Void Ratio (0-8m)	82
0.2 - 0.5		
Intermediate-Period Band	Mean Percent Silt-Clay (9 Depth Ranges)	37-95
0.5 - 1.0 s		
0.5 - 3.33	Percent Saturation	13
Long-Period Band	Quaternary Thickness	100
1.0 - 10.0 s		
1.0 - 3.33	Age	100
3.33 - 10.0		
	Holocene Thickness	99
Total-Period Band	Depth to Water Table	97
0.2 - 10.0		
0.3 - 10.0	Sediment Type (Very Coarse to Fine)	99
	Depth to Crystalline Basement	100
	Depth to Cementation	91
	Mean Bore-Hole Shear Velocity (4 intervals)	51

containing engineering boreholes for construction projects. The data, therefore, would be of great value if related to site response in some quantifiable manner.

To examine the relation between site response and the geological parameters, the most straightforward approach is to group the sites according to variations in one of the geologic factors and to compute mean response for each group. Table 2 indicates the following ground response characteristics: 1.) Sites underlain by Holocene and Pleistocene sedimentary deposits undergo levels of shaking 2.6 to 3.4 times greater than those underlain by crystalline rock for all period bands; 2.) The void ratio has a strong influence on short-period response, with void ratios in the 0.8-0.9 range indicating a mean response on soil 6 times greater than on crystalline rock and 3 times greater than on low-void-ratio soils; 3.) Amplitudes in the long-period band generally increase with increasing thickness of Quaternary deposits and/or depth to basement.

More detailed examination of the influence of all the geologic parameters using the methods of exploratory data analysis (Mosteller and Tukey, 1977; Velleman and Hoaglin, 1981) indicate that the strongest changes in site response were correlated with changes in void ratio, Holocene thickness, depth to basement, and Quaternary thickness. Applying the smoothing techniques of exploratory data analysis, it is frequently possible to extract the influence of one factor on the response variable given a body of data in which several factors are changing simultaneously. Figure 4, for instance, shows the smoothed short-period ratio plotted against Holocene deposit thickness (A) and void ratio (B). The peak in short-period response for Holocene thicknesses near 15 m is due to the shift through this period band of the fundamental resonance period of the Holocene layer. The general increase in the short-period response as void ratio increases is principally due to the increasing shear-wave velocity contrast at the Holocene-Pleistocene boundary. For comparative purposes, theoretical spectral ratios were computed using a horizontally layered SH-body-wave model and assuming constant Q. The physical properties of the geologic column used in this modelling were generated by computer from the geologic data matrix with variable surface layer velocities, fixed lower layer velocities, and depths to velocity contrasts determined by Holocene deposit thickness, Quaternary thickness, and depth to basement (Figure 5). Surface layer velocities were either measured borehole shear

Table 2. Comparison of Shaking Response (relative to crystalline rock) from distant nuclear explosions recorded at sites within the Los Angeles region.

A) Response in three period bands compared with age of materials.

Age	Short-period (0.3 - 0.5s)	Intermediate Period (0.5 - 3.3s)	Long-period (3.3 - 10.0s)
Holocene	3.4	3.3	2.6
Pleistocene	3.2	3.1	2.6
Pliocene	1.4	1.6	2.0
Miocene	2.5 [1.9]	2.2 [1.8]	1.4 [1.3]
Mesozoic	1.7	1.1	0.8

B Short-period response compared with average void ratio in the uppermost 8 m.

Void Ratio	Short-period Mean Response (0.3 - 0.5s)
0.2 - 0.4	2.3
0.4 - 0.6	3.1
0.6 - 0.7	3.0
0.7 - 0.8	4.2
0.8 - 0.9	6.2

C) Intermediate- and long-period response compared with thickness of Quaternary deposits.

Quaternary Thickness (M)	Intermediate Period Mean Response (0.5 - 3.3s)	Long-period Mean Response (3.3 - 10.0s)
0	1.6	1.3
0-75	2.3	1.4
75-200	3.6	2.9
200-500	3.6	3.1
500-1000	4.1	5.9
> 1000	3.4	3.1

D) Intermediate- and long-period response compared with depth to basement rocks.

Depth to Basement (km)	Intermediate Period Mean Response (0.5 - 3.3s)	Long-period Mean Response (3.3 - 10.0s)
0	1.1	0.8
0-2	2.6	1.3
2-4	2.8	2.5
4-6	3.8	4.1
6	3.8	3.9

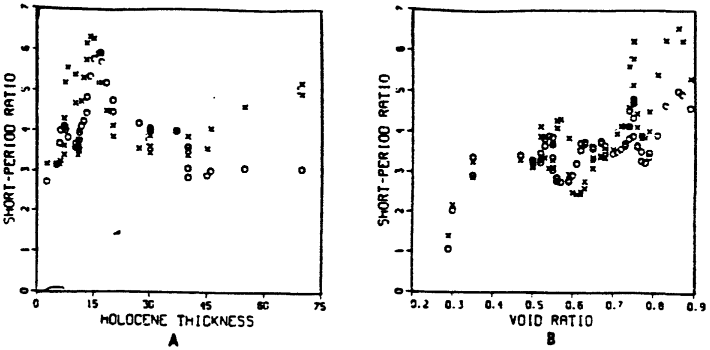

Figure 4.—Smoothed short-period response at sites on Holocene deposits in comparison with Holocene thickness (1) and void ratio (B). In order to minimize the influence of void ratio in (A), only sites with void ratios greater than 0.65 were included. A circle or x indicates theoretical or observed values, respectively.

		SHEAR VELOCITY (m/s)	DENSITY (gm/c³)	Q	REMARKS
HOLOCENE	t_H	V_m V_c	1.46 g/cm³	50	V_c=42.9+94.1/e²
PLEISTOCENE 1	t_{p_1} = QTHK — t_H	350m/s	2.10	200	t_{p_1} NOT ALLOWED TO EXCEED 100m
PLEISTOCENE 2	t_{p_2} = QTHK — t_{p_1} — t_H	600m/s	2.10	200	ONLY PRESENT WHEN QTHK-t_H>100m
TERTIARY ROCK	T_t=dtb—QTHK	1500m/s	2.35	500	ONLY PRESENT WHEN dtb>QTHK
CRYSTALLINE BASEMENT		3000m/s	2.50		

QTHK=THICKNESS OF QUATERNARY DEPOSITS
dtb=DEPTH TO CRYSTALLINE ROCK

Figure 5. Shear-wave velocity model used to compute theoretical damped SH-wave response.

velocities (Fumal and others, 1982) or were computed from void ratios. The theoretical spectral ratios and mean spectral values were processed in exactly the same fashion as the observed quantities. The concordance between the observed data and the theory supports our interpretation of the observed behavior. While similar analysis of other factors indicates that variables such as depth to basement and Quaternary thickness have an effect on the short-period response, these and other variables are secondary in importance to Holocene deposit thickness and near-surface void ratio.

CLUSTERING OF SITES BY GEOLOGIC ATTRIBUTES TO REFLECT SHAKING RESPONSE VARIABILITY

Sites with similar response characteristics can be clustered by computing an analytical measure of similarity between a list of items based on their attributes. In our analysis, the items are recording sites and the attributes are the geotechnical properties of each site. (Note that we do not use the response factor as an attribute because we are attempting to predict response as a function of the geologic properties of the site.) The clustering algorithm (Anderberg, 1973; Hartigan, 1975; IMSL, 1982) uses a computing rule to establish those items most nearly alike and the similarity level at which clusters of similar items are alike. The results can be plotted as an inverted hierarchical tree of similarity nodes. The choice of a similarity level below which clusters form is a subjective judgment and in practice may change across the cluster diagram.

Once a set of clusters is formed by this procedure on a chosen set of factors it is possible to analyze the degree to which these factors define unique groups using discriminant analysis (Morrison, 1969; Morrison, 1974; Cooley and Lohnes, 1971; Nie and others, 1975), which determines the significance of each factor's discriminating power using the statistics of factors within and between clusters. A set of discriminant functions are computed that enable one to calculate the probability that a single member of a cluster belongs to that cluster or any other cluster. Given a table of these probabilities it is possible to calculate the percentage of sites that have been correctly classified.

In our application, this procedure was a trial and error process, during which some of the data analysis described above was done concurrently with the cluster and discriminant analysis. At the start, the site response data were first divided into rock and alluvium groups, and then clusters were examined in which many or all of the measured factors were part of the clustering model. This approach, however, produces too many clusters, with too few station members. The number of factors in the clustering process was reduced iteratively by gradually discarding factors having low statistical significance in the discriminant analysis.

Comparison of the mean response values in each cluster set for the three period bands revealed that some clustering parameters reduced the response variance in each cluster better than others, in accord with the results of the preliminary data analysis showing that the most important factors in each period band should be different. Ultimately, the cluster sets selected were chosen because they had the lowest dispersion in the defining variables while employing those factors having the strongest effect in a given period band; in addition, we required that the probability of misclassification be low, and that each cluster in the set had a sufficient amount of data to estimate the mean cluster properties. The final sets of clusters are a compromise between the many clusters required to preserve the complexity in the site response as a function of geology, and the requirement that each cluster contain enough cases to estimate its average response.

Figure 6 shows the set of two rock and eight alluvium clusters that were derived for the short-period band. This figure can be understood by using cluster 4A as an example; that cluster includes sites with depth to basement rocks greater than 0.5 km, Holocene deposit thickness greater than 20 m, void ratios in the range 0.6–0.7, and a geometric mean response of about 3.6. Using these clusters to predict response preserves the important features of site behavior noted above. For instance, for a fixed Holocene deposit thickness, response increases as void ratio increases (i.e., compare clusters 1A, 3A, and 6A, for example). Response also increases, for a constant void ratio, as the Holocene deposit thickness increases to the critical range (i.e., compare clusters 6A, 7A, and 8A, for example). Note that the clusters with thin Holocene cover also contain most of the Pleistocene sites; those Pleistocene sites in the thickness range 11 to 20 m, however, are grouped with

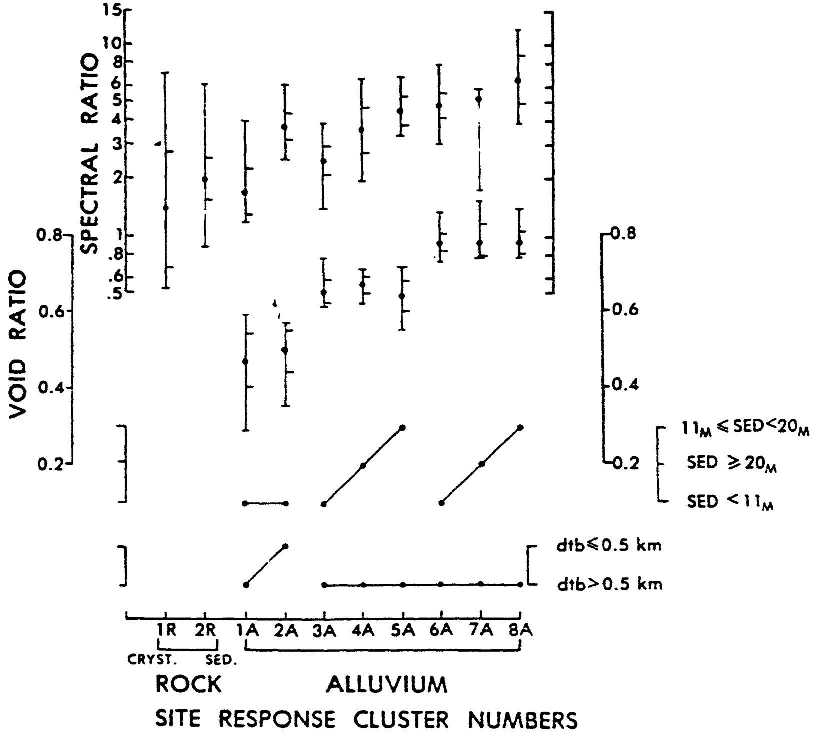

Figure 6.—Site clusters for short-period ground motions in the Los Angeles region. Solid dots indicate the mean of the short-period spectral ratios for a given cluster, mean void ratio Holocene thickness and depth to basement rock groups, as appropriate. Vertical bars indicate the range in the variable for a give cluster, and side ticks indicate the 90% confidence intervals.

the Holocene sites in this range. The rock sites 1R and 2R indicate a geometric mean response that is typically lower than that of the alluvium clusters, as might be predicted on the basis of surface velocities. A comparison of clusters 1A and 2A shows that sites underlain by shallow alluvium over crystalline rock (2A) have a response two times higher than the same type of site overlying a deep sedimentary basin, further emphasizing the role of high impedance contrasts at shallow depths. Even though we were able to divide the sites into only ten clusters, resulting in a moderate range in the geologic and response factors in each cluster, a useful result can be demonstrated by comparing average spectral level with shaking intensity. It is reasonable to assume that a factor of two in mean spectral level corresponds to a change of one MM (Modified Mercalli) intensity unit (Borcherdt and others, 1975); we infer, then, from the data of Figure 6 that these clusters predict the true site-response more closely than one intensity unit increment for 90 percent of the cases, because the geometric 90 percent confidence interval is less than a factor of two (1.45). Clusters were derived for the intermediate- and long-period bands based on Quaternary thickness and depth-to-basement rock. These clusters, however, will not be discussed here.

SITE-RESPONSE PREDICTION MAPS FOR A PORTION OF THE LOS ANGELES REGION

Response maps for the intermediate- and short-period bands for a small area approximately centered on the Los Angeles Civic Center are shown in figures 7a and 7b, which are based on the clusters discussed above and on a set of maps delineating the geographic distribution of the important geotechnical factors. The intermediate-period map (Figure 7a) of significance to structures between 5 and 30 stories in height, predicts that low response will characterize areas underlain by rock and by thicknesses of alluvium less than about 150 m; intermediate levels of response will occur where alluvial thickness is greater than 150 m and/or depth to basement rocks is in the 0.15 to 4 km range; highest levels of response will occur in areas where the depth to basement rocks ranges between 4 and 6 km. Slightly lower levels of response are predicted in the deepest parts of the Los Angeles basin. The lowest response will be in the areas where crystalline basement is located at or near the surface, and slightly higher response occurs where crystalline rocks are overlain by sedimentary rock. South of Burbank and west of Pasadena, the

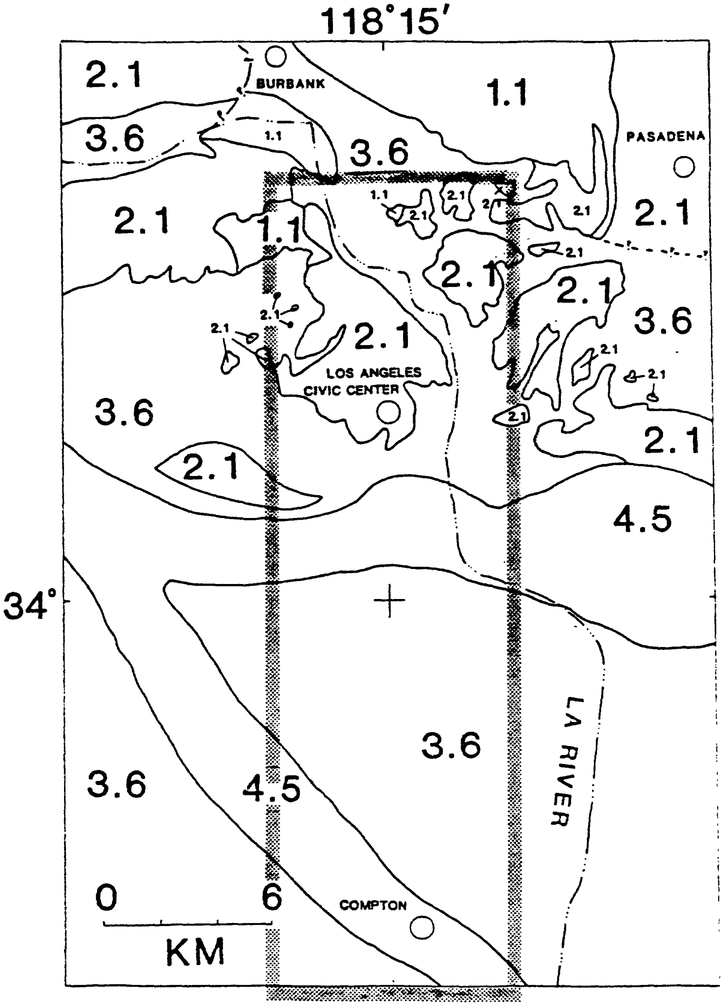

Figure 7 (a & b). Maps of predicted relative shaking response for part of the Los Angeles basin. Numbers are mean amplification factors, comparing levels of shaking to sites on crystalline basement rock.

a. Map for intermediate-periods (0.5-3.3 seconds) a band of significane of buildings 5 to 30 stories in height. Stippling outlines area of Figure 7b.

relatively thin alluvium in the intermontaine basin areas and along the Los Angeles River valley north of the Los Angeles Civic Center also will exhibit a low to intermediate response. In the southwestern part of the map, intermediate levels of response will be observed where crystalline basement rock is about 3 km deep along the Newport-Inglewood structural zone (Yerkes and others, 1965). The long-period map is similar to this map except that it predicts low response in regions where alluvium thicknesses are less than 300 m and/or depth to basement rock is less than 3 km.

The short-period map (Figure 7b), which is most relevant to buildings in the 2 to 5 story class, has been prepared for the central third of the area shown in the long-period map. The lowest response is predicted for areas underlain by crystalline and sedimentary rock, and the highest response occurs in regions where thicknesses of near-surface alluvium (11-20 m) and high void ratios (greater than or equal to 0.7) produce significant resonant response in this period band. In some respects, this map closely resembles a surficial geologic map; thus, details of the alluviated valleys, including that of the Los Angeles River, are delineated. The southwest part of the map depicts an area where silt (characterized by high-void-ratios) deposited by the Los Angeles River thin to the west and wedge out along the east flank of the Newport-Inglewood zone where deformed Pleistocene deposits characterized by low-void-ratios are exposed. It should be noted that high short-period response may occur at sites underlain by rock, if these sites are near the crest of a ridge or other pronounced topography, as shown by the range of high response for clusters 1R and 2R (Figure 6).

SUMMARY

This technique has important implications for earthquake-hazard reduction in urban areas because it can be used to predict future relative shaking response from geotechnical data that are ordinarily obtained in the course of urban development. The ability of this technique to provide estimates of shaking response for period bands pertinent to structures of varied size suggest that the mapping procedure could be used for seismic zonation in many earthquake-prone urban areas, provided that the specific geologic framework of the regions being studied fit the clustering scheme developed here.

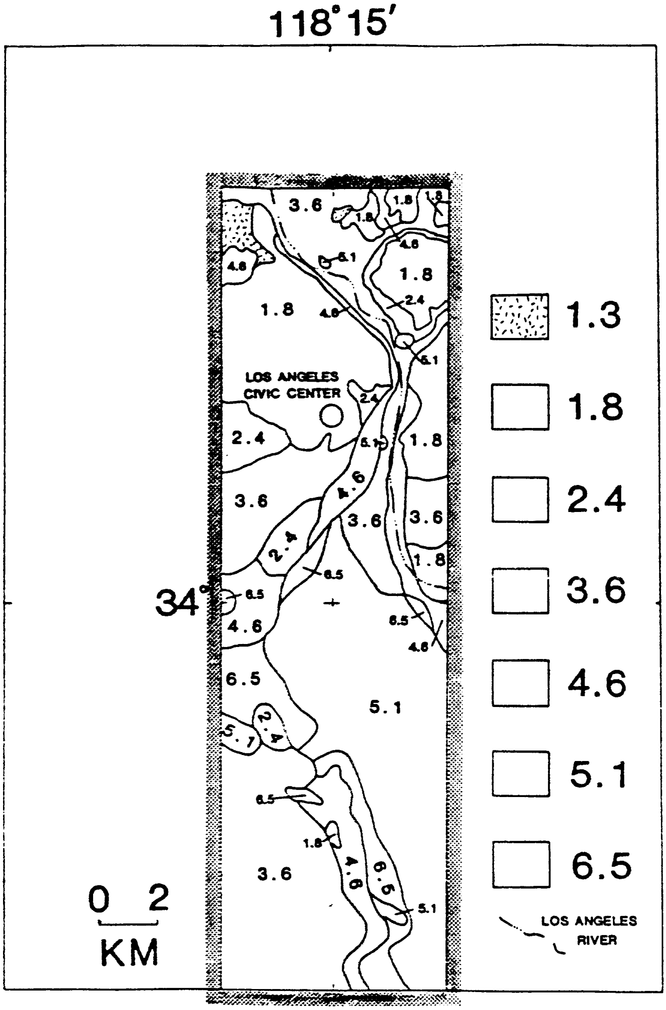

Figure 7 (a & b). Mpas of predicted relative shaking response for part of the Los Angeles basin. Numbers are mean amplification factors, comparing levels of shaking to sites on crystalline basement rock.

b. Map for short-periods (0.2–0.54 seconds), a band of significance to buildings 2 to 5 stories in height.

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GEOLOGIC EVIDENCE OF EARTHQUAKES:
SOME GEOLOGIC STUDIES AND APPLICATIONS IN ESTIMATING THE SEISMIC HAZARD

By

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INTRODUCTION

The purpose of this lecture is to 1) present a general overview of some geologic investigations that have been performed in a variety of seismotectonic settings, 2) provide a perspective on the research effort required to obtain useful geologic information relevant to estimating the earthquake hazard within these settings, 3) review the general conclusions of these studies, and 4) discuss some applications of this research to estimating the regional seismic hazard. The intention is to contrast the nature of geologic studies and applications between seismic areas of active surface faulting and seismic areas having no surface faulting, hence, uncertain association between seismicity and geologic structure. Example studies are drawn from the San Andreas fault system, an active transform plate boundary; the Great Basin, a region of active intraplate extension; and the New Madrid region of the northern Mississippi Embayment, an intra-plate seismic zone (Figure 1).

GEOLOGIC INVESTIGATIONS OF ACTIVE FAULTS

State-of-the-art geologic investigations of active faults require the talents of geologists trained in Quaternary geology and geomorphic processes. This fundamental requirement may not be easily fulfilled in countries where organized geologic-hazard investigation programs do not exist. Professional and monetary rewards for the trained geologist most often lie in the classical fields of geologic investigation such as petroleum geology and economic ore deposits. To best motivate evolving and lasting

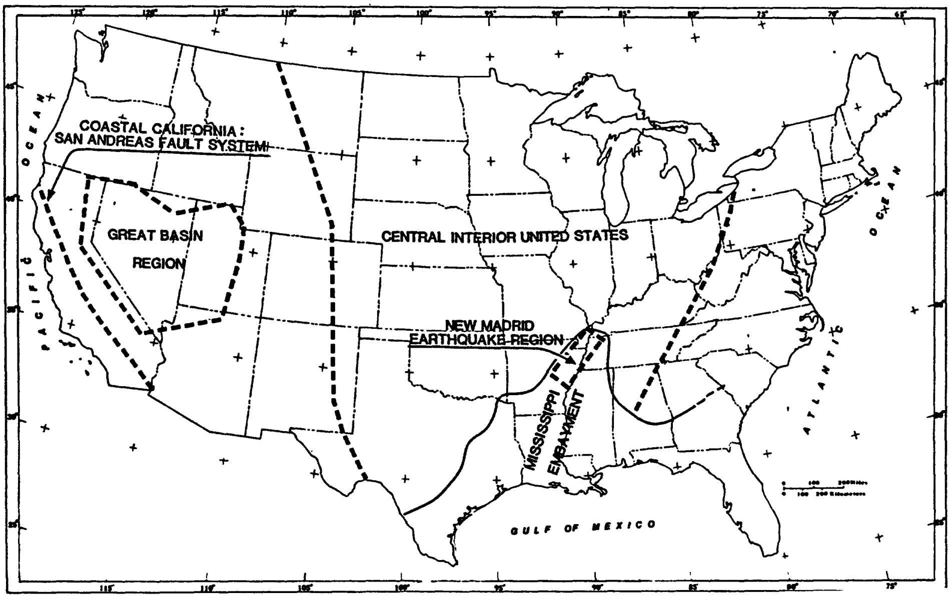


Figure 1.—Index of map of regions referred to in text. Reference for figures 2 through 6 are cited in text.

research contributions to seismic fault studies, an infrastructure for financial support and professional recognition of such work needs to be established within existing geological organizations.

SOME USEFUL METHODS OF INVESTIGATION

Fault-Trenching Studies

A widely used, relatively inexpensive method of investigation to determine ages of geologically young fault movements is to excavate a trench across the surface trace of an active fault. By mapping in detail the walls of the trench, noting relations of fracture zones or disturbed zones within the alluvium, offset soil horizons, and cross-cutting relations such as sand-dike injections, a geologist can reconstruct the sequence of faulting episodes. If the location of the trench has been carefully chosen to optimize the opportunity of cutting carbonaceous layers, absolute ages of the paleo-soil horizons that are offset by faulting can be determined through Carbon-14 radiometric dating. These data then limit the ages of the faulting events, and a recurrence time of assumed large, fault-rupturing earthquakes can thus be estimated.

The most acclaimed trenching study in the United States was performed on the San Andreas fault by Dr. Kerry Sieh of the California Institute of Technology. The trenchsite neighbors a small creek bed that crosses the San Andreas fault and has a repeated history of a swamp environment within the last 2,000 years. C^{14} dating of peat layers within the surficial stratigraphy provided excellent time-stratigraphic control of offset soil horizons. The conclusion drawn from this study was that the recurrence of large earthquakes along this segment of the San Andreas fault is nonuniform; long intervals alternate with short intervals between large earthquakes. The <u>average</u> recurrence interval is about 160 years with periods between large earthquakes ranging from as short as 57 ±9 years to as long as 275 ±68 years within a 1400 year period. The size of the ancient fault-rupturing events is critical but difficult to determine precisely. By comparing the offsets of the various events exposed in the trench with that of an historic great earthquake that broke this segment of the fault in 1857, Seih (1978) bounded this problem with three possible models: 1) all 9 earthquakes are truely great earthquakes ($M > 8$), 2) seven of the nine earthquakes might be less than $M=8$, but still large events; 3) five of the nine events are truely great earthquakes with the intervening events being less than $M=8$ but greater than $M=7$. Comparison of the rates of great earthquake occurrence derived from trenching data with rates derived from historic seismicity shows that the geologic recurrence intervals are 2.5 to 4.5 times shorter than those derived from the historic earthquake catalog (Thenhaus and others, 1980). Because of uncertainties in both types of estimates, such comparisons provide a valuable perspective that can guide future investigations depending on the proposed application of the hazard map, or the objectives of a particular earthquake hazards investigation project. An important point is that trenching studies address, in general, only high magnitude earthquake occurrences. In regions of moderate to low earthquake activity, the most probable earthquake ground motions to affect buildings having normal lifetimes will come from the relatively more frequent moderate-sized earthquakes rather than the rare, very large earthquake.

Rates of Fault Slip and Earthquake Recurrence

A recently developed method allows the rate of earthquake occurrence along a fault to be calculated if the long-term average slip rate of the fault is known (Molnar, 1979; Anderson, 1979). By assuming that all of the fault slip is accommodated by seismic slip (that is, no aseismic fault creep has occurred), and a relative occurrence frequency of earthquakes within a range of magnitudes (b-value) up to some maximum-size earthquake expected on the fault, an absolute frequency of earthquakes can be obtained.

In areas of active faulting, rates of fault slip can usually be assessed from existing literature or existing geologic mapping by noting the age and distance of relative displacement of a feature, or rock unit, across a fault. The seismic moment (M_0) of any given earthquake is: $M_0 = \mu A \overline{u}$ where μ is the shear modulus, A is the fault area (the length and depth of fault rupture), and $\overline{u}$ is the average slip on the fault during the earthquake. Given a sufficiently long period of time, the sum of all individual earthquake displacements along the entire fault will have caused the noted offset of the geologic feature. The average recurrence interval for earthquakes having moments greater than M_o but not greater than $M_o^{\max}$ is given by the equation:

$$T\ (M_o) = \frac{M_o^{\max\ 1-\beta}}{(\ 1-B)\ \dot{M}\ \S}\ M_o^{\beta}$$

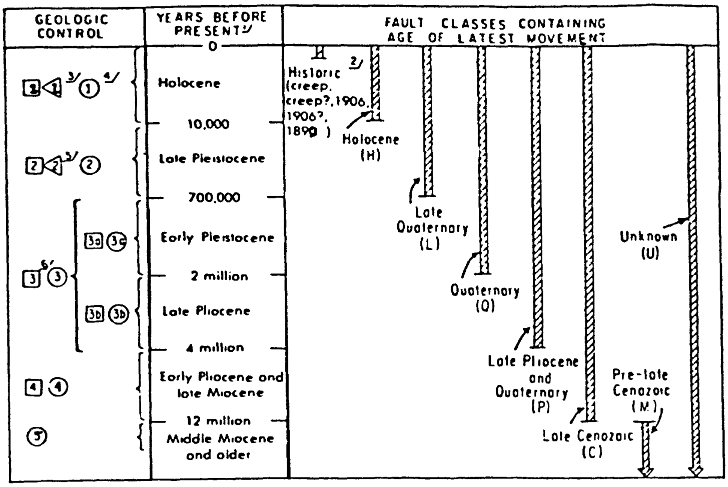
where $M_o^{\max}$ is the moment of the maximum earthquake expected on the fault; $\dot{M}_o$ § is the rate of occurrence of seismic moments ($\dot{M}_o$ § $=\overline{\mu A v}$, where $\overline{v}$ is the average slip rate on the fault, $\overline{\mu}$ and A were defined previously); M_o is the lower end of the range of moments of interest; and § is constant defined as b/c, where b can be defined from observed historical seismicity data, or assumed to be 1 (Anderson, 1979) and c is rationalized as 1.5 based on worldwide observations (Molnar, 1979).

Complete theoretical derivations of this method can be found in Molnar (1979) and Anderson (1979). A comparison of seismic rates derived from this method and rates derived from the historic record of seismicity for several faults of the San Andreas system (Anderson, 1979; Thenhaus and others, 1980) indicates generally good agreement between the two methods and provides a measure of confidence in the seismic rates used in the hazard study. In regions having an inadequate historic data base of earthquakes to derive statistical estimates of recurrence, fault slip-rates could be used to estimate earthquake frequency. However, because considerable uncertainty exists in all of the different methods of estimating earthquake recurrence times (Molnar estimates results of the slip-rate method could be in error by as much as a factor of 3 to 5), every effort should be made to collect and compare data and results from a variety of methods.

Detailed Field Mapping of Faults and Fault Breaks

An ambitious field mapping project has been performed in coastal California by the U.S. Geological Survey aimed at mapping faults and fault-strands in detail and assessing bounds on ages of latest displacements (for example, Ziony and others, 1974; Buchanan-Banks and others, 1978). Figures 2-A and 2-B provide insight into the detail of investigation. Such mapping projects are very useful in earthquake hazard studies. However, such

AGE RANGE CHART OF GEOLOGIC CONTROLS AND AGE CLASSES



GEOLOGIC CONTROL SYMBOLS

Symbols below indicate location of 1) late Cenozoic stratigraphic unit not displaced by fault movement, 2) unit displaced by fault movement, and 3) geomorphic feature formed by fault movement. Number within each symbol indicates age of each unit based on the generalized time spans of the Age Range Chart; the youngest reasonable age is assumed for deposits whose age is uncertain

○ Oldest known unfaulted stratigraphic unit deposited across or intruded along the fault. Latest fault movement predates age of unit

□ Youngest known stratigraphic unit displaced by fault. Latest fault movement postdates age of unit

◁ Geomorphic feature formed by fault movement. Latest fault movement inferred from type of feature (see footnotes 3/ and 5/ to Age Range Chart)

Figure 2A.—Figure Explanation for map shown on figure 2B. Note the geologic criteria for dating ages of latest fault movement.

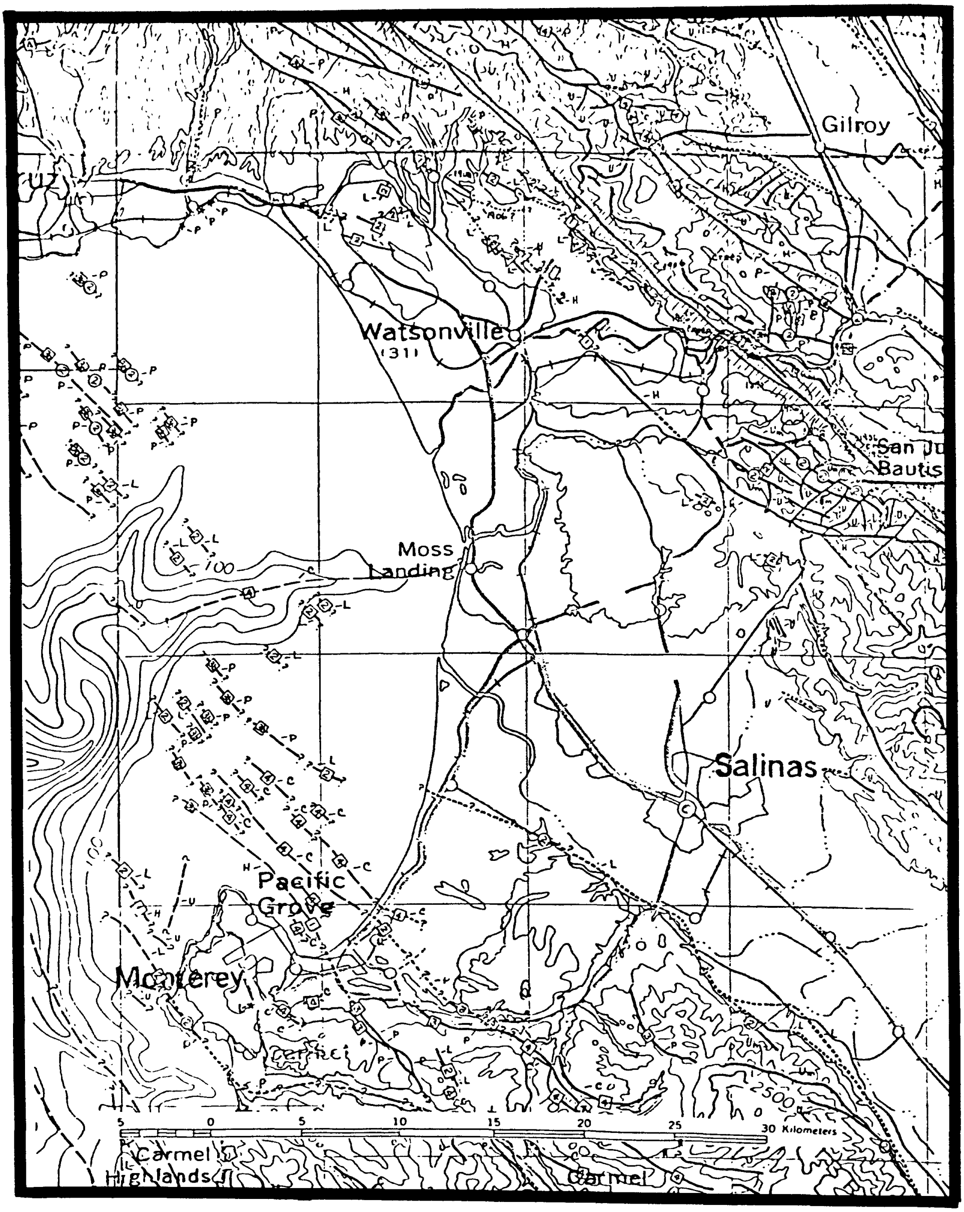


Figure 2B.—Fault map of Monterey Bay area, coastal California showing latest ages of fault movements according to the geologic criteria of figure 2A.

compilations covering an area of about 10,000 km^2 can take four or more years to complete with a staff of several geologists.

It is this type of mapping that provided data for detailed seismic source zones of a seismic ground motion hazard map of coastal California (Thenhaus and others, 1980). In this study faults of regional extent are considered seismogenic zones in themselves, if an association between the fault and seismicity can be inferred. This inference may be based on either evidence of historic or Holocene surface faulting on the fault and/or the close spatial association of seismcity with the fault. The boundaries of such seismogenic zones are drawn arbitrarily 10 km on either side of the fault in order to enclose the smaller historic earthquakes that may be associated with any particular fault. No attempt is made to differentiate the segments of individual faults into separate zones on the basis of different ages of latest displacements. Instead, the occurrence of Holocene or historic rupture along any segment of a fault is taken as an indication that the entire fault is active and that events are equally likely along its entire length.

An Innovative Approach to Studying Young Faults: Fault Scarp Profiling

Geomorphic studies in the semiarid Great Basin region of the Western United States (Fig. 1) showed that degradation of fresh, steep fault scarps having normal displacements results in a subdued fault profile in time. The crest and toe of the scarps become rounded and the initially steep fault scarp face flattens as the scarp is eroded (Wallace, 1977). Bucknam and Anderson (1979) quantified this geomorphic process through development of a field method of fault scarp profiling. Comparison of profiles of scarps of a known age (Pleistocene lake shoreline scarps) with profiles of tectonic fault scarps provides relative dates as to when the tectonic fault scarps formed. Hence, a rate of surface faulting activity per unit area can be assessed based on the geologic record of fault scarps extending back tens of thousands of years. Figure 3 compares regional rates of earthquake activity as determined from the historic record of earthquakes and rates determined from application of the scarp-profiling method. For broad regions the rates compare quite well; however, for very small areas or individual faults the comparison is much poorer and may result from periodic earthquake behavior of the individual

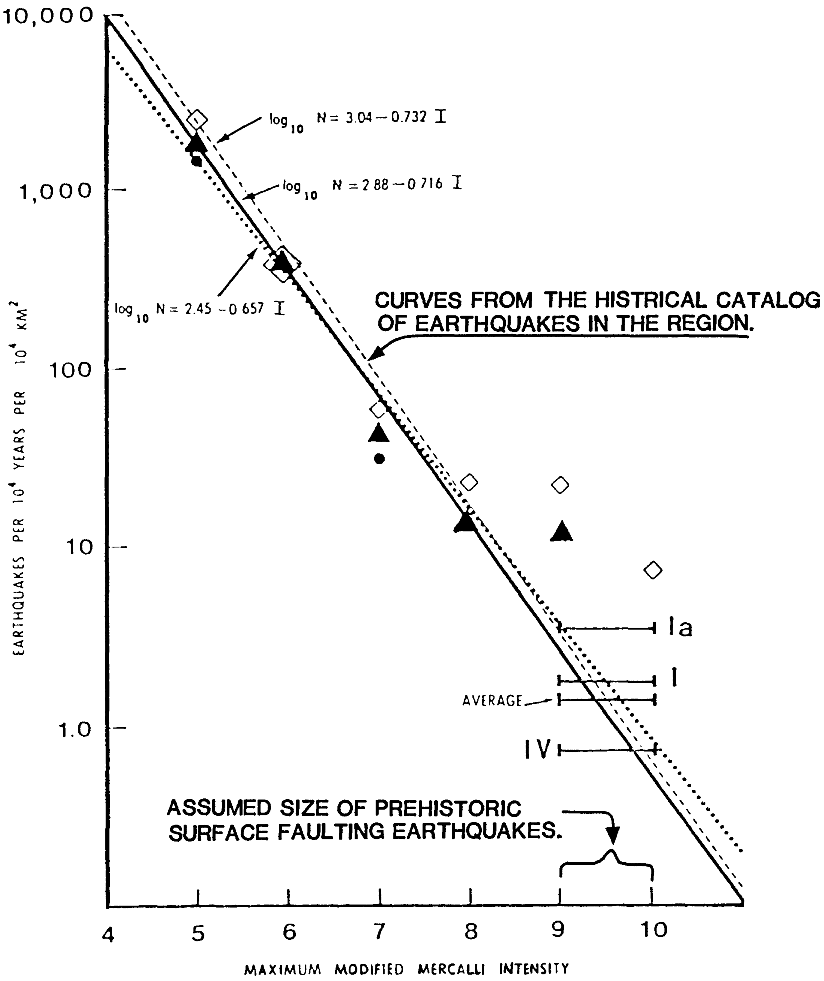


Figure 3.—Comparison of earthquake occurrence rates determined from fault scarp studies and historic seismicity in the eastern Great Basin.

fault or unreliable statistical results because of a very small data base of historic earthquakes.

Most outstanding is the ease of application of the scarp-profiling method and the rapid data collection technique that provides very useful information for seismic hazard studies. By scanning standard stereo air photo images (scale $\approx$ 1:60,000) in the office, a geologist can quickly identify linear features, or vegetation lines in Quaternary alluvium. Typically, reconnaissance field investigation is all that is required to identify the feature as a probable fault scarp. The profiling itself can be performed by a single person, although two is more convenient. Care needs to be taken in locating scarp profiles so that washed areas, animal burrows, or hummocky areas due to buried plant roots do not introduce error into the profile slope. It is also important that the profile be measured normal to the scarp.

Although the application of this method is rapid, several years of study were required to understand landform degradation and erosional processes in this semiarid environment. This evolution of study could be generalized to any geologic environment in that, several years (or more) of basic research on fundamental geologic processes may be required in a region before innovative, efficient methods of study focussed on specific questions are developed.

Maximum Magnitude Earthquakes and Fault Lengths

As in the above fault-slip formula, and in virtually every application of geologic data to seismic hazard studies, some estimate of the maximum potential size of an earthquake on a given fault is needed. Much analysis has been performed on the correlation of length-of-fault-rupture to earthquake size (for example Slemmons, 1977; Mark and Bonilla, 1977). Regressions of magnitude on log length ($M = a + b \log L$) can be used to estimate most probable magnitude given rupture length and regressions of log length on magnitude ($\log L = a + bM$) can be used to estimate most probable rupture length given magnitude. Care needs to be taken to use the appropriate relationship for the variable being investigated. Most often some percentage of the total fault length (ranging from 50% to 100%) is assumed to rupture in an earthquake and the resulting magnitude is assessed. Usually some measure of conservatism is added by assuming a rather long fault rupture length. However, a little recognized fact is that the regression provides a mean estimate of magnitude given a fault rupture length and dispersion of the data about this mean at certain rupture lengths could be on the order of two or more magnitude units. Accounting for this uncertainty in the mean magnitude estimate at given rupture lengths could easily boost maximum magnitudes higher than assigning what is thought to be a consertively long rupture length.

Seismic Zones Having No Surface Faulting: Intraplate Earthquakes

The earthquake that affected the largest geographical area of the United States historically did not occur in California, but rather in the mid-continent region. A series of earthquakes persisted through the years of 1811 and 1812 in the northern Mississippi Embayment near the town of New Madrid, Missouri (Fig. 1). The largest earthquake of this series had an epicentral intensity of X-XI and was felt from the Rocky Mountain front to the east coast. The tectonic setting of this earthquake long-remained an enigma. Surface faulting is not obvious in the earthquake area, although sandblows are widely present. The landsurface in the embayment is characterized by monotonous low relief; an extremely benign appearance in comparison to the dramatic fault-bounded mountain blocks of the Basin and Range Province in the Western United States. It was obvious that the relatively simple, straightforward field methods applied to studies of active faults in the Western United States and that provided so much useful information on the earthquake hazard would provide little information in this seismically active area. In 1975 the U.S. Geological Survey initiated an integrated multidisciplinary effort to resolve this long-standing seismological problem. Seismologic monitoring, gravity and magnetic studies, seismic-reflection profiling, geomorphic studies, stratigraphic and structural studies, and heat-flow investigations, were focused on the New Madrid earthquake region to identify the geologic source of the 1811-1812 New Madrid earthquakes (McKeown and Pakiser, 1982). The results indicate a northeast-trending zone of down-faulted crustal rocks more than 200 km in length and about 75 km wide (Fig. 4). Inferred mafic plutons occur along the margins of this structure, and in some cases are near the central axis. The overall geologic setting indicates that this structure is an ancient buried rift that has been seismically

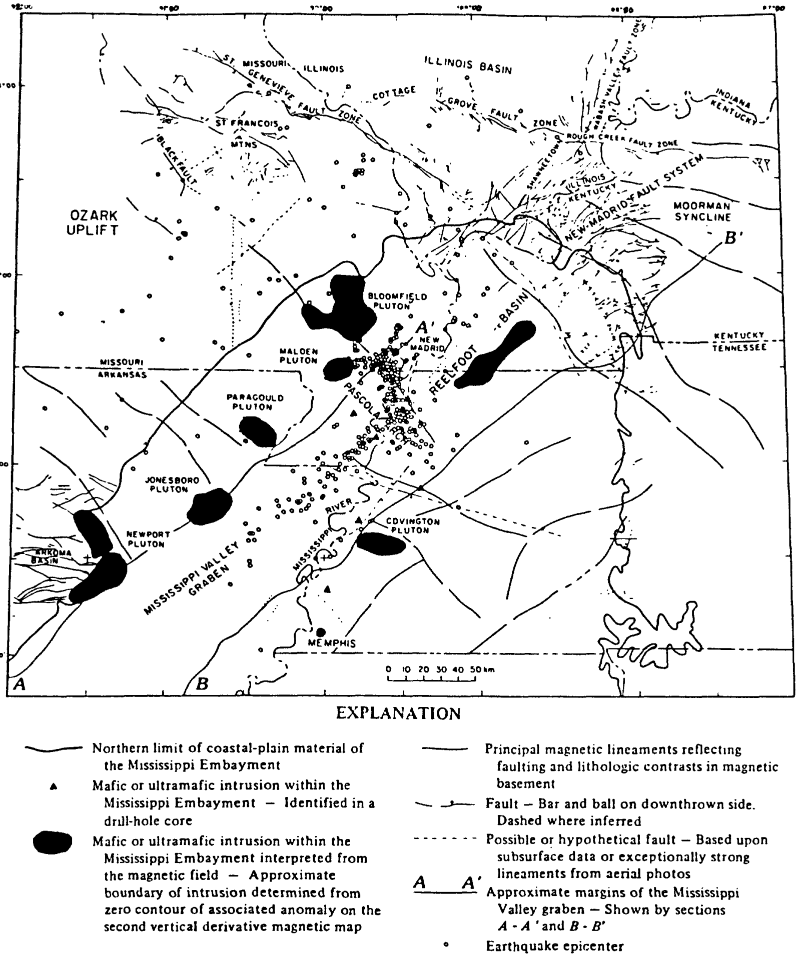


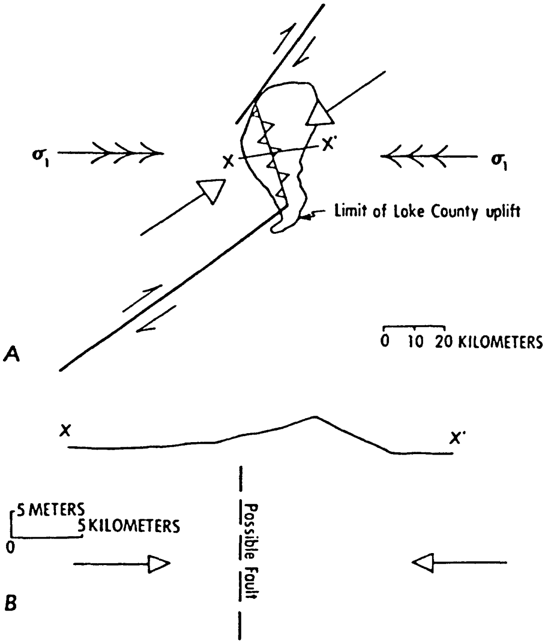
FIGURE 4.--Seismotectonic map of the New Madrid earthquake region.

reactivated in the present-day stress regime. Where microseismic activity follows a northeast trend, central to the rift, predominantly strike-slip motion occurs; where the seismicity trend jogs to the northwest, predominant thrusting motion occurs. At the surface, above the zone of thrusting, warping of the Mississippi River Valley floor has been identified through geomorphic studies. Russ (1982) has unified this information in a proposed tectonic model (Fig. 5). East-west directed compressive stress results in strike-slip motion on northeast trending planes of weakness. Right-lateral motion is required for a zone of convergence and thrusting on a northwest-trending plane. The zone of convergence also explains the geometry of local uplift at the surface.

Seismic Source Zones in The Interior United States

The New Madrid investigations provide an excellent example of the scope and the research commitment that may be necessary to obtain lasting scientific results pertinent to earthquake hazard questions in regions where the geologic origins of seismicity are not known. However, the New Madrid area is only one of many interior regions that have experienced historical earthquakes. The New Madrid area gained distinction because, within historic times, it has had the largest earthquake. Rates of seismic strain release in interior plate regions are in general low, hence recurrence intervals of large earthquakes are long, on the order of at least many hundreds to thousands of years. There may very well be (and most probably are) other zones in the central United States capable of sustaining high magnitude earthquakes. It is just fortuitous that the large earthquakes have not occurred within the geologically short time period of recorded history. Clearly, a research program of the scope carried out in New Madrid region is not feasible for every area that has experienced earthquakes historically.

Figure 6 shows the historic earthquake catalog of the central United States along with seismic source zones used in a study that estimates the earthquake ground-motion hazard throughout the United States (Algermissen and others, 1982). Surface faulting is unknown throughout this region and active faults having a probable association with historically damaging earthquakes are unknown outside of the New Madrid zone. The source zones are based



Plan view (*A*) and cross section (*B*) of hypothetical tectonic model for the origin of the Lake County uplift. Model interrelates the modern stress field with fault movement and the location of uplift. Barbed arrows indicate σ_1, the direction of maximum compressive stress; single arrows indicate right-lateral displacement on faults of major northeast-trending seismic zones; large open-headed arrows indicate compression generated by movement along faults; and saw teeth indicate area of highest compression.

FIGURE 5.--Seismotectonic model for the New Madrid earthquake region.

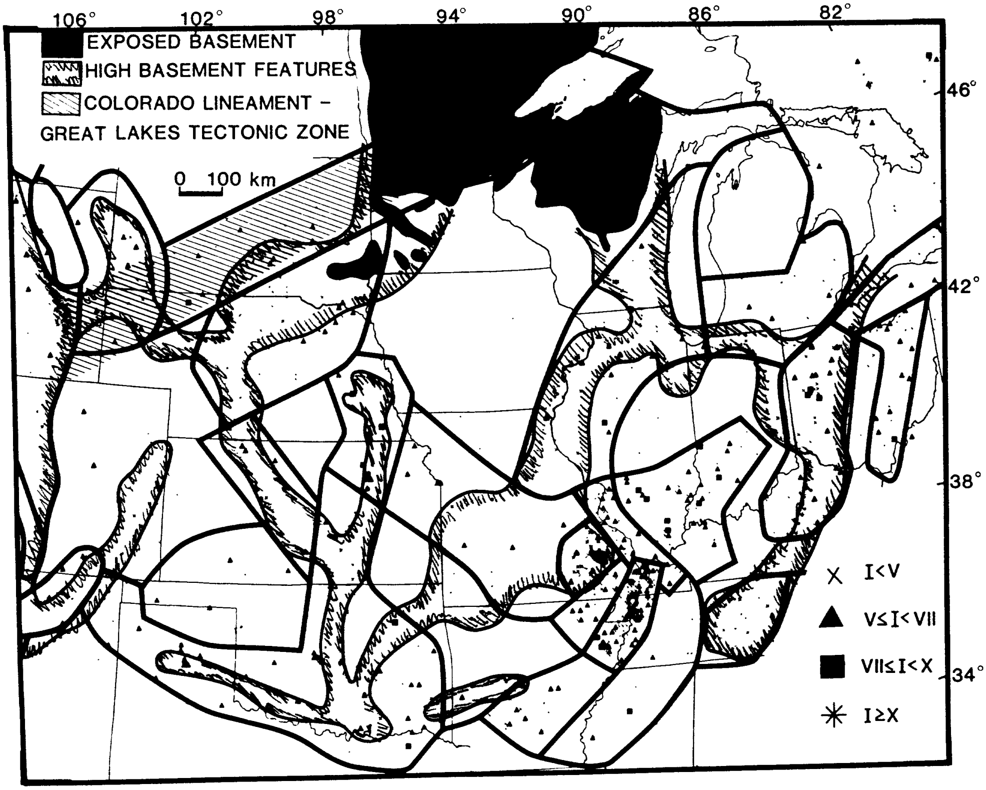


Figure 6.—Central United States seismicity, source zones, and general basement structure.

primarily on the geographic distribution of historic seismicity, and the rates of earthquake activity are also determined from the historic record of events. For the most part, the distribution of earthquake activity appears to be strongly associated with high basement features such as domes and broad arches. These regional basement features, where they could be at least associated with a number of lower intensity historical earthquakes, provided a useful guide in drawing the zone boundaries. The total seismic rate of earthquake occurrence within the zones of high basement features is a factor of 10 greater than the total seismic rate of the intervening areas, excluding the New Madrid earthquake area and adjacent source zones. An important aspect of an earthquake hazard map is to identify areas of possible future occurrences of large earthquakes. Strong regional correlation of historic seismicity with geologic structure can provide a useful approach to this problem when causes of seismicity are unknown.

SUMMARY

Much useful information on earthquake recurrence intervals and estimates of maximum earthquake magnitudes on faults can be obtained in regions of active faulting through the application of relatively simple and inexpensive methods of geologic investigation. The results are usually not unequivocal but are useful for regional comparison to statistically derived rates of earthquake recurrence. In site-specific hazard studies where the seismic potential of a small number of faults is in question, these methods can provide an earthquake history of large earthquake occurrence on individual faults extending back tens of thousands of years.

In regions where active faulting is unknown, investigations requiring a much broader scope and involving much more effort in terms of man-power and capital investment may be required to merely identify the seismically active geologic structure. Where such an effort is not feasible nor practical, useful estimates of the seismic hazard can be made on a regional scale noting correspondence between the historical catalog of events and regional geologic structure.

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HISTORICAL EVIDENCE OF EARTHQUAKE ACTIVITY

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INTRODUCTION

Historical evidence of earthquake activity comes from many sources. This paper will focus on earthquake catalogs and the information that goes into them, especially the historical information.

We will first look at some examples of earthquake catalogs and what they contain. Second, we will discuss the minimum information that an earthquake catalog should contain to be useful for seismic hazard planning. Finally, we will discuss a large number of other things from the historical record that might be included in a good earthquake catalog. Emphasis in this paper will be on historical data, particularly intensity data, rather than on modern instrumental information.

EXAMPLES OF EARTHQUAKE CATALOGS

There are several different kinds of earthquake catalogs. Table 1 gives examples of each of these catalog types. The first three catalogs in table 1 list earthquakes located all over the earth, the remainder list shocks for a specific country, continent, or region.

Some catalogs, such as the Gutenberg and Richter catalog, give only the time, location, magnitude, and reference for each earthquake listed. Others contain much more information on each earthquake. For example, the ISC catalog gives the readings recorded by each seismograph station reporting the earthquake. The last two catalogs on the list, "Earthquake History of the United States" and "United States Earthquakes," are examples of catalogs giving descriptive and intensity information.

TABLE 1.--Examples of earthquake catalogs

[Complete information in reference list]

A. Gutenberg and Richter, 1954 -- world

B. ISC (International Seismological Centre) -- world

C. PDE (Preliminary Determination of Epicenters) -- world
(National Earthquake Information Service)

D. Shebalin, 1974 -- Balkans

E. Earthquake History of the United States
(Coffman and others, 1982) -- United States

F. United States Earthquakes, 1928 to present --
United States

Most earthquake catalogs today are lists on computers, rather than in books. Many things can be done easily with a computer catalog that would be extremely time consuming with a catalog in a book. Computer catalogs can be updated or corrected quickly and easily. Subsets of the data can be extracted for a specific purpose. The data can be sorted in many different ways. Plots can be made of the data by the computer. Keeping a catalog on a computer makes it far more versatile and useful. However, the book form is also useful for ready reference to such things as maps, graphs, and photographs, even though the maps and graphs can be kept in digital form on the computer.

ESSENTIAL CONTENTS OF EARTHQUAKE CATALOGS

Many different kinds of information may be included in earthquake catalogs. The information kept depends on the purpose for which the catalog is used. Table 2 shows the most essential contents of an earthquake catalog

TABLE 2.--Essential contents of an earthquake catalog that is to be used for seismic hazard evaluation

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A. LOCATION

1. Epicenter (+ quality + reference)
2. Depth (+ quality + reference)
3. Region and/or name (+ reference)

B. TIME

Year-month-day-hour-minute-second (+ reference)

C. SIZE

1. Magnitudes: M_s (+ reference)
   m_b (+ reference)
   other (+ reference + scale)
2. Maximum intensity: I_o (+ reference + scale)

---

that would be of use for seismic hazard evaluation. At minimum, such a catalog should contain the location, time, and size of the earthquakes. Since the information for a single shock has most likely been assembled from several different sources, it is also necessary to give a reference for each piece of information in the catalog. Moreover, a quality factor, or estimate of accuracy, should also be given for each measured, calculated, or estimated number in the catalog. Finally, all units (kilometers, centimeters, degrees, minutes, seconds, hours, etc.) and all scales (M.M., M_s, m_b, etc.) should be marked and a complete explanation of each scale appended to the catalog.

Location

Location includes the latitudes and longitudes of the epicenters, depth of focus, and a name and (or) region number. Name or region number might include such things as country, state, county, physiographic province, seismic source zone, or location on an arbitrary grid system. The more such keys are incorporated into a computer catalog, the more versatile that data base is. Before a new catalog is compiled, careful thought must be given to all the uses to which the catalog is likely to be put, what information may be needed, and the format in which it is to be organized.

With locations alone, a seismicity map may be made for the region covered by the catalog. A seismicity map of this kind is useful for defining zones of major seismic activity, but does not show the sizes of the earthquakes, how often they occur, or how widespread are their effects.

If a catalog contains preinstrumental or historic earthquakes as well as instrumental earthquakes, such differences should be noted. Epicenters of preinstrumental earthquakes are usually estimated from the location of the maximum reported intensity. Modern earthquakes that are both instrumentally recorded and well reported prove that the maximum intensity location may be 50 km or more away from the instrumental epicenter. In addition, for old earthquakes in sparcely populated areas, the reported location of the maximum intensity itself may be far from the epicenter. The epicenter is usually assumed to be at the location of the observer who reported the maximum intensity. If there were no observers within the meizoseismal area during or shortly after the earthquake (or if they left no written accounts), then the reported intensity center may be far removed from the actual epicenter.

Since the amount, distribution, and quality of data available for historical earthquakes varies so much from one earthquake to another, the parameters estimated from that data also vary widely in quality. Estimated epicenters, felt areas, magnitudes, etc. may be quite good or very poor. Estimates of the quality of the various estimated parameters and descriptions of the methods of estimating the parameters are useful. At least, instrumental values should always be distinguished from values estimated from intensity or other information.

Time

The time of the earthquake is the year, month, day, hour, minute, second, and tenths of a second. Precise time is important in order to distinguish among the many aftershocks that frequently follow large earthquakes. Greenwich Mean Time or the local standard time(s) should be specified. The date and time of an old shock is often a problem. Lack of precision in the way dates were reported in old newspapers and letters can easily lead to the same earthquake being listed in a catalog at two different times, most likely exactly one hour, one day, or one week apart. Such catalog entries should be regarded with suspicion. Also suspect are the times listed for earthquakes before standard time zones were established. Usually, each town had its own time, based on local noon. A town's time would be several minutes different from that of the next town down the road. This confuses the time of the main shock, and makes separating a series of aftershocks virtually impossible.

Size: Magnitude

Size of an earthquake is denoted by two methods. The first is magnitude, an instrumental measure of the amount of energy generated at the source. A number of different magnitudes are used. A useful catalog should contain at least M_s (surface-wave magnitude) and m_b (body-wave magnitude) for each earthquake. Other magnitudes, such as a local magnitude M_L or a moment magnitude M_o, may also be included. Text accompanying the catalog should completely define how each type of magnitude reported in the catalog is calculated.

A common use of an earthquake catalog is to extract a subset of earthquakes from it, for example, all the shocks with M_s greater than 7.5. To do this, it is necessary to have the same type of magnitude (in this case M_s) for each earthquake in the catalog. Since the same type of magnitude is not always reported for every earthquake, a way must be found to calculate the desired type of magnitude for the shocks that have some other magnitude(s), but not the desired one. Because the relationship between two types of magnitudes is usually approximately linear over the magnitude range of interest, a simple linear regression may suffice to estimate the missing magnitudes. For example, suppose M_s is the type of magnitude desired for every entry, but M_s is not given for every entry. However, there are some entries giving both M_s and some other measure of size (m_b, M_L, I_o (maximum intensity)). Then, relationships can be derived such that:

$$M_s=a(1)+b(1)m_b$$
$$M_s=a(2)+b(2)M_L$$
$$M_s=a(3)+b(3)I_o$$

These relationships can then be used to calculate estimates of the missing M_s values. Of course, each such estimated magnitude should be clearly marked in the catalog as having been estimated rather than derived from instrumental amplitudes, and the method of estimation should be explained fully.

Magnitudes of preinstrumental earthquakes may also be included in an earthquake catalog. These may be estimated from intensity or geologic information, and there is likely to be a very wide range in the quality of such estimates. Where there is little information about an old earthquake, a magnitude may be estimated based on the maximum reported intensity I_o, or, better, on the felt area of the earthquake. If enough information exists about the old shock to make at least a rough isoseismal map, a better estimate of the shock's magnitude may be obtained from the attenuation of intensity with epicentral distance. (Because rate of intensity attenuation varies from region to region, a knowledge of the attenuation in the region of the earthquake is necessary for use of the latter two methods; this can be obtained from modern earthquakes.) If there is geological evidence of surface faulting, the magnitude may also be estimated from the length of the rupture zone.

Size: Maximum Intensity

A less precise method of denoting earthquake size is maximum intensity. Maximum intensity I_o is needed in addition to magnitude for continuity between preinstrumental and modern earthquakes. Maximum intensity is primarily of interest for moderate and larger earthquakes ($I_o \geq$ VI M.M.), since the smaller shocks are probably not known very many years back in time. A catalog of microearthquakes recorded by a local seismic network, for example, might have little need of I_o's because few of the earthquakes may be large enough to be felt.

Whenever any kind of intensity information is included in a catalog, it is important that the catalog specify the intensity scale that is used. The scale should, moreover, be reproduced in full in the text accompanying the catalog, along with any necessary comments on the local variations in the use of the scale. Whenever possible, it is best to have all the intensities in a catalog assigned by a single researcher, thus eliminating personal variations in interpretation of the intensity scale.

There are several intensity scales in use in the world today. In addition to the Modified Mercalli Scale in use in the United States, there is the Japanese scale and the Geofian Scale (Russia). The Rossi-Forel (R.F.) Scale was commonly used in the United States until 1931, and there are many pre-1931 isoseismal maps and intensity studies evaluated in that scale. Table 3 compares these four scales and affords a "quick and dirty" method of converting intensity values from one scale to another without actually going back to the original accounts and reassigning the intensities in the new scale.

Estimate of Completeness

Besides location, time, and size of the earthquakes, there is one more essential of a good catalog if the catalog is to be useful for seismic hazard evaluation. That is an estimate of the completeness of the records for each size of shock in the catalog. For example, in the New England region of the United States, we think we know of every earthquake with $I_0 \geq$ VIII M.M. since the mid-1600's. There were enough people, newspapers, letters, diaries, etc. in that place at that time so that an earthquake of intensity VIII would almost surely have been reported and be in our catalog. Reports of maximum intensity VI earthquakes, however, are probably not complete for the New England region until the mid-1800's. In the Western Mountain and Pacific Northwest regions of the United States there are no earthquake records at all prior to the mid-1800's. There are unpopulated places in the world today where a maximum-intensity-VIII event would probably not be reported at all if we still had to depend only on felt reports rather than seismograph records.

OTHER USEFUL HISTORICAL INFORMATION FOR AN EARTHQUAKE CATALOG

In addition to location, time, and size of the earthquakes, and an estimate of the completeness of the catalog, there are a number of other useful things in the historical records that might be included in an earthquake catalog (see table 4). Which of the following are chosen for inclusion in a particular catalog depends on local circumstances and on the intended purposes of the catalog.

Isoseismal Maps

Isoseismal maps are plots of all the assigned intensities for a particular earthquake contoured for each level of intensity. The simplest isoseismal map is simply an outline of the felt area with a notation at the epicenter. More useful isoseismal maps show all the intensities and all the contours.

Information on which to base intensity assignments for old earthquakes is often scarce. Since there are few data points on the intensity map of the old earthquake, the contours tend to be smooth and rather circular. With better information, modern earthquakes have much more interesting isoseismal maps, showing protrusions of higher intensities (for example, along river valleys) and reentrants of lower intensities (for example, where there is bedrock).

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Estimate of Completeness

Besides location, time, and size of the earthquakes, there is one more essential of a good catalog if the catalog is to be useful for seismic hazard

TABLE 3.--Comparison of seismic intensity scales

ROSSI-FOREL	MODIFIED MERCALLI	GEOFIAN	JAPANESE
I	I	I	O
		II	I
	II		
II		III	
III	III		
IV	IV	IV	II
V			
	V	V	III
VI			
	VI		IV
		VI	
VII			
VIII	VII	VII	V
	VIII	VIII	
IX			
	IX	IX	VI
X	X	X	
	XI	XI	VII
	XII	XII	

(Taken from Barosh, 1969, and Wood and Neumann, 1931) evaluation. That is an estimate of the completeness of the records for each size of shock in the catalog. For example, in the New England region of the United States, we think we know of every earthquake with $I_o \geq$ VIII M.M. since the mid-1600's. There were enough people, newspapers, letters, diaries, etc. in that place at that time so that an earthquake of intensity VIII would almost surely have been reported and be in our catalog. Reports of maximum intensity VI earthquakes, however, are probably not complete for the New England region until the mid-1800's. In the Western Mountain and Pacific Northwest regions of the United States there are no earthquake records at all prior to the mid-1800's. There are unpopulated places in the world today where a maximum-intensity-VIII event would probably not be reported at all if we still had to depend only on felt reports rather than seismograph records.

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TABLE 4.--Other useful historical information for an earthquake catalog

---

A. Isoseismal maps

B. Intensities at specific sites

C. Areas at each intensity level

D. Felt area

E. Unusual effects and special circumstances

F. Surface faulting

G. Surface faulting and rupture length

H. Duration

I. Casualties and injuries

J. Estimate of damage cost

K. Tsunami

L. Ground effects

M. Long-period effects

N. Aftershocks

O. Rockbursts and explosions

P. Photographs

Q. Duplicate events

R. Format

The collection of isoseismal maps for a region can be used to generate a map showing the maximum intensities ever reported for every point in the region. Such a maximum intensity map is a very basic kind of seismic risk map. It gives the highest intensity known to have occurred at any given point, but gives no information about how often that intensity might have occurred, or what is the likelihood that that intensity might be exceeded in, say, 500 years.

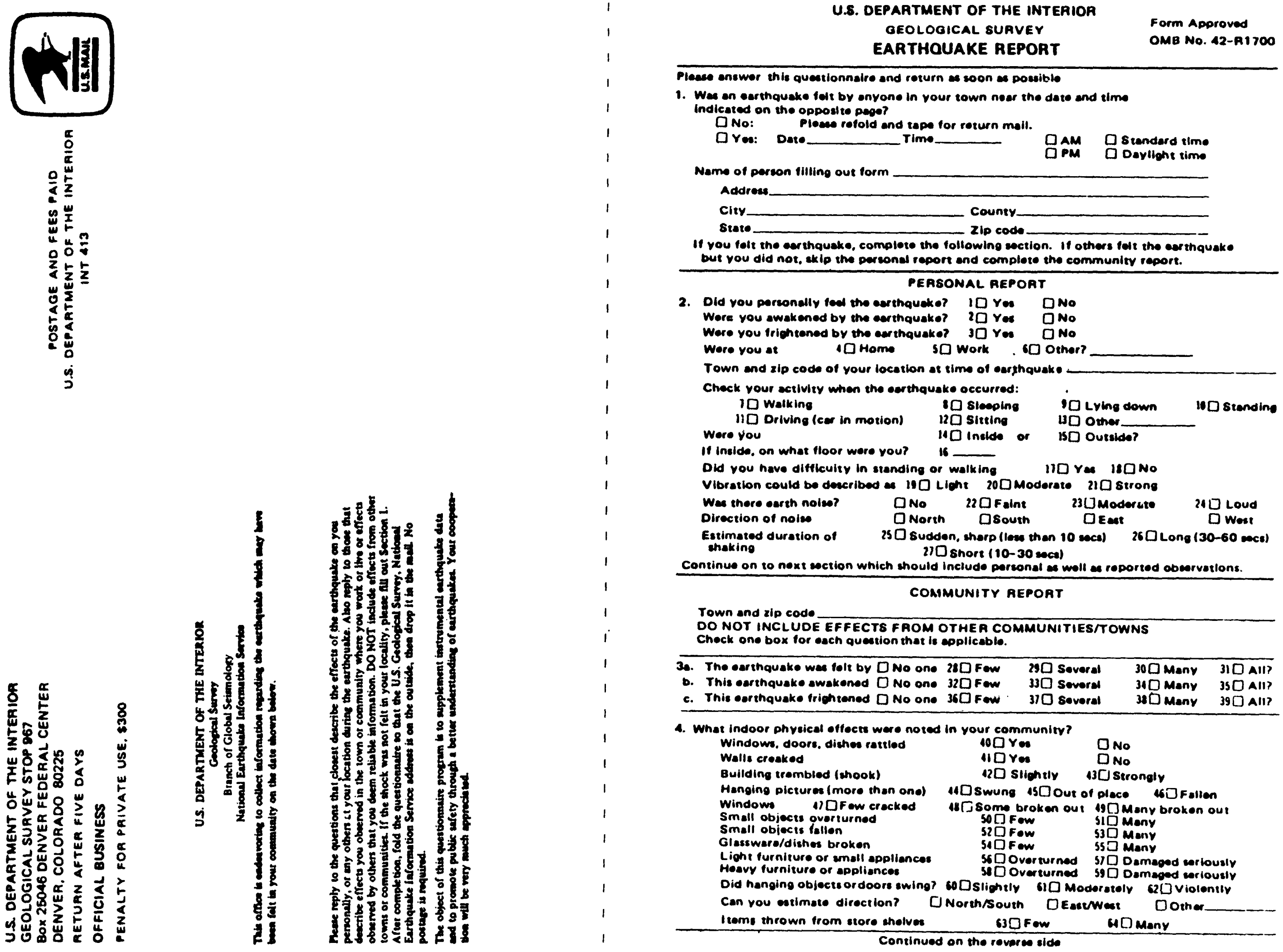
Intensities at Specific Sites

This is the data used to make an isoseismal map. It may be as simple as a list of locations, each with its assigned intensity, or as thorough as a complete descriptive report of the effects at each site. An example of the latter type of catalog is the publication, "United States Earthquakes." This is a series of booklets, one published every year since 1928, each containing information about all the earthquakes felt in the United States in a single year. Descriptions of effects at each site are given for all the places reporting moderate to high intensities. For the lowest intensities, all the towns assigned each intensity are listed. Contoured isoseismal maps showing the distribution of intensities are included for the larger and more important shocks. More recent books in the U.S. Earthquakes series also have photographs of both typical and unusual damage. This series of books is also kept as a computer catalog. The computer version lists time, location, magnitude, town name, town latitude and longitude, and town intensity.

The intensity data presented in "United States Earthquakes" is gathered using questionnaire cards like the one shown in figure 1. The cards are computer-addressed and are sent to post offices. About 20,000 questionnaires like this are mailed each year. For a small earthquake, 50-100 cards might be used, depending on the location and importance of the earthquake. For a larger shock, 2000-3000 cards might be sent (Carl Stover, oral commun., 1984).

Areas at Each Intensity Level

This information is obtained by measuring the areas on an isoseismal map. The areas can be used to plot an intensity-attenuation curve (a graph of the fall-off of intensity with epicentral distance) for the earthquake. The curve is



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U.S. DEPARTMENT OF THE INTERIOR
Geological Survey
Branch of Global Seismology
National Earthquake Information Service

This office is endeavoring to collect information regarding the earthquake which may have been felt in your community on the date shown below.

Please reply to the questions that closest describe the effects of the earthquake on you personally, or any others at your location during the earthquake. Also reply to those that describe effects you observed in the town or community where you work or live or effects observed by others that you deem reliable information. DO NOT include effects from other towns or communities. If the shock was not felt in your locality, please fill out Section 1. After completion, fold the questionnaire so that the U.S. Geological Survey, National Earthquake Information Service address is on the outside, then drop it in the mail. No postage is required.

The object of this questionnaire program is to supplement instrumental earthquake data and to promote public safety through a better understanding of earthquakes. Your cooperation will be very much appreciated.

U.S. DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
EARTHQUAKE REPORT

Form Approved
OMB No. 42-R1700

Please answer this questionnaire and return as soon as possible

1. Was an earthquake felt by anyone in your town near the date and time indicated on the opposite page?
☐ No: Please refold and tape for return mail.
☐ Yes: Date________ Time________ ☐ AM ☐ PM ☐ Standard time ☐ Daylight time

Name of person filling out form ________
Address________
City________ County________
State________ Zip code________

If you felt the earthquake, complete the following section. If others felt the earthquake but you did not, skip the personal report and complete the community report.

PERSONAL REPORT

2. Did you personally feel the earthquake? 1☐ Yes ☐ No
Were you awakened by the earthquake? 2☐ Yes ☐ No
Were you frightened by the earthquake? 3☐ Yes ☐ No
Were you at 4☐ Home 5☐ Work . 6☐ Other? ________
Town and zip code of your location at time of earthquake ________
Check your activity when the earthquake occurred:
7☐ Walking 8☐ Sleeping 9☐ Lying down 10☐ Standing
11☐ Driving (car in motion) 12☐ Sitting 13☐ Other________
Were you 14☐ Inside or 15☐ Outside?
If inside, on what floor were you? 16 ______
Did you have difficulty in standing or walking 17☐ Yes 18☐ No
Vibration could be described as 19☐ Light 20☐ Moderate 21☐ Strong
Was there earth noise? ☐ No 22☐ Faint 23☐ Moderate 24☐ Loud
Direction of noise ☐ North ☐ South ☐ East ☐ West
Estimated duration of shaking 25☐ Sudden, sharp (less than 10 secs) 26☐ Long (30-60 secs) 27☐ Short (10-30 secs)

Continue on to next section which should include personal as well as reported observations.

COMMUNITY REPORT

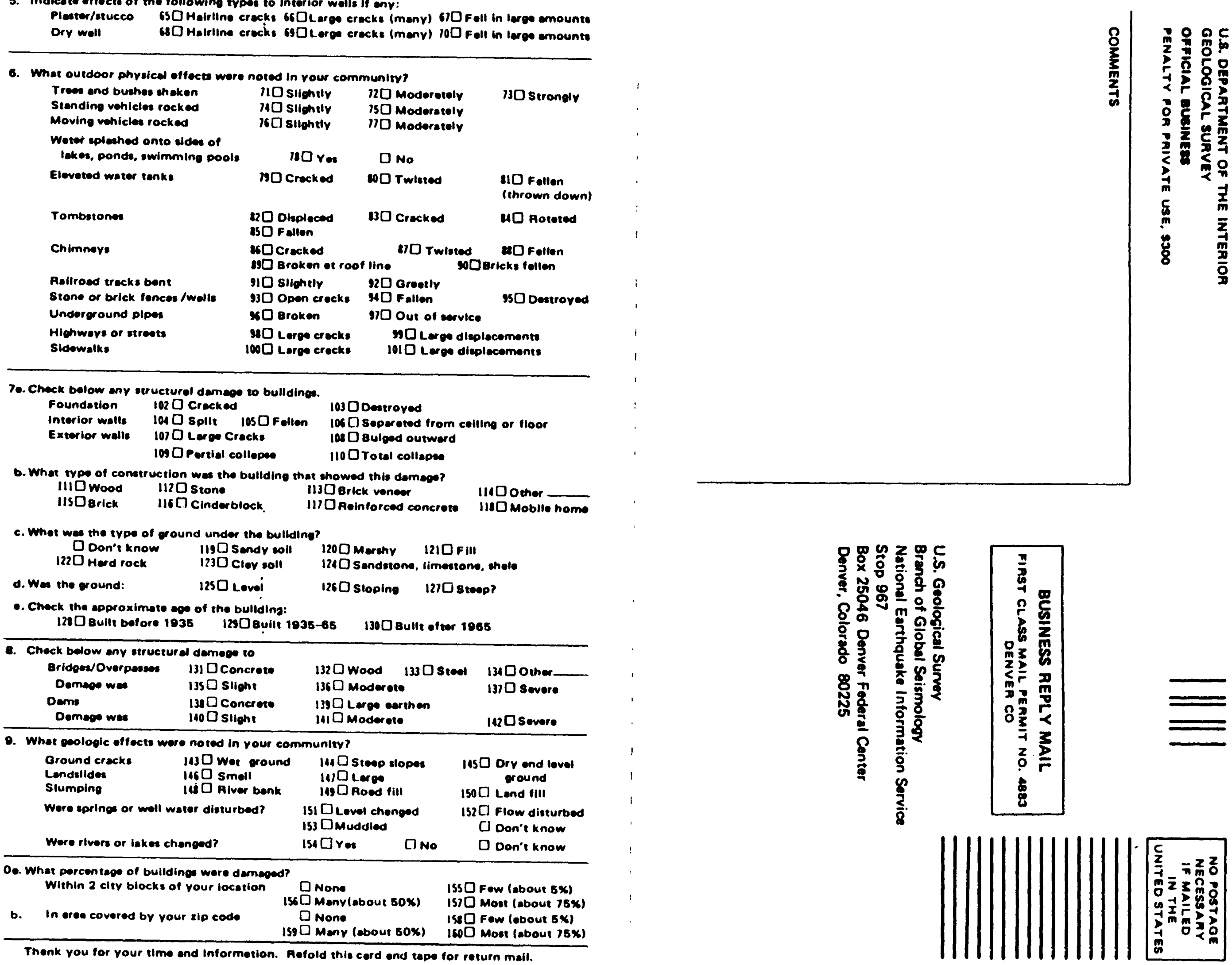
Town and zip code ________
DO NOT INCLUDE EFFECTS FROM OTHER COMMUNITIES/TOWNS
Check one box for each question that is applicable.

3a. The earthquake was felt by ☐ No one 28☐ Few 29☐ Several 30☐ Many 31☐ All?
b. This earthquake awakened ☐ No one 32☐ Few 33☐ Several 34☐ Many 35☐ All?
c. This earthquake frightened ☐ No one 36☐ Few 37☐ Several 38☐ Many 39☐ All?

4. What indoor physical effects were noted in your community?
Windows, doors, dishes rattled 40☐ Yes ☐ No
Walls creaked 41☐ Yes ☐ No
Building trembled (shook) 42☐ Slightly 43☐ Strongly
Hanging pictures (more than one) 44☐ Swung 45☐ Out of place 46☐ Fallen
Windows 47☐ Few cracked 48☐ Some broken out 49☐ Many broken out
Small objects overturned 50☐ Few 51☐ Many
Small objects fallen 52☐ Few 53☐ Many
Glassware/dishes broken 54☐ Few 55☐ Many
Light furniture or small appliances 56☐ Overturned 57☐ Damaged seriously
Heavy furniture or appliances 58☐ Overturned 59☐ Damaged seriously
Did hanging objects or doors swing? 60☐ Slightly 61☐ Moderately 62☐ Violently
Can you estimate direction? ☐ North/South ☐ East/West ☐ Other________
Items thrown from store shelves 63☐ Few 64☐ Many

Continued on the reverse side

Figure 1a.--Questionnaire used to gather intensity data for "United States Earthquakes."



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DENVER CO

U.S. Geological Survey
Branch of Global Seismology
National Earthquake Information Service
Stop 967
Box 25046 Denver Federal Center
Denver, Colorado 80225

COMMENTS

5. Indicate effects of the following types to interior walls if any:

Plaster/stucco	65☐ Hairline cracks	66☐ Large cracks (many)	67☐ Fell in large amounts
Dry wall	68☐ Hairline cracks	69☐ Large cracks (many)	70☐ Fell in large amounts

6. What outdoor physical effects were noted in your community?

Trees and bushes shaken	71☐ Slightly	72☐ Moderately	73☐ Strongly
Standing vehicles rocked	74☐ Slightly	75☐ Moderately	
Moving vehicles rocked	76☐ Slightly	77☐ Moderately	
Water splashed onto sides of lakes, ponds, swimming pools	78☐ Yes	☐ No	
Elevated water tanks	79☐ Cracked	80☐ Twisted	81☐ Fallen (thrown down)
Tombstones	82☐ Displaced 85☐ Fallen	83☐ Cracked	84☐ Rotated
Chimneys	86☐ Cracked 89☐ Broken at roof line	87☐ Twisted 90☐ Bricks fallen	88☐ Fallen
Railroad tracks bent	91☐ Slightly	92☐ Greatly	
Stone or brick fences/walls	93☐ Open cracks	94☐ Fallen	95☐ Destroyed
Underground pipes	96☐ Broken	97☐ Out of service	
Highways or streets	98☐ Large cracks	99☐ Large displacements	
Sidewalks	100☐ Large cracks	101☐ Large displacements	

7a. Check below any structural damage to buildings.

Foundation	102☐ Cracked		103☐ Destroyed
Interior walls	104☐ Split	105☐ Fallen	106☐ Separated from ceiling or floor
Exterior walls	107☐ Large Cracks		108☐ Bulged outward
	109☐ Partial collapse		110☐ Total collapse

b. What type of construction was the building that showed this damage?

111☐ Wood	112☐ Stone	113☐ Brick veneer	114☐ Other ______
115☐ Brick	116☐ Cinderblock	117☐ Reinforced concrete	118☐ Mobile home

c. What was the type of ground under the building?

☐ Don't know	119☐ Sandy soil	120☐ Marshy	121☐ Fill
122☐ Hard rock	123☐ Clay soil	124☐ Sandstone, limestone, shale	

d. Was the ground: 125☐ Level 126☐ Sloping 127☐ Steep?

e. Check the approximate age of the building:
128☐ Built before 1935 129☐ Built 1935-65 130☐ Built after 1965

8. Check below any structural damage to

Bridges/Overpasses	131☐ Concrete	132☐ Wood	133☐ Steel	134☐ Other ______
Damage was	135☐ Slight	136☐ Moderate		137☐ Severe
Dams	138☐ Concrete	139☐ Large earthen		
Damage was	140☐ Slight	141☐ Moderate		142☐ Severe

9. What geologic effects were noted in your community?

Ground cracks	143☐ Wet ground	144☐ Steep slopes	145☐ Dry and level ground
Landslides	146☐ Small	147☐ Large	
Slumping	148☐ River bank	149☐ Road fill	150☐ Land fill
Were springs or well water disturbed?		151☐ Level changed 153☐ Muddied	152☐ Flow disturbed ☐ Don't know
Were rivers or lakes changed?		154☐ Yes ☐ No	☐ Don't know

0a. What percentage of buildings were damaged?

Within 2 city blocks of your location	☐ None 156☐ Many (about 50%)	155☐ Few (about 5%) 157☐ Most (about 75%)
b. In area covered by your zip code	☐ None 159☐ Many (about 50%)	158☐ Few (about 5%) 160☐ Most (about 75%)

Thank you for your time and information. Refold this card and tape for return mail.

Figure 1b.—Questionnaire used to gather intensity data for "United States Earthquakes."

useful for comparing the attenuation of several earthquakes. In particular, the magnitude of an old, preinstrumental earthquake may be estimated by comparing its intensity-attenuation curve with those of better documented modern shocks in the same region. An intensity-attenuation curve can also be used to estimate the distances to which damaging intensities are likely to extend for a hypothetical future large earthquake in a region.

Isoseismal areas for earthquakes along international borders or along sea coasts are not complete. These partial areas should be denoted as different from complete areas. If complete areas are essential, they may be obtained by sketching in the missing parts of the partial contours. However, magnitudes estimated from sketched-in contours may easily be over- or underestimated by as much as a whole magnitude unit.

Felt Area

Felt area, along with maximum intensity I_0, is frequently reported for old, preinstrumental earthquakes. A map showing the rough boundaries of the felt area may be the best isoseismal map that is possible for some large, but poorly documented, earthquakes. Felt area can also be used to estimate magnitude by comparing an earthquake's felt area to felt areas of other earthquakes of known magnitude in the same region.

Felt areas limited by international borders or the sea are frequently given in catalogs as the area within a country or on land only. These partial felt areas should be clearly denoted as different from complete felt areas.

Unusual Effects and Special Circumstances

Unusual or unexpected effects occur during almost every earthquake. Some examples: (1) An old, poorly designed, poorly constructed, and decaying building within an intensity-VIII area is, nevertheless, not badly damaged. (2) A new, modern, well designed, well constructed, and well maintained building, supposed to be earthquake resistant, is severely damaged within an intensity-VII area. (3) Intensities are higher in one city block than in the adjacent block that has structures of similar age, quality, and type.

Special circumstances are things that sometimes explain unusual effects. They include (1) amplification or damping of seismic waves due to local geologic, topographic, or water table conditions; (2) liquefaction effects, ranging from sand blows to major landslides caused by the liquefaction of a layer at depth; (3) age, design, quality of materials, quality of workmanship, and upkeep of structures; and (4) previous earthquakes affecting the site.

Some special circumstances are needed to assign intensities. The intensity scale takes into account the quality of damaged structures. That is, a poor structure is expected to fail at a lower intensity than a good structure (see Richter, 1958, chap. 11). Records of types of circumstances that resulted in unusually low or high intensities are useful for future planning purposes in other similar regions.

Unusual effects also include those things that are reported frequently but that are not easily demonstrated or explained after the earthquake. For example, people often report seeing large, relatively slow, waves in the ground during an earthquake. But after the earthquake has ended, no trace can be found of the expected cracks, even when the waves were observed in an expanse of some rigid substance like concrete. The appearance of such waves may be a phenomenon of the inner ear, but it is not well understood. Other unusual effects about which a body of data is growing are earthquake lights and unusual animal behavior. Although none of these are useful for disaster planning purposes, they may some day prove useful in understanding the nature of the earthquake process and how structures and living things respond to that process.

Surface Faulting and Rupture Length

Damage is usually highest along the strike of a surface fault. Any structure built across the fault is expected to be torn apart. Also, the rupture length of a surface fault is related to the magnitude of the earthquake.

Faulting does not necessarily occur at the surface. Earthquake faulting at depth produces different patterns of seismic waves at the surface and different attenuation patterns.

Duration

Larger earthquakes, in addition to producing larger waves, higher accelerations, etc. than smaller shocks, also continue for a longer time. This increase in the duration of strong ground motion with increase in magnitude is an important component of the greater damage done by larger earthquakes. Since seismic waves attenuate with distance from the epicenter, duration of strong motion is also longer nearer the epicenter than farther away.

Duration for modern earthquakes is commonly defined as the time during which the instrumentally recorded acceleration remains above 5 percent g, or, sometimes, above 10 percent g. Sometimes, observers estimated a duration for preinstrumental earthquakes. Unfortunately, six observers in the same place are likely to report six different durations ranging, for example, from 15 seconds to 3 minutes.

Casualties and Injuries

Not only the numbers are of interest here, but also the circumstances. If all the dead and injured were in a single structure that collapsed, information about that structure is significant for future building codes. If many were in a major landslide or killed by a tsunami, that is significant for future zoning and land-use restrictions. This type of information is especially important if deaths and injuries from future earthquakes are to be minimized.

Estimate of Damage Cost

A number of different estimates of the dollar amount of damage may appear for a single earthquake. It is important to know who collected the information and for what purpose. Federal/local officials, federal/local disaster assistance personnel, insurance investigators, people gathering information for tax records, newspaper reporters, etc., all have different motives for collecting this kind of information and different methods of estimating the damage cost.

A fire following an earthquake (perhaps difficult to extinguish because of damaged water pipes) may cause more damage than the original shaking; so may a large tsunami. Afterwards, it may be impossible to distinguish earthquake shaking damage from fire or wave damage.

Tsunami

Earthquake-generated tsunamis can cause damage even at great distances from the epicenter of an earthquake. Locally, a tsunami may be more devasting than the shaking effects of the earthquake. For example, in coastal areas near the epicenter, a tsunami may strike within a few minutes of the shock, destroying whole harbors and small towns.

Seiches may also be generated in enclosed bodies of water at large distances from an earthquake.

Ground Effects

Ground effects caused by earthquakes include rock falls, landslides, slumps, lateral spreads, and liquefaction effects. These effects are known to occur on a small scale at lower intensities than their location in the Modified Mercalli Scale implies. The ground effects become large scale and common in areas where the right conditions exist, and where intensities are VIII M.M. or more. For old earthquakes, many or most of the reports may be about ground effects rather than shaking effects. Since ground effects occur over a wide range of intensities, it is difficult to assign a precise intensity based on ground effects alone. Therefore, intensities based solely on ground effects data are likely to be much less accurate than, for example, intensities based on damage to man-made structures.

A special category of ground effects is liquefaction effects. Repeated intense vibrations can cause a loose, water-saturated soil layer at depth to liquefy or loose all its shear strength. When this happens, water and soil may be pumped to the surface by continuing shaking, resulting in sand blows. Large buildings situated on top of a sand blow may sink or tilt, and buried structures (such as pilings or tanks) may float to the surface. There were many such effects in the 1964 Niigata earthquake (Kawasumi, 1968). If the liquefied layer slopes toward an open face or bluff, the entire mass above the liquefied layer may slide toward that face. This circumstance caused much damage in the 1964 Alaska earthquake (Wood, 1967).

Areas where the geologic, topographic, and hydrologic conditions suggest susceptibility to ground effects should be compared with areas that are close enough to a possible earthquake source to experience high intensities. Thus, areas having potential for severe ground effects may be delineated.

Long-Period Effects

The farther away from the epicenter, the more the longer period waves with low acceleration but high displacement begin to predominate over the short-period, high-acceleration vibrations that do the most damage in the epicentral region. At distances of several hundred kilometers, the predominant period is about the same as the natural period of tall buildings. Thus, damage may be done to such buildings in locations distant from the epicenter.

Other possible long-period effects are landslides, oscillations in well water, swinging and swaying objects, and dizziness reported by particularly sensitive people.

Aftershocks

Aftershocks, some quite large, may continue for days, or even years, after a great earthquake. Aftershocks of lower magnitude than the main shock sometimes cause damage equivalent to that of the main shock. For example, a structure weakened by the main shock may finally collapse during an aftershock.

Reports of long-continuing aftershocks may be useful for delineating the epicentral region of an old, preinstrumental earthquake. The larger aftershocks occurring within a few days or weeks of the main shock will have been reported felt over a fairly large area, but reports of the smaller shocks, which may have continued for years, will have been limited to the epicentral region.

For some purposes, such as calculating earthquake return periods, it is desirable to temporarily remove known aftershocks from the earthquake list.

Rockbursts and Explosions

Rockbursts or mine collapses and artifically generated explosions frequently find their way into catalogs of earthquakes. Whenever such a cause is known, it should be noted in the catalog to avoid future confusion with actual earthquakes.

Photographs

Since earthquake damage is seldom left undisturbed long enough to be studied very carefully, photographs provide a means of examining damage, showing it to others, and comparing it with damage from other earthquakes. Pictures are useful for earthquake researchers, for disaster planners, and for educating the public about earthquake safety.

Duplicate events

Before a catalog is useful for plotting seismicity or determining recurrence rates, multiple entries referring to the same shock must be removed. A catalog is usually compiled from several pre-existing catalogs. Epicenters, magnitudes, and times will not be reported as the exact same values by all the source catalogs. Confusion can also be caused by local time or Greenwich time.

Rules should be established for deciding which listing to accept when two or more entries thought to refer to the same earthquake are found. If one input catalog is generally more reliable than others, its values would be preferred. Experience with the data set is the best guide.

Format

Consider carefully the format to be used before starting to develop a new earthquake catalog. Changing the format of a large catalog can be a time-consuming problem, even with a computer.

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MODIFIED MERCALLI INTENSITY SCALE OF 1931
(From Wood and Newmann, 1931)

I

Not felt--or, except rarely under especially favorable
circumstances.
Under certain conditions, at and outside the boundary of
the area in which a great shock is felt:
I sometimes birds, animals, reported uneasy or disturbed;
R.F.[1] sometimes dizziness or nausea experienced;
sometimes trees, structures, liquids, bodies of water, may
sway--doors may swing, very slowly.

II

Felt indoors by few, especially on upper floors, or by sensitive, or
nervous persons.
Also, as in grade I, but often more noticeably:
I sometimes hanging objects may swing, especially when
to delicately suspended;
II sometimes trees, structures, liquids, bodies of water, may
R.F. sway, doors may swing, very slowly;
sometimes birds, animals, reported uneasy or disturbed;
sometimes dizziness or nausea experienced.

III

Felt indoors by several, motion usually rapid vibration.
Sometimes not recognized to be an earthquake at first.
Duration estimated in some cases.
III Vibration like that due to passing of light, or lightly
R.F. loaded trucks, or heavy trucks some distance away.
Hanging objects may swing slightly.
Movements may be appreciable on upper levels of tall
structures.
Rocked standing motor cars slightly.

[1] Indicates corresponding degree of intensity in the Rossi-Forel scale, an intensity scale widely used in the United States before the publication of the Modified Mercalli Scale in 1931. Intensity scales used in other parts of the world are discussed in Barosh (1969). An amplified version of the Modified Mercalli scale is given by Richter (1958).

IV

Felt indoors by many, outdoors by few.
Awakened few, especially light sleepers.
Frightened no one, unless apprehensive from previous experience.
Vibration like that due to passing of heavy, or heavily loaded trucks.
IV to V R.F. Sensation like heavy body striking building, or falling of heavy objects inside.
Rattling of dishes, windows, doors; glassware and crockery clink and clash.
Creaking of walls, frame, especially in the upper range of this grade.
Hanging objects swung, in numerous instances.
Disturbed liquids in open vessels slightly.
Rocked standing motor cars noticeably.

V

Felt indoors by practically all, outdoors by many or most; outdoors direction estimated.
Awakened many, or most.
Frightened few--slight excitement, a few ran outdoors.
Buildings trembled throughout.
Broke dishes, glassware, to some extent.
V to VI R.F. Cracked windows--in some cases, but not generally.
Overturned vases, small or unstable objects, in many instances with occasional fall.
Hanging objects, doors, swing generally or considerably.
Knocked pictures against walls, or swung them out of place.
Opened, or closed, doors, shutters, abruptly.
Pendulum clocks stopped, started, or ran fast, or slow.
Moved small objects, furnishings, the latter to slight extent.
Spilled liquids in small amounts from well-filled open containers.
Trees, bushes, shaken slightly.

VI

Felt by all, indoors and outdoors.

Frightened many, excitement general, some alarm, many ran outdoors.

Awakened all.

VI to VII R.F. Persons made to move unsteadily.

Trees, bushes, shaken slightly to moderately.

Liquid set in strong motion.

Small bells rang--church, chapel, school, etc.

Damage slight in poorly built buildings.

Fall of plaster in small amount.

Cracked plaster somewhat, especially fine cracks chimneys in some instances.

Broke dishes, glassware, in considerable quantity, also some windows.

Fall of knick-knacks, books, pictures.

Overturned furniture in many instances.

Moved furnishings of moderately heavy kind.

VII

Frightened all--general alarm, all ran outdoors.

Some, or many, found it difficult to stand.

Noticed by persons driving motor cars.

Trees and bushes shaken moderately to strongly.

Waves on ponds, lakes, and running water.

Water turbid from mud stirred up.

Incaving to some extent of sand or gravel stream banks.

Rang large church bells, etc.

Suspended objects made to quiver.

VIII-- R.F. Damage negligible in buildings of good design and construction, slight to moderate in well-built ordinary buildings, considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc.

Cracked chimneys to considerable extent, walls to some extent.

Fall of plaster in considerable to large amount, also some stucco.

Broke numerous windows, furniture to some extent.

Shook down loosened brickwork and tiles.

Broke weak chimneys at the roof-line (sometimes damaging roofs).

Fall of cornices from towers and high buildings.

Dislodged bricks and stones.

Overturned heavy furniture, with damage from breaking.

Damage considerable to concrete irrigation ditches.

VIII

Fright general--alarm approaches panic.

Disturbed persons driving motor cars.

Trees shaken strongly--branches, trunks, broken off, especially palm trees.

Ejected sand and mud in small amounts.

Changes: temporary, permanent; in flow of springs and wells; dry wells renewed flow; in temperature of spring and well waters.

VIII+ to IX- R.F. Damage slight in structures (brick) built especially to withstand earthquakes.

Considerable in ordinary substantial buildings, partial collapse: racked, tumbled down, wooden houses in some cases; threw out panel walls in frame structures, broke off decayed piling.

Fall of walls.

Cracked, broke, solid stone walls seriously.

Wet ground to some extent, also ground on steep slopes.

Twisting, fall, of chimneys, columns, monuments, also factory stacks, towers.

Moved conspicuously, overturned, very heavy furniture.

IX

Panic general.
Cracked ground conspicuously.
Damage considerable in (masonry) structures built especially to withstand earthquakes:
IX+ threw out of plumb some wood-frame houses built especially to withstand earthquakes;
R.F. great in substantial (masonry) buildings, some collapse in large part; or wholly shifted frame buildings off foundations, racked frames;
serious to reservoirs; underground pipes sometimes broken.

X

Cracked ground, especially when loose and wet, up to widths of several inches; fissures up to a yard in width ran parallel to canal and stream banks.
Landslides considerable from river banks and steep coasts.
Shifted sand and mud horizontally on beaches and flat land.
X Changed level of water in wells.
R.F. Threw water on banks of canals, lakes, rivers, etc.
Damage serious to dams, dikes, embankments.
Severe to well-built wooden structures and bridges, some destroyed.
Developed dangerous cracks in excellent brick walls.
Destroyed most masonry and frame structures, also their foundations.
Bent railroad rails slightly.
Tore apart, or crushed endwise, pipe lines buried in earth.
Open cracks and broad wavy folds in cement pavements and asphalt road surfaces.

XI

Disturbances in ground many and widespread, varying with ground material.

Broad fissures, earth slumps, and land slips in soft, wet ground.

Ejected water in large amount charged with sand and mud.

Caused sea-waves ("tidal" waves) of significant magnitude.

Damage severe to wood-frame structures, especially near shock centers.

Great to dams, dikes, embankments, often for long distances.

Few, if any (masonry), structures remained standing.

Destroyed large well-built bridges by the wrecking of supporting piers, or pillars.

Affected yielding wooden bridges less.

Bent railroad rails greatly, and thrust them endwise.

Put pipe lines buried in earth completely out of service.

XII

Damage total--practically all works of construction damaged greatly or destroyed.

Disturbances in ground great and varied, numerous shearing cracks.

Landslides, falls of rock of significant character, slumping of river banks, etc., numerous and extensive.

Wrenched loose, tore off, large rock masses.

Fault slips in firm rock, with notable horizontal and vertical offset displacements.

Water channels, surface and underground, disturbed and modified greatly.

Dammed lakes, produced waterfalls, deflected rivers, etc.

Waves seen on ground surfaces (actually seen, probably, in some cases).

Distorted lines of sight and level.

Threw objects upward into the air.

EFFECTS OF SHAKING ON BUILDINGS
BY CLASS OF CONSTRUCTION

by

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[Note: Halftone figures not included in these class notes.]

A study of the effects of earthquake shaking on buildings is useful from many standpoints. Among their various uses, statistical applications by government for earthquake vulnerability studies and disaster response planning are becoming increasingly important. The property insurance industry also needs this kind of information. This paper summarizes a classification system which has had widespread application and also summarizes United States experience from shaking damage.

American practices are described in this paper, particularly those of the western United States where many earthquake resistive buildings exist. The basis for much of this presentation comes from earthquake insurance methods used by property insurance companies.

The classification methods and their applications given herein do not supplant a detailed analysis which a structural engineer may prepare for a specific building. Rather this discussion is directed towards low cost approximations which are useful only when many similarly constructed buildings are considered as a group.

The description of building classification methods relates only to buildings in general and not to their sites. When individual site-specific structures are considered, the overall hazard must include not only the construction characteristics (i.e. building class) but also the possible site-specific geologic hazards. These hazards include soil liquefaction, landsliding, and faulting; geologic hazards are not a direct part of a building classification system.

A fuller description of the classification methods and their applications to buildings may be found in Chapters 5 and 6 of "Earthquakes, Volcanoes, and Tsunamis: An Anatomy of Hazards" by Karl V. Steinbrugge (Skandia Insurance Group, 280 Park Avenue, N.Y., N.Y. 10017). Reproduction of Skandia copyrighted material in this paper is by their permission.

PART A: BUILDING CLASSIFICATION

Occupancy vs. Construction Classification Systems

Rather generally, building classes for earthquake vulnerability or insurance purposes are determined by construction characteristics rather than by occupancy characteristics. Examples of construction characteristics include structural systems such as steel frame, reinforced concrete frame, wood frame, etc. while occupancy characteristics include building-use such as hotels, schools, hospitals, etc.

A limited argument for a classification system based solely on occupancy may involve the following reasoning. Consider a warehouse occupancy vs. a hotel occupancy, each in structurally identical 5-story reinforced concrete frame buildings. In the event of a moderate earthquake in which the reinforced concrete frame adequately resists the seismic forces in bending without structural damage, it can be clearly shown by engineering analyses that the hotel occupancy will potentially have a somewhat greater dollar loss on a percentage basis than will the warehouse occupancy. The difference in losses will be due to the damage to the additional non-structural partitions, plumbing, air conditioning, etc. found in the hotel but not found in the warehouse (as long as structural damage does not occur).

The foregoing argument is weak when the construction characteristics become a variable -- it was a constant in the previous example. Assume a 5-story non-reinforced brick bearing wall warehouse having sand-lime mortar in its masonry walls and having wood-joisted roof and floors. This structure will probably partially collapse or be severely damaged in the postulated moderate earthquake; on the other hand, its counterpart reinforced concrete hotel (should it have appropriate earthquake resistant shear walls) will have no or negligible damage. In this construction comparison, the damage range is much greater than that for the occupancy comparison.

As one investigates occupancy vs. construction in more detail, it becomes increasingly evident that the structural characteristics of a building substantially predominate over occupancy characteristics with respect to the degree of damage. However, it is also evident that a classification system based solely on materials of construction and how they are used will not always give best results. In conclusion, a simple practical classification system is normally based on materials of construction.

Building Classes, Building Codes, and Earthquake Resistive Construction

It is valid to question the use of building classification systems which differ from those used in building codes, particularly for newer buildings having earthquake resistive designs.

A building code containing earthquake bracing provisions is one criteria for a building classification but not the sole criteria. Examples exist of significant damage to modern earthquake resistive buildings designed according to code provisions. This seeming anomaly is due, in part, to the very important philosophical background for the seismic provision of building codes which allows for serious property damage, provided that life safety is not reduced. A hotel may have losses exceeding 50%, but if all the occupants can leave safely, then the building would meet today's intents of the earthquake provisions of the usual building code. It therefore follows that building classifications for monetary loss estimation purposes must take a different direction by recognizing the potentials for direct damage, functional impairment, as well as life safety.

Identification of Building Classes

Definitions of individual building classes used in this paper are given in Appendix A. These classes may be placed into one of two general categories:

Category A. By materials of construction: Classes 1 through 5,

Category B. By special damage control design: Class 6.

The "special damage control design" wording for Category B buildings does not convey a mental image of the details and methods of construction to a non-engineer field inspector. Whenever a Category B building has an equivalent (or a near equivalent) to one in Category B, then it is desirable to use a Category A classification. More often than not, Classes 3A and 4A have this dual capacity with Class 6. Some basic concepts on damage control engineering are discussed in the section describing buildings which fall into Class 6.

Classes 1A and 1B -- Wood Frame

Wood frame construction for dwellings and other small habitational occupancies is usually sufficiently well understood that perhaps it need not be illustrated in greater detail for classification purposes than shown in Figures 1 and 2.

Wood frame habitational structures, be they single or multifamily dwellings or non-dwellings, often have exterior wall finishes other than wood. These may include corrugated or aluminum metal siding, stucco, asbestos-cement ("transite"), and veneer. But what constitutes veneer? In general, it is brick, stone, concrete, marble, or other heavy material attached to the wood studs. It is appropriate to consider any masonry which is over 3 inches in thickness as being veneer; a 4-inch thick brick clearly qualifies as veneer and is the usual minimum in rule-making.

Certain multistory habitational structures, such as motels, condominiums, and apartment houses, may have a special classification problem due to fire resistive construction required for first story (ground level) automobile parking. For one example, the first story (including the floor of the second story) may be of reinforced concrete construction while above is rarely more than three full stories of wood frame. An appropriate classification is 4B or possibly 4A. This is correct from an engineering standpoint since the concrete provides the earthquake resistance (or its lack) against building collapse; the wood frame contributes very little mass by comparison.

Finally, wood frame as intended in Classes 1A and 1B includes all kinds of wood construction, be it the small wood studs of a dwelling (Figures 1 and 2) or the heavy timber of a wooden warehouse or processing plant. The area limitation of 3,000 square feet for Class 1A is somewhat arbitrary, although not unreasonable. The same may be said for height limitations.

Classes 2A and 2B -- All Metal

All metal buildings are easily classified using Figure 3. However, it is reasonably probable that very large modern Class 2B structures should receive favorable consideration for superior earthquake resistance. It can be easily shown on a mathematical basis that tall aircraft hangers, or similar large structures, are earthquake resistive if they have survived (or can survive) strong windstorms; this is due to wind forces being much greater than seismic as a result of the light mass of all-metal structures.

The phrase "all-metal" construction need not be taken too literally from a damageability standpoint. Corrugated cement-asbestos or wood siding (Figure 3) on an otherwise all-metal building is reasonably equivalent, as is fiberglass. One story height and 20,000 square feet limitations are somewhat arbitrary and probably conservative.

Classes 3A, 3B, and 3C -- Steel Frame

By definition, steel frame buildings have structural steel columns, beams, and girders. Figures 4 and 5 schematically show some of the material assemblies found with steel frames. These assemblies may also include steel trusses. Floors, roof, and walls may be of any material. Normally, steel frame structures are multistory buildings, but they need not be multistory. Figure 6 shows structural steel with a reinforced concrete shear wall core (also discussed in next section on concrete).

Older steel frame structures, such as those constructed in the 1920's and earlier, have poured-in-place concrete floors and roofs, and the beams and columns become fireresistive with poured-in-place concrete. This kind of fire protection develops substantial reserve seismic strength and no doubt has prevented collapses in some earthquakes. Exterior walls of these older buildings are usually unreinforced brick or poured-in-place reinforced concrete, and are supported by the steel frame. Visually, a concrete fireresistive steel frame building is about the same as a concrete building, and often construction drawings must be seen to distinguish between them. Normally, these older steel frame buildings will be Class 4B.

Modern steel frame buildings, including those being currently constructed, often have a lightweight concrete fill on metal deck, in turn supported by steel beams (Figure 7). The metal deck is commonly welded to the steel frame. Upon occasion, precast prestressed concrete "planks" or poured-in-place reinforced concrete may replace the metal deck. Modern fire protection of beams and columns may be gypsumboard or more usually of sprayed-on materials (Figure 8). While lighter and less expensive than poured-in-place concrete, these modern fireresistive materials do not contribute to the reserve seismic strength as does concrete fireprotection. Wall materials may be brick, concrete, precast concrete, glass, metal, or any number of other materials, all supported by the steel frame. These modern multistory steel frame buildings will qualify for Class 4A if they have good damage control features such as appropriate size shear walls.

Earthquake loss experience clearly indicates that, as a construction type, steel frame buildings should be kept distinct from reinforced concrete buildings. Fortunately, there are only a moderate number of buildings in which one of these materials does not predominate from an earthquake standpoint; the classification exceptions are considered in the next section on concrete buildings.

Damage Control Features:

The identification of some of the damage control features necessary for placing a building in Class 3A is possible by a visual field inspection in many cases. One of the favorable features is a height-to-least-width ratio not exceeding 4 to 1. A regular shaped building in plan and elevation is also favorable. A triangularly shaped building is unfavorable, unless very unusual engineering design precautions were taken. Well-covered and scarcely visible structural separations may divide an apparent T-shaped building or H-shaped building into two structures and thereby eliminate the seemingly irregular shape; such structural separations may be detected by trained field personnel. Exterior panel walls of poured-in-place concrete, precast concrete, brick, glass skin, metal skin, etc. are identifiable. Skins of metal or glass should be treated as open wall areas, as is usually true for precast concrete wall panels. Large interior open areas are unfavorable. The use of reinforced concrete shear walls (Figure 5), year built (as an indication of the seismic code provisions used in the design), and other identifiable features may be determined from a field inspection.

While a field inspection may identify favorable or unfavorable characteristics, it can not determine how flexible the steel frame will be under earthquake forces. It also often can not detect engineering provisions taken to minimize adverse flexibility effects. More specifically, the distance that one story will move horizontally with respect to the story beneath it can not be visually determined ("story drift" in engineering jargon). It is the amount of story drift between two floors which cracks exterior walls, smoketowers, stairs, ceilings, partitions, etc.

It must be remembered that Class 3A earthquake resistive buildings having superior damage control systems could be placed into the Class 6 series, but normally the equivalent within the Class 3 series is preferred since it is often easier for the user to relate to materials of construction than relate to abstract design features.

Classes 4A through 4D -- Reinforced Concrete

Reinforced concrete buildings are placed in Classes 4A through 4D (Figures 9 through 12), and their classifications parallel those of steel frame (Class 3 series).

An important distinction must be made between poured-in-place concrete and precast concrete. Classes 4A and 4B must have structural systems of poured-in-place reinforced concrete. Precast concrete, on the other hand, is commonly fabricated at a plant, and then completed beams and columns are hauled to the jobsite. Assembly at the jobsite (Figure 12) may require the welding together of metal inserts placed in each precast concrete member or interconnection by other means. Lift slab construction requires the reinforced concrete floors and roof to be poured one on top of another, then individually jacked into place (Figures 11 and 13).

Prestressed concrete is a material having high strength reinforcing steel which is stressed and located in the concrete in such a manner as to make the concrete much more effective. The special stressing of the reinforcing steel may be accomplished before the concrete is poured into forms containing the steel (pretensioned prestressed concrete) or after it is poured (post-tensioned prestressed concrete). Thinner floors are a result, and the building usually is of lighter mass. The process is not without its tradeoffs, however. The high stresses in the concrete due to prestressing will cause the concrete to "creep" or shorten. When this is not carefully considered by the designer, it may result (and has) in a badly damaged building even without an earthquake occurring.

The use of concrete walls for load bearing purposes has no steel counterpart, and therefore these concrete walls are frequently found in steel frame buildings (Figure 6). These walls may support all vertical loads tributary to them, thereby becoming bearing walls. They may additionally act as earthquake bracing walls, or "shear walls", as shown in Figure 10. All Class 4 series allow the substitution of concrete bearing walls for concrete columns.

The identification of some of the damage control features necessary for Class 4A is possible by a visual field inspection in many cases. The previous remarks under "Damage Control Features" for steel frame (Class 3 series) also apply here.

Classes 5A through 5C -- Mixed Construction

Mixed construction by its definition has many variants and thereby becomes complex to classify. Some of the principal variants are shown in Figure 14, and these should be adequate for most of the usual cases. Carefully made classifications and loss evaluations are often impractical for low value buildings, even if they are earthquake resistive. However, it is possible to develop general guidelines which directly or indirectly identify the use of reinforcement and quality of workmanship in materials such as reinforced hollow concrete block (cinder block) and reinforced brick masonry. Approximate guidelines may also identify the quality of wood roof ties to reinforced concrete tilt-up panels and the like. Some of these guidelines are given in the following paragraphs.

Non-reinforced Brick Construction (Class 5B):

Unreinforced brick, stone, and other unit masonry bearing wall buildings always have been associated with large life loss and extensive property damage, even in moderate earthquakes such as the 1933 Long Beach, California, earthquake. The 1983 Coalinga, California, earthquake essentially destroyed all buildings of this class in the central business district. Indeed, an examination of all historic records always confirms this. These kinds of masonry buildings are Class 5B. Brick is by far the most common unit masonry material in the United States, and these paragraphs therefore emphasize that material.

Except for the development and use of earthquake resistive brickwork in some regions of the United States since the 1933 Long Beach earthquake, usual brickwork was and continues to be laid in a manner whereby header courses show (Figure 15). A Class 5B multistory unreinforced brick building may often be identified from the exterior as being "brick bearing" (i.e. floor loads carried by the brick walls) by observing the increased wall thickness in the lower stories at windows or at other wall openings.

This type of construction (Class 5B) has been time-tested for normal gravity (vertical) loads and for the walls' fire resistivity. However, when horizontal forces such as earthquakes occur, the usual sand-lime mortar is completely inadequate since its shear and tensile capabilities are very deficient. Sand-lime mortar may often be identified by digging at the mortar joint with a sharp object such as a screwdriver. If the mortar is readily removed, it probably is sand-lime. The author has had the experience of inspecting a building with its owner and, while the owner was explaining how substantial his structure was, the author dug out a brick and handed it to him. Even the poorest quality concrete sidewalk will be stronger than most sand-lime mortars -- try digging a hole in a concrete sidewalk with a nail!

The use of cement in brick mortar contributes substantial amounts of shear and tensile strength. But cement mortar is more difficult to apply by brick masons, therefore more costly, and since time immemorial "construction costs are too high."

Reinforced Brick Construction (Class 5A):

Reinforced brickwork was introduced into building codes as a result of the 1933 Long Beach (California) earthquake. Figures 16 and 17 are examples of reinforced brick walls during the course of construction. These kinds of buildings, when completed, qualify for Class 5A or Class 6. Note that the center of the wall is reserved for reinforcing steel and concrete (more appropriately termed "grout", being a very fluid concrete having small aggregates, i.e. small rock and sand). This material assembly has performed excellently.

Visual identification of reinforced brickwork relies on the premise that no bricks extend through the center grout core (Figure 16). Therefore, when viewing the completed wall, a person sees only the sides of the brick ("stretchers") and not their ends ("headers"). Mortar will be much stronger and therefore difficult to dig out.

Hollow Concrete Block (Cinder Block):

Hollow concrete block (cinder block) is a new material compared to brick or stone, and therefore much less earthquake experience exists. Such experience as does exist has been poor when the block is not reinforced, consistent with theory, and Class 5C applies when no reinforcement information is available.

Wall construction of this type also can be made earthquake resistive by reinforcement placed as shown in Figure 18. Significant variants exist in the type and placement of the reinforcement. Workmanship problems can exist. For example, Figure 19 shows an unfilled cell from a collapsed building in the 1964 Alaskan earthquake. Similar workmanship problems can also occur with brick when dirt, sawdust, and mortar drop to the bottom of the grout area during construction, then is not removed, and the result is a very weak section in the wall.

Visual identification of reinforced hollow block can sometimes be made by the slight change of color in the face of grout filled block. Lightly tapping the walls with a hammer may detect different sounds at the grout filled cells. None of these methods test the quality and workmanship, but are indicative. Only comparatively expensive sampling ("coring") or inspection during construction is adequate.

Tilt-up Concrete Wall Construction (Class 5A):

Tilt-up concrete wall construction is currently very common for one and two story warehouses, light manufacturing, and several other occupancy types having wood roofs. It has largely replaced poured-in-place reinforced concrete walls for these occupancy types due to substantial cost savings.

Figure 20 shows a precast concrete wall panel being "tilted-up" into place. Reinforced concrete tilt-up walls are cast flat on the concrete floor (Figure 20). After curing and when the concrete has obtained adequate strength, the wall panels are "tilted" up and into place (Figure 20). They are then temporarily restrained by diagonals until the roof system is in place.

The earthquake resistance of these buildings is important at floor-to-wall and wall-to-wall connections.

Figure 21 shows some of the major variants for pilasters (columns) with tilt-up walls. Not shown is the reinforcing steel, bolting from wood column to tilt-up wall, and welding. Based on this diagram, certain generalized guidelines may assist in determining superior, average, or poor in class:

Superior: Types A and B. Type E if complete steel frame (or Class 4B)
Average: Type C. Type E if incomplete steel frame
Poor: Type D

The foregoing are general rules which are not necessarily applicable in every case.

Class 6 -- Earthquake Resistive Buildings

Alternatives to the use of Class 6 have been discussed in connection with the other classes. When these alternatives are not available or practical, then the best answer is a building classification determined by a structural engineer who is knowledgeable in earthquake damage control. These reviews may not always be cost-effective for other than the larger buildings which fall into Classes 3A, 4A, and 6.

A brief overview of some of the concepts behind damage control engineering may be of interest. The amount of earthquake damage is not only a function of the building's resistance against collapse, but to a great extent it is a function of the design characteristics used in achieving the required strength.

One and two story light industrial and warehousing buildings are often of the "box system" concept (Figure 22). By box system is meant that the roof and the exterior walls are intended to act in a manner similar to the top and sides of a box when the top of the box is pushed sideways. The plywood roof deck (top of the box), acts as a horizontal diaphragm, i.e., stressed skin. The exterior (and interior) unit masonry walls or tilt-up concrete walls parallel to the direction of ground motion act as bracing walls, or more appropriately are termed "shear walls". Interior columns are normally not intended to resist any significant amount of earthquake force in the box system concept. This box system concept for earthquake resistive design is quite valid, and its effectiveness depends primarily on the connections among the structural elements. Excessive openings in any shear wall are of major concern in the effectiveness of this system. Roof-to-wall ties are often critical.

In a more general sense, earthquake bracing for all kinds of buildings may be divided into two types: a. frame action (by columns and beams interacting in bending), and b. shear walls (by bracing walls). A combination of these frequently exists. Additionally, there are variants such as X-bracing systems which are more commonly found in one and two story buildings.

In buildings where the steel or reinforced concrete frame resists earthquake forces, the columns and beams resist these forces in bending as may be seen in Figure 23. The relative story-to-story movement, often termed story drift, may be in terms of inches and still not overstress the columns and beams. However, story drift in amounts which normally are expected to occur in an earthquake will shatter floor-to-floor partitions, continuous elevator shafts and stair walls, stairs, plumbing, exterior walls, and other items extending between stories. Therefore, occupancies with numerous partitions, such as hotels and the like, can have substantial interior and exterior damage, possibly approaching 50% of building value, and still be considered safe by code standards.

A shear wall building (Figure 23 right) is normally quite rigid compared to a steel or concrete framed structure which resists earthquake forces by frame action. In shear wall buildings, the earthquake forces are resisted by the shear walls acting as bracing walls. Shear walls usually are of reinforced concrete, but may be of reinforced grouted brick (Figure 17), reinforced hollow concrete block (Figure 18), or wood in wooden buildings. Shear walls may be exterior walls, interior walls, or both.

PART B: SELECTED DAMAGE STATISTICS AND CHARACTERISTIC DAMAGE

As has been stated, it is important for vulnerability studies, disaster response planning, and earthquake insurance purposes to identify damage and damage patterns, and also to quantify the results in terms of casualties, monetary losses, and functional impairments. The person seeking detailed information other than that briefly given here must turn to the accounts of specific earthquakes. Alternatively, a much more complete summary of United States experience than given here may be found in the previously cited "Earthquakes, Volcanoes, and Tsunamis; An Anatomy of Hazards". The tabular information and the illustrations which follow are from that reference.

Class 1 Series: Wood Frame and Mobile Homes

The great 1906 San Francisco earthquake approaches the maximum credible in its magnitude, intensity, and horizontal fault displacement which may be expected to occur in the maximum probable earthquake in California. Examples of satisfactory performance of wood frame are shown in Figures 24 and 25. Tables 1 and 2 are compilations of wood frame dwelling damage in cities close to the San Andreas fault on which the 1906 San Francisco earthquake occurred. Houses are better constructed today in that they are more securely anchored to better foundations. Also brick chimneys are now reinforced in California.

In the 1933 Long Beach, California, earthquake, a larger than normal number of wood frame dwellings had serious damage due to failures at or near the foundation level (Figure 26). This was a result of a lack of lateral force bracing or was due to the deterioration of such bracing as might have existed. Table 3 shows that of 4,575 wood frame residences studied in Compton about 95% of them had less than 5% damage. Dollar losses were obtained from the building department of the county assessor's office, supplemented by information from Compton city building permits and field surveys.

After the 1971 San Fernando earthquake, a field inspection was conducted of over 12,000 single family dwellings in the most heavily shaken areas. Dwelling construction was almost universally of conventional wood frame. Exterior wall foundations were concrete and were continuous for the exterior stud walls of newer buildings. Soils throughout the inspected areas were generally weak alluvium.

Damage distribution by construction component is given in Table 4. Figure 27 shows the results of the percentage loss determination made for 12,037 inspected dwellings in the hardest hit areas. The data in Figure 27 do not include the many thousands of cases of negligible to very slight damage, usually far less than 5% of value, found throughout much of eastern San Fernando Valley away from the hardest hit areas; nor does it include the extremely few cases of damage which exceeded 5% throughout this same area. Using "all dwellings" as a reference in Figure 27, it is evident that older dwellings such as the pre-1940 construction fared worse than did more recent construction. Two story dwellings fared even more poorly; however, the sample was small (less than 3% of the total) and it includes many older buildings. Dwellings of mixed one and two story fared poorest of all; no doubt this reflected the particular design characteristics for one of the newer construction types. Interestingly, no essential difference could be detected in the relative performance of wood joist floors vs. concrete slab floors on grade, even though the concrete slab construction tended to be the newer construction.

Dwelling loss data on the 1983 Coalinga, California, earthquake are still being compiled. The author's ongoing study of every 1 to 4 family dwelling (total of 2041 dwellings) found 95% of them to be 1 story and the others to be either 2 story or 1-and-2 story. Also, a total of 91% were 1 story wood frame, with the balance being other than 1 story or of brick, adobe, or tile. Table 5 shows preliminary results based on all Coalinga dwellings grouped together. Permanent dwelling movement (or shift) with respect to its foundation is a clear indicator of significant damage. From Table 5, one may conclude that older buildings are more vulnerable than are the newer ones. Also one may conclude that dwellings having concrete floor slabs performed better than supported wood floors; this finding is not supported by the 1971 San Fernando experience and is attributed to differences in regional construction practices.

Mobile Homes:

Five earthquakes in the last decade in California have shown that mobile homes in California are particularly vulnerable to earthquake damage. Damage and damage patterns recorded after the 1971 San Fernando, 1978 Santa Barbara, 1979 Imperial Valley, 1980 Livermore, and 1983 Coalinga earthquakes indicate a growing problem due to the rapid increase in the number of mobile homes throughout California. Earthquakes prior to 1971 were in areas where no significant number of mobile homes existed or were before the advent of mobile homes as they are known today.

Field observations by the author and others following the aforementioned four shocks also showed that mobile homes suffered much more damage than did nearby conventional single family dwellings. A few exceptions after the 1971 San Fernando shock could usually be related to special geologic conditions such as a dwelling astride a fault rupture, on a landslide, or affected by another geologic hazard. Significant exceptions have not been observed after other shocks.

A mobile home is a factory-built dwelling, commonly is a wood siding or a metal clad wood frame structure, and is erected on a steel frame. It is then towed on wheels to a site where it is most often structurally linked with a second similar structure to form a double-wide coach. The coach is raised off the wheels, leveled, and in California supported on piers spaced at a maximum of six feet on center along the coach frame below the floor unless specified differently by the coach manufacturer. Utilities are normally connected with flexible connections, lessening the chances of failure and fire when the coach falls off its supports. Normally, no earthquake bracing exists between the mobile home floor and the ground.

Representative damage is shown in Figure 28. It is important to note that once a coach falls off its supports, damage and losses will not greatly increase beyond the initial damage since further loss will essentially be restricted to effects from the coach sliding over fallen supports. Figure 29 compares mobile home expected losses with those expected from dwelling classes.

Classes 3 and 4 Series: Multistory Construction

Detailed comparative monetary loss data are very meager from United States experience. Fire followed earthquake after the 1906 San Francisco earthquake, and therefore no reliable segregated earthquake loss information is available. In the moderate 1971 San Fernando earthquake, the high-rise buildings were a number of miles from the heavily hit area. Anchorage, Alaska, experience in 1964 is probably the best in print (Table 6). This Alaskan experience clearly reflects long period effects since the energy release was distant (see next section).

Perhaps the best relevant foreign experience is from the 1967 Venezuelan earthquake. An estimated 1000 high-rise buildings existed in Caracas at the time of the earthquake. Only about 150 high-rise structures in the Los Palos Grande District and vicinity were examined in a preliminary fashion by the author but this is where almost all of the city's damage was concentrated. There were 142 buildings in that district which were 7 stories or higher, 81 buildings which were 10 stories or higher, and 16 buildings which were 15 stories or higher. Four collapsed, and damage to the remainder was frequently heavy. Detailed loss information may be found in "A Study of Soil Amplification Factors in Earthquake Damage Areas, Caracas, Venezuela", NOAA Technical Report ERL 280-ESL 31 (1972). These loss data may have limited transferability to other regions since unusual soil conditions apparently amplified the earthquake motions resulting in significantly increased damage in one small section of the city.

Long Period Effects and High-Rise Damage:

Not commonly understood are the special problems which occur with high-rise buildings (Classes 3 and 4) when located at large distances from a great earthquake. The potential for damage in future earthquakes is rapidly increasing with the increasing number of high-rise buildings.

In a destructive earthquake, the seismic motions at their source are generally of rapid and irregular oscillatory types having large amplitudes. Of considerable significance is the fact that earthquake waves change in character as they travel from their energy source (strained rock in and near regions of faulting). Human observations as well as seismographic records show that the very rapid and violent ground oscillations (short period motion) in the epicentral region are quickly damped and dispersed, leaving principally the gentle swaying motion (long period motion) at large distances away from the earthquake. The greater the distance, the slower the observed predominant oscillations. These predominant oscillations can be so gentle that they may not be felt by all persons, and yet be strong enough to cause water in reservoirs and in fiords to oscillate with sometimes destructive effects.

Each building has its own specific vibrational characteristics based on its stiffness and each building will therefore respond to the particular ground motion at the site in a specific manner. One of these vibrational characteristics is termed the structure's natural period of vibration. In general, the taller the building, the longer is its natural period of vibration. If the building's natural period of vibration roughly coincides with a few cycles of the principal motions of an earthquake, a case of quasi-resonance will occur, or a condition similar to near-resonance. As a result of this quasi-resonance, the vibratory motions of the building may dramatically increase, along with damage. Damage from quasi-resonance is generally observed in taller buildings from distant earthquakes.

Based on the changes in ground motions as a function of increasing distance, observed damage patterns tend to reverse with distance. Low, rigid (short period) building damage predominates over high-rise (long period) damage in the epicentral and energy release regions; the comparative damage patterns reverse at a distance, with the degree of reversal being a function of increasing distance (all other factors being equal). In other words, at distances over 100 miles (for example), high-rise building damage may predominate over that of even poorly built one story structures.

An excellent example of long period effects is the 1952 Kern County, California, earthquake which resulted in numerous instances of non-structural damage to multistory fire-resistive steel or concrete frame buildings in Los Angeles and Long Beach. These cities are located about 70 and 90 miles from the epicenter, respectively. Generally, the affected buildings were 10 to 12 stories high and had a measured natural period of vibration of 1 to 2 seconds, but buildings as low as 6 stories were also damaged. It was quite apparent that the period range of the ground motions more nearly coincided with the periods of the taller buildings than it did with the shorter periods of the low rigid buildings.

Ground motion with an estimated period of 1.5 to 2.0 seconds damaged a number of multistory buildings and caused several to collapse in Mexico City in 1957. This shock's epicenter has been variously reported as 170 to 220 miles from Mexico City. Merchandise did not fall from shelves of low buildings in Mexico City. Also poorly built, low, rigid buildings were not damaged at these long distances from the epicenter, including the "collapse hazard" adobe structures.

The 1964 Alaskan earthquake caused extensive damage to multistory buildings in Anchorage (Table 6) which was 75 miles from the epicenter and much farther from the center of energy release.

Class 5 Series: Unit Masonry Construction

Mixed construction loss data in United States are essentially those related to brick, concrete, and/or hollow concrete walls.

The 1933 Long Beach earthquake experience (Table 7) is typical for unreinforced brick having sand-lime mortar. Figure 30 shows one example of the severe damage that occurred. In his study on 1,264 brick-bearing walled structures, R.R. Martel concluded that the percentage of damage decreased with an increase in the number of stories: 23% for one story, 21% for two stories, 16% for three stories, and 12% for four stories.

The 1952 Kern County, California, earthquake was significant since it provided a partial comparison between unreinforced brick and reinforced brick as well as among other unit masonry types. Table 8 shows the performance of unit masonry public schools in Kern County; clearly the use of reinforcing steel and other earthquake resistant techniques was successful although not perfect.

The 1971 San Fernando experience is found in Table 9. These data from the mercantile district do not represent those of the hardest hit areas which were a few miles to the north, and therefore should not be compared with tilt-up construction discussed in the next section. However, one may compare the pre-1940 construction losses with the post-1949 construction losses, and it is evident that the modern buildings having varying degrees of earthquake bracing performed better than the older ones.

The 1983 Coalinga, California, earthquake resulted in essentially 100% loss to all non-reinforced brick buildings in the central business district since they were demolished after the event.

Class 5 Series: Light Industrial Construction

After the 1971 San Fernando earthquake, a detailed loss study was made of light industrial construction which included 61 buildings, all located in or near the ground disturbed areas near the faulting. Probably all were less than 10 years old at the time of the earthquake, and therefore should have been earthquake resistive.

The buildings were one story, and one-equals-two stories in height, and few had mezzanines. All were located in 2 industrial tracts. These structures represented essentially all of the completed light industrial construction having plywood roofs and tilt-up concrete or unit masonry walls in the heavily hit area. Exception to the foregoing was a small section of one otherwise plywood roof which was of undamaged poured gypsum. Examples of damage are shown in Figures 31 and 32, and observed damage is summarized in Table 10.

The buildings were examined in detail to accurately determine dollar losses and establish pre-earthquake "present worth" values as summarized in Table 11. From the table it may be seen that the average upper limit loss is 17.7% for this earthquake.

The other industrial tract located outside of the heavily shaken area had far less damage, usually negligible or none. It is reasonable to believe that the average loss for buildings throughout the San Fernando Valley was less that 3%, possibly less than 1%.

Figure 33 shows the loss distribution by type of wall construction. Clearly, almost half of the buildings examined had less than 10% loss; the right side of Figure 33 shows losses under 5% to be common.

Figure 33 also shows that unit masonry (reinforced brick and reinforced hollow concrete block) performed well. The data in Figure 33 should not be interpreted to mean that unit masonry walls outperformed tilt-up walls, despite the rapid comparative drop-off in loss to unit masonry. The sample is too small and other problems, such as the larger roof diaphragm areas found in tilt-ups, do not allow adequate numerical comparison between the relative performance of brick or hollow concrete block walls and tilt-ups. However, on a judgmental basis, their performance appeared to be about equal. In any event, the data showed that unit masonry and tilt-up walls could be equally effective earthquake bracing elements, but there was no available information to compare them with poured-in-place reinforced concrete walls.

APPENDIX A

CONSTRUCTION (BUILDING) CLASSIFICATIONS

Any building which fully qualifies under more than one definition should be placed in the lower numbered (i.e. best) classification.

Wood Frame Buildings

Class 1A: Single through four family wood frame dwellings. No limitations on story height, area, and construction materials.

Other wood frame and frame stucco habitational buildings which do not exceed 2 stories in height, regardless of area.

Non-habitational wood frame and frame stucco buildings, except: (1) buildings which are over 3 stories in height; and (2) buildings which are over 3,000 sq.ft. in ground floor area.

Class 1B: Wood frame and frame stucco buildings not qualifying under Class 1A.

Class 1C: Mobile homes.

All-metal Buildings

Class 2A: All-metal buildings which are one story in height and 20,000 sq. ft. or less in ground floor area. Wood or cement-asbestos are acceptable alternatives to metal roofing and/or siding.

Class 2B: Buildings which would qualify as Class 2A except for exceeding area or height limitations.

Steel Frame Buildings

Class 3A: Buildings with a complete steel frame carrying all loads. Floors and roofs must be of poured-in-place reinforced concrete or of concrete fill on metal decking welded to the steel frame (open web steel joists excluded). Exterior walls must be non-load bearing and of poured-in-place reinforced concrete or of reinforced unit masonry. Buildings having column-free areas greater than 2,500 sq.ft. (such as auditoriums, theaters, public halls, etc.) do not qualify.

Class 3B: Buildings with a complete steel frame carrying all loads. Floors and roofs must be of poured-in-place reinforced concrete or metal, or any combination thereof, except that roofs on buildings over three stories may be of any material. Exterior and interior walls may be of any non-load bearing material.

Class 3C: Buildings having a complete steel frame with floors and roofs of any material (such as wood joist on steel beams) and with walls of any non-load bearing materials.

Reinforced Concrete Buildings,
Combined Reinforced Concrete and Structural Steel Buildings

NOTE: Class 4A and 4B buildings must have all vertical loads carried by a structural system consisting of one or a combination of the following: (a) poured-in-place reinforced concrete frame, (b) poured-in-place reinforced concrete bearing walls, (c) partial structural steel frame with (a) and/or (b). Floors and roofs must be of poured-in-place reinforced concrete, except that materials other than reinforced concrete may be used for the roofs of buildings over 3 stories.

Class 4A: Buildings with a structural system as defined by the note above with poured-in-place reinforced concrete exterior walls or reinforced unit masonry exterior walls. Not qualifying are buildings having column-free areas greater than 2,500 sq.ft. (such as auditoriums, theaters, public halls, etc.).

Class 4B: Buildings having a structural system as defined by the note above with exterior and interior non-bearing walls of any material.

Class 4C: Buildings having: (a) partial or complete load carrying system of precast concrete, and/or (b) reinforced concrete lift-slab floors and/or roofs, and (c) otherwise qualifying for Class 4A and 4B.

Class 4D: Buildings having a reinforced concrete frame, or combined reinforced concrete and structural steel frame. Floors and roofs may be of any material (such as wood joist on reinforced concrete beams) while walls may be of any non-load bearing material.

Mixed Construction

Class 5A: Buildings having load bearing exterior walls of (a) poured-in-place reinforced concrete, and/or (b) precast reinforced concrete (such as "tilt-up" walls), and/or (c) reinforced brick masonry, and/or (d) reinforced hollow concrete block masonry. Floors and roofs may be of wood, metal, poured-in-place concrete, precast concrete, or other material. Interior bearing walls must be of wood frame or any one or a combination of the aforementioned wall materials.

> NOTE: No class distinction is made between newer highly earthquake resistive buildings and older moderate earthquake resistive buildings having these construction materials.

Class 5B: Buildings having load bearing walls of unreinforced brick or other types of unreinforced solid unit masonry, excluding adobe.

Class 5C: Buildings having load bearing walls of hollow tile or other hollow unit masonry construction, adobe, and cavity wall construction. Also included are buildings not covered by any other class.

Earthquake Resistive Construction

Class 6: Any building with any combination of materials so designed and constructed as to be highly earthquake resistant and also with superior damage control features in addition to the minimum requirements of building codes.

TABLE 1

DWELLING DAMAGE FROM
1906 SAN FRANCISCO EARTHQUAKE IN SAN MATEO COUNTY

	Character of Foundation			
	Wood	Concrete	Brick	Total
Houses examined	387	225	230	842
Houses moved	73	59	58	190
Percent of houses moved	17	26	26	23
Foundations cracked	--	51	94	145
Percent of foundations cracked	--	23	41	--

Data from Volume 1 of the "Report of the State Earthquake Investigation Commission" by A.C. Lawson, et al

TABLE 2

DAMAGE TO PLASTER AND HOUSEHOLD ARTICLES ON HILLS AND LOW LANDS OF SAN MATEO COUNTY, SAN FRANCISCO EARTHQUAKE OF 1906

Location	Amount of Damage	Percentage in cases of cracking of plaster	Percentage in cases of falling of dishes, etc., in varying amounts
San Mateo and Redwood City	Slight	40	40
	Medium	30	20
	Great	30	40
Belmont	Slight	79	74.3
	Medium	11	23.3
	Great	10	2.3

Data from Volume 1 of the "Report of the State Earthquake Investigation Commission" by A.C. Lawson, et al.

TABLE 3

DAMAGE TO WOOD FRAME RESIDENCES IN COMPTON AS A RESULT OF THE 1933 LONG BEACH EARTHQUAKE

Damage (percent)	Number of buildings	Fraction of total number (percent)
0-4	4,334	94.7
5-24	131	2.9
25-49	63	1.4
50 and more	36	0.8
Demolished	11	0.2
Total	4,575	100.0

From Professor R. R. Martel´s paper "Earthquake Damage to Type III Buildings in Long Beach, 1933" which was published in "Earthquake Investigations in the Western United States, 1931-1964".

TABLE 4

WOOD FRAME DWELLING DAMAGE
1971 SAN FERNANDO EARTHQUAKE

Construction Component	Percentage of Buildings Having Described Damage			
	None	Slight	Moderate	Severe
Foundation	91.9%	5.8%	1.6%	0.7%
Damage to frame	78.8%	16.0%	3.3%	1.9%
Interior finish - plaster	4.2%	78.4%	11.1%	6.3%
Interior finish - gypsumboard	12.1%	78.0%	6.5%	3.4%
Exterior finish - stucco (plaster)	20.7%	74.1%	4.0%	1.2%
*Brick chimney damage	67.6%	16.1%	6.6%	7.4%

*Total brick chimney damage was found in 2.3% of the cases. "Total" means exactly that; essentially no bricks were left standing, or the chimney was otherwise so damaged as to be non-repairable.

TABLE 5

OBSERVED MOVEMENT BETWEEN DWELLINGS AND THEIR FOUNDATIONS

Very slight to several feet -- summary of yet-to-be published study

Age Group	Number of Dwellings Having: Wood floor	Concrete floor	Number of Dwellings with Observed Movement Between Superstructure and Foundation: Wood floor	Concrete floor
Pre-1940	812	27	197	1
1940-49	301	49	16	1
Post-1949	363	405	6	0
All ages	1,476	481	219	2

TABLE 6

DAMAGE TO MULTISTORY BUILDINGS IN ANCHORAGE, ALASKA

1964 ALASKAN EARTHQUAKE

Building Name (occupancy)	Year Built	Stories	Structural Frame	Floors	Exterior Walls	Earthquake Bracing System	Monetary % Loss
R/C buildings, or principally R/C:							
Airport control tower	1952	6B	R/C	R/C	Insulated metal	R/C frame	100
Penney, mercantile	1962	5	None	R/C	R/C, HCB, precast	R/C	100
*Four Seasons, apts.	1964	6	None	*	Plastered studs	Shear walls	100
Mt. McKinley, apts.	1951	14B	R/C	R/C	R/C	Shear walls	40
1200 "L", apts.	1951	14B	R/C	R/C	R/C	Shear walls	30
Elmendorf Hospital	1955	7B	R/C	R/C	Non-structural HCB	Shear walls	***
Knik Arms, apts.	1950	6B	R/C	R/C	R/C	Shear walls	Negligible
Structural Steel, or principally structural steel:							
Hill, offices	1962	8	Steel	R/C on steel	Insulated metal	Shear walls	20-25
Cordova, offices	1960	6B	Steel	R/C on M/D	Insulated metal	Shear walls	20
**Anchorage-West., hotel	1960/64	14B	Steel	R/C on M/D	Insulated metal	Shear walls	12
Providence Hosp.	1961	5B	Steel	R/C on M/D	Insulated metal	Shear walls	2.5

Notes:

*Prestressed post-tensioned lift slabs using steel columns. Shear walls failed in overturning.

**Steel frame also contained earthquake bracing. Some exterior wall was concrete.

***Structural damage 1%; nonstructural greater.

Abbreviations:

R/C	Reinforced concrete
M/D	Metal deck supported by steel beams and girders.
HCB	Hollow concrete block
B	Basement

TABLE 7

DAMAGE TO MASONRY WALLED BUILDINGS IN COMPTON (1933 LONG BEACH EARTHQUAKE)

	Commercial		Residential	
Damage, %	Number of buildings	Fraction of total number, %	Number of buildings	Fraction of total number, %
	2	2	13	47
5-24	5	4	3	16
25-49	26	21	4	27
50-75	25	20	1	10
100 (demolished)	64	53	0	0
Total	122	100	21	100

For all practical purposes, the masonry-walled construction consisted of unreinforced brick-bearing walls. Interiors were wood, although occasionally some steel or brick might have existed. Data from "Earthquake Investigations in the Western United States, 1931-64", in the chapter by R. R. Martel, Table 5.

TABLE 8

DAMAGE TO UNIT MASONRY PUBLIC SCHOOLS
1952 KERN COUNTY, CALIFORNIA, EARTHQUAKE

	Number of schools	
Extent of damage	Earthquake resistive	Earthquake nonresistive
None	21	1
Slight	6	9
Moderate	1*	9
Severe	0	13
Collapse	0	1

*Administration Building, Arvin High School.

TABLE 9

MERCANTILE BUILDING DAMAGE
IN DOWNTOWN SAN FERNANDO
1971 SAN FERNANDO EARTHQUAKE

Wall construction	Number of Buildings				
	No damage	Slight damage	Moderate damage	Severe damage	Total examined
Brick:					
Pre-1940	10	25	25	14	74
1940-1949	3	1	0	0	4
Post-1949	4	0	1	0	5
Hollow concrete block:					
Pre-1940	9	7	6	4	26
1940-1949	6	0	0	0	6
Post-1949	25	1	0	0	26
Reinforced concrete:					
Pre-1940	5	1	0	0	6
1940-1949	1	1	0	0	2
Post-1949	3	0	0	0	3
Wood frame and other:					
Pre-1940	13	4	2	1	20
1940-1949	3	0	0	0	3
Post-1949	4	1	0	0	5
Total	86	41	34	19	180

Note: Roof and floors were wood. Normally 3 stories and under in height.

TABLE 10

SUMMARY OF OBSERVED DAMAGE TO LIGHT INDUSTRIAL BUILDINGS 1971 SAN FERNANDO, CALIFORNIA, EARTHQUAKE

Damage	Wall Construction: Tilt-up	Wall Construction: Unit Masonry	All Buildings
Concrete floor slab:			
None, or hairline cracks (to one-eighth inch)	15	11	26
Moderate cracking (to one inch)	13	7	20
Severe cracking (over one inch)	7	3	10
*Walls:			
None, or slight damage	11	9	20
Moderate damage to some portions	12	6	18
Severe damage to some portions	14	4	18
Collapse of some portions	2	3	5
Roofs:			
None, or slight damage	11	13	24
Moderate damage to some portions	5	3	8
Collapse of some portions	20	4	24

*Includes 5 buildings for which access was denied.

TABLE 11

DOLLAR LOSSES TO LIGHT INDUSTRIAL BUILDINGS

1971 San Fernando, California, Earthquake
Arroyo and Bradley Tracts

Tract	*Number of Buildings	Pre-EQ Worth	Earthquake Damage	Loss %
Arroyo	33	$ 4,527,000	$ 788,000	17.4%
Bradley	23	$ 7,172,000	$1,277,000	17.8%
Total	56	$11,699,000	$2,065,000	17.7%

*Excludes 5 buildings where access was denied.

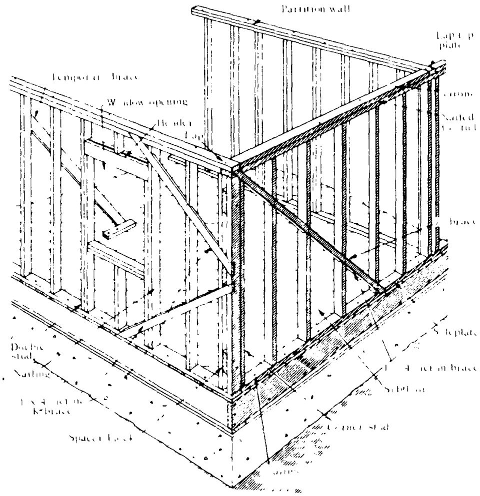


FIG. 1 Wood frame dwelling with platform construction. In platform construction, the subfloor sheathing extends to all outside edges of the dwelling, and the walls and partitions are erected on and nailed to this flat surface. Variations exist throughout the United States.

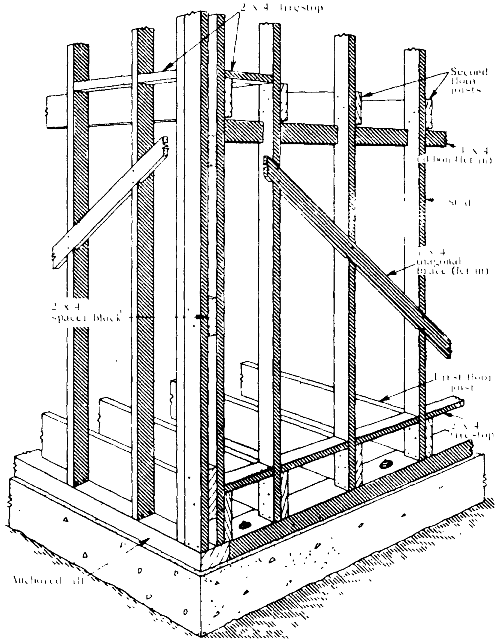


FIG. 2 Wood frame dwelling with balloon construction. The studs and joists rest on the anchored sill (compare with Figure 1). Variations exist throughout the United States.

Simplified Description of
ALL-METAL CONSTRUCTION

Class 2A: 1 story, floor not exceeding 20,000 sq. ft.

Class 2B: All-metal of any size.

Note: Cement-asbestos or wood acceptable alternate to metal roof and/or siding.

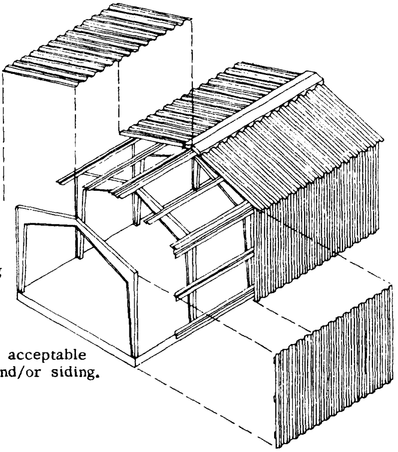


FIG. 3 Simplified classification of all-metal buildings.

Simplified
Description of
STEEL FRAME

No shear walls, i.e. no bracing walls.

•Class 3A: Superior damage control features.
Class 3B: Ordinary damage control features.
Class 3C: As shown, except floors and roof not concrete or metal deck.

•Engineering review recommended.

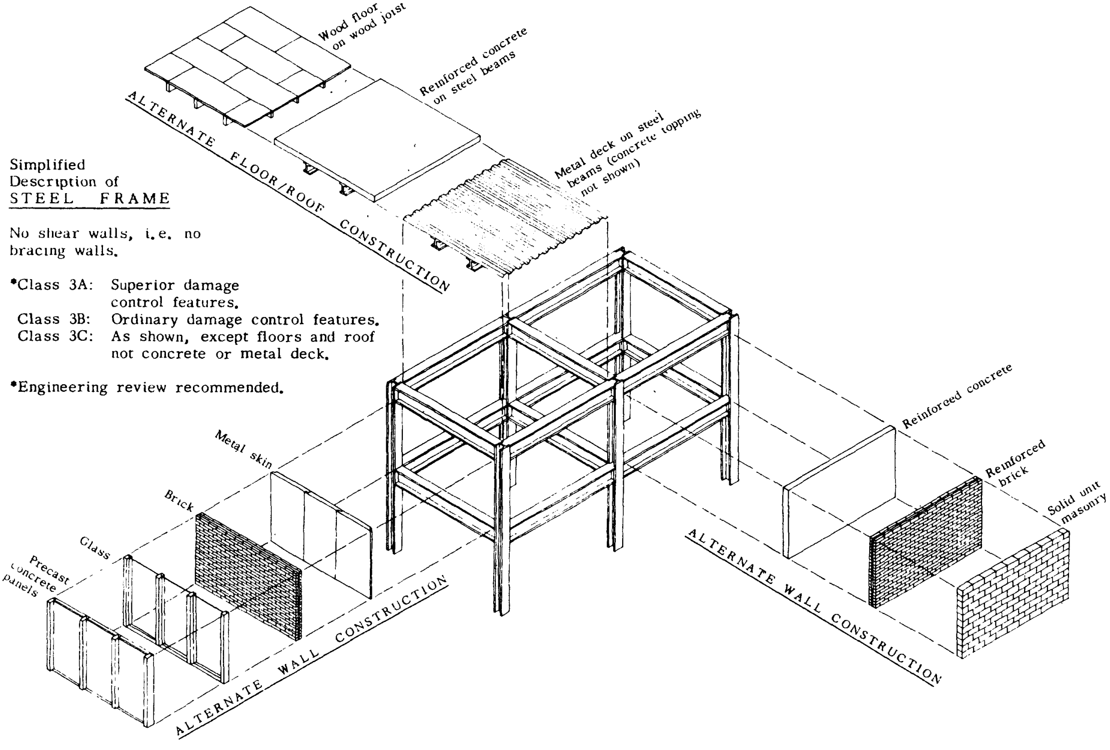
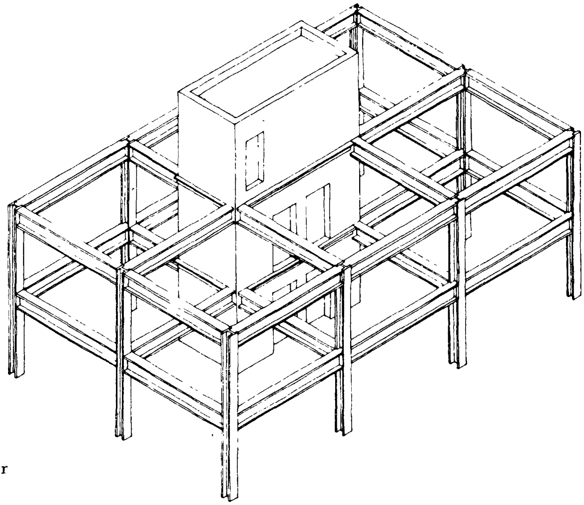


FIG. 4 Simplified classification of steel frame buildings without shear walls.



Simplified Description of
STEEL FRAME

Reinforced concrete shear walls around central core, i.e. central elevators and stairs. Alternate locations may be acceptable.

See previous diagram for kinds of exterior wall construction which may be used.

This type of construction often qualifies for Class 3A.

FIG. 5 Simplified classification for steel frame buildings with shear walls.

Figures 6, 7, and 8 are not included in these class notes.

Simplified Description of
REINFORCED CONCRETE FRAME

No shear walls, i.e. no bracing walls.

*Class 4A:	Superior damage control features.
Class 4B:	Ordinary damage control features.
Class 4C:	Precast reinforced concrete, lift slab (see next figure).
Class 4D:	As shown, except floors and roof not concrete.

*Engineering review recommended.

Note: Only Classes 4A and 4B shown in this diagram.

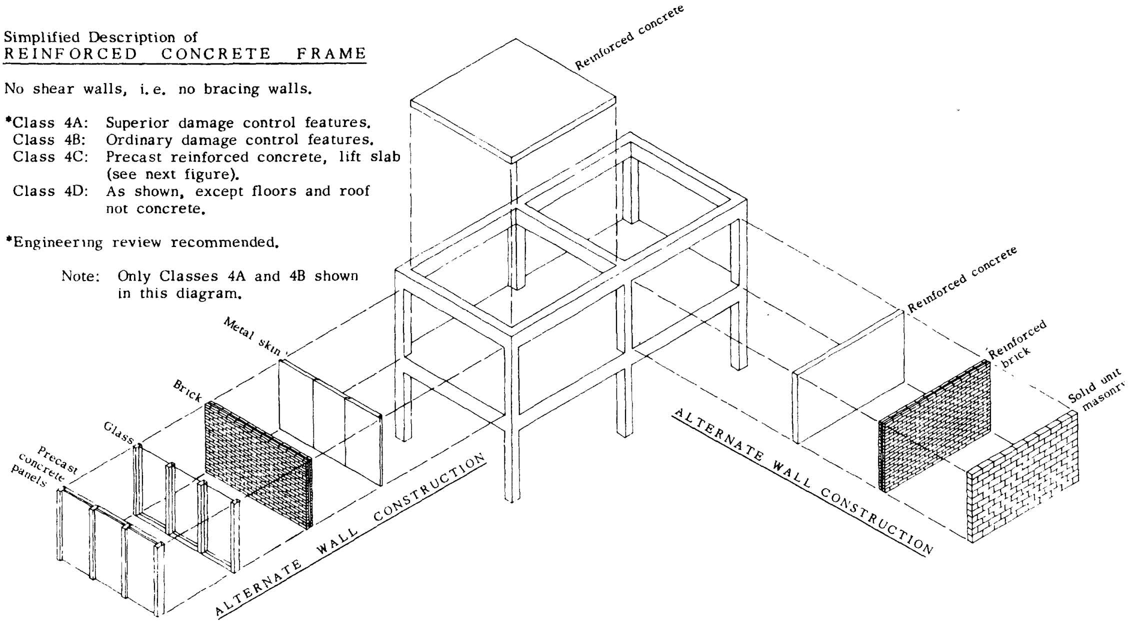
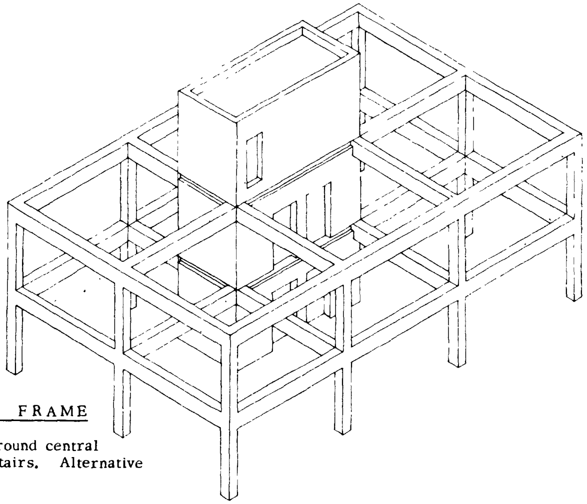


FIG. 9 Simplified classification of reinforced concrete frame buildings without shear walls.



Simplified Description of
REINFORCED CONCRETE FRAME

Reinforced concrete shear walls around central core, i.e. central elevators and stairs. Alternative locations may be acceptable.

See previous diagram for kinds of exterior wall construction.

This type of construction qualified for Class 4A.

FIG. 10 Simplified classification of reinforced concrete frame buildings with shear walls.

Simplified Description of
LIFT SLAB CONSTRUCTION

Class 4C lift slab construction shown here; Class 4C precast concrete construction shown in Figure 12.

Floors and roof are poured reinforced concrete and are usually prestressed. Floors and roof are cast one-upon-another at ground level, then jacked up the columns (usually steel) to their respective final levels, and often braced by a central core of reinforced concrete. This central core normally contains the stairs and elevators.

Exterior walls may be of any materials.

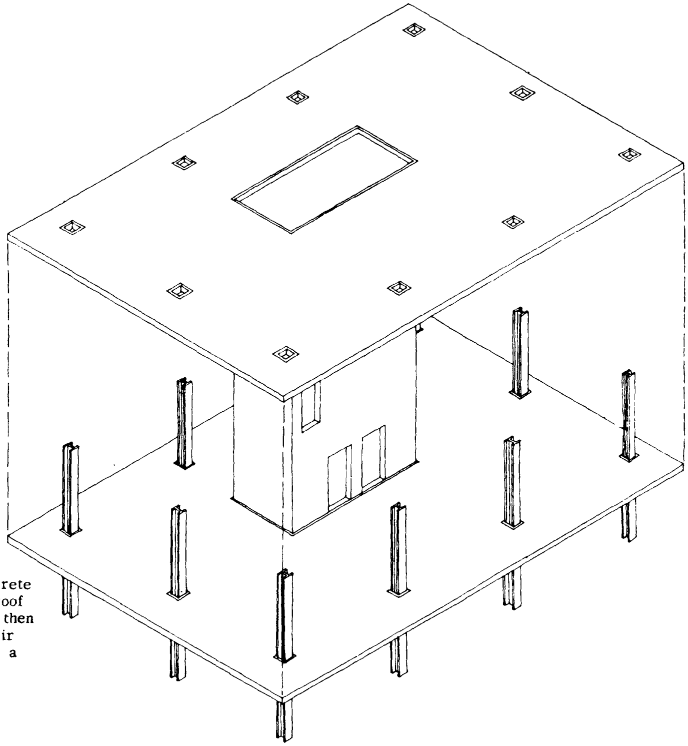


FIG. 11 Simplified description of lift slab construction.

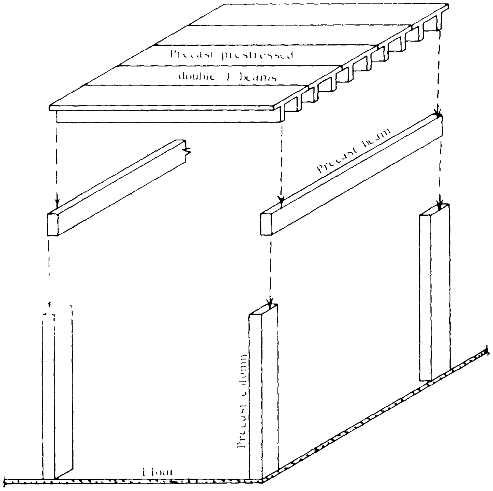
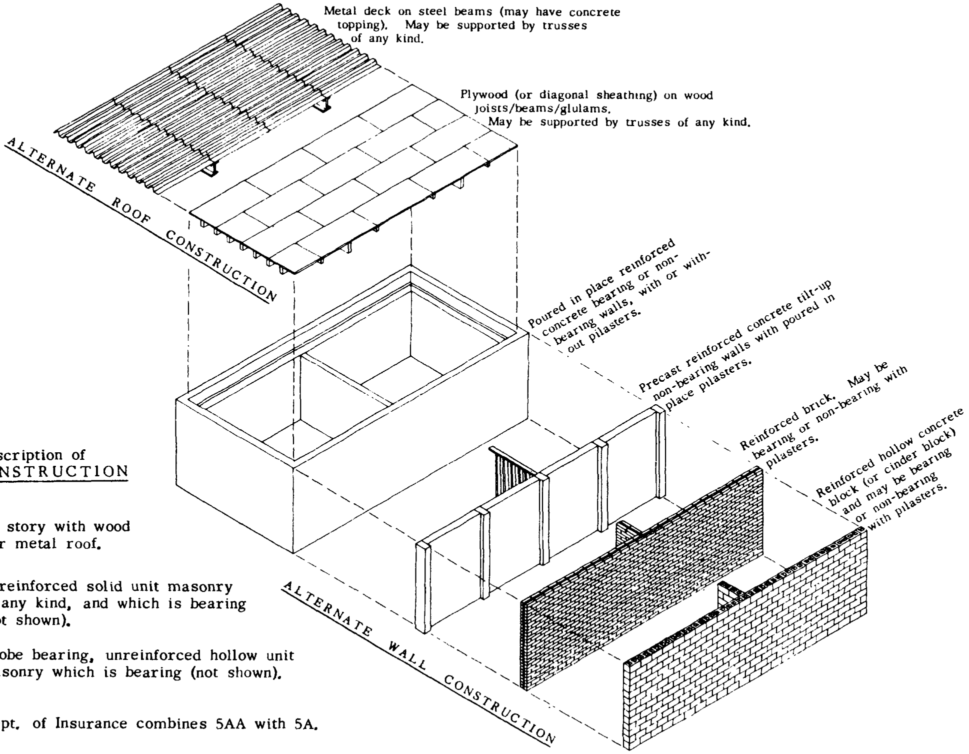


FIG. 12 Class 4C, showing an exploded diagram of one of many common types of precast concrete assemblies. Interconnections may be made by welding together metal plates cast into the concrete members as suggested in this figure, or by pouring a concrete topping slab over the precast floor elements, or by pouring concrete instead of using precast concrete for some members.

Figure 13 is not included
in these class notes.



Simplified Description of
MIXED CONSTRUCTION

Class 5A:	1 story with wood or metal roof.
Class 5B:	Unreinforced solid unit masonry of any kind, and which is bearing (not shown).
Class 5C:	Adobe bearing, unreinforced hollow unit masonry which is bearing (not shown).

*California Dept. of Insurance combines 5AA with 5A.

FIG. 14 Simplified classification of mixed construction.

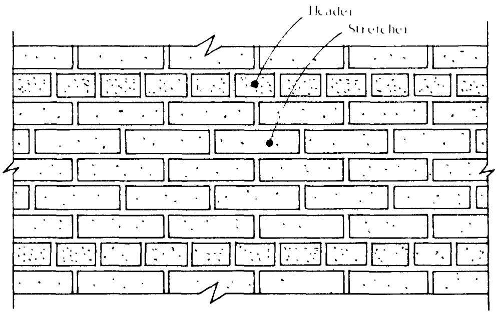


FIG. 15 Face of an unreinforced brick wall laid in common bond. In common bond, header courses are laid in every fifth, sixth, or seventh course with stretchers between. Headers are here shown in every sixth course.

Figures 16 and 17 are not included in these class notes.

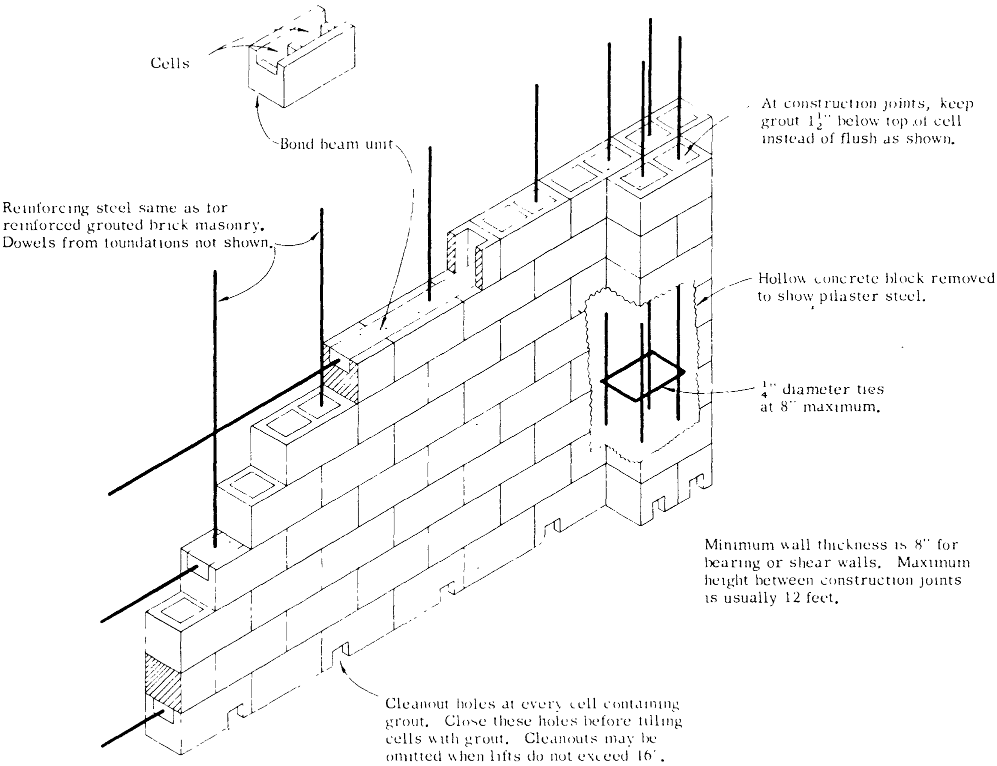


FIG. 18 Diagram of reinforced hollow concrete block construction. Cleanout holes are often omitted, thereby usually reducing the earthquake shear resistance at the foundation level. When omitted, dirt, sawdust, and mortar droppings accumulated at the bottom of the cell cannot be removed, and the grout does not bond to the foundation.

Figures 19 and 20 are not included
in these class notes.

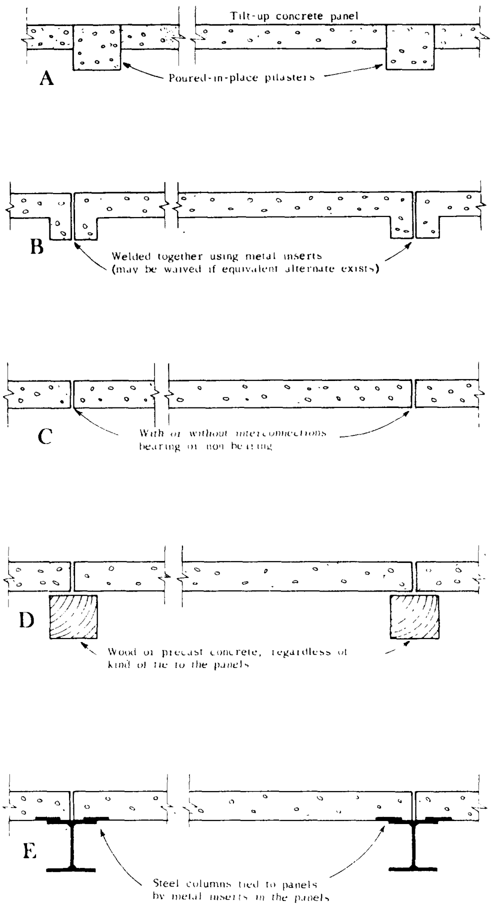


FIG. 21 Plan view showing types of pilasters (columns) between two tilt-up wall panels. The building classification may vary as a function of the type of pilaster or its absence. Reinforcing steel, bolts, and welding not shown.

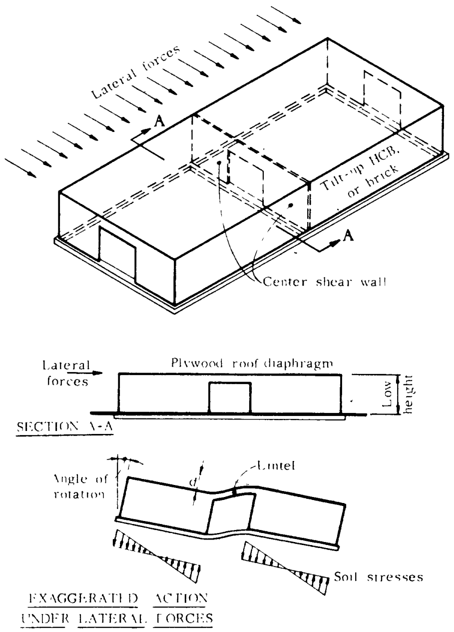


FIG. 22 "Box" concept in earthquake resistive design.

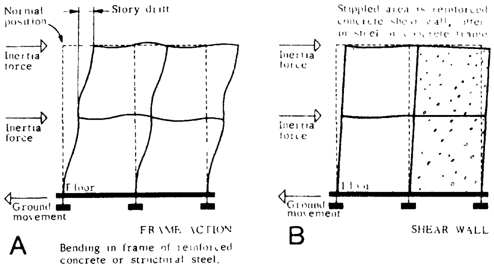


FIG. 23 Two earthquake bracing methods. In (*A*), columns and beams resisting earthquake forces in bending. (*B*) A shear wall resists earthquake forces instead of using the columns and beams.

Figures 24, 25, and 26 are not included in these class notes.

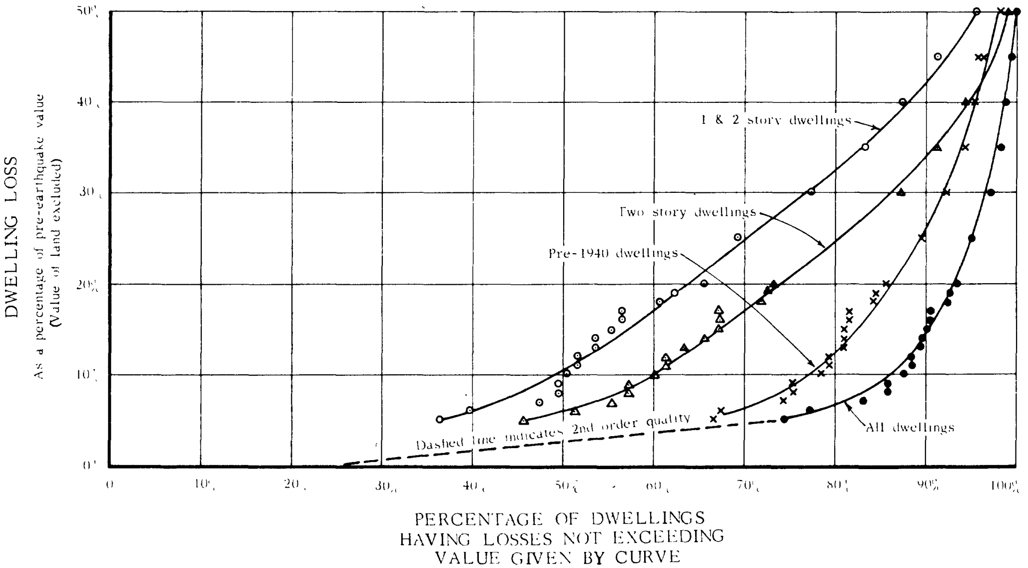


FIG. 27 Single family wood frame dwelling loss in the most heavily shaken areas of the 1971 San Fernando, California, earthquake.

Figure 28 is not included
in these class notes.

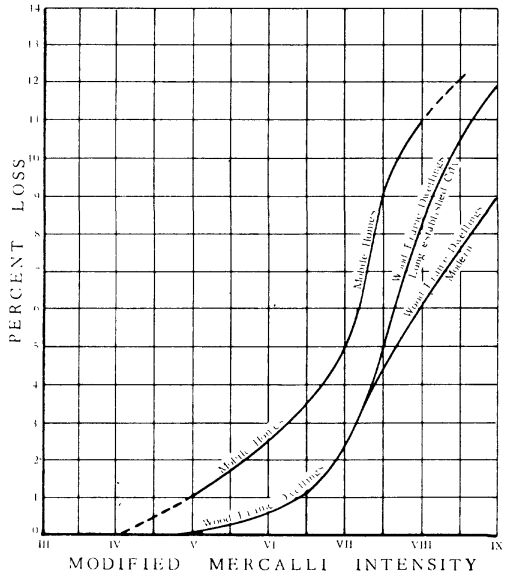


FIG. 29 Monetary loss to mobile homes (in percent of value) as a function of Modified Mercalli Intensity. *Dashed line* is inferred.

Figures 30, 31, and 32 are not included in these class notes.

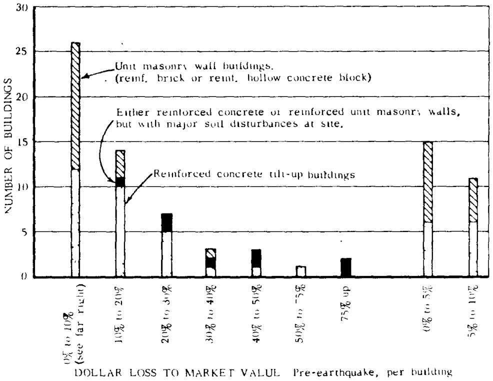


FIG. 33 Loss distribution by wall type for light industrial construction, 1971 San Fernando earthquake.

GEOLOGIC EFFECTS - LIQUEFACTION AND ASSOCIATED GROUND FAILURE

By

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INTRODUCTION

Geologic effects caused by earthquakes are divided into two principal categories, primary effects and secondary effects. Primary effects are those caused by deep-seated forces in the earth and include fault rupture and tectonic uplift and subsidence. Other lecturers in this course discuss these primary effects and the paper by Bonilla (1981) entitled Surface Faulting (in the volume entitled Facing Geologic and Hydrologic Hazards which is included in the course materials) gives information on this topic. Secondary geologic effects are those geologic effects caused by ground shaking and include liquefaction and compaction of sediment and various forms of ground failure. This presentation treats liquefaction and those ground failures associated with this phenomenon. The presentation by Wilson and Keefer entitled Earthquake-induced Ground Failures, treats other forms of ground failure including various types of landslide, rock falls, etc., that occur on steep slopes and in areas not affected by liquefaction.

Ground failure caused by liquefaction are a major cause of earthquake damage and casualties. For example, most of the extensive damage caused by the 1964 Alaska earthquake was a consequence of liquefaction. Major and spectacular damage triggered by liquefaction also occurred during earthquakes in India in 1897 and 1905, California in 1906, in Japan in 1923 and 1964, and Chile in 1960. Because of the potential for damage, considerable study of the liquefaction phenomenon has occurred in the past few years to provide criteria for evalulating liquefaction and ground failure potential. Such evaluations are a major element in earthquake hazard assessment and mitigation studies.

LIQUEFACTION PROCESS

Liquefaction is a process by which clay-free soil deposits, primarily sands and silts, temporarily loose strength and behave as a viscous liquid rather than as a solid. The actions in the soil which produce liquefaction are as follows: Seismic waves, primarily shear waves, passing through a saturated granular layer, distort the granular structure, and cause some loosely packed groups of particles to collapse (Figure 1). Disruptions to the particulate structure generated by these collapses cause transfer of load from grain-to-grain contacts in the soil to the interstitial pore water. This transfer of load increases pressure in the pore water, causing drainage to occur. If drainage is restricted, a transient build up of pore-water pressure will occur. If this porewater pressure rises to a critical level, usually just short of the overburden pressure, grain-to-grain contact stresses approach zero and the granular layer temporarily behaves as a liquid rather than as a solid and liquefaction has occurred. In the liquefied condition, soil deformations may occur with little shear resistance. Deformations large enough to cause damage to constructed works (usually about 0.1 m) are called ground failure.

The ease with which a soil can be liquefied depends on the looseness of the soil, the packing arrangement of soil grains, the amount of cementing between particles, and the amount of drainage restriction. The amount of soil deformation following liquefaction depends on the looseness of the material, the areal extent of the liquefaction, and the ground slope and the distribution of loads applied by buildings and other structures on the ground surface.

Liquefaction does not occur at random, but is restricted to certain geologic and hydrologic environments, primarily recently deposited sands and silts in areas with high ground water levels. Generally, the younger and looser the sediment, and the higher the water table, the more susceptible the soil is to liquefaction. Sediments most susceptible to liquefaction include Holocene (less than 10,000 year-old) delta, river channel, flood plain, and aeolian deposits and poorly compacted fills. Few soils deposited before the Holocene era have liquefied during modern earthquakes. Liquefaction has been

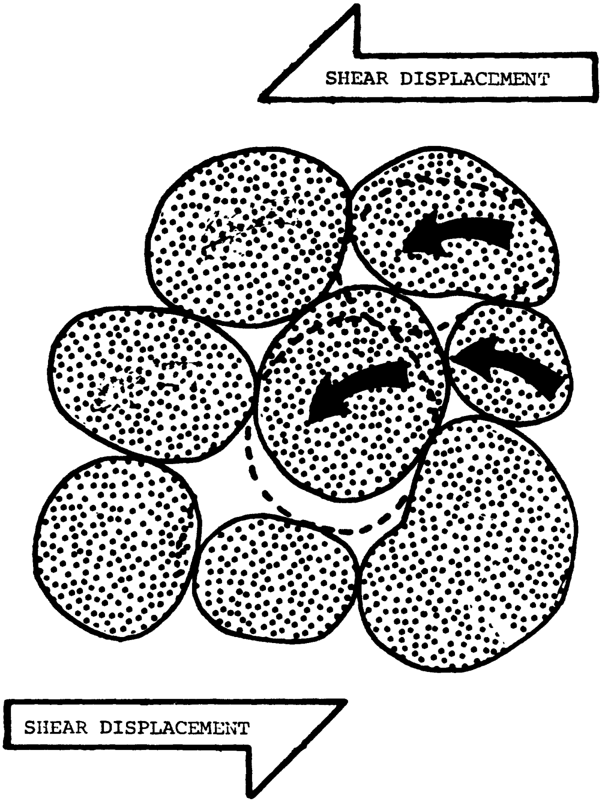


Figure 1.--Sketch of water-saturated group of sand grains illustrating the process of liquefaction. Shear deformations (indicated by large arrows) brought about by earthquake ground shaking distort the granular structure causing some loosely packed groups to collapse as indicated by the small arrows. Each collapse transfers stress from grain-to-grain contacts to the pore water, increasing the pressure in that water. When pore-water pressures reach a critical level (grain-to-grain contact stresses approach zero), the granular material suddenly behaves as a liquid rather than as a solid. At this point, liquefaction has taken place. (After Youd and Keefer, 1981)

most abundant in areas where ground water lies within 10 m of the ground surface; few instances of liquefaction have occurred in areas with ground water deeper than 20 m.

GROUND FAILURE TYPES

Four primary types of ground failure are caused by liquefaction-lateral spreads, ground oscillation, flow failure, and loss of bearing strength. In addition, liquefaction may enhance ground settlement and leads to the eruption of sand boils (fountains of water and sediment emanating from the pressurized, liquefied zone as shown in Figure 2).

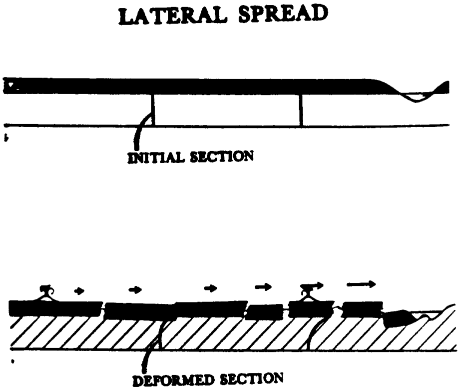
Lateral Spreads

Lateral spreads involve lateral displacement of large, surficial blocks of soil as a result of liquefaction in a subsurface layer (Figure 3). Movement occurs in response to combined gravitational forces and inertial forces generated by an earthquake. Lateral spreads generally develop on gentle slopes (most commonly between 0.3 and 3 degrees) and move toward a free face such as an incised river channel. Horizontal displacements on lateral spreads commonly range up to several meters, but where slopes are particularly favorable and ground shaking durations are long, displacements may range up to several tens of meters. Ground shifted by lateral spreading usually breaks up internally, causing fissures, scarps, horsts, and grabens to form on the failure surface. Lateral spreads commonly disrupt foundations of buildings built on or across the failure, sever pipelines and other utilities in the failure mass, and compress or buckle engineering structures crossing the toe of the failure.

Damage caused by lateral spreads, though seldom catastrophic, is severely disruptive and often pervasive. For example, during the 1964 Alaska earthquake, more than 200 bridges were damaged or destroyed by spreading of floodplain deposits toward river channels. The spreading compressed bridges over the channels, buckled decks, thrust stringers over abutments, and shifted and tilted abutments and piers. Figure 4 shows a buckled bridge deck typical of damage inflicted during that earthquake. Similar damage has occurred in many other large earthquakes.

Figure 2.--Sand boil generated during the 1981 Westmorland, California, earthquake. Sand boils are caused by water laden with sediment venting from a subsurface liquefied layer in which artesian pore-water pressures develop during the liquefaction process.

Figure 3.--Diagram of lateral spread before and after failure. Liquefaction occurs in the cross-hatched zone. Surface layer moves laterally down mild slope breaking up into blocks bounded by fissures. The blocks also may tilt and settle differentially with respect to one another.



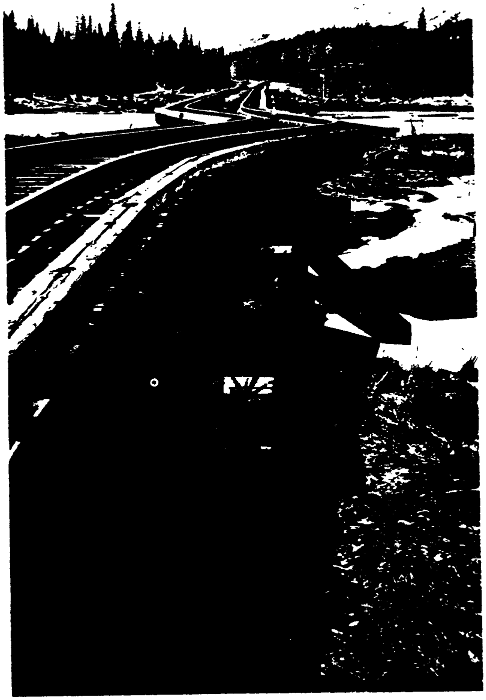


Figure 4.--Bridge in Alaska compressed and buckled by lateral spread that moved toward the stream channel during the 1964 earthquake.

Lateral spreads are particularly destructive to pipelines. For example, every major pipeline break in the city of San Francisco during the 1906 earthquake occurred in areas of ground failure. These pipeline breaks severely hampered efforts to fight the fire that ignited during the earthquake; that fire caused about 85 percent of the total damage to the city. Thus, rather inconspicuous ground-failure displacements of less than 2 m were in large part responsible for the devastation to that city (Youd and Hoose, 1978).

Ground Oscillation

Where slopes are flat or too gentle to allow lateral displacement, liquefaction at depth commonly decouples overlying soil blocks allowing them to jostle back and forth on the liquefied layer in response to earthquake shaking (Figure 5). This jostling of blocks produces ground oscillation often seen by observers as traveling ground waves. The oscillations are accompanied by opening and closing fissures and commonly ground settlement; these disruptions of the ground can inflict serious damage to overlying structures and to pipelines and other facilities buried in the ground.

Flow Failures

Flow failures are the most catastrophic ground failure caused by liquefaction. These failures commonly displace large masses of soil tens of meters and in a few past instances, large masses of soil have traveled tens of kilometers down long slopes at velocities ranging up to tens of kilometers per hour. Flows may be comprised of completely liquefied soil or blocks of intact material riding on a layer of liquefied soil. Flows usually develop in loose saturated sands or silts on slopes greater than 3 degrees (Figure 6).

Many of the large and most damaging flow failures have developed in coastal areas. For example, flow failures, mostly under water, carried away large sections of port facilities at Seward, Whittier, and Valdez, Alaska during the 1964 earthquake and generated large sea waves that overran these and other coastal areas, causing additional damage and casualties.

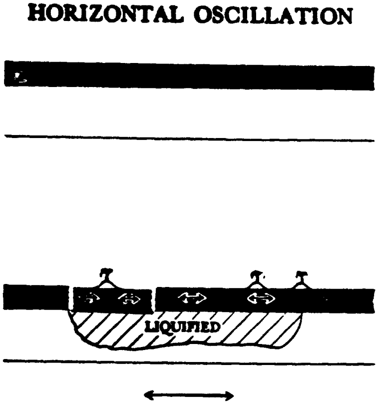


Figure 5.--Diagram showing mechanism of ground oscillation. Liquefaction occurs in the cross-hatched zone decoupling the surface layer from the underlying firm ground. The decoupled layer vibrates in a different mode than the underlying and surrounding firm ground causing fissures to form, and impacts to occur between oscillating blocks and adjacent firm ground. Travelling ground waves and opening and closing fissures are commonly seen during ground oscillation.

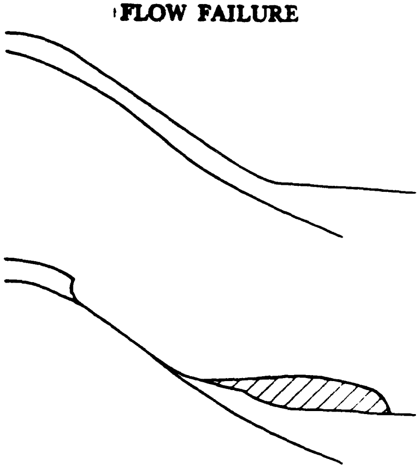


Figure 6.--Diagram of a flow failure. Liquefaction develops beneath ground surface causing soil to loose strength and flow down steep slope. On land, the liquefied soil and blocks of intact soil riding on the flow commonly come to rest as a mass when the flow reaches the bottom of the steep slope. Beneath water, some flows have traveled many kilometers down long gentle slopes.

On land, flow failures commonly called debris flows have been even more catastrophic, though perhaps not so numerous as submarine failures. Massive flow failures in collapsible loess deposits were reported during earthquakes in China and Russia. The 1920 Kansu, China, earthquake-triggered flow failures ranging up to 1.6 km in length and breadth. Some failures flowed downslope for several kilometers. Liquefaction in this instance was apparently caused by pressures generate in entrapped air rather than in water. An estimated 200,000 people were killed in these flows.

Loss of Bearing Strength

When the soil supporting a building or other structure liquefies and loses strength, large deformations can occur within the soil which may allow the structure to settle and tip (Figure 7). Conversely, buried tanks and piles may rise buoyantly through the liquefied soil. Many buildings settled and tipped during the 1964 Niigata, Japan earthquake. The most spectacular bearing failures during that event were in the Kwangishicho apartment complex where several four-story buildings tipped as much as 60 degrees (Figure 8). Apparently, liquefaction first developed in a sand layer several meters below ground surface and then propagated upward through overlying sand layers. The rising wave of liquefaction weakened the soil supporting the buildings and allowed the structures to slowly settle and tip. Most of the buildings were later jacked back into an upright position, underpinned with piles, and reused.

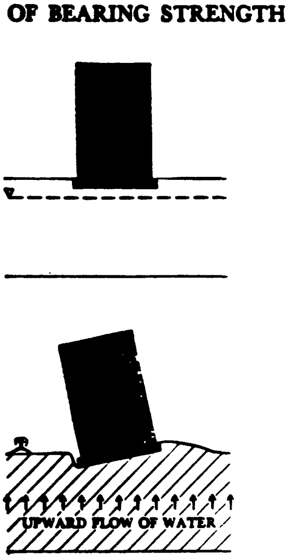
EVALUATION OF LIQUEFACTION POTENTIAL

Field and laboratory studies over the past few years have led to the development of criteria for predicting the occurrence of liquefaction. Criteria have been developed for both regional and site specific scales.

Regional Maps of Liquefaction Potential

The U.S. Geological Survey has developed a technique for making regional maps showing potential for liquefaction (Youd and Perkins, 1978). In this procedure two constituent maps; a susceptibility map and an opportunity map,

Figure 7.--Diagram tilting of building due to liquefaction and loss of bearing strength in soil. Liquefaction weakens soil reducing, foundation support for building, causing it to settle and tilt. While heavy structures settle, buried empty tanks and pipes may float upward through the liquefied soil.



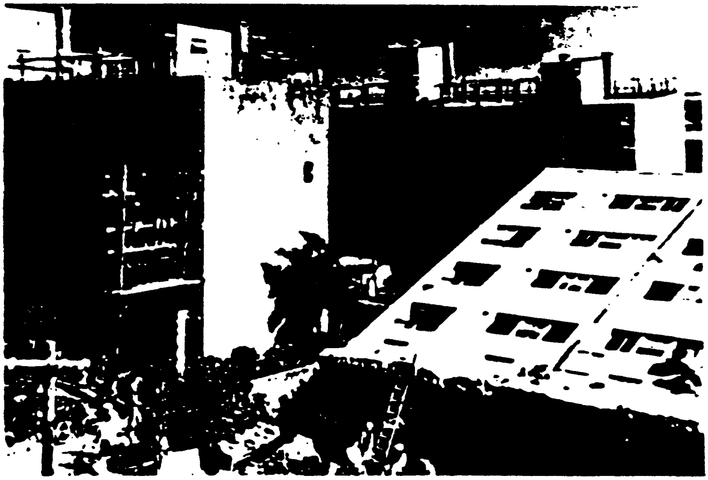


Figure 8.--Four-story apartment buildings in Niigata, Japan, that settled and tipped because of loss of strength in supporting soils during the 1964 Niigata earthquake. Loss of bearing strength was caused by liquefaction of a sand layer that extends 10 to 15 meters below the base of the foundations.

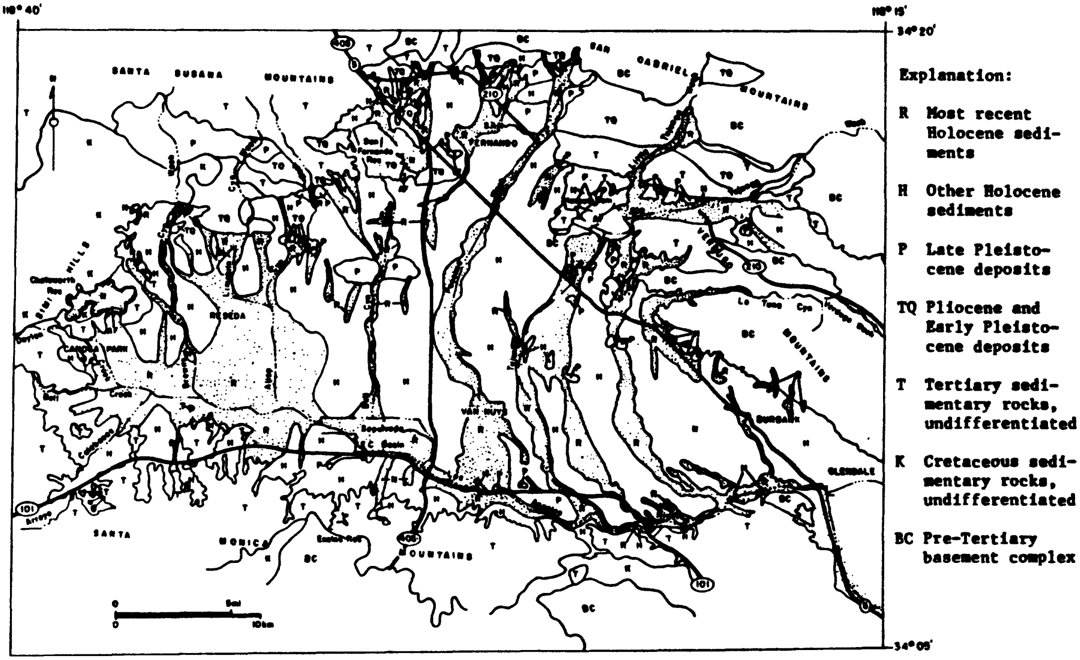


Figure 9.--Map showing distribution of geologic units in the San Fernando Valley, California. This map is the geologic base used in preparing a liquefaction susceptibility map for the area. (After Youd and others, 1978).

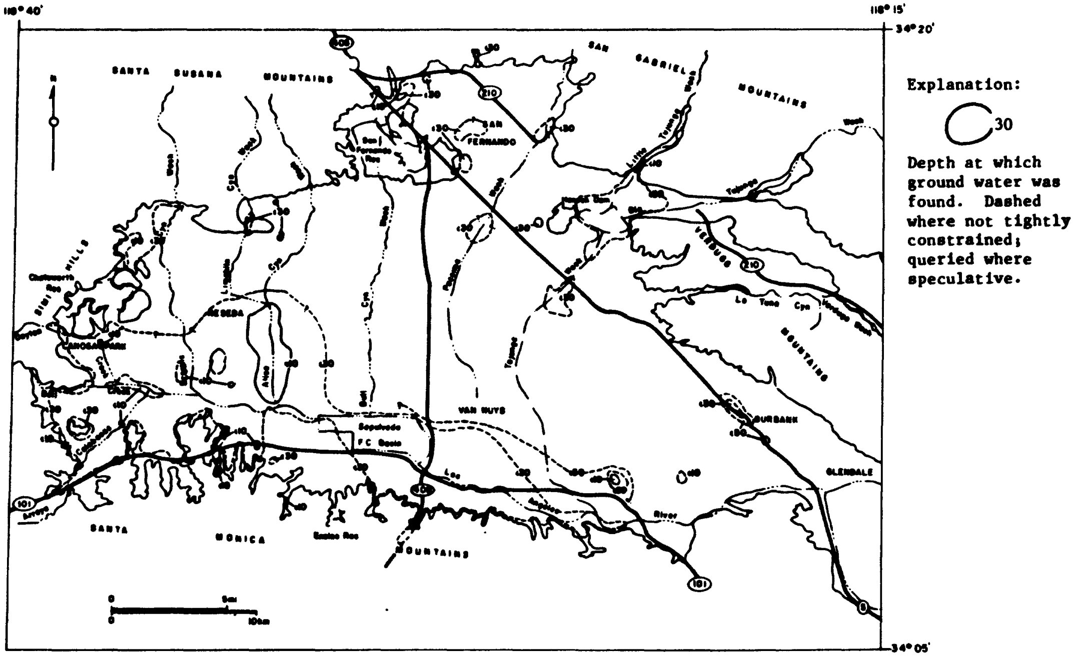


Figure 10.—Map showing approximate depth to ground water, including perched ground water, for the San Fernando Valley, California. (After Youd and others, 1978).

are compiled and then superimposed to form the final liquefaction potential map.

The susceptibility map separates areas underlain by sediment that could contain liquefiable deposits from areas where liquefiable sediment is highly unlikely. Geologic maps provide the primary data used to compile susceptibility maps. Geologic maps are required that show detailed delineation of sediment units deposited in the past 100,000 years, with emphasis on those units deposited most recently. For example, the geologic map shown in Figure 9 was used in the preparation of a liquefaction susceptibility map for the San Fernando Valley near Los Angeles, California. Maps that show depth to ground water are also required; one for the San Fernando Valley is shown in Figure 10. Criteria, such as those listed in tables 1 and 2, can then be used to qualitatively differentiate susceptibilities of the various geologic units. A map compiled using this procedure for the San Fernando Valley is shown in Figure 11.

Sediment property data, where available from geologic and engineering studies, can be used to better characterize sedimentary units and make the map more quantitative. In fact, if sufficient engineering data are available the map can be delineated in quantitative probabilistic values.

Regional susceptibility maps show only general or average susceptibility for an entire map unit; susceptibility, however, may vary somewhat from locality to locality within the unit. Detailed site-specific investigations are required to determine the exact susceptibility at a particular point or site within the unit.

A liquefaction opportunity occurs when the intensity of ground shaking reaches a level strong enough to cause liquefaction in susceptible materials. Liquefaction opportunity maps show the distribution of probabilities that such an opportunity will occur in a given period of time. Data needed to compile opportunity maps are estimates of future earthquake activity including delineation of seismic source zones and estimates of the frequency of occurrence of earthquakes with magnitude greater than 5 for each seismic source zone. Compilation of these data require specialized

Type of deposit (1)	General distribution of cohesionless sediments in deposits (2)	Likelihood that Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr (3)	Holocene (4)	Pleistocene (5)	Pre-pleistocene (6)
***(a)* Continental Deposits**					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
***(b)* Coastal Zone**					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave energy	Widespread	Moderate	Low	Very low	Very low
Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
***(c)* Artificial**					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Table 1.—Estimated Susceptibility of Sedimentary Deposits to Liquefaction during Strong Seismic Shaking. (After Youd and Perkins, 1978).

Sedimentary unit	Probable susceptibility of clay-free granular layers		
	Ground water depth, ft(m)		
	<30(9.1)	30(9.1)-50(15.2)	>50(15.2)
Most recent Holocene	High	Low	Very Low
Other Holocene	Moderate	Low	Very Low
Late Pleistocene	Low	Low	Very Low
Late Pliocene and early Pleistocene	Very Low	Very Low	Very Low
Tertiary	Very Low	Very Low	Very Low

Table 2.—Criteria used in compiling a liquefaction susceptibility map for the San Fernando Valley, California. (After Youd and others, 1978).

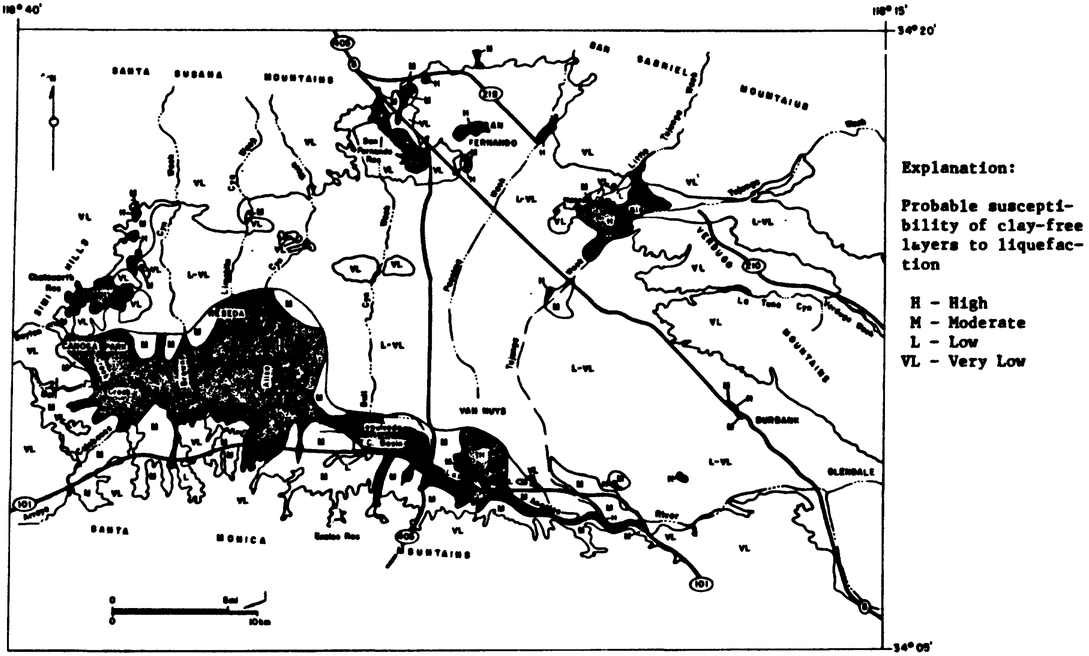
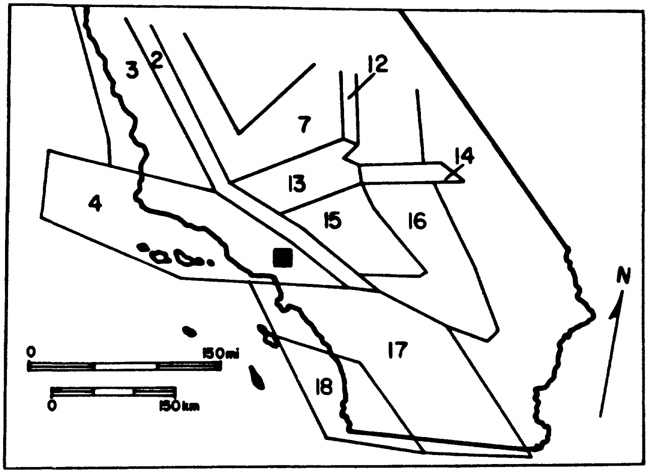


Figure 11.—Liquefaction susceptibility map for the San Fernando Valley, California, with zones showing the relative likelihood that liquefiable sediment exists beneath the ground surface.



Zone No.	Number of intensity V's/100 yrs/10^4 km^2 N	b-value for epicentral intensity[2] b_I	Maximum intensity[1] I_o	Maximum magnitude[1] M_c
2	60.6	-.40 up to XI then flat	XII	8.5
3	12.5	-.45	XI	7.9
4	24.2	-.45	XI	7.9
13	129.65	-.45	XI	7.9
14	136.56	-.45	XI	7.9
15	----	-.53	VIII	6.1
16	20.8	-.50	X	7.3
17	70.4	-.45	XI	7.9
18	2.5	-.50	X	7.3

[1] Modified Mercalli Intensity

[2] b-value for magnitude (b_M) = 0.6 b_I

Figure 12.--Seismic source zones near the San Fernando Valley (dark square) and table of estimated seismicity values for each source zone. These data were used to prepare a liquefaction opportunity map for the San Fernando Valley.

seismological studies. For example, the map and table in Figure 12 show seismic data used to prepare a liquefaction opportunity map for the San Fernando Valley. To complete the analysis, a correlation is required that relates earthquake magnitude to the distance to which liquefaction can occur. The relation shown in Figure 13 is useful and conservative for most parts of the world; the relation may be improved and refined, however, with further study and experience. This magnitude-distance relation is compatible with standard methods of probabilistic hazard analyses and can be used to prepare liquefaction opportunity maps with a probabilistic basis. This analysis was made for the San Fernando Valley and the results are shown in Figure 14.

The superimposed opportunity and susceptibility maps form a regional map showing potential for liquefaction. The latter map delineates areas where there ia a high likelihood that liquefiable sediments might be present and where there is a relatively short return period between seismic events capable of causing liquefaction. Such maps are useful for preliminary planning purposes in identifying areas where liquefaction and ground failure could pose a substantial hazard and where additional site-specific investigations may be needed to evaluate site safety and to make land-use decisions.

Site-Specific Investigations

Analyses of field test data from sites that have and have not liquefied during earthquakes have led to the development of empirical engineering criteria for evaluating liquefaction susceptibility. The most commonly used procedure and criteria at the present time were developed by Seed and others (1983). In this procedure, calculated cyclic stresses generated in the ground at a particular site for an expected earthquake are compared with the amplitude and duration of stresses that would be required to cause liquefaction in soil layers beneath the site. The key parameters used to calculate the cyclic stresses generated by the earthquake are peak acceleration and earthquake magnitude. The soil property commonly used to index stresses required to cause liquefaction is modified penetration resistance, N_1, a property calculated from the results of a field test called the "standard penetration test." Curves used to evaluate liquefaction susceptibility using modified penetration resistance are plotted on Figure 15.

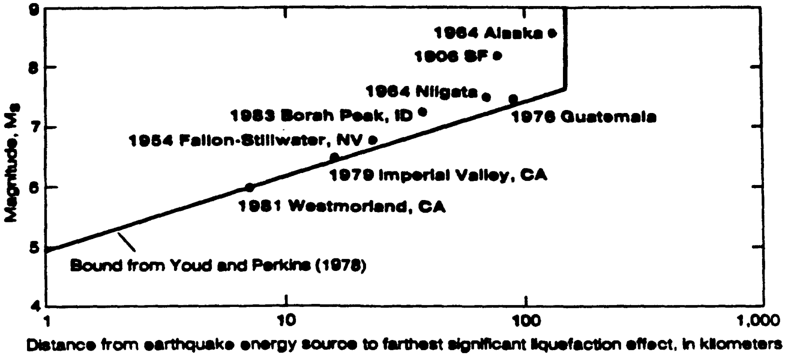


Figure 13.--Data and relation developed by Youd and Perkins (1978) to determine distance from the earthquake energy source to the farthest distance significant liquefaction effects are expected to occur.

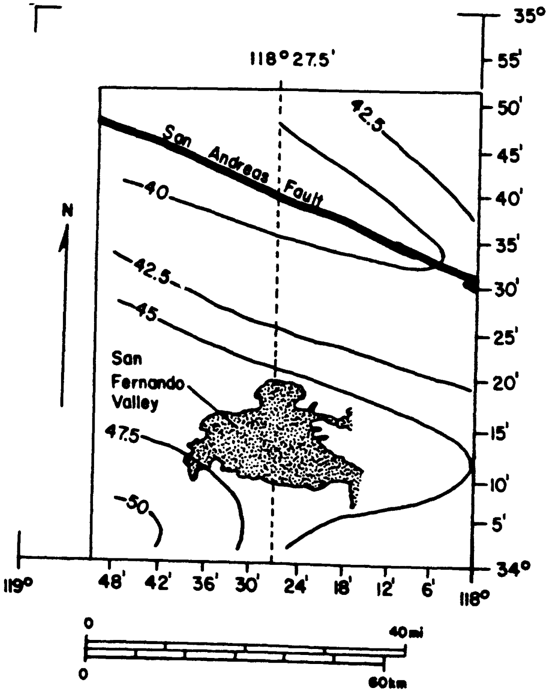


Figure 14.--Liquefaction opportunity map for the San Fernando Valley showing expected return period for earthquake shaking strong enough to cause liquefaction in highly susceptible soil layers. (After Youd and others, 1978)

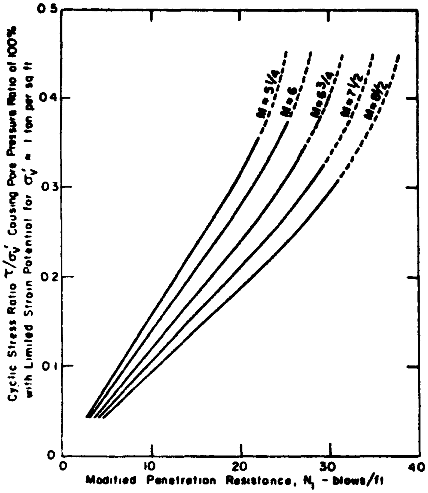


Figure 15.—Chart for evaluating liquefaction susceptibility for different magnitude earthquakes (after Seed and others, 1983).

Because of severe criticisms that have been made against the standard penetration test, other field tests are being developed to evaluate soil resistance to liquefaction. One test that is coming into common use is the cone penetration test. This test has advantages of a simpler field procedure which reduces operator error, and measures a continuous record of penetration with depth, whereas the standard penetration test provides only intermittent values. Considerable research is underway to develop correlations to evaluate liquefaction susceptibility directly from cone-penetration resistance records. Some other soil properties, such as shear-wave velocity and dilatancy response, show promise of usefulness for correlating with liquefaction resistance. Studies are also being made using these parameters.

Another published technique for evaluating liquefaction susceptibility is to take soil samples from the field to the laboratory and perform cyclic-load tests which simulate earthquake loading. Procedures for conducting and analysing these tests are outlined by Seed (1979). The expense and great difficulty of taking, transporting, and preparing soil samples for laboratory testing without causing disturbance to the specimen that invalidates test results, however, has proved to be a major problem. Consequently, laboratory liquefaction tests on soil samples presently are used primarily for research investigations and for high-cost engineering studies for critical facilities, such as nuclear power plants.

MITIGATION OF LIQUEFACTION HAZARDS

Four general approaches to mitigation of earthquake hazards (avoidance, prevention, engineered design, and post-earthquake repairs) apply to the mitigation of liquefaction hazards.

Avoidance

A prime way to limit damage due to liquefaction is to avoid areas susceptible to liquefaction. This approach is not always possible because some facilities such as transportation routes, irrigation canals, pipelines, etc., commonly must cross susceptible areas. Also past development may have already occurred in hazardous areas, eliminating the alternative of avoiding the site. Some communities in the U.S.A. are taking some actions, however, to avoid hazardous areas. These efforts generally incorporate zoning regulations based on regional maps of liquefaction potential of the type described above. The regulations require site-specific engineering investigations to evaluate liquefaction hazards in areas marked by high liquefaction potential on the regional maps. Follow-up steps are required if a liquefaction hazard is discovered; these steps require either diacarding of the site or incorporation of engineered design measures to prevent damage. Emergency planning in some communities also requires alternate or redundant water mains and transportation routes to bypass areas of high liquefaction potential.

Prevention

Where the ground can be stabilized to prevent liquefaction, the hazard can be avoided. Several techniques are available and have been applied in a few instances to stabilize hazardous sites. In general, these techniques are very expensive to apply and some techniques do not work well in silty soils. Because of these deterrents, stabilization procedures have not been widely used. The common techniques are: 1) removal and replacement of the susceptible soil with nonliquefiable material; 2) dynamic compaction of the soil inplace by repeatedly dropping a heavy weight (tens of tons) on the ground surface; 3) dynamic compaction by pushing a vibrating probe into the soil; 4) compaction by setting off explosions to vibrate the soil; 5) driving relatively large diameter piles into the soil at close spacings to compact the soil; 6) injecting a cement or chemical grout into the liquefiable layer to bond the soil particles together, 7) dewatering the susceptible layer with the use of gravity or pumped drains; 8) placing gravel-filled columns into the liquefiable layer to provide vents to the ground surface for relieving water pressures as they develop during the liquefaction process; and 9) constructing buttresses to prevent ground-failure displacements should liquefaction occur.

Accommodation Through Engineering Design

In some situations structures can be designed to resist damage due to liquefaction. A common example is buildings founded on piles that bear in soil strata below all liquefiable layers. This measure has worked well in flat areas where liquefaction has not been accompanied by lateral displacement of the soil. For example, buildings in Niigata, Japan, founded on deep piles suffered little damage during the 1964 earthquake, whereas many nearby buildings, founded on shallow footings or shallow piles settled and tipped (Youd, 1980). Structures that can not be easily designed to resist ground failure include canals and pipelines.

Post-Earthquake Repairs

It may not be possible to avoid, prevent, or design against liquefaction in all instances. Where these deterrent measures are not applied, post-earthquake repairs and emergency responses may be the only alternatives. Studies of damage during past earthquakes show that damage as a consequence of liquefaction can be severe and disruptive to essential community facilitites such as pipelines and transportation routes. An important aspect of emergency response following these earthquakes has been the ability to quickly repair essential facilities, by having necessary equipment and materials readily available. Where repairs were made quickly, greater disruption and damage was commonly avoided. For example, damaged irrigation canals were repaired in time to deliver water to avoid major crop damage following the 1979 Imperial Valley, California earthquake. On the other hand, if repairs are not quickly made, additional damage and suffering may be severe. For example, about 85 percent of the damage to San Francisco following the 1906 earthquake was caused by fire. Severance of pipe lines in zones of ground failure, prevented delivery of water to the fire and greatly inhibited efforts to contain the conflagration. If repairs to the pipelines could have been made more quickly, a considerable amount of fire damage might have been averted.

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APPLICATION OF SEISMIC HAZARD EVALUATIONS TO BUILDING CODES

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The application of seismic hazard evaluation to building codes has progressed significantly since the first United States seismic building code was developed in about 1927. Hazard evaluations are used to determine the seismic zoning for all areas in the United States and in some other parts of the world. Also they are used on a more micro scale for specific important, essential, or potentially hazardous facilities. The following text discusses the use of hazard evaluation and the principles of seismic design including:

- Facility Criteria
- Construction Materials
- Design Considerations - Structural
- Design Considerations - Nonstructural
- Design Considerations - Lifelines
- ATC-3 Tentative Provisions

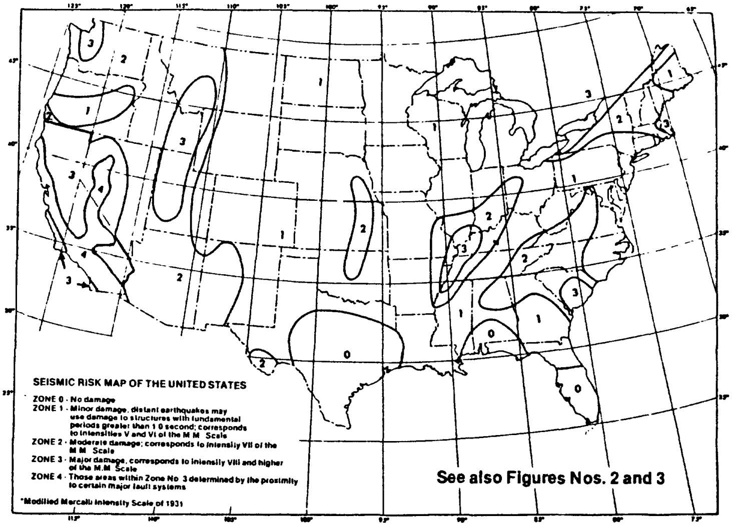
FACILITY CRITERIA

When hazard evaluations are used to determine design criteria for a specific facility, the site characteristics, and the site seismicity should be assessed. When selecting a new site, an investigation should be made to determine the existence of any active earthquake faults, and the potential for landslides, liquefaction, or consolidation of foundation soils when subjected to vibratory ground motion. Large differences in elevation should also be studied. Soils-foundation reports should be reviewed with the assistance of a qualified professional to determine the possible liquefaction potential of underlying soils during a seismic event. Detailed field investigations might be necessary to determine the existence and extent of any of the above such possible hazards. The extent to which such investigations are employed will depend on the size and importance of the proposed facility, probability of safety related or large economic risk, and the possibility of or suspicion of potential site hazards.

The site seismicity should be evaluated including a study of earthquake history (the occurrence of earthquakes) and earthquake damage history. The magnitude of earthquakes that have or might occur should be studied together with the maximum ground acceleration that might occur. The location of faults and possible fault breakage should be studied together with the duration of potential shaking.

For a normal usage facility it may be appropriate to select design coefficients based upon the seismic zoning shown in the building code that is applicable to the particular region. A facility that must be able to function during and after a major earthquake, or only be functional after a major event, should have the effective peak ground motions that might occur at the site, and for which the facility should be designed, determined as part of the study. For most sites the possible peak ground motions could be several times greater than the building code coefficient. Thus the amplitude of motions used in design should depend on the degree of risk acceptable for the facility. After the amplitude of the ground motion is determined, response spectra or acceleration time histories should be developed considering the soil and foundation conditions at the site and the distances to potential earthquake sources.

On a macro scale seismic hazard evaluations are used to develop maps such as those that are currently used in the Uniform Building Code or those that have been published with the Applied Technology Council "Tentative Provisions for the Development of Seismic Regulations for Buildings", which is often referred to as ATC3-06. The ATC-3 provisions will be discussed later in this paper. The seismic risk map from the Uniform Building Code is shown in Figure 1 and a seismic contour map from the ATC-3 publication is shown in Figure 2. After the design criteria for the facility is determined, either specific for the site or that from the Building Code, the next factor to consider is the construction materials.



SEISMIC RISK MAP OF THE UNITED STATES

ZONE 0 - No damage

ZONE 1 - Minor damage, distant earthquakes may use damage to structures with fundamental periods greater than 1 0 second; corresponds to intensities V and VI of the M M Scale

ZONE 2 - Moderate damage; corresponds to intensity VII of the M M Scale

ZONE 3 - Major damage, corresponds to intensity VIII and higher of the M.M Scale

ZONE 4 - Those areas within Zone No 3 determined by the proximity to certain major fault systems

*Modified Mercalli Intensity Scale of 1931

See also Figures Nos. 2 and 3

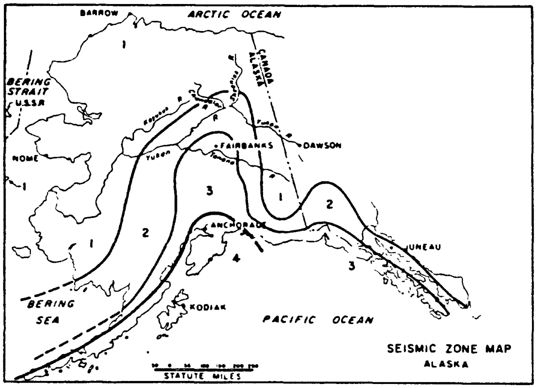


FIGURE NO. 2

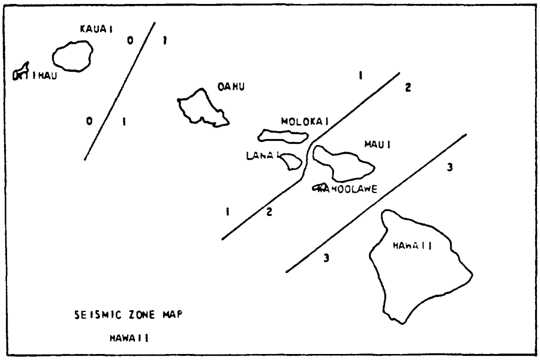
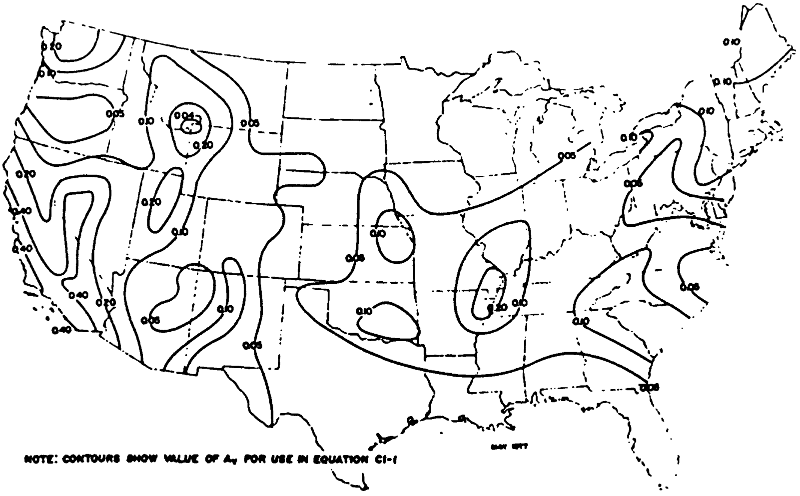


FIGURE NO. 3

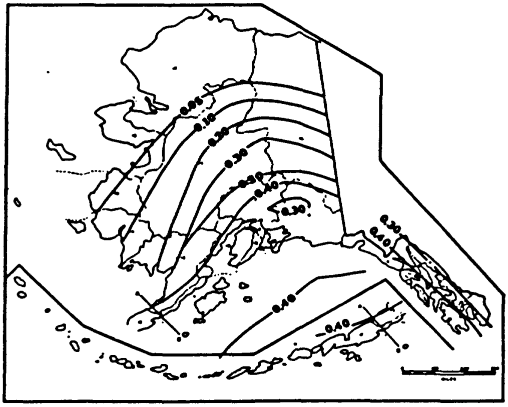
FIG. 1 SEISMIC RISK MAP OF UNITED STATES (FROM UBC 1982 EDITION)



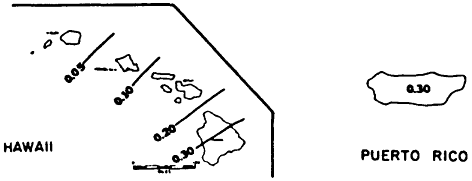
NOTE: CONTOURS SHOW VALUE OF A_v FOR USE IN EQUATION C1-1

MAY 1977

CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT



ALASKA



HAWAII

PUERTO RICO

CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT

FIG. 2 SEISMIC RISK MAP OF UNITED STATES (FROM ATC-3-06, 1978 EDITION)

CONSTRUCTION MATERIALS

In the United States there are four basic construction materials used for most buildings and other structures. They are structural steel, reinforced concrete, reinforced masonry, and wood or timber. Each of these materials have unique characteristics that make them applicable for different types of facilities. The type of structural framing used for a building or a structure has a major effect on how the building will react when subjected to earthquake motions. A frame structure made up of vertical columns and horizontal beams and girders is usually the most flexible and tends to dissipate the earthquake energy by deforming and bending or flexure. If a frame structure has the capability to remain stable when subjected to major earthquake motions then a lower total seismic force coefficient can be used in its design. The reason being that a flexible structure generally has lower inertial forces than a stiff structure.

On the other hand, a structure that has a number of solid walls (called shear walls) will normally have larger induced inertia forces during an earthquake and, as a consequence, a higher total seismic force coefficient is used in the design. The shear wall structure, however, has advantages over the frame structure in that the various components of the building will not move or deform with respect to each other as much as they will in a frame. As a consequence, interior partitions, ceilings, stairways, elevator shafts, and similar items may not need to be specially designed to resist the motions of the building during an earthquake.

DESIGN CONSIDERATIONS-STRUCTURAL

The structural design considerations to be considered are listed in Figure 3. One of the most important factors is that a building having a symmetrical layout in plan generally exhibits better structural performance during an earthquake than one that is very irregular. It is also important that a structure be relatively regular vertically. It is essential that all components of the building be tied together so that they do not separate during an earthquake.

DESIGN CONSIDERATIONS - STRUCTURAL

- Basic layout - symmetry is important
- Avoid re-entrant corners or similar locations of stress
- Consider effects of nonstructural elements - partitions, windows, and filler walls
- Relative stiffness of building stories
- Consider moment frames versus shear wall or frame/shear wall construction
  - Moment frames deform more but induce lower seismic forces
  - Shear walls deform less but have higher seismic forces
  - Moment frames exhibit better ductility than shear walls
  - Dual framing - moment frame/shear wall often good solution
- Tie all components together - beams to supports, walls to floors and roofs
- Consider torsional effects
- Provide sufficient space between separate parts of building to avoid impact

FIGURE 3

Figure 4 summarizes the characteristics of structural steel framing for structures. As noted previously, moment frames have lower induced seismic forces but higher lateral deformations which could cause damage to the nonstructural elements in a building. However, structural steel can be braced by either concentric or eccentric bracing. The bracing reduces the lateral deformation of a structure, but then the structure has to be designed for a higher seismic force.

Figure 5 summarizes the characteristics of reinforced concrete buildings. Properly designed reinforced concrete buildings exhibit good ductility, that is, the ability to withstand earthquake motions without collapse. However, the concrete and its reinforcing steel should be designed so that the concrete does not fail in shear, the reinforcing steel does not pull out of the concrete or fail in anchorage, or that the concrete does not fail in compression or crushing. Concrete can be used for stiff structures, such as those designed with shear walls, or for moment frames combined with shear walls. This latter type of framing has proven to be quite popular in many areas of the world.

Figure 6 lists factors to be considered when designing a building with masonry. It has been found in observing the aftermath of a number of major earthquakes that properly designed masonry generally exhibits good performance during an earthquake. Masonry is generally most effective against earthquake motions when the building is constructed with shear walls to resist the earthquake forces and is fairly symmetrical in plan. It is important that the steel reinforcement be placed properly and that the workmanship used in constructing the masonry be of good quality as should the mortar and the blocks or brick. All of the walls should be securely anchored or connected to the building framing, floors and roof.

Figure 7 summarizes the characteristics of wood structures. Wood is normally used for structures three stories or less. In the U.S. it is used for residential dwellings, both apartments and homes, and also for warehouses, small industrial facilities and commercial buildings.

STRUCTURAL STEEL

- **Excellent stress-strain characteristics**
- **Moment steel frames**
  - **Lower seismic forces**
  - **Higher lateral deformation can cause nonstructural damage**
- **Braced steel frames**
  - **Higher seismic forces**
  - **Lower lateral deformation**

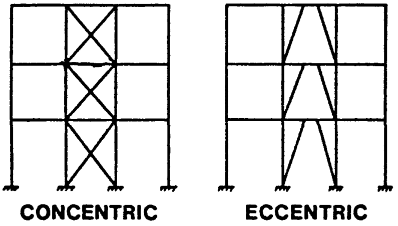


FIGURE 4

REINFORCED CONCRETE

- **Properly designed R.C. exhibits good ductility**
- **Design to avoid:**
  - **Shear failure**
  - **Anchorage failure**
  - **Compression failure**
- **R.C. moment frames - design for ductility**
  - **Confine longitudinal reinforcement at joints in columns and beams**
- **Shear wall structures**
  - **Boundary members**
  - **Anchor to framing**
- **Moment frame/shear wall structures**

FIGURE 5

REINFORCED MASONRY

- Properly designed masonry exhibits good performance
- Shear wall-type construction
  - Provide vertical and horizontal reinforcement
  - Fully grout vertical cells with reinforcement
  - Embed horizontal steel in mortar joint or bond beam
  - Workmanship extremely important
  - Ensure quality materials (mortar and block or brick)
  - Anchor walls to building framing, floors and roof

FIGURE 6

WOOD STRUCTURES

- Wood structures generally perform well during earthquakes
- Wood has good inherent energy dissipation characteristics
- Important that components are well connected together
- Usually designed as shear wall structures for seismic

FIGURE 7

Structural framing systems can be stiffened by using cross-bracing or shear walls to reduce the deformations induced by seismic motions. For such systems, the nonstructural components are less likely to be damaged. However, braced framing has much less ductility than moment framing. Bracing members act primarily in tension and therefore, when failure occurs, it is somewhat abrupt. Unlike a moment frame system, a brace system has little capability to dissipate energy by continued deformation. A recent development that is gaining favor in the U.S. is the eccentric-braced steel frame which does allow some ductile deformation, see Fig. 4.

The general design philosophy for earthquake building codes until quite recently have been that buildings should be designed to suffer no damage during minor earthquakes; no structural damage but some nonstructural damage during moderate earthquakes; and no building collapse but some damage during a major earthquake which would be the maximum earthquake that you would expect to occur in a given region.

It is essential that occupants be able to leave a damaged building and that rescuers be able to enter. In case of a major earthquake, certain essential facilities should be functional during and immediately after the event. These include emergency command centers, communication centers, hospitals, ambulance facilities, and similar. There is also increasing concern that damage to nonstructural components and systems can cause injury to people. In addition, their cost of repair can often greatly exceed the cost of structural repair.

DESIGN CONSIDERATIONS - NONSTRUCTURAL

Nonstructural design considerations are summarized in Figure 8. It is important when considering the design of nonstructural elements that the deformation of the building frame be considered. As most nonstructural elements have limited capacity to resist earthquake motions, they are generally rather easily damaged if the building frame should deform excessively. To minimize the possibility of injury to occupants, the partitions and ceilings should have seismic restraints as should the air ducts, fans, boilers, elevator equipment, and light fixtures. During an earthquake, the structure and its contents move, therefore good details should be used - either fully connect the nonstructural to the structural frame or fully separate the two elements. Keep in mind that a falling ceiling or light fixture can severely injure an occupant or in some cases cause fatalities.

As part of the nonstructural, the lifelines systems should be considered. These are summarized in Figure 9. Lifelines include the means of ingress and egress from a building, and the critical electrical and mechanical systems necessary to allow movements such as elevators. In addition; external systems such as utilities, roads, and bridges should be designed to be in adequate condition after an earthquake so that water can be supplied for firefighting, and the roads and bridges provide access for emergency rescue crews to tend to the injured occupants, and/or to provide fire protection.

ATC - 3-06 TENTATIVE PROVISIONS

The ATC-3-06 tentative provisions were first conceived in 1970 when the Structural Engineers Association of California recommended that earthquake design codes for buildings should be reviewed and updated on a national basis. The National Science Foundation and the National Bureau of Standards sponsored efforts by the Applied Technology Council to conduct such a study. The ATC assembled a group of 85 participants from throughout the U.S. including researchers, practicing engineers, building officials, government agency representatives, and building code promulgating organizations. The ATC-3 provisions philosophy is summarized in Figure 10. The design steps to be followed using the provisions are summarized in Figure 11. The coefficients Aa and Av and the seismicity indices for the U.S. are listed in Figure 12. Figure 13 lists the seismic performance categories. The categories vary from A through D with Category A requiring nominal earthquake design and Category D requiring detailed design. The seismic hazard exposure groups are based on the function of the building. Group III are essential facilities, Group II are buildings with high occupancies or restriction of occupants, retail stores, hotels, and similar facilities and Group I are all other facilities.

DESIGN CONSIDERATIONS - NONSTRUCTURAL

- Consider whether nonstructural will be overstressed by deformation of building frame - If so provide connections to minimize or design nonstructural to resist induced forces or displacements
- Design connections of precast concrete or other nonstructural element to withstand differential building deformations
- Nonstructural elements
  - Provide seismic restraints for partitions and ceilings
  - Provide seismic restraints for air ducts, fans, boilers, elevator equipment, light fixtures, etc.

FIGURE 8

DESIGN CONSIDERATIONS - LIFELINES

- Lifelines include
  - Means of ingress and egress - stairs, corridors, elevators
  - Critical electrical and mechanical systems
  - External systems - utilities, roads, bridges, and fire protection
- Design should include
  - Consideration of means for evacuation from building
  - Possibility of fire from short circuits, broken pipe lines, etc.
  - Need for communications after major earthquake
  - Need for access of emergency equipment, ambulances, fire trucks, or other equipment

FIGURE 9

ATC-3-06
PROVISIONS PHILOSOPHY

- Design emphasized over analysis
- Design for minimum life risk
- Structural and nonstructural components included
- Performance type specifications where possible
- Provisions should not inhibit ingenuity

FIGURE 10

DESIGN STEPS

- Locate site on map
- Determine map area number
- Table 1-B
  - Select coefficients A_a and A_v
  - Select seismicity index
- Table 1-A
  - Select seismic performance category
- Review design requirements
  - Select soil profile
  - Select framing system
  - Design category (Table 1-A)
  - Is building requiar or irregular
  - Select analysis procedure
- Analysis and design
  - No design
  - Equivalent lateral force
  - Modal analysis

FIGURE 11 - ATC DESIGN STEPS

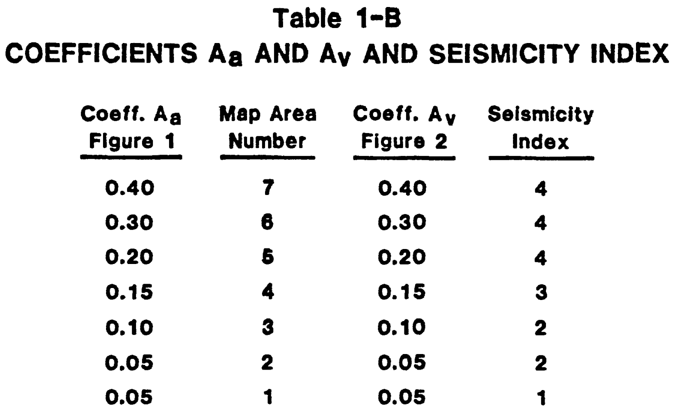


Table 1-B
COEFFICIENTS A_a AND A_v AND SEISMICITY INDEX

Coeff. A_a Figure 1	Map Area Number	Coeff. A_v Figure 2	Seismicity Index
0.40	7	0.40	4
0.30	6	0.30	4
0.20	5	0.20	4
0.15	4	0.15	3
0.10	3	0.10	2
0.05	2	0.05	2
0.05	1	0.05	1

FIGURE 12 - TABLE 1-B OF ATC-3-06

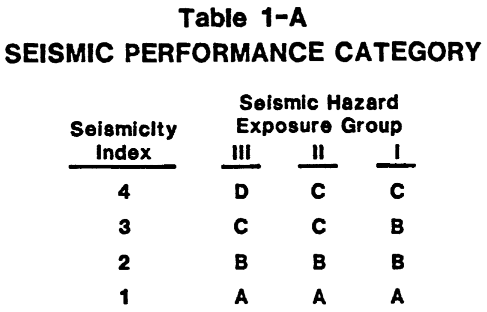


Table 1-A
SEISMIC PERFORMANCE CATEGORY

Seismicity Index	Seismic Hazard Exposure Group III	II	I
4	D	C	C
3	C	C	B
2	B	B	B
1	A	A	A

FIGURE 13 - TABLE 1-A OF ATC-3-06

The ATC-3 provisions utilize response modification coefficients which are based on the type of structural system including the type of vertical seismic resisting system. Figure 14 summarizes the major factors to be considered in building configuration. As noted previously, regular or symmetrical buildings perform much better in earthquakes than those that are irregular. Figure 15 summarizes the analysis procedures that are followed for the different seismic performance categories A to D.

The ATC-3 provisions also consider the effect of the underlying or supporting soils on the response of a building during an earthquake. Figure 16 summarizes the structural design requirements and notes the soil factors S1 through S3 which consider the site effects. Figure 17 shows the seismic design base shear formula, for the UBC and ATC-3-06. Cs is the percent of gravity load (weight of building) that is applied as a horizontal load on the structure. Figure 18 outlines the determination of the seismic design coefficient Cs and the building period T or Ta to be used in the Cs formula.

The ATC provisions require design of nonstructural elements for earthquake forces. This includes ceilings, partitions, mechanical and electrical systems in the building.

The ATC provisions took about four years to develop, from 1974 to 1978. They have been under review by various groups in the U.S. for the past several years and have been used wholly or in part by a number of large industrial companies in the U.S. and by some developers of building code provisions. The U.S. Federal Emergency Management Agency plans to issue the ATC-3 provisions, as modified by a 3-year study, for use by federal government agencies in the design of buildings for earthquake forces.

SUMMARY

The preceding text has provided a brief overview of the use of seismic hazard evaluations in building code provisions. There are a number of factors to consider when using risk evaluations in developing building code provisions and in designing buildings and structures for earthquake motions as follows:

BUILDING CONFIGURATION

- Plan configuration - building irregular when any of following occurs
  - Not symmetrical or has significant re-entry corners
  - Significant eccentricity between seismic resisting system and mass at any level
  - Diaphragm at any level has significant changes in strength or stiffness
- Vertical configuration - building irregular when any of following occurs
  - Not symmetrical or has significant horizontal offsets
  - Mass-stiffness ratios between adjacent stories vary significantly

FIGURE 14 - ATC-3-06 CONFIGURATION

ANALYSIS PROCEDURES

- Seismic performance category A
  - No overall seismic analysis
- Seismic performance category B
  - Equivalent lateral force (ELF) procedure
- Seismic performance categories C and D
  - Regular buildings - ELF procedure as minimum
  - Irregular buildings - special consideration of dynamic characteristics (for vertical irregularities only - can use modal analysis given)

FIGURE 15 - ATC-3-06 ANALYSIS

STRUCTURAL DESIGN REQUIREMENTS

- Internal forces using linearly elastic model
- Limit deflections
- Provide continuous load path or paths with adequate strength and stiffness to transfer all forces to final resisting point
- Site effects
  - Soil profile types

    S_1 - rock or over 2500 fps shear wave velocity or stiff soil under 200 ft. deep

    S_2 - deep cohesionless or stiff clay or soil depth over 200 ft.

    S_3 - soft-to-medium clays and sands over 30 ft. deep
  - Site profile coefficient S

	S_1	S_2	S_3
S	1.0	1.2	1.5

  - Soil-structure interaction

FIGURE 16 - ATC-3-06 STRUCTURAL DESIGN

EARTHQUAKE LOADS

$V = KI(ZCS)W$

- Z = zone factor (seismic risk)
- C = response factor (period)
- S = soil factor
- W = weight of structure
- K = system factor (ductility)

SEISMIC BASE SHEAR - UBC

SEISMIC BASE SHEAR

$V = C_s W$

where

C_s = seismic design coefficient
W = total gravity load including partitions and permanent equipment, 25% of storage and warehouse floor loads, and effective snow load

SEISMIC BASE SHEAR - ATC-3-06

FIGURE 17 - SEISMIC BASE SHEAR

- The initial ground motions developed in the risk evaluations normally must be modified. Normally it is not feasible, economically, to design for zero damage. Therefore the building code forces have implicitly assumed that there will be some damage in case of a major or moderately severe earthquake.

- Therefore, the forces to which a building or structure may be subjected during an earthquake may be much greater than those specified in the Building Code and used in the structural design of the building.

- It is most important to prevent collapse of the building or the dislodging and falling of structural members or of nonstructural members such as ceilings, partitions, and mechanical and electrical equipment.

- Wherever feasible, it is desirable to make the building symmetrical or regular, both in plan and in elevation.

- It is important that the structural framing and the nonstructural elements be well tied together. However, the relative deformation or deflections of the structural frame and the nonstructural elements that might occur must be considered in the design details.

- It is essential that the structural framing have the capability to withstand relatively large lateral deformations and still remain stable and not collapse.

It is important in the design of buildings that the architect, structural engineer, and the mechanical and electrical engineers work together as a team so as to provide an integrated structure that considers the potential effects of earthquakes on all elements. By working as a team, a cost efficient and adequately strong structure can be developed.

PROBLEMS OF EARTHQUAKE RISK ASSESSMENT

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ABSTRACT

Objective data on earthquake risk to buildings and structures are needed for the purpose of physical and economic planning, insurance and civil defense planning. A method of deriving such data from the analysis of damgae caused by recent earthquakes is described and is illustrated by examples of recent experience in the Balkans.

INTRODUCTION

In the process of physical planning for economic development, decisions have to be made on the location of new settlements, factories, powerplants, etc., and of all the technical infrastructure that goes with them. In most cases, these investments have to be integrated into an already existing urban fabric, whose vulnerability to earthquakes must be taken into account. For instance, a new factory or chemical plant may be designed and built to resist seismic ground motion of the greatest intensity likely to occur at the site, but it will still be at risk if the houses in which its workers live, or its supply system, are liable to damage or destruction in case of earthquake.

It goes without saying that no viable system of insurance against earthquake losses can function without reliable information and data on the seismic risk, if not to each individual building, at least to the various types of buildings to be insured in any given locality.

Finally, for planning civil defense measures to deal with eventual earthquake emergencies, it is useful to prepare "scenarios" of possible events. To do this, one has to make an inventory of buildings and of the people living or working in them, and to foresee the amount of damage, loss of life and injury that may occur in earthquakes of various intensities in the area concerned.

For all these purposes, data are needed on the vulnerabilty of the various types of existing buildings to seismic ground motion, and these data can be obtained from the careful observation and analysis of the damage actually suffered by such buildings in recent earthquakes. During the past few years, the systematic recording and analysis of earthquake damage has been practiced in the Balkan countries, and this has provided a unique data base for the study of vulnerabilty.

Before proceeding further, it may be wise to define some of the terms that will be used in this paper. The following definitions are based on those adopted by UNDRO (1979).

DEFINITIONS

When an earthquake occurs in a settled area, some buildings may be damaged. The degree of damage suffered by each building may be determined by inspection and expressed on an agreed scale running, say, from 0 (no damage) to a 1 (total collapse). In economic terms, the loss associated with the damage may be expressed as the cost of repair or replacement of the building and its contents (although in practice the building may not be restored to its original state): in some cases, consequential losses (loss of life, injury, loss of production, etc.) may also be taken into account. The specific or proportional loss will be the loss expressed as a fraction of the value of the building at the time of the earthquake. The relation between degree of damage and specific loss will depend on the nature of the building, the materials used in its construction and on other economic factors which will vary from place to place and from country to country.

The degree of damage to any individual building will be determined by two mutually independent factors: 1) the intensity of the ground motion at the site; and 2) the vulnerability of the building to the ground motion of that intensity.

The vulnerabilty of a building or other structure is determined by its physical and dynamical characteristics and is independent of location. It may be represented by a curve or mathematical function relating degree of damage to ground motion intensity.

The concept of intensity is easy to define but difficult to measure, because of the complexity of seismic ground motion and the impossibility of representing it by the value of any single physical parameter. Nor is the response of structures to ground motion determined by the values of any one parameter of this motion. For the purposes of risk analysis, one is therefore, for the time being at least, obliged to fall back on the concept of intensity as "destructive power," measured by reference to observed damage and expressed as degrees on one or other of the commonly-used macroseismic scales. The snag is, that if both the vulnerabilty of a building and the intensity of ground motion are derived from observations of damage, they can no longer be regarded as independent variables. We shall see how this difficulty has been partially resolved in the analysis of the damage caused by the earthquake of 4 March 1977 in Bucharest.

The term specific risk denotes the probability distribution (or exceedance) function of specific loss to a given building (or type of building) at a given site during a given period of time; it is the product of the convolution of the vulnerability of the building and the seismic hazard at the site, the latter term being defined as the probability distribtuion (or exceedance) function of ground motion intensity at the site during the given period of time.

Specific risk applies to a given structure or type of structure at a given site. If one is concerned with a whole city or settlement, in which there are buildings of various types and where the seismic hazard may vary from place to place due to a variety of soil conditions (as revealed by microzoning), it is necessary to introduce the notion of risk, defined as the probability of loss in a given period of time, and obtained by multiplying the specific risk to each structure by the value of the structure. Risk, as distinct from specific risk, is an additive function.

Finally, it may be worth while making clear the distinction between seismic hazard and seismicity. The latter term denotes the seismic activity in a given area and is usually expressed by a magnitude-frequency function for earthquakes in the area; it is not directly related to ground motion.

INVENTORY AND CLASSIFICATION OF ELEMENTS AT RISK

Whether it be for risk assessment or for civil defense preparedness planning, one must start by making an inventory of the buildings and other structures in the area concerned and by classifying them according to structural type and material. For economic purposes one must also assign a value to each building, while for civil defense planning one must also compile a consensus of the people living or working in each building. This is a laborious but straightforward task if there already exists a cadastral survey and property register, as in many European countries. Where no such cadastre exists, a very considerable effort is needed to compile such information, but it may nevertheless appear justified in areas of high seismic risk.

As an example, we may take the French cadastre, which was first elaborated during the 19th Century, mainly for the purposes of taxation. The whole country is surveyed and mapped on a scale ranging from 1:2,000 in rural areas to 1:500 in urban areas; 540,000 map sheets cover the area of 546,000 km^2. The maps show some natural features such as rivers, the limits of individual properties and the location and horizontal plan of all buildings and structures; they do not show contours of height. An example is shown in Figure 1.

Each building or other structure is classified according to usage (and each floor in the case of multistory buildings), and a value assigned for taxation according to the type and quality of construction. All the relevant information is collected and stored on microfiches (see Fig. 2) for use by the administration at the level of the municipality ("commune"), county ("department)" or the country as a whole. The whole system will shortly be computerized (system MAJIC 2).

The establishment and maintenance of such a cadastre involves considerable effort, as mentioned above. In France, the cadastre employs a permanent staff of 9,000 including 1,600 qualified surveyors, grouped in some 300 area offices. Ten percent of the map sheets are revised each year, to take account of change of ownership, new construction, etc. It may be difficult to justify expending such effort merely for the purpose of reducing seismic risk, but it may be argued that it also serves to establish property rights, to provide an inventory of capital investment and to lay the basis for a fair taxation system.

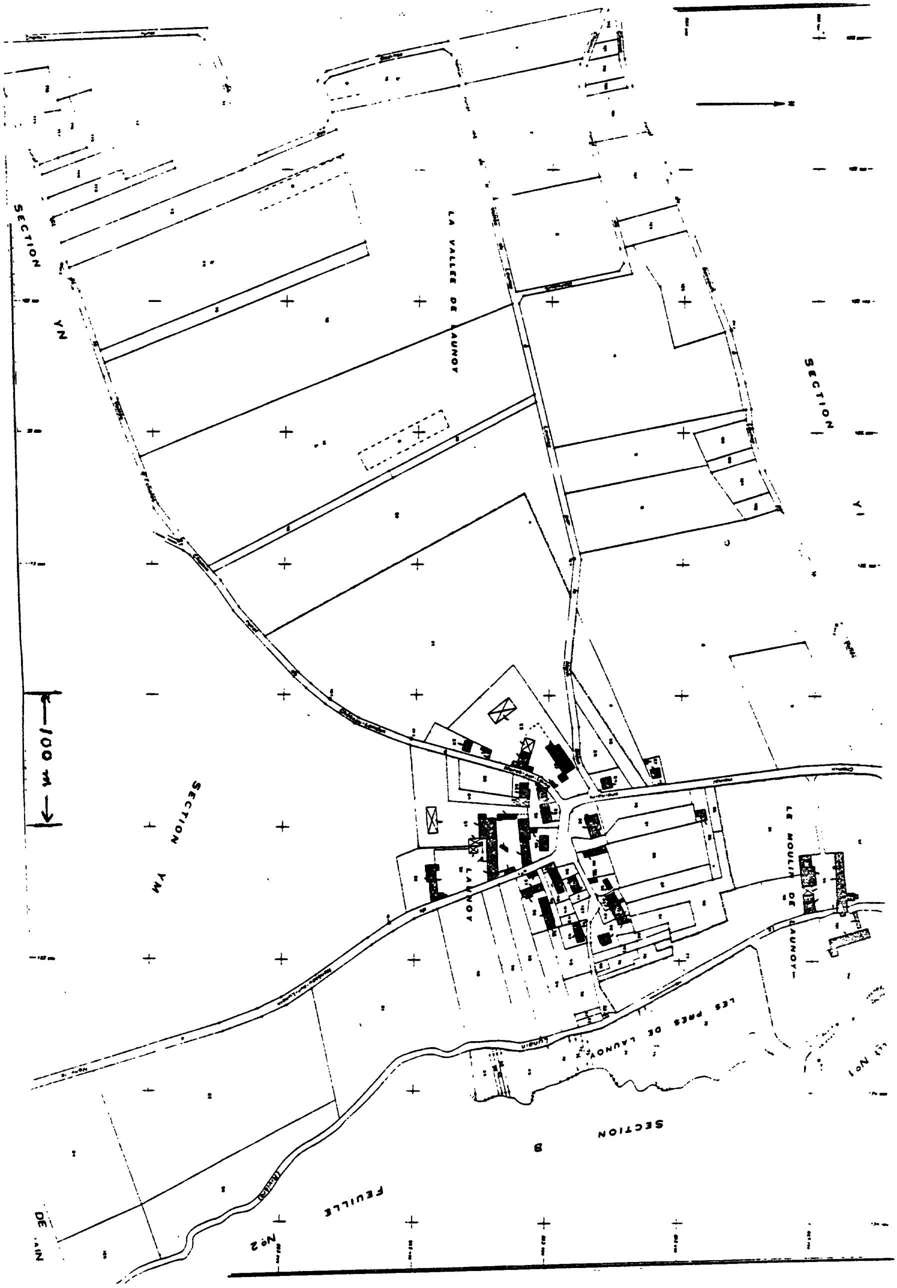
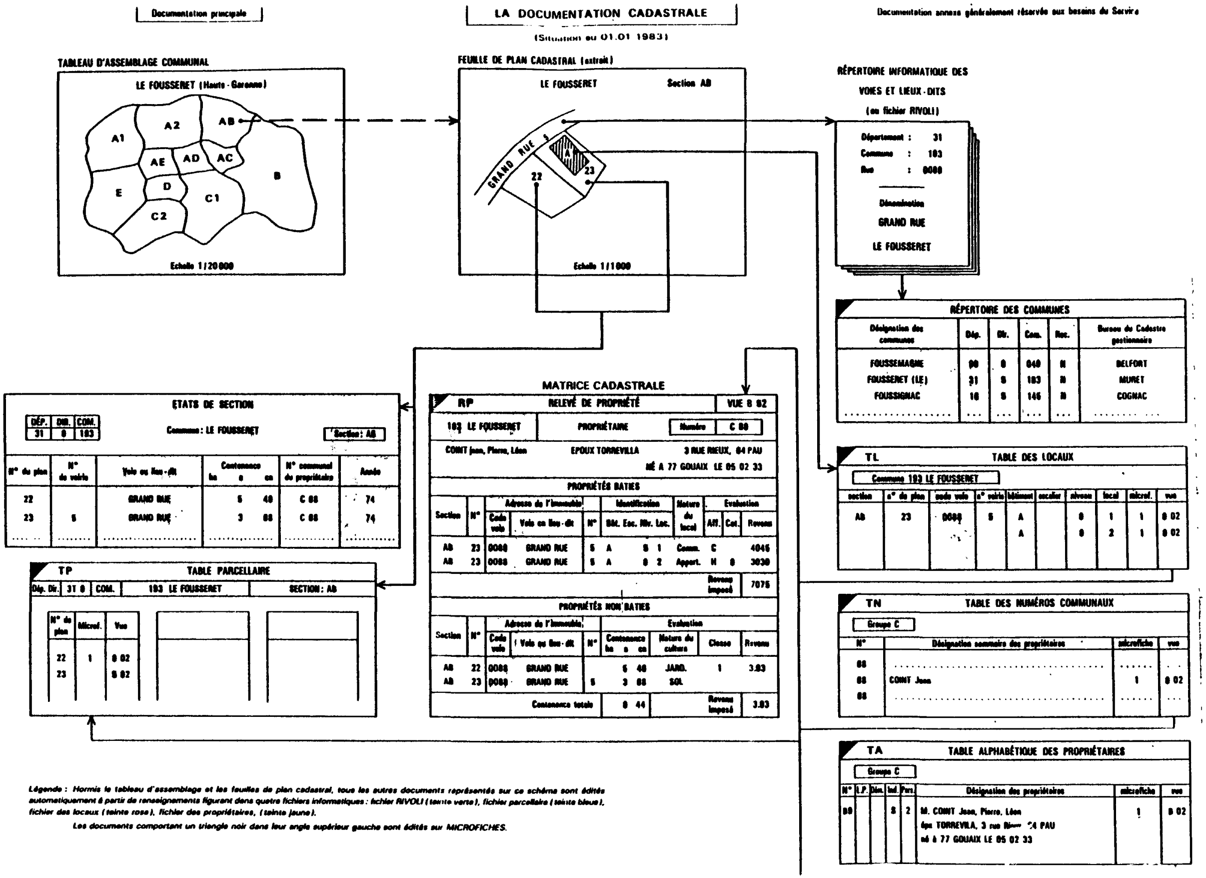


Figure 1. Example of French Cadastral map.



Légende : Hormis le tableau d'assemblage et les feuilles de plan cadastral, tous les autres documents représentés sur ce schéma sont édités automatiquement à partir de renseignements figurant dans quatre fichiers informatiques : fichier RIVOLI (teinte verte), fichier parcellaire (teinte bleue), fichier des locaux (teinte rose), fichier des propriétaires, (teinte jaune).

Les documents comportant un triangle noir dans leur angle supérieur gauche sont édités sur MICROFICHES.

Figure 2.--Cadastral documentation of a French municipality ("commune").

Once an inventory of buildings has been compiled, the next step is to classify them according to structural type and construction materials and to their expected dynamic characcteristics, with the expectation that all buildings in each category will have similar vulnerabilty to seismic ground motion. As an example, Table 1 shows the classification adopted for Bucharest (Romania) for the analysis of the damage caused by the earthquake of 4 March 1977. Further details of this classification are given in the report of Working Group B of UNDP/UNESCO project RER/79/014 (Earthquake Risk Reduction in the Balkan Region).

TABLE 1

Category	Subcategory
1. Low quality (adobe-like)	-
2. Masonry (stone or brick) bearing walls and flexible floors	2.1. Pre-1940 2.2. Post-1940
3. Masonry bearing walls with regid (R.C.) floors	3.1. Pre-1940 3.2. Post-1940
4. R.C. frame structures	-
5. High-rise buildings with R.C. bearing walls	5.1 Closely-spaced walls 5.2 Widely-spaced walls

The division of categories 2 and 3 into subcategories pre-1940 and post 1940 was due to the fact that many buildings in these categoreis had been damaged in the 1940 Bucharest earthquake and subsequently repaired.

For economic risk analysis, in which it is necessary to take into account possible consequential losses due to earthquake damage, it is useful also to classify buildings according to their function or use. The following classification was adopted in Yugoslavia for studies of the damage caused by the earthquake of 15 April 1979:

1. Schools and hospitals
2. Residential buildings (publicly owned)
3. Residential buildings (privately owned)
4. Hotels and other buildings concerned with tourism
5. Industrial buildings, warehouses, and other production facilities

6. Administrative buildings and other important public buildings
7. Infrastructures (e.g., water supply and sewerage systems)
8. Transportation facilities
9. Port structures

Finally, it is necessary to compile a census of the people living or working in each building. The risk (or rather, specific risk) to human life is particularly difficult to assess, since so much depends on the time of day at which an earthquake occurs. While the economic cost of injury may be estimated, that due to loss of life can only be assessed on the basis of criteria of a social and ethical nature which lie outside the scope of this discussion.

THE ASSESSMENT OF DAMAGE

In the case of some recently-constructed, engineered buildings, it may be possible to assess their vulnerability to ground motion of various intensities if data are available from laboratory model tests or from structural analysis. However, the only source of information on the vulnerability of the great majority of buildings will be the analysis of the damage suffered by buildings of similar type during past earthquakes.

The most systematic study yet made of earthquake damage is that carried out under the UNDP/UNESCO project on earthquake risk reduction in the Balkan region, where recent earthquakes have caused extensive damage. In Greece, detailed assessments were made of the damage to 344 buildings in Perahora caused by the earthquakes of 24-25 February 1981; in Romania, of 19,000 buildings in Bucharest damaged by the earthquake of 4 March 1977; and in Yugoslavia, of the damage to some 60,000 buildings in the earthquake of 15 April 1979.

The damage anlaysis was carried out in accordance with the methodology developed by the projects's Working Group on Vulnerability and Seismic Hazard (Chairman: H. Sandi) and set out in the report of this Group, already referred to. To give an example of the methods and of the results obtained, we shall quote from the Romanian contribution to this report.

The buildings in Bucharest damaged by the earthquake of 4 March 1977 were classified into the eight categories and subcategories shown in Table 1.

The damage to each building was assessed on a scale of 0 to 5, according to the criteria set out in Table 2.

TABLE 2: DAMAGE ASSESSMENT

Building Category	Nature of damage	Damage degree
Category 1	Walls:	
Low-quality construction	not affected	0
	slightly affected	1
	cracked	2
	severely cracked	3
	collapsed	5
	Bearing walls:	
	not affected	0
	slightly affected	1
Categories 2&3	cracked	1-3/4
	diagonal cracking	3
Masonry construction	leaning	4
	collapsed	5
	Nonbearing masonry:	
	not affected	0
	cracked	1
	partially collapsed	2
	totally collapsed	3

TABLE 2: DAMAGE ASSESSMENT (CONTINUED)

	Columns:	
	not affected	0
	cracked	2
	severely cracked	4
	crushed concrete	4
	buckled reinforcement	4
Category 4		
	Beams:	
Reinforced concrete frame structues	not affected	0
	slightly affected	1
	cracked	2
	severely cracked	3
	failed	4
	Infill masonry:	
	not affected	0
	boundary cracks	1
	cracked	1-1/2
	severely cracked	2
	dislocation	2-1/2
	collapse	3
	Bearing walls:	
Category 5	not affected	0
	slightly affected	1
	cracked	2
High-rise buildings with RC bearing walls	severely cracked	3
	failed	4
	Columns/infill masonry:	
	as for RC frame structues	

Note: In Categories 2,3,4, and 5, the damage degree for the whole building was taken as the highest of the damage degrees to the components.

THE ASSESSMENT OF INTENSITY

In order to derive vulnerability functions for the various types of buildings, a value must be assigned to the intensity of ground motion at each site. This is difficult to do, for the following reasons:

1) Even when accelerograph records of ground motion have been obtained in or near the buildings, it is not always clear which of the many parameters of motion is the most significant in determining the response of the structue and the eventual damage.

2) Without detailed knowledge of the dynamical properties of the soil and subsoil in the surrounding area, and of the soil structure interaction, it is not safe to assume that the motion affecting a building is identical or even similar in its spectral distribution to that recorded even a few hundred meters away.

3) If no accelerograph record has been obtained at the site, the only way to assign a value to the intensity is to assess the damage to typical buildings in the neighborhood and to translate this into intensity, using one or other of the standard macroseismic scales.

In Bucharest, the earthquake of 4 march 1977 was recorded by three instruments: one accelerograph and one seismoscope in the basement of a single-story building; and one accelerograph at the top of a 10 story building some 2 km away from the first. The records of the latter instrument and of the seismoscope are reproduced in Figure 3.

Because of the scarcity of instrumental records, the intensity at each site in Bucharest was assessed in the following manner: the statistical data on damage degree were interpreted to give intensities on the MSK scale; a least error approach led to the identification of zones of uniform intensity of ground motion and of similar dominant frequency; intensity was expressed for each such zone in degrees and half-degrees on the MSK scale.

From UNESCO project working group B:
national report of Romania.

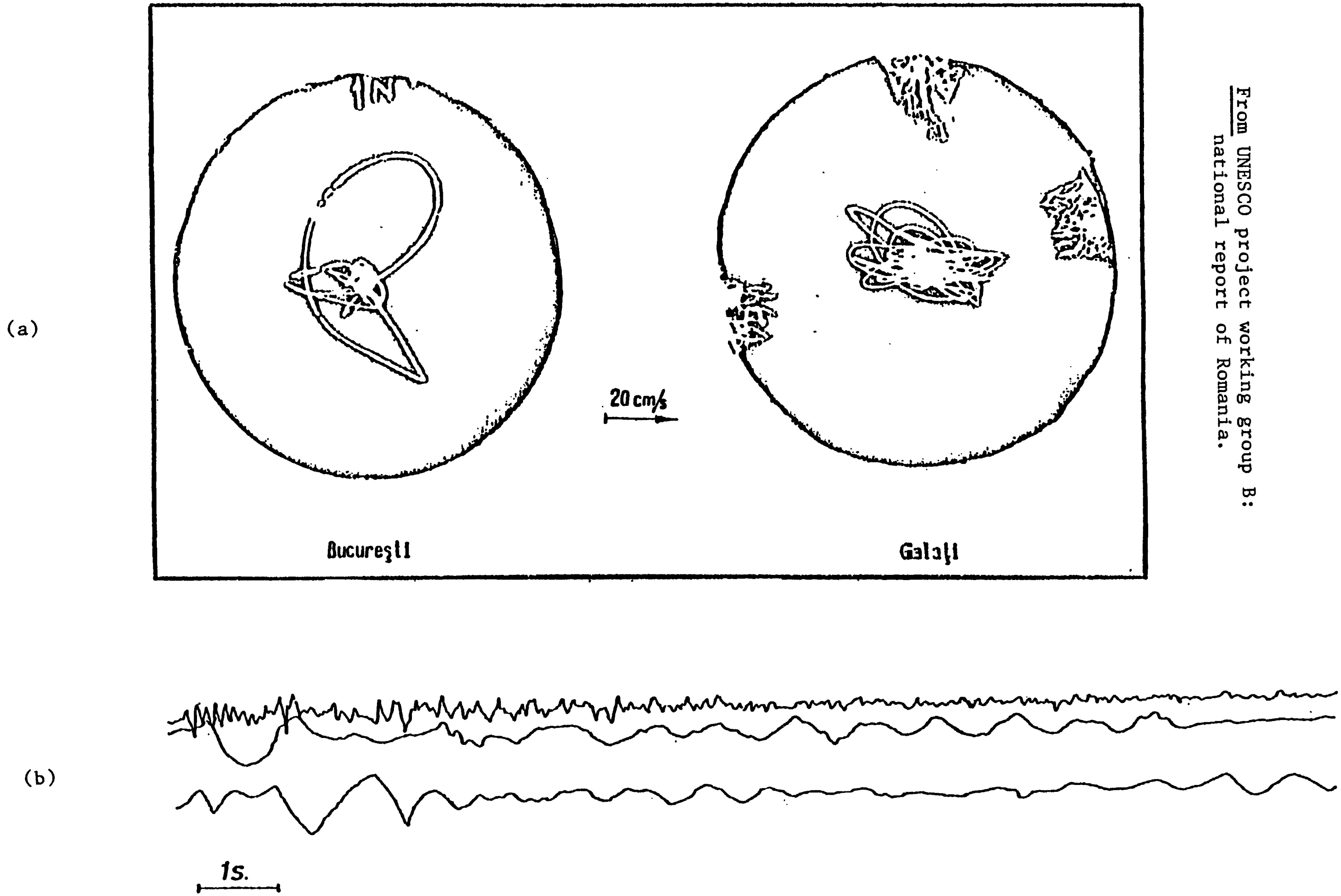


Figure 3.--(a) Seismoscope records from Bucharest and Galati. (b) Buildings with masonry bearing walls and flexible floors built before 1940.

In Yugoslavia, over 300 accelerograph records were obtained of the earthquake of 15 April 1979 and its aftershocks. It should, therefore, be possible in this case to relate the damage suffered by various types of buildings to one or more of the observed parameters of ground motion.

THE ASSESSMENT OF VULNERABILITY

The 19,320 buildings covered by the detailed damage survey in Bucharest were grouped into the eight categories and subcategories shown in Table 1. The degree of damage to each building was assessed using the criteria shown in Table 2.

Figures 4-6, reproduced from the Working Group report referred to above, show the results of the statistical analaysis of damage to three types of buildings: those with masonry bearing walls and flexible floors built before 1940; reinforced concrete frame structures; and high-rise buildings with closely-spaced R.C. bearing walls. On the right-hand side of each figure are shown, for each intensity level, the number (n) of damage buildings examined, the mean damage degree (x) of the buildings subjected to ground motion of that intensity, and the standard deviation (s) of damage degreee; the figures also show, for each intensity level, the fractions of the n building in each intensity zone which suffered each degree of damage.

Inspection of these figures reveals some interesting differences between the vulnerability characteristics of different types of buildings, in the interpretation of which it must be remembered that this was an intermediate-depth earthquake and that an unusually high proportion of the energy was transmitted at frequencies less than 1 Hz (periods greater than 1 sec). High-rise buildings were particularly sensitive to this.

Characteristic of all types of buildings is the wide scatter in the observed degree of damage at all levels of intensity. In all categories, the upper limit of standard deviation of damage degree at intensity VI-1/2 or VII was greater than the lower limit at an intensity 1-1/2 degrees higher (i.e. VIII or VIII-1/2). This wide scatter must be attributed mainly to differences in the dynamic characteristics of individual buildings in the same category, and it seems that further effort is needed to define building types more narrowly.

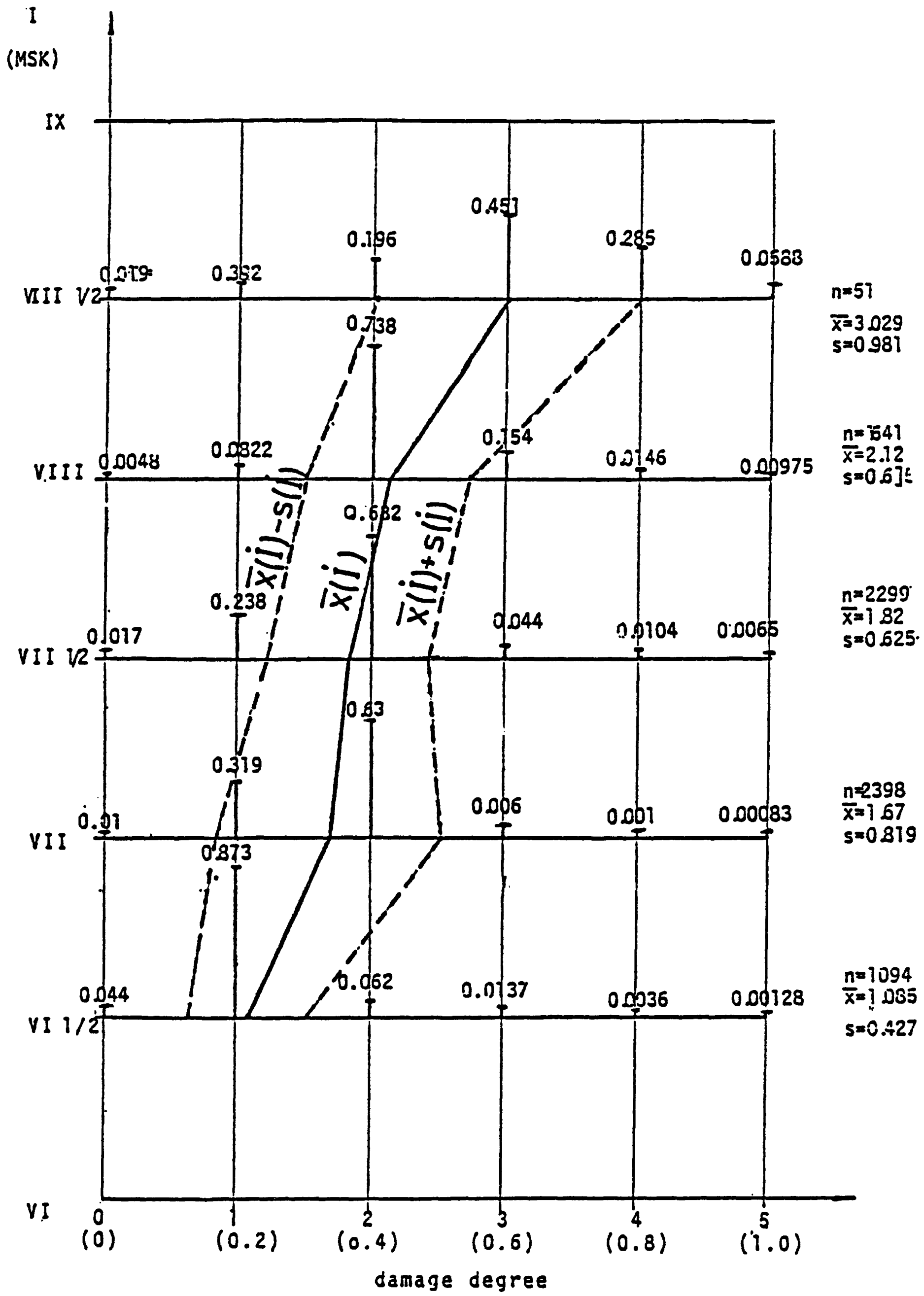


Figure 4.—Buildings with masonry bearing walls and flexible floors built before 1940.

Reinforced concrete frame structures

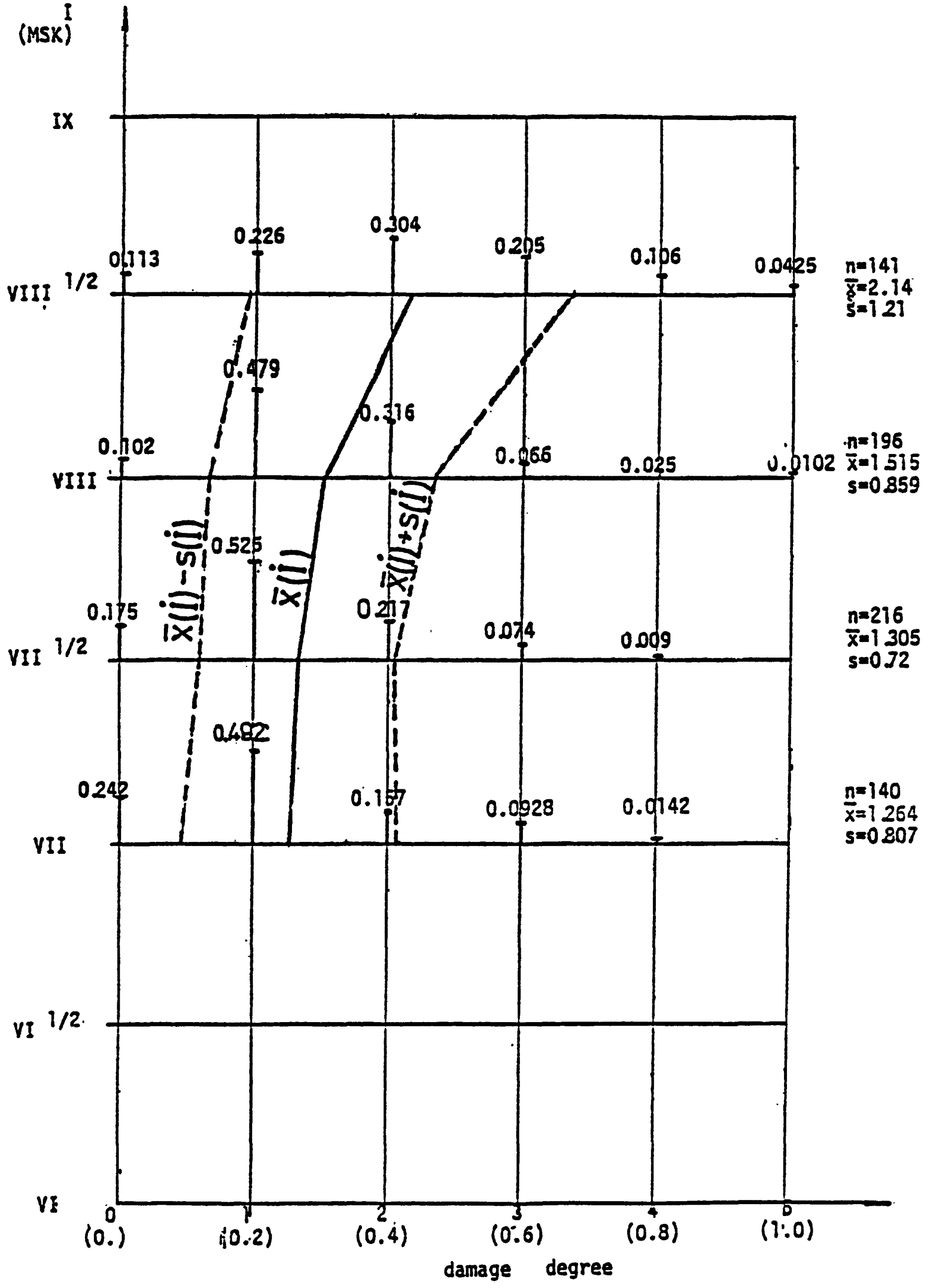
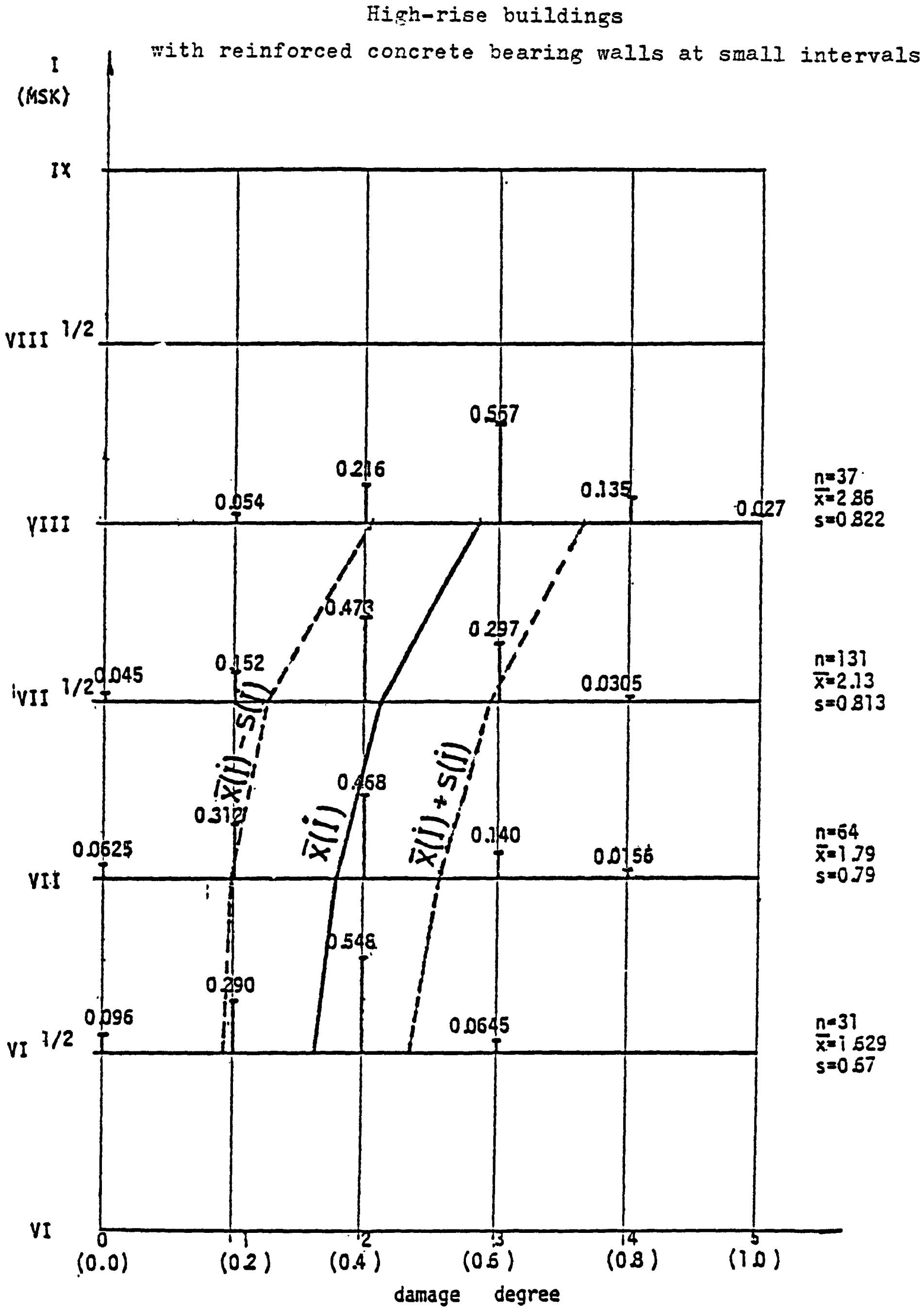


Figure 5.—UNESCO project working group B: National Report of Romania.



High-rise buildings
with reinforced concrete bearing walls at small intervals

Figure 6.—UNESCO project working group B: National report of Romania.

Despite the care taken to derive objective values of ground motion intensity at each site, it is possible that unknown local variations in soil properties over short distances may have influenced the ground motion at particular sites. Soil-structure interaction may also have been significant.

These results reveal some of the difficulties that may arise when one attempts to apply in practice that concepts defined in Section 2 above. However, as will be seen in what follows, they do provide a basis for an objective assessment of the specific risk to buildings in areas liable to be affected by intermediate-depth earthquakes in Romania and elsewhere.

THE ASSESSMENT OF HAZARD

Seismic hazard is defined as the probability of occurrence of seismic ground motion of various intensities at a given site during a given period of time. The units in which hazard is expressed, and the degree of precision required, vary according to the purpose for which this information is needed.

Physical and economic planners have to make, on behalf of the community, decisions on the siting of new settlements and investment projects, on the layout of new towns, etc. Seismic hazard is usually only one of the many factors that they have to take into account, and high precision is not required.

For planning at the regional level, it is sufficient to have maps on a scale of 1:100,000 or thereabouts, showing relative levels of hazard, for instance as intensitites on a macroseismic scale. For detailed urban planning, larger-scale maps are needed, usually on scales of 1:10,000 or even 1:5,000, again showing relative hazard levels on a macroseismic scale. Such maps must of course be based on the results of detailed microzoning surveys.

For earthquake-resistant design and construction, hazard must be expressed in terms of physical parameters of ground motion. Precisely which parameters are the most significant in determinig structural response and therefore most useful for design purposes, remains a subject for debate.

However, for the purposes of seismic risk assessment, such maps are of little use. What is needed is, for each site, the probability distribution of all possible ground motion intensities. Up to the present, seismologists appear to have concentrated on providing data for engineering design and to be wedded to the idea that seismic hazard must be given to the provision of data for risk assessment.

Fortunately, the national report of Romania contained in the report of the UNDP/UNESCO project's Working Group A (Seismology, seismotectonics, seismic hazard and earthquake prediction) includes estimates of the annual probabilites (Pi) of occurrence of seismic ground motion of various intensities for Bucharest. They are reproduced in the following Table, together with the corresponding values of Pi for half-degree intervals of intensity.

TABLE 3. SEISMIC HAZARD IN BUCHAREST

Intensity (MSK)	VI	VI-1/2	VII	VII-1/2	VIII	VIII-1/2	IX
P_i (Radu et al.)	.099		.035		.014		.007
P_i (adopted)	.063	.036	.022	.013	.009	.0055	.0045

NOTE: The value in brackets for half-degree intensitites are interpolated by us.

The figures in the above Table are derived from an analysis of data on past earthquakes for each of which a single value of intensity was adopted for Bucharest as a whole; the value adopted for the earthquake of 4 March 1977 was VIII. However, the more detailed analysis of the damage caused by this latter event appears to indicate that the intensity in various parts of the urban area varied between VI-1/2 and VIII-1/2, presumably because of variations in local soil conditions. If these variations are confirmed by microzoning, then in estimates of the hazard of future earthquakes the intensity figures in the above table should be adjusted as appropriate for each zone.

THE ASSESSMENT OF SPECIFIC RISK

To illustrate the way in which estimates of specific risk may be derived from observed vulnerability, we may take as an example the reinforced-concrete frame structures in Bucharest (Figure 5). The method of analysis is similar to that proposed by Sandi (1982).

We have to assume that the 693 buildings in this category which were surveyed after the earthquake of 4 March 1977 are typical of all such buildings in Bucharest, in so far as their vulnerability to seismic ground motion is concerned. If this assumption is correct, than the observed distribtuion of damage degree at each intensity level, as shown in Figure 6, may be taken as representing the probability distribtuion of damage degree in any future event of that intensity. We may then read off from Figure 5 the values of p_{id}, that is to say the probability of damage of degree D = d to any building of this category in the event of ground motion of intensity I = i at the site.

Denoting by p_i the annual probability of occurrence of ground motion of intensity i, the annual probability P_d of occurrence of damage degree D = d due to all possible events will be:

$$p_d = \sum_{i = o}^{i = max} p_i \cdot p_{id}$$

Table 4.1 shows the computed values of $p_i \cdot p_{id}$ for reinforced concrete structures in Bucharest within the zone of intensity I = VIII during the earthquake of 4 March 1977.

TABLE 4.1

ANNUAL PROBABILITY OF DAMAGE IN ZONE OF I = VIII ON 4 MARCH 1977

Intensity	p_i	$p_i \cdot p_{id}$				
		d = 1	d = 2	d = 3	d = 4	d = 5
VII	.022	0.0108	0.0035	0.0020	0.0003	-
VII-1/2	.013	0.0069	0.0028	0.0010	0.0001	-
VIII	.009	0.0039	0.0026	0.0005	0.0002	0.0001
VIII -1/2	.0055	0.0013	0.0017	0.0012	0.0006	0.0002
P_d		0.0229	0.0106	0.0047	0.0012	0.0003
Cumulative ($D \geq d$)		0.0397	0.0168	0.0062	0.0015	0.0003

The survey of damage carried out in Bucharest after the earthquake of 4 March 1977 indicated that the macroseismic intensity ranged from XX-1/2 VI-1/2 to VIII-1/2 in the urban area, due persumably to differences in local soil conditions. If we assume that soil factors will influence the pattern of intensity in the same way in future earthquakes (and this may be a valid assumption since Bucharest is likely to be affected only by events at the same Vrancea focus), we may calculate the probability of damage to reinforced concrete frame structures in these other zones. The results are set out in the following Tables.

TABLE 4.2

ANNUAL PROBABILITY OF DAMAGE IN ZONE OF I = VII-1/2 ON 4 MARCH 1977

Intensity	p_i	$p_i \cdot p_{id}$				
		d = 1	d = 2	d = 3	d = 4	d = 5
VII	.036	0.0173	0.0056	0.0032	0.0005	-
VII-1/2	.022	0.0115	0.0047	0.0017	0.0002	-
VIII	.013	0.0059	0.0039	0.0008	0.0003	0.0002
VIII-1/2	.009	0.0018	0.0024	0.0017	0.0008	0.0003
P_d		0.0365	0.0166	0.0074	0.0018	0.0005
Cumulative ($D \geq d$)		0.0628	0.0263	0.0097	0.0023	0.0005

TABLE 4.3

ANNUAL PROBABILITY OF DAMAGE IN ZONE OF I = VII-1/2 ON 4 MARCH 1977

Intensity	p_i	$p_i \cdot p_{id}$				
		d = 1	d = 2	d = 3	d = 4	d = 5
VII	.013	0.0065	0.0021	0.0012	0.0002	-
VII-1/2	.009	0.0046	0.0019	0.0007	0.0001	-
VIII	.0055	0.0028	0.0019	0.0004	0.0001	0.0001
VIII-1/2	.0045	0.0009	0.0012	0.0008	0.0004	0.0001
P_d		0.0148	0.0071	0.0031	0.0008	0.0002
Cumulative ($D \geq d$)		0.0260	0.0112	0.0041	0.0010	0.0002

TABLE 4.4

ANNUAL PROBABILITY OF DAMAGE IN ZONE OF I = VII ON 4 MARCH 1977

Intensity	p_i	$p_i \cdot p_{id}$				
		d = 1	d = 2	d = 3	d = 4	d = 5
VII	.009	0.0043	0.0014	0.0008	0.0001	-
VII -1/2	.0055	0.0020	0.0008	0.0003	-	-
VIII	.0045	0.0020	0.0013	0.0003	0.0001	0.0001
VIII-1/2	.003	0.0007	0.0009	0.0006	0.0003	0.0001
P_d		0.0090	0.0044	0.0020	0.0005	0.0002
Cummulative ($D \geq d$)		0.0161	0.0071	0.0027	0.0007	0.0002

The question now arises as to what specific loss corresponds to each damage degree. This question can only be answered by the building industry, but we may take it that for damage degree 5 the specific loss will be 100%. For lower degrees of damage, we may for the purpose of this exercise adopt two alternative assumptions:

Assumption A: Specific loss is proportional to damage degree; this is certainly an overestimate.

Assumption B: Specific loss is proportional to the square of the damage degree; this is probably an underestimate.

On the basis of these assumptions we may calculate, from the value of P_d shown in the above Tables, the annual probability $p_{ł}$ of each level of specific loss. The sums of the values of $P_{ł}$ for all damage degrees is a measure of the overall specific risk to buildings of this category in each intensity zone in Bucharest. As an example of this procedure, we may calculate the specific risk to reinforced concrete frame structures in the zone of intensity VIII in the earthquake of 4 March 1977 in Bucharest, on the bases of the above two assumptions.

TABLE 5
SPECIFIC RISK IN R.C. FRAME STRUCTURES IN INTENSITY ZONE I-VIII

Damage degree, d	1	2	3	4	5
Annual probability P_d	0.0229	0.0106	0.0047	0.0012	0.0003
Assumption A:					
Specific loss ł	0.2	0.4	0.6	0.8	1.0
$P_{ł} = ł \times P_d$	0.0046	0.0042	0.0028	0.0010	0.0003
Specific risk $= \Sigma P_{ł}$	0.0129 = 1.29%				
Assumption B:					
Specific loss ł	0.4	0.16	0.36	0.64	1.0
$P_{ł} = ł \times P_d$	0.0009	0.0017	0.0017	0.0008	0.0003
Specific risk $= \Sigma P_{ł}$	0.0054 = 0.54%				

The "specific risk" in the above Table is in fact the average expected annual loss from earthquake action, expressed as a percentage of value. The probability distribution of various specific losses is indicated by the values of P_d.

Using the data from Tables 4.2, 4.3 and 4.4, similar calculations may be made of the specific risk to reinforced-concrete frame structures in the other urban zones where different intensities were observed on 4 March 1977. The results are summarized in Table 6.

TABLE 6

Intensity Zone	Specific risk (expected average annual loss (%))	
	Assumption A	Assumption B
VII	0.53%	0.23%
VII 1/2	0.85%	0.35%
VIII	1.29%	0.54%
VIII 1/2	2.02%	0.84%

We must emphasize that these figures only illustrate the method of assessing specific risk, and cannot be taken as reliable estimates of the actual specific risk to buildings in Bucharest. This is because (a) they are based on data from one earthquake only, (b) it is not known whether the spectral characteristics of the ground motion in this earthquake are typical of the earthquakes affecting Bucharest, (c) the sample of buildings covered by the damage survey may not be typical of existing or future reinforced concrete frame structures in Bucharest, and (d) in the absence of a sufficient number of recordings of the ground motion, the assessments of intensity and of its local variations are necessarily somewhat uncertain.

The method of analysis appears nevertheless to offer good prospects for deriving from systematic damage surveys such as this, objective assessments of specific risk to specified types of buildings.

THE ASSESSMENT OF RISK

Specific risk is not an additive quantity. If one wishes to assess the specific risk to an assemblage of buildings of different types, or in zones of different hazard level, one must first calculate the risk to each building in monetary terms by assigning a value to each building and use this value as a weighting factor in assessing the specific risk to the whole set of buildings.

If one wishes to assess the total seismic risk to a community, one must also take into account the consequential risks of loss of life, injury, loss of production, loss of markets, etc. Further losses may occur due to the secondary effects of earthquakes such as landslides, tsunamis, conflagrations, etc., and as a result of damage to lifelines (roads, railways, power lines, water and sewerage networks, telephone systems, etc.). However, these are beyond the scope of the present lectures.

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EARTHQUAKE HAZARD STUDIES IN SOUTH AMERICA

by

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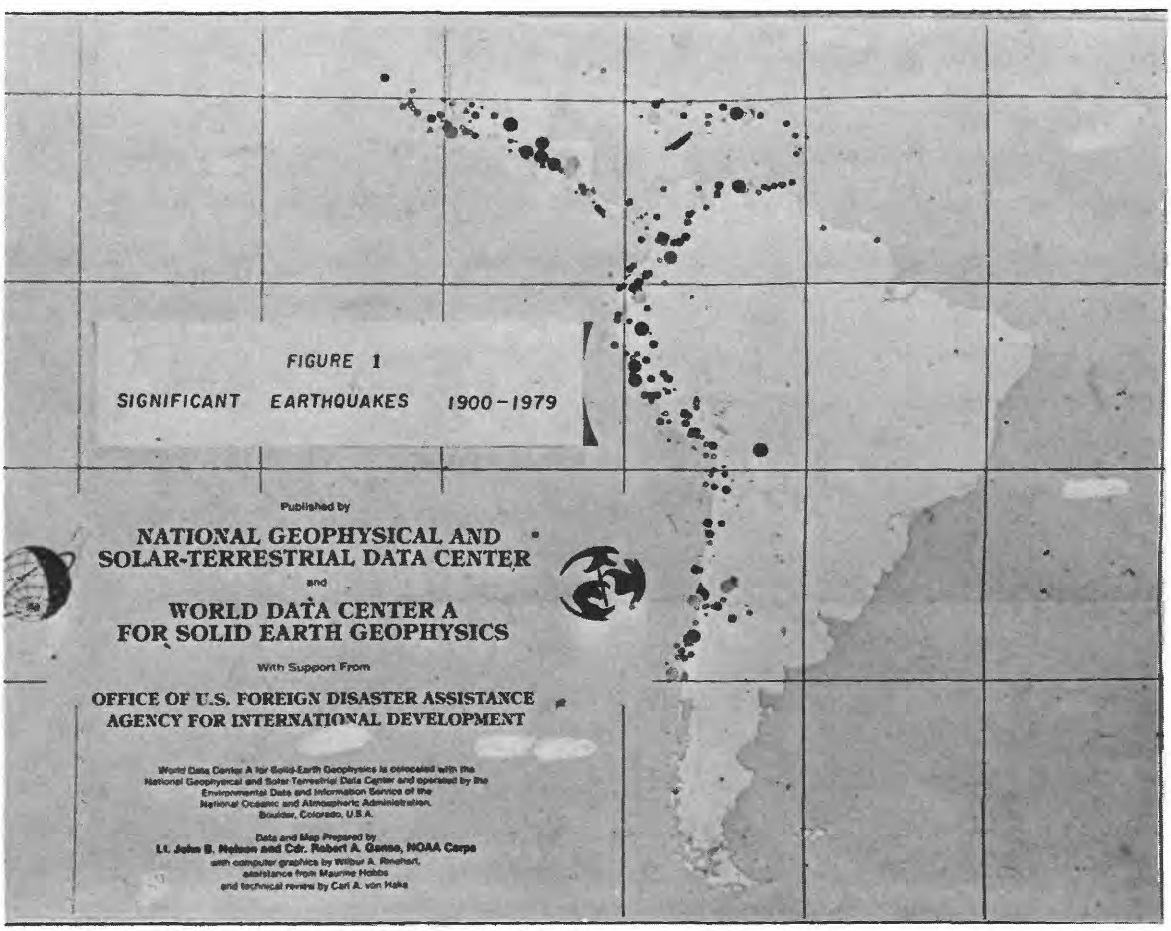
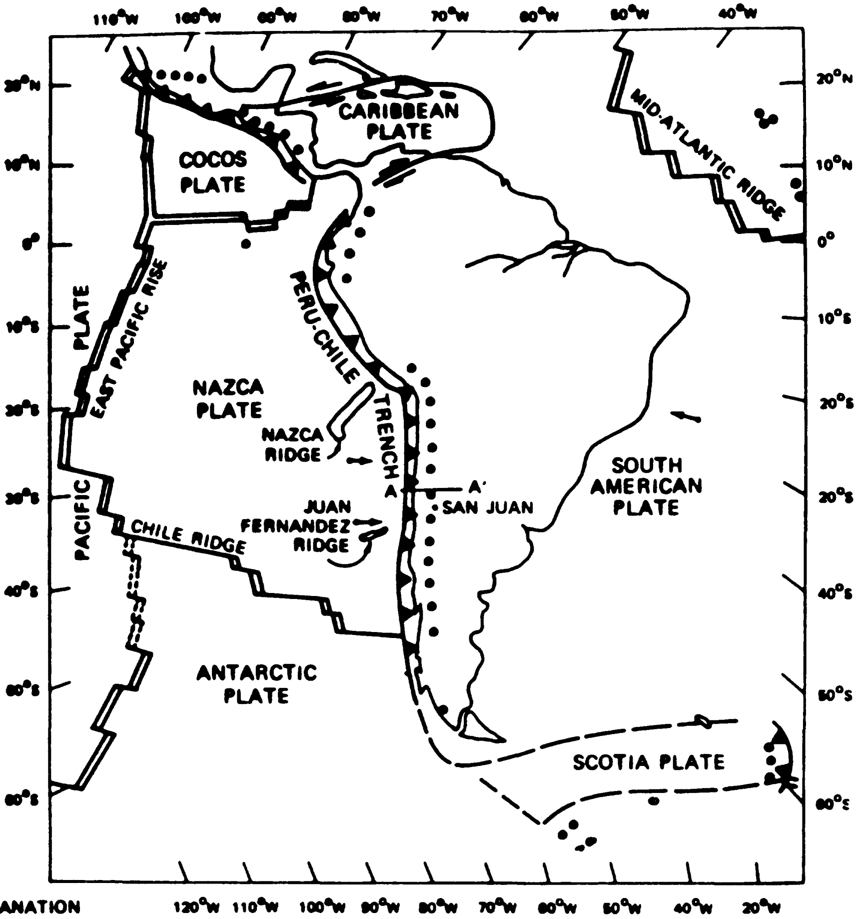
INTRODUCTION

About 15 percent of the world's total seismic energy release corresponds to South America, making this region the most seismically active of the earth. Although most of South America presents some degree of seismic activity, there is no doubt that the Andean region is the one where almost all the activity is concentrated, as can be seen from the map of Figure 1.





EXPLANATION

— — Approximate plate boundary

—— Major strike slip fault, arrows show sense of relative displacement

▲▲ Trench and megathrust zone, sawteeth are on overriding plate

● Location of volcanoes

Spreading center or rise, offset along transform faults

→ Direction of plate motion

NOTES

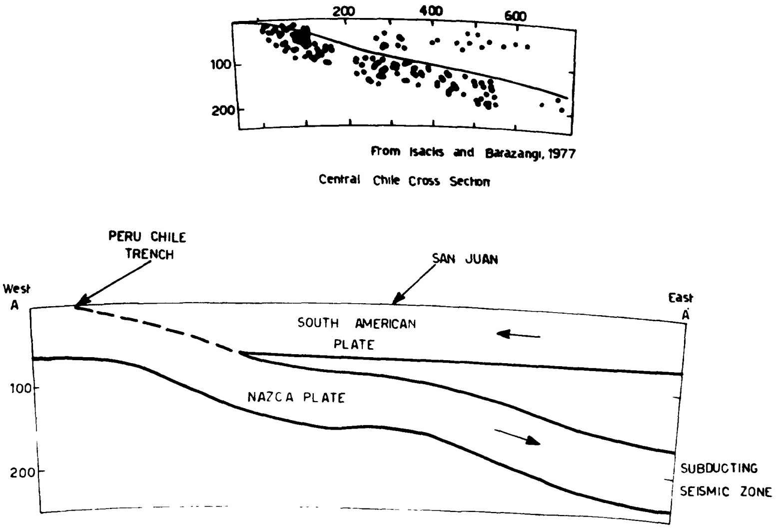
1. Modified after Lowman (1978)
2. Section A - A' is shown in Figure 3
3. Volcano symbol represents one or more volcanoes

0 500 1000 1500 kilometers

FIGURE 2: PLATE TECTONIC SETTING

According to the current plate tectonic concepts, the boundary between the South American plate and the Nazca plate is convergent. The axis of this zone of convergence occurs along the Peru-Chile Trench (Figure 2). Along this boundary the eastward-moving Nazca plate is being subducted beneath the westward-moving South American plate. The rate of convergence in the latitude range 30,-32, south is relatively high, approximately 11 cm/yr. This includes eastward movement of 9.7 cm/yr movement for the Nazca plate and a westward movement of 1.4 cm/yr for the South American plate.

The orientation and dip of the Nazca plate beneath the South American plate appears to be defined by the seismicity of the Benioff zone (Figure 3). This zone of seismicity shows that the subducted plate has an eastward dip that ranges from approximately 5, to 30, beneath the South American plate. Different interpretations are used to explain these changes in dip of the subducting plate.



From Isacks and Barazangi, 1977

Central Chile Cross Section

Figure 3: Plate Tectonic Cross Section in Central Argentina

The South American countries, especially those from the Andean region, have been working on seismological research programs for a long time. This has enabled them to improve the knowledge of seismicity in each country and in the whole region. Besides, since 1966, they gathered in a Regional Seismological Center for South America (CERESIS) which has permitted them a continuous exchange of information and scientific personnel.

With the purpose of developing an overall study on seismic hazards in South America, CERESIS started in 1981 a regional program named "Earthquake Mitigation Program in the Andean Region" (SISRA Project). This program is being performed by an agreement with the U.S. Geological Survey with the financial support of the Agency for International Development (A.I.D.).

EARTHQUAKE MITIGATION PROGRAM IN THE ANDEAN REGION - (SISRA)

The main purpose of this program is to get the best knowledge of the levels of seismic hazard in South America in such a way that each country can take actions to reduce this hazard and mitigate the loss of lives and properties.

In order to accomplish this purpose the program has 3 phases in this first step:

Phase 1:
- a) To complete the Seismic Data Bank for the region.
- b) To obtain a Preliminary Seismotectonic Map with the existing data.
- c) To draw a Map of Maxima Seismic Intensities occurring in the region and a Map of Liquefaction and Landslides produced by historical earthquakes.

Phase 2: To estimate the economic losses of three destructive earthquakes that occurred in the region: the 1967 Caracas, Venezuela earthquake; the 1970 Chimbote, Peru earthquake and the 1977 San Juan, Argentina earthquake, and to obtain a statistical model for the potential probable losses that may occur in the region combining the seismic hazard and the type of existing construction.

Phase 3: To obtain probabilistic regional maps of seismic hazard.

Phase 1 is almost finished and a meeting will be held in September 1984 to present the final results. Phase 2 is scheduled to finish at the end of this year. Phase 3 is now starting.

All the products that are being obtained are the results of the work done in each country by the National Groups under the guidance of a Coordinating Committee.

Seismic Data Bank

The seismic data bank is being implemented with historical and instrumental data. It will consist of two main catalogs: a hypocenters catalog and an intensities catalog.

The historical data have been compiled in a uniform way. There will be summaries of macroseismic effects of major events. All the events will have an intensity table with coordinates of the place where the intensity was estimated. This information will aid to estimate an approximate value of their magnitudes and also to construct attenuation curves.

The hypocentral coordinate data bank will have the hypocentral coordinates and magnitudes for each seismic event in South America. This catalog will be the basic hypocentral data to obtain the probabilistic regional maps of seismic hazard corresponding to phase 3.

Preliminary Seismotectonic Map

The Preliminary Seismotectonic Map has been complete using the existing information with a minimum of new research. It consists in a Plio-Quaternary Tectonic Map (1:5,000,000 scale) showing the following main features:

a) Fractures, faults and lineaments that are thought to have been active during the Plio-Quaternary and/or show a proven or possible relationship with the seismic activity.

b) Volcanic centers which are known to have been active in historical times in the Quaternary.

c) Thermal hot springs of elevated temperature where they are considered to be important.

d) Uplifted and down dropped areas.

Seismic Hazard Maps

In the first phase where the Seismic Data Bank and the Plio-Quaternary Tectonic Map were developed, it was decided to obtain as a first approximation to the zonation of the hazard, a Maxima M.M. Intensities Map for the region, which is presented in Figure 4. This map (1:5,000,000 scale) was the result of the work done in each country according to the following scheme:

a) To restudy the seismic M.M. intensities distribution for each of the most important earthquakes.

b) To consider only the levels of damage reached for each earthquake, disregarding if such damage was produced only due to ground shaking or if, besides, other phenomena such as soil liquefaction or landslides, contributed to the damage.

c) To obtain a maximum intensity map by superimposing the curves obtained for each earthquake without considering the number of times in which this maximum value was reached.

d) To express by means of a point with the corresponding maximum intensity value, those high intensities which are local values.

Having in mind the statement in b), it was necessary to complete the information condensed in the maximum intensity map with that corresponding to other phenomena such as soil liquefaction and landslides. A new map was obtained which is shown in Figure 5.

Probabilistic Maps of Seismic Hazard

The third phase of the SISRA Project will give as final product regional maps for different levels of seismic exposure. These maps will be ready for the end of this year. However, each country has been working on problems related to seismic hazard estimation. Taking as a sample the case of Argentina, the studies

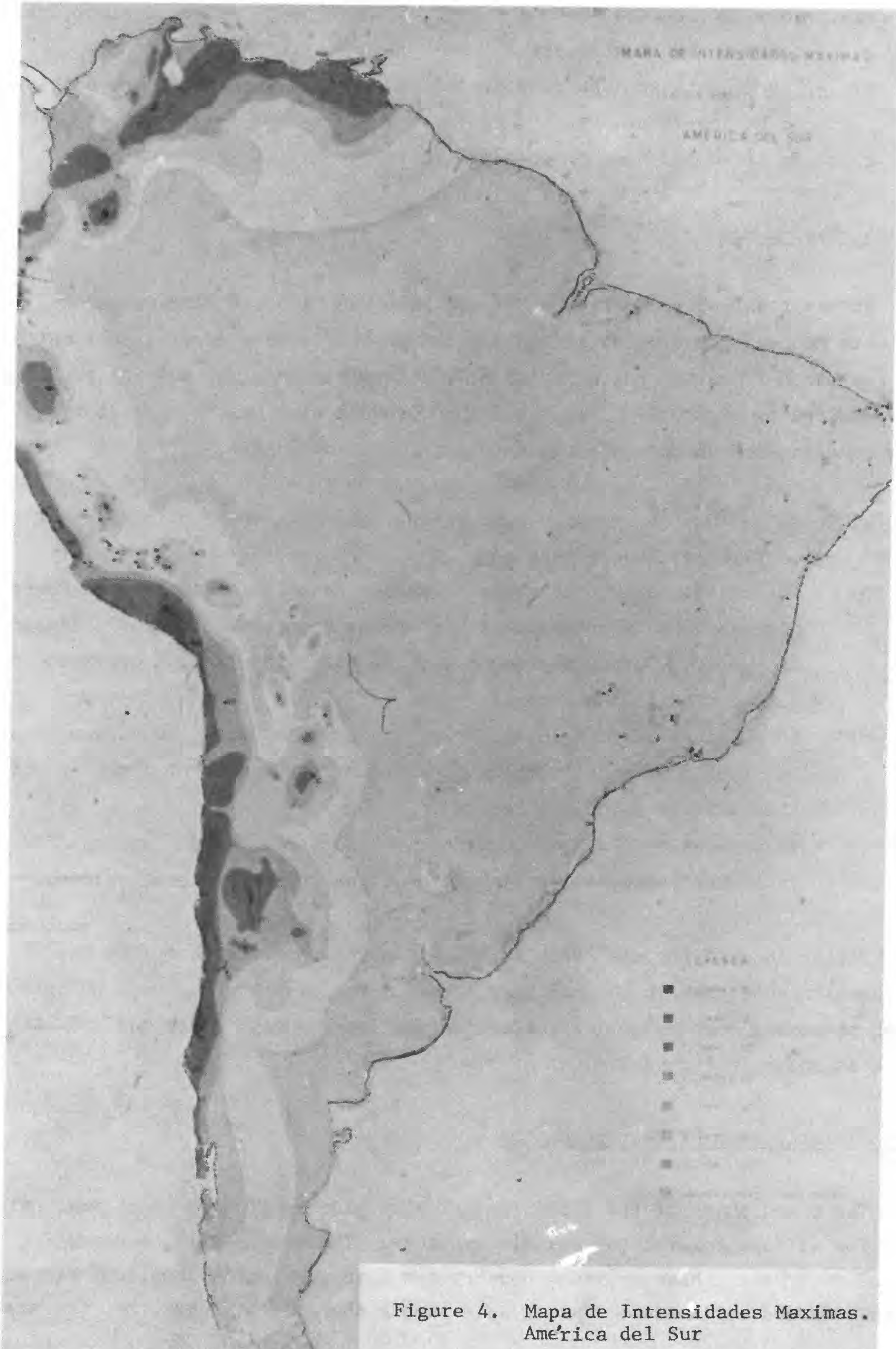


Figure 4. Mapa de Intensidades Maximas. Ame´rica del Sur

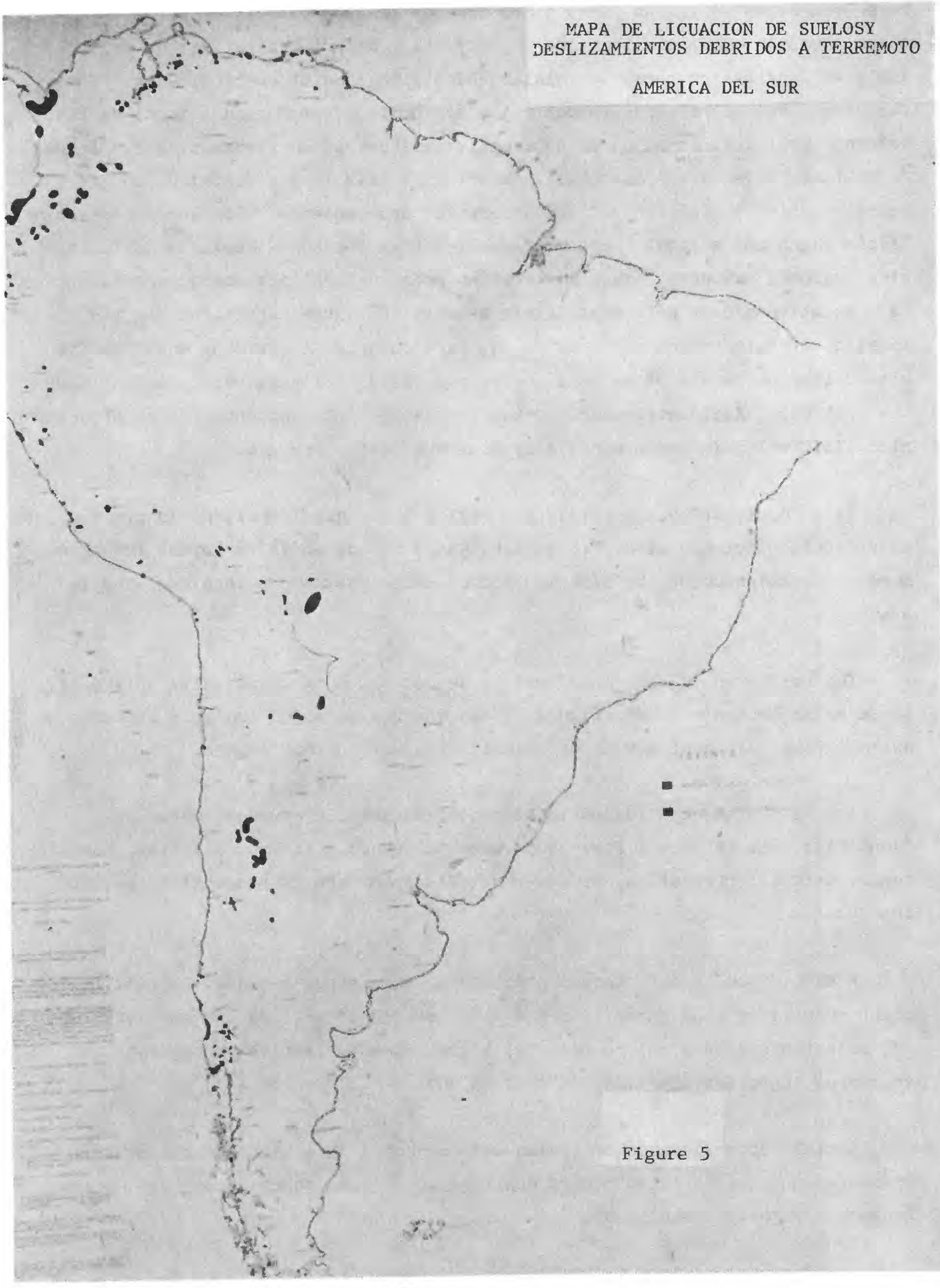

MAPA DE LICUACION DE SUELOSY
DESLIZAMIENTOS DEBRIDOS A TERREMOTO
AMERICA DEL SUR

Figure 5

have been directed in the first place towards the identification of seismotectonic regions within the country. The main faulting systems were analyzed looking for possible relationships with some of the following features: epicentral and hypocentral distributions, predominant focal depths, volcanic activity, location of destructive historical earthquakes, etc. It was found that there is a remarkable lack of uniformity in the distribution of seismic activity (location of epicenters and hypocenters), with areas presenting levels which are strongly higher or smaller than the normal seismic activity. Also regional patterns common to large morphostructural phenomena were identified as a relationship at different levels between the large structures and the location of seismic activity was found, particularly in dividing environments with different levels of seismic activity. The division is very remarkable when the historical destructive earthquakes are taking into account. Large structures also limit volcanic areas especially from the Quaternary age.

As a result of this analysis the region under consideration was divided into eleven seismotectonic areas, (Figure 6) which may be modified in the future when more information about the distribution of seismicity and modern faulting is available.

The next step in this study was to select the most seismically active of those seismotectonic areas (Figure 7) and start a detailed analysis in order to estimate the different levels of seismic exposure in that region.

Active faults, considered to be potential seismic sources, were identified. To get that, five steps were followed: literature review, remotely sensed data interpretation, aerial reconnaissance, ground reconnaissance, and trenching.

Remotely sensed data interpreted during this study included LANDSAT false-color composites (MSS bands 4, 5 and 7) at scales of 1:1,000,000 and 1:250,000 and selected standard stereo pairs of black-and-white aerial photographs at scales of approximately 1:18,000 to 1:30,000.

Aerial reconnaissance of faults was conducted as a part of this study. These reconnaissance flights were designed to provide observations during low-sun-angle lighting conditions.

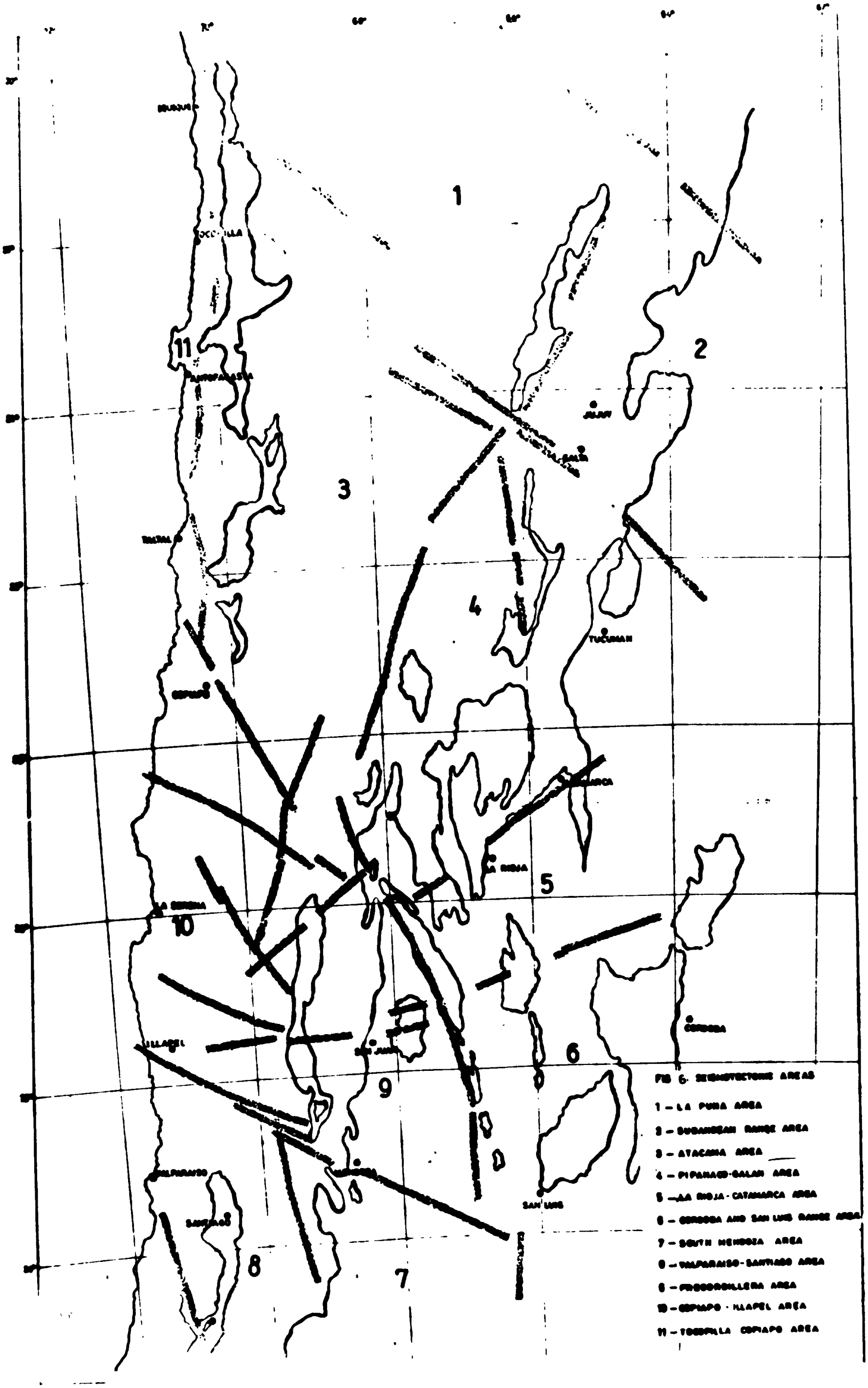


Figure 6. Seismotectonic Areas

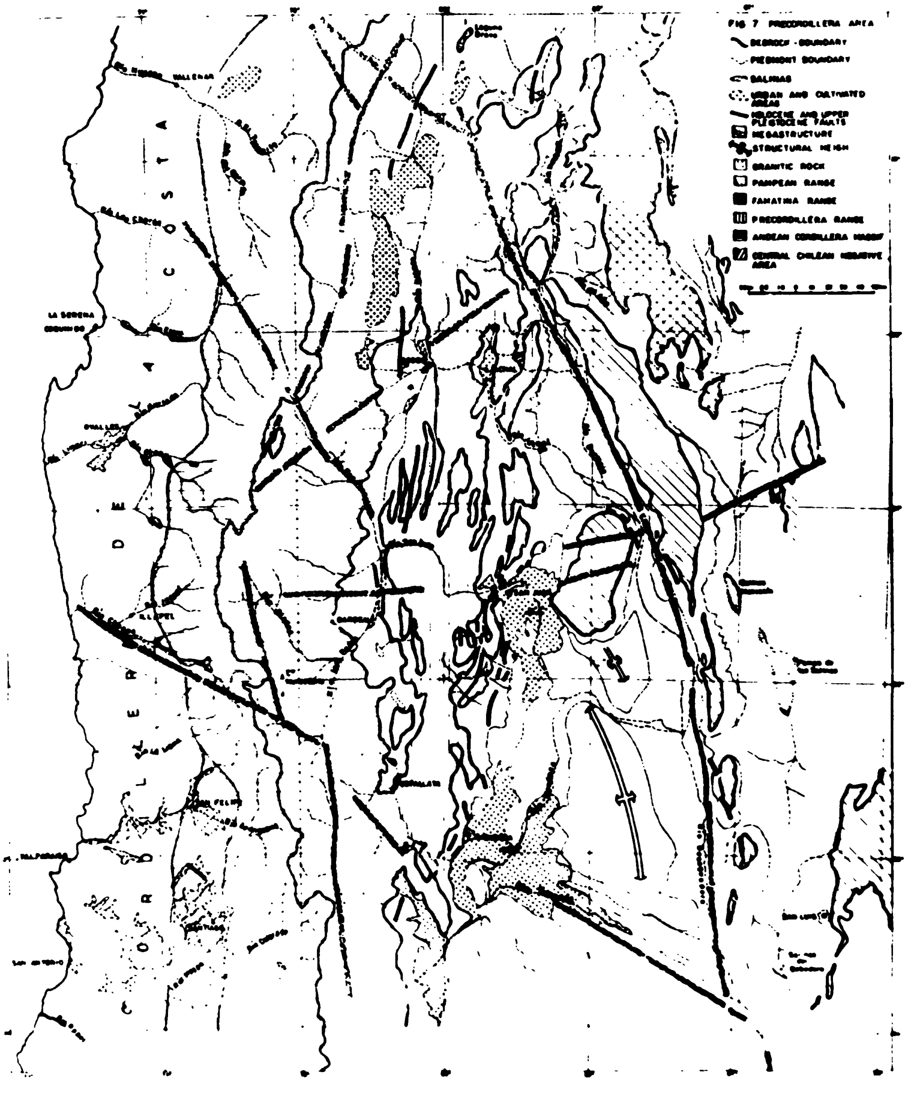


Figure 7. Precordillera Area

Ground studies were conducted at selected locations along all of the recognized potential seismic sources. The ground studies were designed to provide estimates of the fault parameters, based on scarp morphology, and to aid in the selection of sites at which to excavate trenches.

Using methods described by Wallace (1977; 1978a; 1978b) and Bucknam and Anderson (1979), among others, fault scarp height, profiles, and maximum slope angles were used to estimate: maximum displacement and surface rupture length for prehistoric earthquakes; the ages of those earthquakes, the number of displacement which form the present scarps, and the recurrence intervals.

The trenching program consisted of several excavations made with a 2-m wide front-end loader and were prepared in accordance with the methods of Taylor and Cluff (1973). Level lines were established in the trenches with a bubble-type line level; stratigraphic horizons were marked at one- to two-meter intervals with colored flags on nails or with spray-paint near marker nails. Organic materials encountered in several of the trench walls were collected for age dating.

The next step was to determine the seismicity parameters for the area and to estimate suitable attenuation relations.

With all that information seismic exposure maps were drawn (Figure 8).

In order to get results applicable to land-use planning, engineering design and emergency preparedness and response, the analysis was completed with subsurface condition studies in order to assess ground behavior in the presence of earthquakes and to estimate the liquefaction potential. Also the earthquake resistant characteristics of the existing constructions and the probable damage levels due to earthquakes were evaluated.

The results show a seismic hazard zonation map (Figure 9) and a seismic zonation for structural design (Figure 10) with its correspondents design spectra (Figure 11).

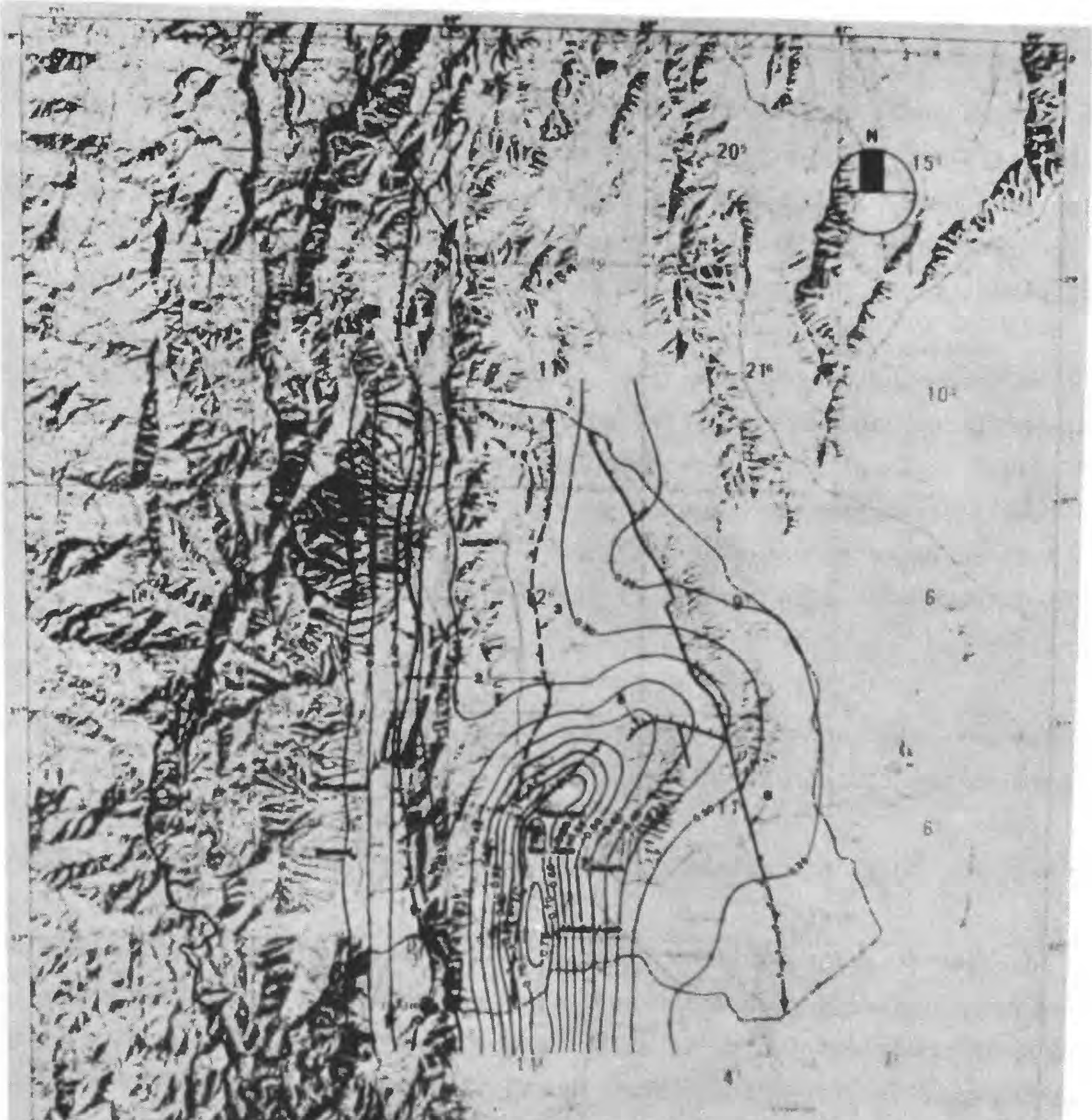


Figure 8. San Juan Province: Seismic exposure map contours of peak instrumental accelerations having a 10% probability of exceedance in 50 years.

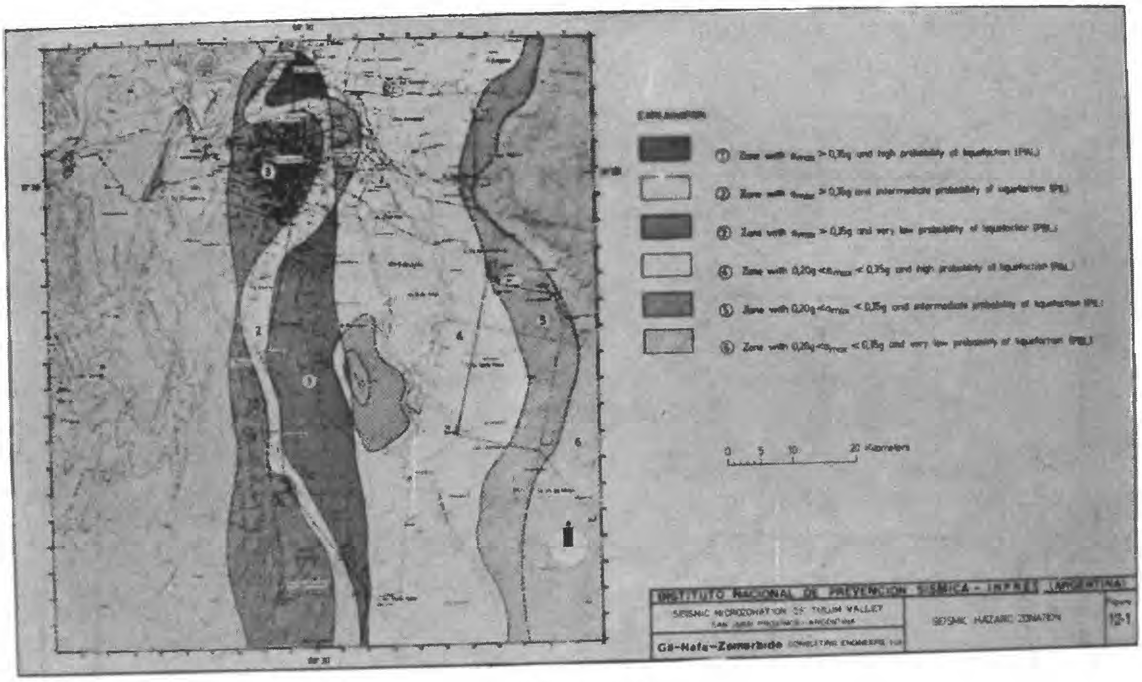


Figure 9

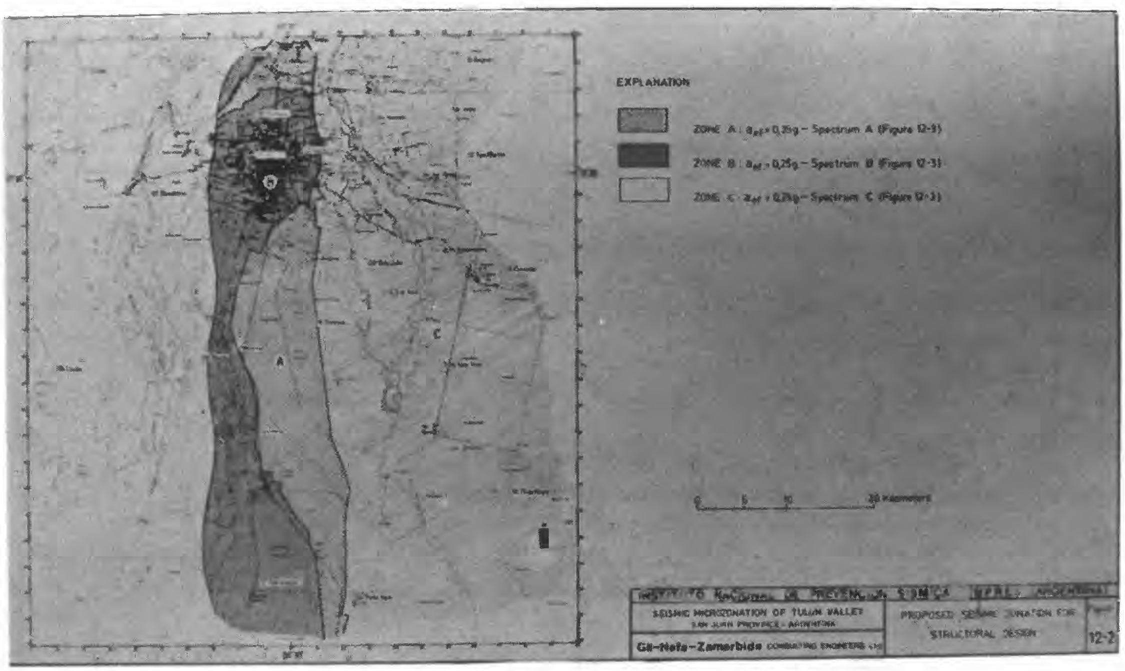
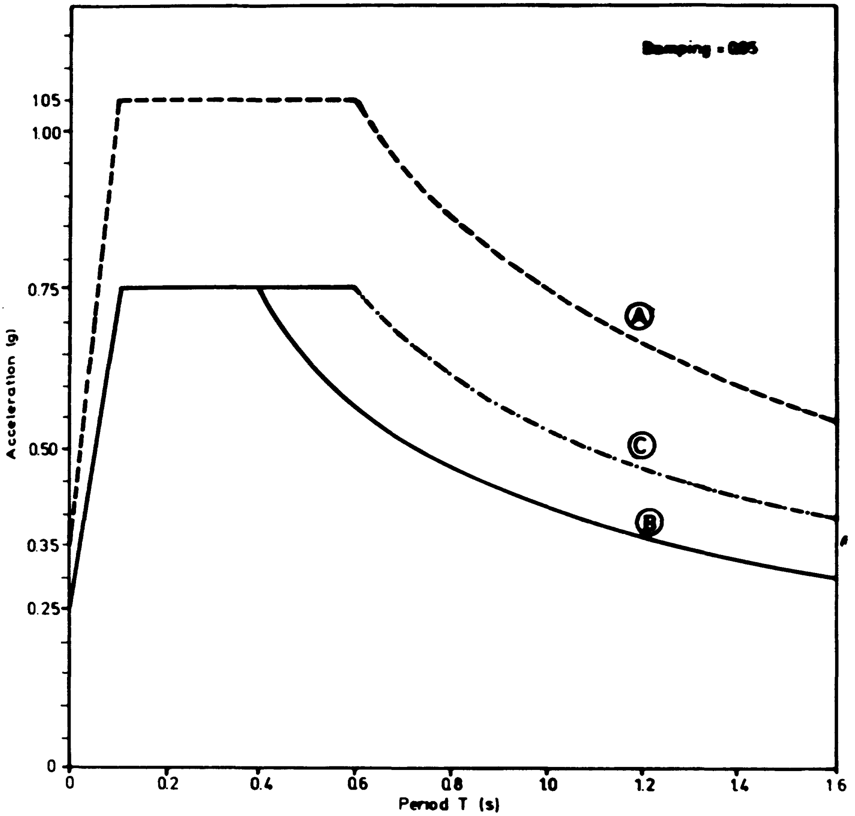


Figure 10



EXPLANATION

ZONE	$a_{ef.}$ (g)	b (g)	T_1 (s)	T_2 (s)
A	0.35	1.05	0.1	0.6
B	0.25	0.75	0.1	0.4
C	0.25	0.75	0.1	0.6

NOTES

1- $a_{ef.}$ = ground effective acceleration (for T = 0), (in g units)
2- b = flat portion ordenate of the spectrum (in g units)
3- T_1 = period corresponding to the beginning of the flat portion (seconds)
4- T_2 = period corresponding to the end of the flat portion (seconds)

STITUTO NACIONAL DE PREVENCION SISMICA – INPRES (ARGENTINA)

SEISMIC MICROZONATION OF TULUM VALLEY
SAN JUAN PROVINCE – ARGENTINA

I-Nafá-Zamarbide CONSULTING ENGINEERS Ltd.

RECOMMENDED DESIGN SPECTRA

Figure
12-3

Figure 11

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ESTIMATION OF EARTHQUAKE LOSSES

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INTRODUCTION

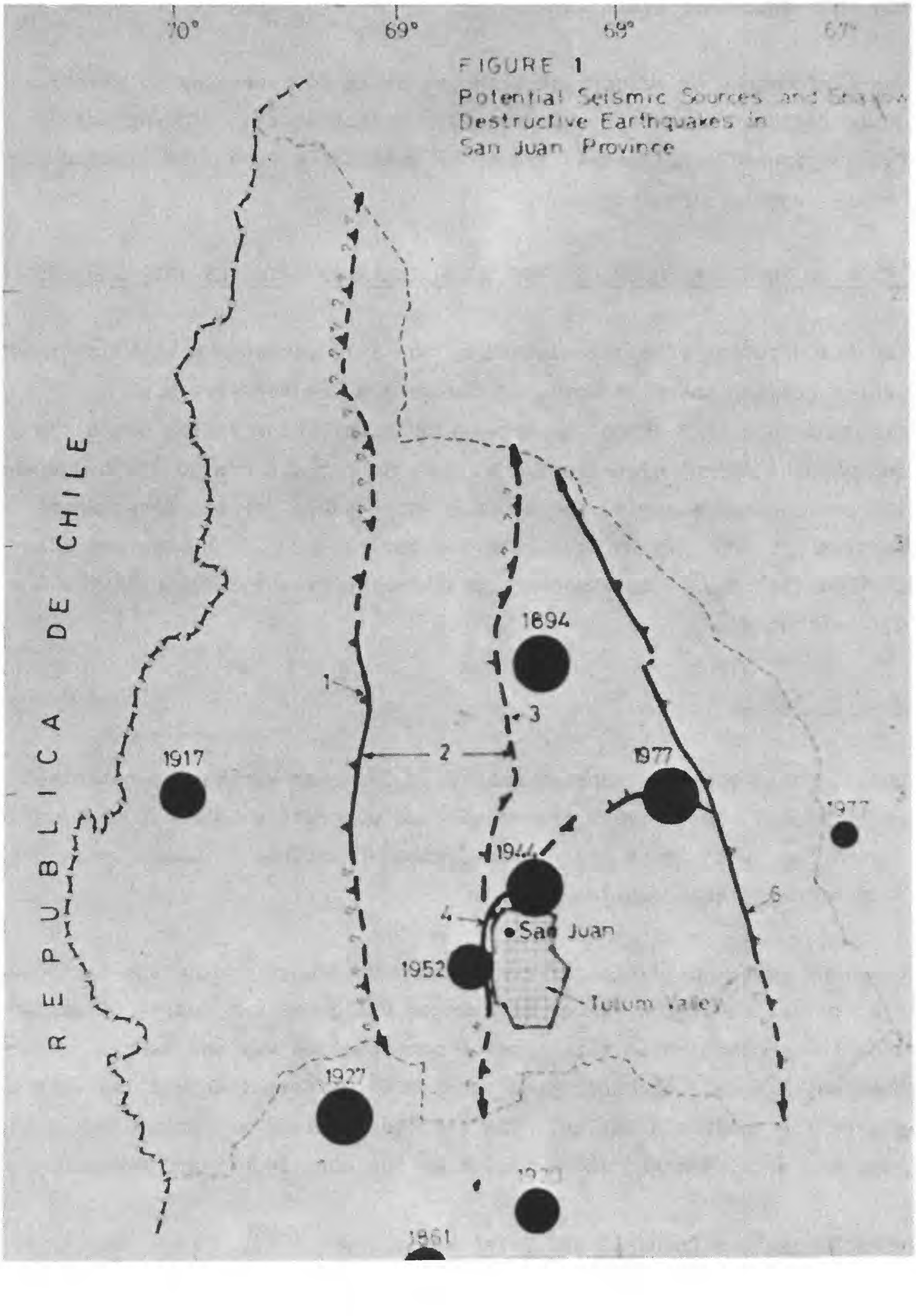
This paper is based on a research that INPRES is doing in Argentina as part of a comprehensive program called "Evaluation of the Economic Effects of Earthquake in Latin America (ECOSIS). This program is supported by AID through the U.S. Geological Survey. Data used in this paper were taken mainly from References 1 and 2. This paper has two main purposes:

1) to evaluate the economic losses in buildings and infrastructure in the Tulum Valley caused by the November 23, 1977, earthquake.

2) to estimate the probable potential losses in existing buildings of the same area considering their earthquake resistance capacity and the degree of seismic exposure.

The San Juan Province (Figure 1) located in the central west region of Argentina along the eastern flank of the Andes Mountains, has been the site of three large earthquakes in the past Century. The Tulum Valley (Figure 1) is the most important region in the province because it contains almost 90% of the population and more than 90% of the economic activity. An earthquake in 1894 caused considerable damage in the towns at the northern end of the valley. Another one in 1944 almost totally destroyed the capital and killed about 10,000 people. The most recent event, the 1977 earthquake, has been studied in detail and offers an excellent opportunity to observe damage patterns in the Tulum valley and to correlate them with physical parameters.

The epicenter of this earthquake (M_s=7.4) was located northeast of the Tulum valley at a distance of about 70 km from the city of San Juan. About 70 people were killed and very extensive property damage occurred throughout the


70°
69°
68°
67°
FIGURE 1
Potential Seismic Sources and Shallow
Destructive Earthquakes in
San Juan Province
REPUBLICA DE CHILE
1894
1917
1977
1977
1944
San Juan
1952
Tulum Valley
1927
1861
1
2
3
4
6

valley. The city of Caucete, 30 km southeast of the capital was affected more severely than any other urban area.

The performance of masonry structures, designed according to existing earthquake resistant regulations, was quite satisfactory. Wide-spread liquefaction caused considerable damage to cultivated areas, irrigation channels, water wells, roads, pipelines, etc.

EVALUATION OF ECONOMIC LOSSES IN THE TULUM VALLEY DUE TO THE 1977 EARTHQUAKE

The distribution of damage caused by the 1977 earthquake shows a direct correlation between the high level of damage and the occurrence of liquefaction. Figure 2 shows the areas within the Tulum Valley where the computed probability of liquefaction exceeds 50 percent due to the occurrence of the 1977 earthquake, assuming the minimum water table level. Areas where liquefaction was reported to have occurred during this earthquake are also shown. This figure will be used as a base map to evaluate the economic losses of the 1977 earthquake.

Damage to Buildings

Buildings in the area consist mainly of low-rise earthquake-resistant masonry or adobe. There are a few reinforced concrete constructions located in the city of San Juan; these are either framed structures infilled by masonry panels or concrete shear walls.

Only the adobe buildings suffered extensive damage during the earthquake. A complete evaluation of the number of damaged buildings and degree of damage caused by the earthquake in this type of construction was not made. Nevertheless, the data obtained from samples in different localities were used to extrapolate for the whole Valley. The results obtained are presented in Figure 2 where the number of damaged adobe buildings for each county are shown.

Damage to Irrigation Channels and Water Wells

The climate in the San Juan Province is arid. For this reason crops grown in this region are irrigated by channels and water wells. The irrigation system


FIGURE 2
DAMAGE TO BUILDINGS
31°30'

in the valley is 1,616 km long. One-third of this length consists of concrete channels and the rest is earth channels. Almost all of the damage to channels in the valley is due to ground failure at locations where liquefaction took place. Eighty km of concrete channels and 15 km of earth channels suffered heavy damage requiring rebuilding.

One thousand eight hundred and eight (1,808) water wells with average depth of 200 m were inspected in the area of heaviest damage. 315 of them were so heavily damaged that they were impossible to repair, while another 431 were repaired at an average cost of 25 percent of the total original cost.

Damage to Roads

Paved and unpaved roads suffered damage in the 1977 earthquake. The unpaved roads, which are not rigid structures, experienced deformations and fissures, but they were easily covered and were rapidily repaired at a very low cost. On the other hand, paved roads, which are more rigid, were heavily damaged with deep cracks. It was necessary to remove the pavement and build them again. Most of this damage (Figure 3) was also due to ground failure, as in the case of concrete channels. A total of 140 km of paved roads were destroyed and rebuilt.

Damage to Water Pipelines

Water pipelines were affected by the earthquake. Five small towns located in the area where liquefaction was widespread suffered heavy damage in their water distribution systems. Figure 4 shows the points where water pipelines were broken in the city of Caucete. If we define the damage ratio as the number of ruptures in the pipelines per 100 meters and plot it as a function of their diameters, the results obtained (Figure 5) show that the damage ratio diminishes as the diameter increases. Also, the curve depicting the damage ratio for cast-iron pipelines is far below the curve for asbestos-cement pipelines, revealing that damage was produced by differential settlement of the ground.

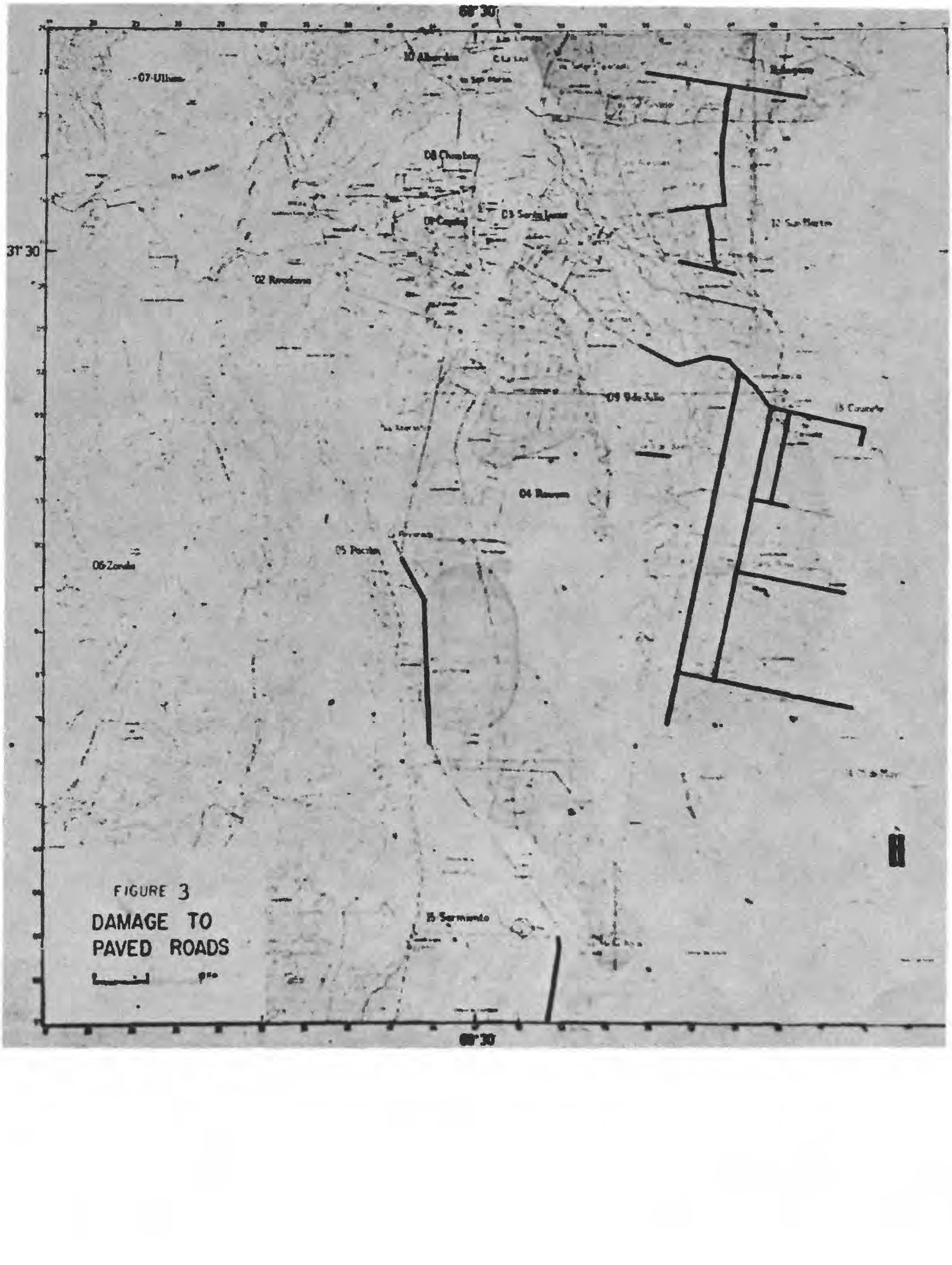


FIGURE 3
DAMAGE TO PAVED ROADS

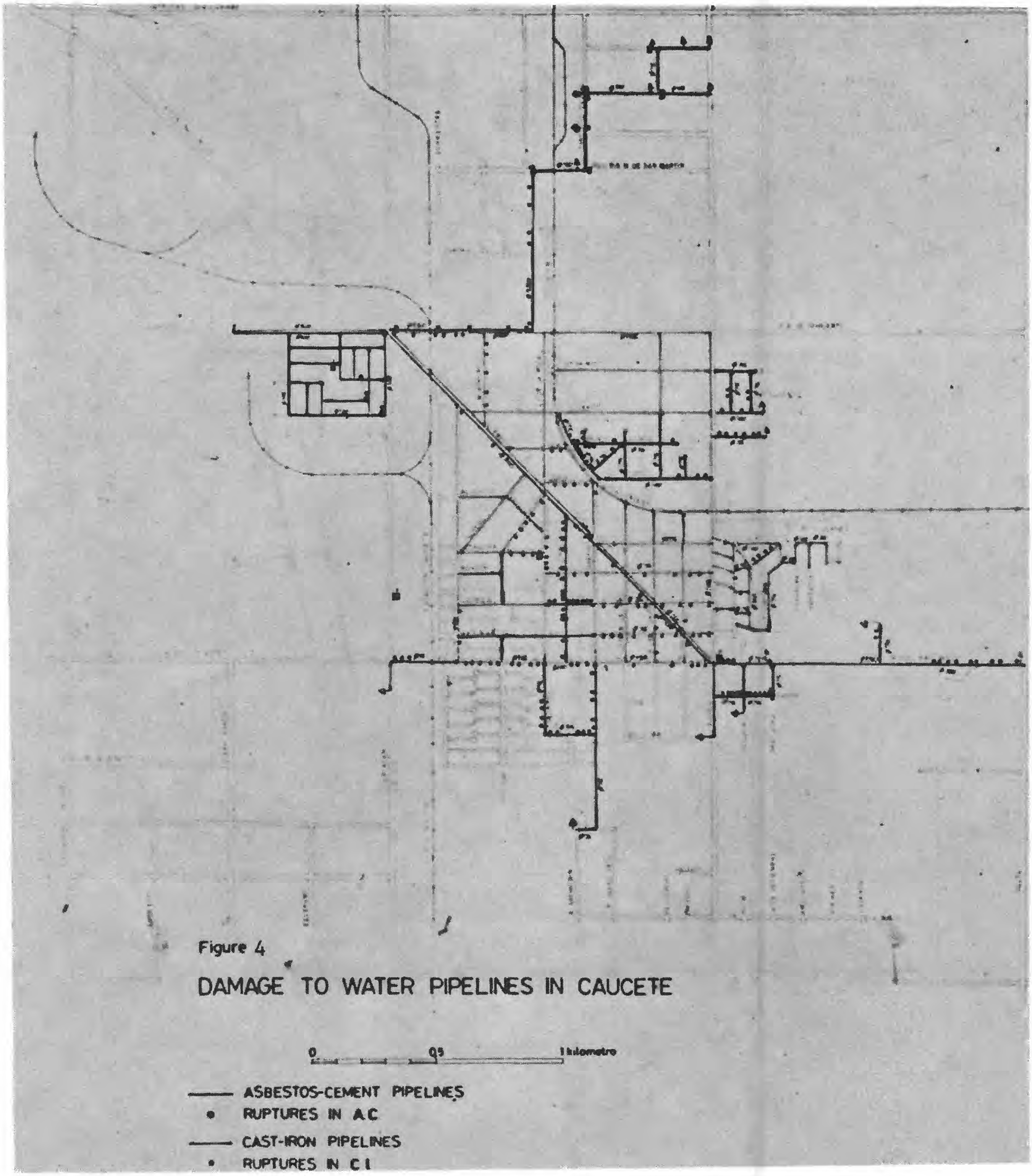


Figure 4

DAMAGE TO WATER PIPELINES IN CAUCETE

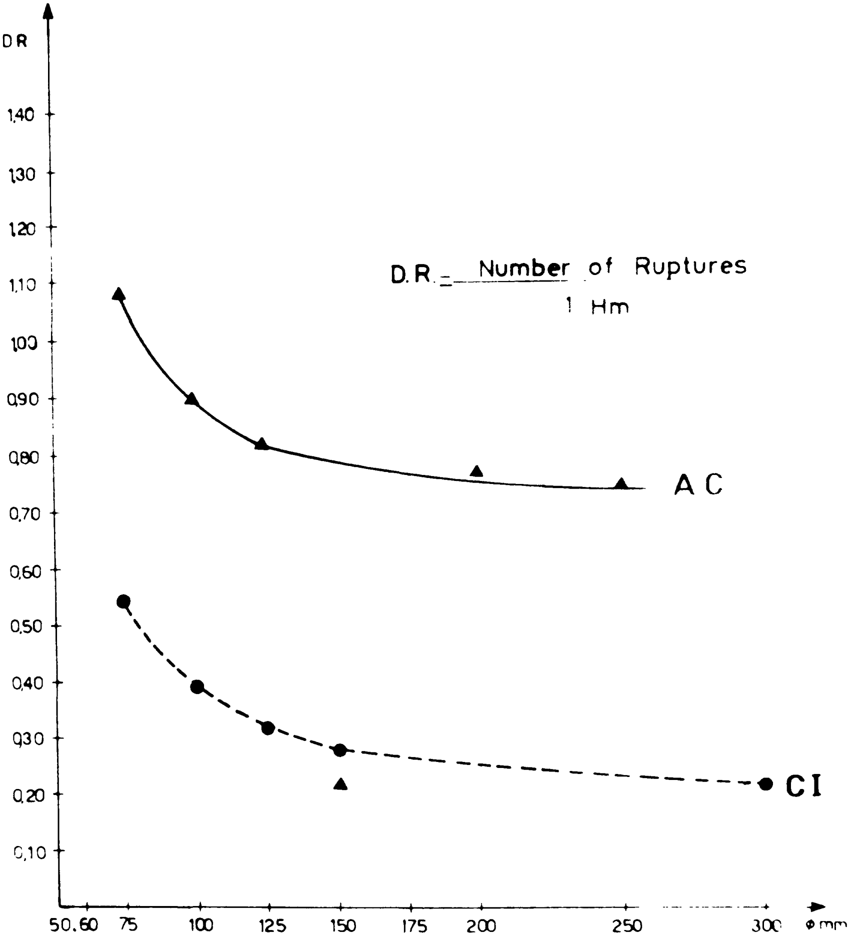


Figure 5: Damage Ratio in Water Pipelines in Caucete

Remarks

Up to this point the evaluation has taken into account only the direct costs due to the 1977 earthquake, but some indirect costs are also important in a complete evaluation. These include: damage to cultivated areas as a result of differential settlement of the ground, partial flooding due to liquefaction, increase of salinity in water wells, damage to plantations and crops caused by lack of water due to broken channels or wells, and damage to machinery on farms.

All of these costs are being compiled and the complete evaluation will be available at the end of this program.

ESTIMATION OF PROBABLE POTENTIAL LOSSES

In order to evaluate in a rigorous way the potential damage to existing buildings, the following factors should be considered: 1) type of lateral load resisting system, 2) distribution of mass and stiffness, either in plan or in elevation, 3) quality of workmanship, 4) non-structural elements. The problem is actually very complex and a methodology that includes all of these factors is beyond the scope of this paper. Thus, the approach adopted here is sxxended to express the potential for damage in a global fashion. The results obtained will be useful for economic decisionmaking by governmental officials. It is important to point out that existing buildings in the area are mainly low-rise earthquake-resistant masonry or adobe construction having no significant eccentricities in plan nor uneven distribution of mass and rigidity in elevation. Therefore, good performance is expected in future earthquakes. Accordingly, no correction or weight factors will be employed.

Seismic Exposure Analysis

The purpose of a seismic exposure analysis is to assess the probability that specific values of peak ground acceleration may be exceeded at different sites and to identify the seismic sources that have a dominant contribution to that probability. The seismic exposure at a point depends on: 1) the location and geometry of earthquake sources near the site, 2) the recurrence rate of earthquakes of various magnitudes in each source, and 3) the attenuation of ground motion from each source to the site. Figure 6 shows curves of seismic exposure for the Tulum Valley and location of populated areas where the economic losses were determined.

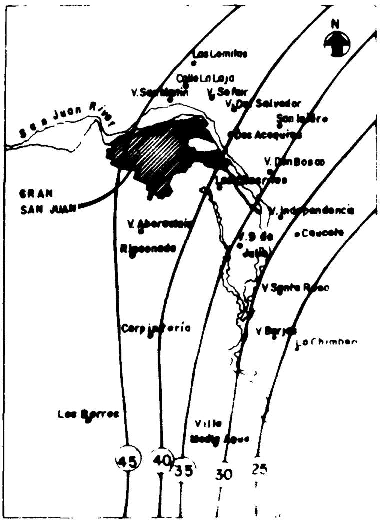


FIGURE 6 Contours of PGA (%g) having 10% probability of exceedance in 25 year

The regional geologic and tectonic setting of San Juan Province is characterized by predominantly east-west compression and related volcanism, plutonism, faulting, and sedimentation in fault-bounded basins. Major units or features of significance to an understanding of the regional geology and tectonics include the subducting Pacific plate, the Andes, the Precordillera, and the Pampean Ranges. On the basis of previous work six potential seismic sources were identified in the region. Figure 1 shows the Tulum Valley area and the location of these seismic sources. For each source, the magnitude of the maximum earthquake and its recurrence rate were estimated combining evidence from historical seismicity and fault studies. Several indices were used, including: 1) observed amount and sense of displacement along faults in the trenches that were excavated during the investigations, 2) scarp height, 3) scarp length, 4) number and height of steams terraces, and 5) fault length. Only a few of the faults in the province have been exposed to a sufficient number of earthquakes to be able to accurately define frequency-magnitude relationships, therefore, the regional b value for shallow (crustal) seismicity was used to characterize each fault. The a values were calculated using the geologically-estimated recurrence values. The data were normalized for time and magnitude intervals. Table 1 shows a summary of potential seismic sources used in the seismic exposure analysis.

Only a few strong motion accelerograms have been obtained during earthquakes in Argentina; therefore, there are insufficient data for regression analyses. Attenuation curves developed by INPRES were based on the available MMI intensity data and some seismoscope data in Argentina from shallow (crustal) earthquakes.

Table 1
Summary of Potential Seismic Source Data Used for
Seismic Exposure Analysis

Source		Length (km)	Strike	Dip	Maximum Earthquake	Fault Recurrence (years)
Number	Name					
1	Quebrada de la Caida de Tigre Fault	500	N-S	75°E	8	1,600
2	Precordillera Region Faulting Area	400	N-S	45°W	7-3/4	7,500
3	Frente Norte Fault	250	N-S	45°W	7-3/4	3,200
4	Precordillera Faulting System	250	Note 4	35°E	7-1/2	400
5	Sierra Pie de Palo Fault	60	Note 4	90°	7-3/4	4,000
6	Sierra Valley Fertil Fault	400	N70°W	45°E	7-3/4	15,000
7	Benioff Zone	-	-	Horizontal	7-1/2	15,500

Soil and Subsurface Conditions

During the 1977 earthquake most of the ground failure in the valley was related to liquefaction. Also, reports of liquefaction are well documented for the 1894 earthquake. Thus, it is important to determine the soil and subsurface conditions.

The potential for the ocurrence of liquefaction in the valley was computed, combining results of the seismic exposure analysis and the soil characteristics. The results indicated that three different zones of liquefaction hazard can be established: high, intermediate, and low.

The Tulum Valley is bordered on the east by Sierra Pie de Palo and on the west by Sierra Chica de Zonda, Sierra Villicum and the Tapias and Salado hills. The surface and near-surface rock and sediments of the valley, can be separated into four distinct zones, as follows: 1) rock, 2) alluvial fan deposits of the old course of the San Juan River, 3) flood plain deposits of the present course of the San Juan Rivers, and 4) transition zone of old fan deposits and flood plain deposits. Both free and constrained aquifers exist in the valley. Water used in the irrigation of the land percolates only to the impervious near-surface layers of fine-grained material where it stays as "perched" ground water. The depth of the free aquifer in the fan area, west of the river, varies from 10 to 100 m below the surface; the perched water table in the rest of the valley is as shallow as 1 m below the surface, depending on the time of the year. During a previous study 10 cross sections of the valley were prepared, using available information from water wells and from 15 new boreholes. The in-situ density was estimated from SPT measurements. Also the shear-wave velocities of both the alluvial fan and the flood plain materials were measured by field geophysical methods.

Damage Assessment

During a detailed survey carried out in 1981, of the existing construction in all localities of the valley having 100 or more dwellings, more than 83,600 buildings were identified and classified into seven different types according to their earthquake resistance. Types and percentage of dwellings (in brackets) with respect to the total are: 1) earthquake-resistant masonry (57.4%), 2) non-earthquake-resistant masonry (1.6%), 3) reinforced-concrete moment-resisting frame systems with infilled masonry panels (0.08%), 4) shear wall buildings (0.02%), 5) earthquake-resistant buildings for industries, warehouses, supermarkets, etc. (0.8%), 6) adobe dwellings (39.7%), 7) other types (wood, metallic, mixed, etc.) (0.4%). Because types 3, 4 and 5 jointly represent less than 1% of the total (0.9%) and considering their characteristics and response to ground motions, they were included as part of type 1. Therefore, neglecting dwellings of type 7, three principal types of buildings (earthquake-resistant masonry, non-earthquake-resistant masonry and adobe buildings) were considered. Another important feature of the existing construction in the area is that 95.8% is one story high; 3.9% two stories high and 0.3% are three or more stories high. The fundamental period of vibration for the majority of the construction in the valley was estimated to be less than about 0.2 to 0.3 seconds.

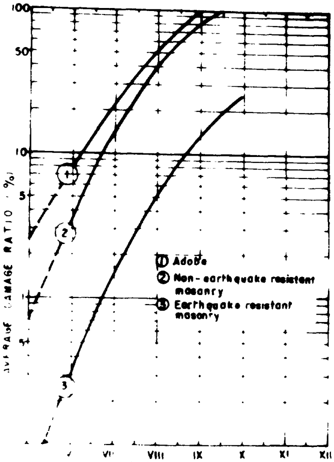


FIGURE 7 Damage ratio vs M.M Intensity
(From Ref 3)

To estimate the potential damage in existing buildings, a direct relationship between the earthquake loading and damage ratio is needed. The earthquake loading will be represented by peak ground acceleration, which can be correlated with Modified Mercalli intensity (MMI). Several authors have proposed equations to correlate peak ground acceleration and MMI. Although there is a wide range of disagreement among them, it is agreed that the relationship is affected mainly by magnitude and epicentral distance. The question of whether peak ground acceleration is a good parameter to be correlated to Modified Mercalli Intensity will not be discussed here. Other ground motion parameters may be used, but the reliability of each one is still a controversial matter. For this reason, ranges of peak acceleration instead of one specific value of acceleration will be correlated with MMI (see Table 2). The relationship between MMI and damage ratio adopted is shown in Figure 7. These curves were proposed by Sauter, McCann, and Shah (1980) (Ref. 3) and they are a kind of "average" of different proposals of several authors. They agree fairly well with the damage observed during the 1977 San Juan earthquake and with historical information of the 1894 and 1944 earthquakes.

Table 2. PGA Ranges vs MM Intensities

MMI	PGA(%g)
V	5
VI	5-10
VII	10-20
VIII	20-35
IX	35-50
X	50

Probable Potential Losses

In an attempt to consider the liquefaction potential in the probable losses, soil factors are proposed (see Table 3). The values included in the table are

Table 3

Soil Factors Adopted for Liquefaction Hazard

Liquefaction Hazard	High	Intermediate	Low
Soil Factor	1,2	1,1	1,0

To evaluate the probable potential losses (PPL) the following equation was used:

$$PPL = \Sigma_j \; Bj \,.\, EDj \,.\, Cj \quad (j \text{ all types of buildings}) \qquad (1)$$

where:

Bj = number of type j buildings

EDj = expected damage for type j buildings

Cj = equivalent cost ratio

$$= \frac{\text{mean cost per sq. meter of a type j building}}{\text{mean cost per sq. meter of an earthquake-resistant masonry building}}$$

The expected damage was obtained as:

$$EDj = \sum_{\text{all } i} P(MMI = i) \,.\, DRj \,.\, S \qquad (2)$$

being:

P (MMI = i) = probability of occurrence of a MMI = i

DRj = damage ratio for a type j building (see Figure 6)

S = soil factor from Table 3

based solely on engineering judgement and tend to quantify the fact that the larger the liquefaction potential, the larger the probable potential losses.

The probable potential losses were evaluated for two periods of time, 1 and 25 years. Adopted values for Cj are, respectively, 1, 0.5, and 0.1 for earthquake resistant masonry, non-earthquake-resistant masonry and adobe buildings. The results are shown in Table 4 for each construction type and for each locality. Total PPL are also shown. These PPL were represented by the equivalent number of earthquake-resistant masonry dwellings. To evaluate the losses in financial terms, this number must be multiplied by the cost of such a dwelling.

Concluding Remarks

The results obtained show that the Tulum Valley area has a high degree of seismic risk. This risk may be quantified as probable potential losses in terms of the cost of typical earthquake-resistant masonry dwellings. Assuming a mean cost of $11,000 (US) per dwelling, the probable economic losses would be approximately $3,800,000 (US) per year and $53,400,000 (US) per twenty five years. It should be pointed out, however, that the economic losses specified above are only related to buildings. Highways, bridges, irrigation channels, pipelines, etc., were not considered in computing the losses.

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Table 4 - Probable Potential Losses

County	Locality	Masonry Buildings						Adobe Buildings			P. P. L.	
		Earthquake Resistant			Non-Earthquake Resistant							
		Number of Buildings	F. D. % 1 year	E. D. % 25 years	Number of Buildings	F. D. % 1 year	E. D. % 25 years	Number of Buildings	E. D. % 1 year	E. D. % 25 years	1 year	25 years
Albardon	Va. San Martin	242	0.4	6.6	2	3.2	45.0	164	6.3	63.9	2.0	26.9
	Calle La Laja	189	0.4	6.4	2	3.1	44.0	345	6.2	62.6	2.9	34.1
	Las Lomitas	14	0.4	6.3	2	3.0	43.6	138	6.1	62.2	0.9	9.9
Angaco	Sefair-Talacasto	42	0.4	6.3	-	-	-	142	6.2	62.7	1.1	11.6
	Va. El Salvador	293	0.4	6.3	1	3.1	43.8	116	6.2	62.5	1.9	25.9
Capital	Gran San Juan 1	3.835	0.3	5.8	-	-	-	81	5.5	55.8	12.0	226.9
	Gran San Juan 2	4.596	0.3	5.8	47	2.8	39.5	931	5.5	55.8	19.6	327.8
	Gran San Juan 3	5.929	0.3	5.8	101	2.8	39.5	2.837	5.5	55.8	34.8	522.1
	Gran San Juan 4	4.188	0.3	5.8	26	2.8	39.5	2.036	5.5	55.8	24.1	361.6
	Gran San Juan 5	3.560	0.3	5.8	170	2.8	39.5	2.322	5.5	55.8	25.8	369.6
Caucete	Ciudad de Caucete	2.187	0.2	3.4	-	-	-	1.507	5.0	41.7	11.9	137.2
	Va. Independencia	26	0.3	4.1	-	-	-	157	5.3	46.7	0.9	8.4
Chimbas	Gran San Juan 1	1.764	0.3	5.8	146	2.8	39.5	1.707	5.5	55.8	16.7	226.4
	Gran San Juan 2	1.734	0.3	5.8	65	2.8	39.5	2.101	5.5	55.8	17.7	230.6
	Gran San Juan 3	124	0.3	5.8	6	2.8	39.5	428	5.5	55.8	2.8	32.3
9 de Julio	Las Chacritas	48	0.4	6.2	-	-	-	226	6.1	61.6	1.6	16.9
	Va. 9 de Julio	258	0.3	4.7	-	-	-	112	5.5	50.5	1.4	17.8
Pocito	Gran San Juan	191	0.3	5.8	-	-	-	854	5.5	55.8	5.3	58.7
	Va. Aberastain	576	0.4	6.7	-	-	-	852	6.0	62.5	7.4	91.8
	La Rinconada	115	0.4	6.7	2	3.1	-	177	6.0	62.5	1.6	19.2
	Carpinteria	82	0.3	5.4	-	-	-	137	5.4	52.5	1.0	11.6
Rawson	Gran San Juan	8.530	0.3	5.8	268	2.8	39.5	6.725	5.5	55.8	66.3	922.9
Rivadavia	Gran San Juan	2.063	0.3	5.8	118	2.8	39.5	1.605	5.5	55.8	16.7	232.5
	Gran San Juan	1.290	0.3	5.8	48	2.8	39.5	1.490	5.5	55.8	12.7	167.4
	Gran San Juan	1.727	0.3	5.8	260	2.8	39.5	1.129	5.5	55.8	15.0	214.5
San Martin	Dos Acequias	42	0.4	6.4	-	-	-	102	6.2	62.7	0.8	9.1
	San Isidro	29	0.3	5.9	-	-	-	107	5.9	59.4	0.7	8.1
	Don Bosco	190	0.3	5.3	-	-	-	109	5.7	55.5	1.2	16.1
Santa Lucia	Gran San Juan	3.578	0.3	5.8	91	2.8	39.5	2.689	5.5	55.8	26.8	375.5
	Alto de Sierra	181	0.4	6.6	-	-	-	547	6.3	64.2	4.2	47.1
Sarmiento	Cienequita	56	0.3	5.1	-	-	-	54	5.0	50.7	0.4	5.6
	Los Berros	171	0.3	5.1	-	-	-	295	5.0	50.7	2.0	23.7
	Va. Gral. Sarmiento	347	0.2	3.7	-	-	-	367	5.0	40.1	2.5	27.6
25 de Mayo	Va. Santa Rosa	193	0.2	2.9	2	2.0	23.4	194	4.7	37.1	1.3	13.0
	Va. Borjas	262	0.2	2.7	6	1.9	21.6	171	4.6	34.5	1.4	13.6
	La Chimbera	125	0.2	2.2	1	1.7	18.4	301	4.4	30.5	1.6	12.0
	Total	48.777			1.364			33.255			347.1	4851.5

LAND USE PLANNING RELATIVE TO SEISMIC HAZARDS

By

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INTRODUCTION

Most of the death and destruction resulting from earthquakes occur in urban areas, ranging from the smallest town to the largest city. The location, design and construction of a city can have a significant effect on its vulnerability under seismic conditions. The professional land use planner (also termed city planner or urban planner) has the responsibility to advise governments regarding urban development. It therefore is incumbent on the land use planner to become familiar with seismic problems and incorporate this information into his ongoing planning work. To do this, he needs to receive information from the work with other professionals including seismologists, geologists, soils engineers and structural engineers. He also needs to put his work in the context of the full range of responses that can be developed to deal with seismic hazards. His main contributions can be grouped under two headings: pre-earthquake hazard mitigation and post-earthquake reconstruction.

RESPONSE TO HAZARDS

There are five basic ways in which people respond to seismic hazards:

1) Avoid hazardous areas.
2) Stabilize unstable ground.
3) Build or reinforce structures to withstand ground failure or intense shaking
4) Limit types and intensities of land use to be commensurate with hazards.
5) Establish warning systems.

The land use planner is most concerned with the long-term safety of an urban area and therefore prefers to avoid placing development in hazardous areas. This approach works best in vacant areas that are just being developed. When a hazard cannot be avoided, the other responses should be considered. Each of the responses has its place in hazard mitigation.

PRE-EARTHQUAKE HAZARD MITIGATION

Many factors affect the success of pre-earthquake hazard mitigation programs. These factors can be grouped under four headings: information, new development, existing development and political support.

Information

The provision and interpretation of basic information is fundamental to a successful hazard mitigation program.

1) Availability of geologists: The single most effective way in which to bring geologic and seismic concerns into land use planning is to have a professional geologist available to consult with land use planners on a regular and continuing basis.

2) Geologic maps: The basis for a carefully prepared plan that takes geology into consideration is good geologic mapping and interpretive maps which cover the planning area and are available for ready use. (Example: Town of Portola Valley, California.)

3) Land capability maps: Land capability maps take geologic and other data and carefully rate and weight these factors with respect to identified land uses. These maps help identify those areas most capable of supporting the land uses considered. (Example: Santa Barbara County, California.)

New Development

The task of the planner is easiest in those areas that are vacant and are to be developed in the future. Here one can identify the most hazardous areas and recommend that they be avoided to the maximum extent feasible. If hazardous areas cannot be avoided, then other solutions such as strengthening structures, stabilizing ground, reducing the amount of development or installing warning systems can be considered.

A system is needed in which seismic information can be fed into the decision-making process. One convenient way to view this process is to consider that it has three phases, general development plans, development regulations and construction regulations.

1) General development plans: Plans that indicate the general development of an urban area should take seismic hazards into account. These plans address large areas such as metropolitan regions, individual cities or major developments within cities. The plans are general in nature and in the United States are referred to as "general plans." At this general level of land use planning detailed geologic data is usually not needed. As the area of concern is reduced in size, however, more detailed information is needed.

2) Development regulations: While generalized geologic information is adequate for the general development plan phase of land use planning, there is a need to develop more specific information as development plans become more detailed. In the United States, this more detailed information is usually required by the state or local governments of those proposing to undertake development. (Examples: Alquist-Priolo Special Studies Zones Act, California Law: landslide mapping and related development policies of the Town of Portola Valley, California: Portola Valley Ranoh residential development, Town of Portola Valley, California.)

3) Grading and construction regulations: The most detailed geologic information is needed at the time grading and constructions plans are being prepared for a development. These plans are the domain of engineering geologists, soils engineers, civil engineers, structural engineers, and architects more than land use planners. Nevertheless, since the final action in the land use planning and development process relates to grading and construction, it is at this point highly critical that proper procedures be followed: if they are not, the preceding land use planning efforts will have been in vain.

Existing Development

The problem of reducing seismic hazards in areas that are already developed is much more difficult to solve than in areas that are being newly developed. In many cities, the large inventory of structures that do not meet adequate seismic safety standards is staggering. Nevertheless, it is incumbent on land use planners in concert with others to seriously study the problem and attempt to find solutions. Three aspects of existing development deserve discussion; strengthening of buildings, redevelopment and critical facilities.

1) Strengthening of buildings: Where old structures pose a threat to life and property, they need to be studied to determine what can be done to increase their strength. Structural engineers should play a major role in these studies and recommendations. Nevertheless, the land use planner needs to be knowledgeable about the topic in order to try and include it as a significant factor in his planning work. (Example: City of Los Angeles, California.)

2) Redevelopment: When structural conditions are so bad that the cost of strengthening a building exceeds the value of the building, then the more advisable action may be to demolish the building and build a new, more modern building. A new building may have many benefits in addition to greater safety, such as better energy conservation and more functional space to meet

current demands. Most redevelopment projects in the United States have been undertaken for a variety of reasons, and the earthquake threat has not usually been of major concern except in the aftermath of a major earthquake. After damaging earthquakes, cities often are more willing to consider redevelopment of areas which have been damaged. (Example: 1969 Santa Rosa earthquake, California.)

3) Critical facilities: All cities have what are called "critical facilities." Critical facilities are those buildings and other structures that if damaged would have a major effect on the community including hindering post-earthquake emergency response. These include: hospitals, fire stations, communication centers, major utility systems, dams, and major power plants. Also of concern are structures which involve involuntary occupancy, such as schools (attendance is compulsory in the United States) and jails. Critical facilities need special attention as to damage under seismic conditions. The proper response may be to strengthen the facility, provide redundancy, or remove it and build a new facility.

Political Support

Planners realize that their best efforts to reduce seismic vulnerability will not come to realization unless there is political support for their recommendations. It is therefore fundamental to the land use planning program to educate governmental officials as to seismic hazards and methods for their reduction. This is not an easy task. Most people understand earthquakes to be events that occur infrequently and believe chances are that they won't be affected. Others believe that earthquakes are inevitable and that there is little that can be done to reduce vulnerability. Nevertheless, vigorous efforts need to be made to help governmental officials realistically appraise the earthquake potential and urban vulnerability.

There is one time when political support for seismic safety is usually very high--right after a damaging earthquake. It behooves the land use planner to have recommendations ready which can be given to governmental officials immediately after an earthquake when there is a high level of concern.

POST-EARTHQUAKE RECONSTRUCTION

It would appear that the period immediately following a damaging earthquake would be an ideal time to reduce vulnerability to future earthquakes. This should be the time to clear away old and damaged buildings and start fresh. The record has indicated, however, that it is not so easy to make such improvements. There are in fact many forces operating which call for rapid rebuilding in almost any fashion just to have shelter available. While many aspects of reconstruction can be discussed, five topics are of major concern: reconstruction planning process, reconstruction funds, unstable ground, damaged structures, and emergency and temporary housing.

Reconstruction Planning Process

If reconstruction planning is going to reduce seismic vulnerability and result in a more carefully designed city, it is critical that a process be established in which all of the concerns that should be taken into account in decision-making are considered. This will not automatically happen in most cases. It is important to consider what such a process should include. Major factors to be included are:

1) Mapping of hazardous areas
2) Mapping of structural damage
3) Identification of sources of aid for reconstruction
4) Preparation of preliminary reconstruction plan
5) Review of plan by all affected parties
6) Adoption of reconstruction plan
7) Commencement of reconstruction

While undertaking this program, the emergency requirements of the urban area will have to be met first, such as temporary housing and restoration of utilities.

Reconstruction Funds

Review of past earthquakes in the United States indicates that implementation of reconstruction plans depends on the amount of funds available. Major projects requiring stabilization of land and construction of new buildings and other structures are usually dependent on the amount of outside funds available, since the costs usually far exceed the financial capability of the local city. If outside funds are not available, it is not likely that significant improvements in seismic safety will result. In the United States, the outside funds for local cities come almost exclusively from the Federal government.

Unstable Ground

Large earthquakes often reveal unstable ground in a dramatic way such as by landslides, liquefaction and differential settlement. In reconstruction such areas need to be identified; either the instabilities should be corrected as a part of the reconstruction or the areas should be avoided. This requires good mapping, engineering and land use planning. The proper treatment of unstable ground after an earthquake is one of the major areas of concern by land use planners. (Examples: 1964 Alaska earthquake: 1980 Campania-Basilicata earthquake, Italy.)

Damaged Structures

After a damaging earthquake there is usually a wide range of structural damage from little or none to complete destruction. It is important to inventory buildings to determine which ones can be used with no structural work, which ones can be saved through reinforcement and which ones are beyond saving. These decisions are not easy to make and involve considerations of cost, risk and alternatives. The evaluation of damaged structures provides the planner with important information for making decisions about the clearance or rehabilitation of damaged areas. (Examples: 1980 Campania-Basilicata earthquake, Italy; 1983 Coalinga earthquake, California.)

Emergency and Temporary Housing

One of the major problems after an earthquake is the provision of emergency and temporary shelter for businesses, government services and housing. The major part of the problem relates to housing. Observations from past earthquakes indicate that this housing problem usually involves three steps:

Step 1 - Emergency shelter, such as tents or small trailers
Step 2 - Temporary shelter, such as mobile homes or prefabricated structures
Step 3 - Permanent new housing

In some instances it has been possible to move from emergency shelter directly into quickly constructed permanent housing. The planner should become deeply involved in the siting and construction of housing since these decisions, which are usually made rather rapidly, will affect the long-term future development of a community. To the extent that locations have been identified prior to an earthquake for the location of temporary housing, the easier the task will be of providing space after the earthquake. (Examples: 1976 Tangshan earthquake, China: 1976 Friuli earthquake, Italy: 1980 Campania-Basilicata earthquake, Italy: 1980 El Asnam earthquake, Algeria.)

CONCLUSION

The land use planner's primary job is to help direct the overall development of urban areas both before and after disasters. In his work he must consider all of the factors that affect urban areas including physical, economic, social and political. Seismic concerns are but one of the factors in the wide range of matters the planner must consider. All too often, seismic factors are ignored. If they are to be taken into account, it is important that professionals from other disciplines such as seismology, geology, soils engineering and structural engineering inform the planner about seismic problems and work with the planner in developing responses to potential hazards.

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SOCIOECONOMIC SIGNIFICANCE OF LANDSLIDES AND MUDFLOWS

LANDSLIDE HAZARDS

By

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GENERAL SOCIOECONOMIC CONSIDERATIONS

The social and economic costs of landslides and mudflows in the world are extreme and apparently growing. In addition to killing people and livestock, damaging or destroying residential and industrial developments, as well as villages and even entire towns, slope failures are destructive to agricultural and forest lands, and have negative impact on the quality of water in rivers and streams. Human activities in disturbing large volumes of geologic materials in construction of transportation routes, building developments, mines and quarries, dams and reservoirs, canals, tunnels, and lines for communications systems have been a major factor in the 20th Century's increase slope failures harmful to the lithosphere.

The following note by Marinatos (1960, p. 186) may well refer to the earliest record of a catastrophic landslide resulting in serious social and economic losses: "In the year 373/2 B.C., during a disastrous winter night, a strange thing happened in central Greece. Helice, a great and prosperous town on the north coast of the Peloponnesus, was engulfed by the waves after being leveled by a great earthquake. Not a single soul survived." Research by Marinatos indicates that Helice probably was engulfed as the ground slipped toward the sea a distance of about 1 km. Seed (1968) has concluded that this was a major landslide resulting from soil liquefaction caused by the earthquake.

Another such catastrophe (probably a debris flow) was noted by Spanish conquistadors in Bolivia in the 16th century (Sanjines, 1948). According to the priest Padre Calancha, who observed the event from a distance, Hanco, a community of about 2,000 inhabitants, disappeared "in a few minutes and was swallowed by the earth without more evidence of its former existence than a cloud of dust which arose where the village had been situated."

In the 20th Century, there have been well-recorded examples of the destructiveness of slope failures. As an extreme example, massive landslides triggered by a major earthquake in Kansu Province, China, in 1920 destroyed dozens of villages and killed on the order of 100,000 people. Three major landslides in Peru in the past 20 years have killed a total of over 20,000 people and destroyed villages, agricultural lands, and highways. Landslides and mudflows also have caused devastating secondary effects resulting in death and destruction. For example, in 1963 a large rockslide into the Vaiont Reservoir in northern Italy displaced a huge volume of water that overtopped the dam, killing nearly 3,000 people.

Slope failures are responsible for considerably greater social and economic loss than is generally recognized; they represent a significant element of many major disasters in which the magnitude of their effects is overlooked by the news media. For example, in the United States the tremendous destruction in central Virginia in 1969 is remembered as an effect of Hurricane Camille, even though most of the people killed during the storm died as a direct result of debris flows caused by the heavy rain.

Governmental agencies and policymaking individuals need to develop a better understanding of the socioeconomic significance of landslides and mudflows. Such knowledge will allow officials at all levels of government to make rational decisions on allocation of funds to landslide and mudflow warning, avoidance, prevention, and control, and to postfailure repair and reconstruction. This report is intended to encourage governmental agencies to keep more complete records of such costs and to utilize these records in future planning.

LOSSES DUE TO LANDSLIDES AND MUDFLOWS

Categories of Damage Costs

Direct versus indirect costs—Landslide and mudflow costs include both direct and indirect losses from slope failures affecting public and private properties. For the purpose of this report, a direct cost is defined as the cost of replacement, repair, or maintenance due to damage to installations or property within the boundaries of the responsible landslide or mudflow.

All other costs of landslides and mudflows are indirect. Examples of indirect costs are: (a) loss of productivity of agricultural or forest lands, (b) reduced real-estate values in areas threatened by landslides or mudflows, (c) loss of tax revenues on properties devalued as a result of landslides or mudflows, (d) measures to prevent or mitigate additional landslide or mudflow damage, (e) adverse effects on water quality in streams and irrigation facilities outside the landslide limits, (f) loss of industrial or agricultural productivity due to interruption of transportation systems by landslides or mudflows, and (g) secondary physical effects, such as flooding, that produce both direct and indirect costs. Some indirect costs of landslides and mudflows are difficult to evaluate; thus their estimates often tend to be conservative or ignored. Where indirect costs have been rigorously determined, they sometimes have been found to exceed direct costs. The possibility of a major landslide that could destroy port facilities and create a wave that might inundate downtown Kodiak, Alaska, is an example of a landslide threat that is alleged to have caused indirect costs relating to planning for expansion of the port area.

Public versus private costs—Possibly of greater interest than whether a cost is directly or indirectly attributable to a landslide or mudflow is a breakdown on the basis of who actually pays for the damages. On this basis landslide costs can be divided into costs to public and private entities (Fleming and Taylor, 1980).

Public costs are those costs borne by government agencies; private costs are all others. The heaviest public costs traditionally have been for repairing or relocating roads and highways and appurtenant structures such as storm drains and sidewalks. Other examples of public costs are those incurred to dams and reservoirs, sewer systems, public buildings, and harbor and port facilities.

Private costs consist mainly of damage to real estate and structures, either private homes or industrial facilities. Indirect costs to private individuals often occur when the value of their property is reduced, not by direct damage, but by the threat of landslide or mudflow activity.

Examples of Losses Due to Landslides and Mudflows

Financial losses--Reliable estimates of the overall costs of landslides and mudflows are difficult to obtain for geographical entities as large as individual nations; thus they are not available for the world as a whole at present. Because of the relative availability of data on landslide damages in the United States and because of the authors' familiarity with these data, this section will relate primarily to losses in the United States. In most cases, these estimates can be related directly to other developed countries with similar terrains and mixes of urban and rural habitats. However, the costs presented here may be somewhat higher than might be expected in developing countries where property and labor values may be lower than in the United States. In addition, estimated losses are given here as United States dollar losses at the time of damage, i.e., no attempt has been made to account for inflation by converting original dollar losses into today's values.

In perhaps the first national assessment of United States landslide costs, the Highway Research Board (Smith, 1958) reported that "the average yearly cost of landslides in the United States runs to hundreds of millions of dollars," probably a realistic estimate for that time. However, in the 25 years since Smith obtained his data, a combination of inflation, increased development in landslide- and mudflow-prone areas, and the use of larger cuts and fills in construction has resulted in considerably increased slope-failure costs.

Krohn and Slosson (1976) estimated the annual landslide damage to privately owned buildings and their sites in the United States to be about $400 million (1971 dollars), a figure not including other damages such as those to public property, transportation facilities, mines, agricultural and forest lands, etc. Nor did it include any indirect costs. Thus, the $400 million is only a fraction of direct and indirect, public and private annual costs of landslide damage for the United States at that time. Jahns (1978) studied the combined costs of damages from landslides and subsidence in the

United States for the period 1925-75 and reported that direct losses were at least $75 billion. This figure was more than three times the combined losses from floods, hurricanes, tornadoes, and earthquakes during the same period.

Using the above information, plus indirect costs and previously unpublished data, Schuster (1978) concluded that direct and indirect losses to public and private property in the United States exceeded $1 billion per year. By 1983 this annual figure probably has increased to nearly $1.5 billion, most of the increase being due to inflation.

N. Ohhira, Director-General of the Japanese National Research Center for Disaster Prevention, has noted that annual losses in Japan due to landslides and mudflows also total about $1.5 billion (oral commun., 1982). The unpublished results of a 1976 UNESCO survey of landslide effects indicated that annual landslide losses in Italy were about $1.14 billion (M. Arnould, written commun., 1982). Based on estimated annual landslide damages of $100 million for 10,000 km of highways and roads in hilly and mountainous parts of northern India, Mathur (1982) has arrived at an annual cost of landslide damages of about $1 billion for the total 88.979 km of roads in this landslide-prone area. In addition to the commonly used reconstruction and maintenance costs, Mathur's estimates include indirect costs such as loss of man and vehicle hours resulting from road blockage, property loss, loss of tourist trade, and failure of communications, that have not generally been included in determining the estimates for the United States, Japan, and Italy. The largest of these indirect costs, loss of man and vehicle time, by itself constitutes $350 million of the above figure. Thus, the given landslide losses for India cannot be compared directly to those of the other three countries. It does appear, however, that the annual costs of landslides in the United States, Japan, Italy, and India are of the same order of magnitude. To the authors' knowledge. similar estimates of total annual losses are not available for most other countries, but they probably are considerably smaller than those noted above.

Somewhat more accurate cost estimates can be made for landslides in smaller geographic areas. Arnould and Frey (1978) noted instances of costs of individual landslides in various parts of the world; however, they were unable to obtain significant data from several of the countries most severely affected by slope failures.

In the United States the most financially devastating slope failures have been those triggered by the 1964 Alaska earthquake, the 1980 landslides in southern California, and the 1982 landslides in the San Francisco Bay area of California. Youd (1978) estimated that ground failure caused 60 percent of the $300 million total damage from the 1964 Alaska earthquake; a very high percentage of the ground failure consisted of landslides, including lateral spreads. Five major landslides caused about $50 million of the $85 million damage to nonmilitary facilities in Anchorage, Alaska's largest city. The extent of damage to highways, railways, and bridges from lateral-spread failures amounted to about $50 million. Flow failures in three Alaskan coastal communities carried away port facilities that cost about $15 million.

In 1980, the estimated total losses in the six southern counties of California due to landslides, debris flows, mudflows, and related slope failures caused by high-intensity rainfall approximated $500 million (Slosson and Krohn, 1982). In addition to catastrophic floods, the intense storms of early January 1982 in the San Francisco Bay area triggered thousands of debris flows and a few large landslides. About 30 people were killed and hundreds left homeless in this catastrophe, and property losses due to landslides, debris flows, and associated floods exceeded $280 million, most of which was probably caused by the landslides and debris flows (Smith and Hart, 1982). Some 6,500 homes and 1,000 businesses were destroyed or damaged.

Slope failures have been particularly costly in urban areas. For example, costs of landslides in Allegheny County (Pittsburgh), Pennsylvania, were estimated to be about $2 million annually for the period 1970-74 (Briggs and others, 1975). As another example, a recent survey by the U.S. Geological Survey of costs of landslide damage in San Diego County, California, during the rainy seasons of 1978-79 and 1979-80, documented 120 landslides that caused damages of about $19 million.

In many parts of the world, damages to transportation facilities, mainly highways and railways, constitute a significant part of total landslide and mudflow costs. However, the exact magnitude of these losses often is difficult to obtain because (a) many smaller slope failures are repaired by routine maintenance operations from which landslide and mudflow costs cannot be separated from other maintenance costs, and (b) a significant percentage of slope-failure costs for transportation systems consists of difficult to quantify indirect costs such as traffic disruption and delays, inconvenience to motorists, engineering costs for investigation, and analysis and design of protective measures. For the United States, the Federal Highway Administration in 1973 estimated $100 million as a conservative figure for the total annual cost of slope-failure damage to highways and roads (Chassie and Goughnor, 1976). No parallel figure is available for railroads; however, it undoubtedly is much smaller, because in landslide-prone areas of the United States the total length of railways is much shorter than that of roads and highways.

Costs of landslide damages to highways and roads in India also are available. In addition to the previously cited general cost data by Mathur (1982), Chopra (1977) has noted that catastrophic damages to roads in North Bengal and Sikkim occurred in 1968 and 1973; restoration works were estimated to cost $14 million and $8 million, respectively. For Nepal, Charma (1974) has documented cases of individual landslides that have killed hundreds of people, displaced more than a thousand families, and cost hundreds of thousands of dollars in damages; much of this damage has been to roads and highways.

For planning purposes, other studies have attempted to project costs of landslides into the future. In a study projecting the costs of geologic hazards in California from 1970 to 2000, the California Division of Mines and Geology estimated that the costs of slope movements in California during that period would be almost $10 billion, or an average of over $300 million per year (Alfors and others, 1973). This estimate was based on the assumption that California mitigative practices in use in 1970 would remain in practice. Figure 1, taken from the California study, shows the estimated

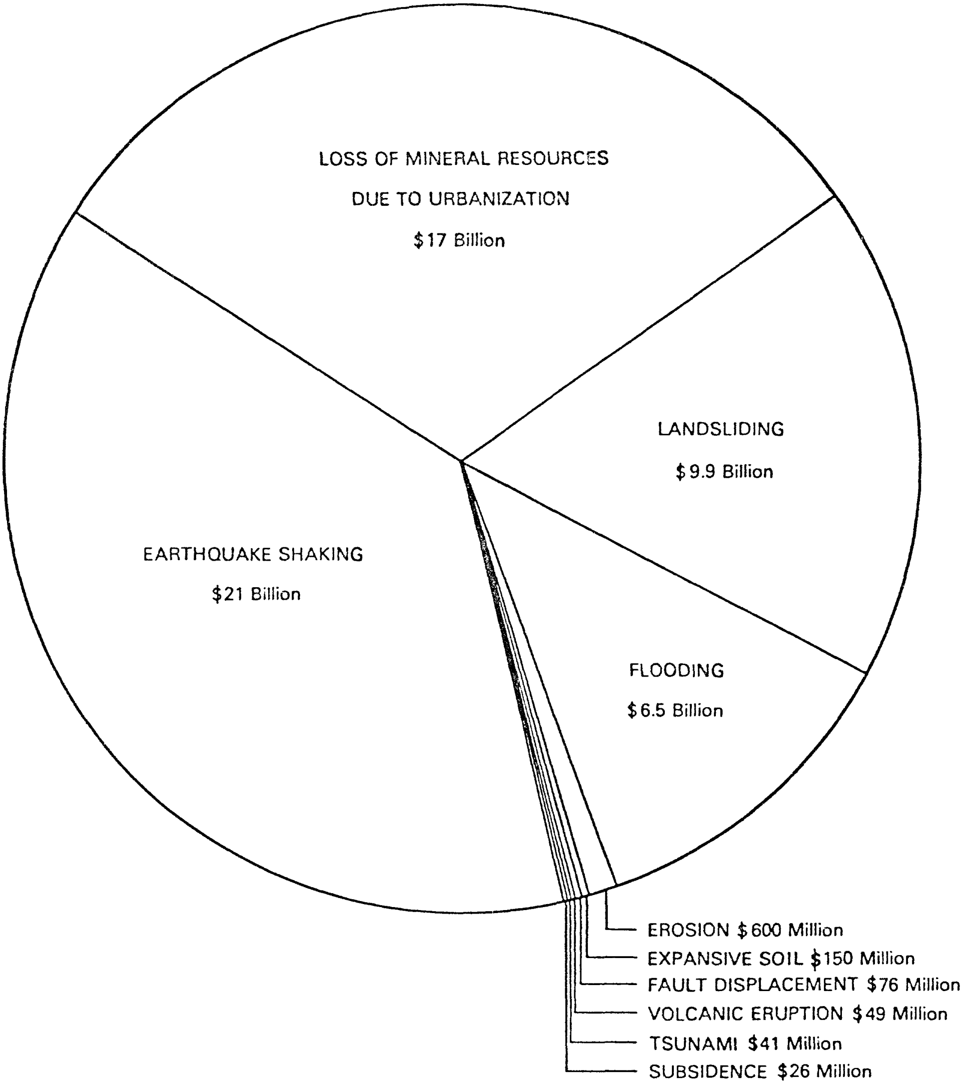

Figure 1.--Predicted economic losses from geologic hazards and urbanization in the State of California, United States, from 1970 to 2000, based on current loss-production practices remaining unchanged (Alfors and others, 1973).

losses due to all types of landslides, including debris flows and mudflows compared to losses due to other geologic hazards and urbanization. Of the so-called "catastrophic" geologic hazards included in the study, losses due to slope failures exceed those predicted for floods and, in turn, are exceeded by those due to earthquakes. California is particularly prone to earthquake activity, and this estimate assumes that one or more catastrophic earthquakes will occur in a major California metropolitan area during the 30-year period.

Casualties

In addition to the economic losses associated with slope movements, a significant loss of life can be attributed directly to catastrophic slope failures in many parts of the world. Fatalities due to catastrophic slope failures have been recorded since people began to congregate in areas subject to such failures, and the burgeoning population of the world in this century has worsened the problem. Varnes (1981) has noted that during the period 1971-74, an average of nearly 600 people per year were killed by slope failures worldwide. Interestingly, about 90 percent of those deaths occurred within the Circum-Pacific region (i.e., in these areas in or on the margins of the Pacific basin). As an example of annual casualties in one country, Krohn and Slosson (1976) have estimated that the total loss of life in the United States from all forms of landslide and mudflow activity exceeds approximately 25 lives per year, a total greater than the average number of deaths due to earthquakes. An analysis of a UNESCO international questionnaire on mass movements provides information on casualties caused by landslides in several other countries (Arnould and Frey, 1978).

Within recorded time, the greatest loss of life due to any single group of landslides has been the aforementioned earthquake-triggered loess landslides in Kansu Province, China, in 1920. Official Chinese estimates of the loss of life from this earthquake due to landslides, collapsed cave homes, falling buildings, and exposure to the harsh winter climate was 200,000 (Close and McCormick, 1922). The description of Close and McCormick suggests that as many as 100,000 were killed by landslides.

Probably the best known of recent catastrophic landslides are the debris avalanches of 1962 and 1970 on the slopes of Mt. Huascaran in the Cordillera Blanca of Peru. In January 1962, a large debris avalanche, which started as an ice avalanche from a glacier high on the north peak of Mt. Huascaran, but soon became a mixture of water, ice, rocks and soil, roared through mountain villages, killing some 4,000-5,000 people (Cluff, 1971). Eight years later, in a sequel to this tragedy, an even greater number were killed when a magnitude 7.75 earthquake occurred off the coast of Peru, triggering another disastrous debris avalanche on the slopes of Huascaran (Cluff, 1971; Plafker and others, 1971). This debris avalanche descended at average speeds of roughly 320 km/hr into the same valley but over a much larger area, burying the towns of Yungay and Ranrahirca, killing over 18,000 people.

In 1974, another massive landslide in the Andes Mountains of Peru killed approximately 450 people, mostly in the village of Mayunmarca (Hutchinson and Kojan, 1975). This landslide, which occurred in the valley of the Mantaro River, had a volume of 1.6 Gm^3, making it one of the largest in recorded history. It temporarily dammed the Mantaro River, forming a lake with a depth of about 170 m and a length of about 31 km. The key road along the river above the landslide dam was destroyed by many "reservoir-induced" landslides, and, as of 1982, had not been reopened. In overtopping and breaching the dam, the river caused extensive damage downstream destroying approximately 20 km of road, three bridges, and many farms.

In Europe, the most disastrous landslide in recent time has been the 1963 Vaiont Reservoir slide in northeastern Italy. This reservoir-induced massive rockslide having a volume of about 250 Mm^3 slid at high velocity into the reservoir, sending a wave 260 m up the opposite slope and at least 100 m over the crest of the dam into the valley below, where it destroyed five villages and took 2,000-3,000 lives (Kiersch, 1964).

Among industrialized nations, Japan probably has suffered the largest continuing loss of life and property from landslides and other slope movements. The oldest recorded landslide damage in Japan occurred in A.D. 989; many people were killed and injured in this event, and many houses were destroyed (Fukuoka, 1982). Although a few slope failures in Japan are caused by earthquakes, most are a direct result of heavy rains during the typhoon season. When urban areas are stricken by rapid landslides, extensive damage may be expected to occur. For example, in July 1938, Kobe, one of Japan's largest cities, was swept by debris flows generated by torrential rainfall, resulting in 450-600 deaths and destruction of more than 100,000 homes (Nakano and others, 1974; Fukuoka, 1982; Japan Ministry of Construction, 1983).

Similarly, 1,154 people were killed and nearly 2,000 homes destroyed or badly damaged in the City of Kure in 1945 by debris flows generated by heavy rains accompanying a typhoon (Nakano and others, 1974; Japan Ministry of Construction, 1983). Other examples of deaths and damage in Japan due to slope failures for the 16-year period from 1967-82 are provided in Table 1. Table 2 indicates losses due to the most catastrophic landslides and mudflows in Japan from 1945-76 (Japan Ministry of Construction, 1983); note that Japan is affected almost annually by catastrophic slope failures resulting in large loss of life and property.

The Soviet Union also has experienced large loss of life due to landslides and mudflows, most of the deaths occurring in isolated catastrophes. The greatest catastrophe occurred in 1949, when an intensity 9 Bals, magnitude 7-1/2 earthquake in the Tien Shan Mountains of Soviet Tadzhikistan triggered a series of massive landslides and debris flows that buried some 33 population centers. Estimates of the number of deaths in this disaster range as high as 12,000 (Jaroff, 1977) to 20,000 (Wesson and Wesson, 1975). Some large cities of the U.S.S.R., such as Alma-Ata, Erevan, Dushanbe, and Frunze, are located in dangerous debris-flow and mudflow areas (Gerasimov and Zvonkova, 1974). In 1921, a large debris flow in Alma-Ata, the capital of the Kazakh Republic, killed 500 people and inflicted considerable damage to the city (Yesenov and Degovets, 1982).

Although there have been some very large and catastrophic slope failures in North America, most of them have occurred in mountainous, relatively unpopulated areas; thus these failures commonly have not resulted in major losses of life. However, there have been several notable exceptions in this century. The first was in Canada in 1903 when a great rockslide killed about 70 people in the coal mining town of Frank, Alberta (McConnell and Brock,

Table 1.--Deaths in Japan from 1967-82 due to (A) flood and all types of slope failures and (B and C) slope failures only. For this 16-year period, slope failures caused 55 percent of all deaths due to natural disasters. (After Japan Ministry of Construction, 1983)

Year	(A) Deaths due to floods and all types of slope failures	(B) Deaths due to mudflows and debris flows	(C) Deaths due to other types of slope failures	Percentage of deaths due to slope failures $\left(\frac{B+C}{A} \times 100\%\right)$
1967	603	297	158	75.4
1968	259	154	5	61.4
1969	183	32	82	62.3
1970	175	22	27	28.0
1971	376	53	171	59.6
1972	637	194	239	68.0
1973	81	19	18	45.7
1974	239	40	129	70.7
1975	202	71	49	59.4
1976	242	72	81	63.2
1977	54	12	8	37.0
1978	110	16	24	36.4
1979	202	4	23	13.4
1980	114	0	25	21.9
1981	92	13	20	35.9
*1982	508	152	185	66.3
Total deaths----	4,077	1,151	1,244	
Average deaths/year---	255	72	78	
Overall percentage of deaths due to slope failures-----				58.7

*1982 unpublished data provided by the Japan Ministry of Construction.

Table 2.--Losses due to major slope failure disasters in Japan, 1938-81. All of these disastrous mass movements were caused by heavy rainfall, most commonly related to typhoons; none were triggered by earthquakes or volcanic activity. (From Japan Ministry of Construction, 1983.)

Date	Prefecture	Severely affected area	Number of dead or missing	Number of houses destroyed or badly damaged
July 1938	Hyogo-------	Mount Rokko (Kobe area)------	505	130,192
July 1945	Hiroshima---	Kure City and its environs---	1,154	1,954
Sept. 1947	Gumma-------	Mount Akagi------------------	271	1,538
July 1951	Kyoto-------	Kameoka----------------------	114	15,141
June 1953	Kumamoto----	Mount Aso--------------------	102	------
July 1953	Wakayama----	Arita River------------------	460	4,772
Aug. 1953	Kyoto-------	Minamiyamashiro--------------	336	5,122
Sept. 1958	Shizuoka----	Kanogawa River---------------	1,094	19,754
Aug. 1959	Yamanashi---	Kamanashi River--------------	43	277
June 1961	Nagano------	Ina Valley Region------------	130	3,018
Sept. 1966	Yamanashi---	Lake Saiko-------------------	32	81
July 1967	Hyogo-------	Mount Rokko------------------	92	746
July 1967	Hiroshima---	Kure City and its environs---	88	289
July 1972	Kumamoto----	Amakusa Island---------------	115	750
Aug. 1972	Niigata-----	Kurokawa Village-------------	31	1,102
July 1974	Kagawa------	Shodo-shima Island-----------	29	1,139
Aug. 1975	Aomori------	Mount Iwaki------------------	22	28
Aug. 1975	Kochi-------	Niyodo River-----------------	68	536
Sept. 1976	Kagawa------	Shodo-shima Island-----------	119	2,001
May 1978	Niigata-----	Myoko-Kogen------------------	13	25
Oct. 1978	Hokkaido----	Mount Usu--------------------	3	144
Aug. 1979	Gifu--------	Horadani---------------------	3	16
Aug. 1981	Nagano------	Ubara------------------------	10	56

1904). A more recent Canadian catastrophe was the 1971 debris flow in sensitive clay that demolished part of the town of Saint-Jean-Vianney, Quebec, destroying 40 homes and killing 31 people (Tavenas and others, 1971).

In terms of casualties, one of the worst natural disasters to hit the United States this century was the 1969 flooding and associated debris flows in central Virginia from Hurricane Camille (Williams and Guy, 1973). Although no exact number can be ascertained, most of the 150 who died as a result of Hurricane Camille are thought to have been victims of the debris flows resulting from the hurricane.

Another major catastrophe involving landsliding in the United States was the 1972 Buffalo Creek dam failure in West Virginia (Davies, 1973). A debris flow resulting from slope failure of three coal-refuse impoundments due to heavy rains traveled 24 km downstream, killing 125 people and leaving 4,000 homeless.

Interestingly, the world's two largest landslides in recent history have resulted in a relatively small number of casualties. The 1911 Pamir landslide in Soviet Tadzhikistan (then Russia), with an estimated volume of 2.5 Gm^3, was a truly catastrophic event. However, in spite of its great volume, and apparently high velocity, casualties were low because the area was sparsely inhabited. Most of the casualties apparently occurred in the village of Usoy, whose 54 inhabitants were buried (Bolt and others, 1975, p. 178-179). In May 1980, a massive rockslide/debris avalanche with a volume of 2.8 Gm^3 descended at high velocity from the north slope of Mount St. Helens, State of Washington, United States, as a result of the eruption of the mountain (Voight and others, 1981). The debris avalanche progressed about 22 km westward, burying the North Fork Toutle River valley under about 60 km^2 of poorly sorted debris. Although it buried and destroyed nine highway bridges, many kilometers of highway and roads, and numerous public and private buildings (Schuster, 1981), only 5-10 people were killed by the debris avalanche. The low casualty rate was a direct result of the evacuation of residents and visitors in anticipation of a possible eruption.

Programs to Reduce Economic Losses Due to Landslides and Mudlows

Probably the most comprehensive national program of landslide loss reduction has been that of Japan where a strong national program for landslide control has developed since World War II (Japan Society of Landslide, 1980). Initially, landslide control activities were tied to other legislation; for example, river improvement, erosion control, and agricultural and forest land maintenance. The first program in Japan devoted exclusively to landslides began with the 1958 "Landslide Prevention Law." Other legislative measures have been adopted since 1958, culminating in 1969 with the "Law for the Prevention of Disasters Caused by the Collapse of Steep Slopes." This legislation provides for governmental assumption of expenses and guidance for recovery from natural disasters for which no individuals bear responsibility. The measures provide not only for repair of damage and restoration of property to original form, but also for construction efforts to prevent future slope failures. The estimated annual cost of this program is about $500 million (N. Ohhira, personal commun., 1982).

Other countries also have developed national programs. Some have developed the technical data base necessary to recognize areas of differing likelihood of failure. For example, in France the ZERMOS (Zones Exposed to Risks of Movements of the Soil and Subsoil) plan is responsible for production of landslide-hazard maps at scales of about 1:25,000 or larger. These maps portray degrees of risk of various types of slope failures, and include activity, rate, and potential consequences (Humbert, 1977). Recently the scope of the ZERMOS plan has been enlarged to provide guidelines for suitable locations of development and permissible land use (Porcher and Guilliope, 1979).

Insurance programs may reduce losses due to slope failures. At a minimum, they reduce the impact to individual property owners by spreading the losses over a larger base. Insurance that includes standards for site selection and construction techniques can actually reduce total losses.

In New Zealand, a national insurance program assists homeowners whose dwellings have been damaged by landslides or other natural hazards that are not within the reasonable control of homeowners to prevent (Arnould, 1976). This natural-hazard insurance program is an outgrowth of the Earthquake and War Damage Act of 1944. A disaster fund, accumulated from a surcharge to the fire insurance program, reimburses property owners for losses (O'Riordan, 1974).

In the United States, the National Flood Insurance Program (NFIP) was created by the Housing and Urban Development Act of 1968. An amendment extended the application to "mudslides" in 1969. The range of phenomena the term "mudslide" encompasses was not clear; however, "mudslide" commonly has been used by southern California news media to refer to a variety of landslide phenomena that includes mudflows (Committee on Methodologies for Predicting Mudflow Areas, 1982). As presently worded, the regulations include mudslides (i.e., mudflows) that are proximately caused or precipitated by accumulations of surface or ground water. The insurance on these "water-caused" landslides is provided by private insurance companies, but is little used; however, it is underwritten and subsidized by the federal government. Other than the NFIP, landslide insurance generally is not available in the United States.

SUMMARY

Estimates of economic losses due to landslides and mudflows have been reported for Japan, Italy, and the United States, which show annual costs exceeding U.S. $1 billion each. Unfortunately, for most other countries comprehensive economic data on landslide and mudflow losses are not readily available except for scattered information on major catastrophes. Similarly, data on loss of life from landslides and mudflows are incomplete. Where they have been compiled, the year-to-year totals are surprisingly large. In Japan, during the period 1967-82, an average of 150 people was killed annually by landslides and mudflows, a total greater than the number killed by floods. In the United States about 25 people are killed annually by landslides and mudflows. Major landslides in China, Italy, Peru, and the Soviet Union have killed from a few thousand to many thousands of people in single events during this century. Socioeconomic data from other countries of the world, particularly from developing countries, are urgently needed as a basis for making priorities for hazard mitigation activities.

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TECHNIQUES FOR REDUCING LANDSLIDE HAZARDS*

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INTRODUCTION

Numerous techniques for reducing landslide hazards are available to planners, engineers, and decisionmakers. Some of these techniques are well known to the engineering profession, such as structures to control landslides; or to the planning profession, such as public acquisition of hazardous areas. Others are obvious and practical, such as warning signs and regulations, but these require consistant enforcement. Still others are innovative when applied to landslides, but have been successfully used in solving flood and soil problems. These and other techniques are listed in the table under the headings of discouraging new development, regulating new development, and protecting, converting, or removing existing development. The techniques may be used in a variety of combinations to help solve both existing and potential landslide problems.

A prerequisite for the successful use of any of these techniques is the availability of adequate, detailed information in a form and language understandable to nongeologists. Recognition and identification of landslides are discussed by Rib and Liang _in_ Schuster and Krizek, eds. (1978, p. 34-80). The slope-stability maps prepared by Nilsen and others (1979) for the nine-county San Francisco Bay region are examples of the types and scale of hazard maps adequate for county and regional planning and decisionmaking. The relevance and application of these maps to land-use planning are discussed by

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Vlasic and Spangle in the report by Nilsen and others (1979, p. 55-87). The feasibility and cost of using a computer to prepare landslide susceptibility maps are discussed and illustrated in a report by Newman, Parodis, and Brabb (1978).

Table

SOME TECHNIQUES FOR REDUCING LANDSLIDE HAZARDS

Discouraging new development in hazardous areas by:

- Adopting public-facility and utility-service policies
- Creating special assessment districts
- Disclosing the hazard to real-estate buyers
- Informing and educating the public
- Posting warnings of potential hazards
- Providing financial disincentives
- Recording the hazard on public records
- Requiring nonsubsidized landslide insurance related to the level of hazard

Regulating new development in hazardous areas by:

- Adopting grading ordinances
- Adopting hillside-development regulations
- Adopting sanitary ordinances
- Adopting subdivision ordinances
- Creating special hazard reduction zones and regulations
- Requiring appropriate land-use zoning districts and regulations

Protecting existing development in hazardous areas by:

- Controling landslides
- Directing mudflows
- Operating systems for monitoring, warning, and evacuating

Removing or converting existing unsafe development through:

- Acquiring or exchanging hazardous areas
- Abating public nuisances
- Discontinuing nonconforming uses
- Reconstructing public facilities
- Redeveloping blighted areas

DISCOURAGING DEVELOPMENT

There are several ways to discourage development in hazardous areas. These include public information and education, posting warning signs, recording the hazard on public records, levying special assessments, adopting lending policies or public-facility extension policies, disclosing hazards to real-estate buyers, or requiring the payment of insurance premiums. Any of these may be used to discourage developers and home buyers from locating projects or purchasing homes in landslide areas.

Public Information and Education

Public information programs can help to bring landslide information to the attention of the public. Prudent citizens, when told of slide hazards, may not wish to risk property losses and expose their families to the danger and trauma of a landslide. Since any program of land-use control depends on the support of an informed public, educating the public is of great importance. Preparing, announcing, and disseminating information on slide damage and susceptibility can be done through conferences, workshops, newsletters, press releases, bulletins, and letters to key officials.

Another way to disseminate information on slide damage and site instability is through the use of maps, such as those showing five zones of instability in urbanized areas of West Virginia. A bulletin accompanying these maps (Lessing and others, 1976) advises buyers, builders, and homeowners about "danger signals" and slide correction methods, and explains the legal responsibilities of state agencies.

Warning Signs

Warning signs that are posted and readily visible to buyers, developers, and the public help alert prospective developers and purchasers who visit the site. Such signs should be based upon adequate data and be posted where the slide areas intersect or abut public rights-of-way. Similar warning signs have been used by the U.S. Bureau of Reclamation to identify flood-hazard areas on the Sacramento River near Redding, California.

Warnings can also take the form of rubber-stamp notations. When these notations are placed on subdivision plats or on building and zoning permits, they will alert potential buyers that the sites are in slide areas.

Recording the Hazard

Public records on land ownership also help alert land purchasers, local assessors, and lenders to potential slide hazards. Such records can be made by filing maps of slide areas with the appropriate recorder of deeds, along with listings of the subdivisions or the sections (as identified under a public land survey system), and requesting entry onto tract indexes. Abstracts of titles for affected properties and subsequent conveyances then would contain an entry referring to the hazard. If subdivision ordinances requiring identification of slide areas were adopted, such information would be filed automatically with a recorder of deeds.

In Santa Clara County, California, if the required geological report indicates hazardous conditions, the property owner must sign a statement before any development takes place. This statement acknowledges that the owner is aware of the hazards, accepts the risks of development, and relieves the county of liability. The statement is recorded in the county recorder's office and can be expunged only if subsequent information -- approved by the county geologist -- indicates that the hazard no longer exists or has been reduced. However, no new structures for human occupancy can be located on active landslides until they have been stabilized by acceptable engineering practices (Santa Clara County Board of Supervisors, 1978).

Special Assessments

If it becomes necessary to construct public works to control or prevent slides, or to repair damaged public facilities, the costs could be assessed wholly or in part against the lands that will benefit from the construction. For instance, the city of San Jose anticipated subsidence damage to public roads and utilities in an area with new development. The city council passed an ordinance and resolution establishing a 17.9 acre maintenance district prior to the development of the area, and the city's director of public works prepared a budget, an assessment formula, and a proposed assessment. These were adopted by the city council after public hearings and validation by the Superior Court (San Jose City Council, 1979).

The California Legislature (1979) provides for the formation of geologic-hazard abatement districts. When created, these districts are authorized to acquire real estate; acquire, construct, operate, manage, or maintain improvements; and pay for such improvements. Improvements are defined as "any activity necessary or incidental to the prevention, mitigation, abatement, or control of a geologic hazard." Such a hazard is defined as "an actual or threatened landslide, land subsidence, soil erosion, or any other natural or unnatural movement of land or earth." A district is comprised of an area specially benefited by and subject to special assessment to pay the cost of an improvement.

Financal Disincentives

Private lenders and government agencies can help reduce slide hazards by denying loans or requiring insurance for development in slide-prone areas. Almost all construction today involves loans or mortgages by private lenders. Many of these loans are insured by government agencies, such as the Department of Housing and Urban Development, and the government has defined minimum requirements for eligibility for federally insured mortgages and housing aasistance (U.S. Department of Housing and Urban Development, 1973). These include a mandated minimum level of construction and design. Natural hazards like landslides are also considered. Private lending institutions could take a similar approach. To do so, however, they need to have accurate information and risk and an assessment of risk.

It is also possible to use tax credits to discourage development. In agricultural areas, state farmland-preservation laws could provide encouragement to leave hazardous lands undeveloped. For example, the Wisconsin Legislature (1977) provides for a state income-tax credit of up to $4,200 annually as an incentive to farmers to preserve farmland. The credit doubled in 1982 if a county created an exclusive agricultural zone.

Public-Facility and Utility-Service Policies

Local governments and utilities could adopt policies that prohibit financing and constructing public facilities or utilities in potentially hazardous areas. If such areas lacked streets, utilities, and other community facilities, they would be far less likely to attract development activities. This technique could be used in conjunction with other regulatory techniques, such as land-use regulations. However, to be fair to prospective developers, the action needs to be taken long before any development plans are made.

Disclosure

Preparing landslide-hazard information for nongeologists and enacting federal, state and local hazard-disclosure laws can make many people aware of slide hazards. Disclosing such hazards at the time of purchase can alert property owners to the potential danger and loss. For example, to provide for protection against flood losses through a Federally subsidized flood-insurance program, the U.S. Congress requires lenders to notify prospective borrowers that the real estate being mortgaged is located in flood-hazard areas, as identified by the federal insurance administrator (U.S. Congress, 1974).

To provide for public safety from earthquakes, the California State Legislature (1972) requires a seller or his agent to tell the prospective buyer that the real estate is located within a fault-rupture zone, as delineated by the state geologist.

In the ordinance enforcing on-site geologic investigations before construction, the Santa Clara County Board of Supervisors (1978) also requires all sellers of real estate lying partly or wholly within the county's flood, landslide, and fault-rupture zones to provide the buyer with a written statement of the geologic risk.

To help Realtors* comply with these Federal, State, and county disclosure laws, five local Boards of Realtors in the San Francisco Bay area prepared

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*A Realtor is a member of the National Association of Real Estate Boards.

street maps showing some or all of the flood, landslide, and fault-rupture zones. The five maps cover one entire county and parts of three others, and include more than 50 cities. In addition, the San Jose Board of Realtors has designed a form (San Jose Board of Realtors, 1978) to be attached to a real-estate contract to comply with the county disclosure ordinance. This example of disclosure is discussed and illustrated by Brown and Kockelman (1983, p. 84-91).

Prerequisites for effective disclosure of hazards by real-estate sellers include a seller's or real estate agent's knowledge and integrity, a buyer's awareness of the potential danger or financial loss before making the commitment to purchase, and a buyer's concern about hazards vis-a-vis his or her other priorities. According to Palm (1980), disclosure of fault-rupture hazards by real estate agents in Berkeley and Contra Costa County, California, indicates that these prerequisites are often lacking.

Cost of Insurance

The cost of landslide insurance can discourage development in hazardous areas or encourage land uses that are less likely to experience damage. Landslide insurance from private sources is costly for areas with known landslide potential because landslides lack the random nature necessary for a sound insurance program. In this respect, slide areas are comparable to flood areas, and the report on flood insurance prepared by the American Insurance Association (1956) may be applicable to slide areas:

> Flood insurance covering fixed-location properties in areas subject to recurrent floods cannot feasibly be written because of the virtual certainty of loss, its catastrophic nature, and the reluctance or inability of the public to pay the premium charge required to make the insurance self-sustaining.

Therefore, insurance can be made available at reasonable rates in slide-prone areas only if subsidized. In addition, there is the problem that government subsidies for property owners who suffer slide damage can lead to highly undesirable development in anticipation of indemnity.

The Federal government attempts to discourage development in mudflow areas through the National Flood Insurance Program. As amended by Congress, this program requires that some kind of flood insurance must be purchased before any form of Federal financial assistance will be given for construction or acquisition in identified mudflow-hazard areas (U.S. Congress, 1968).

REGULATING DEVELOPMENT

Land-use regulations can be used to reduce landslide hazards; these are often the most economical and effective means a local government can use. It is unrealistic to assume that development can be discouraged indirectly for a long period, and other measures, such as building protective structures or purchasing the land, can be very costly.

A community, through its development regulations, can prohibit or restrict development in landslide areas. It can zone landslide areas for open-space uses like parks, grazing, or certain types of agriculture. Or, if more vulnerable development is allowed in slide areas, the intensity of development can be kept to a minimum -- both to reduce its effects and the potential for property damage. Zoning, subdivision, and sanitary regulations can be used to meet these objectives.

Land-Use Zoning Districts

Land-use is an accepted way to control development. It can provide direct benefits by restricting future development of vacant lands in slide-prone areas and by limiting the expansion of existing development in those areas.

Zoning districts that are most compatible with landslide-hazard areas include agricultural, open-space, conservancy, park, and recreation land-use zones. These districts permit such uses as grazing, woodlands, wildlife refuge, and public and private recreation. Provisions can be incorporated into district regulations to prohibit specific uses that could trigger slides or that would be vulnerable to slide damage. Examples of necessary prohibitions might include farm dwellings, off-road vehicles, irrigation, and the permanent sheltering or confining of animals.

District regulations can also reduce the density of development. In San Mateo County, for instance, the board of supervisors created a resources management zoning district to carry out the objectives and policies of their open-space and conservation plans. The district regulations limit the number of dwellings in slide-prone areas to one unit per 40 acres (10 hectares). However, the number of dwellings permitted may be accumulated over large areas and clustered in the nonhazardous areas (San Mateo County Board of Supervisors, 1973).

Special Hazardous-Area Regulations

Land-use regulations on the use of slide areas can supplement the basic use and site regulations in zoning ordinances. They can be designed to:

1. Preserve vegetation, maintain drainage, control off-road vehicles, establish vibration performance standards, avoid the most hazardous slide areas, require clustering of dwellings, and reduce development densities.

2. Prohibit certain operations that increase loads, reduce slope support, or otherwise cause instability. These include such operations as filling, irrigating, disposing of solid and liquid wastes, and cutting away the toe of the slide.

3. Prohibit certain uses -- such as storage of radioactive, toxic, flammable, and explosive materials -- that could cause serious health and safety hazards.

The town of Portola Valley sets out criteria for land uses in unstable or potentially unstable areas (Nilsen and others, 1979). Roads, houses, utilities, and water tanks are prohibited in slide areas. The Town also requires use of the same criteria in administering it's subdivision, site development, and building ordinances. The town council subsequently adopted additional regulations that reduce the maximum number of dwellings permitted in areas of potential slides (Portola Valley Town Council, 1979).

Hillside-Development Regulations

Some communities have adopted regulations to limit the amount of development that may take place in hillside areas; sometimes these are added to grading regulations. The regulations usually include some formula for determining the density of development to be permitted for a given degree of slope.

Hillside density regulations are used both for safety and aesthetic reasons. Limited development protects the beauty of the hillsides. Low building densities minimize danger to slopes. Proper grading procedures, however, are needed to supplement the density restrictions. Correct excavation and fill practices are imperative to ensure slope stability. Without them, landslide potential remains the same for individual sites; only the number of affected sites is reduced. A geologist or geotechnical engineer is needed to make an accurate judgment about where slope density and other regulations can be useful. The steepness of the slope is not always the main determinant of slope stability. For instance, the nature of some of the soils around Washington, D.C., can make even gentle slopes highly susceptible to slides.

There are three principal types of hillside regulations:

1. Slope-density provisions that decrease allowable development densities as slope increases.

2. Soil-overlay provisions that assign use and density on the basis of soil characteristics in sloping areas.

3. The "guiding principles" approach that is relatively free of precise standards, but emphasizes case-by-case evaluation on the basis of a number of specific policies.

In Performance Controls for Sensitive Lands, these types are discussed in detail by Thurow and others (1975).

Subdivision Ordinances

Regulating the design and improvement of subdivisions is another way to control the development of landslide areas. A dilemma occurs when government officials approve a subdivision, accept public rights-of-way, extend utilities, and then attempt to impose zoning and other regulations that would prohibit further development. This dilemma can be avoided by adopting a subdivision ordinance designed to:

1. Require the delineation and designation of slide areas on subdivision plats and certified survey maps.
2. Require dedication or reservation of slide areas for public or private parks or other community purposes; and require dedication of, or easements along, those waterways necessary for adequate drainage.
3. Require that public and private roads, bridges, utilities, and other facilities be located or designed and constructed to avoid slide areas or to withstand anticipated movement.
4. Select road and utility alignments and grades to minimize cuts and fills.
5. Prohibit the creation and improvement of building sites in slide areas.

The Southwestern Wisconsin Regional Planning Commission (1969, Appendix F) has prepared a model ordinance for certain soils and geological conditions. The model ordinance has been adopted by many local units of government, and some have been in effect for over 10 years without successful legal assault.

Sanitary Ordinances

On-site soil absorption and sewage disposal systems (septic tanks, absorption fields, and seepage beds and pits) can trigger slides and are likely to become inoperative when slides occur. Sanitary ordinances can help eliminate some of the problems caused by the disruption of private sewage-disposal systems or the contamination of private water-supply systems. Useful ordinances can be designed to:

1. Require a permit before any such system is installed, and require that the permit application show the boundaries of slide areas.

2. Prohibit on-site soil-absorption sewage-disposal systems and private water-supply systems in slide areas.

3. Require that on-site soil-absorption sewage-disposal systems in slide areas be replaced with alternate systems, such as public sanitary sewage or holding tanks.

Another model ordinance by the Southwestern Wisconsin Regional Planning Commission (1969, Appendix H) is a good example of how soils and geological requirements can be incorporated into sanitary, health, and plumbing ordinances .

Grading Ordinances

Grading ordinances can be used to ensure that excavating, cutting, and filling in landslide areas are designed and conducted in such a way as to avoid overloading, cutting into the toe of the slide, or otherwise reducing its stability. Such goals can be obtained by a grading ordinance designed to:

1. Require a permit prior to scraping, excavating, filling, or cutting any lands.

2. Prohibit, minimize, or carefully regulate the excavating, cutting, and filling activities in slide areas.

3. Provide for the proper design, construction, and periodic inspection and maintenance of drainageways including culverts, ditches, gutters, and diversions.

4. Regulate the disrupting of vegetation and drainage patterns.

5. Provide for proper engineering design, placement, and drainage of fills, including periodic inspection and maintenance.

The success of the City of Los Angeles' slide-reduction program is discussed by Fleming, Varnes, and Schuster (1979, p. 434-437).

PROTECTING EXISTING DEVELOPMENT

Development has taken place in landslide areas and will continue to do so. Because many such areas offer highly desirable vistas, they are subject to development pressures. Nevertheless, there are several ways in which the land and residents in such areas can be protected. The three to be discussed here are landslide control, mudflow diversion, and monitoring and warning residents about imminent slide dangers.

Landslide Control

Property damage from landslides often leads to a demand for public works to provide protection for existing development. This includes constructing restraining structures to prevent further sliding; taking steps to control water problems that may be contributing to instability; or excavating areas on, or near, the sliding mass to stabilize it. This type of landslide control is usually limited to small slides because of the costs involved and the necessity for careful and accurate engineering design, inspection, and maintenance.

Current technical literature contains many examples and descriptions of these techniques; important references include: Reviews in Engineering Geology (Coates, 1977); Landslides: Analysis and Control (Schuster and Krizek, eds., 1978); and Landslide Remedial Measures (Royster, 1979, p. 301-352). The Allegheny County Department of Planning and Development has evaluated some of these tools and analyzed construction costs (A.W. Martin Asaociates, Inc., 1975).

Slide control can be self-defeating. Remedial public-works construction may encourage development by leading the public to believe that slide problems have been eliminated, not simply reduced. Also, such construction does not prevent earthquake-related slides. Intelligent management and regulation is still needed for slide areas.

Mudflow Divisions

One type of landslide, mudflows, sometimes can be diverted around development. Steps can also be taken to reduce the velocity of the flow. Specific measures include:

1. Covering the slopes in some way, such as with vegetation or mulch.
2. Building catchment basins and other structures to prevent damage to existing structures that cannot be economically removed or that have to be maintained, such as roads or utilities.

The Los Angeles County flood control district has experience in using some of these measures (Davis, 1979).

Monitoring, Warning, and Evacuating

Potentially unstable land can be monitored so that residents can be warned and, if necessary, evacuated. The most common forms of monitoring at present are field observation and electrical fences or trip wires. Other monitoring tools currently being tested include: vibration meters, television observation, guided radar, and laser beams.

Immediate relay of information is vital in areas where slides (such as rockfalls and debris flows) happen rapidly. Therefore, the U.S. Geological

Survey was delegated under the Disaster Relief Act (U.S. Congress, 1974) to develop the necessary warning system (U.S. Geological Survey, 1977). As a result, local, state and Federal officials in Billings, Montana, Kodiak, Alaska, and Wrightwood, California, have been warned of potential rockfalls, landslides on Pillar Mountain, and cyclic landsliding, respectively. Although the time, place, and magnitude of slides can be predicted only in relatively small areas in which detailed geological and engineering studies have been conducted, susceptible areas can be identified on a larger scale. The other techniques described may then be more appropriate.

REMOVING OR CONVERTING EXISTING UNSAFE DEVELOPMENT

Recurrent damage from landslides may be avoided by permanently evacuating areas that have undergone slides. Structures may be removed or converted to a use less vulnerable to slide damage. The feasibility of such action depends on the value of the structures, their potential for contributing to unstable conditions, whether they can be successfully reinforced, and the level of citizen concern. Techniques for removal or conversion include public acquisition, urban redevelopment, abatement of a public nuisance, nonconforming-use provisions in zoning ordinances, and reconstruction of existing public facilities.

Public Acquisition

A government agency may acquire the land through purchase, condemnation, tax foreclosure proceedings, dedication, devise (will), or donation. The agency can then control development in the public interest. It might choose to sell or lease part or all of it, stipulating that no structure be built that would be vulnerable to landslides. It might lease the land for crops or grazing, thus recovering part of the acquisition costs. In some cases, simple purchase may be more economical than extending and maintaining public services or utilities in slide areas.

The agency might also acquire less-than-fee interest in slide areas. This costs the public less than purchase because only certain property rights are purchased. Such interest may be in the form of scenic easement to protect vistas, conveyance of development rights to assure the continuation of existing private parks and open spaces, or grants of public access and development rights for construction and use of park facilities. The purchase of easement or development rights can limit development, and the owner receives fair compensation for the release of these rights. Easement lines do not require accurate landslide information (as does zoning, for instance) because the boundaries to be acquired can be determined by agreement. The use of easements should include periodic inspections and enforcement of the land use permitted by the agency holding the easement. Easements should be obtained in perpetuity or for as long as the slide hazard exists. Generally, easements have been found most effective in areas without heavy development pressures.

Numerous state and Federal financial aid programs are available for acquiring land for purposes compatible with slide hazards, such as park and recreation use. One example is the Federal Land and Water Conservation Fund Act (U.S. Congress, 1965) described in the Catalog of Federal Domestic Assistance (U.S. Office of Management and Budget, 1983, p. 414, 415).

Urban Redevelopment

Landslide areas can be redeveloped publicly or privately by purchasing land that has been determined to be blighted or to have deteriorated. State laws authorizing the creation of public agencies usually provide for the preparation and adoption of redevelopment plans, acquisition, clearance, disposal, reconstruction, and rehabilitation of blighted areas, and relocation of persons displaced by a redevelopment project. Redevelopment agencies usually are empowered to issue bonds, receive a portion of taxes levied on property in the project, and use federal grants or loans available under various programs of the Federal Housing and Community Development Act. These programs are discussed in the Catalog of Federal Domestic Assistance (U.S. Office of Management and Budget, 1983, p. 360-362, 365, 366).

Public Nuisance Abatement

Buildings and structures damaged by minor slides, such as creep and swelling soils, tend to initiate a cycle of deterioration because they often go unrepaired. Government agencies can remove such structures by applying their public nuisance abatement powers. Section 203 of the Uniform Building Code (International Conference of Building Officials, 1979, p. 25), adopted as a model by many cities and counties, gives an example of how this can be done:

> All buildings or structures regulated by this code which are structurally unsafe or not provided with adequate egress, or which constitute a fire hazard, or are otherwise dangerous to human life are, ... unsafe. All such unsafe buildings ... are hereby declared to be public nuisances and shall be abated by repair, rehabilitation, demolition, or removal ....

Nonconforming-Use Provisions

New or amended zoning ordinances may mean that some existing land uses become nonconforming. Nonconforming uses are defined as those that exist at the time a zoning ordinance is adopted or amended and do not conform to the use restrictions. For example, if an ordinance prohibits residences within a slide area, residences existing in the area at the time of adoption become nonconforming.

Zoning ordinances may provide that nonconforming uses can be continued but not extended or enlarged. They may stipulate that, if the current uses are discontinued for a designated period, any future use must conform with the ordinance. Regulations may also require that total structural repairs or alterations over the lifetime of a nonconforming building be limited to a percentage of the assessed or market value. Enabling legislation may also permit the elimination of nonconforming uses by providing for amortization over a reasonable period of time. This concept and practice of eliminating nonconforming uses are discussed in The Effect of Nonconforming Land-Use Amortization (Scott, 1972). Model ordinance provisions and comments on the discontinuance of existing land uses are described in A Model Land Development Code (American Law Institute, 1975, p. 142-166).

Public-Facility Reconstructions

Reconstructing public facilities located in slide areas may afford an opportunity to reduce the risk of damage from landslides. Such facilities could include roads, bridges, utilities, and community facilites that are subject to rebuilding by reason of functional or structural obsolescence. This end can be achieved by reinforcing, designing to accommodate displacement, relocating in areas not subject to landslides, or bridging. Bridging refers to the construction of spans over slide areas. It is primarily used for highways; it is expensive and consequently is used only as a last resort.

CONCLUSION

Even if adequate landslide research is available, presented in a language understandable by nongeologists, effectively communicated, and properly used, the lasting effectiveness of any landslide-hazard reduction technique depends upon many other factors, for example:

- Continued awareness and interest by the public
- Careful revision of enabling legislation if needed by state legislatures
- Accurate site investigations by registered geologists or geotechnical engineers
- Conscientious administration of regulations by inspectors
- Consistent enforcement by government attorneys
- Sustained support of enforcement officials by local political leaders
- Judicious adjustment of regulations by administrative appeal bodies
- Skillful advocacy (if challenged) and proper interpretation by the courts
- Concern for individual, family, and community safety by real estate buyers, developers, insurers, and financiers.

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QUANTITATIVE ASSESSMENT OF LANDSLIDE HAZARD FOR NATURAL RESOURCES MANAGEMENT

by

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Introduction

To effectively reduce landslide hazard, decisionmakers need information that is technically accurate and understandable. Both factors should be of equal concern to geologists responsible for advising decisionmakers. As technical specialists, we are inclined to concentrate on developing better landslide data rather than on improving communication of this information. This neglects the fact that most users, i.e., decisionmakers, are non-geologists unfamiliar with terminology, principles, and implicit assumptions involved with landslide hazard assessment efforts. The most effective methods for assessing landslide hazards are based on a sound technical approach and presentation of the results in a readily understood manner appropriate to the user's needs. Examples can be found illustrating significant reduction in losses from landslides through a cooperative effort by technical specialists and decisionmakers (Fleming and others, 1979).

Natural Resources Management

I advise managers of a National Forest. They make decisions directing a wide variety of activities affecting the land and its natural vegetation, water, and wildlife. Managers must balance economic and political forces influencing natural resource uses to ensure future productivity and multiple beneficial use of the land. Managers are commonly trained as foresters. They receive advice concerning management from teams of individuals with diverse backgrounds such as soil science, wildlife biology, logging systems' engineering, and archaeology. Landslide hazard information that I develop must address the management activity under consideration. It must also be understood by the non-geologists influencing and making the decision.

The relationship of landslide hazard to natural resources management can be illustrated by the Sierra National Forest in which I work (Figure 1). Management addresses an area of nearly 1.5 million acres. Timber is our main natural resource product. Wildlife, recreation use, and water for hydroelectric power and agricultural irrigation are other major resources. A variety of management activities are needed to accomplish this production. For example, roads are built to remove harvested timber and to reach recreation sites. The environmental consequences of these activities must be weighed against the resources produced. One important consequence is reactivation of existing landslides or creation of new ones (DeGraff, 1980).

Landslide hazard is generally greater on the Sierra National Forest than on nearby populated areas. Unlike the adjacent San Joaquin Valley, the forest hosts steep, rugged terrain. Climatic conditions vary widely each year and over longer periods of time. These natural conditions are conducive to landslide occurrences. The activities required to manage the natural resources of the forest can increase this natural landslide hazard potential. (Swanson and Dyrness, 1975; Ziemer, 1981).

Landslides increase the cost of producing needed natural resources. An obvious cost is the money spent to remove landslide debris and repair or replace structures. One road used to haul timber in the Sierra National Forest sustained damage totally nearly $1 million in 1982 (DeGraff and others, 1984). Less obvious costs are lost reservoir capacity from increased sediment in streams, siltation of fish spawning areas, and reduced soil productivity on hillslopes. One small debris flow in 1983 generated sediment that entered a major stream draining part of our forest. Resulting siltation forced suspension of fish stocking for weeks. This sediment was trapped in a small reservoir. There are no clear costs associated with these impacts; however, it was certainly not beneficial to fishing or long-term water storing ability.

Assessment Characteristics

In assessing landslide hazards for natural resources management, I use serveral methods sharing some common characteristics. Each method meets the needs specific to a certain level of land management. The methods use

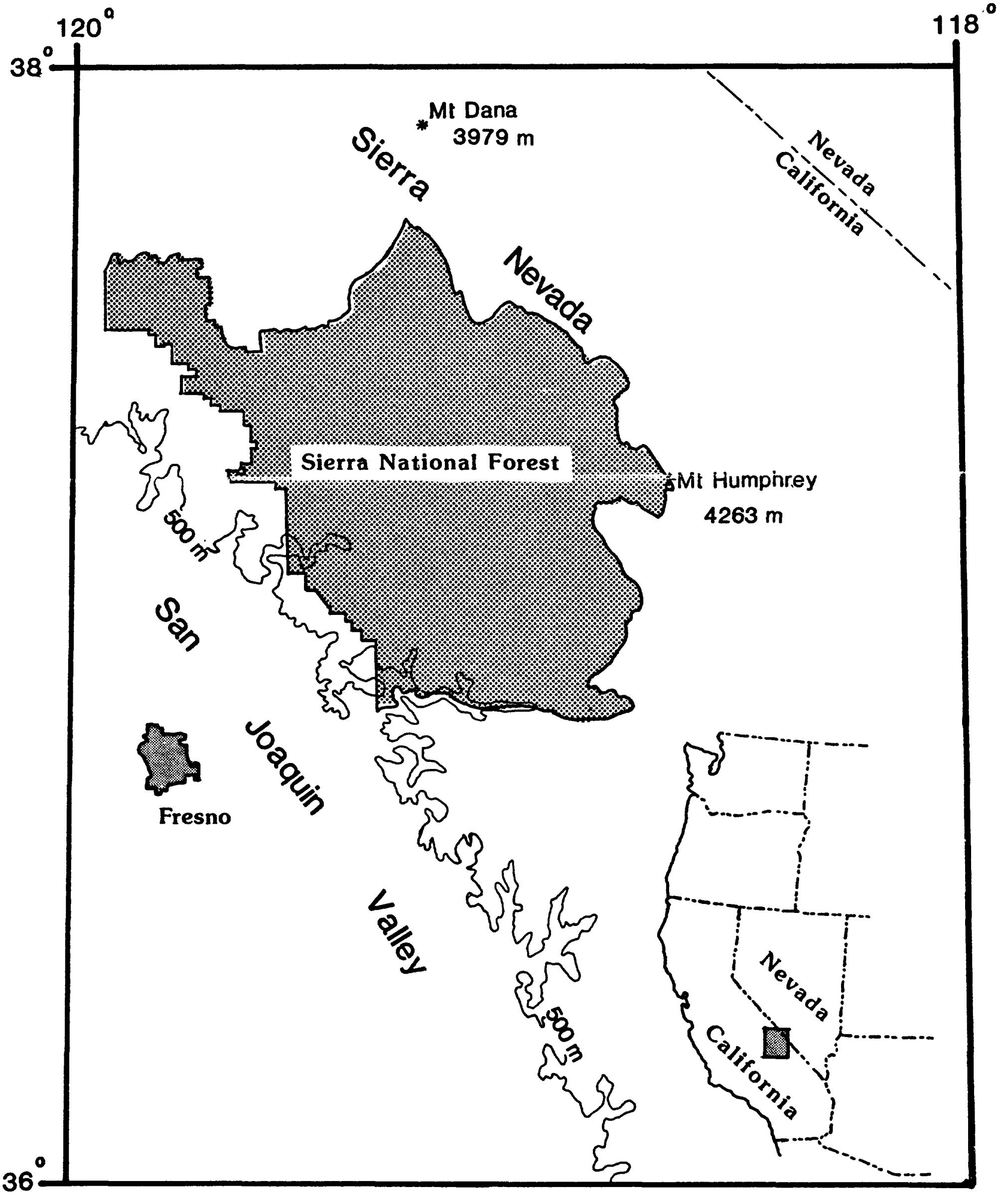


Figure 1. Location of the Sierra National Forest and surrounding geographic features. The 500-meter contour marks the approximate boundary between the Sierra Nevada and the San Joaquin Valley.

quantitative approaches to define relative landslide hazard. Finally, each method assumes that future landslide activity can be predicted by examining existing or previous landslide activity.

Management of national forests needs landslide hazard information at three different levels: 1) planning, 2) project, and 3) site. No single approach can adequately meet the needs of all three levels. A hierarchy of assessment methods is needed. A similar approach for land use planning in urban environments was promised by Leighton (1976). Leighton estimated that this approach would reduce damaging landslides by 95 to 99 percent. Using three different but compatible methods, I use an equivalent approach suitable for natural resources management. Each assessment method increases the detail of information obtained and decreases the area being evaluated. Table 1 shows the purpose, information obtained, and area considered for each level of assessment.

Table 1. Description of Landslide Hazard Information and Management Levels

Level of Management	Purpose	Hazard Information Needed	Area (Sq. Mi.)
Planning (Regional*)	Land and resource allocation; setting priority for further study.	General landslide susceptibility zones based on physical condition most frequently associated with past landslides.	5
Project (Community)	Location and suitability of developments and facilities; identify areas needing detailed study.	Landslide susceptibility zones based on the intensity of past landslides or specific types of past landslides.	1-5
Site (Site)	Criteria guiding design of developments and facilities; establishes where subsurface investigation and related techniques should be applied first.	Identification of active movement and likelihood of future movement based on activity occurring in recent time (200 yrs.).	1

*Levels of study proposed by Leighton (1976).

Quantitative approaches are used to define relative landslide hazard. Using a quantitative method minimizes differences arising from the diverse experience and educational background of different investigators doing the work. This adds a degree of reliability to the interpretation of landslide hazard. Information expressed in terms of relative hazard helps in comparing alternative courses of action.

Landslides occur where basic conditions creating landslide susceptible slopes coincide with initiating conditions causing failure (Sharpe, 1938). Examples of basic conditions are steep slopes, weak rock, and tilted strata. Initiating conditions include strong ground motion, intense precipitation, and excavation by man. The distribution of landslides over an area is evidence for locations where basic and initiating conditions combined to exceed the stability threshold of the slopes. Only the relationship of past landslides to basic conditions is usually discernible. Therefore, slopes with unfavorable basic conditions are assumed to be ones most susceptible to initiating conditions. While this is not always true, difficulty in predicting future initiating conditions precludes their explicit use. It is also assumed that localities with a high density of existing landslides are more susceptible to future landslide activity than localities with a low density of existing landslides. The last assumption involving existing landslides addresses potential for movement. Future movement is assumed to occur with the same frequency as recent (less than 200 years) movement. These assumptions enable evaluation of existing landslides to serve as the basis for assessment methods.

Matrix assessment, isopleth mapping of landslides, and dendrogeomorphology are methods that provide information needed for different levels of management, use quantitative approaches to yield relative landslide hazard, and predict future landslide activity based on existing landslides. Other methods exist that meet these criteria. The three methods I use are also cost-effective when applied to areas managed for their natural resources. Natural resources management usually involves large areas. Commonly, the terrain is rugged and accessibility is limited. Information useful to landslide hazard evaluation is scant. Few historical records of past landslides or detailed landslide investigations exist. The basic information available is usually geologic mapping, topographic mapping, and an inventory of existing landslides. This inventory is a product of aerial photointerpretation. The type of landslide, state of activity, and direction of movement are inferred from identifiable evidence in the aerial photography. Only limited field verification is done. This makes the best use of limited money and manpower over these large areas. Recent inventory work by myself and an assisting geologist produced maps showing landslides in a 100,000-acre area for about $.04 acre.

The following sections explain in detail each landslide hazard assessment method. For purposes of illustration, examples are given for the imaginary Tall Trees National Forest. These hypothetical situations provide a clearer understanding of the methodology with the absence of a specific locality requiring detailed description. Cited references contain actual cases where these methods have been used.

Matrix Assessment

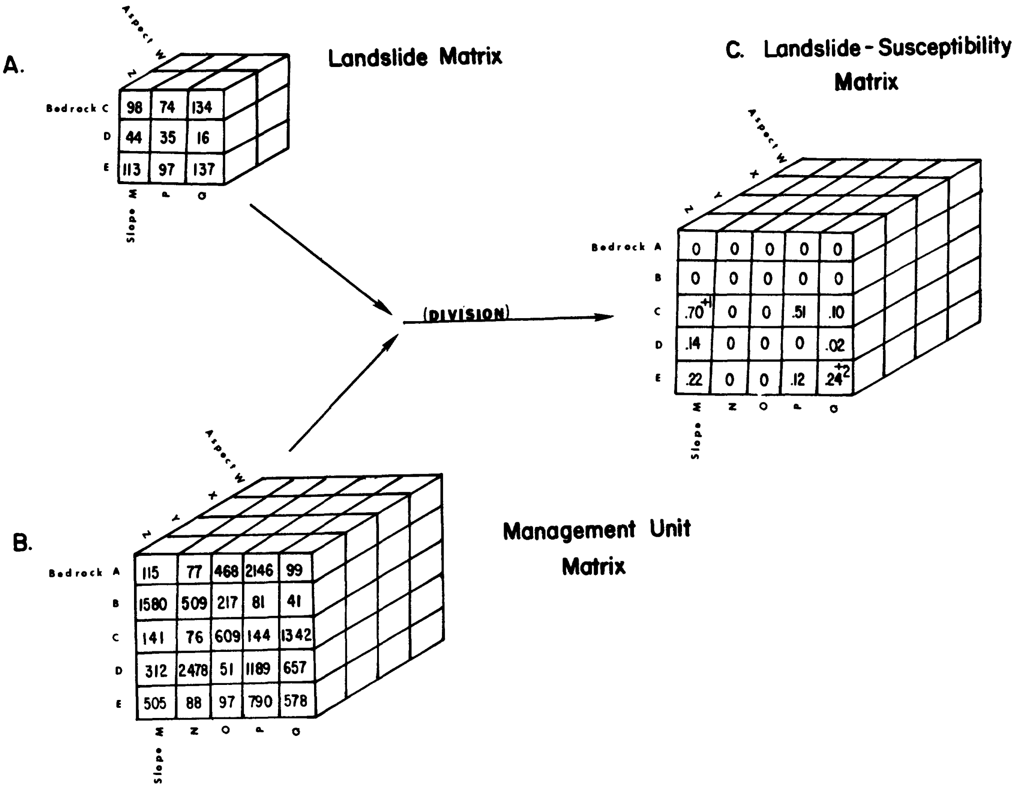
The matrix assessment method (DeGraff and Romesburg, 1980; DeGraff, 1982) provides landslide hazard information needed by management for planning. It uses three factors basic to landslide susceptibility conditions: bedrock, slope inclination, and slope orientation (DeGraff, 1978). A cluster analysis procedure classifies combinations of these factors defining four classes of landslide susceptibility. The combinations for each class are used to produce map units having this susceptibility to landslide occurrence. Management can allocate management activities likely to produce landslides to classes with lower susceptibility or plan for greater costs required to avoid landslides in higher susceptibility localities.

This assessment method starts with an inventory of existing landslides. Topographic maps and geologic maps are used to find the bedrock, slope inclination, and slope orientation associated with each landslide. This differentiates bedrock units with greater susceptibility to failure. This greater susceptibility may result from physical and chemical factors, including permeability, fractures, and mineral composition. It may be a function of landslide-prone regolith derived from a specific bedrock (Nilsen and Brabb, 1975). Bedrock is usually expressed as a formation, member, or other mappable unit. Slope inclination identifies topography conducive to landsliding. It is expressed in percent and grouped in 5% or 10% classes. Slope orientation differentiates slope elements where structural controls or climatic variations tend to increase landslide occurrence. It is defined by degrees of azimuth and expressed as 8 or 16 classes of compass direction (N, NE, SW, etc.). Each combination of bedrock, slope inclination class, and slope orientation class is assigned the total number of acres for all landslides associated with them. This is the landslide matrix (Figure 2a).

Using the same topographic and geologic maps, the total number of acres within the area being studied is determined for each combination represented in the landslide matrix. This forms the management unit matrix (Figure 2b). All combinations of bedrock, slope inclination class, and slope orientation class not represented in the landslide matrix, define areas with low landslide susceptibility.

The acres for each combination in the landslide matrix is divided by the acres for the corresponding combination in the management unit matrix (Figure 2c). The resulting values are the proportion of area disturbed by landslides. For example, 98 acres of landslides are found to involve bedrock formation A on 40-50% slopes that face northwest. A total of 141 acres having this same combination is present in the management area. This means that 0.70 is the proportion of the area affected by landslide activity. This normalizes the data. Without this step, it is impossible to determine whether the higher values of landslide-disturbed acres represent combinations of bedrock, slope inclination, and slope orientation especially susceptible to failure or widespread in the area under study.

The proportional values can range from .01 to .10. A nonhierachical cluster analysis is applied to these values (Anderberg, 1973, p. 44). Cluster analysis is a numerical technique for grouping objects. In this instance, an initial grouping separates the proportional values into three groups. This is achieved by dividing their range into three equal increments. The cluster analysis tries to improve this initial grouping by minimizing the sum of



Bedrock	Slope M	P	Q
C	98	74	134
D	44	35	16
E	113	97	137

Bedrock	Slope M	N	O	P	Q
A	115	77	468	2146	99
B	1580	509	217	81	41
C	141	76	609	144	1342
D	312	2478	51	1189	657
E	505	88	97	790	578

Bedrock	Slope M	N	O	P	Q
A	0	0	0	0	0
B	0	0	0	0	0
C	.70 +1	0	0	.51	.10
D	.14	0	0	0	.02
E	.22	0	0	.12	.24 +2

Figure 2. Diagram of the matrix assessment procedure for determining landslide susceptibility. The proportional vlaue of 0.70 shown for Bedrock C on slope M and aspect Z is computed by dividing the acres of landslide-disturbed terrain (98) by the number of acres having this same combination of factors in the management unit (141).

squared deviations about each group mean (W function). After calculating the initial sum or W value, it moves the boundary of the lower and middle value groups to the left (Table 2). It recalculates the W value. If it decreases, the boundary is moved again. This is repeated until the W value increases. The boundary for the previous lower W value is retained. If the first move to the left increases the W value, the initial boundary is retained. Than the boundary is moved to the right. This continues until the W value increases. The boundary for the previous lower W value is retained. Then the same procedure is applied to the boundary between the middle and higher value groups. The final grouping of proportional values represents three additional classes of landslide susceptibility. Combinations of bedrock, slope inclination, and slope orientation represented by values in the lower group define areas with moderate landslide susceptibility. The middle group defines areas with high landslide susceptibility. Extreme landslide susceptibility is defined by combinations in the higher group.

Table 2.

Landslide susceptibility partitioning and related W function values that lead to final partition of proportional values.

Partition	Landslide Susceptibility			W Value
	Moderate	High	Extreme	
Initial	0.01 to 0.31	0.31+ to 0.51	0.51+ to 1.00	0.200 676
Left	0.01 to 0.25	0.25+ to 0.51	0.51+ to 1.00	0.168 319
Left	0.01 to 0.21	0.21+ to 0.51	0.51+ to 1.00	0.142 149
Left	0.01 to 0.15	0.15+ to 0.51	0.51+ to 1.00	0.138 630
Left	0.01 to 0.14	0.14+ to 0.51	0.51+ to 1.00	0.159 149
Initial	0.01 to 0.15	0.15+ to 0.51	0.51+ to 1.00	0.138 630
Left	0.01 to 0.15	0.15+ to 0.47	0.47+ to 1.00	0.261 244
Right	0.01 to 0.15	0.15+ to 1.00	1.00+ to 1.00	0.805 297
Final	0.01 to 0.15	0.15+ to 0.51	0.51+ to 1.00	0.138 630

A landslide susceptibility map can be prepared for the management area (Figure 3). Every point in the area is described by a specific bedrock, slope inclination, and slope orientation combination. The landslide susceptibility

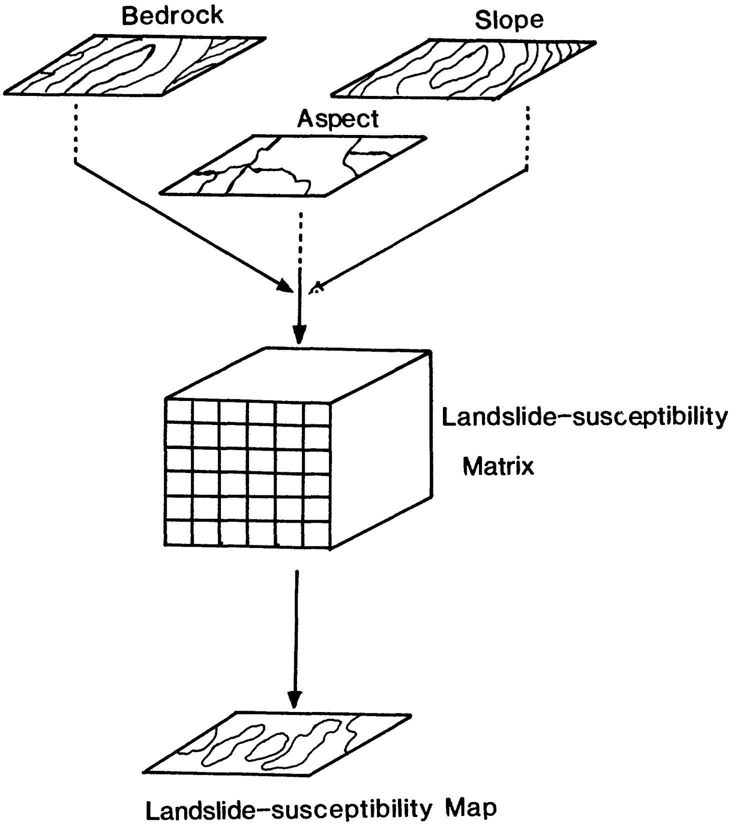


Figure 3. Schematic representation of preparing a landslide-susceptibility map using the matrix assessment results.

class for that value is assigned to that point. After all points are assigned to classes, boundaries are drawn around contiguous points with identical susceptibility classification. This produces a map of low, moderate, high, and extreme landslide susceptible areas.

Application of matrix assessment to planning can be illustrated by a transportation analysis. Our hypothetical Tall Trees National Forest includes a major mountain range. Forest managers are faced with a proposal to connect a major forest road to a recently completed interstate highway crossing the forest. Three corridors are identified for possible location of the road. Detailed studies will produce designs minimizing landslide occurrence. However, road sections requiring these designs will be expensive. If the corridor chosen during the feasibility study has the least amount of unstable land, the road can be built for the least cost. Figure 4 shows the landslide-susceptibility map for the corridors. It is clear that managers of the Tall Trees National Forest should choose corridor A. Several locations with high landslide susceptibility would have a high priority for later detailed stability investigation.

Isopleth Mapping of Landslide Deposits

Isopleth mapping of landslide deposits develops landslide hazard information appropriate to project work (DeGraff, 1983). It generalizes the distribution of landslides into six classes of landslide susceptibility. It minimizes nonrecognition or misidentification of landslides in the basic inventory. Valid comparison can be made between areas with a few large landslides and ones with many small landslides. The landslide hazard information provided by this method addresses questions on location and suitability of proposed management activity.

Preparation of isopleth maps of landslide deposits is well-described by Wright and others (1974). Like matrix assessment, it starts with the basic landslide inventory (Figure 5). A transparent grid is placed over a topographic map showing the location of existing landslides. The grid intersections on the overlay are spaced at 0.5 inches. A 1-inch diameter counting circle enclosing an inscribed 20 x 20 per inch grid is placed over

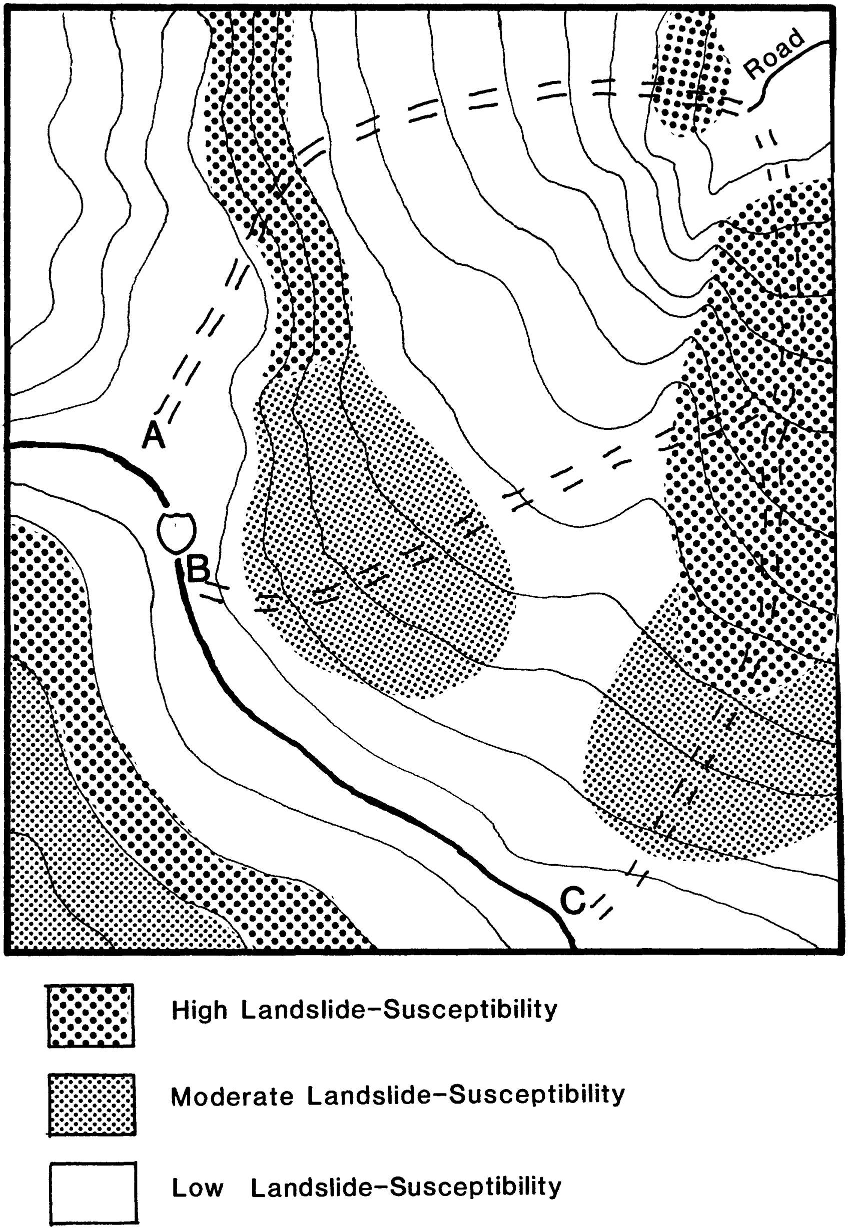


Figure 4. Map showing the three road corridors proposed to connect the forest road to the interstate highway crossing the imaginary Tall Trees National Forest.

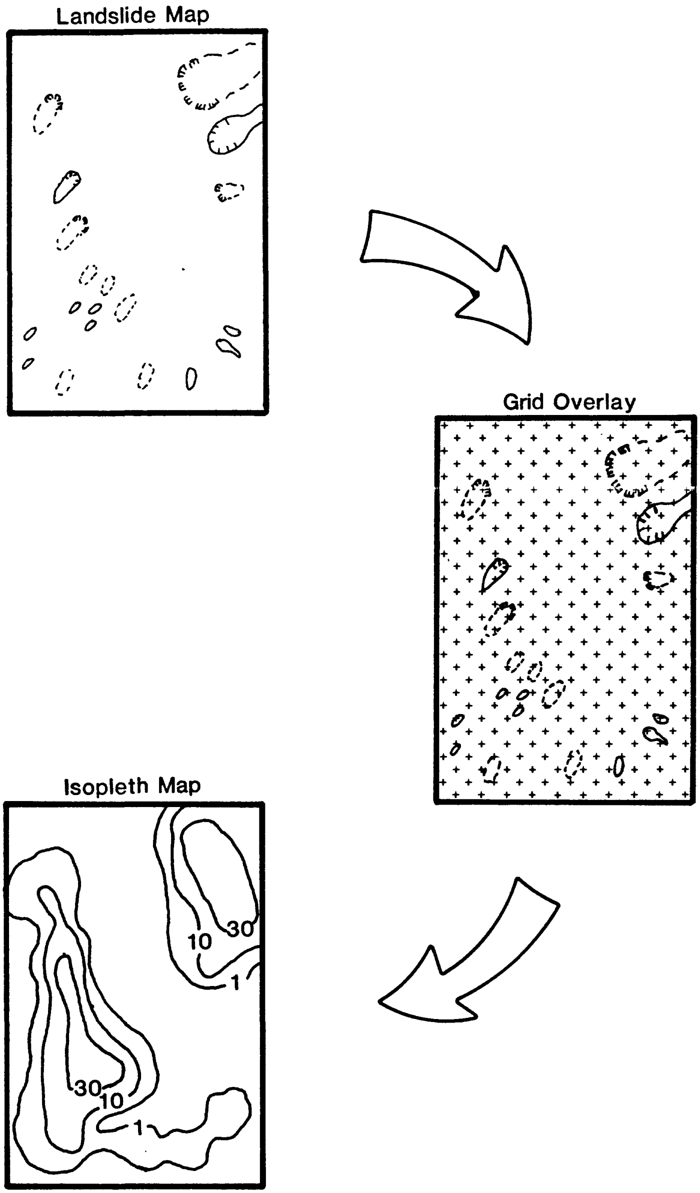


Figure 5. Diagram showing the steps in preparing an isopleth map of landslide deposits. The numbers are the percent values for each contour (isopleth).

each grid intersection. The number of grid squares within the circle coinciding with mapped landslide deposits are counted. Using the total number of grid squares in the circle, the number is converted into a percent value. This value is assigned to the grid intersection on the map overlay. Contour lines (isopleths) are drawn for 1, 10, 30, 50 and 70 percent of area underlain by landslide deposit. Relative landslide-susceptibility classes are: 1) negligible for less than 1 percent, 2) low for 1 to 10 percent, 3) moderate for 10 to 30 percent, 4) high for 30 to 50 percent, 5) very high for 50 to 70 percent, and 6) extreme for greater than 70 percent.

Preparation of a timber sale will serve to illustrate this method. We will assume that our imaginary Tall Trees National Forest follows the typical seven-year program leading to a timber sale. Sale areas are initially defined on the basis of suitable timber stands, existing access roads, and similar criteria. At this point, the proposed sale area and tentative harvest plan are reviewed. This identifies situations that might render proposed harvest units unsuitable or alter road locations. Information on stability or similar concerns is based on existing data without additional field study. Figure 6 shows an isopleth map for a proposed timber sale. Harvest units proposed to be clear cut and temporary roads in high landslide susceptibility areas would need reconsideration. Sale activities proposed for moderate landslide susceptibility areas will require detailed review. Stability would be of no real concern for the sale area designated as having low or negligible landslide susceptibility.

Dendrogeomorphology

Dendrogeomorphology or, more simply, tree-ring analysis provides landslide hazard information applicable to site investigations (DeGraff and Agard, 1984). It helps determine whether existing landslides were active in recent time. Some estimate of the likelihood of movement in the future can be made. The landslide hazard information gained aids in designing management activities.

Tree-ring analysis is a developing technique applicable to the study of various geomorphic processes and geologic hazards (Alestalo, 1971, Shroder,

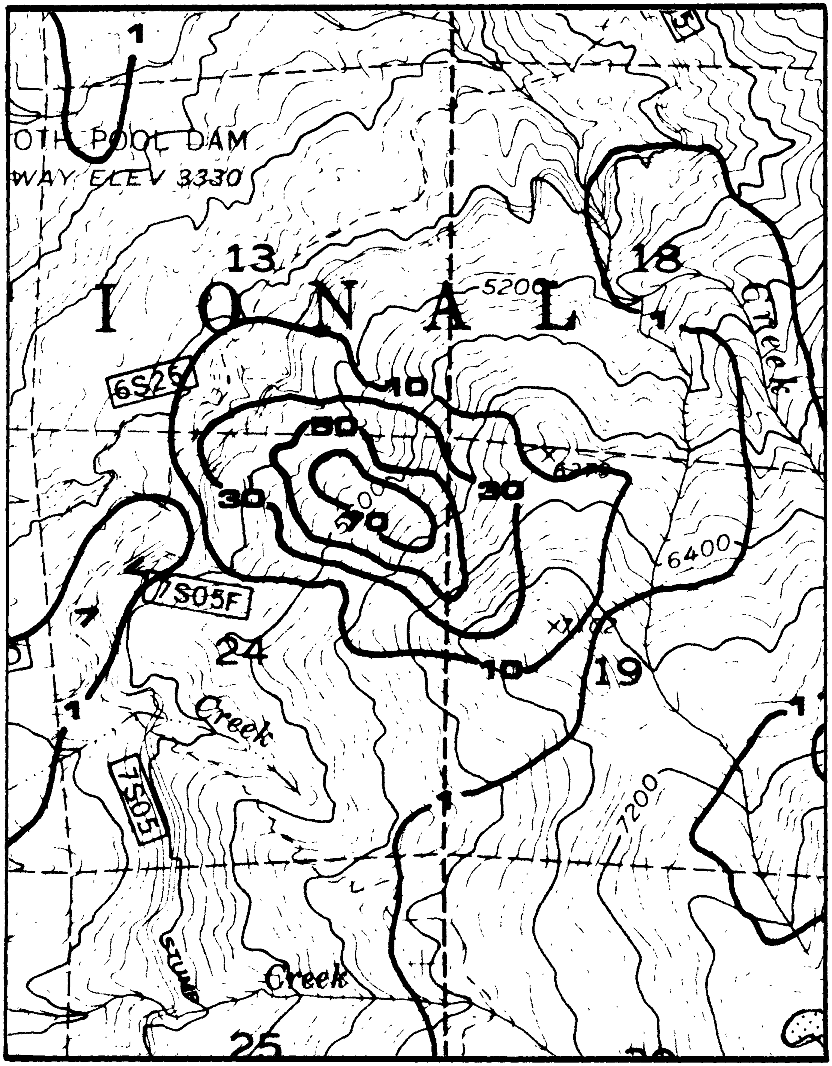


Figure 6. Isopleth map for a sample timber sale being prepared on the hypothetical Tall Trees National Forest.

1978). Tree growth can be altered or limited by geomorphic processes. External events affecting a tree can produce a datable internal response. For example, tilting of the tree by landslide movement can produce eccentric growth (Figure 7). This results from growth changes initiated by the tree to restore a vertical orientation to the growth tip. A curve in the tree stem may also be evident.

Applying this method begins with an existing landslide recognized on the ground or identified by the landslide inventory. Coring with an increment borer is the common non-destructive sampling technique. These cores are commonly taken from the tree near the base and at breast height. Data describing the tree, local site, and sampling effort are taken. The cores are dried and glued into slotted wooden mounts. It is then sanded and polished to provide a longitudinal cross-section. Ring width, reaction wood, and related features are noted by year. Ring widths are plotted with downslope width as a percentage of total annual growth. Abrupt increases or decreases in eccentricity beyond 60 percent or less than 40 percent are assumed to result from disturbance by tilting. A datable event is considered the onset of eccentric growth after a year or more of concentric growth. Consecutive years of eccentric growth may either indicate continued tilting or growth response to correct only the initial tilting. A series of trees should be sampled to allow construction of an event history. Duplication of events between different trees increases the reliability of interpretation. Control trees on adjacent undisturbed sites provide data to clarify the event history. Suppressed growth and reaction wood are other internal growth responses useful in reconstrucing movement histories.

Tree-ring analysis is less widely applicable than matrix assessment or isopleth mapping of landslide deposits. Its principal value is as a means for preliminary investigation or for data supplementing traditional subsurface sampling or geophysical methods. It should only be substituted for these traditional methods when circumstances such as access prevent their use. There is greater opportunity for misinterpretation of data through casual use of this method than of other methods.

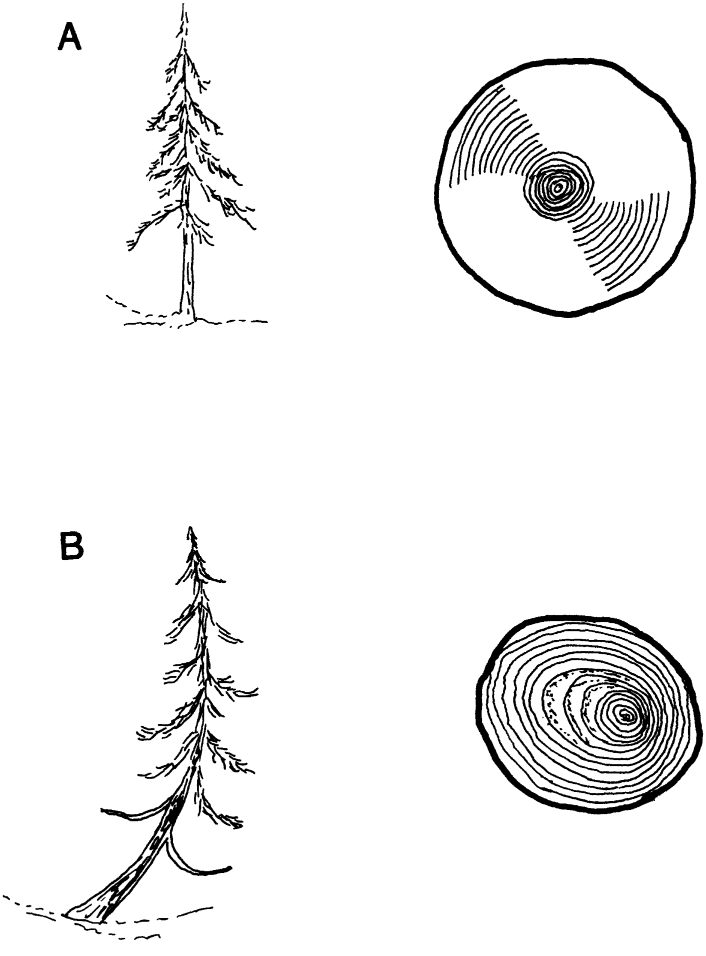


Figure 7. The upper tree (A) is undisturbed by landslide movement with a cross-section showing concentric growth rings. The lower tree (B) was tilted by landslide movement. The eccentric growth pattern that resulted has narrower rings on the downtilt side. The narrower rings are on the uptilt side for angiosperms.

Initial investigation for a large landslide next to a road will show how this method could be applied. Management of our hypothetical Tall Trees National Forest is considering widening the road. Considerable expense would be involved to drill exploration holes for the entire extent of this feature. A preliminary identification of any sections active in recent time would narrow the scope of this work. Tree-ring analysis is applied to the feature. Figure 8 shows two areas with significant movement in the past hundred years. Subsurface investigation will be directed to these areas first. The results will guide the need and extent of stabilizing design for this road widening.

CONCLUSION

Economic, social, and public policy factors restrict options available to decisionmakers to achieve landslide hazard reduction (Hays and Shearer, 1981). This holds true for natural resources management as well as urban planning. Therefore, it is important to provide decisionmakers with technically sound landslide information in an understandable form appropriate to their needs. When geologists fail in this effort, options for effective landslide hazard reduction are further restricted.

The described program serves landslide information needs encountered in managing national forests. Different local environments or emphasis in natural resources management may require use of different landslide hazard assessment methods. Choosing methods that quantify relative hazard, use information derived from existing landslides, and is specific to levels of user need will likely achieve comparable results.

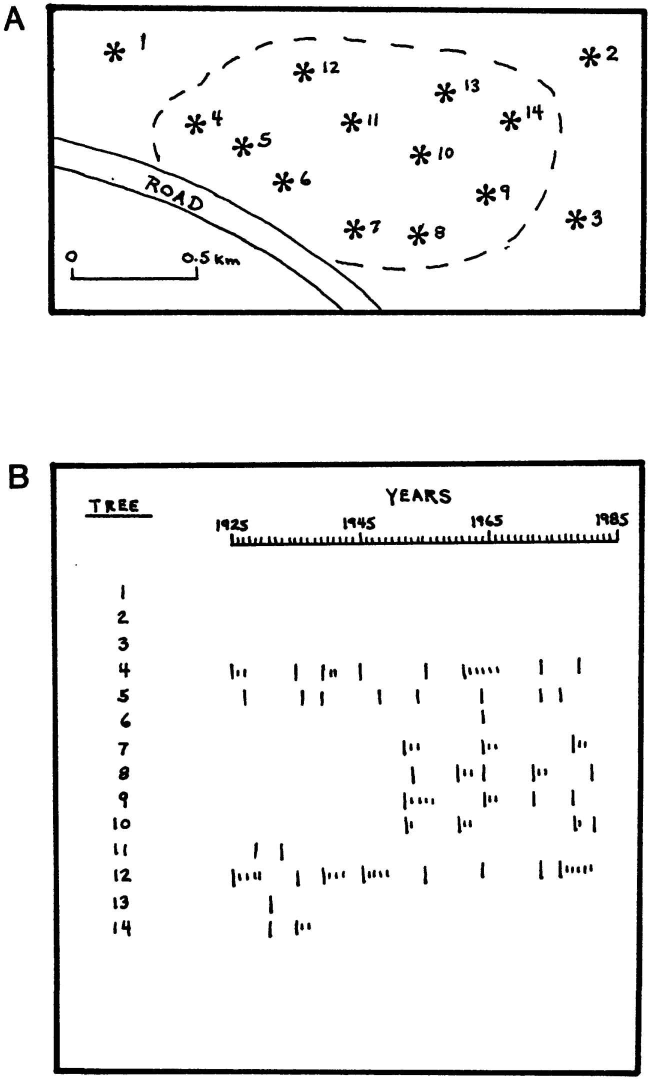


Figure 8. The map (A) shows the landslide and road problem on the Tall Trees National Forest. The asterix are the sampled trees with a number corresponding to the chart (B) of years with eccentric growth from landslide movement. No eccentricity is shown by trees 1, 2, and 3 on undisturbed ground. Areas needing further study are shown by similar patterns of disturbance for trees 4, 5, and 12 and trees 7, 8, 9, and 10.

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LANDSLIDES CAUSED BY EARTHQUAKES

By

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INTRODUCTION

Earthquakes have long been recognized as a major cause of landslides. Earthquake-induced landslides have been documented from at least as early as 373 or 372 B.C. (Seed, 1968) and have caused tens of thousands of deaths and billions of dollars in economic losses during the present century. In some earthquakes, landslides have denuded thousands of square kilometers. To reduce losses from these landslides it is important to understand their basic characteristics. In order to do that, I address the following major questions: (1) How do the number and distribution of landslides depend on earthquake magnitude, ground-shaking intensities, and other seismic parameters? (2) What types of landslides are caused by earthquakes? (3) Which of these types are most hazardous to human life and property? (4) What geologic materials are most susceptible to landslides in earthquakes?

Information I use to answer these questions has been taken from a study of landslides in 40 historical earthquakes chosen to sample many climatic, geologic, and seismic settings in the earth's major seismic regions (Keefer, 1984). These earthquakes have magnitudes from 5.2 to 9.5; to study landslides in smaller events, I have also examined intensity reports from several hundred earthquakes in the United States.

TYPES, NUMBERS, AND GEOLOGIC ENVIRONMENTS OF EARTHQUAKE-INDUCED LANDSLIDES

Landslide Classification

The term "landslide" encompasses many phenomena involving lateral and downslope movement of earth materials. Numerous landslide classifications based on morphology, materials, mechanism of initiation, or other criteria have been proposed. The classification of earthquake-induced landslides shown in Table 1, based on the principles and terminology of Varnes (1978), categorizes landslides primarily by material and character of movement and secondarily by such other attributes as degree of internal disruption and water content.

Material is classified as "rock" or "soil" based on its state prior to landslide initiation. "Rock" signifies firm, intact bedrock. "Soil" signifies a loose, unconsolidated, or poorly cemented aggregate of particles, which may or may not contain organic material. The term "soil" thus encompasses the entire regolith and all man-made fills.

Movement characteristics of earthquake-induced landslides are summarized in Table 1 and discussed in more detail in the section below on landslide characteristics and geologic environments. Landslides are grouped by similarities in movement, internal disruption, and geologic environments into major categories of disrupted slides and falls, coherent slides, and lateral spreads and flows. In classifying earthquake-induced landslides, the term "rock avalanche" is used for the sake of brevity to be synonymous with both "rock-fall avalanche" and "rock fall-debris flow" as defined by Varnes (1978). In addition, I have grouped all subaqueous landslides together for ease in discussing their geologic environments.

Numbers of Landslides in Historical Earthquakes

The number of landslides caused by an earthquake generally increases with increasing magnitude. For example, earthquakes with $M < 5.5$ commonly cause a few tens of landslides at most whereas earthquakes with $M > 8.0$ commonly cause several thousands at least. However, local geologic conditions and seismic parameters other than magnitude also determine the number of landslides triggered.

The relative abundance of different types of landslides caused by the 40 historical earthquakes studied by Keefer (1984) are shown in Table 2. The most abundant landslides were rock falls, disrupted soil slides, and rock slides; their abundance indicates both that they are especially susceptible to

TABLE 1. CHARACTERISTICS OF EARTHQUAKE-INDUCED LANDSLIDES
(From Keefer, 1984)

Name	Type of movement	Internal disruption	Water Content				Velocity	Depth
			D	U	PS	S		
		LANDSLIDES IN ROCK						
		Disrupted slides and falls						
Rock falls	Bounding, rolling, free fall	High or very high	X	X	X	X	Extremely rapid	Shallow
Rock slides	Translational sliding on basal shear surface	High	X	X	X	X	Rapid to extremely rapid	Shallow
Rock avalanches	Complex, involving sliding and/or flow as stream of rock fragments	Very High	X	X	X	X	Extremely rapid	Deep
		Coherent slides						
Rock slumps	Sliding on basal shear surface with component of headward rotation	Slight or moderate	?	X	X	X	Slow to rapid	Deep
Rock block slides	Translational sliding on basal shear surface	Slight or moderate	?	X	X	X	Slow to rapid	Deep

Note: Internal disruption: "slight" signifies landslide consists of one or a few coherent blocks; "moderate" signifies several coherent blocks; "high" signifies numerous small blocks and individual soil grains and rock fragments; "very high" signifies nearly complete disaggregation into individual soil grains or small rock fragments.
Water content: D = dry; U = moist but unsaturated; PS = partly saturated; S = saturated.

Velocity:

0.6 m/yr 1.5 m/yr 1.5 m/mo 1.5 m/day 0.3 m/min 3 m/sec

extremely slow very slow slow moderate rapid very rapid extremely rapid
(velocity terminology from Varnes, 1978).

Depth: "shallow" signifies thickness gererally less than 3 m; "deep" signifies depth generally greater than 3 m.

TABLE 1 CONTINUED

Name	Type of movement	Internal disruption	Water Content				Velocity	Depth
			D	U	PS	S		
LANDSLIDES IN SOIL								
Disrupted slides and falls								
Soil falls	Bounding, rolling, free fall	High or very high	X	X	X	X	Extremely rapid	Shallow
Disrupted soil slides	Translational sliding on basal shear surface or zone of weakened, sensitive clay	High	X	X	X	X	Moderate to rapid	Shallow
Soil avalanches	Translational sliding with subsidary flow	Very high	X	X	X	X	Very rapid to extremely rapid	Shallow
Coherent slides								
Soil slumps	Sliding on basal shear surface with component of hearward rotation	Slight or moderate	?	X	X	X	Slow to rapid	Deep
Soil block slides	Translational sliding on basal shear surface	Slight to moderate	?	?	X	X	Slow to very rapid	Deep
Slow earth flows	Translational sliding on basal shear surface with minor internal flow	Slight			X	X	Very slow to moderate with very rapid surges	Generally shallow occasionally deep
Lateral spreads and flows								
Soil lateral spreads	Translational on basal zone of liquefied gravel, sand or silt or weakened sensitive clay	Generally moderate occasionally slight occasionally high			X	X	Very rapid	Variable
Rapid soil flows	Flow	Very high	?	?	?	X	Very rapid to extremely rapid	Shallow
Subaqueous landslides	Complex, generally involving slumping, lateral spreading, and/or flow; occassionally involving slumping and/or block sliding	Generally high or very high, occassionally moderate or slight			X	X	Generally rapd to extremely rapid; occassionally slow to moderate	Variable

Table 2. Relative Abundances of Earthquake-Induced Landslides

(From Keefer, 1984)

Landslide type- listed in order of decreasing total numbers

Very abundant-- 100,000 in the 40 historical earthquakes.

Rock falls
Disrupted soil slides
Rock slides

Abundant-- 10,000 to 100,000 in the 40 historical earthquakes

Soil lateral spreads
Soil slumps
Soil block slides
Soil avalanches

Moderately common-- 1,000 to 10,000 in the 40 historical earthquakes

Soil falls
Rapid soil flows
Rock slumps

Uncommon-- 100 to 1,000 in the 40 historical earthquakes

Subaqueous landslides
Slow earth flows
Rock block slides
Rock avalanches

initiation in seismic conditions and that geologic environments that produce them are widespread in seismic regions. Subaqueous landslides, slow earth flows, rock block slides, and rock avalanches were uncommon in the 40 earthquakes. The apparent rarity of subaqueous landslides is due partly to difficulties in observation. The reported numbers of the other uncommon types, however, are probably good approximations of their actual numbers. The rarity of these landslides indicates low susceptibility to initiation by seismic shaking, restricted distribution of environments that produce them, or both.

LANDSLIDE CHARACTERISTICS AND GEOLOGIC ENVIRONMENTS

Disrupted Slides and Falls

Rock falls: Rock falls are individual boulders or disrupted masses of rock that descend slopes by bounding, rolling, or free fall. Although earthquake-induced rock falls occur in virtually all types of rock, most are in closely jointed or weakly cemented materials. Most well-cemented rocks that produce rock falls are broken by joints spaced a few centimeters apart. In many such rocks, joints are opened by physical weathering or stress relief or are filled with weak chemical-weathering products. Rock falls originate on slopes steeper than 40°.

Rock slides: Rock slides are disordered during movement into masses of rock fragments and blocks that slide on planar or gently curved surfaces where joints, bedding planes, or other surfaces of discontinuity dip out of slopes. Rock slides involve the same types of materials as rock falls, or, occasionally, older rock-slide deposits. They originate on slopes steeper than 25°.

Rock avalanches: Rock avalanches are landslides that disintegrate into streams of rock fragments that can travel several kilometers on slopes of a few degrees at velocities of hundreds of kilometers per hour. All rock avalanches reported from the 40 historical earthquakes were large, with volumes of at least 0.5×10^6 m^3. Because of their size and speed rock avalanches are among the most hazardous of earthquake-induced landslides.

The kinetic energy necessary for long-distance transport of rock-avalanche material is produced by intial fall from steep, high slopes. Avalanche paths consist of steep source slopes where the avalanches accelerate and, except where constrained by narrow canyons, of gentle run-out slopes where they decelerate and come to rest. Data on source-slope inclination and height show that the minimum inclination and height are respectively 25° and 150 m. All but one of the earthquake-induced rock avalanches originated on slopes undercut by active fluvial erosion or by active, Holocene, or late Pleistocene glacial erosion. Most slopes that produce rock avalanches are intensely fractured, with the rock being broken by several sets of fractures spaced a few centimeters apart. Most such slopes also exhibit at least one of the following additional signs of low strength or potential instability: (1) conspicuous planes of weakness--faults, master joints, bedding planes, or foliation surfaces--dipping out of the slope, (2) significant weathering of the rock, (3) weak cementation of the rock, or (4) geologic or historical evidence of previous landsliding.

Soil falls: Soil falls are blocks or disrupted masses of soil that descend slopes by bounding, rolling, or free fall. Most soil blocks break apart during transport or impact. In earthquakes, soil falls originate on steep slopes such as coastal bluffs, canyon walls, stream banks, terrace faces, and cut slopes. Although 63° was the minimum slope inclination reported for soil falls in the historical earthquakes, these landslides probably take place on slopes at least as gentle as 40°, the minimum observed for rock falls. Most soil falls involve weakly cemented sand or gravel; a few occur in unconsolidated or weakly cemented clay.

Disrupted soil slides: Disrupted soil slides consist of sheets of soil, a few decimeters to a few meters thick, that disintegrate during movement into chaotic jumbles of small blocks and individual soil grains. Most slide on basal shear surfaces formed at soil-bedrock contacts or at boundaries between different soil layers; a few move on basal zones of weakened, sensitive clay. The material most commonly involved in disrupted soil slides in the historical earthquakes was loose, unsaturated, residual or colluvial sand with little or no clay. However, earthquakes in volcanic areas have also caused many disrupted soil slides in saturated volcanic soils consisting of alternating layers of scoria and sensitive clay produced by weathering of fine-grained ash. Other materials susceptible to disrupted soil slides include sandy and silty man-made fill; fault gouge; floodplain alluvium; terrace deposits; cemented sand, silt, and clay; till; colluvium; fluvioglacial deposits; and old landslide material. The minimum slope inclination for disrupted soil slides is 15°.

Soil avalanches: Soil avalanches, more disaggregated and faster moving than disrupted soil slides, generally consist of streams of grains and small blocks of soil. Many travel far beyond the bases of the slopes on which they originate. The originate on slopes steeper than 25° in unsaturated sand, older soil-avalanche deposits, and volcanic soils containing sensitive clay.

Coherent Slides

Rock slumps: Each rock slump consists of one or a small number of coherent, deep-seated blocks that slide on basal shear surfaces curved so that movement involves a component of headward rotation. Earthquake-induced slumps are initiated on slopes steeper than 15° in igneous and metamorphic as well as in sedimentary rocks. Most rocks that produce slumps are weak, either because they are poorly cemented, closely jointed, weathered, or sheared. Older rock-slump deposits are occasionally reactivated by earthquakes.

Rock block slides: Rock block slides, also generally deep-seated, consist of one or a few blocks that slide on planar or gently curved basal shear surfaces. Movement, thus, involves little or no rotation. Basal shear surfaces are bedding planes or other discontinuities that dip out of slopes, allowing the blocks to move without significant distortion. Rock block slides originate on slopes steeper than 15°.

Soil slumps: Soil slumps, generally deep-seated, consist of one or a few coherent blocks that slide on basal shear surfaces curved so that movement involves headward rotation. These landslides are characterized by crescent-shaped scarps, blocks with surfaces tilted back toward the crests of slopes, and bulging toes. Man-made fill is the most common material in earthquake-induced soil slumps. Many slumped fills are uncompacted or poorly compacted. Slumps in fill composed of sand or silt are more common than in fill composed of clay. Earthquakes cause fill to slump both on hillsides and on alluvial and coastal floodplains, where slumps are most abundant in embankments built over marshes or filled river channels. Floodplain alluvium produced soil slumps in more earthquakes than any other natural material, and sandy alluvium produced more slumps than coarser or finer material. Other materials involved in slumps are listed in Table 3. Except for one slump on a 7° slope triggered by a 1978 Japanese earthquake, the minimum slope inclination reported for soil slumps in the historical earthquakes was 10°.

Soil block slides: Soil block slides, also generally deep-seated, slide in a translational manner on planar or gently-curved shear surfaces. These slides have grabens at their heads, some have internal fissures or grabens as well, and pressure ridges mark the toes of many.

Large earthquake-induced block slides, with volumes of several hundred thousand or million cubic meters, generally occur in old landslide areas on river or coastal bluffs higher than 16 m and steeper than 6°. These block slides generally move on layers of sensitive clay or of loose, saturated sand or silt. Most smaller block slides occur in sandy or silty floodplain alluvium or man-made fill. Most fills that produce block slides are on floodplains and are composed of alluvial materials. Other materials producing small numbers of block slides are till, volcanic ash, colluvium, clayey playa sediment, lacustrine sediment, terrace gravel, sandy eolian sediment, sandy alluvial-fan sediment, and periglacial sediment. Basal shear-surfaces of most block slides are probably saturated. Many soil block slides involve flat-topped slopes and near-horizontal basal shear surfaces. The frontal slopes in areas of soil block slides are reported to be as gentle as 5°.

Slow earth flows: Slow earth flows are tongue- or teardrop-shaped bodies of clay, silty clay, or clayey silt bounded by discrete, lateral and basal shear surfaces. These landslides move primarily by boundary shear; internal deformation is minor. Basal shear surfaces are saturated. Slow earth flows caused by earthquakes are in clayey residual soil, clayey loam, till, volcanic ash, colluvium, and older earth-flow deposits. The minimum reported slope inclination for these slow earth flows is 10°.

TABLE 3. SOIL-SLUMP MATERIALS

- Man-made fill (23)
- Floodplain alluvium (12)
- Old slump deposits (5)
- Colluvium (3)
- Till (3)
- Alluvial fan deposits (3)
- San dunes (2)
- Terrace deposits (2)
- Cemented sand and gravel (1)
- Volcanic ash (1)
- Pyroclastic deposits (1)
- Sensitive clay (1)
- Fluvioglacial deposits (1)
- Ground moraine (1)
- Playa deposits (1)
- Lacustrine deposits (1)
- Deltaic deposits (1)
- Barrier island deposits (1)

Note: Numbers in parenthesis are total number of the 40 historical earthquakes (Keefer, 1984) in which slumps occurred in particular material.

TABLE 4. SOIL LATERAL-SPREAD MATERIALS

- Floodplain alluvium (19)
- Man-made fill (18)
- Deltic deposits (5)
- Sand dunes (4)
- Coastal sand spits (4)
- Alluvial fan deposits (3)
- Lacustrine sediments (3)
- Beach deposits (3)
- Beach bar deposits (2)
- Estuarine deposits (2)
- Periglacial sediments (1)
- Barrier islands (1)
- Tidal flats (1)
- Terraces not on floodplains (1)
- Glacial outwash (1)
- Glaciolacustrine deposits (1)
- Playa sediments (1)
- Organic, marine mud (1)
- Carbonate silt in a thermal spring (1)
- Sensitive clay of undetermined origin (1)

Note: Numbers in parenthesis are total number of the 40 historical earthquakes (Keefer, 1984) in which lateral spreads occurred in a particular material.

Lateral Spreads and Flows

Soil lateral spreads: Soil lateral spreads move in a translational manner on zones of liquefied gravel, sand, or silt or, occasionally, on sensitive clay rendered fluid by disturbance. All these basal zones are saturated. More disrupted than soil slumps or soil block slides, soil lateral spreads contain numerous internal fissures and grabens. Soil lateral spreads are most common in granular man-made fill and floodplain alluvium. Most alluvial lateral spreads are along active or abandoned river channels or in marshes. Composition of alluvium in lateral spreads is predominantly silt, silty sand, or fine-grained sand. Most alluvial materials in lateral spreads are Holocene; a few are Pleistocene. Man-made fills that fail by lateral spreading range in composition from silt to sandy gravel, but most are composed of fine-grained sand; fills that produce lateral spreads are commonly uncompacted. Other materials that produce soil lateral spreads are listed in Table 4. The minimum slope inclination reported for lateral spreads is 0.3°.

Rapid soil flows: Rapid soil flows are streams of soil grains, usually but not always mixed with water, that flow in a fluid-like fashion at high velocities. Some rapid soil flows travel several kilometers on slopes of only a few degrees yet transport boulders weighing hundreds of tons. Rapid soil flows have probably killed more people in the present century than any other type of seismically-triggered landslide. Flows in loess, probably caused bv a combination of seismic shaking and heavy rainfall, killed 15,000 people near Khait, U.S.S.R. following an earthquake in 1949 (A. M. Sarna Wojcicki, 1980, oral commun.). Seismically-tiggered flows in loess may have killed as many as 100,000 people in a 1920 earthquake in Kansu Province, China (Varnes, 1978). In addition to loess, rapid soil flows form in volcanic soils containing sensitive clay; dune sand; sandy residual and colluvial soils; silty and sandy alluvial fan deposits; floodplain alluvium; and man-made fill. All flows in the historical earthquakes, possibly excepting those in Kansu Province, were in saturated soils. These saturated flows were triggered either by a combination of shaking and high rainfall, or by shaking at sites with high water tables or flowing springs. The minimum slope reported for a saturated flow is 2.3°.

Subaqueous landslides: Although a few earthquake-induced subaqueous landslides move primarily by slumping or block sliding, most involve lateral spreading, rapid flow, or both. Submarine landslides caused by earthquakes occur in granular Holocene deltaic sediments, or, less commonly in, coarse-grained till or material from sand spits, tidal flats, beaches, alluvial fans, or offshore areas. Sublacustrine landslides occur in lake silt, outwash gravel and sand, alluvial fans, and most commonly, in Holocene deltaic sediments composed of sand, gravelly sand, sandy gravel, or gravel. The minimum slope inclination reported for subaquous landslides was 0.5°, but at most landslide localities slopes were steeper than 10°.

LANDSLIDE DISTRIBUTION AND SEISMIC PARAMETERS

The Smallest Earthquakes That Cause Landslides

To determine the smallest earthquakes that cause landslides, intensity reports for United States earthquakes from 1958 to 1977 inclusive were examined (Keefer, 1984). These data suggest that the following are the smallest earthquakes likely to cause landslides of various types: (1) $M \approx 4.0$--rock falls, rock slides, soil falls, and disrupted soil slides, (2) $M \approx 4.5$--soil slumps and soil block slides, (3) $M \approx 5.0$--rock slumps, rock block slides, slow earth flows, soil lateral spreads, rapid soil flows, and subaqueous landslides (4) $M_s \approx 6.0$--rock avalanches, and (5) $M \approx 6.5$--soil avalanches.

In spite of the lack of reports of landslides in earthquakes smaller than these, the possibility of smaller events occasionally causing landslides cannot be discounted. All types of earthquake-induced landslides can also be triggered by non-seismic causes and, if failure of a slope is imminent before an earthquake, a landslide could be initiated even by weak shaking.

Magnitude and Area Affected by Landslides

Data relating magnitudes of historical earthquakes to the areas affected by landslides are plotted in Figure 1. Each area was measured by drawing a

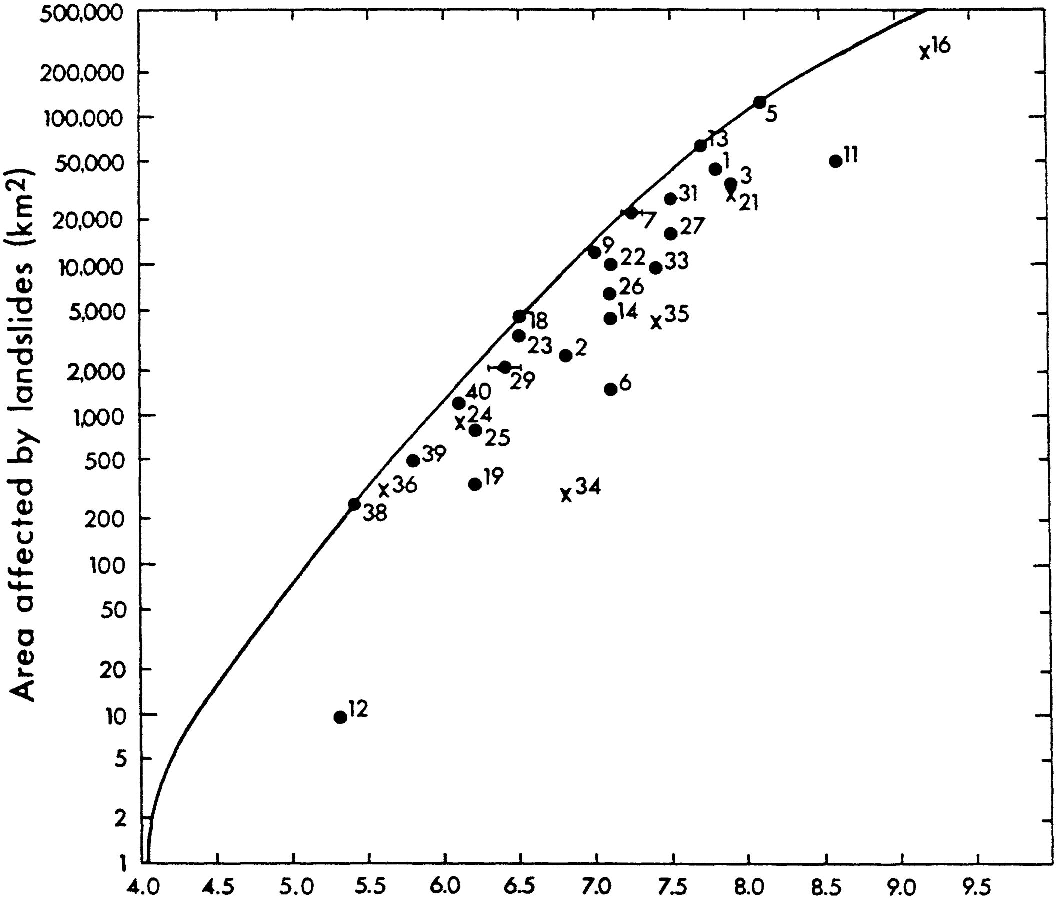

Figure 1. Area affectd by landslides in earthquakes of different magnitudes. Numbers beside data points are referenced to earthquakes in Keefer (1984). Dots = onshore earthquakes. X's = offshore earthquakes. Horizontal bars indicate range in reported magnitudes. Solid line is approximate upper bound enclosing all data. (Figure from Keefer, 1984).

boundary around all reported landslide localities and calculating the size of the region enclosed. The areas are those where ground shaking was strong enough to trigger landslides on susceptible slopes. Not all slopes within these areas produced landslides, and zones of high landslide density were commonly interspersed with zones having few or no landslides. Most areas affected by landslides were irregular in shape and asymmetric with respect to epicenters and fault ruptures. Nevertheless, areas affected by landslides show a strong correlation with magnitude. An approximate upper bound fit to the data in Fig. 1 gives the greatest area likely to be affected by landslides in an earthquake of given magnitude.

Magnitude and Maximum Distance of Landslides From Epicenter and Fault Rupture

Other measures of the relation between earthquake magnitude and landslide distribution are the maximum distance from the epicenter or fault rupture at which an earthquake causes landslides. Determinations were made for each of the following major three categories of landslides: (1) disrupted slides and falls--rock falls, rock slides, rock avalanches, soil falls, disrupted soil slides, and soil avalanches; (2) coherent slides--rock slumps, rock block slides, soil slumps, soil block slides, and slow earth flows; and (3) lateral spreads and flows--soil lateral spreads, rapid soil flows, and subaqueous landslides (Table 1).

The maximum distance from the epicenter to a reported landslide in each category was determined for each historical earthquake in which landslide identifications and locations were sufficiently complete and precise. An example of the data, showing the maximum distance of disrupted slides or falls from the epicenters of the historical earthquakes, is illustrated in Figure 2. Approximate upper bounds were fit to these data for all three categories of landslides (Figs. 2 and 3). The relations between the upper bounds (Fig. 3) suggest that disrupted slides and falls can be triggered by weaker shaking than coherent slides and that coherent slides can be triggered by weaker shaking than lateral spreads or flows.

Because seismic energy is released throughout a zone of fault rupture rather than at a single point, maximum distance of landslides from a fault-

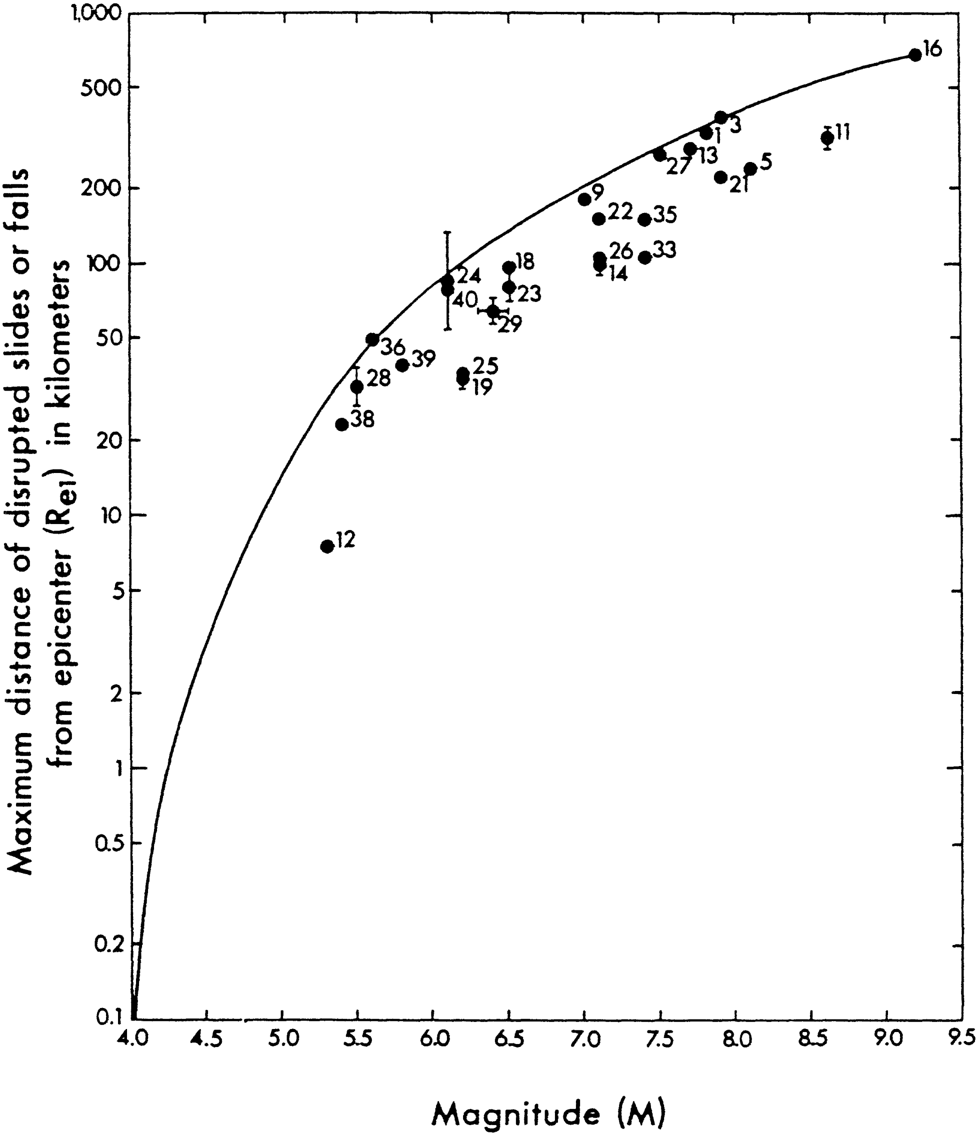

Figure 2. Maximum distance from epicenter to disrupted slides or falls in earthquakes of different magnitudes. Numbers beside data points are referenced to earthquakes in Keefer (1984). Vertical bars indicate uncertainties, where known, in locations of epicenters, in locations of most distant landslides or both. Horizontal bars indicate range in reported magnitudes. Solid line is approximate upper bound enclosing all data. (Figure from Keefer, 1984).

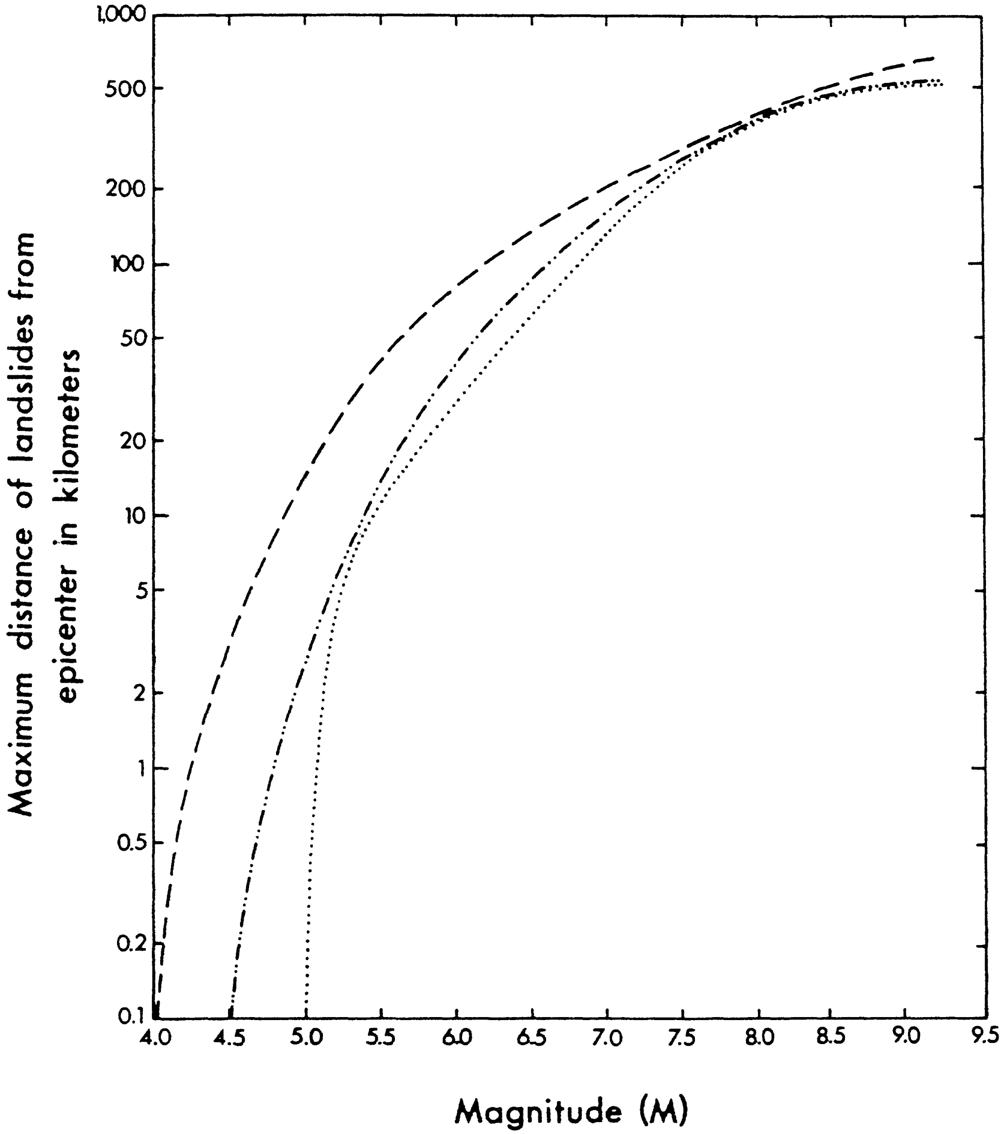

Figure 3. Upper bounds showing maximum distances from epicenters at which landslides of different types are likely to occur. Dashed line is bound for disrupted slides and falls, dash--double-dot line is bound for coherent slides, and dotted line is bound for lateral spreads and flows. (Figure from Keefer, 1984).

rupture zone may be a more refined measure than epicentral distance. Fault-rupture zones were determined for the historical earthquakes using surface faulting, aftershock hypocenter distributions, combinations of surface fault-ruptures or mapped fault-traces and aftershock hypocenters, or, in 4 cases discussed by Keefer (1984), using other criteria. As suggested by Kanamori (1977), aftershocks used to define fault-rupture zones were limited to those that occurred within 24 hours after main shocks wherever data permitted. Distances were measured to the nearest point, whether surface or subsurface, on the dipping fault-plane defined by the aftershock hypocenters; where no surface rupture was present, the top of the aftershock-hypocenter zone was taken as the top of the fault-rupture zone. Figure 4 shows upper-bound fits to the data relating magnitudes to maximum distances of landslides from fault-rupture zones.

Landslides and Seismic Shaking Intensity

Numerous intensity scales have been devised to characterize the severity of earthquake shaking using human perceptions, movement of objects, shaking damage to engineering structures, landslides, and other ground failures. The scale most commonly used in the United States is the Modified Mercalli (MMI) scale proposed by Wood and Neumann (1931) and revised by Richter (1958). The Modified Mercalli scale uses landslide-related criteria to help define some levels of intensity. These criteria are used in conjunction with others, and the intensity assigned to a locality is based on a preponderance of evidence. Intensity data from different localities are then commonly smoothed to produce an isoseismal map showing contours of equal intensity. Because of the combination with other criteria and because of the smoothing, landslide-related criteria are given relatively little weight in many isoseismal maps, and landslides are found in areas with intensities lower than specified on the scale.

To determine the lowest intensities for landslides according to the preponderance of evidence, isoseismal maps were compared to maps showing landslide distribution in the historical earthquakes, and the lowest intensity at which landslides were triggered in each earthquake was determined. Determinations were made separately for disrupted slides and falls, coherent

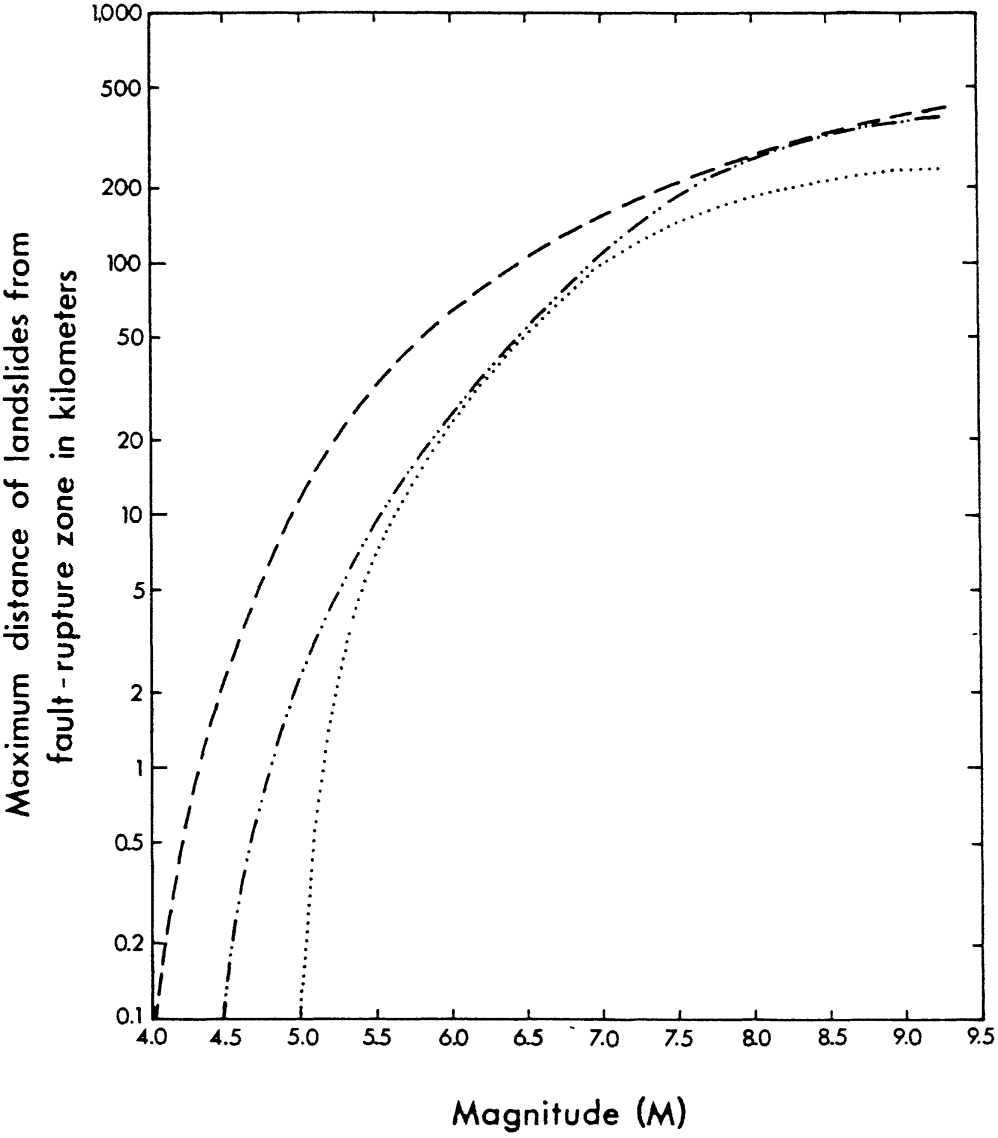

Figure 4. Upper bounds showing maximum distances from fault-rupture zones at which landslides of different types are likely to occur. Dashed line is bound for disrupted slides and falls, dass--double-dot line is bound for coherent slides, and dotted line is bound for lateral spreads and flows. (Figure from Keefer, 1984).

slides, and lateral spreads and flows. The predominant minimum intensity for disrupted slides and falls was MMI VI, and the lowest intensity reported in any earthquake was MMI IV. The predominant minimum intensity for coherent slides, lateral spreads, and flows was MMI VII and the lowest intensity reported was MMI V. Thus, disrupted slides and falls are triggered, on the average, at one intensity level lower than other types of landslides, and the minimum intensity for coherent slides approximately equals that for lateral spreads and flows.

HAZARDS CAUSED BY EARTHQUAKE-INDUCED LANDSLIDES

Whereas most or all types of earthquake-induced landslides pose some hazard to human life, rapid soil flows, rock avalanches, and rock falls together caused at least 90 percent and possibly more than 99 percent of the reported landslide deaths in the 40 historical earthquakes. Rapid soil flows killed at least 25,000 people, rock avalanches at least 21,000, and rock falls at least 800 in these earthquakes (Keefer, 1984). Rock avalanches and rapid soils flows, the two leading causes of death, are similar in that they can travel several kilometers at high velocities on slopes of a few degrees. Most deaths caused by these landslides were due to burial of cities or villages located on gently sloping ground several kilometers from the sites of landslide initiation. Rock falls, the third leading cause of death, are also the most abundant landslides in seismic events (Table 2) and occur in virtually all types of rocks on slopes steeper than 40°. Areas at risk from rock falls are limited by distances that boulders can bound or roll once they reach the bases of the steep slopes on which the rock falls originate; the maximum such distance reported in the historical earthquakes was approximately 800 m.

Earthquake-induced landslides have also damaged many types of engineering structures and caused a large percentage of the total economic losses in several earthquakes. Economic losses were reported from all types of earthquake-induced landslides except slow earth flows. Rapid soil flows, rock avalanches, and rock falls are as hazardous to civil works as they are to human life. In addition to these, the leading causes of landslide-related damage in the historical earthquakes were soil slumps and soil lateral spreads. The propensity of soil slumps and soil lateral spreads for causing economic losses in earthquakes is due to their abundance (Table 2) and to their common occurrence on gentle slopes in man-made fill and in alluvium on floodplains, which are likely locations for human habitation.

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QUANTITATIVE PREDICTION OF EARTHQUAKE-INDUCED LANDSLIDES

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INTRODUCTION

The purpose of this lecture is to describe a newly developed method for predicting the areal limits of various types of landslides triggered by large earthquakes. The method is based both on empirical studies of landslides in historical earthquakes and on a numerical analysis of dynamic slope-stability. The lecture by David Keefer described the characteristics of earthquake-induced landslides and the empirically-observed relations between earthquake magnitude and areal extent of landslides. This second lecture describes a dynamic slope-stability analysis that links slope failure with levels of ground motion, followed by a magnitude/distance relationship derived from strong-motion seismology, and closing with an example of the application of these techniques to a hypothetical event in the Los Angeles region.

A SIMPLE MECHANICAL MODEL FOR EARTHQUAKE-INDUCED LANDSLIDES

Whether or not a particular slope will fail during an earthquake depends on the answers to two questions: 1) How stable was the slope before the earthquake? and, 2) How severely did the earthquake shake the slope? Thus, we seek some way to numerically express both the preearthquake stability of the slope and the severity of the shaking.

We begin the analysis by examining slope stability under static (nonseismic) conditions. Figure 1 depicts a hypothetical landslide of the block-slide type. Two forces act on the center of mass of this potential landslide: the gravitational load, L, which is the downslope component of the weight of the landslide mass; and an opposing force, R, which is the resistance of the slope to deformation from downslope movements. The maximum

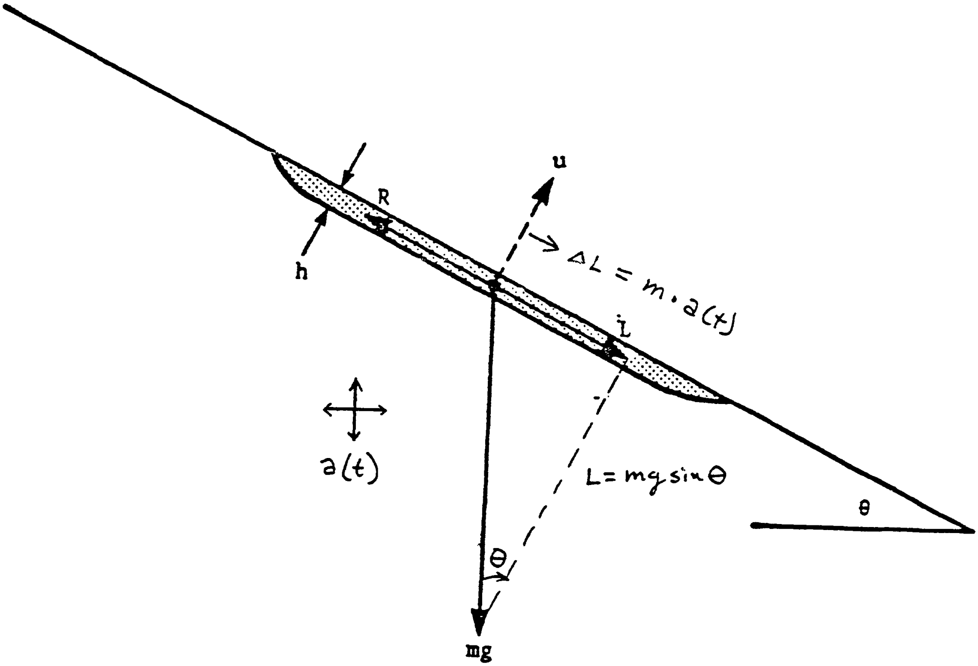

Figure 1. Sketch of a hypothetical slope with potential landslide of blockslide type, showing interaction of load, L, and resistance, R, forces, and the seismic acceleration field, a(t).

resistance, Rmax, is a function of the strength of the slope material, and may be expressed as the integration of the shear strength over the area of the sliding surface. For following standard slope engineering nomenclature, we express the static slope stability as the ratio of the maximum resistance to the load and call this ratio the factor of safety:

$$FS = Rmax/L$$

For a seismic situation, this analysis must be taken a step farther. During an earthquake, seismic waves passing through the slope create an acceleration field, a(t), which exerts an additional body force on the mass, m, of the potential landslide. During the time of strong shaking, this seismically induced force will fluctuate and sporadically add to the downslope load. If this downslope seismic force is sufficiently large, the total downslope load, L + m a(t), may exceed the maximum resistance, Rmax, and the slope will undergo irreversible displacement. Following Newmark (1965), this level of ground motion may be expressed in terms of a critical acceleration, A_c, which is the seismic acceleration and, when multiplied by the mass, m, is equal to the difference between Rmax and the gravitational load, L

$$mA_c = R_{max} - L$$

$$\text{or,} \quad mA_c = \left(\frac{R_{max}}{L} - \frac{L}{L}\right)L = (FS - 1)L$$

so that: $$A_c = (FS-1)\, L/m$$

therefore: $$A_c = (FS-1)g \sin \Theta$$

where: g = acceleration due to gravity, and Θ = the slope angle. Thus, the critical acceleration is the minimum ground acceleration required to overcome the maximum resistance of the slope to sliding. In order to cause a landslide on a particular slope, the ground acceleration must exceed A_c for a finite length of time. Therefore, we use the critical acceleration as the numerical measure of the stability of a slope and its susceptibility to an earthquake induced landslide.

If the slope steepness and the strength of the slope materials is known, we may estimate the critical acceleration of a shallow landslide on an infinitely long slope (Fig. 1) from the equation:

$$A_c = \frac{c'}{\delta h} + (\tan \phi \cos \theta - \sin \theta)$$

where: θ = slope steepness angle; δ = specific weight of slope material; h = thickness of the potential landslides; c' = effective cohesion; and ϕ = effective friction angle of the slope material.

This infinite-slope analysis may be further generalized by grouping various types of slope materials into three broad lithologic categories, each with assumed, average values of c' and ϕ' (for dynamic conditions) (Figure 2). Figure 2 is a plot of Ac versus slope steepness for each of these three categories of slope materials. This plot was used, in conjunction with the techniques described below, to derive susceptibility units for an experimental seismic slope stability map of San Mateo County, California, (Wieczorek and others, 1984).

THE NEWMARK ANALYSIS

The critical acceleration, A_c, represents the ground motion necessary to begin the process of slope failure (Newmark, 1965). However, because small irreversible displacements do not necessarily result in a slope failure of practical or engineering significance, how far the landslide would move must now be determined. This may be estimated using a technique developed by Newmark (1965), which hereafter will be called the "Newmark analysis." The analysis, which models the landslide as a rigid friction block resting on a slope, utilizes a record of ground acceleration versus time (a strong-motion record) to calculate the expected displacement of the block.

The significance of the critical acceleration is illustrated in Figure 3a, a plot of acceleration versus time for an actual strong-motion record. Superimposed on this strong-motion record is a horizontal line corresponding to the critical acceleration, A_c, of a hypothetical slope. If the strong-

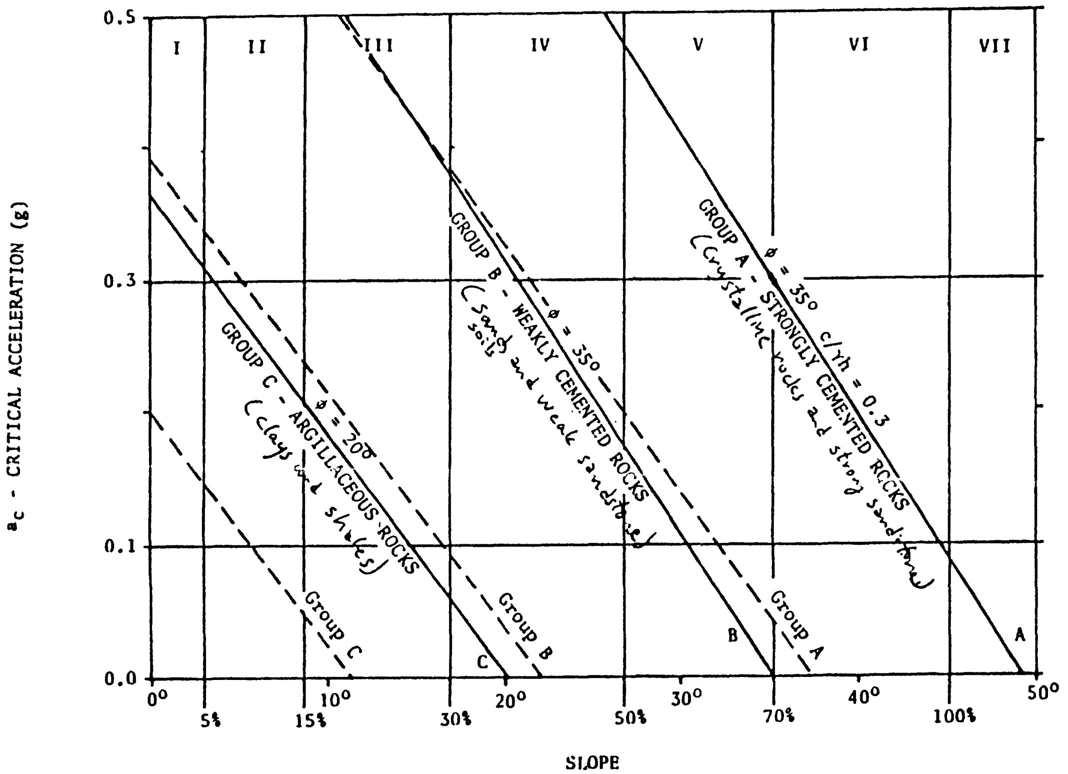

Figure 2. Plots of critical acceleration, Ac, versus slope steepness for three sets of lithologies: A - crystalline rock and well-cemented sandstone, B - sandy soil and poorly-cemented sandstone, and C - clayey soil and shale. The cohesion factor, c'/ γh, for the A group assumes values of c' = 300 psf (67 kP), δ= 100 pc (1600 kg/m^3), and h= 10 ft (3.3 m). The angle of internal friction, θ (peak strength, undrained conditions) is equal to 35° for clayey soils and shales. The solid lines depict dry slope materials and the dashed lines depict saturation from the slide plane to the surface.

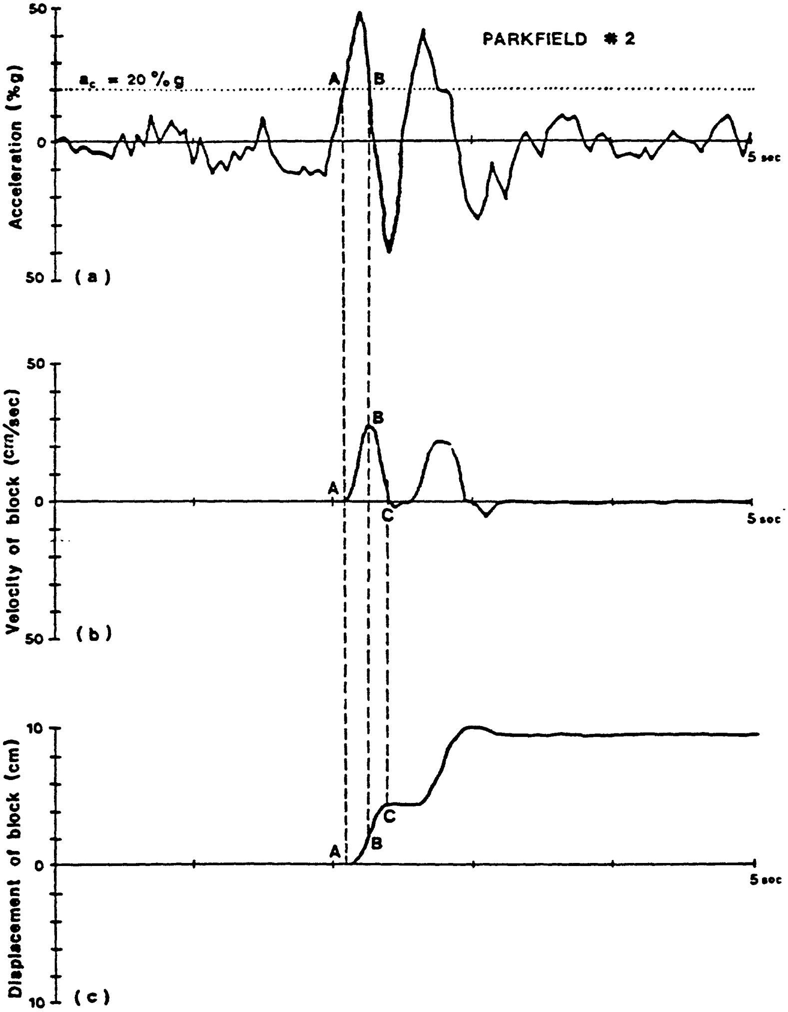

Figure 3. Demonstration of the algorithm for the Newmark analysis: a) Strong-motion record (Parkfield, Station 2, 1966) with critical acceleration, Ac = 20% g, for a hypothetical slope superimposed. b) Velocity of block versus time. c) Displacement of block versus time.

motion record contained a peak acceleration less than the critical for our hypothetical slope, then the displacement of the block would be zero and the slope would remain stable. However, when the ground motion does exceed the critical acceleration, the next step is to integrate those portions of the strong-motion record which lie above the critical acceleration (Figure 3).

The integration begins at point A (Figure 3a, b, c), where the ground acceleration first exceeds the critical acceleration. We integrate that portion AB of the ground motion above A_c over time, to calculate the velocity of the block (Figure 3b). At point B, the block reaches the maximum velocity for this acceleration pulse. After point B, when the ground acceleration drops below the critical acceleration, the block is still in motion because of its inertia, but is now being decelerated by the friction that still exists beneath the block. Also, the ground acceleration may well swing around in the opposite direction and directly oppose the motion of the block (as at the end of the second pulse). Eventually, the motion of the block will cease and the block will come to rest at point C. Finally, the displacement of the block is calculated by integrating the velocity of the block over time (Fig. 3c).

In his original report, Newmark (1965) was concerned with artificial embankments, and the computed displacement was used as a design factor: one may either design the embankment to tolerate the seismically induced displacement without significant loss of strength or design a stronger embankment to reduce the seismic displacement. In either case, the resistance of the slope to sliding, expressed as A_c, may be treated as a constant during the dynamic analysis. In actual cases, however, as the displacement becomes significant, the slope material may lose some shear strength, thus decreasing the A_c. If the displacement continues, the slope may lose so much strength that A_c will go to zero and the slope will become statically unstable (FS 1). In this case, the slope may continue to move even after the seismic shaking ceases, and a landslide results.

For a regional application, such as mapping the susceptibility, however, it is not practical to consider decreases in A_c as a function of displacement in the numerical analysis. Instead, the numerical analysis of the strong-motion record is performed under the assumption of a constant value of critical acceleration. The displacement thus calculated is then compared to a "benchmark" value of displacement, herein termed the "critical displacement," beyond which it is likely that the slope would lose a significant amount of strength.

The actual value of the critical displacement may vary widely, depending on the mechanism of slope failure (rock fall, slump, block glide, etc.), the lithology, the slope geometry, and the previous history of slope movement. In order to quantify this parameter for purposes of regional application, we assign a value of 10 cm for the critical displacement of coherent slides (slumps and block glides). A value of 10 cm is consistent with two ideas: 1) In moving 10 cm, a landlside will probably have broken most of the cohesion bonds along the slip surface and across the scarp at the top of the landslide, as well as having begun to override the toe at the bottom. 2) A displacement of 10 cm is our estimate of the displacement that a typical house foundation could withstand without severe damage. Rock falls and most other types of disrupted landslides involve a significant component of tensile failure, a more brittle mechanism than shear for which the critical displacement is lower. So, we shall assume a value of 2 cm for the critical displacement for disrupted landslides.

By taking the same given strong-motion record and repeatedly running the Newmark analysis (as depicted in Figure 3) for a family of values of the critical acceleration, A_c, from zero to the peak acceleration (A) of the record, a predicted displacement for each value of Ac can be calculated. In this way, we may calculate the response of any possible slope to the seismic ground motion represented by the given strong-motion record. (This procedure is analogous to the "response spectrum" analysis used in engineering design of manmade structures to resist earthquakes.)

Figure 4 is the plot of critical acceleration versus the corresponding displacements predicted from a number of strong-motion records. By taking the value of A_c which corresponds to a predicted displacement equal to the critical displacement (10 cm for coherent landslides) for a given strong-motion record, a measure of the seismic shaking intensity may be derived that

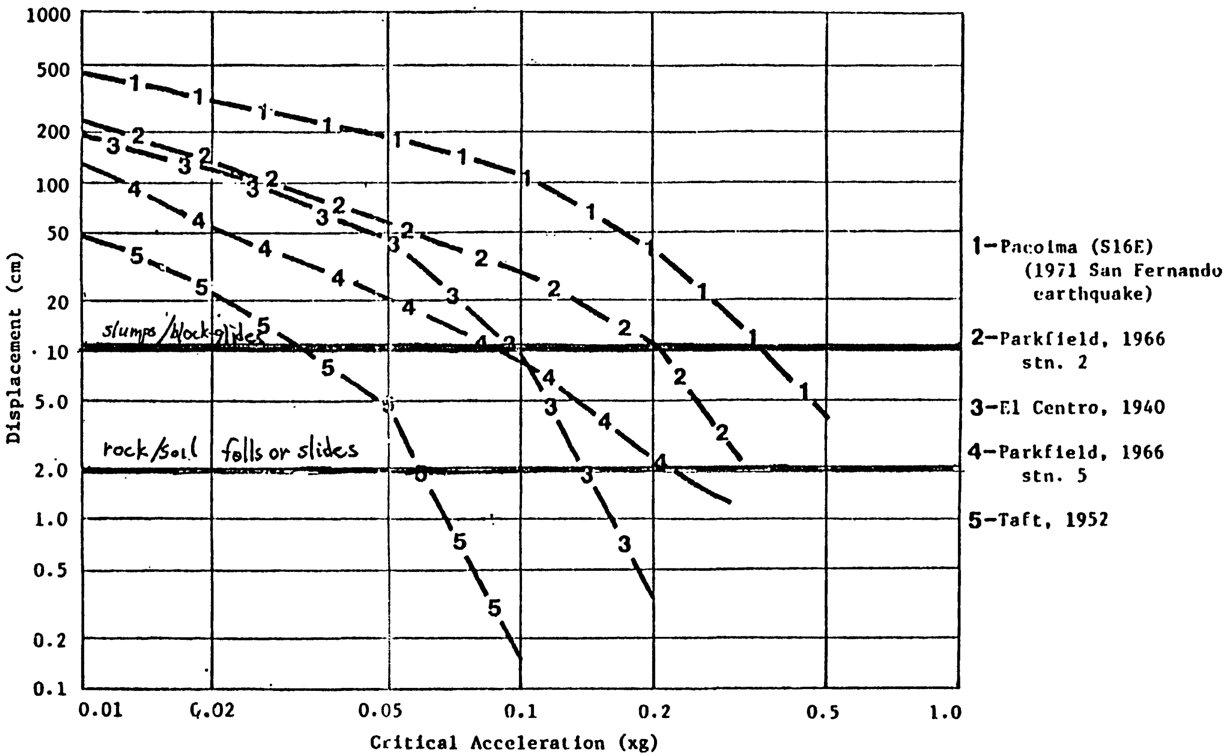

Figure 4. Plots of displacement predicted by the Newmark analysis, as a function of assummed critical acceleration, for several strong-motion records from California earthquakes. The stronger of the two horizontal components of the records were used in each case. Also indicated for reference are the values of critical displacement for coherent landslides, 10 cm, and for discoherent landslides, 2 cm.

is directly related to slope stability. We abbreviated this value of A_c as $(A_c)_{10}$. Thus, for example, the Taft record has an $(A_c)_{10} = 0.03$ g, while the Pacoima record has and $(A_c)_{10} = 0.33$ g (Figure 4).

The $(A_c)_{10}$ value expresses the critical acceleration of the most resistant slope that would be expected to undergo significant displacement as a coherent slide for this level of seismic shaking. All slopes with $A_c > (A_c)_{10}$ should survive this shaking without significant failure. In the same way, one may reference the severity of seismic shaking to the critical displacement for disrupted landslides (Uc = 2 cm) by deriving a value for$(A_c)_2$. Therefore, $(A_c)_{10}$ and $(A_c)_2$ become useful parameters for expressing seismic shaking severity for predicting earthquake induced landslides.

RELATING SHAKING SEVERITY TO SLOPE STABILITY

A somewhat less complicated way of expressing the severity of seismic shaking as recorded by a strong-motion record is that developed by Arias (1970). Arias intensity, Ia, is defined as an integration over time of the acceleration squared (Arias, 1970):

$$Ia = \frac{\pi}{g} \int_0^{\infty} (a(t)^2 \, dt$$

The Arias intensity is expressed in units of velocity, usually in meters per second. We may use the Arias intensity to replace the hundreds of points in a digitized strong-motion record with a single value and simplify the seismic input to the Newmark analysis by taking advantage of newly developed empirical relations between $(A_c)_{10}$, $(A_c)_2$, and the Arias intensity (Fig. 5).

Figure 5 plots $(A_c)_{10}$ and $(A_c)_2$, versus the Arias intensity. Each data point represents a single strong-motion record for which both the Arias intensity and the value of $(A_c)_{10}$ and $(A_c)_2$ were calculated. Figure 5 shows a linear logarithmic relation plotted between the Arias intensity and both $(A_c)_{10}$ and $(A_c)_2$. For many cases, where only approximate estimates of $(A_c)_{10}$ and $(A_c)_2$ are required, we can avoid performing a number of Newmark analyses by simply calculating the Arias intensity and using the plot in Figure 5.

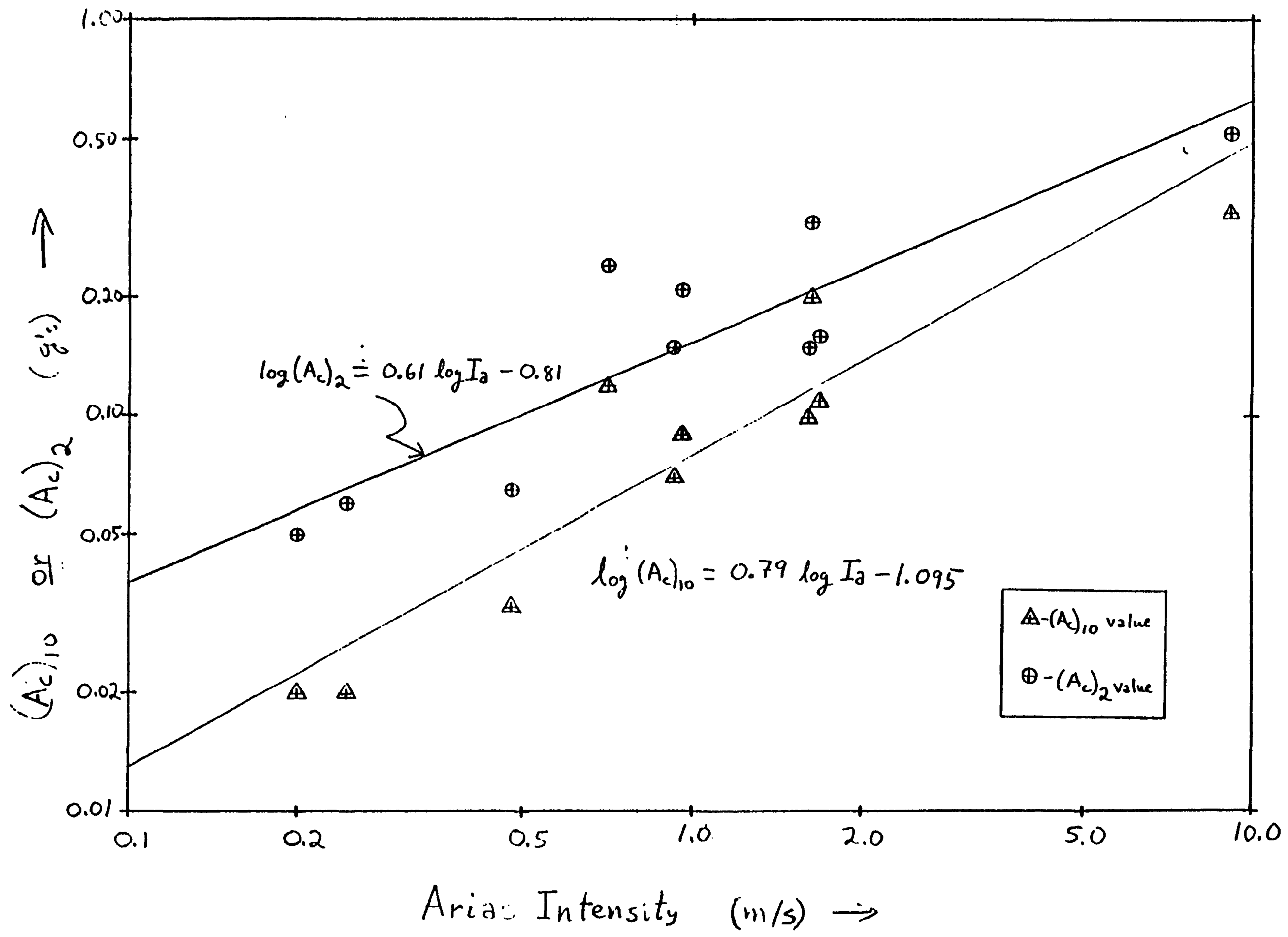

Figure 5. Plots of $(A_c)_{10}$ (triangles) and $(A_c)_2$ (circles) versus Arias intensity (I_a) for ten strong-motion records. Regression lines are also plotted for $\log (A_c)_{10}$ versus $\log I_a$ and $\log (A_c)_2$ versus $\log I_a$. The standard deviations for these regressions are 0.144 and 0.151, respectively.

SEVERITY OF SHAKING AS A FUNCTION OF MAGNITUDE AND DISTANCE

Because strong motion records are available for only a limited number of sites in a relatively small number of earthquakes, an independent technique is needed to estimate the distance over which a particular earthquake could trigger landslides on slopes with a given value of critical acceleration. To fill this need, we have developed an empirical relation for the severity of seismic shaking (as measured by the Arias intensity) as a function of earthquake magnitude and the distance from the seismic source.

Because the data set of reported values of Arias intensity is rather limited, two assumptions were used to constrain the statistical analysis: 1) For a given earthquake magnitude, an inverse square relation was assumed between the Arias intensity and the source distance. 2) for a given distance from the seismic source, the logarithm of the Arias intensity is assumed to be a linear function of the magnitude of the earthquake as measured by the moment-magnitude scale of Hanks and Kanamori (1979). To have a common reference for magnitude, we corrected the reported Arias intensity value, Ia, for a variety of magnitudes to values equivalent to $\underline{M}$ 6.5 event for the same source distance, I_a*, using the linear assumption; that is $\log I_a^* = \log I_a + (6.5-\underline{M})$. We then compare these "corrected" values of Arias intensity with the log of the source distance (Figure 6) and derived the value of the proportionality constant, K which best fits the data, K = -4.1. The source distance, r, was also corrected for focal depth. The standard deviation was found to be equal to 0.44. Thus, our magnitude-distance relationship for Arias intensity becomes:

$$\log I_a = 4.1 + \underline{M} - 2 \log r + 0.44 P$$

With this relationship between magnitude, source distance, and Arias intensity, we can estimate the seismic shaking severity at any site given the magnitude of the event, the distance from the site to the seismic source, and the probability of the actual I_a exceeding the predicted I_a (P).

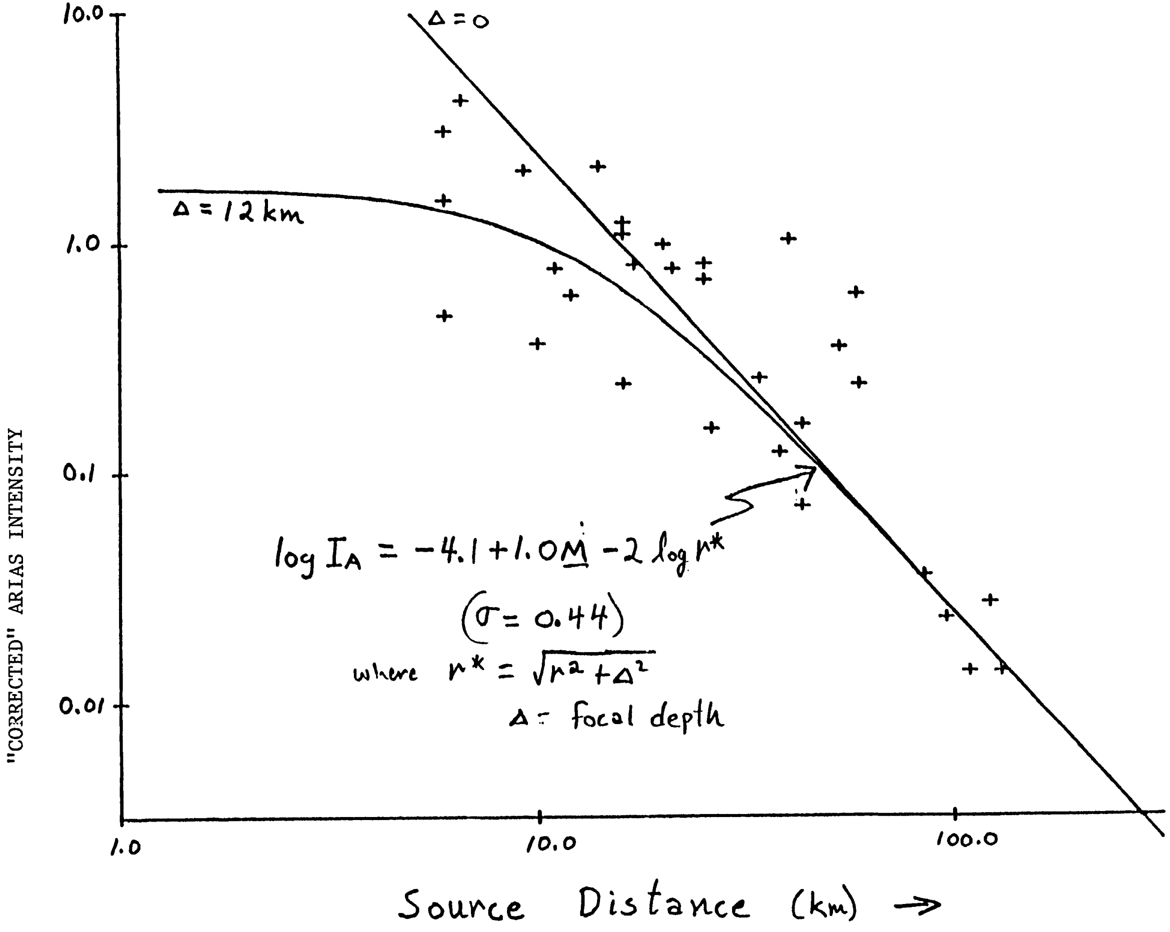

Figure 6. Plots of Arias intensity versus distance form the seismic source area. The value of Arias intensity were calculated from strong-motion records, then adjusted to be equivalent to M 6.5 (moment-magnitude) event at the same source distance, based on an assumed log-linear relationship between magnitude and Arias intensity for a given distance.

PREDICTING THE LIMITS OF LANDSLIDING FROM AN EARTHQUAKE

What is the minimum seismic shaking intensity required to trigger landsliding? From a comparison of our $M/r/I_a$ relation and the relations between magnitude and most distant landslides developed from our studies of historic earthquakes (see Keefer notes) and taking the statistical variations into account, the "best fit" to the historic earthquake data is a "threshold value" of Ia = 0.5 m/s for coherent landslides and I_a = 0.15 m/s for disrupted landslides.

We may now use our $M/r/I_a$ relation (Fig. 6) to estimate the source distance from a given earthquake at which the probability of the seismic shaking severity exceeding any given value is 50% (P = 0). Taking I_a = 0.5 m/s as the threshold shaking severity for coherent slides, we estimate this distance to be approximately 24 km for a M 6.5 event. Figure 7 is a map of the Los Angeles area with a hypothetical seismic source zone for a M 6.5 event on the northern Newport-Inglewood fault. A line is drawn 24 km from the source to indicate the area within which there is a greater than 50% chance of the ground motion exceeding the threshold severity. Figure 7 also includes a line 65 km from the source zone, corresponding to the predicted limit for coherent slides as determined empirically from worldwide historical data. Beyond this 65-km limit, coherent slides are unlikely during this event, even for susceptible slopes. In a middle zone, between 24 km and 65 km from the fault rupture, the probability that shaking would be severe enough to cause coherent slides on susceptible slopes is finite but less than 50%.

SUMMARY

The probability of a landslide occurring on a particular slope during a particular earthquake is a function of both the preearthquake stability of the slope and the severity of the seismic ground motion. The preearthquake stability of the slope is controlled by both the strength of the slope material and the steepness of the slope. The susceptibility of a slope to seismically induced landsliding may be expressed as the critical acceleration, A_c, which may be calculated from the static factor of safety as determined by a standard slope stability analysis.

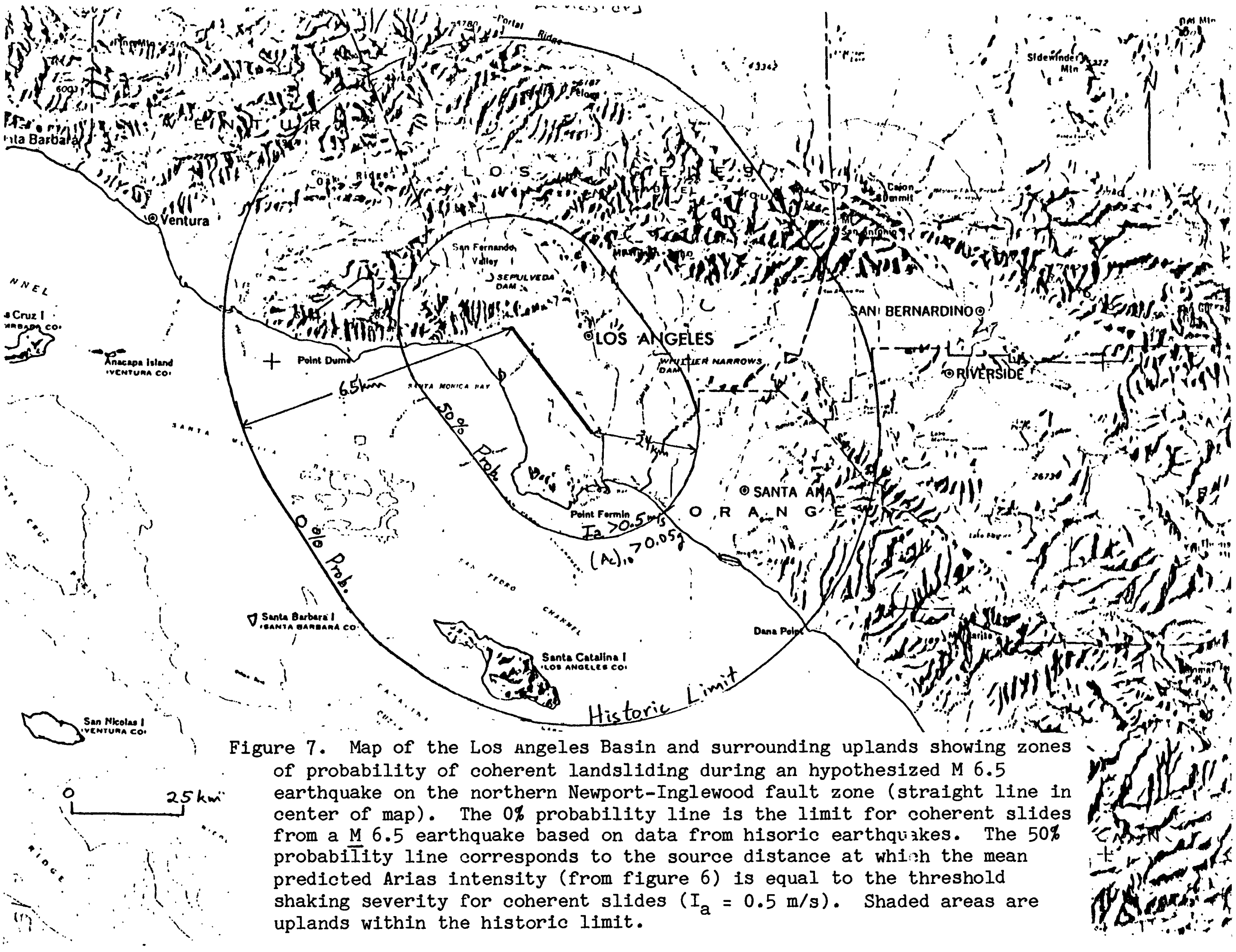

Figure 7. Map of the Los Angeles Basin and surrounding uplands showing zones of probability of coherent landsliding during an hypothesized M 6.5 earthquake on the northern Newport-Inglewood fault zone (straight line in center of map). The 0% probability line is the limit for coherent slides from a M 6.5 earthquake based on data from hisoric earthquakes. The 50% probability line corresponds to the source distance at which the mean predicted Arias intensity (from figure 6) is equal to the threshold shaking severity for coherent slides (I_a = 0.5 m/s). Shaded areas are uplands within the historic limit.

The second factor, severity of the ground shaking required for earthquake-induced landsliding, has been investigated using both empirical data from historical earthquakes (Keefer, 1984) and a numerical technique developed by Newmark (1965). The Newmark analysis allows for inputs of both the static slope stability and a seismic strong-motion record, thus linking slope stability and seismic ground-motion. The Newmark analysis computes the displacement of a rigid friction block which is used to represent a potential landslide on the slope under study. We have defined the "critical displacement" as that beyond which the slope can be considered to have produced a landslide. We have assigned values of 10 cm and 2 cm as the critical displacements for coherent and disrupted landslides respectively. The severity of seismic shaking required to cause coherent slides is thus defined as that which according to the Newmark Analysis, would produce a displacement > 10 cm on a slope with a given A_c, that is $(A_c)_{10}$.

Using both the theoretical and the historic/empirical studies, we have delineated three zones surrounding the seismic event: 1) a zone (within the 50% probability line) within which there is a high probability of failure of susceptible slopes ($A_c < 0.05$ g); 2) a zone with a less than even, but still finite probability of failure of susceptible slopes; and, 3) a zone, beyond the outer limit defined by data from worldwide historical events, which is so far from the seismic source that the probability of landslides is very small, even on susceptible slopes.

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LANDSLIDE CLASSIFICATION FOR IDENTIFICATION
MUD FLOW AND OTHER LANDSLIDE HAZARDS

By

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INTRODUCTION

The adoption and summary review of a widely accepted, systematic landslide classification is a necessary first step in a study to evaluate the feasibility of a nationwide program to identify areas at risk from mud flows and other landslides. At the request of the Federal Emergency Management Agency (FEMA), a joint effort was undertaken by the U.S. Geological Survey (USGS) and FEMA to address the needs of FEMA programs in flood insurance, disaster assistance and hazard mitigation. The report of the National Research Council (NRC) Committee on Methodologies for Predicting Mudflow Areas (1982) addressed FEMA concerns that floodplain maps do not identify areas of mud flow hazard for the purposes of the "mudslide" provisions of the National Flood Insurance Program (NFIP), and has indicated that "clear water floods, mud floods, mud flows, and other landslides" consitute a flood-landslide continuum. The NRC Committee addressed the distinction between floods and landslides, and identified mud floods as floods in contrast with mud flows as landslides, but did not address the distinctions between mud flows and other landslides. Discriminating between "mudslides (i.e. mudflows)" and other landslides remained a primary concern in the FEMA administration of the mudslide provisions of the NFIP. Mud flows and other landslides also pose a nationwide potential for disaster (causing major damage annually to 4/5 of the States of the U.S.) and are, therefore, also important to FEMA administration of programs in Disaster Assistance and Hazards Mitigation.

NFIP regulations treat floods and "mudslides (i.e., mudflows)" together, while addressing other landslides as different phenomena, even though many individual events display some properties common to all three. Consequently, evaluating the feasibility of nationwide identification of areas susceptible to "mudslides (i.e., mudflows)", and of areas susceptible to other landslides that might constitute a National disaster, requires addressing several questions:

- What hazardous conditions and natural processes are technically appropriate for inclusion under terms such as "floods", "mudslides", and "other landslides"?
- What levels of comprehensiveness and accuracy are required in delineating the potentially hazardous areas?
- What are the probable costs for different methods of identifying and delineating areas of potential hazards?
- Can a methodology having a satisfactory level of comprehensiveness and accuracy be developed, that can be applied on a nationwide basis, for which the costs and schedules for completion are within reasonable bounds?

The actual selection of specific kinds of events to be included in a nationwide program of identifying mud flow and other landslide hazards is a socio-political decision, as is the choice of standards for accuracy and comprehensiveness. On the other hand, technical expertise can provide a uniform descriptive terminology for adoption as an aid to consistency within and between technical and regulatory language regarding landslides. Technical analysis can also suggest options that permit the socio-political decisions to be made in a context that includes consideration of the "trade-offs" in accuracy, comprehensiveness, and costs of implementing hazard identification programs.

Using an established classification system, the evaluation of the feasibility of a program to identify areas at risk from mud flows and other landslides can proceed to examine various methods that might be utilized in delineating landslide susceptibility, and to make some approximate estimates of their costs. In seeking that objective, technical aspects of the above questions have been addressed in a sequence of steps:

1. Adoption of a well-documented widely accepted classification system and systematic nomenclature--specifically, that of Varnes (1978)--as a basis for naming and describing various kinds of landslide processes.

2. Selecting a number of landslide terms to represent the wide variety of different kinds of landslides in the geologic-hydrologic continuum of slope and channel mass-transport events, and listing them in an array that reflects some concensus as to the relative proportion of wet flow among the mechanisms associated with each kind.

3. Listing, with the above array, the general characteristics that might be used to recognize which class of landslide has occurred, the most common kinds of damage to be expected, and the most common triggering events. The array can then be considered as one of the means for correlating regulatory language regarding NFIP distinctions between mud flows and other landslides with technical aspects of recognition and classification. It can also be used to identify and describe the kinds of mud flow and other landslide events that are most likely to result in disaster declarations, and those most likely to be associated with other disaster-causing events such as rainstorms, earthquakes, and volcanic eruptions.

To identify the hazardous conditions and events in a context that avoids inconsistencies between technical definitions and regulatory statements, requires agreement on terminology used in both administrative and technical definitions. For this report, the classification system of Varnes (1978) has been adopted because it is a widely accepted, well documented, systematic technical classification that is readily available as a published document. As such, it can be used in an interactive process of review and revision by both technical and administrative personnel, as well as a basis for further discussion of questions of comprehensiveness, accuracy, schedule, and cost.

LANDSLIDE CLASSIFICATION AND NOMENCLATURE

The evolution of landslide nomenclature (distinctive names for different kinds of events) has been largely unsystematic. The introduction of new terms has generally been a one-at-a-time reflection of new observations and physical measurements. Classification systems generally have been devised subsequent to the first uses of the names that appear in the classifications. Classification systems are generally erected with the purpose of examining the similarities and differences between events for systematic changes. Numerous landslide classifications have been devised by geologists and engineers. Several different systems have been widely used by different workers in different parts of the world, different parts of the U. S., and even by different workers in the same areas. Each system has been devised with a somewhat different perspective regarding the most important factors. One result has been that some similar events have been called by different names in different classification systems, and, conversely, different kinds of events have been called by similar names. Consequently, names for different classes of landslides, that are not referenced to some specific classification system can be uninformative and confusing. Popular media usage of terms such as "mudslide" also contribute to confusion in landslide nomenclature. The most comprehensive recent classification available, and one that also approaches nomenclature systematically, is that of D. J. Varnes (1978).

The technical terminology used in this report follows the classification described in Landslides, Analysis and Control, Transportation Research Board Special Report 176 (Varnes, 1978). An abbreviated tabulation is shown on Figure 1. Varnes' classification and nomenclature gives primary consideration to the type of movement and secondary consideration to type of material. The systematic nomenclature of Varnes (1978) utilizes names compounded of terms for mechanism of mass movement downslope and for the kind of material constituting the moving mass. The simple names in Figure 1 can be made progressively more detailed and informative by expansions using carefully-defined descriptor words. Where important components of complex landslides can be identified, names are compounded

TYPE OF MOVEMENT			TYPE OF MATERIAL		
			BEDROCK	ENGINEERING SOILS	
				Predominantly coarse	Predominantly fine
FALLS			Rock fall	Debris fall	Earth fall
TOPPLES			Rock topple	Debris topple	Earth topple
SLIDES	ROTATIONAL	FEW UNITS	Rock slump	Debris slump	Earth slump
	TRANSLATIONAL		Rock block slide	Debris block slide	Earth block slide
		MANY UNITS	Rock slide	Debris slide	Earth slide
LATERAL SPREADS			Rock spread	Debris spread	Earth spread
FLOWS			Rock flow (deep creep)	Debris flow	Earth flow
				(soil creep)	
COMPLEX			Combination of two or more principal types of movement		

Figure 1. Classification of slope movement. (From Varnes, 1978).

(e.g., slump-earth flow). Additional modifiers or special names can be used to describe rates of movement, wetness and special triggering factors, if known. For a more comprehensive discussion and for references to original sources the reader should consult Varnes' (1978) article.

For the purposes of this report the orderly tabulation of types of movements and source materials shown by Varnes (1978) (reproduced here as Plate 1) has been revised to present an array that retains Varnes' grouping according to dominant type of movement, but separates wet flows and dry flows, and arranges landslide terms in order of decreasing general importance of wet flow as a component of the mechanism of movement (Plate 2.). The array of Plate 2 is intended to illustrate the similarities and differences that are of greatest concern to the FEMA's National Flood Insurance Program (NFIP). The inclusion of common triggering events also assists in identifying the kinds of landslides most likely to involve the FEMA Disaster Assistance Programs. Most of the landslide classes listed in Plate 2 are described and illustrated by Varnes (1978). Plate 2 also includes field recognition criteria by which most of the factors critical to classification can be identified.

TRANSPORTING MECHANISMS AND RATES OF MOVEMENT

Varnes (1978) describes landslide mechanisms using the terms "Falls, Topples, Rotational Slides, Translational Slides, Lateral Spreads, Flows, and Complex." The following summary descriptions have been excerpted or abstracted from Varnes (1978), and the reader is referred to that paper for more detailed discussions together with references to usage of the terms by other workers:

Falls

Detached masses of any size move down steep slopes or cliffs with little or no shear displacement between the materials originally in contact, and descend mostly through the air by free fall, bouncing, or rolling.

Topples

The moving masses detach and pivot outward from the slopes about axes or points below the center of mass (commonly near their basal contacts with the stable parts of the slope), imparting overturning moments (out of the slope) to units of mass in motion, similar to the earliest stages of rolling.

Slides

The detached masses remain in constant sliding contact with the underlying stable slope materials, with shearing movement taking place along the surface of contact. In many slides, chiefly those that move slowly over few irregularities in the basal slip surface, relative movement within the moving mass, if any, takes place on a relatively few internal shear surfaces that divide the mass into a relatively few discrete subunits. Many other slides, however, are greatly disrupted and consist of many individual units. The principal resistance to mass movement in slides results from the frictional forces acting on the rupture surfaces.

Rotational Slides occur along surfaces of rupture that are curved concavely upward, imparting a backward (into the slope) tilt to the moving mass or its major subunits. Concave-upward rupture surfaces most commonly form within relatively homogeneous materials.

Translational Slides occur along rupture surfaces that are generally planar or gently undulatory. Planar rupture surfaces most commonly follow pre-existing discontinuities such as bedding planes, faults, joints, or the bedrock-regolith interface. The degree to which the moving mass is disrupted during displacement is commonly a function of the degree to which the rupture surface departs from the planar, and the distance the mass has moved.

Lateral Spreads

Flat to gently dipping translational displacement resulting in overall extension of detached, coherent masses, which move toward a free face on a basal shear surface or on subjacent material that has deformed by liquefaction or plastic flow. Spreading can also occur as a result of distributed shear in a mass of bedrock without evidence of a well-defined basal shear surface or zone of plastic flow.

Flows

Moving masses having internal differential movements that are distributed throughout the mass are termed "flows." In earth and debris masses the differential movements can be intergranular, and visible shear surfaces may not form; or differential movements can take place on closely spaced shear surfaces, some or all of which may be short-lived and heal without leaving clear evidence in the deposits. Movements in bedrock that may be described as a form of flow are extremely slow and distributed among many closely spaced fractures. Because wet flow and associations with flooding are important to FEMA's administration of the NFIP, flows are here described as "wet" or "dry." Wet flows are further subdivided depending on whether the moving masses consist of cohesive materials capable of retaining their water content during slow plastic deformation, or of non-cohesive materials from which water tends to drain relatively quickly, leaving a relatively stable deposit.

Flows (and flowlike movements) in Bedrock Flow movements in bedrock include deformations that are distributed among many large or small fractures, including microfractures, without the concentration of displacement along a through-going rupture surface. They may result in folding, bending, bulging, or other manifestations of plastic behavior, and the internal distribution of velocities may simulate that of viscous fluids.

Wet Flows (in earth and debris) In non-cohesive (relatively) flows, internal distribution of differential velocities varies systematically throughout the fluid mass. In cohesive (relatively) flows, the distribution of velocities resembles that for a plastic body in plug flow, and the moving masses are usually bounded by visible shear surfaces. Gradations between these extremes include plug flows bounded by zones of distributed velocities that lack visible boundary shears or internal slip surfaces.

In describing viscous fluids, the terms "Newtonian" and "non-N wtonian" are frequently used to describe differences in properties that affect flow rates.

Newtonian fluids are single-phase fluids having viscosities that are unique to a particular fluid composition at specified temperatures. Examples of Newtonian fluids are water, machine oil, honey, glycerine, and pitch. In Newtonian fluids, the flow rate varies inversely with viscosity and is a linear function of applied shear stress.

Non-Newtonian fluids include the polyphase fluids (such as mixtures of Newtonian fluids with solid particles or immiscible liquids) that behave as though their viscosities depend on the amount of admixed solid particles. Examples of non-Newtonian fluids are clay slurries, oil well drilling muds, ketchup, paint, mustard, mayonnaise, wet concrete, natural mud flows (including debris flows), and flowing lava. In non-Newtonian fluids, the flow rate of the material varies in a non-linear way with the shear stress, so that velocity and runout are difficult to predict.

Dry Flows Granular materials such as silt, sand, and fine gravel can flow downslope with internal distribution of velocities accommodated by grain to grain interaction, without the development of boundary or internal slip surfaces. Trapped or entrained air may provide an important fluidizing component, especially in the development of earthquake-triggered loess flows and in extremely rapid rock fragment flows initiated as rock falls.

Complex

Slope movements that exhibit more than one of the major modes of movement are termed "complex." Nearly all landslides are complex in some degree, and "complex" is only rarely used in naming the landslide class. Different parts of a landslide mass can move by flowing and sliding simultaneously, or a single mass may move by toppling, falling, sliding, and flowing in sequence. Landslide class names generally serve to identify the dominant mechanism and material, with or without the addition of descriptor terms. In instances of major changes during the movement of a mass, where two or more kinds of movement can be clearly identified, terms are commonly hyphenated. For example, an extremely rapid rock fragment (or debris) flow that begins as a rock fall from a high mountain cliff can be termed a "rock fall-debris flow" (or a "rock fall-debris avalanche"), or a rapid debris flow generated by a soil slip can be termed a "soil slip-debris flow."

Mechanisms and rates of movement are closely associated; however, most mechanisms operate over a range of rates that overlaps with the ranges of other mechanisms. Consequently, names that emphasize high or low rates of movement, such as "avalanche" or "creep," may not provide satisfactory information about the mechanism of movement. The rate scale used by Varnes (1978) includes:

Extremely Rapid	10 ft/sec (3 m/sec) or greater
Very Rapid	1 ft/min (0.3 m/min) to 10 ft/sec
Rapid	5 ft/day (1.5 m/day) to 1 ft/min
Moderate	5 ft/mo (1.5 m/mo) to 5 ft/day
Slow	5 ft/yr (1.5 m/yr) to 5 ft/mo
Very Slow	1 ft/5yr (0.06 m/yr) to 5 ft/yr
Extremely Slow	less than 1 ft/5 yr

The rate of movement of any slide or flow can, of course, change quickly. Slides that move at rates described as moderate to slow are commonly at or near equilibrium between gravitational forces tending to drive the mass downslope and the frictional forces that resist sliding and/or internal deformation. Movement at rates described as moderate to very rapid commonly reflects sufficient disequilibrium so that rapid acceleration and deceleration can occur. An extreme example is provided by rapid debris flows that develop from soil slips in colluvial soils on steep hillsides. Soil creep movement in colluvial soils on hillsides is generally extremely slow and may be undetectable. When saturated as a result of heavy rainfall or rapid snowmelt, thin slabs of the material can fail as soil slips, with a relatively slow initial rate of movement. If the moisture content of the moving mass equals or exceeds the liquid limit of the saturated material, remolding caused by the slide movement can result in a change-of-state from a solid, sliding slab to a fluid slurry that moves by flow. The change-of-state to a fluid is accompanied by a sudden, drastic reduction in the frictional resistance to movement, so that even though the gravitational driving forces remain constant, the mass accelerates downslope and can reach very rapid speeds.

LOCATION AND GEOMETRY

The geometric forms of underlying slip surfaces and the steepness of the slopes where landslides originate and over which they move provide important limits on the kinds of transporting mechanisms, distance and the rate of movement, and the degree of internal disruption of the materials in the moving masses. In addition, the runout distances for debris flows and mud flows are greater for thicker (or deeper) flows, and depths can be maintained over relatively gentle gradients if the flows are confined within channels.

Slabs and blocks of bedrock or cohesive soil, for example, can move farther with less internal disruption over planar slip surfaces than over surfaces that are concave upward, such as are characteristic of rotational slides. In general, geologic discontinuities such as bedding planes, joints, and faults, that may serve as preexisting surfaces of weakness along which sliding can occur, tend to be planar in form. Consequently, landslides with slip surfaces along such discontinuities tend to be blocky or slabby in form, and may originate in areas having slopes that range from very steep to very gentle, and the principal factor in determining the susceptibility to movement is the geometric relation between the orientation of the surfaces of weakness and that of the topographic slope.

For many kinds of landslides, hillslope steepness is a major factor in determining susceptibility to slope movement. Campbell (1980) presented generalized graphic representations of these relations, reproduced here as Figures 2, 3, and 4. (Note that because the diagrams were developed for an area where some kinds of landslides are not known to have occurred, the landslide classes represented are not comprehensive. Important landslide classes, such as quick clay flows, are missing because the diagrams apply only to a specific area that lacks quick clay flows.) Nearly all kinds of landslides are most common on slopes greater than 3:1 (18^o); fewer, but still significant kinds of landslides originate on slopes greater than 5:1 (11^o)and less than 3:1. Landslides that originate on slopes less than 5:1 generally require extremely weak materials or preexisting surfaces with low shear resistance; commonly, but not invariably, earthquake shaking or construction excavation serves to trigger their movement toward a free face.

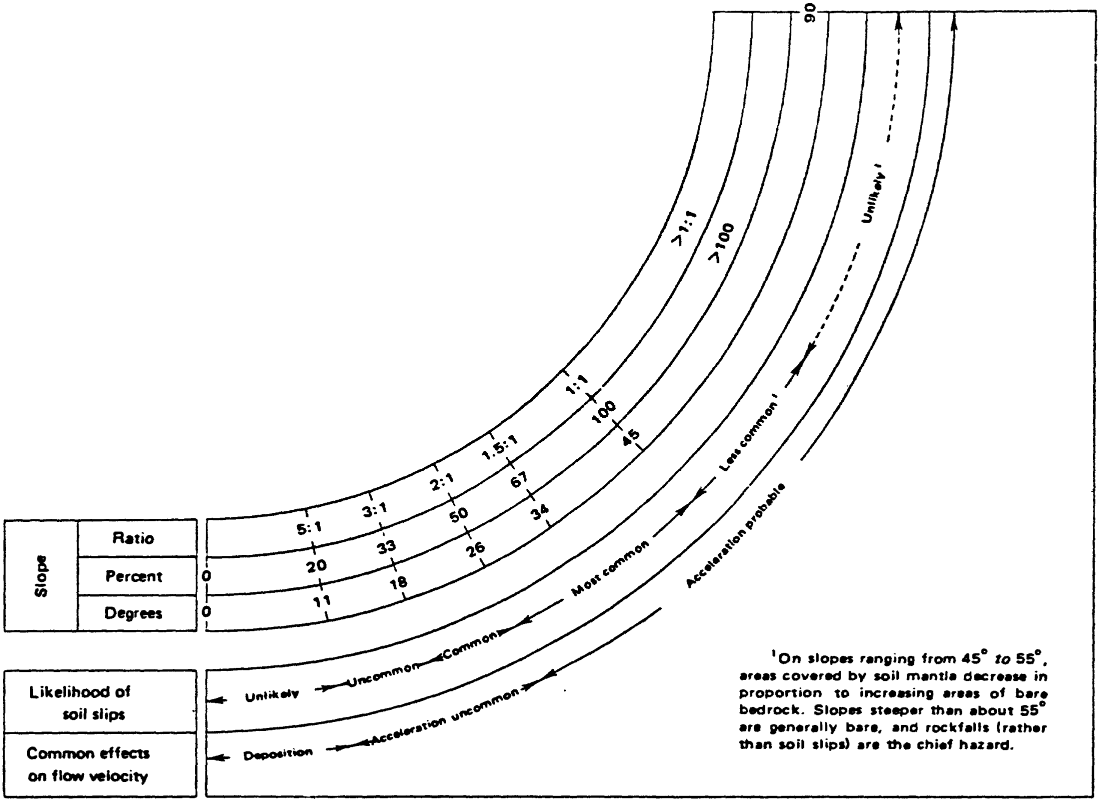
Rainstorm-triggered soil slip-debris flow failures commonly originate on slopes steeper than 3:1 and deposition generally begins on slopes gentler than 5:1. Moreover, runout over slopes less than 5:1 of debris flows generated on steeper slopes above generally does not exceed a few hundred feet, except where flow is confined to a channel (See Figure 2 notes.).

MATERIALS

Varnes adopted four terms to describe material involved in slope movements--"bedrock" and "engineering soils," with "debris" and "earth" as subsets of engineering soils:

1. Bedrock designates hard or firm rock that was intact and in its natural place before the initiation of movement.

2. Engineering soil includes any loose, unconsolidated or poorly cemented aggregate of solid particles, generally of natural mineral, rock, or



[1] On slopes ranging from 45° *to* 55°, areas covered by soil mantle decrease in proportion to increasing areas of bare bedrock. Slopes steeper than about 55° are generally bare, and rockfalls (rather than soil slips) are the chief hazard.

Figure 2. Relation of of soil slips and debris flows to slope. (From Campbell, 1975)

Note: According to an inventory by D. M. Morton (written communication, 1975) of soil slips that occurred during the heavy rainstorms of the 1969 winter season in the area adjacent to the Santa Clara River Valley, Ventura County, California, there were a total of 32,730 individual scars counted in an area of about 550 mi^2 (about 10.5 7-1/2 min, quadrangles). The slope interval for each was identified from slope maps that had been prepared by photomechanical methods from the topographic quadrangles. The results are tabulated below:

Slope Interval (percent)	No. of scars	Percent of total scars
0-20%	126	0.4%
20-33%	812	2.5%
33-50%	2,390	7.3%
50-67%	4,978	15.2%
67% +	24,424	74.6%

The figures above have not been adjusted for local irregularities in the slope intervals. Examinations of selected scars in the low slope intervals (0-20%, 20-30%) shows that many are headed on slopes that are locally steeper than the slope interval on the map.

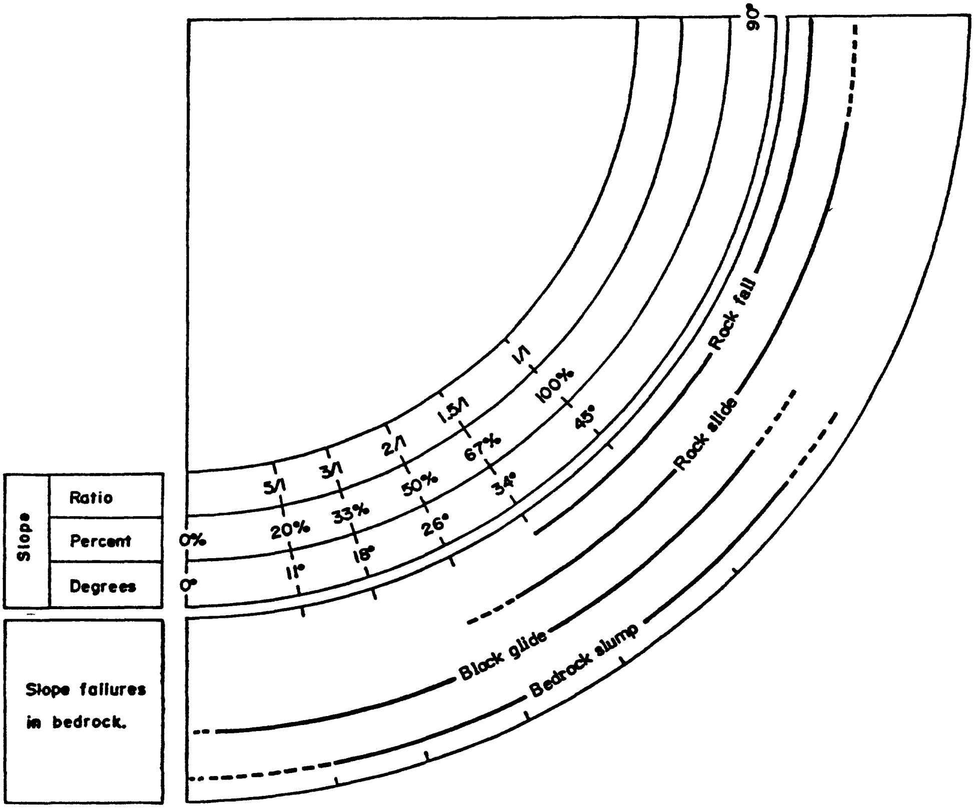


Figure 3. Relations of landslides in bedrock to slope. (From Campbell, 1980)

(Note that because this figure was developed for landslides in a small part of southern California, it is not comprehensive with respect to all landslide classes in the U.S.)

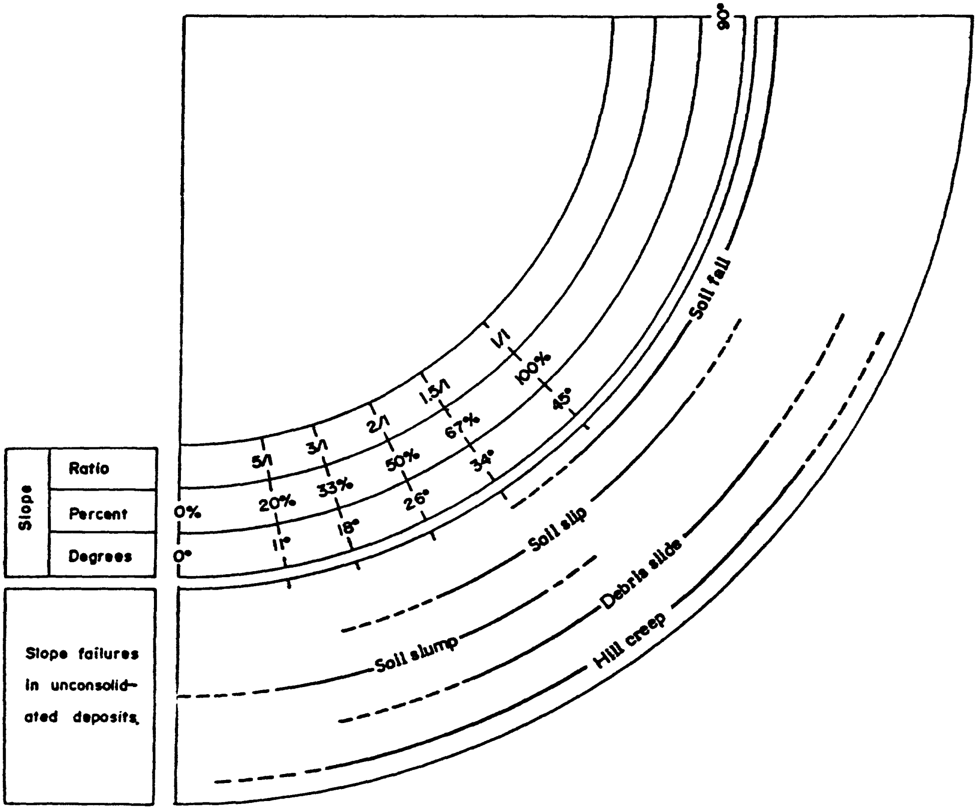


Figure 4. Relations of landslides in unconsolidated materials to slope. (From Campbell, 1980)

(Note that because this figure was developed for landslides in a small part of southern California, it is not comprehensive with respect to all landslide classes in the U.S.)

inorganic composition and either transported or residual, together with any interstitial gas or liquid. Natural soils may contain variable amounts of organic material.

a. Debris refers to an engineering soil, generally surficial, that contains a significant proportion of coarse material. "Debris" may be used to specify material in which 20 to 80 percent of the fragments are greater than 2 mm (0.08 in) in size, and the remaining fragments less than 2 mm. [1]

b. Earth connotes material in which about 80 percent or more of the fragments are smaller than 2 mm; it includes a range of rock and mineral fragments from nonplastic sand to highly plastic clay.

During landslide movement, slope materials are subjected to different degrees of internal disruption, therefore, parts of names that describe materials can change accordingly. Falling bedrock blocks can be broken by impact and bouncing into mixtures of rock fragments appropriately termed "debris." Engineering soils (including debris) can be disrupted by discrete slip surfaces into blocks, or they may become completely disaggregated into their constituent particles. Some special kinds of parent materials can be altered in place by dynamic shaking during an earthquake. For example, some layers of wet sand can undergo liquefaction and flow, causing "sand boils", and dry loess (wind-deposited silt and sand) with porous structure may collapse into dry flow.

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[1] In distinguishing debris flows from mud flows, Varnes (1978) restricts the term "mud flow" for rapidly moving, wet earth flow that contains at least 50 percent fragments smaller than 2 mm, whereas "debris flow" denotes a rapidly moving earth flow that contains more than 50 percent fragments coarser than 2 mm in size. It should be noted, however, that both "debris flow" and "mudflow" have been used in a broader sense (e.g., Sharp and Nobles, 1953, Middleton and Hampton, 1973, and Campbell, 1975) to refer to flowing slurries with more regard for their flow and depositional characteristics than to their proportional contents of rock fragments coarser than coarse sand size (2 mm).

MOISTURE

Varnes (1978) adopted the following descriptive terminology to describe the moisture content of landslide masses:

1.	Dry	Contains no visible moisture.
2.	Moist	Contains some water but no free water and may behave as a plastic solid but not as a liquid.
3.	Wet	Contains enough water to behave in part as a liquid, has water flowing from it, or supports significant bodies of standing water.
4.	Very Wet	Contains enough water to flow as a liquid under low gradients.

It is important to note that the foregoing terminology refers to the visible "wetness" of the moving landslide masses, rather than to the degree of saturation of pores with water or to the pore pressures in and beneath the mass.

SCAR AND DEPOSIT CHARACTERISTICS

The scars remaining on hillsides from which masses have detached and moved downslope can, in many instances, provide definitive evidence of the character of the parent material and the geometry of the initial slip surface (whether planar or curved, steep or gentle). The geometry of crown scars can also indicate whether break-away occurred on preexisting geologic structures, which commonly have linear traces and planar shapes, or on arcuate surfaces propagated through relatively homogeneous materials. For falls, examination of fresh scars on cliffs and bluffs can provide a basis for estimating initial volumes. For flows, clues as to moisture content and rates of movement may be found by examination of drainage channels through which they have passed for erosional and depositional features.

The form and geometry of deposits provide evidence from which interpretations of mechanism can be made. Non-cohesive flows leave trains of debris in confined channels at low gradients, and leave lobes, fans, and sheets of debris or mud on unconfined gentle slopes; the deposits tend to drain rapidly and to remain stable except when overridden by subsequent debris flows. Cohesive flows commonly have steep lobate toes with transverse ridges, radial cracks, and longitudinal ridges. Solifluction and soil creep tend to leave no scars (or very inconspicuous ones), and small, multiple lobes are characteristic of solifluction. Gulls, ridge-crest trenches, and less distinct open cracks are commonly associated with cambering, sackung and the slow toppling deformation of near-surface bedrock structures. Blocks rotated into the slope are diagnostic of rotational slumps, whether in bedrock or in engineering soils. Blocks rotated out of the slope are identified as topples.

The internal fabric of the deposits generally provides diagnostic evidence by which mud flood origins and mud flow origins can be distinguished; however, the depositional landforms and the geologic settings of the deposits can also be of assistance in recognizing the mechanism of emplacement. Matrix-supported conglomerates in bodies with lobate fronts or as steep "alluvial" fans at the mouths of small, steep drainage basins indicate a debris flow origin; internal bedding lamination, graded bedding, and a relatively flat "floodplain" with an incised channel indicate that a flood or a mud flood is the origin. Of course, mud flows and mud floods can both occur in mountain canyons and some "alluvial" fans include both mud flow and mud flood deposits; where this can occur, a careful field examination can generally yield diagnostic evidence. Where soil slip-debris flow events have occurred recently enough so that the scars remain visible, the likelihood that debris deposits downslope or downstream were emplaced by debris flow mechanisms is high. However, the rapid obliteration of scars by new vegetation and the regeneration of colluvial slope soils causes the absence of scars to be of virtually no value as evidence that debris flow activity has not occurred. The principal evidence for past debris flows is provided by the debris flow deposits, most of which remain recognizable for thousands of years, even if dissected and partly removed by major stream erosion or human construction activity.

The nomenclature of the principal features of landslide scars and deposits is illustrated for a slump-earth flow in Figure 2.1t on Plate 1. The names are adaptable to nearly all kinds of landslides, although terms like "main body", "head", and "toe" become much more obscure when applied to flows that have moved completely out of the vicinity of their crown areas. The distances between crown scars and displaced masses (from a few feet for small rotational slumps to a few miles for some large debris flows) provide indications of the rates of movement and the sizes of the areas having a potential for hazard from recurrences of the events.

DAMAGING FORCES AND EFFECTS IN FLOODS, MUD FLOWS, AND OTHER LANDSLIDES

A standard nomenclature needs to be developed and adopted to describe the effects of various events on structures. Several terms are suggested here, some of which apply to effects of both floods and landslides.

Inundation (that is, immersion of an object in a fluid) is a principal cause of damage from floods, and debris/mud flows. Damage is caused by physical and chemical changes in materials that are ordinarily dry and protected from wetting--from below by foundations, and from above by roof structures.

Drag, the lateral force acting on objects in moving streams of viscous fluids, can cause damage to structures surrounded by flowing water, mud, or debris by displacing them laterally from their foundations.

Buoyancy, the lifting force that tends to cause structures to float off their foundations, is another effect that is shared by floods and mud flows.

Burial or partial burial by the deposition of sediment occurs from both floods and mud flows, but is also an effect of other landslides.

Lateral pressure is built up when mud or debris pushes against one side of a structure, whether from mud or debris flows, or at the toes of slides. Rapidly flowing, turbulent floods can also develop lateral hydraulic forces.

Lateral impact by a fast-moving debris flow poses a destructive capability from the momentum of relatively small masses. Lateral impact from boulders, trees, and other objects carried by floods is accompanied by the hydraulic effects of deep, fast-flowing water. Lateral impact can also result from other fast-moving landslides such as debris slides and rock falls.

Impact from above requires steep slopes (or gradients). Rock and debris falls and topples are the most common sources of these impacts; however, debris flows in some mountain areas have become air-borne by flow over cliffs, and have impacted structures from above.

Foundation failure refers to the adverse effects on structures from tilting, shearing (both laterally and vertically), and pulling apart of the earth materials on which structures are founded, where a structure is founded astride the boundary of a landslide, or astride the boundaries of individual blocks that move differentially within a landslide.

COMMON TRIGGERS

Landslides can be triggered during readily identified exogenous events such as earthquakes, heavy rainstorms or rapid snowmelt, volcanic eruptions, and other landslides. However, the details of the triggering mechanisms are sometimes not clearly defined by simple association with such events. Earthquake shaking imparts cyclical changes in loading on slope materials and can also cause liquefaction of susceptible materials. Heavy rainstorms and rapid snowmelt tend to raise pore fluid pressures at shallow depths, as does rapid melting of snow and ice during volcanic eruptions. These dramatic events may be associated with other adverse effects, both direct and indirect. Rainstorms may be associated with floods as well as landslides. Earthquake shaking can directly affect structures, cause liquefaction in low-lying areas, and trigger landslides on slopes; if the slope materials are saturated, some masses may move as flows. Landslides also result from the cumulative effects of less obvious events such as wedging by freeze-thaw or shrink-swell cycles, root wedging, animal burrows, tectonic uplift, undercut/surcharge by natural erosion and deposition, undercut/surcharge from construction activity, and thaw of ice-bearing soils (permafrost). Rising ground water level is an important variable triggering factor that may be ephemeral over a short term during a single rainstorm, or moderate to long-termed, from the cumulative rainfall of several storms or a season. The toes of large, slow moving landslides may become oversteepened and fail as small slumps and slides which, if sufficiently wet, can change to flows.

MUD FLOW, MUDFLOW, MUD SLIDE, AND MUDSLIDE

The definitions that presently guide administration of the National Flood Insurance Program, as published in Title 44, Code of Federal Regulations (1983, p. 167-173), include:

"Flood or Flooding means:

(a) A general and temporary condition of partial or complete inundation of normally dry land areas from:

(1) The overflow of inland or tidal waters.
(2) The unusual and rapid accumulation or runoff of surface waters from any source.
(3) Mudslides (i.e., mudflows) which are proximately caused or precipitated by accumulations of water on or under the ground.

(b) The collapse or subsidence of land along the shore of a lake or other body of water as a result of erosion or undermining caused by waves or currents... or by an unanticipated force of nature, such as a flash flood or an abnormal tidal surge, or by some similarly unusual and unforseeable event which results in flooding...

"Mudslide (i.e., mudflow) describes a condition where there is a river, flow or inundation of liquid mud down a hillside usually as a result of a dual condition of loss of brush cover, and the subsequent accumulation of water on or under the ground preceded by a period of unusually heavy or sustained rain. A mudslide (i.e., mudflow) may occur as a distinct phenomenon while a landslide is in progress, and will be recognized as such... only if the mudflow, and not the landslide, is the proximate cause of damage that occurs."

Ambiguities, inaccuracies, and inconsistencies attend attempts to reconcile the foregoing definitions with established technical classification systems and descriptive terminology.

Mud Flow is the term used by Varnes (1978) to denote a rapidly moving, wet earth flow in which at least 50 percent of the earth material is in grains smaller than 2 mm. In the usage of the NRC Committee on Methodologies for Predicting Mudflow Areas (1982), this distinction was not rigidly maintained, and the term "mud flow" was used to refer to flows having sufficient viscosity (by virtue of sufficient wet mud matrix) to support large boulders, regardless of the relative proportion of matrix mud to larger debris particles.

Mudflow is a term that has been used in a variety of ways. Mudflow has been used as a generic term to refer to both debris flows and mud flows, regardless of the percentage of coarse material contained (e.g., Sharp and Nobles, 1953). In the Soviet and Chinese literature, mudflow is a term used to include heavily laden mud floods, mud flows, and debris flows. In other technical literature (e.g., Hutchinson, 1970) mudflow has been used to describe features subsequently ascribed to mudslides by Hutchinson and Bhandari (1971), and which would be termed mud slides or slow earth flows in the usage of Varnes (1978).

Mud Slide is a two-word term, following the naming system of Varnes (1978), accepting the technical usage to features like those described by Hutchinson and Bhandari (1971) as "mudslides". Usage should be restricted to material consisting of mud that moves by sliding on basal and lateral shear surfaces. The moving mass is internally deformed by plastic flow which may take place on internal shear surfaces (a more common term in the U. S. is "earth flow"; see Plate 1).

> <u>Mudslide</u>, the specific term used in the Housing and Urban Development Act of 1969, P.L. 91-152, Sec. 409 (a), which amended the National Flood Insurance Act of 1968 (42 U.S. Code, 1983, rec 4001 (f)), is a popular term, commonly used by the news media to refer to a wide scope of events, ranging from debris-laden mud floods to many kinds of wet rotational and translational slides, and even including some wet rock fall events. Priority usage of "mud slide" in technical literature probably goes to Fleming (1878) and Cailleux and Tricart (1950), according to Hutchinson and Bhandari (1971), who applied the term as one word, "mudslide", to carefully described features that would be named "slow earth flows" in the terminology of Varnes (1978).

The report of the Committee on Methodologies for Predicting Mudflow Areas (National Research Council, 1982) identified the kinds of events that should be termed "mud flows" and distinguished them as transitional from "mud floods", at the wet end, of a gradational continuum, and "other landslides", at the drier end. Figure 5, reproduced from that report, illustrates the distinctions. The Committee described those distinctions as follows:

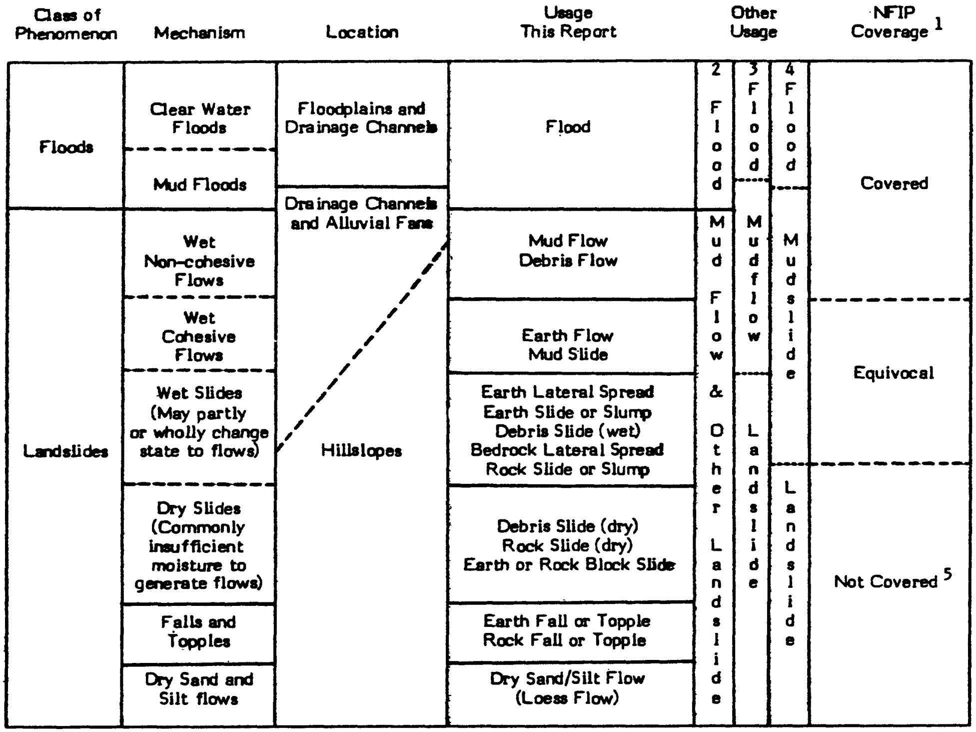
> "The key characteristic in differentiating between mud floods and mud flows is that a mud flow possesses a combination of density and strength that will support inclusions of higher density than water, such as boulders, both during transport and when the mass comes to rest. The ability to support an inclusion during transport stems from a velocity-dependent strength (the matrix viscosity) and a velocity-independent strength (the shearing resistance of the mass). When the flow comes to rest, the velocity-dependent strength goes to zero. However, ...(if a) high density inclusion does not sink in the mass (at rest,) ... it is supported by ... static shearing resistance. A mud flood ... does not have static shearing resistance. Transport of inclusions is possible only because of the strength provided by the motion of the fluid. When the fluid velocity goes to zero, the strength also goes to zero and the supporting capability of the mass is lost. All inclusions of higher density than the fluid (water, in most cases) then sink at Stokes Law rates that depend on the viscosity of the fluid."

<table>
<tr><th>Class of Phenomenon</th><th>Event</th><th colspan="2">Location</th><th>Covered by NFIP</th><th>Status of Hazard Mapping</th></tr>
<tr><td rowspan="2">FLOODS</td><td>Clear Water Floods</td><td rowspan="2">Floodplains</td><td rowspan="2">Drainage Channels and Alluvial Fans</td><td rowspan="3">Yes</td><td>Now Mapped</td></tr>
<tr><td>Mud Floods</td><td rowspan="3">Not Now Mapped</td></tr>
<tr><td rowspan="2">LANDSLIDES</td><td>Mud Flows</td><td colspan="2" rowspan="2">Hillslopes</td></tr>
<tr><td>Other Landslides</td><td>No</td></tr>
</table>

Figure 5. Flood and landslide phenomena: the locations in which they occur and their status under the National Flood Insurance Program (NFIP). (From National Research Council, 1982).

The continuum of natural processes represented by "clear water floods, mud floods, mud flows, and other landslides" on Figure 5 can be further subdivided. Figure 6 is an adaptation of the format of Figure 5 to attempt to further subdivide the various phenomena collectively grouped as "mud flows and other landslides". The right-hand column of Figure 6, titled "NFIP Coverage", reflects our interpretation of how events described by technical terminology derived from the Varnes (1978) classification (as listed in the column headed "Usage, This Report") would correlate with the present language of NFIP insurance policies and regulations regarding coverage for "mudslides, i.e., mudflows".

The array shown in Figure 6 illustrates that debris flows and mud flows, in the technical terminology adopted for this report, correlate with the mud flow part of "mud flows and Other Landslides" in the usage of the 1982 NRC report. It also indicates that dry sand and silt flows, dry earth falls and topples, dry rock falls and topples, dry rock block slides, dry rock slides, and dry debris slides, correlate with the dry part of the "mud flows and other landslides" group identified by the 1982 NRC report. Wet cohesive flows such as earth flows and mud slides fall into an "equivocal" category, because they



Class of Phenomenon	Mechanism	Location	Usage This Report	Other Usage[2]	Other Usage[3]	Other Usage[4]	NFIP Coverage[1]
Floods	Clear Water Floods	Floodplains and Drainage Channels	Flood	Flood	Flood	Flood	Covered
	Mud Floods	Drainage Channels and Alluvial Fans			Mudflow	Mudslide	
Landslides	Wet Non-cohesive Flows		Mud Flow Debris Flow	Mud Flow & Other Landslide			
	Wet Cohesive Flows	Hillslopes	Earth Flow Mud Slide				Equivocal
	Wet Slides (May partly or wholly change state to flows)		Earth Lateral Spread Earth Slide or Slump Debris Slide (wet) Bedrock Lateral Spread Rock Slide or Slump		Landslide		
	Dry Slides (Commonly insufficient moisture to generate flows)		Debris Slide (dry) Rock Slide (dry) Earth or Rock Block Slide			Landslide	Not Covered[5]
	Falls and Topples		Earth Fall or Topple Rock Fall or Topple				
	Dry Sand and Silt flows		Dry Sand/Silt Flow (Loess Flow)				

[1] This column shows as:
Covered - the kinds of events that seem to meet all the regulatory requirements to be called "floods" or "mudslides (i.e. mudflows)" within the scope of section 59.1, 44 Code of Federal Regulations (1983);
Equivocal - the kinds of events over which disputes have arisen about whether or not they are "mudslides (i.e., mudflows)"; and
Not Covered - the kinds of events that do not seem likely to be considered "mudslides (i.e., mudflows)" under current interpretations of section 59.1, 44 Code of Federal Regulations (1983)

[2] Usage in Figure 5; from National Research Council, 1982.

[3] Approximate usage in Russian, Chinese, and eastern European landslide literature.

[4] Approximate usage in popular media.

[5] Although dry slides, falls, topples, and flows may not be included as "mudslides", there are circumstances where they provide the mechanisms by which stream banks collapse or subside when eroded or undermined by flooding streams.

Figure 6. Flood and landslide phenomena: mud flows and other landslides.

commonly show evidence of movement by sliding as well as by internal wet flow. Some wet slides are also considered equivocal because they commonly give rise to wet flows, at least in part. Indeed, most of the "other landslides" that generally lack sufficient moisture to initiate flows may, at times, contain sufficient moisture and contribute to the development of flow failures.

Although Figure 6 shows dry slides, dry falls, dry topples, and dry flows to be excluded from coverage because they are not, themselves, "mudslides", it should be noted that many, relatively dry, kinds of landslides can be initiated in association with flooding. Landslides are common forms of erosion along the banks of streams, rivers, lakes, and reservoirs. Banks and shoreline materials can be undercut by flood currents and waves, leading to their collapse or subsidence by landslide processes. High water levels in flooding streams, lakes, and reservoirs cause ground water levels to rise in adjacent earth materials, reducing their internal resistance to failure by landslide processes. Although many relatively dry landslides triggered in this way do not appear to meet the criteria of Figure 6 for coverage as "mudslides", some could occur in circumstances where questions about NFIP coverage under other parts of the definition for "flood or flooding" (44 Code of Federal Regulations, sec. 59.1, p. 169) might be raised for further consideration.

DISASTROUS AND NON-CATASTROPHIC LANDSLIDES

Landslides are events that occur throughout the range between sudden and catastrophic at one extreme and slow and insidious at the other. Landslide disasters are commonly associated with exogenous triggering event such as a major earthquake, rainstorm, or rapid snowmelt. The triggering event may cause additional damage from earthquake shaking or flooding, and landslide effects may not be readily separable from such other effects without intensive study. The identification of areas in which there is is some potential for landslide disaster can be achieved by delineating areas susceptible to landslide (including debris flow and mud flow) hazards during non-disaster conditions, and combining that evaluation with information on the relative potential for levels of seismic shaking or rainfall intensity and duration that are capable of triggering landslides.

The following list of some 20th century landslide disasters in the U.S. is not comprehensive, but can serve as a general representation:

- 1906--Numerous "flow-slides" and other landslides were triggered in the San Francisco Bay Region by the great earthquake, which also caused liquefaction in the wet lowlands of the area (Youd and Hoose, 1978).

- 1853(?), 1874, 1936, 1958--Giant waves, as much as 1,720 feet high in the 1958 event, generated in Lituya Bay, southeastern Alaska by large landslides, probably triggered by earthquakes (Miller, 1960).

- 1964--The great earthquake in Alaska triggered major damaging landslides over a vast area of south-central Alaska. Submarine landslides at Seward, Valdez, and Whittier generated large local waves that added to the destruction already caused by the shaking and landslides (Eckel, 1970).

- 1969--Most of the 150 people killed in central Virginia during the Hurricane Camille "flood" died from blunt force injuries rather than by drowning. Debris supplied by the numerous debris flow avalanches caused by the heavy rain, is believed to have been the instrument of injury (Williams and Guy, 1973).

- 1969--"Mudslides" (in this instance determined to be soil slip-debris flows) in southern California killed 20 people in the course of two intense rainstorms. Campbell (1975) notes that all 20 fatalities were caused by a total of 10 soil slip-debris flow events that originated in chaparral-, sagebrush-, and grass-covered slopes. Even though mud floods and mud flows were also generated in recently burned parts of the San Gabriel Mountains, resulting in great damage on the alluvial fans along the south flank of the range, much damage and all of the fatal injuries resulted from flows that originated on unburned slopes having natural or landscaped vegetation cover.

- 1971--Damaging landslides triggered by the San Fernando earthquake, southern California, caused major losses at the San Fernando

Reservoirs, Juvenile Hall, Olive View, and Camp Holton localities, as well as hundreds of debris slides on the slopes of the San Gabriel Mountains (Yerkes, 1973).

- 1972--Three coal refuse impoundments at Buffalo Creek, West Virginia, failed during heavy rains, forming a debris flow or mud flood that traveled 24 km downstream, killing 125 people and leaving 4,000 homeless (Davies and others, 1973).

- 1978--Two major rainstorms triggered destructive debris flows and also caused extensive flooding in southern California. Storm-triggered landslide damage in the City of Los Angeles, alone, was estimated at $50 million (Slosson and Krohn, 1982, p. 291).

- 1980--A series of six major rainstorms hit southern California and Arizona in a period of 8 1/2 days, causing major flooding and landsliding. Landslide damage in the City of Los Angeles was estimated at $70 million (Slosson and Krohn, 1982, p. 291).

- 1980--A minor earthquake triggered a massive rock slide-debris avalanche on the north side of Mt. St. Helens, Washington, uncorking the pressurized magma chamber which erupted explosively. The debris avalanche progressed about 22 km westward, burying the North Fork Toutle River valley under about 60 km^2 of debris. Mud flows (debris flows) generated during the eruption extended down the Toutle and Cowlitz Rivers to and into the Columbia River. In addition to debris flow damage along its course, the channel-filling debris increases the danger of later flooding on the Cowlitz (Cummans, 1981).

- 1982--Major damage (totals estimated at about $300 million), 19 confirmed deaths, and five more missing and presumed dead in the San Francisco Bay area. The dead and missing were the result of 11 debris (flow) avalanches and one major block slide (with associated peripheral debris flows). Small but damaging landslides occurred in large numbers scattered over a large area (see Brown, 1984, for summary).

- 1982-83--Major damage from landslides (estimates in the hundreds of $ millions) occurred in central and northern California from exceptional total rainfall of the winter rainy season, and in the Washoe Lake area, Nevada, along the Wasatch Front and in the Wasatch Range, Utah, from rainfall and rapid thaw of exceptional snowpack. A general rise in ground water levels in many landslide areas of the western States is associated with a major global weather perturbation (Utah Geological and Mineral Survey, 1983).

Although landslide "disasters" are plentiful enough, the annual cumulative losses which are dominated by non-catastrophic landslide events are staggering. Schuster (1978), using data from various sources, concluded that direct and indirect losses to public and private property in the United States exceeded $1 billion per year. Fleming and Taylor (1980) acquired and compared damage figures for a number of U.S. communities and list the following average annual losses for selected areas:

$5,900,000	San Francisco Bay region (nine counties)
4,000,000	Allegheny County, Pennsylvania
5,170,000	Hamilton County, Ohio

In southern California, notable landslide events occurred in the following winter rainy seasons years. Estimated damages are from Slosson and Krohn (1982).

$ 7,500,000	1951-52
7,500,000	1961-62
6,000,000	1968-69
50,000,000	1977-78

CLASSIFICATION FACTORS AND FEMA PROGRAMS

The technical classification described in this report can be applied to evaluating methods for nationwide hazards identification for the purposes of the Disaster Assistance, Hazards Mitigation, and National Flood Insurance Programs, either alone or in combination.

The mudslide provisions of the NFIP seem concerned primarily with flow as a mechanism, with wet debris or mud as the materials, and with inundation, buoyancy, drag, lateral impact, lateral pressure, partial or complete burial, and (possibly) erosional undercutting as the kinds of damage-producing forces and effects. Association with times of flooding appears important where both floods and "mudslides" are caused by the same triggering events, such as heavy rainstorms. In addition to damage by "mudslide i.e., mudflow" events themselves, some relatively dry landslides, including earth or rock falls, topples, and slides, that occur because of erosion or undermining of slopes by flooding streams might also be of concern to the NFIP within the context of the definitions of "flood" and "flooding" in 44 Code of Federal Regulations, (1983).

Disaster Assistance requires damage of sufficient social and economic importance to be classified as disastrous. This tends to focus attention on large events that occur with such rapidity that people are killed or injured and much property damaged. Landslide events of this kind occur mostly in combination with triggering events such as severe rainstorms, earthquakes, and volcanic eruptions. Generally, steep slopes and mountainous terrain are required to provide the gravitational drive for acceleration of falls, slides and flows to high speeds. Debris flow disasters such as that of January 1982 in the San Francisco Bay region consist of a large number of destructive but relatively small events in hilly and mountainous terrain. Earthquake-triggered lateral spread and quick clay flows, however, can move very rapidly on low slopes, as evidenced by the Turnagain Heights landslide in Anchorage, triggered by the 1964 Alaska earthquake.

Usually, disaster declarations have been made in circumstances where the landslide activity is associated with flooding, earthquake shaking, or volcanic eruption. Because landslides may continue to cause damage long after the triggering events have ceased to be active, it is difficult to specify the end of the "disaster" by direct reference to the landslide activity. However, if the landslides are triggered by other events, such as those listed above, temporal associations with the triggering events could be used for assigning times for termination of the disaster period.

Hazards mitigation planning and engineering design for landslides mitigation requires knowledge of the dominant type of landslide in any given region, and of the kinds of events most likely to trigger movement. Although mitigation activities are generally specific for specific sites, land-use planning and preliminary engineering design are greatly aided by the identification and delineation of areas of potential hazard in terms of type and relative susceptibility.

The nationwide delineation of relative susceptibility to mud flows and other landslides would be of unquestioned usefulness in the implementation of the FEMA programs noted above. Clearly, there are many options for different levels of effort that might be appropriate to the information needs of such a program. Moreover, the costs and benefits of acquiring and applying different levels of information must be considered. The cost of the identification and delineation, nationwide, of the relative susceptibility of different areas to hazards from mud flows and other landslides is directly dependent on: a) the detail, comprehensiveness, and reliability with which the risks are identified, and b) the accuracy with which the distribution of risk is depicted.

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HAZARDOUS VOLCANIC EVENTS
AND
ASSESSMENTS OF LONG-TERM VOLCANIC HAZARDS

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INTRODUCTION

This section describes the types of hazards posed by various volcanic events and the ways in which these hazards can be assessed by determining a volcano's record of past eruptive activity. This discussion deals primarily with assessment of long-term hazards, or those which have a low frequency of occurrence.

The majority of the more than 1300 volcanoes that have been active in the last 10,000 years (Simkin and others, 1981) lie on or near convergent or divergent boundaries of lithospheric plates; others lie in the interiors of plates in areas of crustal extension. Of this total, more than 500 have been active in historic time; about 25 percent of these have had eruption-related fatalities. Almost 50 percent of all the reported fatalities have occurred in three countries: Indonesia, The Philippines, and Japan. Compared with other hazards, such as floods, landslides, and earthquakes, the mean annual losses of life and property due to volcanic eruptions are small. Nevertheless, there is great potential for catastrophic losses of life and property from volcanic eruptions in specific areas, and the risk is increasing as populations grow and developments expand in volcanic terranes. A critical need in many volcanic areas is for assessments of the long-term hazards posed by the many volcanoes that have been active in the last 10,000 years, but not necessarily in historic time. Most of these are viewed as extinct volcanoes by local populations and authorities; however, many are likely to erupt in the future. In addition, the historic eruptions of some active volcanoes may not adequately characterize the frequency and (or) types of possible activity and therefore studies of long-term hazards should also be carried out at these volcanoes.

Much of the material presented here is taken or modified from "Sourcebook for Volcanic-Hazards Zonation" by D. R. Crandell, B. Booth, K. Kusumadinata, D. Shimoruzu, G. P. L. Walker, and D. Westercamp, which will be published soon by UNESCO.

U.S. GEOLOGICAL SURVEY VOLCANO-HAZARDS PROGRAM

The Volcano-Hazards Program (VHP) of the U.S. Geological Survey is composed of four main elements (Bailey and others, 1983).

1. Volcanic-Hazards Assessment--Prepare assessments of potential geologic and hydrologic hazards for volcanoes and volcanic areas in the United States. These are based largely on studies of the past activity at each center.

2. Volcano Monitoring--Develop and deploy systems to monitor eruption precursors at potentially hazardous volcanoes, and to monitor eruptions.

3. Fundamental Research--Study volcanic processes and products; the generation, transport, and eruption of magma; the hydrology of volcanic areas; and develop new monitoring techniques.

4. Emergency-Response Planning and Public Education--Provide information about volcanic hazards to the public and to Federal, State, and local emergency-response agencies, who are responsible for taking the lead in this element.

The Geologic and Water Resources Divisions are engaged in hazard assessments, monitoring, and related research; the National Mapping Division provides cartographic and geographic data and products for activities of the Geologic and Water Resources Divisions and other organizations.

Funding levels (in millions of dollars) for the VHP for the last 5 yr reflect the 1980 eruption of Mount St. Helens:

1979	1980	1981	1984
1.1	4.4	12.7	7.4

The locations and number of professional staff involved in volcano-hazards work are: Cascade Volcano Observatory (about 40); Hawaiian Volcano Observatory (about 15); and Reston, Denver, Menlo Park, Flagstaff, and Anchorage offices (about 25).

Figure 1 shows the volcanoes and volcanic areas in the U.S. for which hazard studies are completed, in progress, or planned.

TYPES OF HAZARDOUS VOLCANIC EVENTS

Volcanic events both directly and indirectly affect peoples' lives and health, their activities, and their property. Direct effects include interaction with the products of the eruption themselves, such as tephra falls, whereas indirect effects include such phenomena as ground deformation and tsunamis. This section discusses the character of various volcanic events, the hazards posed by them, some examples, and references to published sources, in English, of additional information. Most of these are available in the U.S. Geological Survey library.

Tephra Falls

Tephra includes fragments of all sizes of rock and lava that are ejected into the atmosphere by the force of an eruption and by hot ascending gases, and accumulate as airfall tephra deposits upon falling back to the Earth's surface. Historic eruptions that have been accompanied by widespread tephra falls (Plinian) have had main eruptive phases lasting from 1 hour to one day or longer (table 1). Single events may be separated by long repose intervals or occur as multiple events closely spaced in time over a period of months or years (e.g., Paricutin, 1946-1952; Foshag and Gonzales, 1956). Airfall tephra

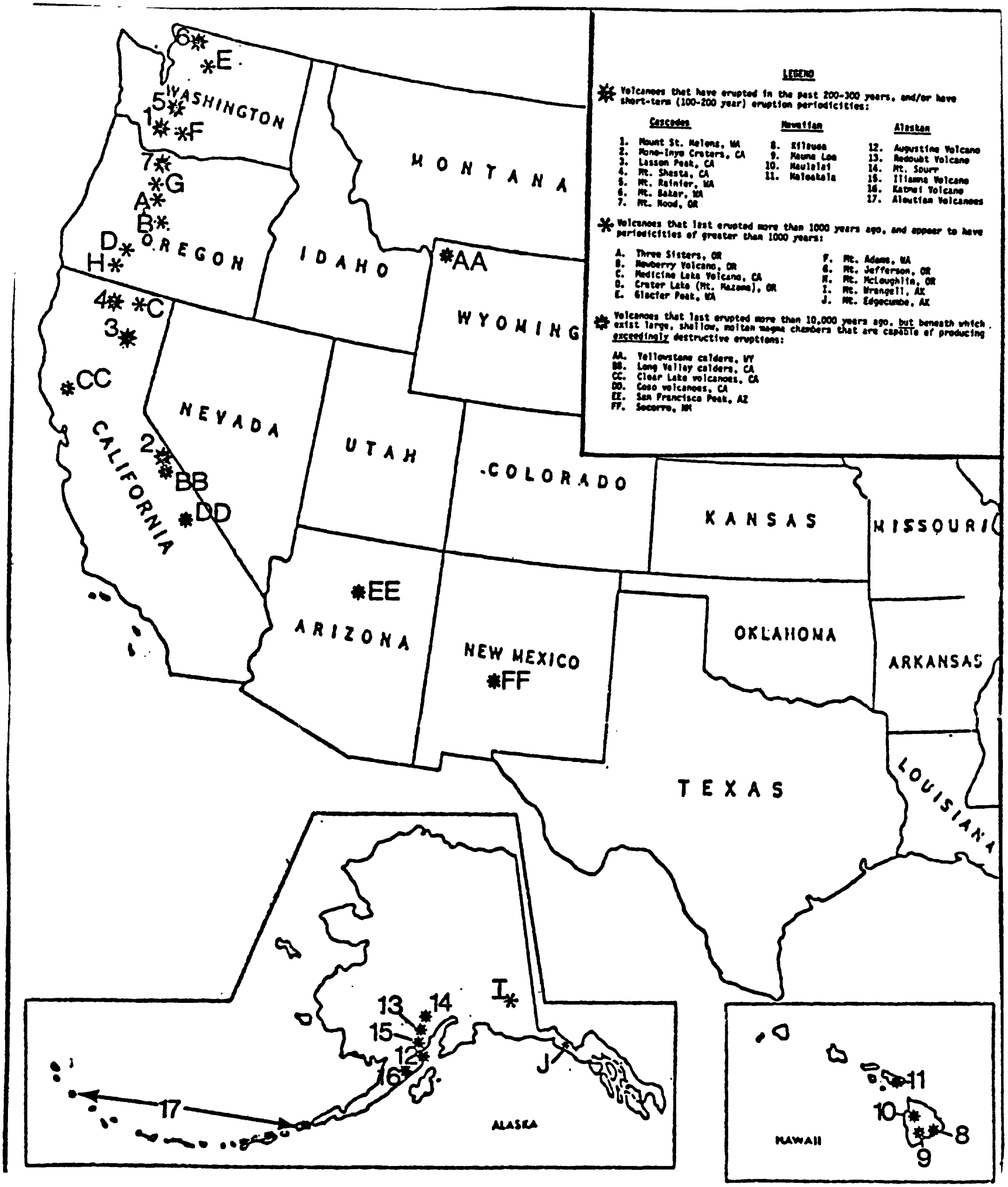

Figure 1. Location of potentially hazardous volcanoes in the United States.

deposits consist of variable proportions of low-density pumice and scoria, dense lithic fragments, and crystals. These materials may be juvenile (formed from the magma involved in the eruption) or accidental (derived from older rocks). Sizes of the fragments (Fisher, 1961) range from ash (2 mm), to lapilli (2-64 mm), to blocks and bombs (64 mm) that may be several meters in diameter.

Table 1. Duration of Main Eruptive Phase of Some Historic Plinian Eruptions (Modified from Blong (1982).

Volcano	Year	Duration	Source
Krakatau	1883	24-30 hr	Self and Rampino (1981)
Katmai-Novarupta	1912	<20 hr	Curtis (1968)
Santa Maria	1902	18hr	Rittman (1962)
Coseguina	1835	3 da	Rittman (1962)
Tambora	1812	2 da	Blong (1982)
Vesuvius	79	24 hr	Lirer and others (1973)
Fogo	1563	2 da	*In* Lirer and others (1973)
Asama	1783	4 da	*In* Lirer and others (1973)
Hekla	1970	2-3 hr	Thorarinsson and Sigvaldson (1972)
	1947	1 hr	Thorarinsson (1954; 1967)
	1846	4 hr	Blong (1982)
	1768	5-6 hr	Blong (1982)
	1693	<1 hr	
Askja	1875	8.5 hr	*In* Lirer and others (1973)
Mount St. Helens	1980	9 hr	Christiansen and Peterson (1981)

Tephra fall poses the widest-ranging direct hazard from volcanic eruptions (Wilcox, 1959). For instance, areas of 10^3 to 10^4 km^2 may be covered with >10 cm of tephra during large Plinian eruptions (Figure 2; Walker, 1981). The areal distribution of the tephra is influenced by many variables. Among the most important are the volume and height of the eruption column, the speed and direction of the wind, the grain-size of the tephra, the variation of these during the eruption, and the duration of the eruption. Tephra deposits are generally lobate in form and thin progressively downwind from the volcano (Figure 3); however in some historic eruptions, isolated areas of increased thickness are present in distal portions of the lobe (Quizapu, 1932-Larsson, 1937; 5 eruptions of Mount St. Helens in 1980-Sarna-Wojcicki and others, 1981). In general the effects of a tephra fall are more severe where the deposits are thick and coarse and decrease in severity as the deposit thins and becomes finer grained.

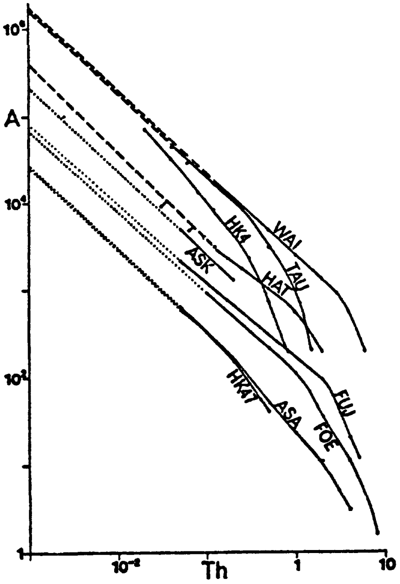

Figure 2. <<Area>>plot, showing area A enclosed by isopachs in km^2 versus thickness Th in metres for several pumice deposits the isopachs of which are well constained at the low-thickness end. Dashed lines - extrapolations required to produce the same volume within the 1 μm isopach as that determined by the crystal concentration method. Dotted lines - extrapolations parallel with the Taupo slope. Pumice deposits in this and subsequent figures identified by the abbreviations in Table 1. (Walker 1981)

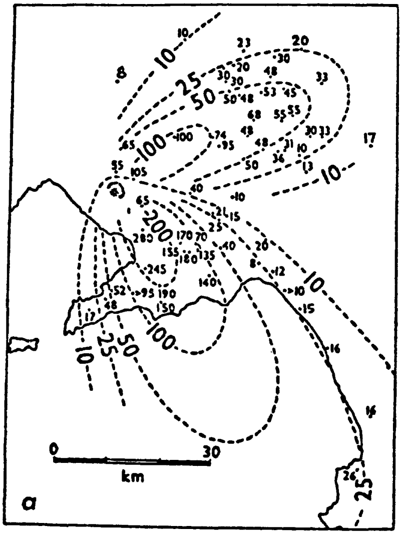

Figure 3. Isopach maps showing the thickness, in centimeters, of the Pompei and Avellino Pumice. a. White and gray components combined. b. Lower white component only. c. Upper gray component only. (Liren and others 1973)

Tephra fall endangers life and property by (1) the force of impact of falling fragments, (2) forming a layer covering the ground, (3) producing a suspension of fine-grained particles in air and water, and (4) carrying toxic and noxious gasses, acids and salts, and, close to the volcano, heat. During historic eruptions, mean horizontal velocities of tephra clouds ranged from <10-100 km/hr (see compilation in Blong, 1982), so that advance notice to downwind areas may provide up to several hours of warning of impending tephra fall.

The hazard from the impact of large fragments is greatest close to the vent and decreases with increasing distance. Walker's (1980) plots of maximum pumic and lithic clast sizes with distance from vents for a range of sizes of Plinian eruptions show that small bombs of pumice were carried horizontal distances as much as 20-80 km, whereas lithic bombs were carried up to 10-30 km (Figure 4). People can survive falls of small bombs with minimal shelter; however, falls of larger bombs can harm people even in substantial shelters. Large ballistic and airfall bombs generally fall within 10 km of vents. In the 1938 explosions of Asama, 1-m-diameter bombs were thrown as far as 4.5 km (Minakami, 1942). Blong (1981) presents the limited data available on the effects of bomb impacts on some common building materials.

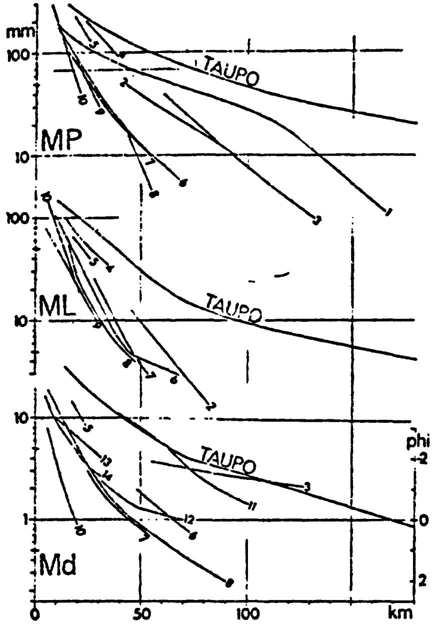

Figure 4. "Distance" plots of the Taupo and other selected plinian pumice deposits. 1 = Ontake (Kobayashi et al., 1967); 2 = Shikotsu (Katsui, 1959); 3 = Askja 1875 (G.P.L. Walker, unpublished data); 4 = Tenerife J (Booth and Walker, 1980); 5 = Granadilla (Booth, 1973); 6= Pompei (Lirer et al., 1972); 7 = La Primavera B (Clough et al., 1980); 8 = Toluca upper (Bloomfield and Sanchez Rubio, 1977); 9 = Fogo 1563 (Walker and Croasdale, 1971); 10 = Fogo A (Walker and Crosdale, 1971); 11 = Crater Laker (Fisher, 1964); 12 = Hekla 1947 (Thorarinsson, 1954); 13 = Tenerife L (Booth and Walker, 1980).

Burial by tephra can collapse roofs of buildings, break power and communication lines, and damage or kill vegetation. Dry, uncompacted tephra has densities ranging from 0.2-1.2 g/cm^3; addition of water can increase densities by as much as 150 percent for low-density tephra and 50 percent for higher-density tephra. Moisture also increases the cohesiveness of the tephra. The effects of burial on structures vary greatly with their design and construction. Flat roofs are more prone to failure than those that are steeply pitched. The effects of burial on vegetation are also highly variable. In general falls of 1 m will kill most types; even thin falls can be highly damaging depending on species, time of year, and stage of growth (Rees, 1979; Blong, 1982).

The suspension of fine particles in air affects visibility and peoples' health, especially those with respiratory problems, and can damage unprotected machinery, especially internal combustion engines. Air, rail, and highway traffic are especially vulnerable; the effects of the relatively thin tephra fall from the 1980 eruption of Mount St. Helens lasted for days to almost 2 weeks (Figure 5; Warrick, 1981). Wind can resuspend fine tephra and therefore many of these problems can last long past the original eruption. Even in thin

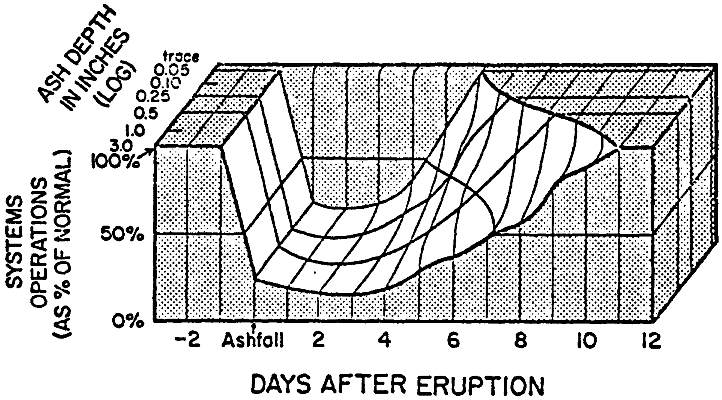

Figure 5. Generalized disruption surface from volcanic ashfall. Mount St. Helens 1980 (Warrick, 1981).

Table 2. Tephra thicknesses and duration of accompanying darkness (Blong, 1982).

Volcano and year	Distance from volcano (km)	Duration of darkness (hr)*	Thickness of uncompacted tephra (cm)
Tambora, 1815	65	29-30	9.5
	460?	22	5.0
	380	19*	3.2
	400	25*	20.0
	400-500	72*	5.0+
	600	712	5.0
Taal, 1911	20	0.5	0.1
Soufriere, 1920	20	4.0?	2.5
Coseguina, 1835	80	28*+48	6.5
	40	5*+1.5	7.5
Askja, 1875	60	7*+1.5	15.2
Bezymianny, 1955	45	3.5	2.0
	80	8.5	2.0
Vesuvius, 1906	5-6	48*	25-30
Mayon, 1871	10-12	2-3	0.4
Katmai, 1912	160	11*	12.5-15
		26*	25.0
St. Helens, 1980	135	12	1.5

*= total darkness

gas is water vapor; other important gases include carbon dioxide, carbon monoxide, sulfur dioxide, sulfur trioxide, hydrogen sulfide, chlorine, and fluorine (Williams and McBirney, 1979). These gases are transported away from the vent as acid aerosols, as compounds adsorbed on tephra, and as microscopic salt particles (Rose, 1977; Thorarinsson, 1979).

The sulfur compounds and chlorine and fluorine react with water to form acids that are poisonous and, even in low concentrations, are damaging to eyes, skin, and respiratory systems of animals. These acids can also damage or kill vegetation, depending on their concentration and on type of vegetation; they can also damage metals and fabrics. (See summaries in Wilcox, 1959; Bolt and others, 1977; Thorarinsson, 1979.)

Carbon monoxide and carbon dioxide are odorless and unlike the noxious gases can't be detected by people. Carbon monoxide is poisonous and carbon dioxide, which is denser than air, can collect in low-lying areas and asphyxiate any animals that enter the area. In some instances, carbon dioxide can be emitted from large areas away from active vents, which makes detecting hazardous areas very difficult (Thorarinsson, 1979).

The effects of a volcanic gas are directly related to its concentration, which generally decreases downwind from the vent. Although harmful effects are usually restricted to within 10 km of vents, in exceptional circumstances they can occur tens or even thousands of kilometers away. Climatic changes induced by very large eruptions that release a large volume of ash and acid aerosols into the statosphere can, of course, have worldwide effects (Rampino and Self, 1982).

Pyroclastic Flows

Pyroclastic flows are masses of hot (300-> 800^{o} C), dry pyroclastic material and hot gases that move rapidly along the ground surface. Pyroclastic flows consist of two parts: 1) a basal flow that moves along the ground and is the pyroclastic flow proper and 2) an overriding turbulent ash cloud that includes both hot pyroclastic surges (ash-cloud surges), which are discussed in the next section, and towering columns of ash that are derived from the basal flow and surges (Figure 6; Anderson and Flett, 1903; Perret, 1935; van Bemmelen, 1949; Taylor, 1954; Smith, 1960; Davies and others, 1978; Fisher and Heiken, 1982).

Basal flows are dense mixtures of ash and gases with variable proportions of lapilli, blocks, and bombs. Their mode of flow is mostly laminar; their

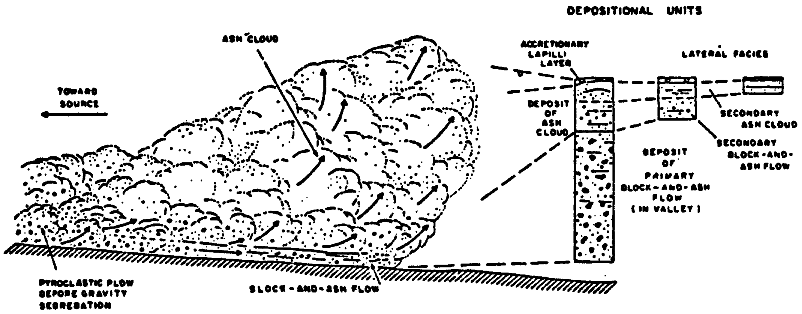

Figure 6. Pyroclastic flow and depositional units at Pelee. Initial turbulent flow separates into block-and-ash flow at base with overriding turbulent flow. Depositional units and facies as shown. See text for further explanation. Not to scale. (Fisher and Heiken, 1982).

flowage is sustained by several processes. These include movement under the influence of gravity along topographic depressions, the exsolution of gas from juvenile material (Fenner, 1923), and the incorporation of air by the flow that is then heated and expands and leads to fluidization of the fine-grained particles (Perret, 1935; McTaggart, 1960; Sparks, 1976, 1978; Wilson, 1980). Rates of flow vary depending on the origin, volume, and composition of the flow but range from 50-150 km/hr for small flows and may exceed several hundred km/hr for large flows.

Pyroclastic flows originate in several ways: (1) by the gravitational collapse of vertical eruption columns (Hay, 1959; Sparks and others, 1978), (2) by upwelling of frothing magma from circular or linear vents; (3) by laterally directed explosions at the base of a dome (Perret, 1935), and (4) by collapse of portions of volcanic domes.

Pyroclastic flows are common at many andesitic and dacitic composite volcanoes and at silicic calderas throughout the world. They range greatly in length, from 1 to at least 200 km; in area, from <1 to $>20{,}000$ km^2; and in volume, from <0.001 to >1000 km^3. In general, pyroclastic flows formed by explosions or collapses of domes, which tend to have a large content of lithic material, lie in the lower parts of these ranges. These small pyroclastic flows tend to follow topographic depressions. In contrast, very large pyroclastic flows are formed by collapse of large eruption columns or by upwelling from vents and are generally pumiceous and rich in ash. These may be of great mobility and cover broad areas with sheet-like deposits around volcanoes and surmount topographic barriers to heights of several hundred meters (Yokoyama, 1974; Miller and Smith, 1977; Barberi and others, 1978). Some pyroclastic flows of very great velocity and mobility may cover very wide areas, but leave a very thin deposit and therefore may be overlooked in the geologic record (Ui, 1973; Walker, 1980; Walker and others, 1981).

Because of their mass, high temperature, high velocity, and potentially great mobility, pyroclastic flows and accompanying pyroclastic surges (see next section) pose grave hazards from asphyxiation, burial, incineration, and impact. The hazards of small pyroclastic flows can be dealt with by avoiding development in areas of high hazard and (or) by evacuating these areas when eruptions are imminent. Very large pyroclastic flows would constitute a national disaster in that the affected areas would be so large that evacuation would be difficult, if not impossible, and there would be catastrophic loss of property and resources.

Pyroclastic Surges

Pyroclastic surges (Sparks, 1976) are turbulent, low-concentration, density currents of gases, rock debris, and, in some cases, water, that move above the ground surface at high velocities. They deposit thin, generally fine-grained, bedded sequences in contrast to relatively thicker, massive, and poorly sorted pyroclastic flows. Fisher (1979) mentions three genetic types of pyroclastic surges. (1) Ground surges (Sparks and Walker, 1973) are generated from collapsing eruption columns or directly from open-vent or dome explosions and may precede a pyroclastic flow. These are hot, although some may carry a component of cold material. (2) Ash-cloud surges (Fisher, 1979) are derived from moving pyroclastic flows; they are also hot. (3) Base surges (Moore and others, 1965; Moore, 1967; Nairn, 1979) originate from phreatic or phreatomagmatic explosions, contain a large amount of water and (or) steam, and are generally colder than the boiling point of water.

Pyroclastic surges of types (1) and (2) are formed by explosive magmatic eruptions and are generally found in association with pyroclastic flows (Figure 7), but because of their greater mobility they may affect areas adjacent to and beyond pyroclastic flows (Taylor, 1958; Moore and Melson, 1969) and even on flanks of the volcano unaffected by pyroclastic flows. Pyroclastic flows controlled by topography may be deflected, but the overriding surge may continue in the initial direction (Fisher and others, 1980). In turn, highly energetic surge clouds, once detached from basal flows, can undergo partial gravity segregation to form secondary pyroclastic flows (Fisher and Heiken, 1982; Fisher, 1983). During some eruptions, pyroclastic surges may form without accompanying pyroclastic flows.

The hot types of pyroclastic surges pose a variety of hazards including asphyxiation, incineration, presence of noxious gases, destruction by high-velocity clouds, impact by rock fragments, and burial by pyroclastic-surge deposits. Their great mobility makes escape impossible once they have formed, so that evacuation of zones of potential pyroclastic-surge inundation should begin at the start of an eruption that may be accompanied by hot pyroclastic surges. Many great historic volcanic catastrophies have been caused by hot pryoclastic surges including Vesuvius, 79, $>$ 2000 people killed (Sheridan and others, 1981); Mt. Pelee, 1902, 30,000 people killed (Lacroix, 1904); and Mt. Lamington, 1951, 3000 people killed (Taylor, 1958).

Pyroclastic surges of type (3), base surges, generally occur where magma interacts with abundant surface and (or) ground water. Therefore eruptions in marine, lake, flood-plain, and other wet environments may generate base surges. Base surges are cold (generally $< 100^{o}$ C) and therefore typically stop within 10 km from their source. Hazardous aspects of base surges include destruction of vegetation and structures by the force of the rapidly moving turbulent cloud, impact by rock fragments, and burial. Their rapid velocity is too great to allow escape, so that evacuation from likely surge zones when eruptions are imminent and (or) avoidance of development in hazardous areas are the only available mitigation procedures.

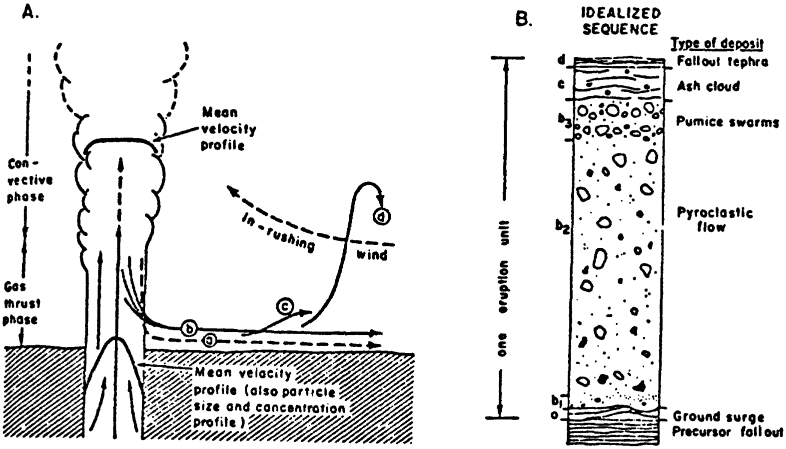

Figure 7. Eruption column collapse related to idealized eruption unit. Letters a–d in Figure 7A correspond to those in Figure 7B. A. Outer part of column collapses to form a pyroclastic surge (a); followed by (b) progressive collapse of interior parts to give voluminous, high-concentration pyroclastic flow. Pyroclastic material segregates from surface of flow (dilute phase) to form an ash cloud (c). Finest-grained material continues to be elutriated into atmosphere from dilute phase flow (d). It falls back on flow deposits or is swept back to join main eruption column by inward rushing atmospheric winds. B. Idealized depositional sequence of one eruption unit showing ground surge deposit (a), fine-grained basal layer of pyroclastic flow developed by flowage processes (b_1), main body of the pyroclastic flow representing the main bulk of the collapsed eruption column (b_2), a zone of pumice swarms segregated at the top of the ash flow (b_3), an ash cloud deposit elutriated from the top of the moving pyroclastic flow (c) and a thin fallout deposit (d). (Fisher 1979).

Laterally Directed Blasts

Laterally directed blasts are large explosions that have an important low-angle component and are driven by magmatic gases and(or) a high-pressure hydrothermal system. These events apparently require only modest volatile contents in the magma (Eichelberger and Hayes, 1982). The explosion may be initated by volatile pressures that exceed lithostatic pressures, by sudden reduction of lithostatic pressure by landslide, or by other means. That a laterally directed blast is a complex process is shown by the resulting deposits that have evidence of pyroclastic flow, surge, and fall processes and perhaps others not yet recognized (Hoblitt and others, 1981). An important aspect of laterally directed blasts is that the blast moves at speeds that greatly exceed those due to simple gravitational acceleration; at Mount St. Helens, 1980, the blast cloud had initial velocities on the order of 600 km/hr that slowed to about 100 km/hr near its margin 25 km from the volcano (Moore and Sisson, 1981). Laterally directed blasts contain mixtures of cold and hot debris; the hot debris may be hot enough to scorch vegatation and start fires. Survivors of the Mount St. Helens blast reported that the blast was cold for a few seconds followed by a period of several minutes when it was hot (Figure 8; Rosenbaum and Waitt, 1981).

Laterally directed blasts have affected areas ranging from a few to hundreds of km^2. The largest events known are the 1956 blast at Bezymianny that devastated 500 km^2 (Gorshkov, 1959, 1963) and the 1980 blast at Mount St. Helens that devastated 600 km^2 (Moore and Sisson, 1981). Within the areas affected by laterally directed blasts, virtually all life can be expected to be killed and all structures destroyed by the force of the blast cloud, impact by debris, abrasion, burial, and heat.

Lava Flows and Lava Domes

The rate of effusion and the viscosity of lava and the slope of the surface onto which it erupts control the morphology of extrusions (Williams and McBirney, 1979). Fluid lavas form long, thin flows on slopes and flat-topped lava lakes in depressions, whereas viscous lavas will form short, stubby flows on slopes and steep-sided domes in depressions. Fluid lavas such

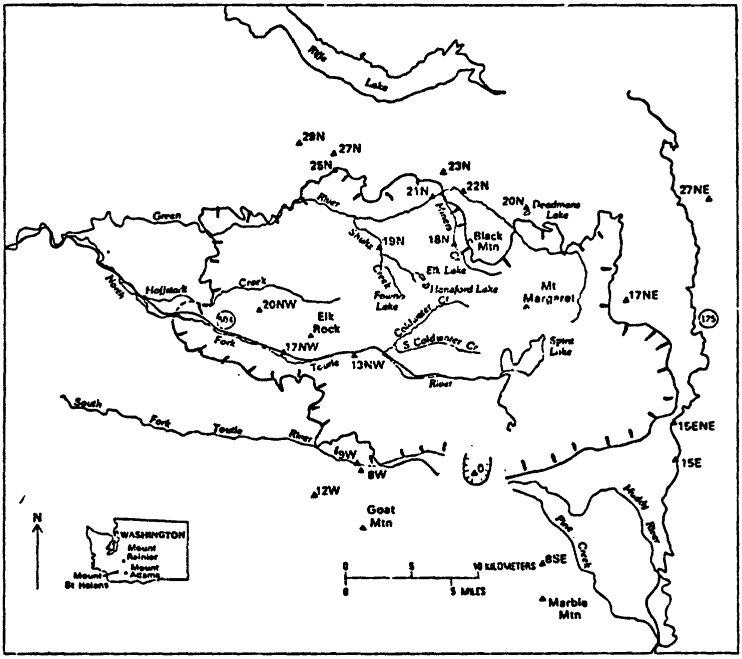

Figure 8. Map showing location of eyewitnesses at time of the May 18 eruption (triangles). Shaded region, area of devastation in which the forest was destroyed by the directed blast. Hachured area, post-May 18 crater rim. (Rosenbaum and Waitt 1981).

as basalt form flows that are typically several tens to as much as 100 km long; flows of some historic fissure eruptions have covered areas of many to hundreds of square kilometers (Thorarinsson, 1969; Fedotov and Markhinin, 1983). Flows of viscous lava such as dacite seldom exceed 10 km and are generally only a few kilometers long. In many cases, dacite and rhyolite lavas are so viscous that they pile up over vents as steep-sided domes. Rates of movement of lava flows vary considerably from tens to hundreds of m/hr for silicic lavas to several km/hr for basaltic lavas. Because of their slow rates of movement, lava flows seldom threaten human life. However, in exceptional circumstances rates may be much higher as in the case of the rapid draining of very fluid lava from a lava lake on Mt. Nyiragongo, Zaire, in which as many as 300 people were killed by thin lava flows advancing at rates of 30-100 km/hr (Tazieff, 1977).

The major hazard of lava flows is damage or total destruction by burying, crushing, or burning everything in their paths. In addition, fires started by lava flows or floods and lahars generated by rapid melting of snow and ice can affect areas far beyond the borders of the lava flow. Because lava flows follow valleys, disruption of surface drainage can result in damming and backflooding of stream valleys.

Lava domes seldom cover large areas, but as mentioned in preceding sections they pose grave pyroclastic-flow and surge hazards.

During several recent eruptions, authorities have attempted to protect property and resources by diverting lava flows (see discussions in MacDonald (1972) and in Bolt and others (1977)). Methods used include building of artificial barriers, bombing or blasting levees and channels in order to spread lava higher up the flow path rather than allowing it to reach the flow front, and spraying water on the flow to cool the lava and cause the flow to thicken and therefore reduce its possible extent. Although these attempts have met with variable success, it seems likely that in certain cases some control of lava flows, especially small ones, is possible.

At present, the best mitigation methods appear to be evacuation of areas as lava flows approach and by keeping expensive or critical developments out of areas likely to be affected in the future.

Lahars and Floods

Lahars are rapidly flowing mixtures of rock debris mobilized by water that originate on the slopes of a volcano (Crandell, 1971). The term is also used for the deposits that result from this process. The physical properties of lahars are controlled by grain size and water content (Fisher, 1971), but in general include high-yield strength, high-bulk densities (as great as 2000 kg/ m^3), and high apparent viscosities (Neall, 1976). Velocities of historic lahars have varied with physical properties and topography. Lahars at Mount St. Helens in May 1980 had velocities as low as 5 km/hr on low-gradient valley bottoms and as great as 140 km/hr on and near the volcano (Janda and others, 1981). Mean velocities of many other historic lahars have been on the order of 40-60 km/hr (MacDonald, 1972).

Areas of inundation and lahar length are greatly influenced by topography (Figure 9). For a given volume, lahars channeled in narrow valleys will be thicker and extend to greater distances than those that spread out over broad valley bottoms or fans. Lahars with high velocities can surmount topographic

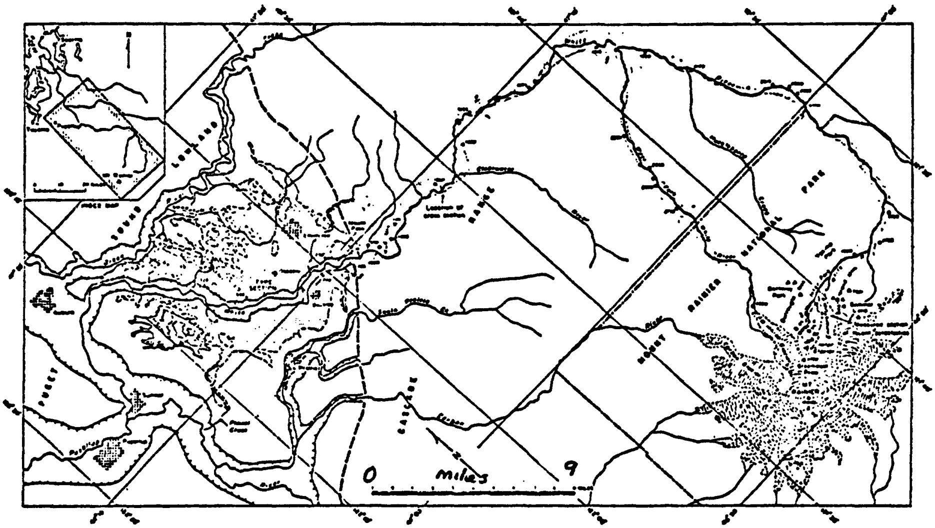

Figure 9. Extent of the Osceola mudflow from Mt. Rainier. (Crandell and Waldron 1956).

barriers and rise up on the outsides of bends. Large lahars may extend for hundreds of kilometers and cover areas of hundreds of square kilometers and thus constitute a wide-ranging volcano hazard.

Lahars can be generated in many ways that result in the mixing of rock debris and water on a volcano. (Crandell, 1971; MacDonald, 1972; Neall, 1976). Among them:

1. Release of water from a crater lake either by volcanic explosions or by collapse of a crater wall.

2. Mixing of pyroclastic flows with water.

3. Rapid melting of snow and ice on the slope of a volcano by hot rock debris or hot gases.

4. Brecciation of a lava flow on snow, ice, or very wet ground on the slope of a volcano.

5. Avalanching of water-saturated debris on a volcano, where the source of water might be atmospheric, hydrothermal, or from some other source.

6. Torrential rainfall on unconsolidated fragmental deposits on the slope of a volcano.

7. Collapse of dams formed by lava flows, lahars, mass movements, etc.

8. Incorporation of debris in floods caused by rain or by melting of glaciers or snow.

Some lahars are generated directly by volcanic activity, while others are unrelated to volcanic activity and can be triggered by storm, earthquake, or mass-movement events. So monitoring of volcanic activity itself may not be capable of detecting the onset of conditions that may lead to generation of lahars.

Lahars threaten lives and property both on volcanoes and in the valleys draining from them. Because of their high-bulk density and velocity they can destroy vegetation and even substantial structures in their paths, such as bridges. The deposits of lahars can deeply bury crops and developments and fill stream channels, thus decreasing their capacity to carry flood flows (Janda and others, 1981; Lombard and others, 1981). In addition, increased sedimentation in streams can affect capacity and navigation of channels far downstream from lahars (Schuster, 1983). Some lahars are hot and may contain fragments that are hot enough to start fires.

Dikes, diversions, and sedimentation basins may help to control lahars, especially those of small volume. Likewise, reservoirs built for flood control or power generation can also be used to trap lahars if the water level is lowered prior to an eruption (Crandell and Mullineaux, 1978).

In the absence of effective control measures, mitigation measures for lahar hazards are evacuation if an eruption is imminent and the avoidance of siting developments in high-hazard areas.

The hazards associated with floods in volcanic areas are similar to those elsewhere. Floods that originate on volcanoes may become lahars as they entrain debris. Also, floods may continue downstream from lahars and debris avalanches.

Phreatic explosions

Phreatic or steam-blast explosions occur when the vapor pressure of ground water exceeds the lithostatic load of overlying rocks and sediments. They have occurred without warning (Iwo Jima in 1957; Corwin and Foster, 1959) and with seismic and other presursors. The initial explosion can throw out particles of varied sizes on ballistic trajectories, which may be followed by up to many minutes of gas streaming that can transport rock debris upward. The finer-fraction can then be carried downwind and deposited as airfall tephra. In addition, phreatic explosions can release harmful gases and can cause lahars.

The materials ejected during phreatic explosions are generally nonmagmatic, but phreatic eruptions can grade into those that involve partly or entirely magma.

Phreatic explosions in volcanic areas can be caused by water in a volcano being heated by hot rock (as at Kilauea in 1924; MacDonald, 1972) or by rising magma. Those caused by the heat of rising magma may precede magmatic eruptions by weeks or months (as at Mt. Pelee in 1902 and at Mount St. Helens in 1980; Christiansen and Peterson, 1981) or occur immediately before (as at Hekla in 1947; Einarsson, 1951). In other cases, phreatic activity dies out and no magmatic activity occurs (as at Siranesan in 1932; Tsuya, 1933; and at Soufriere de Guadeloupe in 1975-1976). Both phreatic and magmatic explosions can occur simultaneously along a zone of shallow magmatic intrusion that has differing ground-water conditions (Nairn, 1979).

Phreatic explosions of steam, water, and mud also occur in hot spring areas (White, 1955; Lloyd and Keam, 1974).

Indirect Volcanic Hazards

In addition to the direct effects of volcanic eruptions discussed above, other events triggered by eruptions or even events on volcanoes that occur during periods of repose can be hazardous and are grouped here as indirect volcanic hazards.

Volcanic earthquakes generally precede and accompany eruptions by periods of days to many years. This activity typically consists of swarms of small earthquakes, rather than major shocks, so that damage from earthquakes may be minimal. However, in some circumstances the intensity has been severe (as during the Fayal eruption of 1957-1958; Machado and others, 1962). Currently, earthquake swarms near the town of Pozzuoli, Italy, which lies in the Vesuvius area, are causing considerable damage, especially to old masonry structures.

Tsunamis are long-period waves that are caused by the sudden displacement of water, travel at high speeds, and can build to great heights as they approach shores. Most are caused by tectonic earthquakes; however, tens of historic tsunamis of volcanic origin have caused many deaths and much destruction of property even at great distances from eruptions. Latter (1981) discusses many causes of tsunamis of volcanic origin including volcanic earthquakes; explosions; collapse or subsidence; landslides, lahars, or pyroclastic flows entering bodies of water; and shock or air waves accompanying large explosions.

Structural collapse in volcanic areas includes gradual and catastrophic subsidence and landslides. Subsidence can be caused by withdrawal of magma by eruptions or subsurface drainage; affected areas can range from small pit craters to large calderas of hundreds of square kilometers. Landslides on volcanoes can range from small slips to large, catastrophic debris avalanches on the flanks of composite volcanoes (Schuster and Crandell, in press; Siebert, in press) to huge landslides on island volcanoes.

Large mass movements probably constitute the greatest hazard related to structural collapse. Large debris avalanches have occurred at several composite volcanoes in historic time and have been triggered by phreatic explosions (Bandai Volcano, Japan, 1888; Sekiya and Kikuchi, 1889), by magmatic eruptions (Bezymianny and Shiveluch Volcanoes, Kamchatka, in 1956 and 1964, respectively; Gorshkov, 1959; Gorshkov and Dubik, 1970), and by earthquakes and deformation associated with magmatic intrusions (Mount St. Helens in 1980; Lipman and others, 1981; Voight and others, 1981). Large slope failures may also occur independently of these triggering mechanisms where stability is gradually lost through hydrothermal alteration, erosion, and changing pore-water conditions (Hyde and Crandell, 1978).

Debris avalanches from volcanoes are among the largest mass movements known; the recently recognized middle Quaternary debris avalanche from ancestral Mount Shasta covers an area of 450 km^2 and with an estimated volume of 26 km^3 is the largest known Quaternary landslide on Earth (Crandell and others, 1984). Historic volcanic debris avalanches have covered areas of tens of square kilometers, and have extended several tens of kilometers from sources.

Debris avalanches bury and destroy everything in their paths and greatly alter preexisting topography. Lahars and floods may originate from debris avalanches and extend many kilometers down valleys; tsunamis can also be generated by large debris avalanches entering a lake or the sea.

On island volcanoes, gravitational collapse of limited sectors to very large parts of edifices can occur where slopes are oversteepened by erosion or faulting and where hyaloclastite or other pyroclastic materials are present, especially if they are altered to clay.

On Tenerife, sector collapses have displaced areas as large as 100 km^2 through vertical distances of hundreds of meters (Figure 10; Booth, 1979). In the Lesser Antilles, single prehistoric landslides have affected areas of tens of square kilometers on the western sides of some islands, which have steep submarine slopes toward the Grenada Basin (Figure 11; Roobol and others, 1983).

On Kilauea, Hawaii, the sudden displacement of large blocks toward the sea has generated large, damaging earthquakes and tsunamis (Tilling and others, 1976). At Piton de la Fournaise on La Reunion, and Kilauea, forceful injection of magma into rift zones is causing seaward displacement of entire unbuttressed flanks of large shield volcanoes at the rate of meters per century (Figure 12; Swanson and others, 1976; Duffield and others, 1982).

ASSESSMENT OF LONG-TERM VOLCANIC HAZARDS

Much of the following discussion is modified from Crandell and others, in press.

Long-term, or potential volcanic hazards result from events that have such a low frequency of occurrence that people are unlikely to experience them in their lifetimes. A problem for the volcanologist is determining which volcanoes constitute a long-term hazard, which of these should have the highest priority for study, and which present a great enough risk to warrant some type of monitoring. An issue that requires attention by authorities and the public is the desirability of land-use restrictions, especially around

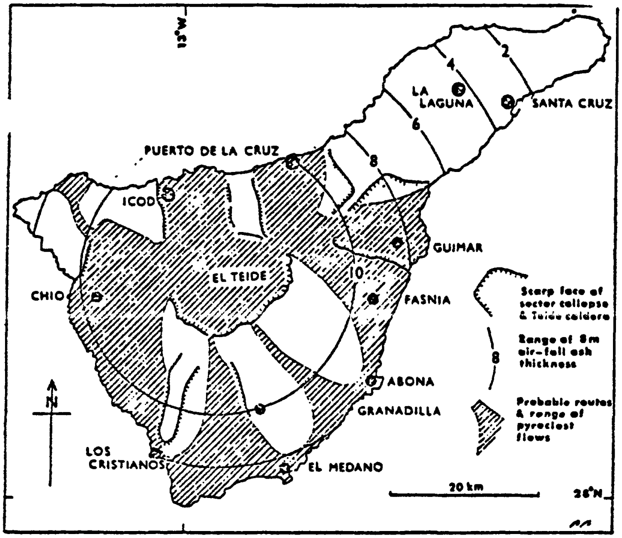

Figure 10. Distribution of some sector collapse depressions on Tenerife, Canary Isles, and probable routes of future pyroclastic flows. (Booth 1979).

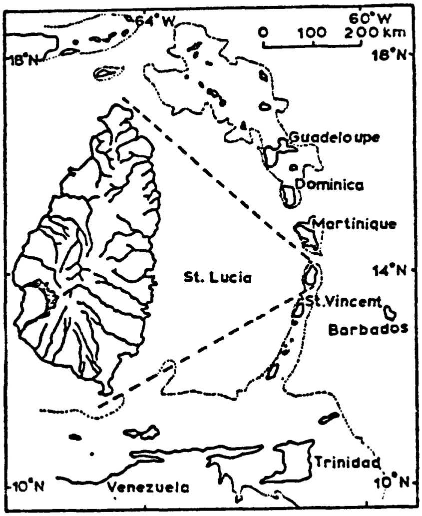

Figure 11. The Lesser Antilles showing the position of St. Lucia. The dotted line is the 1000 m submarine contour. In the enlarged inset of St. Lucia the walls of the "caldera" are shown in black. The river system indicates topography. (Roobol and others 1983).

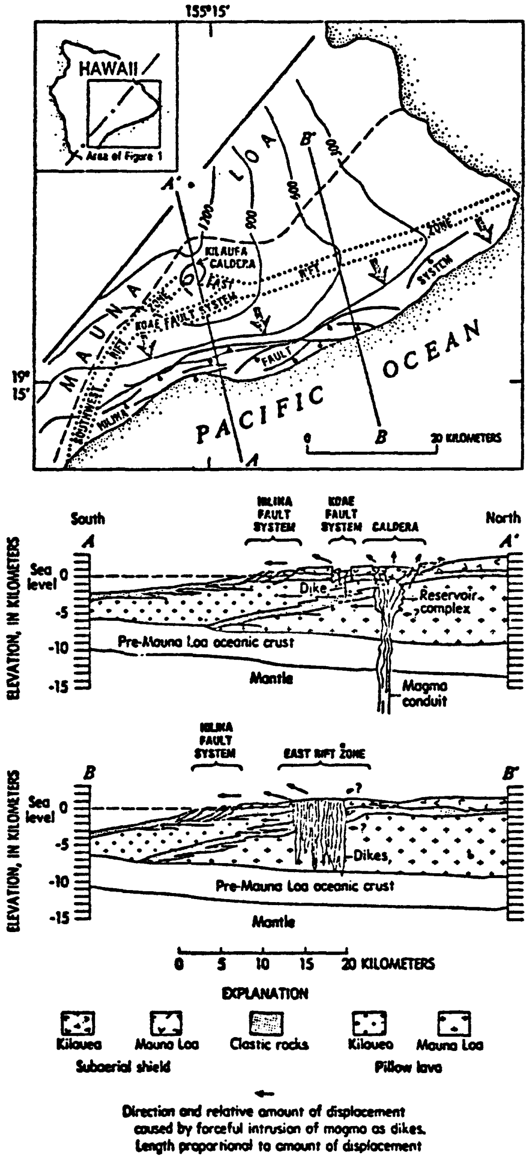

Figure 12. (Swanson and others 1976).

volcanoes where there is a potential for major development or siting of critical facilities. For volcanoes that present long-term hazards, and at which eruptions could cause significant effects, preparation of a hazards assessment based on eruptive history, initiation of some monitoring, development by civil authorities of broad contingency plans for future eruptions, and consideration of procedures for mitigating the effects of future eruptions should be undertaken. Even volcanoes that present hazards of lesser significance should be studied to ascertain their eruptive histories, because these studies may reveal past events of low frequency, but of very great magnitude, that would otherwise not be known. At the least, adequate topographic maps for all volcanoes and volcanic areas should be prepared. These will aid in future studies and will also provide an invaluable aid for rapid hazard assessment if the likelihood of activity were to increase suddenly at an unstudied volcano.

Volcanoes that have been frequently active in our lifetimes constitute a different level of hazard, referred to as short-term or immediate hazard. Those volcanoes that present short-term hazards and threaten life and property should be kept under close surveillance by various monitoring techniques. In addition to surveillance, studies should be undertaken to decipher the volcano's historic and prehistoric behavior. Consideration should be given to restricting various land uses in areas of greatest hazard, educating the public about hazards, and preparing contingency plans for future eruptions. Many of the following assessment techniques apply to short-term hazards, but will be discussed in the context of assessments of long-term hazards.

The type of information that is needed for an assessment of long-term volcanic hazards comes from the character of a volcano's activity in the past and the nature of activity at similar volcanoes in the region of interest and elsewhere. The first gives a view of a volcano's most likely type of future behavior; the second provides insight to events of low probability, but perhaps of great magnitude, that could occur. For instance, the lateral blast of 1980 at Mount St. Helens was much larger than any such previous event there, but was similar in magnitude to the lateral blast at Bezymianny in 1956 (Miller and others, 1981). One problem with this second approach is that dealing with worst cases may be regarded by others as alarmist.

The record of past activity at a volcano comes from both historical accounts and the deposits of past eruptions that are preserved on and around a volcano. Because many historical accounts were made by nonvolcanologists or by volcanologists at a time prior to our present understanding of volcanic phenomena, these accounts need to be substantiated by study of the deposits that resulted from these eruptions (Blong, 1982). Thus the pyroclastic surge that was responsible for most of the fatalities during the AD 79 eruption of Vesuvius, for which a written account exists, was not recognized until recent studies of the deposits were conducted (Sheridan and others, 1981).

Interpretation of past activity from the geologic record is also problematical because the identification of the character of past eruptions relies on an accurate interpretation of the type of event from the deposit it produced, and being able to determine the extent of the area affected by the event.

In principle, the study of long-term volcanic hazards requires:

1. Establishing a stratigraphic sequence of the products of past eruptions, and tracing the extent of individual deposits or groups of deposits.
2. Determining the origin of the deposits in the stratigraphic sequence.
3. Dating the deposits by various techniques in order to develop a chronology of the events.

Stratigraphic Sequence of Products of Past Eruptions

Standard stratigraphic principles are used in establishing the sequence of the products of past eruptions. However, complications in a detailed study of this type result from non-deposition (many eruptions will leave a thin and (or) discontinuous deposit) and erosion (succeeding volcanic events or surficial processes during periods of quiesence can alter or remove part or all of the record of an eruption). Because of these problems, the development of a stratigraphic sequence is a long and complicated task that requires the study of many sections. One should assume that the records of some events will be missed even with very careful work.

The most productive places to begin stratigraphic studies are the lower flanks and ridge areas downwind from the volcano rather than high on the cone. This is because much of the record of past eruptions is found in the fragmental deposits, which are best preserved and exposed beyond the extent of the cone-building lava flows, in areas of gentle slope, and above valley bottoms. In addition, initial studies should be aimed toward discovering and tracing any beds that might be easily and widely recognized. These can serve as stratigraphic markers, and are generally beds of air-fall tephra.

Valley floors leading away from a volcano are sites where deposits of pyroclastic flows and lahars and lava flows will be concentrated. The stratigraphic position of these deposits can be determined by their relation to marker beds of tephra. The extent of flowage deposits then needs to be determined by geologic mapping.

Once a preliminary eruptive history is known, the geologist needs to decide how far back in time a detailed record should be constructed. This will vary among volcanoes, but the record should consist of at least several eruptive episodes in order to assess both the type and frequency of events that have occurred in the past. In general, as one tries to extend the stratigraphic record back in time, both the required effort and incompleteness of the record increase.

Origin of Deposits of Past Eruptions

The determination of the genesis of some volcanic deposits is probably the most difficult aspect of a study. Texture, lithologic composition, sorting, bedding, evidence of heat, topographic expression, and areal distribution are some of the features that are used to determine the mode of origin of fragmental deposits; these are summarized in the following discussion and table. For all of these deposits, it is best to use multiple lines of evidence for determining the origin of a particular deposit.

Tephra-fall deposits generally have mantle bedding that drapes the preexisting land surface and uniform thickness in a given area of moderate to low slope. The deposits are well to moderately well sorted (Figure 13) and are commonly stratified, a single bed may be normally or reversely graded. Successive beds may change in character with changes in the vigor of the eruption, the composition of the erupted material, and the direction and speed of the wind.

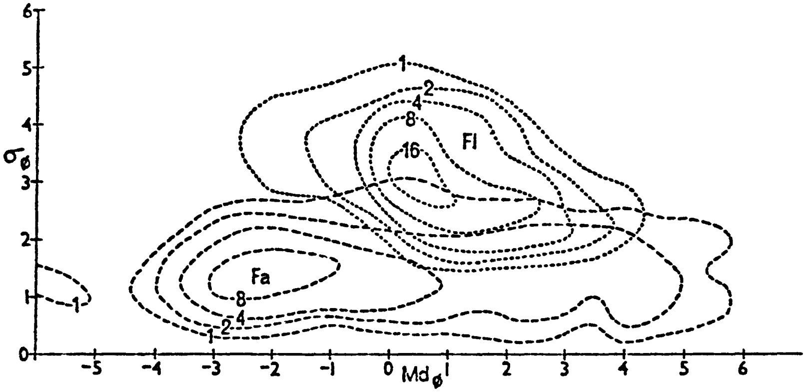

Figure 13. Contoured plot of the data for fig. 1 for pyroclastic fall (Fa) and pyroclastic flow (Fl) deposits, prepared as described in text. Figures are percentages. (Walker 1971).

Exceptions to these characteristics include poor sorting of some air-fall tephras, especially close to vents or where the eruption clouds were flushed by rain. Some deposits that fell while still very hot or were subjected to high temperatures after deposition can weld, which disrupts and disguises many features.

Pyroclastic-flow deposits are typically nonsorted (Figure 13) and nonbedded and tend to thicken in valleys and depressions rather than evenly mantling the landscape. Lahars also have many of these features (table 3); a valuable feature for distinguishing between pyroclastic-flow deposits and deposits emplaced at low temperatures is by evaluating evidence for their emplacement temperature. Evidence for a high emplacement temperature includes welding, carbonized wood, thermally caused brick-red to reddish-gray zones near tops of deposits, and baking of underlying surfaces. Studies of thermoremanent magnetization can provide evidence of emplacement at temperatures either above or below a deposit's Curie temperature (Aramaki and Akimoto, 1957; Mullineaux and Crandell, 1962; Crandell, 1971); more sophisticated studies can provide close estimates of emplacement temperature itself (Hoblitt and Kellogg, 1979; Kent and others, 1981). In some cases, pyroclastic-flow deposits have an apparent stratification that is caused by a sequence of several flow units or by normal grading of lithic clasts and reverse grading of pumice clasts.

Pyroclastic-surge deposits are formed by both hot (ash-cloud surge) and cold (base surge) surges. Deposits of hot pyroclastic surges are typically thin, fine grained, well stratified to massive, and commonly mantle landscapes (Crandell and Mullineaux, 1973; Fisher and Heiken 1982). Individual beds range in thickness from a few millimeters to as much as 1 m, vary greatly in thickness laterally, may be relatively well sorted, and commonly contrast in grain size with adjacent beds. Stratified deposits show planar, wavy, or low-angle crossbedded laminations, and may form dunes. Base-surge deposits have similar bedding features (Crowe and Fisher, 1973; Fisher and Waters, 1970; Schminke and others, 1973), but may form thick deposits around maars in contrast to the generally thin deposits of hot pyroclastic surges.

Table 3. Features that Help in Distinguishing Coarse Unsorted and Unstratified Deposits of Volcanic Rock Debris of Various Modes of Orgin.

Feature	Genesis of deposits			
	Hot pryoclastic flow	Lahar	Avalanche	Glacier
Variety of rock types	Monolithologic	Monolithologic or heterolithologic	Typically monolithologic	Typically heterolithologic
Shape of clasts	Typically angular and subangular	Typically angular and subangular	Typically angular	Typically subrounded; may have facets, striations, snubbed ends.
Evidence of heat	Generally present	Rarely present	Absent	Absent
Distribution within valley	Generally limited to valley floors or other depressions; may be considerably higher on obstacles in path of flow.	Generally limited to valley floors or other depressions; are not appreciably higher on obstacles in path.	Extend directly downslope from source; may be many tens of m higher on obstacles in path.	Limited only by thickness of glacier
Length	Generally less than 15 km	May be many tens of km.	Generally only a few km	Limited only by length of glacier
Topography of deposits	Generally form flat-topped fill in valley	Generally form flat-topped fill in valley; may veneer valley sides up to maximum height reached by lahar. Mounds of rock debris may be present.	Generally very irregular	Generally irregular
Nature of margins in flat areas	Typically digitate	Typically digitate	May be digitate or smooth	Generally smooth
Textural gradation	Vertical change in clast size not common	Clast size generally decreases upward.	No vertical change in clast size	No vertical gradation in clast size

Few deposits of laterally directed blasts have been studied in detail, and much of our information about them comes from the deposit of 1980 at Mount St. Helens (Hoblitt and others, 1981). This deposit is composed of several stratigraphic units that have characteristics of pyroclastic flows, surges, and falls; and as a whole is normally graded (Figure 14). Important criteria for the recognition of deposits of laterally directed blasts include its wide lateral distribution and relative thinness, its lack of control by topography, and its evidence for rapid lateral transport.

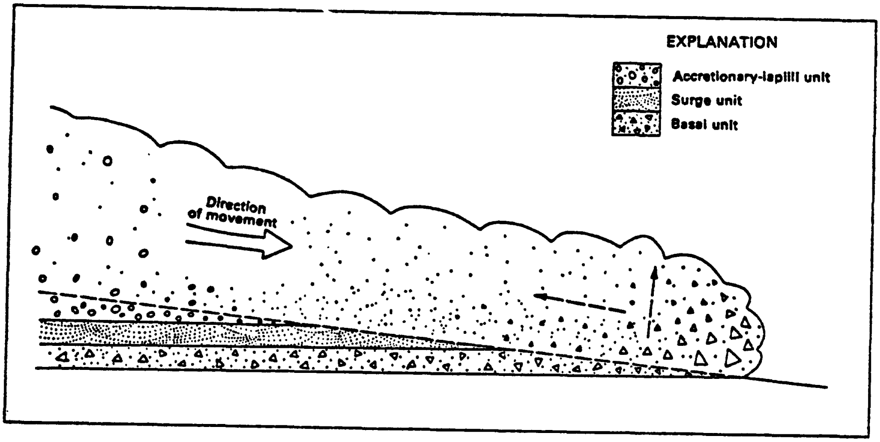

Figure 14. Diagrammatic model showing deposition of basal, surge, and accretionary-lapilli units from blast cloud. Particle size and concentration decrease upward and backward (dashed arrows) in the cloud. (Hoblitt and others 1981).

Lahars have many textural and other characteristics in common with deposits of pyroclastic flows, debris avalanches, and glacial till, and many criteria are needed for differentiating between them (table 3). Features of lahars that may aid in their identification include the common presence of vesicles in their matrix; normal grading of the large clasts in a uniform matrix of some lahars and reverse grading in others; a thin stone-free basal zone in some; vertical, fines-depleted pipes in some hot as well as some apparently cold lahars; and evidence of emplacement temperatures below the Curie temperature (Mullineaux and Crandell, 1962; Crandell, 1971; Hoblitt and Kellogg, 1979). A major problem in interpreting the origin of many of these unsorted and unstratified deposits is that in certain circumstances one may be transitional to another. For instance, pyroclastic flows can become lahars as they incorporate water; lahars can also be derived from debris avalanches.

Dating of Volcanic Deposits

In order to develop a chronology of volcanic events for estimating frequency of past eruptions or for estimating how often certain areas have been affected by eruptions, the stratigraphic record needs to be dated as accurately as possible. A variety of methods are available (Colman and Pierce, 1977), but all have limitations. Goudie (1981) is a good source of information about a variety of dating techniques.

Historical records can be valuable sources of information; however, these records are short in many volcanic areas, and the older records may contain inaccuracies.

Radiocarbon dating is an especially useful technique for use in young (40,000 yr) volcanic rocks because lava flows (Lockwood and Lipman, 1980), pyroclastic flows, hot lahars, and some tephra are hot enough to convert wood and other organic materials that they contact to charcoal, which can be preserved for long periods. Floods, lahars, and cold tephra may also contain or bury organic materials that are suitable for radiocarbon dating. Several factors influence the accuracy of a radiocarbon date. Because of natural variations in the atmospheric concentration of ^{14}C, radiocarbon dates may vary from true, or calendar, ages. The radiocarbon time scale has been calibrated for the last 8,000 years (Klein and others, 1982; Stuiver, 1982); radiocarbon ages can vary from calendar ages by as much as 1,000 years (Figure 15). In addition, one radiocarbon age can have two or more possible calendar ages (Figure 16). Another problem comes from geologic contamination of samples with materials that have a different radiocarbon content; generally contamination with materials having a higher ^{14}C content, and therefore making a date too young, presents the greatest problem. If dated material underlies or overlies a deposit, it can only provide a limiting age for the deposit. Wood or charcoal from the center of a large log can only provide a maximum age for the enclosing deposit. Generally tens of grams of sample are needed for a radiocarbon date; however, a new technique for dating of milligram-size samples by mass spectrometry is now under development (Bennett and others, 1978).

Dendrochronology (Fritts, 1976) can be used in dating some volcanic events in areas where trees develop annual rings. Accurate ages (±1 year) for events can be determined by using trees whose growth was affected but were not killed, by an eruption (Yamaguchi, 1983). The age of the oldest tree growing on a deposit that would have devastated the forested area on which it was deposited provides a minimum age; this age postdates the age of an event by the time necessary for trees to recolonize the deposit, and may range from 1 to tens of years. Problems in applying this technique include finding the oldest trees on a deposit and acquiring skill in coring to the center of large trees. Dating of events by this method is limited by the life expectancy of local vegetation types.

Lichenometry is a dating technique based on the rate of growth of certain lichen species (Webber and Andrews, 1973). For each region and species of interest, growth-rate curves need to be constructed by measuring the diameters of individuals that are living on substrates of known age. The growth curve is then used for determining the age of other deposits. Limits to this method include (1) the large amount of effort involved in developing a growth curve and (2) several factors that govern lichen survival and growth (microclimate, snow kill, etc.) that are difficult to evaluate for study sites. The age of senescence of various lichens limits the usefulness of the technique to the last few thousand years or less.

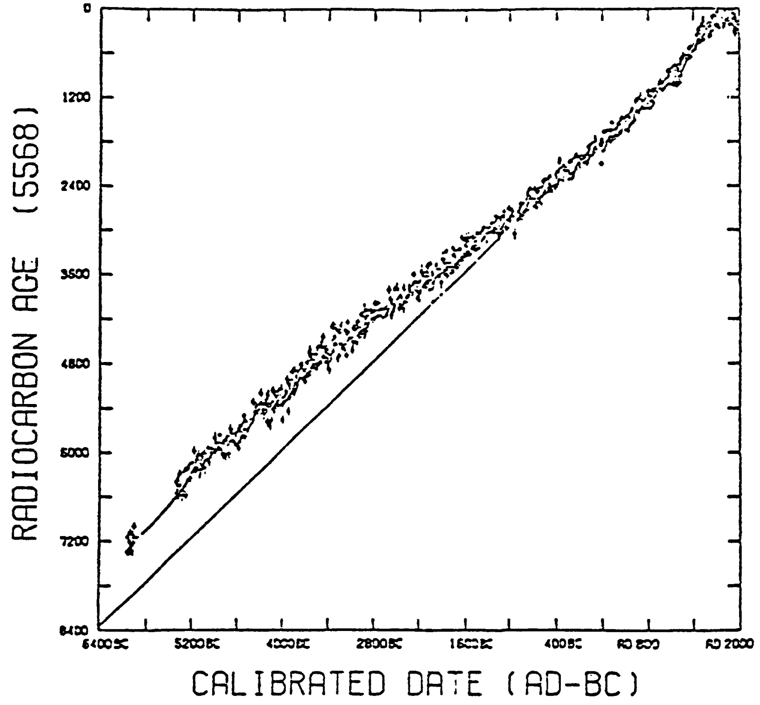

Figure 15. Graphic representation of the period covered by the calibration tables. The ordinate is the conventional radiocarbon age in years or (1950 used as orgin, ages calculated using the 5568-year half-life): the abscissa is the calendric date in years AD-BC. The same data set as in Figure 1 is plotted, but the data here have been winsorized as described in the text. The function includes both the trend analysis and the Fourier equivalent analysis of the residuals around the trend. If conventional radocarbon years were equivalent to calendric years, all the data would fall on the diagonal line that they do not is readily apparent. The maximum deviations between uncalibrated conventional radiocarbon dates and calendric dates occur ca. 5200 B.C. (Klein and others 1982).

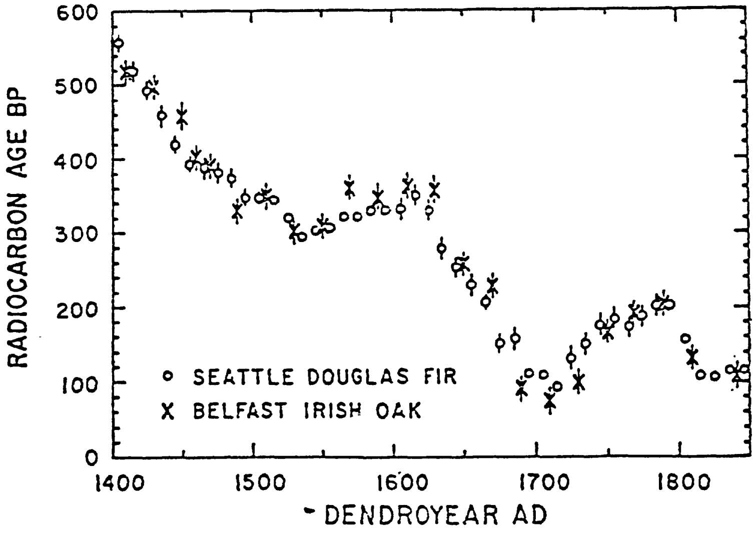

Figure 16. A comparison of Seattle Douglas Fir (O) and Belfast Oak (X) radiocarbon ages. Vertical bars represent the quoted standard deviation. (Stuiver 1982).

Obsidian-hydration dating is based on the rate of hydration of obsidian along cracks and surfaces, which is a function of chemical composition and temperature (Figure 17; Friedman and Obradovich, 1981). The method requires calibration by other dating techniques or by experimental studies; but uncertainties in determining the temperature history of samples add greatly to standard errors. This is less of a problem for young deposits than for those that have been present through major climate changes. The range of the method is from a few thousand to several million years. In addition, techniques are being developed that use the thickness of hydration rinds on glass shards as a dating technique. An attractive aspect of hydration dating is that little expensive equipment is needed other than saws, laps, and a petrographic microscope.

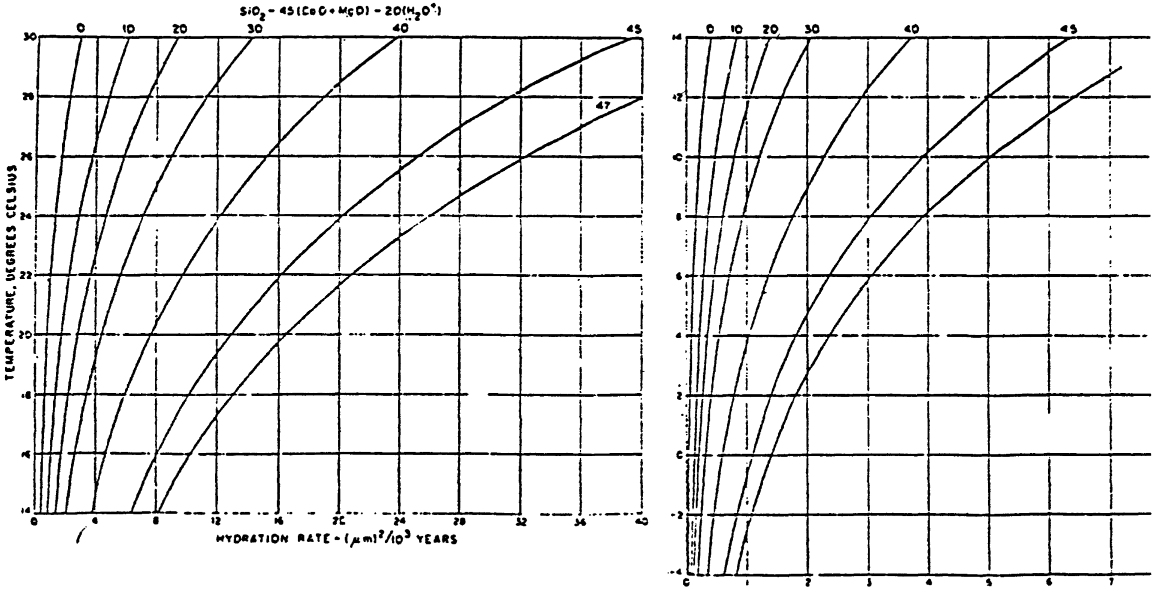

Figure 17. Generalized curves of hydration rate as a function of temperature. The SiO_2 • 45(CaO + MgO) • 20 H_2O^+ index is given for each curve. (Friedman and Long 1976).

Thermoluminescence (TL) dating of volcanic rocks is still in a developmental stage; although some success has been reported with plagioclase from alkalic basalts that range in age up to 250,000 years (May 1979). TL is the emission of light that accompanies heating of a material. The process is incompletely understood; however, the amount of light energy released as TL is proportional to the rate and duration of radiation experienced by the sample. Dating a sample requires measuring both natural and induced TL and the radioactivity of the sample. Calibration by other methods is necessary.

Secular variations of the Earth's magnetic field, which are recorded by volcanic products that cool through their Curie temperatures after coming to rest, offer promise for dating and time correlation (Champion, 1980). A record of the small changes in magnetic-field direction over time need to be constructed before the method can be applied, and this requires a large effort. Lake cores often provide a continuous record, and show that unique directions are few during the Holocene so that other techniques are usually necessary in order to assign a probable age. The method can also be combined with other evidence in showing time equivalence of units.

Several relative-age criteria are useful for estimating the age of a volcanic deposit. All are based on the calibration of the rate of change of some weathering-related feature. In volcanic regions with humid or subhumid moisture regimes, clasts of andesite and basalt in the B horizons of soils acquire a weathering rind that thickens at an exponentially decreasing rate (Figure 18; Colman and Pierce, 1981). Also many soil properties, such as color, clay content of B horizon, calcium carbonate content of C horizon, and depth of oxidation, change with time (Birkeland, 1984). These techniques are also useful for estimating the length of time represented by soils or weathering profiles that occur in a sequence of volcanic deposits. Relative-age methods based on the degree of modification of the origional morphology of volcanic landforms or deposits are highly subjective and generally are only useful in distinguishing broad differences in age.

Volcanic-Hazards Assessments Based on Eruptive History

An example of an eruptive history of a volcano that was compiled by using the techniques described in the preceding sections is shown in Figure 18.

This record is for Mount St. Helens, which during Holocene time has been the most frequently and diversely active of the Cascade volcanoes. In addition to the general information given in Figure 19, other information has been compiled on the extent of the deposits and on the details of each eruptive episode, especially as concerns the timing of events, changes in mineralogy and chemistry of the eruptive products, and the character of eruptions.

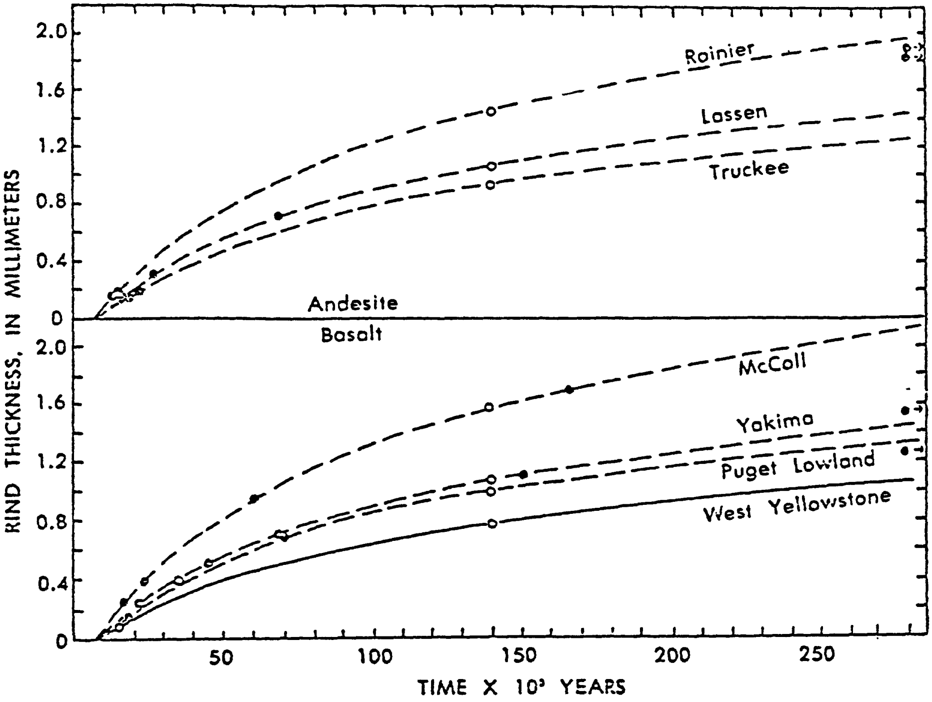

Figure 19. Eruptive history of Mount St. Helens from Crandell and Mullineaux (1978) and Mullineaux and Crandell (1981).

This record then needs to be interpreted in order to assess the types and magnitude of hazards that exist in the area. Most assessments of long-term hazards have used the past eruptive record as a guide for predicting the frequency and character of future eruptions and the areas likely to be affected by them. This information is generally portrayed on maps, so to be of greatest use to authorities, planners, and the public. The philosophy and preparation of hazard maps will be discussed later.

There are several shortcomings of this method of hazard assessment that need to be appreciated by the person making the assessment and by those using it.

1) The farther back in time a record is extended, the less complete it is likely to be. Hence, eruptions may have occurred during the apparent 15,000-year dormant interval between Mount St. Helens' first two recognized eruptive episodes, but the record of these events has not been preserved or found. Times of landscape instability during the major climatic changes that occurred between 20,000 and 10,000 years ago may partly explain the absence of evidence of eruptions during this time interval.

2) Even in the better known and probably well preserved part of the record (the last 10,000 years) apparent dormant intervals have ranged in length from a few hundred to several thousand years. This means that the frequency of past eruptions is subject to a large-statistical error. In this example, the last few thousand years of record provides a better guide to the frequency of future eruptions.

3) Eruptive behavior may change over time at a volcano, which means that past behavior is not necessarily a reliable predictor of future behavior. The apparently abrupt change in the chemical composition of products about 2,000 years ago at Mount St. Helens (during the Castle Creek eruptive episode) would not have been predicted at that time based on the previously erupted products. Also, the more varied eruptive character of the recent episodes at Mount St. Helens make

prediction of eruptive character during the present episode more subject to error.

4) Events may occur of a magnitude that is unprecedented in the eruptive history of a given volcano.

5) Over time a volcano can grow in size and change in shape as volcanic and erosional processes operate. These modifications will change the likelihood of an area being affected by various events. Thus at Mount St. Helens, the existing hazard assessment needed to be revised following the great topographic changes that accompanied the May 1980 eruption (Miller and others, 1981).

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RECOMMENDATIONS FOR VOLCANIC-HAZARDS ASSESSMENTS AND LAND-USE PLANNING BASED ON U.S. GEOLOGICAL SURVEY EXPERIENCES DURING THE 1980 ERUPTIONS OF MOUNT ST. HELENS, STATE OF WASHINGTON, U.S.A.

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Introduction

During eruptions of Mount St. Helens from March to October 1980, the U.S. Geological Survey evaluated volcanic hazards, issued-warnings, and provided geologic information related to understanding and mitigating hazards. In the course of the duties we made observations that may be pertinent to assessment of hazards during future eruptions at other volcanoes.

Hazards assessments at an erupting volcano are greatly facilitated by (1) pre-eruption in-depth studies of the volcano's historic and prehistoric eruptive behavior, (2) the presence of geologists who have studied that specific volcano and are familiar with local geography and land-use patterns, (3) the presence of geologists who are familiar with the international literature on historic eruptions at similar volcanoes, and (4) the assignment of specific responsibility to one or more geologists to frequently evaluate the results of monitoring and to interpret the significance of the results to potential hazards.

Geologists responsible for volcanic-hazards assessments should choose specifically known, and therefore credible, events as models for defining hazard zones and for evaluating risk around an erupting volcano. These evaluations can be based solely on the past behavior of the volcano, or they can be based wholly or in part on more extreme events that are unprecedented at that volcano, but which have occurred at similar volcanoes elsewhere. These two approaches can be illustrated by our experience at Mount St. Helens, where recognized past events included lava flows, pyroclastic flows, lahars, dome emplacement, and pyroclastic eruptions that resulted in airfall tephra. However, only one relatively small directed blast, and no large debris avalanches, had been identified in the past record. Nevertheless, the May eruption included a catastrophic directed blast and a very large debris avalanche, each much larger than any similar events of the past at Mount St. Helens. Consequently, we believe that although the past behavior of a volcano is an appropriate model for an initial hazards assessment for purposes of land-use planning, such an assessment should be modified during eruptive activity if monitoring data suggest events unprecedented at that volcano may occur. Historic events at similar volcanoes should then be used as more appropriate models to modify the initial hazards map.

Placement of hazard-zone boundaries around a volcano can be very difficult because types and sizes of actual events are difficult or impossible to forecast. It is probably necessary to use more than one model of scale of activity to indicate a range in size of potential volcanic events. Although volcanic events of small scale are more frequent than large ones and thus more likely, presentation of this information to the public may result in a disregard for low-frequency, low-probability events that can have severe consequences. At Mount St. Helens, limits of areas closed for public safety did not closely coincide with maximum-extent hazard-zone boundaries based on the past activity of the volcano. This lack of correspondence resulted in large part because the limits of closed areas were based on economic, social, and political considerations as well as potential hazards, and the probability of maximum-scale events was low.

The May 18, 1980 eruption of Mount St. Helens showed that directed blasts present special problems for hazards assessment. Such blasts are among the most hazardous and destructive types of eruptions because they can occur with no direct or immediate warning, and can travel at speeds of hundreds of meters per second. They can extend to a distance of more than 25 km from a volcano and can affect area of hundreds of square kilometers. Unfortunately, previous occurrences and sizes of directed blasts at a volcano may be difficult to establish because deposits from such blasts are difficult to recognize and may not be well preserved.

In order to mitigate the effects of directed blasts, we suggest that if possible precursors occur, such as extensive deformation of a flank of a large steep-sided volcano, prolonged shallow seismicity, or other indications of magma emplacement in such a volcano, the potential blast zone of a hazards map should be based on the largest known blast at a similar volcano. Deformation of the volcano may forecast the azimuth of the axis of a potential blast as suggested by Gorshkov (1963), and as occurred at Mount St. Helens. If so, restrictions on land use can be suggested for a zone centered on the azimuth of deformation with a radius of as much as 30 km, and covering a sector at least 180 wide near the volcano. Within such a zone, possible hazards include high-speed rock fragments and clouds of ash and gases, temperatures of several hundred degrees Celsius for several to many minutes, and deposition of a blanket of rock debris and ash.

A problem encountered at Mount St. Helens involved communicating geologic information to civil authorities who may never before have needed to understand the meaning and significance of such concepts as probability, hazard-zone boundaries, and gradational risk. Some authorities, as well as news-media representatives and the public, often expected yes/no answers to scientifically complex questions concerning the processes of and risk from volcanic activity, and were critical of qualified answers or "we don't know" responses. Whenever possible, scientists should use simple terms to describe geologic concepts, and if data allow, potential volcanic events and risk should be presented in terms of probability.

Another problem concerned the responsibility of issuing hazards-related information to civil authorities and the public. We believe that, to the extent possible, hazards information should incorporate advice from governmental, academic, and international experts. However, this information should be released by a single authoritative source, rather than by several or many scientists or groups of scientists. Severe problems can develop when scientists not directly responsible to civil authorities give their opinions directly to authorities and news media without resolving differences of opinion with the agency responsible for hazards assessments. Resolution of such differences is important because public officials generally do not have the backgrounds necessary to evaluate conflicting scientific advice.

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VOLCANIC-HAZARDS ASSESSMENTS AND ZONATION

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INTRODUCTION

A. Assessment of potential hazards involves determination of the likely types, scales, and locations of future eruptions and areas likely to be affected by products from eruptions.

B. Such assessments can be used for two primary purposes:

1. Long-range land-use planning.
2. To provide information to help mitigate the effects of future eruptions.

C. Hazard assessments are prepared on the assumption that the types of eruptive events that have occurred during the last few thousand years at a volcano will continue into the future.

D. Thus, hazard assessments and hazard-zonation maps are based primarily on the historic and prehistoric eruptive history at a volcano. Extreme events at other similar volcanoes may also be used as models.

EXAMPLES OF HAZARD ASSESSMENTS

A. Volcanic-hazard assessment for Mount Shasta, California.

An example of volcanic hazards at a Cascade stratovolcano. The hazard assessment is based on detailed studies of the eruptive history during the last 10,000 years.

Discussion of hazard zonation for:

1. Flowage-hazards including pyroclastic flows, mudflows, and floods.
2. Lava flows.
3. Tephra hazards.

B. Preliminary volcanic-hazard assessment for the Long Valley-Mono Lake area, California.

An example of volcanic hazards in a region of bimodal volcanism where eruptions have ranged in volume from 1 km^3 to about 600 km^3. The preliminary assessment was prepared in a very short time in response to possible eruption precursors.

1. Small-volume (1 km^3) explosive silicic eruptions.

    - Combined flowage-hazard zone
    - Tephra-hazard zone

2. Caldera-forming eruption.

    - Flowage-hazard zone
    - Tephra-hazard zone

C. Volcanic-hazards assessments at Mount St. Helens, Washington.

An example of volcanic hazards at a young silicic stratovolcano in the Cascade Range. Assessments prepared before, during, and after 1980 eruptions.

1. Discussion of hazard zonation prepared in 1978 for long-range land-use planning.

    - Flowage-hazard zones
    - Tephra-hazard zones

2. Discussion of USGS hazard-zonation maps used by authorities during the 1980 eruptions.

    - Flowage-hazard zones
    - Tephra-hazard zones

3. Discussion of hazard-zonation map prepared by USGS after May 25, 1980.

    - Flowage-hazard zones
    - Directed-blast zone

WHAT WE LEARNED AT MOUNT ST. HELENS AND SOME RECOMMENDATIONS

A. Discussion of factors important for making decisions during eruptions and giving out information to mitigate the effects of an eruption:

    1. Preparation of volcanic-hazard assessment before activity begins.
    2. Extensive monitoring network at volcano.
    3. Onsite agency to help with logistics and communication, etc.
    4. Hazard assessment coordinators familiar with volcano.

B. The past behavior of a volcano is a good basis for an initial hazards assessment for purposes of land-use planning.

However, such an assessment can be modified during eruptions, or precursors to eruptions, if monitoring developments suggest that unprecedented events might occur.

Under such circumstances, historic events at other similar volcanoes can be used as appropriate models to modify an initial assessment.

C. Who has the responsibility for issuing hazards-related information to civil authorities and the public?

1. If possible, obtain and use advice from:

   - Government scientists
   - Scientists from academic and other institutions
   - International experts

2. If possible, all scientists monitoring and interpreting data should try to reach a concensus and the results should be released from a single authoritative source.

3. Serious problems can develop if each scientist gives his own opinion to the authorities. Who do they believe?

D. Communicating volcanic-hazard information to authorities and the public is difficult. Concepts such as probability, hazard-zone boundaries, and gradational risk are difficult to-get across to nonscientists. Use simple terms to describe such concepts if it is possible.

Authorities often expect simple yes/no answers to complex questions; this usually is not possible.

SHORT-TERM FORECASTING OF VOLCANIC HAZARDS

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INTRODUCTION

If a normally quiet volcano begins to show signs of restlessness, volcanic hazards cease to be abstractions in a dusty, long-range planning document or scientific report and become vivid realities that might occur as soon as tomorrow, next week, or next month. Volcanic hazards assessments forecast the time, place, and nature of volcanic eruptions, areas that are likely to be affected by the eruptions, and the likely severity of those effects. Hazards forecasts may be long-term (years, decades, or longer) or short-term (months, weeks, days, or hours). Long-term hazards assessments are based primarily on the geologic and historical record of eruptions at the particular volcano in question (Scott, 1984). Short-term hazards assessments are modifications of the long-term assessment, based on changes in that volcano's behavior over the preceding months, weeks, days, or hours. Long-term forecasts are relatively general, describing a wide range of possible activity and a wide "window" of time, perhaps decades, within which the next eruption is expected. Short-term forecasts are more specific, with a shorter window of time and, if possible, a narrower range of possible activity.

A curious and unfortunate dichotomy has developed in some countries: those who study volcanic stratigraphy and the geological evolution of volcanoes have relatively poor communication with those who use geophysical and geochemical tools to study short-term volcanic processes. In other cases, various geophysical or geochemical parameters are studied by workers in different agencies or institutions, unaware of each other's work and data. Poor communication during times of volcanic quiet results in inefficiency and a delay in learning about the volcano in question. But poor communication during a volcanic crisis can cause a serious underestimate or overestimate of hazards, and a toll of many lives in the first instance or many months of unnecessary disruption in the second. Short-term hazards assessments should integrate all available information--geological, geophysical, and geochemical. These lectures will discuss some ways to integrate various types of information into working hypotheses that can be tested with further geologic studies and monitoring.

About a decade ago, several publications reviewed methods for short-term forecasting of volcanic eruptions (UNESCO, 1971; Decker, 1973; Civetta and others, 1974). A shorter review of these methods appears in papers that use experience at Mount St. Helens to prepare for volcanic hazards in California (Martin and Davis, 1982). The present lecture notes include material from those early reviews, new information gained within the past decade, and some thoughts on short-term volcanic processes and approaches to forecasting volcanic eruptions. These notes are not a substitute for a proper review by experts in each respective field of volcano monitoring; hopefully, a recent book by Tazieff and Sabroux (1983) will be the first of a new round of reviews.

WHAT CAUSES VOLCANOES TO ERUPT?

After many years of study this question remains only partly answered. For the benefit of non-volcanologist participants in this training program, here are some simple, partial answers. The answers are pertinent to this discussion of short-term hazard forecasts, because it is easier to interpret eruption precursors if one has at least some working hypotheses about what is causing those precursors.

Volcanoes are vents at which magma (and often some country rock) is erupted. Magma is a complex mixture of silicate melt, crystals, and dissolved volatiles. Some magma forms by partial melting of the upper mantle or lower crust; such magma is generally basaltic in composition, and has temperatures of between 1000 and 1300°C when it reaches the surface (figs. 1A, 1C, 1E, 1F). When basaltic magma cools through the interval of approximately 1300°C to 700°C (with some variation in the exact temperature range as a function of lithostatic pressure and volatile content), crystals form and the volume of interstitial melt decreases. Volatiles tend to remain in the interstitial melt, now decreasing in volume, so the concentration of volatiles in melt increases as magma crystallizes. Hot magma from depth also heats and partially melts the upper crust, forming silicic magma (figs. 1A, 1B, and 1D). A similar process of volatile concentration occurs as silicic magma cools and crystallizes. Isotopic evidence suggests that some magmas also absorb groundwater, adding to the volatile concentration. Volatiles also tend to rise through magma faster than magma itself rises, leading to an extra concentration of volatiles near the top of a magma reservoir.

As magma rises, decreasing lithostatic pressure and, to a lesser

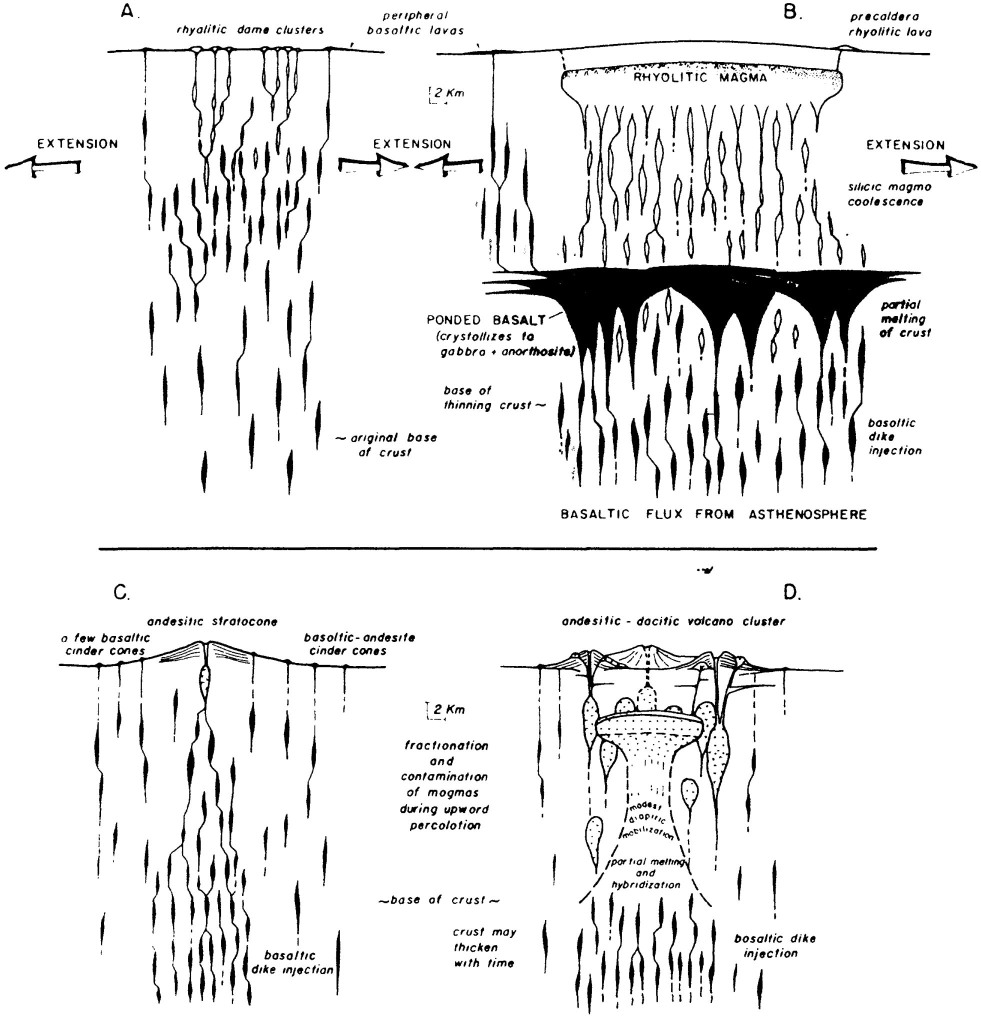
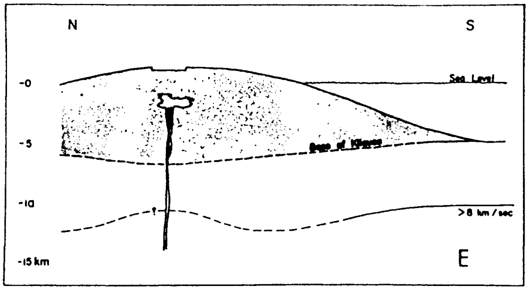
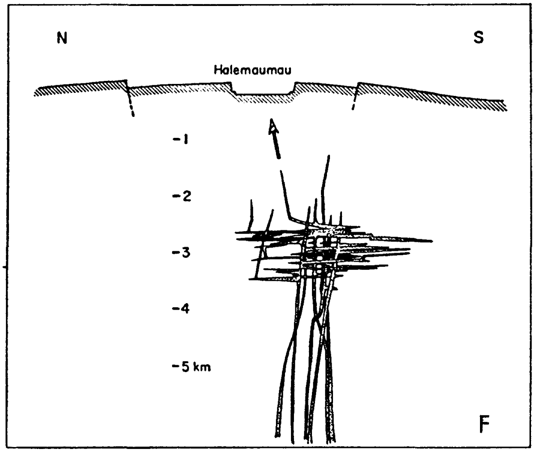


Fig. 1 (caption on next page)





E Cross section of Kilauea volcano showing the position of the magma reservoir lying 2 to 3 kilometers below the surface. F Larger-scale, highly speculative section of the magma reservoir. The arrow shows the possible path of magma as it erupted to the surface on 5 November 1967.

Fig. 1 (continued) Five generalized volcanic systems, to illustrate the general processes of basaltic magma supply from depth, fractionation and contamination of magma, partial melting of crust to form silicic magma, and the various types of volcanoes that form above each type of magma reservoir.

The most fundamental differences between systems are their tectonic setting, their rates of supply of basaltic magma from depth, and their age. A and B are favored in areas of crustal extension, with moderate and high rates of magma supply, respectively. The early development of B may resemble A. C and D are favored where extension, if any, is subordinate and shallow; C may have a lower rate of supply than D, and/or reflect an early stage of development toward D. E and F are cross-sections of Kilauea, representing basaltic shield volcanoes with moderate rates of magma supply from depth. The scale of E is nearly that of A-D; F is enlarged.

A-D are from Hildreth (1981); E-F are from Fiske and Kinoshita (1969).

extent, decreasing temperature reduce the solubility of volatiles in the melt. Any volatile in excess of its solubility will exsolve, forming gas bubbles in the magma. Formation of bubbles ("vesiculation") makes the magma much less dense than surrounding country rock, and can also create outward-directed gas pressures at least a few hundred bars in excess of lithostatic pressure. Heating of nearby rocks and groundwater can facilitate fracturing of that rock. The net effect of these changes is to allow magma to rise as high as its buoyancy will lift it, often onto the surface of the earth, causing eruptions.

A few eruptions are continuous for decades or centuries at a time, but most are episodic, with clusters of eruptions every few decades, centuries, or millenia and individual eruptions within those clusters every few hours to years. One possible explanation of the longer-term episodicity is that influxes of magma from depth are themselves episodic. Over a time scale of decades to millenia, magma generation probably occurs at a constant rate, but a finite period of time is required for melt to accumulate in its source region and overcome physical barriers to its rise. Once those barriers have been overcome, a batch of magma will rise into a shallow magma reservoir, if present, or directly to the surface--an episodic arrival of magma from depth. Magma can be held in a shallow reservoir for years, decades, centuries or even millenia until heat, additional volatiles, and the volumetric addition of magma from depth reactivate the magma of the shallow reservoir and cause it to rise toward the surface. An influx of magma at the base of a shallow reservoir need not push out an equivalent volume of magma from the top of the reservoir, especially if the shallow reservoir is large, but episodicity of influxes from depth is likely to be reflected by equivalent or longer-term episodicity of eruptive periods at the surface (Smith, 1979; Hildreth, 1981).

Shorter-term episodicity, once an eruptive episode has begun, is probably caused by a different process. Magma reservoirs are dynamic systems, and the relative rates of exsolution, diffusion of volatiles through and out of the magma, and passage of magma itself out of the system (onto the surface) exert a great influence on whether and in what manner magma will erupt. If gases escape freely from magma, few or no eruptions will occur, because volatiles will not concentrate in the magma and give it added buoyancy. If gases escape slowly enough to give the magma added buoyancy, but fast enough to prevent a cumulative increase in gas pressures, magma might be erupted continuously. Most commonly, however, exsolved gases are unable to diffuse through melt and cap rock fast enough to keep gas pressures from increasing. Gas pressures increase and finally overcome the confining pressure, permitting magma to vesiculate, rise, and erupt. If the vesiculation is extremely rapid, an explosive eruption results; if a downward propagating shock wave also results, even more magma will vesiculate, resulting in a larger explosive eruption. Eventually, the eruption taps into deeper, volatile-poor portions of the magma, and the decrease in available volatiles plus the increasing lithostatic load of the magma column that still remains in the conduit cause the eruption to stop. The volcano remains in repose until fresh volatiles concentrate in the top of the reservoir and can once again overcome the confines of the conduit to start another eruption. In short, eruptions tend to be episodic because volatiles cannot always escape at precisely the rate required to sustain a continuous eruption, and because once eruptions begin they tend to exhaust themselves. Some external factors, notably regional tectonic strain and even earth tides, can trigger to eruptions at volcanoes that are otherwise ready to erupt.

Some eruptions eject rock from the walls of a vent but do not eject juvenile magma. Such eruptions, called "phreatic eruptions," are powered by steam. Steam can form as (a) cool groundwater seeps downward into contact with hot rock or magma, (b) magma rises upward into contact with groundwater, or (c) confining pressure on hot groundwater is relieved. If a magmatic intrusion is the underlying cause of a phreatic eruption, the precursors to that eruption will probably resemble those for a magmatic eruption, but possibly with more surficial thermal activity than before most eruptions. In such cases, phreatic explosions are often preludes to magmatic eruptions. If a magmatic intrusion is not involved, and the phreatic eruption is instead triggered by a tectonic strain, a landslide, or some other unloading process that relieves confining pressure on hot water, the precursor should be the tectonic strain or unloading itself, possibly accompanied by a visible increase in steaming.

As magma begins to overcome forces acting to contain it in a shallow reservoir or conduit, or as groundwater begins to flash to steam, seismicity and ground deformation will also begin. As the process accelerates, seismicity and ground deformation also accelerate, and are joined in some cases by increased gas emissions. If groundwater is intimately involved, surface thermal activity may also increase. These are the principal four of eight common eruption precursors, described next.

COMMON PRECURSORS TO VOLCANIC ERUPTIONS

Eight main types of precursors provide information useful for modifying a long-term hazards forecast into a short-term hazards forecast. These are: (1) seismicity, (2) ground deformation, (3) gravity changes; (4) magnetic changes; (5) electrical changes; (6) changes in volcanic gas emissions; (7) thermal changes; and (8) visible changes before or during an eruption. It is beyond the scope of these lectures to describe the collection and interpretation of monitoring data in detail. Rather, these lectures will introduce each type of volcano monitoring, provide references to more detailed accounts, and note specific observations or applications to short-term hazards assessment that may not be discussed in the cited references.

Volcanic seismicity

Volcanic earthquakes occur as a magmatic intrusion causes brittle fracturing of country rock, or as movement or explosions occur within the magma. Some volcano-tectonic earthquakes occur when volcanic activity triggers a readjustment of regional tectonic stresses, or vice versa. A few volcano-tectonic earthquakes are as large as M=7, e.g., at Sakurajima in 1914 (Abe, 1981); most volcanic earthquakes are smaller than M=2. Some volcanic earthquakes may be as deep as 60 km, but most are within 5 km of the surface. Some precursory seismicity begins over a year before an eruption (e.g., Krakatau, 1883), most begins a few weeks or months before an eruption (e.g. Mount St. Helens, March-May 1980), and in a few cases seismicity only begins a few days or hours before an eruption (e.g., Krafla, 1975-82; most eruptions of Kilauea).

Several types of volcanic earthquakes have been distinguished. Minakami and others (1951) and Minakami (1960, 1974) defined five broad types of volcanic seismicity, according to focal depths, the nature of the earthquake motion, and associated eruptive activity (fig. 2a):

A-type earthquakes that occur at depths greater than 1 km, are dominated by relatively high frequency waves (greater than or equal to 3 Hz), and have well-defined P and S waves;

B-type earthquakes that occur at depths between 0 and 1 km, are dominated by relatively low frequency waves (less than 3 Hz), and lack clear S waves;

C-type earthquakes that resemble multiple, overlapping B-type earthquakes in a single sequence and occur near the surface during dome growth;

explosion-type earthquakes that also resemble B-type earthquakes but occur during explosions at the surface; and

volcanic tremor, continuous vibration or frequent small earthquakes whose codas overlap to produce a continuous disturbance.

Categories may overlap: some high-frequency events occur at depths less than 1 km, some low-frequency events occur at depths greater than 1 km, and there are strong similarities between B-type, C-type, and explosion type earthquakes (Latter, 1979). The user of Minakami's or any other descriptive classification, therefore, should recognize some gradation and overlap between categories.

Other classifications of volcanic seismicity have been devised, based on the same variables as above, plus location within a volcanic edifice (e.g., summit, rift) and source mechanism. A few examples of other classifications: Koyanagi (1968) for Kilauea and Mauna Loa; Shimozuru and others (1969) for Merapi; Tokarev (1981) for Kamchatkan volcanoes in

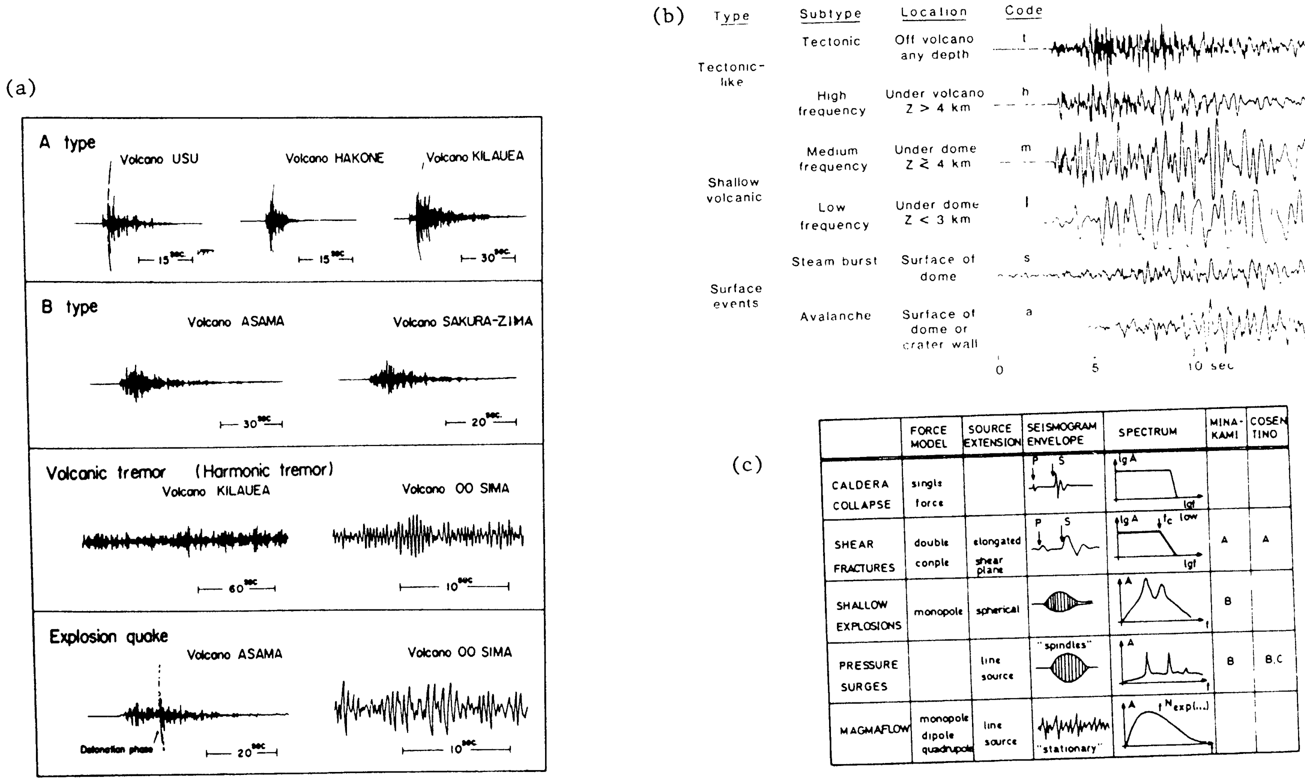


Fig. 2 Common types of volcano-seismic events.
a. largely descriptive classification of Minakami (1974);
b. largely descriptive classification of Malone (1983), for Mount St. Helens;
c. genetic classification of Schick (1981).

general; Zobin (1979 a,b) for Tolbachik; Latter (1979) for Ruapehu and Ngauruhoe; Malone (1983) for Mount St. Helens (fig. 2b); and Schick (1981) for volcanic seismicity in general (fig. 2c).

Two general types of volcanic tremor have been distinguished: harmonic tremor, which is relatively monochromatic (of a single frequency) and has relatively constant amplitude; and "spasmodic tremor", which is less monochromatic and has fluctuating amplitude (Jaggar, 1920). Movement of fluids (magma, water, gas) has often been assumed as the cause of harmonic tremor, although the rapid opening or closing of cracks or the bursting of bubbles in a fluid are alternate sources of harmonic tremor (Aki and others, 1977; Chouet, 1981; Ferrick and others, 1982; Fehler, 1983). Rapid, loud vibration or "water hammer" in household plumbing may be an analogy to harmonic tremor (Ferrick and others, 1982). Empirically, harmonic tremor suggests that an eruption is likely and imminent. Strong harmonic tremor at Mount St. Helens occurred several weeks before the large eruption of May 18, 1980, and tremor in Hawaii often begins a few hours before an eruption. Thus, its appearance is a cause for immediate concern. "Spasmodic tremor," in contrast, may be a composite of many small earthquakes occurring so frequently that their codas overlap. Because the term "spasmodic tremor" has been used to refer to nearly all lengthy seismic signals that cannot be called harmonic tremor, it has little interpretive value.

Genetic classifications of volcanic seismicity according to source mechanism (e.g., Schick, 1981) encourage formulation of hypotheses about the causes and hazards implications of a particular type of seismic unrest. Schick distinguished five source mechanisms--caldera collapse, shear fracturing, shallow explosions, hydraulic pressure surges, and magma flow--on the basis of five categories of seismic signals. These different source mechanisms are undoubtedly the explanation of some differences in volcano seismic signals, but Latter (1979) argues that other differences in volcanic seismic signals are due to modification of signals along their travel paths. B-type earthquakes in particular might have relatively low wave frequencies and poorly defined S waves because they occur in weak (partially molten?) rock in which high-frequency energy is dissipated rapidly. Malone (1983) describes some evidence in favor of different source mechanisms and some evidence that differences in earthquakes arise from path effects. Because it is not always possible to distinguish source and path effects, any genetic classification will necessarily incorporate some uncertainty of interpretation.

Seismic instrumentation:

Ongoing improvements in the technology of seismometers, telemetry systems, and data processing systems render any discussion of seismic instrumentation obsolete within a short time. General principles of volcano seismic instrumentation, including the design of volcano seismic networks, are given in Shimozuru (1971); more recent technical developments are discussed by Lee and Stewart (1981). Briefly, most modern seismic monitoring utilizes electromagnetic seismometers, in which ground vibration is translated into electrical currents. These electrical signals can be recorded onsite or sent by cable or radio telemetry to a central recording station. Signals can be sent directly, in analog form, or can be converted to digital form. At the receiving station, analog signals can be converted to digital signals or vice versa, depending on how the signal is to be used. Analog signals are better than digital signals for a real-time, visual monitoring of seismic activity, and analog instrumentation requires only basic electronic expertise. Digital signals can be transmitted longer distances with less interference than can analog signals, and can also be fed directly into computers for instantaneous location of hypocenters and determination of magnitudes. Digital seismometry usually requires extra signal conversion and an added level of electronic expertise.

In order to obtain accurate epicentral and hypocentral locations, a volcano seismic network should include at least 4 and preferably 6 or more stations within a 10-20 km radius of the vent, timed to within 0.01 second of each other. At least one station should be located near the vent to detect small earthquakes that cannot be detected at the foot of the volcano. At the present time, only a few volcanoes in developing countries have such networks; most volcanoes have lesser networks, occasional monitoring by portable seismographs, or no seismic monitoring at all. When resources are limited, a single, well located, carefully monitored instrument can provide remarkably good information about seismic activity of a volcano. A careful and experienced operator with a single seismograph can obtain daily counts of various types of seismic events, some estimate of the magnitudes of those events and thus their energy release, and a record of S-P differences to show distance ("depth") to the inferred sources.

When less-than-ideal networks are used to locate epicenters or hypocenters, the volcanologist should guard against two common biases in seismic data. The first bias arises in the use of tripartite networks in which three seismic stations are located within a kilometer or so of each other on one side of the volcano. Such an arrangement gives relatively poor resolution, sometimes creating the impression of a wide scatter of epicenters, or of epicenters concentrated on the near or far side of the summit relative to the location of the tripartite network. The wide scatter occurs because closely spaced stations of a tripartite cannot "focus" on the correct epicenters; an apparent concentration of epicenters on the near or far side of the summit will arise if the seismic velocity assumption is incorrect. It is difficult to obtain accurate epicenters using a tripartite network as described (Ward and Gregerson, 1973). The second bias arises with poor timing of stations (as is particularly common with stations that are not telemetered to a central recording site). An error in timing has the same effect as an incorrect assumption about seismic wave velocities in various directions from the vent--it creates a systematic bias to one side of the summit or another. It is advisable to calibrate a new volcano seismic network using man-made explosions at known locations, e.g., quarry blasts, road construction blasts, or blasts set off specifically for the purpose of such calibration.

In order to facilitate comparison with seismic energy release at other volcanoes, it is also necessary to calibrate a new volcano seismic network with earthquakes of known magnitude. This can be accomplished by operating a portable seismograph with known response characteristics, and that has already been calibrated elsewhere, alongside the new network for a period of several weeks or months. Alternatively, one can use a seismograph with "standard electronics," and compute Wood-Anderson equivalent magnitudes. It is advisable to check either method of calibration by comparing the magnitudes of some large tectonic earthquakes, as determined on the Worldwide Seismic Network (WWSN) or a standard Wood-Anderson optical record, to the magnitudes determined on the volcano seismic network (Koyanagi and others, 1977, 1978). Once a network has been calibrated, magnitudes of further volcanic earthquakes can be approximated from their coda lengths (Tsumura, 1967; Lee and others, 1972; Herrmann, 1975).

Spatial and temporal patterns of volcanic seismicity:

Accurate location of earthquake hypocenters (foci) permits delineation of fault planes, zones of high strain rates at the leading edge of a magma intrusion, and aseismic volumes that may reflect rock (or magma) that is too hot to undergo brittle fracture (figs. 3a, 3b). Rarely, hypocenters migrate toward the surface at a discernable rate, permitting a projection of the dates of arrival at the surface, e.g., at Kilauea in 1959 (Eaton and Murata, 1960) or at Plosky Tolbachik in 1975 (Fedotov and others, 1980). A rate of migration can also suggest whether magma is fluid or viscous--and thus less or more likely to erupt explosively.

Accurate locations of earthquake epicenters can suggest the likely vent for an impending eruption. Slight shifts of vent location near the summit of a volcano can shift volcanic hazards strongly to one side or another of the volcano. Thus in cases of summit activity it is important to determine epicenters to within a few hundred meters or less. Occasionally, where potential vents are widely separated and earthquakes are strong enough to be felt by local residents, an iso-intensity map can point to the likely vent. In this way, the writer and Indonesian colleagues identified which among many vents in the Dieng Plateau was the source of earthquakes in late 1981; that location was subsequently

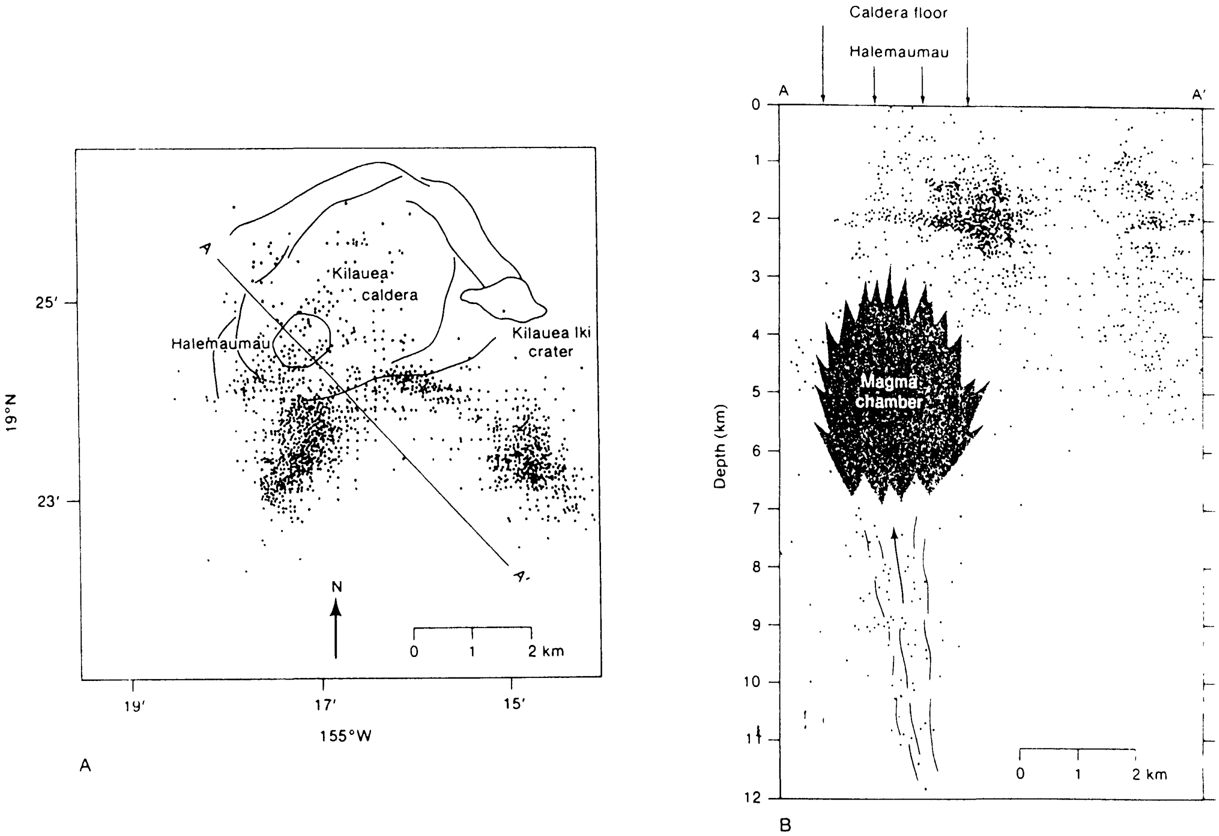


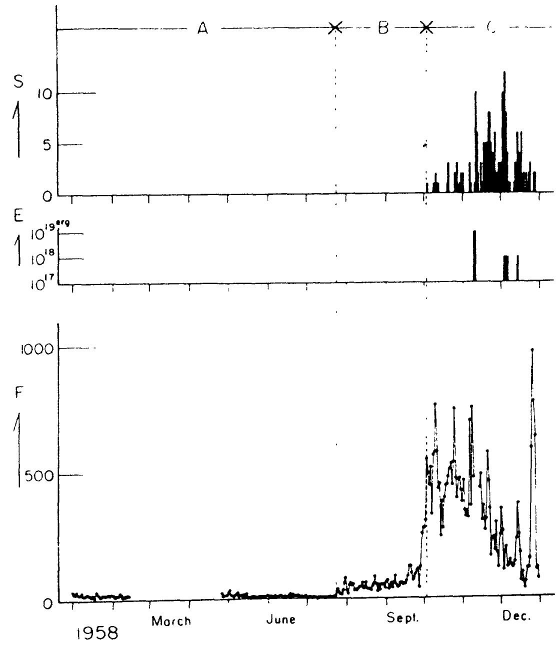
Fig. 3 Earthquake epicenters (A) and hypocenters (B) at Kilauea Volcano, in selected periods of 1971-1973. Hypocenters are along the E rift zone and near the SW rift zone, zones of dike intrusion. Hypocenters are rare in the area interpreted to be a shallow magma reservoir (shaded). From Decker and Decker (1981), after Koyanagi and others (1976).

confirmed with a seismic array, but no eruption occurred.

Several seismic parameters can be examined through time, in relation to the onset of eruptions. The simplest parameter is the number of countable earthquakes per unit time. Minakami (1960, 1974) derived an empirical formula for predicting eruptions of Asama based on a five-day frequency of B-type earthquakes (figs. 4a, 4b). More than about 700 B-type earthquakes in 5 days suggests that an eruption is very likely at that volcano; similar empirical formulae could be derived for other volcanoes that erupt frequently. Shimozuru (1971) and Kieffer (1984) have noted what may be another useful indicator of an impending eruption. At some volcanoes and geysers, seismicity has been observed to decrease shortly <u>before</u> eruptions. One possible physical explanation is that magma has risen so close to the surface that any further brittle fracturing will not be detectable in the 0.5-30 Hz range typically monitored at volcanoes. Kieffer suggests an alternative mechanism: that shallow seismic events just before eruptions may reflect bubble explosion or implosion, and that vesiculating magma near the surface may become acoustically de-coupled from the walls of the conduit, so that continued "bubblequakes" are not transmitted beyond the magma itself. Whatever the mechanism, a sudden decline in seismicity after a long increase can be a sign that an eruption might begin shortly.

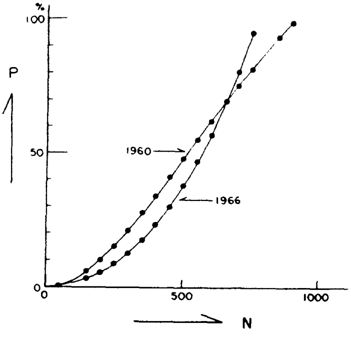
A related approach is to consider the ratios of various types of earthquakes through time, e.g., the ratio of A-type earthquakes to B-type earthquakes, or of high frequency to low frequency earthquakes. Several workers have reported a shift from high frequency earthquakes to low frequency earthquakes as an eruption nears, making the ratio of these two a sensitive prediction tool (e.g., Minakami, 1960, 1974; Malone (1983) (fig. 5).

(a)



A series of B-type quakes preceding explosive eruptions of Asama Volcano. A = quiescent stage of Asama; B = pre-volcanic or pre-eruptive stage; C = stage of explosive eruptions; S = daily frequency of eruption; E = eruptions larger than 10^{18} erg in kinetic energy; F = daily frequency of B-quakes originating in Asama.

(b)



The empirical formulae for predicting the Asama eruptions based on the five-day frequencies of B-type quakes, which were derived in 1960 and in 1966. P = probability of eruptions in the next five days; N = seismic frequency of past five days.

Fig. 4 B-type earthquakes and their relation to explosive eruptions of Asama Volcano (from Minakami, 1960, 1974).

A way to consider the magnitudes and frequencies of earthquakes together is to plot cumulative seismic energy release through time (Tokarev, 1963) (fig. 6; also fig. 18a). The slope and curvature of seismic E release curves have proved to be very useful predictive tools at Kamchatkan volcanoes (Tokarev, 1963, 1971) and at Mount St. Helens (Malone and others, 1981, 1983). In a plot of seismic energy release vs. time, the slope of a tangent to the curve reflects the rate of seismic E release at that given time, and the curvature (second derivative) reflects any acceleration or deceleration of seismic energy release. Typically, the slope and curvature increase exponentially as an eruption nears, and a comparison of a current energy release curve to energy release curves that culminated in eruptions can give a remarkably accurate estimate of when an eruption is likely to begin.

The maximum rate and acceleration of seismic energy release might also help to forecast the explosive or non-explosive character of an eruption. One working hypothesis at Mount St. Helens is that high rates and accelerations of seismic energy release for periods of a few days or less correlate with explosive eruptions, whereas slower seismic energy release over several days or several weeks correlates with dominantly non-explosive eruptions. A possible physical explanation of such a relationship is that gas-rich magma with a high explosive potential can vesiculate and rise more rapidly than can a gas-poor magma with less explosive potential; rapid vesiculation and rise could cause shorter, more intense buildups of pre-eruption seismicity.

Finally, one can plot several different seismic parameters on a single graph, to show at a glance the relation between daily frequency

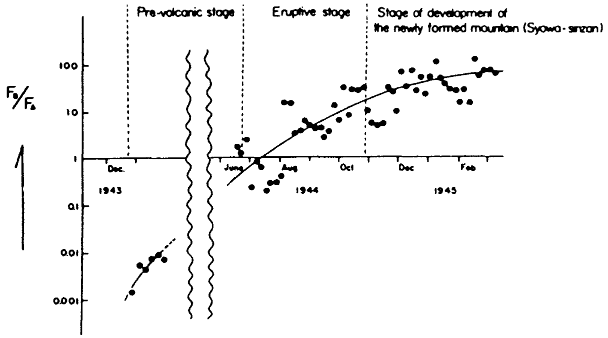


Fig. 5 Changing ratio of B-type to A-type earthquakes, Usu Volcano, 1943-1945. From Minakami and others (1951); Minakami (1974).

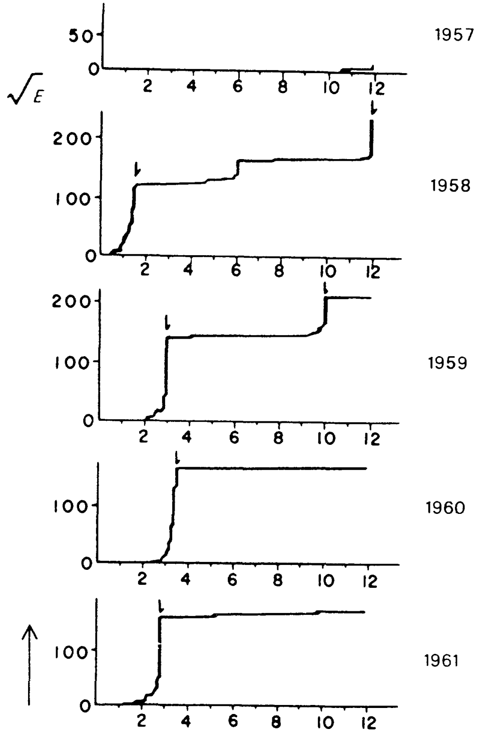


Fig. 6. Seismic precursors to eruptions, Bezymiannny Volcano, 1957-1961, from Tokarev (1963). Abscissa =time, in months; ordinate =seismic strain, or square root of seismic energy release, x 10^7 ergs$^{1/2}$.

and ratios of various types of events, magnitude-frequency relations, and energy release (c.f., Endo and others, 1981) (Fig. 7). In the example of Mount St. Helens from Endo and others (1981), the daily frequency of earthquakes declined in April and early May 1980, but the average magnitude of earthquakes increased, so that the rate of energy release changed (declined) only slightly. This shift toward larger earthquakes may have reflected increasing gravitational instability. In contrast, the number of earthquakes per month in current unrest at Rabaul is increasing but as of the time of this writing (February 1984) the average magnitude seems to have decreased slightly (McKee and others, 1984 and in contributions to the SEAN Bulletin, 1983-84). At least two explanations of such a pattern are possible: rising magma might have "completed" the most difficult fracturing and is now rising more easily near the surface, or heating of groundwater has increased pore pressures and is causing strain to be released in frequent small increments rather than larger, less frequent increments. Plots of several seismic parameters together can be sensitive and informative indicators of changing seismicity.

(2) Ground deformation

Injection of magma beneath or into a volcanic edifice usually results in a small but detectable inflation of that edifice (Decker and Kinoshita, 1971); such inflation can be temporary or permanent. A relaxation of stresses, either by eruptions or by migration of magma into lateral fissures, can result in broad deflation of the edifice. The area of deformation is a function of the size, shape, and depth of the magma body; the magnitude of a measured deformation is a function of these same variables, change in the volume of the magma body, and the location of

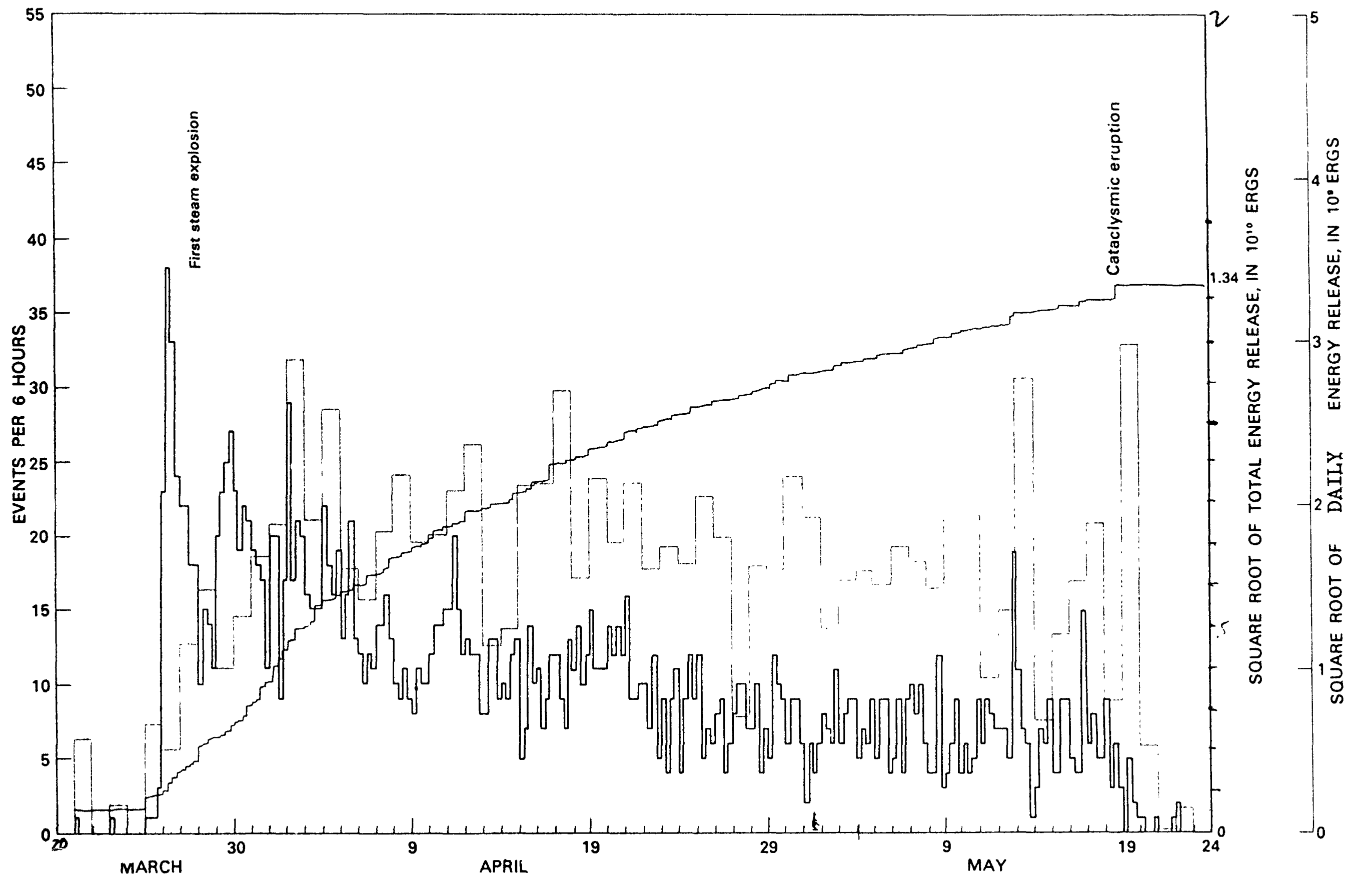


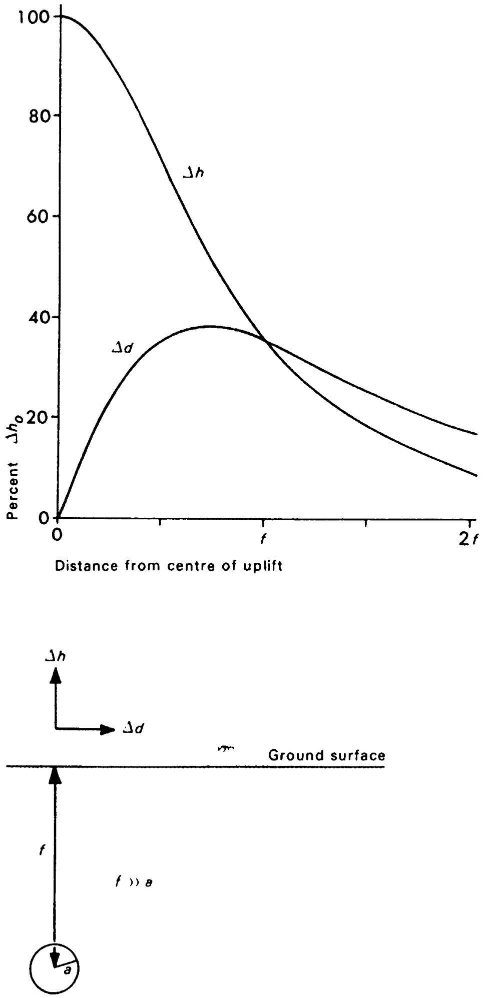
Fig. 7 Counts of earthquakes larger than M=2.5 (dark histograms); daily seismic energy release (light histograms); and square root of cumulative seismic energy release (curving line), from March 20 to May 24, 1980 (GMT) at Mount St. Helens. Earthquake frequency peaked on March 26, but energy release did not peak until April 2, reflecting larger earthquakes after March 27. Energy release declined more slowly than the number of earthquakes per day, again reflecting large earthquakes later in the sequence. From Endo and others (1981).

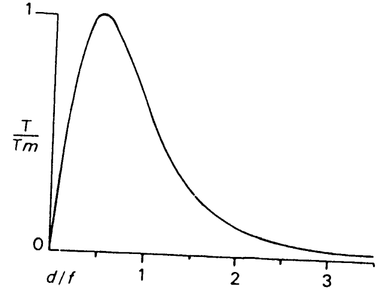
the measuring station relative to the magma body. A measured rate of deformation is a function of all of the above and the rate of magma injection.

Formulas that relate the depth and diameter of a spherical pressure source (magma body) to the pattern of surface uplift, tilt, and horizontal strain are given by Mogi (1958), Eaton (1962), and Decker and Kinoshita (1971) (fig. 8). The depth of a pressure source is approximately 2/3 of the half-width of uplift. (Note: "half-width" is the diameter of a contour of uplift along which uplift equals 1/2 of the maximum uplift; half-width can be measured from contours of uplift, as in Figure 9a, or from a plot of uplift vs. distance from the apex of uplift, as in Figure 9b. If a plot like Figure 9b is used, r, i.e. HW/2, must be doubled to obtain the half-width.)

Non-spherical bodies create different patterns of surface deformation, e.g., a graben directly above a dike, flanked by parallel ridges of uplift (Dieterich and Decker, 1975; Pollard and others, 1983). Deformation patterns are also controlled in part by regional stresses.

Although magmatic intrusions are the most common cause of volcanic ground deformation, one should not automatically assume that all ground deformation is of this origin. Other possible causes include: in-situ expansion of magma due to heating or vesiculation, heating and increased pore pressures of groundwater, thermal expansion of country rock, regional tectonic stresses, and incipient gravitational failure.





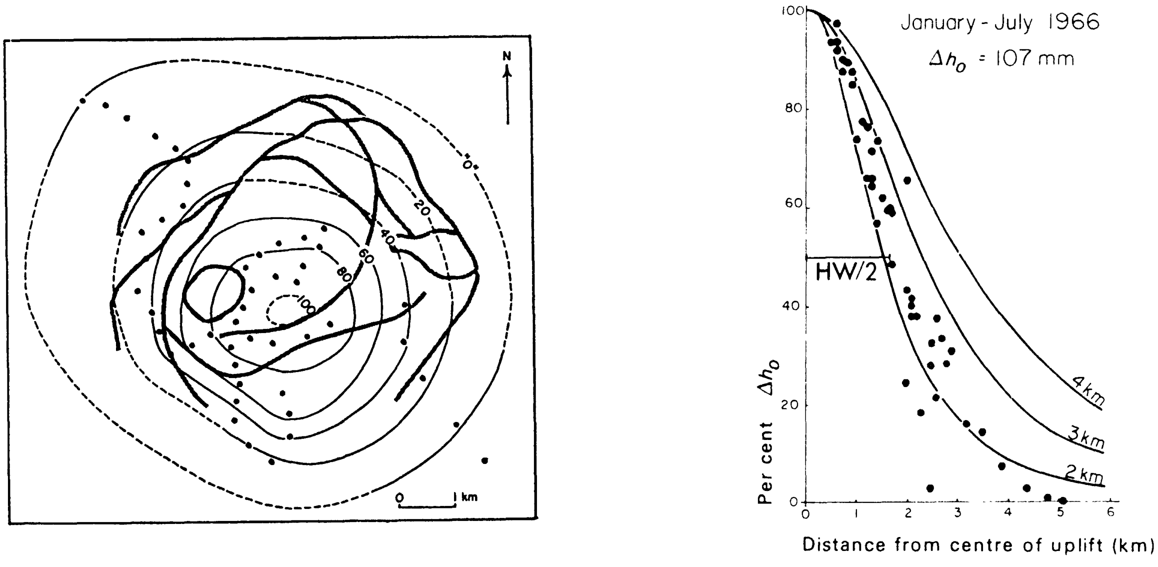
$$T = \frac{\partial \Delta h}{\partial d} \quad T = Tm \text{ at } d = f/2$$

$$\frac{T}{Tm} = 2.5 \frac{d/f}{(1 + d^2/f^2)^{5/2}}$$

$$\Delta h_o = -1.16\, f\, Tm$$

$$\Delta V = \Delta h_o 2\pi f^2$$

Fig. 8 Relations between uplift (Δ h), horizontal displacement (Δ d), tilt (T), depth to pressure source (f), horizontal distance from the apex of uplift (d), and volume of deformation change (Δ V). h_o =maximum uplift; Tm =maximum tilt. A point source of uplift ($f \gg a$) is assumed. From Decker and Kinoshita (1971), after Mogi (1958) and Eaton (1962).



(b)

Fig. 9a Contours of uplift (in mm) at Kilauea Volcano, January-July 1966 (from Fiske and Kinoshita (1969). Heavy lines =major caldera faults.

9b Plot of percent of maximum uplift ($\Delta h/\Delta h_o$ x 100) vs. distance from the apex of uplift, for Kilauea during the same time period. Curves labeled 2, 3 and 4 km are predicted uplift for pressure sources at these depths; the dots are observed uplift, suggesting a pressure source at slightly greater than 2 km depth. HW/2 = half of the half-width of uplift. After Fiske and Kinoshita, 1969).

Various measurements of ground deformation:

a. Precise levelling, to measure uplift and subsidence. Levelling routes can be established radially away from a volcano, or around a volcano, or ideally both. Contours of uplift or subsidence can be drawn (e.g., fig. 9a). Typical pre-eruption uplift at large volcanic centers is in the order of ten centimeters to a few meters; that at smaller stratovolcanoes is a few centimeters. The advantages of precise levelling are its precision and a contour map view of uplift or subsidence; the principal disadvantages of levelling are that it is time-consuming and expensive, and difficult to repeat in winter where snow covers the benchmarks. Also, during very rapid uplift, it may prove impossible to "close" a level loop, because the point of origin and points along the loop are rising even during the survey. Modern spirit levels and temperature-stable invar rods permit first-order levelling with measuring errors of about 1 mm in a kilometer. But where changes are as much as several cm in a kilometer, levelling to first-order standards is unnecessarily precise and levelling to third-order standards may be adequate. In fact, if changes are easily detectable using third-order standards, and if it is possible to repeat the levelling surveys more frequently than would be possible to first-order standards, the data from repeated less precise surveys can be more useful than data from a single precise survey. This must be judged for each individual case, considering the magnitude of expectable changes and whether rates of uplift might be changing rapidly. First-order levelling is not always better than third-order levelling.

Where a body of water exists near or on the volcanic edifice, uplift and subsidence can be measured with one or more tide gauges (Gutenberg, 1933; Tsuboi, 1937; Lipman and others, 1981; Otway, 1984, p. 47-48).

Data from tide gauges can be used to fill out a time history of uplift or subsidence, between infrequent levelling surveys. Changes relative to sea level may be considered absolute changes; changes around the shoreline of an inland lake are relative only to each other unless levelling or other data ties the lake "network" to some fixed elevation. Some information can also be gained even in the absence of a tide gauge, using any simple measuring rod and marks upon the shoreline, but care must be taken to damp out waves and to correct for subtle but ever-present seiches.

b. Electronic-distance measurement (EDM): Pre-eruption inflation of a volcanic edifice results in horizontal as well as vertical strain. Horizontal strain is best measured with an electronic-distance-measuring instrument ("EDM") which sends a beam of visible or infrared light from the instrument to a reflector. The time required for the light to travel to the reflector and back to the instrument indicates the distance. Strictly speaking, measurements are made of slope distance, but because most lines are nearly horizontal, changes are loosely regarded as horizontal strain. If corrections are made for temperature, pressure, and water vapor along the light path, changes of less than 1 ppm (=1 microstrain or 1 mm in 1 km) can be detected (Bomford, 1980). Contraction of lines between the volcano and distant, stable locations suggest inflation of the cone; extension of lines between two points on a volcanic edifice also suggest inflation. Pre-eruption horizontal strains of 10^{-3} to 10^{-5} have been observed (i.e., between 1 cm and 1 m per km of line length) (fig. 10A-C; also fig. 18B). EDM measurements are much faster than levelling, require less manpower and, when strains approach 10^{-4} (10 cm of change in 1 km of line length), can be made with temperature measurements only at the instrument end of lines, and

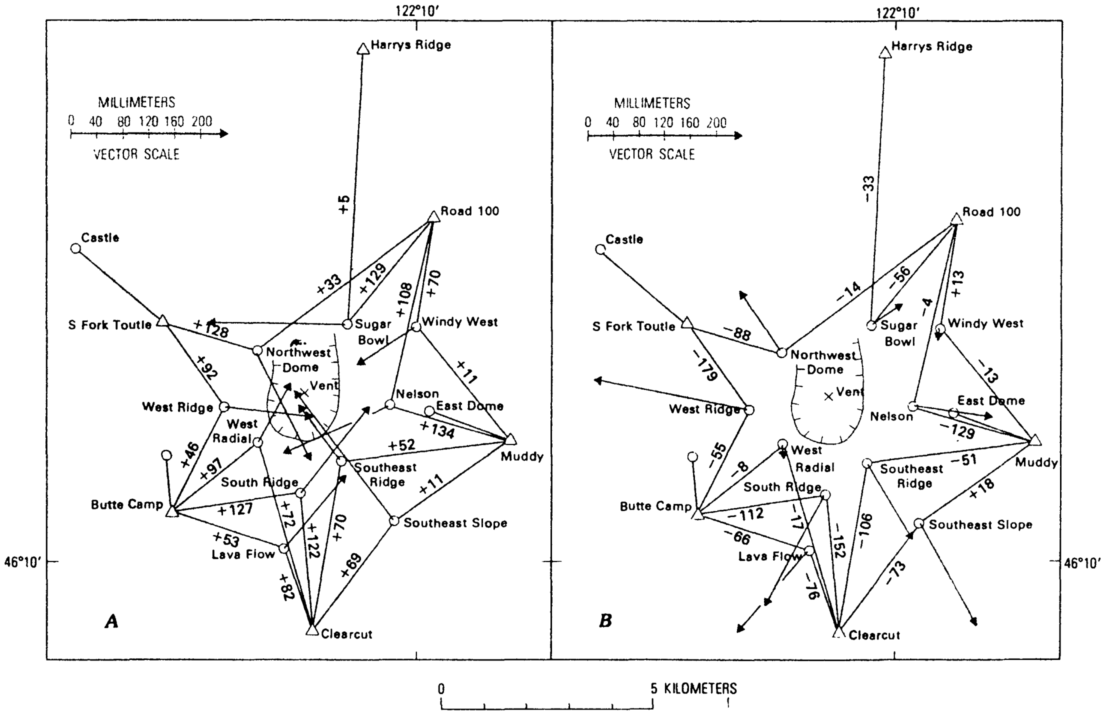


Fig. 10 Cumulative changes (in mm) in slope distances at Mount St. Helens, during periods of (A) contraction (deflation) and (B) expansion (inflation), June-September 1980. Instrument stations (triangles) are around the foot of Mount St. Helens; reflector targets (circles) are on the volcano's upper slopes. Heavy lines with numbers = measured lines and changes (in mm). Light arrows = calculated displacement vectors (scale in upper left of each figure). Periods of contraction = June 8-July 13, July 21-August 4, August 8-September 3, 1980. Periods of expansion, immediately preceding or bracketing magmatic explosions or vigorous gas release = June 8-18, July 13-21, August 4-8, and September 3-24, 1980. From Swanson and others (1981).

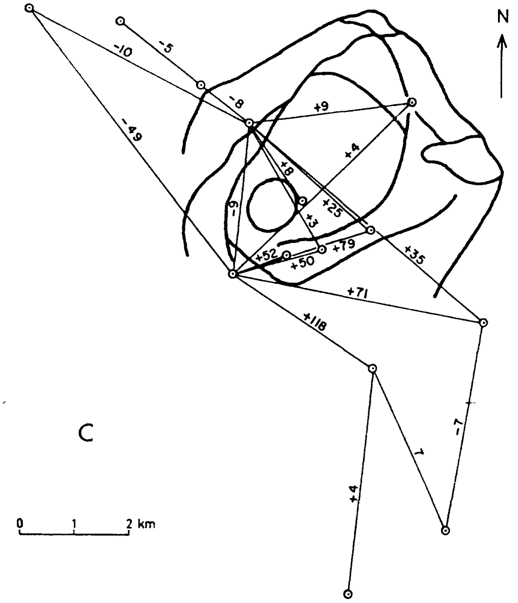


Fig. 10C Cumulative change in length (in mm) of twenty lines at Kilauea Volcano, January 6-February 21, 1967. Heavy lines = main caldera faults. The EDM network at Kilauea emphasizes lines that cross the summit caldera and rift zones, in contrast to the network at Mount St. Helens (a stratovolcano) that emphasizes lines from the base of the cone to points high on the cone. From Fiske and Kinoshita (1969).

permanent reflectors at the opposite ends of the lines. Disadvantages of EDM measurements are that the equipment is expensive, and good weather is required along the entire line of sight.

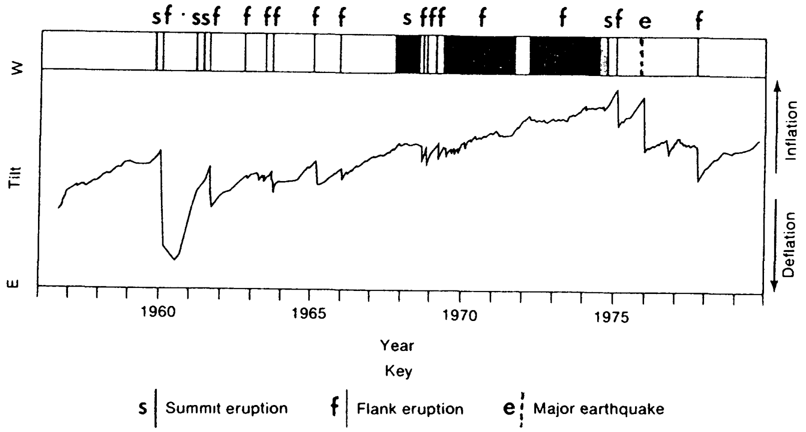
c. Tilt measurements. Points on the flanks of volcanoes commonly show pre-eruption tilts. Tilts of several tens to several hundred microradians are common before eruptions; rarely, local tilts of several thousand microradians occur. Most tilt in volcanic areas has been measured with water-tube tiltmeters (Eaton, 1959), or by precise levelling in small, short-sided arrays of benchmarks ("dry tilt" measurements) (Kinoshita and others, 1974; Yamashita, 1981). Both techniques are widely used. Advantages of water-tube tiltmeters are that they are relatively simple to construct and provide a continuous record; disadvantages are that the quality of data will be poor unless the tiltmeter is in an underground, constant-temperature vault with a long baseline. Advantages of the dry tilt technique are low cost and relative ease of measurements; disadvantages are that the measurements are not as precise as from other tilt methods, and are not continuous.

Recent advances in microelectronics have brought simple, relatively inexpensive electronic bubble tiltmeters (Westphal and others, 1983) that can replace or supplement existing tilt measurements. Three advantages of electronic tiltmeters are that they provide a continuous record of tilt, have a high (adjustable) dynamic range, can be installed high on the slopes of a volcano where tilts are likely to be most pronounced, with data telemetered back to an observatory; disadvantages are that they are subject to drift, other electronic problems, and site instability, and therefore require periodic calibration with other methods.

Use of temporal patterns of tilt to predict eruptions is described in Decker and Kinoshita (1971), Dzurisin and others (1983), and Klein (1984) (e.g., fig. 11a, 11b). Rates and acceleration of tilt are generally more useful than absolute values of tilt. In general, outward edifice-wide tilt occurs at a constant or increasing rate until an eruption begins, but tilt within a few hundred meters of a vent may reverse shortly before an eruption. The latter pattern has been observed inside the crater of Mount St. Helens, where tilt may have reversed as the dome broke apart and ceased to act as a cap, or as new magma gravitationally loaded the dome (Dzurisin and others, 1983).

Tilt measurements will be most reliable if they are made at several locations on a volcano. Any single station may suffer from local site instability, or inertness to deformation, but several stations should show systematic changes if any significant tilt is occurring. Noise in data from a tiltmeter is minimized if the tiltmeter is kept in an underground, constant-temperature vault. Tiltmeters on several sides of the volcano, at roughly equal distances from the summit, can detect any asymmetry in inflation; tiltmeters at various distances along a radial line from the volcano can indicate the depth of the pressure source (fig. 9). Maximum tilt will occur at a horizontal distance from the center of inflation (usually from the summit of a volcano) equal to about one-half the depth of the magma beneath the surface. For example, maximum tilt due to magma 2 km below the summit of a volcano will occur 1 km (map distance) from the summit--which is on the upper slopes of most volcanoes. Therefore, at least one tiltmeter should be located on the upper flanks of a volcano to detect tilts due to magma at shallow depths.

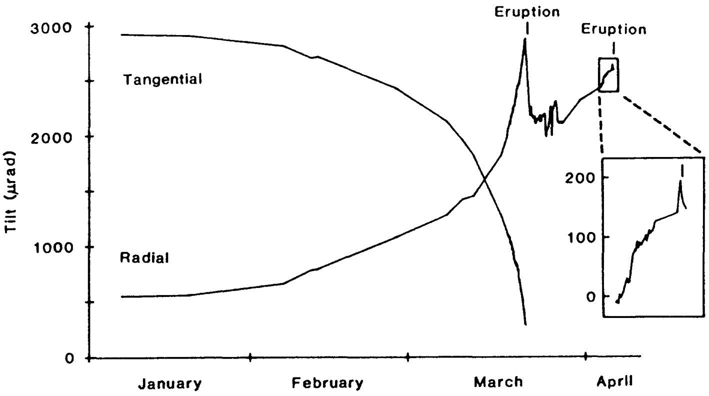
d. Radial cracks, thrust faults, and other surface rupture. Ground deformation may include fractures at the surface that can be measured with a standard surveyor's steel tape, or with an inexpensive, continuously-recording extensometer (Duffield and Burford, 1973; Tilling, A



Twenty-year tilt record of Kilauea Volcano in Hawaii shows the ups and downs related to eruptions. For months to years before an eruption, the summit of Kilauea inflates. When it reaches a level that exceeds the strength of the rocks surrounding the shallow magma chamber, the volcano erupts. Some eruptions rapidly remove large volumes of magma from the summit chamber, and the volcano deflates over a period of days. A high level of inflation indicates a higher probability of eruption, but since the strength of the rocks surrounding the magma chamber varies from one eruption to another, no exact level of tilt can be used to predict the next eruption. (Data from the U.S. Geological Survey.)

Fig. 11A Tilt at Uwekahuna vault, near the summit of Kilauea Volcano. Full vertical scale = 700 microradians. From Decker and Decker (1981).

B



Tiltmeter data from station ROA for the eruption of March to April 1982. Radial uplift began in mid-January and accelerated sharply on 16 March; rapid subsidence began 30 minutes before the explosive onset of the 19 March eruption. Uplift resumed in late March and then again reversed to subsidence 36 hours before extrusion resumed on 5 April (inset).

Fig. 11B Tilt in the crater of Mount St. Helens. From Dzurisin and others (1983).

1976). Such fractures are most likely in the immediate vicinity of a vent. Inflation that is symmetric about a point can result in radial cracks and concentric thrust faults (Chadwick and others, 1983). Small grabens can develop over intruding dikes, often with many small *en echelon* cracks (Pollard and others, 1983).

The dilation of cracks can be monitored from a distance if they are incandescent. Cracks in the lava dome of Mount St. Helens are being monitored by photographing them at night, using a night-vision device that intensifies images in the visible light range (R.T. Holcomb, pers. commun., 1982). Similar devices are used in military and industrial surveillance applications. Increasing numbers of dimly glowing spots along cracks in successive photographs shows that the cracks dilate systematically in the days before each new eruptive pulse. The most rapid dilation occurs 1 to 3 days before the onset of an eruption. This technique is related to thermal infrared monitoring, described below (#7), but is simpler and uses less expensive equipment. But, because it senses only high-temperature sources (above 500°C), its use is limited to situations in which surficial cracking can reveal hot, glowing material beneath a cooler crust.

Benefits of frequent, diversified monitoring of ground deformation:

In general, the various types of ground deformation occur in concert, all suggesting the same underlying phenomenon of inflation, deflation, or intrusion. In the ideal circumstance in which all types of measurements can be obtained, results from one method will help to constrain interpretations of another. For example, levelling or tilt measurements will show what proportions of change along an EDM line are due to horizontal and vertical motions, respectively. Similarly, levelling data can show whether tilts reflect (a) broad, systematic deformation, (b) broad deformation that is complicated by independent movements of blocks of the volcano, or (c) local site instability. EDM and tilt measurements are sensitive enough to detect most pre-eruption ground deformation and can be repeated frequently; these measurements can be supplemented by levelling surveys at longer intervals. Surface fractures are helpful when they occur and if one can visit a vent area frequently; changes that can be visually observed and whose spatial pattern can be seen without recourse to models are inherently more convincing and interpretable to geologists!

The frequency with which ground-deformation measurements should be made depends on the rate at which changes are known to occur, and whether there is any other indication (e.g., seismic) of unrest. Some ground deformation occurs very slowly, over years, decades, or longer. In such cases, measurements every month or year may be sufficient. During rapid deformation, tilt and EDM measurements should be made daily or even more frequently. A rule of thumb is that measurements should be at least 2 times and preferably 3 to 5 times as frequent as the time over which a significant change might be expected to occur. If measurements are less frequent, they are likely to miss important changes or be inadequate to distinguish between noise and significant change. As an eruption nears, rates of deformation commonly increase (cf. Swanson and others, 1983; Chadwick and others, 1983), so the frequency of measurement should also increase. A continuous electronic tilt, crack-widening, or tide gauge record can be a helpful indicator of the instantaneous, real-time rate of ground deformation and of any fluctuation in that rate.

A temporal pattern of ground deformation can indicate the likely date of an eruption, and a spatial pattern can indicate the depth and scale of the source of deformation. The spatial pattern of deformation can also suggest the most likely vent area. Temporal and spatial patterns together, showing a rate of migration of the depth to the source of deformation, might suggest whether an intruding magma is fluid or viscous.

3. Gravity measurements: The local acceleration of gravity is a function of elevation (distance from the center of gravity of the earth) and of density of rocks near the surface of the earth. Changes in gravitational field strength are usually a combination of an elevation (or free-air) effect and a density (or Bouguer) effect. In a volcanic setting, gravity changes can result from: uplift or subsidence (i.e., elevation changes), density changes in near-surface rock caused by magmatic intrusion or changing groundwater level, or (most commonly) a combination of the two. The application of gravity measurements to volcano monitoring is ably discussed by Yokoyama (1971, 1974), so the following remarks are only a brief supplement to his papers.

The magnitude of the gravity effect caused solely by uplift is about -3 microgals/cm. (Note: the worldwide average of the earth's gravitational field strength is about 980 gals, so 1 microgal is roughly one-billionth of the total field!). In other words, raising a gravity meter by 1 cm will result in a 3 microgal decrease in the measured field strength. However, if uplift is caused by the addition of new material below the gravity station (e.g., magmatic intrusion), the resulting mass increase will cause a corresponding gravity increase of about +1 microgal/cm. This value depends on the density of the material added, but 1 microgal/cm is representative for typical magma densities.

If we assume that most uplifts at volcanoes are accompanied by magma movements, the combined elevation and density effects will give rise to a net change of about -2 microgals/cm. This relationship is sufficiently constant that gravity surveys can be a crude substitute for more time-consuming levelling surveys, <u>if</u> vertical deformation exceeds about 5 cm and if gravity can be measured to the nearest 5 microgals. In the case of an island within a volcanic crater lake, gravity surveys may be the only feasible way of measuring absolute elevation changes relative to a stable base far from the lake.

However, at some volcanoes the relationship between gravity and elevation changes is not constant and can differ substantially from the common value of -2 microgals/cm. At Kilauea Volcano, Dzurisin and others (1980) measured values ranging from -1.7 to -6.0 microgals/cm during 1975-1977 (fig. 12). They attributed these changes to the filling and emptying of voids within the volcano as a shallow magma reservoir alternately inflated and deflated. In the extreme case of a "spongy" volcano, magma might move into shallow fractures without causing any surface deformation, so that gravity and elevation changes would be completely unrelated.

Recent advances in the usefulness of gravity surveys are due in large part to improved precision of gravity meters. State-of-the-art gravity meters can detect changes as small as about 5 microgals, if gravity at a stable base station is measured with the same gravity meter shortly before and after measurements at the points in question. Corrections must be made for the effect of earth tides. Drift is a significant problem with gravity meters, so it is critical to refer all measurements to values measured at a base station before, after, and sometimes even during a gravity survey. Drift is also the principal reason that continuous-reading gravity meters have not been widely used as monitoring devices, but a new cryogenic gravimeter has recently been operated with very low drift rates and high precision.

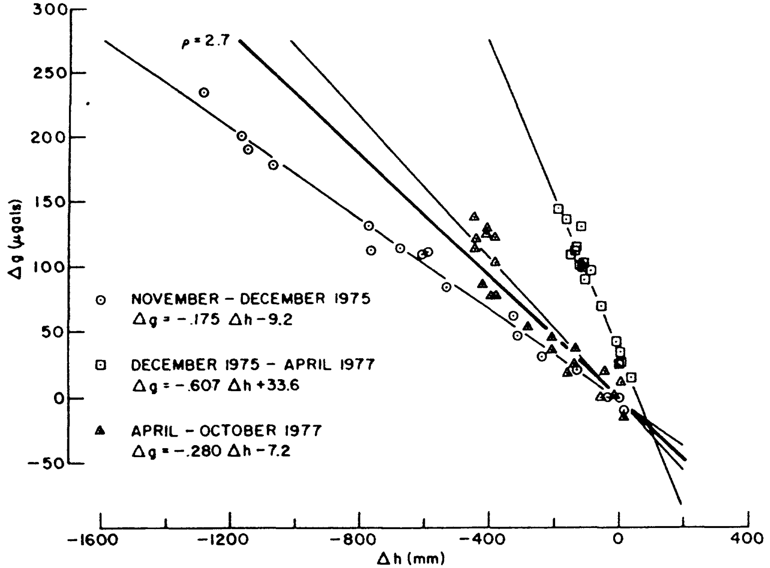


Fig. 12 Plot of gravity and elevation change in the summit region of Kilauea Volcano. Circles = surveys before and after November 1975 earthquake; squares = before and after an east rift zone intrusive sequence; triangles = before and after the September 1977 eruption. Equations are of lines fit to the data. Variability in the slopes (from −1.7 to −6.0 microgals/cm) suggests a sponge-like magma reservoir that can fill and empty with less uplift or subsidence than would be expected if the magma reservoir had no void space. The line labeled =2.7 is the theoretical relation of elevation and gravity changes (for magma of density =2.7 g/cm^3); that line has a slope of −2.33 microgals/cm, or approximately −2 microgals/cm, as noted at many volcanoes. From Dzurisin and others (1980).

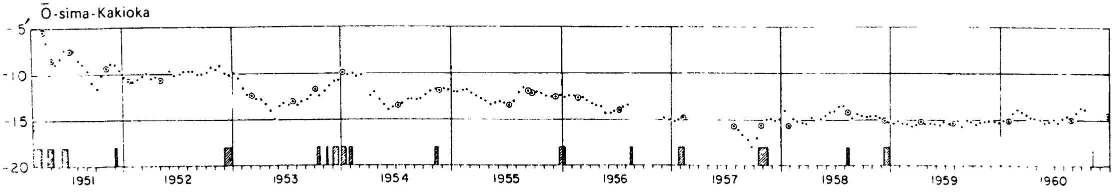
4. Magnetic field measurements: Changes in the total magnetic field strength, or in one component relative to another, can occur as a result of (a) magmatic intrusion, or (b) stress in country rock. Magma is invariably above the Curie temperature of common magnetic minerals, so an intrusion by itself acts to decrease total field strength. The intrusion may also demagnetize country rock, with more variable effect on the magnetic field. Most young volcanoes consist largely or wholly of normally magnetized rock, so demagnetization will result in a decrease in total field strength, but demagnetization of reversely magnetized country rock may result in an increase in total field strength, an increase in the north-directed component of the magnetic field, and a change in the shape of a magnetic anomaly associated with that volcano. Magnetic changes due to heating are relatively slow, limited by the rates of conductive and convective heat transfer.

More rapid changes in the strength and direction of the magnetic field can be caused by stress (Stacey and others, 1965; Yukutake and Tachinaka, 1967; Johnston and Stacey, 1969; Davis and others, 1974, 1979). The precise effect depends on the orientation of the stress axes and on the remanent magnetization of the volcanic edifice. Empirical observations of declination and inclination at volcanoes before eruptions suggests that the magnitude of the stress effect is small compared to that due to thermal demagnetization, but the stress effect usually occurs at a faster rate. Several additional mechanisms for changing the magnetic field at volcanoes are discussed by Johnston and others (1981).

Magnetic field strength and direction is typically measured at one or more sites on a volcano and one site far from the volcano (the abovementioned references, plus Rikitake, 1951; Rikitake and Yokoyama, 1955; Yokoyama, 1969, 1971; Robach, 1983). By considering differences between stations, one can eliminate fluctuations due to non-volcanic causes (e.g., magnetic storms) (figs. 13).

5. Electrical changes: Anomalous stresses and heating of rock and groundwater can cause weak electrical currents or changes in resistivity. Some of these changes may be strictly electrical; others may be electromagnetic effects due to changes in the local magnetic field. Two parameters, electrical self-potential and the behavior of very low-frequency electrical signals from distant sources, have been shown to vary before eruptions of Kilauea Volcano. Electrical and electromagnetic monitoring is largely experimental; the theory and techniques of such monitoring are described by Yokoyama (1971), Anderson and Johnson (1976), Zablocki (1976, 1978), Towle (1983), and Halbwachs (1983).

Electrical self-potential measurements have been most useful in mapping thermal areas on active volcanoes, and in detecting changes in the activity of such areas. Negative ions in groundwater diffuse through rock more slowly than do positive ions, so areas in which warm groundwater rises to the surface have an excess of positive charges. This creates an electrical "self-potential" between survey points and a fixed reference site, that can measured using a high-impedance potentiometer. The magnitude of the self-potential field is typically measured in millivolts, but values greater than 1 volt have been measured at active thermal areas at Kilauea Volcano and Long Valley Caldera. In addition, promising results have been obtained by telemetering data from a permanent self-potential array spanning Kilauea's east rift zone. Self-potential typically increases during magmatic intrusions into the



Variation of the difference of the semi-monthly mean of the westerly declination between Ō-sima and Kakioka. The encircled dots denote the values calibrated by absolute measurements. Hatched columns denote the active periods of Ō-sima Volcano.

Fig. 13. Anomalous changes in magnetic declination, Oshima Volcano, 1951-1960. From Yokoyama (1971).

rift zone, some of which culminate in eruptions; it decreases a few hours to days after an intrusion stops.

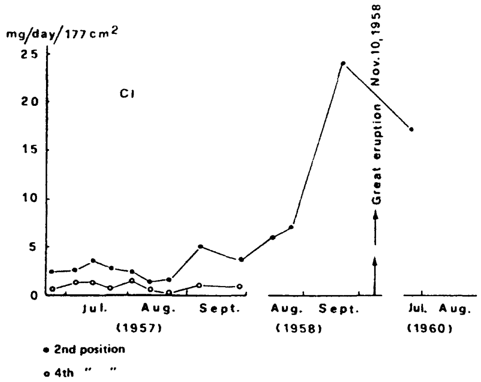
Some radio transmissions to submarines utilize electromagnetic waves with very low frequencies (15-25 kHz). The same waves can serve a completely unrelated but useful purpose in mineral exploration and volcano studies. Normally, the waves travel between the ionosphere and the surface of the earth, but where rocks within about 100 m of the surface are abnormally conductive the waves may dip beneath the surface for a short distance. The VLF waves induce a secondary magnetic field, and measurement of this secondary field helps one to locate conductive strata and to estimate their conductivity. This method has not been used extensively at active volcanoes, but Zablocki (1978) used the technique at Kilauea to: (a) delineate the lateral extent of lava lakes, (b) obtain an indication of the attitude of recent dikes, (c) show that dikes that feed eruptions usually cool more quickly than their strictly intrusive counterparts, (d) show that some fumarolic areas are underlain by highly altered conductive zones, and (e) constrain interpretation of other electromagnetic data. These results suggest that the VLF technique may be a useful monitoring tool at active volcanoes, but its utility in that regard has not yet been demonstrated.

6. Gas monitoring: Physical stress and strain in and beneath the volcanic edifice, and eruptions themselves, are driven by the buoyancy of magma, by gases exsolving from magma, and by gases generated by contact between groundwater and hot rock or magma. The appearance of gases such as H_2O, CO_2, CO, SO_2, H_2S and halogens at the surface of a volcano may be related to degassing of a magma body, hydrothermal activity caused by the interaction of magma with ground water, or both. Monitoring of gas emissions for eruption forecasting examines the chemical composition of gas emissions (volume % of each gas species), the emission rate of gas emissions, and fluctuations in these parameters over periods of months, weeks, days, or even (in the case of continuous monitoring) minutes or seconds.

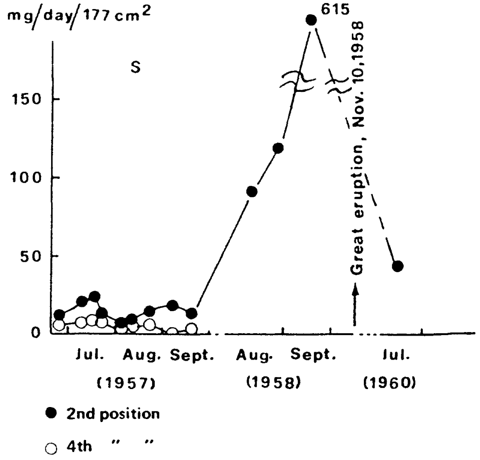
Chemistry of gas emissions:

For more than 150 years chemists have attempted to characterize the chemical composition of volcanic gas emissions. Samples were usually collected in sample vessels, typically evacuated glass bottles or tubes, and analysis were made weeks, months, or even years after collection. Now, with modern methods of gas analysis such as gas chromatography and mass spectroscopy, gas analyses can be made routinely, soon after collection (e.g., Giggenbach, 1975), or even in the field using a field gas chromatograph (LeGuern and others, 1982; LeGuern, 1983). By the early 1970s, the chemical composition of the gas phase was well characterized: the principal gases emitted from volcanoes are H_2O, CO_2, H_2, SO_2, H_2S, and CO, together with a suite of trace gases that include HCl, HF, He, and CH_4.

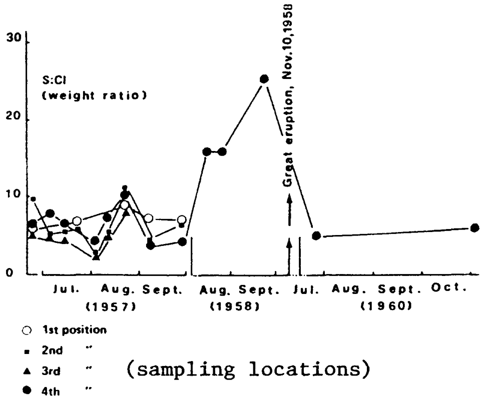
Variation in the composition of the gas phase can be useful for eruption forecasting. Noguchi and Kamiya (1963); Iwasaki and others (1965, 1975); Tazieff (1970, 1971, 1983); Iwasaki (1971); Tonani (1971); Menyailov (1975); and Giggenbach (1983), among others, have noted changes in the composition of fumarolic gases over periods of hours to months before eruptions (figs. 14A-C). In general, such changes are thought to reflect changing saturation and release of various volatiles in magma as that magma rises, cools, or crystallizes. At Kilauea, Thomas and Naughton (1979) concluded that the ratio He/CO_2 tends to increase during quiet periods (when the lightweight He gas can diffuse more easily than CO_2 through cap rock), and to decrease during active periods (when inflation and cracking of the cap rock allows CO_2 and other magmatic volatiles to escape at increased rates). The isotopic ratios of $^{18}O/^{16}O$ and deuterium/hydrogen can indicate the degree to which fumarolic gases are derived from meteoric water; the $^{13}C/^{12}C$ ratio can suggest contributions from organic material or carbonate sediment (Allard, 1983).



a Chlorine absorbed from the atmosphere surrounding Kama-yama, Mount Asama (Japan): its trend in relation to explosion (after Noguchi and Kamiya, 1963).



b Sulphur absorbed, as in a (after Noguchi and Kamiya, 1963).



c Changes in the S:Cl ratio in relation to explosion at Mount Asama (after Noguchi and Kamiya, 1963).

Fig. 14a-c Changes in S, Cl, and S:Cl ratio before an explosive eruption of Mt. Asama in 1958. From Tonani (1971), after Noguchi and Kamiya (1963).

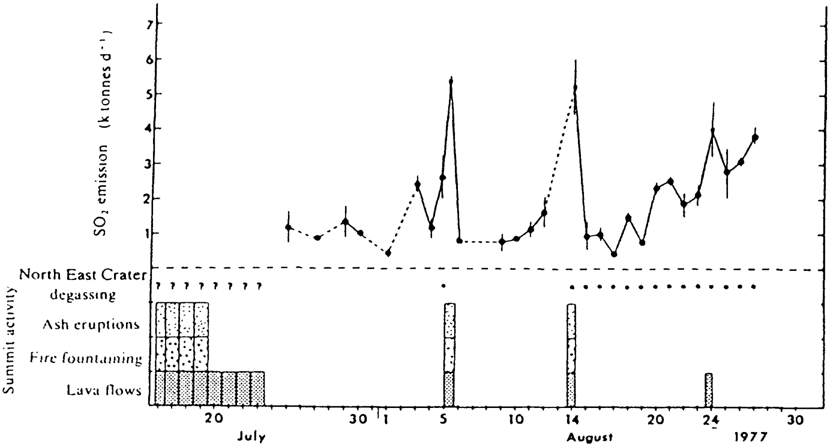
Emission rates:

Fluctuation in the rate of volcanic gas emission is also a useful indicator of volcanic activity. Rates of SO_2 emission have been determined using a correlation spectrometer (COSPEC), developed initially for air-quality monitoring. The COSPEC measures absorption of ultraviolet sunlight by SO_2 in a plume, and thus the concentration of SO_2 in the plume; that concentration times the wind speed gives a flux rate (Stoiber and others, 1983). Early COSPEC studies at volcanoes were pioneered by Stoiber and students; the first use of the COSPEC for an extended period of volcano monitoring was by Malinconico (1979), who made a systematic ground-based measurements of sulfur dioxide emission rates from Etna during a two month period of eruptive activity (fig. 15). Regular ground-based measurements of sulfur dioxide emissions from Kilauea volcano have been made since 1979. Frequent airborne measurements of the volcanic plume from Mount St. Helens have been made since 1980 (Casadevall and others, 1981; 1983) (fig. 16).

Harris and others (1981) modified an infrared spectrophotometer to measure emission rates of carbon dioxide in the plume emanating from the crater at Mount St. Helens. Measurements require direct sampling from an aircraft that is flown along a grid pattern through the plume.

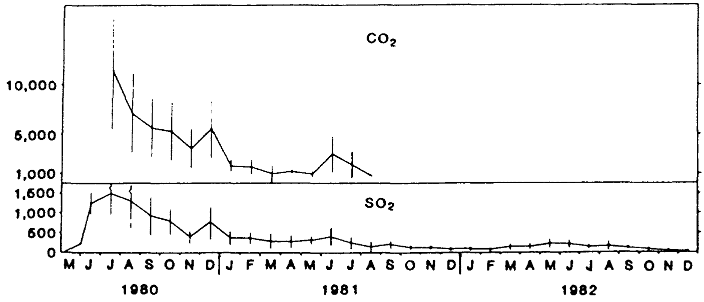
Flux rates of some volcanic gases may also be reflected in changes in water chemistry, and waters are usually easier than gases to sample and analyze. Noguchi and Kamiya (1963) monitored S and Cl emissions by monitoring the absorption of those elements into alkaline solutions that they placed near fumaroles.

Finally, rough estimates of the flux of sulfur and several other gases during explosive eruptions can be made from the amount of leachable sulfate and other salts adsorbed onto ash grains (Rose and others, 1982).



SO$_2$ mass flow averages for the period of study. The dotted lines connecting the SO$_2$ data points are periods with missing days of data. The bars through each data point represent 1 standard deviation of the daily average. The bar graphs represent qualitative estimates of summit activity.

Fig. 15 SO$_2$ emission and eruptions, Mt. Etna. SO$_2$ emission increased by several-fold, several days before eruptions. From Malinconico (1979).



Emission rates of CO$_2$ and SO$_2$ (in metric tons per day) from March 1980 through December 1982. The average daily emission rate was calculated for each month. Thin vertical lines indicate 1 standard deviation.

Fig. 16. CO$_2$ and SO$_2$ emission, Mount St. Helens. SO$_2$ emission was low before and even after the major explosion of May 18, 1980; it peaked in July 1980, and gradually declined to the end of 1982. An interpretation of progressive degassing of the magma is consistent with progressively less explosive activity since 1980. From Casadevall and others (1981).

Continuous monitoring:

Periodic measurements of gas composition and flux rate, even if made frequently, may fail to show short-term fluctuations that are critical for short-term eruption forecasts. We need some continuous, real-time monitors of gas composition and flux to complement continuous, real-time monitoring of seismicity, tilt, and other geophysical parameters. Ideally, the total gas composition could be continuously monitored, but this is not yet practical, so a few relatively abundant and non-reactive gases (H_2, He, and CO_2) have been the focal point of field experimentation (Gantes and others, 1983). H_2 concentrations have been monitored continuously at volcanoes using an H_2/O_2 fuel cell as an electrochemical H_2 sensor (Sato and McGee, 1981). Data is telemetered to a central location for recording and interpretation. Continuous monitoring of H_2 (and a few other species) is low cost, automatic, and works during cloudy, rainy, or windy conditions that often prevent other types of gas monitoring, and even during eruptions when danger precludes other sampling near the vent. Continuous monitoring provides information about short-term (minutes to hours) changes in the chemistry of fumarole emissions and complements other methods of volcanic gas monitoring.

Gases in Soils and Groundwater:

Gases such as radon, helium, mercury, and carbon dioxide occur dissolved in groundwater or adsorbed to minerals in the soil. Changes in the concentrations of these gases may be caused by (a) changes in the temperature of the soil or groundwater, (b) changes in the level of groundwater, or (c) changes in the rate of supply of these gases from the shallow hydrothermal system or magma.

The normal method of monitoring soil gases involves surveys of an area of interest, repeated at a suitable time interval. That interval may be daily, weekly, monthly, or yearly and depends on the concentration of the gas in the soil or groundwater and the expected rates of change. Meteorological effects must be monitored carefully because changes in atmospheric pressure, air temperature, and rainfall affect the concentration of gases in soils and groundwater.

The emission of radon (222) gas from soil and its concentration in groundwater near volcanoes have been observed to increase before a few eruptions for which such measurements have been made (Gasparini and Mantovani, 1978; Cox and others, 1980; Hauksson, 1981 a,b). Although some radon (222) changes may reflect magmatic degassing near the surface, most observed changes are caused when an intrusion fractures the surrounding rock. Groundwater moving into the new fractures is thought to increase the rate of radon emission from the rock, and also perhaps to flush radon from the rock and toward the surface where it is monitored (Hauksson, 1981 a,b).

Background data and the frequency of periodic measurements:

Background geochemical studies must start before a volcano begins to show signs of unrest, in order to establish a baseline set of measurements against which future changes may be compared. Periodic sampling and field visits should be sufficient to reveal changes in the intensity and chemical composition of gas emissions. Once changes have been noted, more frequent sampling or even continuous monitoring should begin. If and when SO_2 is detected as a component of the emissions, remote sensing should begin using the COSPEC technique. Measurements of

SO_2 must be relatively frequent to overcome "noise," from errors in windspeed measurements and from factors that cause variable proportions of sulfur gas to occur as H_2S or H_2SO_4 (not measured by the COSPEC). Experience at Mount St. Helens has demonstrated that daily rather than weekly or less frequent measurements are needed for short-term forecasting.

Interpretation of gas emission data:

In general, emission rates of volcanic gases from an area will remain low if the melt is either depleted in volcanic gases or is under confining pressures that prevent the loss of gases. Conversely, increasing flux rates of magmatic gas suggest that magma is rising to shallow depths where degassing can occur. Ratios of less soluble gases to more soluble gases (e.g., SO_2/HCl or CO_2/SO_2) may increase initially as magma rises, because the less soluble gas begins to exsolve sooner and faster than the more soluble gas; then, as degassing continues those ratios will decrease. Noguchi and Kamiya (1963) documented increases in the S/Cl ratio before eruptions of Oshima Volcano (fig. 14c). Changing ratios of gases and an increase in total emissions can also reflect an influx of gas-rich melt into the shallow parts of a magma reservoir.

An absence of change requires great care in interpretation. Because most magmatic gases will not exsolve until magma rises to shallow depth (less than 1 to 2 km?), a magmatic intrusion at greater depth will cause little or no change in the composition of fumarolic gases. Also, gases that are exsolved from magma can be absorbed into groundwater and never be apparent at the surface. Thus, fumarolic gases cannot be relied upon to reveal a magmatic intrusion that is relatively deep.

After an eruption has begun, gas studies permit inferences about changes in the explosive potential of the magma, by comparison of the measured degassing of magma and the degassing that is predicted from an inferred volume of magma that is shallow enough to degas. Especially in viscous magma, volatiles may not be able to escape as quickly as their solubility decreases. If degassing keeps pace with or exceeds the rate at which a volatile's solubility decreases, explosive potential remains constant or decreases; if, however, the solubility of a gas in magma decreases faster than that gas can be released, the magma will become over-saturated in that volatile and its explosive potential will rise. Water and carbon dioxide are the two most abundant volatiles in magma, and therefore exert the greatest influence on its explosive potential. The pre-eruption content of a particular volatile in the melt can be determined by microprobe analysis of glass inclusions in phenocrysts (Anderson, 1974, 1975); using this technique, Melson (1983) inferred that the pre-eruption water content of Mount St. Helens melt decreased from about 5% in 1980 to about 1% in 1982. The pre-eruption concentration of a volatile times the known volume of magma erupted during an eruptive sequence can be compared to the observed cumulative flux of that same volatile. Casadevall and others (1983) have made such a comparison with SO_2 to demonstrate that substantially more magma degassed at Mount St. Helens in 1980-82 than was erupted.

Thus volcanic gas studies, in combination with previously discussed geophysical studies, are a powerful tool with which to infer the explosive potential of a volcano.

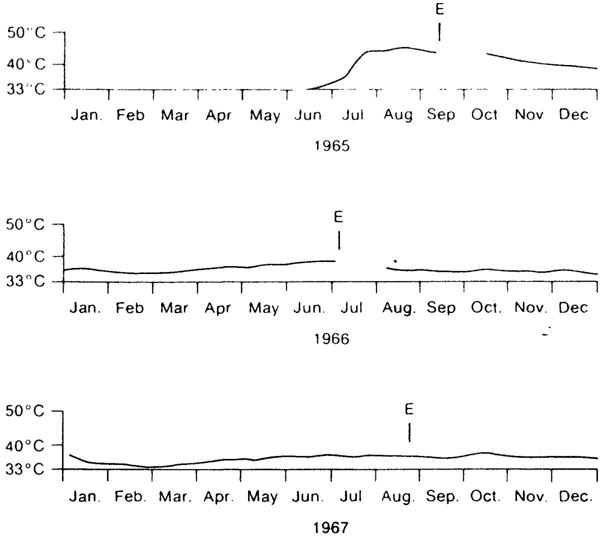
7. Thermal monitoring: Most young and potentially active volcanoes display surficial thermal manifestations such as warm ground, hot springs, and fumaroles. Baseline monitoring, begun well in advance of any unrest, should inventory thermal features by noting their location, areal extent, and maximum temperature (c.f., Casadevall and Hazlett, 1983). Such an inventory can be made in a reasonably short period of time and at a reasonable cost. If magma rises toward the surface it will tend to increase those surface manifestations by heating country rock and groundwater. Ground deformation accompanying a magmatic intrusion can also open cracks that allow cool groundwater to come in contact with hot rock or magma, or allow heated groundwater to rise to the surface. Almost all thermal effects at the surface depend on convective heat transfer by groundwater, vaporized groundwater, or magmatic gases; conductive heat transfer is too slow to cause changes that would be pertinent to forecasting.

On-site temperature measurements can be made at fumaroles, hot springs, crater lakes, or in soil, using simple thermometers or thermocouples. Telemetry can be added if desired, although sensing or telemetry instruments tend to corrode quickly in hot volcanic gases or waters. Remote temperature measurements can be made using thermal infrared (IR) from a terrestrial viewing point near the volcano, or from an aerial survey (Moxham, 1971; Cassinis and Lechi, 1974; Friedman and Frank, 1980; Kieffer and others, 1981). Most such surveys are uncalibrated and show relative differences in temperature; calibrated instruments are available but are more expensive.

Thermal monitoring is attractive in that it is a simple, relative direct expression of the presence of magma at shallow depths. There are a few instances in which temperature increases have been systematic enough to aid in eruption prediction, e.g., at Kelut (Hadikusumo, 1961) and at Taal (Alcaraz, 1969; Alcaraz and Tan Cardoso, 1969) (fig. 17). Most of these instances are at hot springs with relatively constant discharge, or at crater lakes that integrate heat outflow over the area of the lake. Experience has shown that Kelut Volcano is likely to erupt when its crater lake temperatures reach about 40°C; Taal Volcano erupted in 1965 after the temperature of its "Main Crater Lake" reached 45°, but subsequent eruptions have occurred without such notable increases in the temperature of the lake.

Unfortunately, for the few cases where thermal monitoring has been useful, there are many more where it has not. A subtle thermal signal from depth is masked by anything cool at the surface--cloud, snow, surface water, or dense vegetation, and even subsurface "down-hole" measurements are strongly affected by groundwater. Cool groundwater tends to buffer and mask thermal effects, and some temperature fluctuations are merely the result of variable dilution and masking by cool groundwater. Groundwater circulation patterns can also produce complicated patterns of surface heating that bear little resemblance to the shape or location of the magma. Records of groundwater levels, precipitation, and air temperature are necessary in order to distinguish volcanic effects from external, meteorological effects. When meteorological complications are present, or cannot be ruled out by the available meteorological data, thermal monitoring is unlikely to show more than is already known from seismicity, ground deformation, and gas emission.

Thermal monitoring can in some instances indicate whether groundwater temperatures are high enough that a phreatic explosion could occur. If



Changes in the temperature of the crater lake on Taal Volcano in the Philippines sometimes warn of forthcoming eruptions. In 1965 the temperature began to rise well above its background level of 33°C in July; the volcano erupted in September. The 1966 eruption was preceded by a much less obvious temperature rise, and the 1967 eruption could not have been forecast on the basis of temperature alone. (Data from A. Alcaraz, Philippine Commission on Volcanology.)

Fig. 17 Temperature of the Main Crater Lake, Taal Volcano. From Decker and Decker (1981), after Alcaraz (1969) and Alcaraz and Tan Cardoso (1969).

hot springs and fumaroles are at or near the boiling point of water, temperatures are presumably high enough beneath the surface to permit a phreatic explosion. However, lower temperatures at the surface do not preclude high temperatures at depth or the possibility of a phreatic explosion. Chemical study of thermal waters can indicate the maximum temperatures in a hydrothermal system (e.g., Fournier, 1977, 1979).

8. Visual and other observations: Relatively few visible changes occur at a volcano before an eruption begins; exceptions may be glow along fractures in a vent area, increased fume or steaming, changes in the color of a plume or of sublimates at vents, changes in odor of fumes, or unusually dramatic ground deformation. Many important visual observations can be made of successive eruptions and their products. Eruptions typically have multiple phases, so this aspect of monitoring becomes especially important after an eruption begins.

Commonly observed features of eruptions are their frequency and duration; specific eruptive phenomena such as tephra fall, pyroclastic flows, debris flows, and lava flows; the volumes of erupted products; the explosive force with which products are ejected and the heights to which eruption columns or lava fountains reach; the ratio of older wall rock to fresh magma; and the composition and crystallinity of juvenile products. After an eruption begins, it is common for visually observable changes to accompany instrumentally-monitored changes, and the combination of visual and instrumental data is a powerful tool for anticipating the further course of an eruption.

THE RECORD OF HISTORICAL VOLCANISM OF THE WORLD

To the extent possible, forecasts should be based on a known relation between precursors and eruptions of a particular volcano in question. This is possible where numerous eruptions have been monitored, but most volcanoes erupt too infrequently, or have insufficient monitoring (data) upon which to base precise forecasts. Even at well-monitored volcanoes a new pattern of precursors can appear, bringing uncertainty to forecasts. In such instances the record of precursors to eruptions at other, similar volcanoes can be a valuable reference.

More than 5000 eruptions have occurred at more than 500 volcanoes in historical time (Simkin and others, 1981; Simkin and Siebert, 1984). Only a small percentage of these eruptions have been described in detail, and only some of these descriptions contain information on eruption precursors. Nonetheless, there is enough information about precursors to eruptions of the world to serve as a valuable reference during a volcanic crisis. Systematic observations at volcanoes worldwide have increased greatly within the last two decades, spurred in part by systematic reporting programs of the Volcanological Society of Japan and the Smithsonian Institution, and in part by the establishment of more volcano observatories.

When a volcanic crisis develops, we often consult literature references and colleagues who are familiar with eruptions at other volcanoes. Sometimes this works well, but sometimes it is impractical. Library facilities may be inadequate, colleagues may be far away, and the process of checking references systematically may take more time than the volcano is willing to grant us. A compilation of information about precursors to historical eruptions around the world is currently in preparation, for addition to the large Smithsonian volcano data file.

When complete, the compilation will permit quick computer searches for patterns analogous to those being observed at a restless volcano. A search will give a range of possible outcomes, suggest the most likely of these, and provide references to details of any close analogues.

Successful application of the worldwide record to a particular volcano requires identification of key facts about the restless volcano and its precursor activity. These key facts become the criteria for comparisons and searches. For example, one might want to know examples of andesitic stratovolcanoes at which seismic energy release has exceeded 1016 ergs/day for 10 or more consecutive days. Or one might want to know all cases of volcanic tremor of a certain frequency or amplitude. Too small or general a set of criteria will result in a very large number of possible analogues, some of which may not be pertinent. Too large or specific a set of criteria may result in only a few possible analogues, omitting others that are pertinent. In general, it is helpful to start with relatively general criteria and a large set of possible analogues, and then use more specific criteria to shorten the list. The original, longer list can be kept for reference in case the activity might change or some new observation suggests reconsideration of the larger set of possibilities.

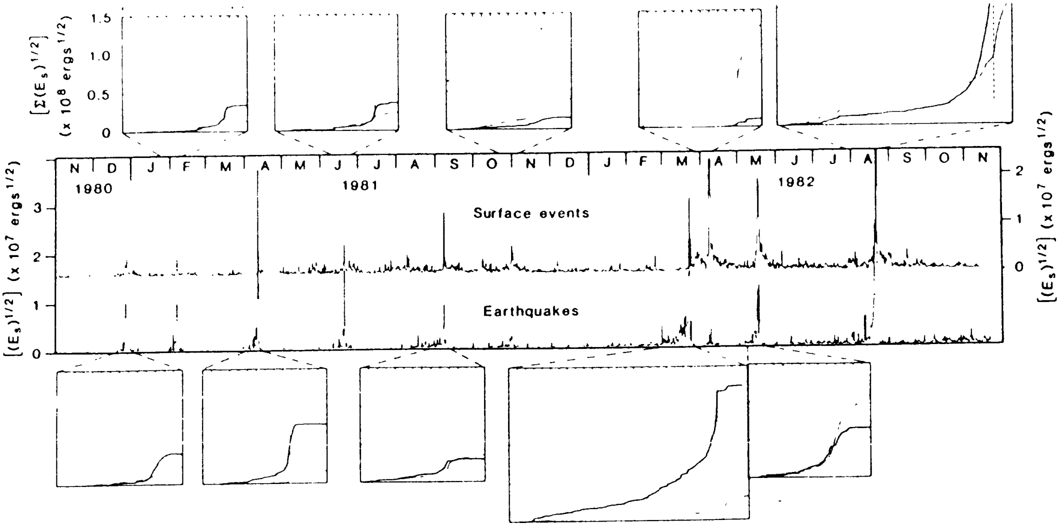
Successful application of the worldwide record also requires a recognition of its limitations. Only rarely will an analogy be so close and unique that it can be the basis for a specific forecast. More commonly, a set of analogues will suggest a range of possible scenarios for the restless volcano. Each possibility can be treated as a working hypothesis, tested by the continuing unrest at that volcano.

FORECASTING THE ONSET OF AN ERUPTION

The simplest and generally the most reliable way to forecast the date and type of an eruption is to compare its precursors to known precursors of previous eruptions. This is pattern recognition--recognition of a specific pattern of unrest and its relation to an ensuing eruption. If a particular pattern of seismicity or deformation or other change is known to have preceded an earlier eruption, recurrence of that pattern suggests that another eruption will occur. The principle is simple; its application is not always so simple.

An eruption can be forecast on the basis of a cumulative amount or a rate of precursory change, if that change matches or can be projected to a threshold level of activity at which past eruptions are known to have begun. Minakami (1960) considered the daily frequency of B-type earthquakes at Asama Volcano and Tokarev (1963) considered cumulative seismic energy release at Bezymianny, as described earlier. At Mount St. Helens, cumulative curves of seismicity (Malone and others, 1983) and ground deformation (Swanson and others, 1983; Chadwick and others, 1983) have been helpful in forecasting eruptions from late 1980 to the present. Fig. 18a shows seismic energy release before eruptions of Mount St. Helens in 1981 and 1982; Fig. 18 shows deformation of the crater floor before the same eruptions. Repetitive patterns (and considerable effort to document those patterns) have been the keys to successful forecasts at Mount St. Helens.

Most of the forecasting described above could have been accomplished without any understanding of the causes. But it is important to understand the causes or mechanisms of precursory activity, as becomes apparent when a new pattern of precursors appears. An understanding of the causes of



The square root of seismic energy release $[(E_s)^{1/2}]$ per half day from 1 November 1980 to 20 November 1982. Surface events (types s and a) are plotted separately from earthquakes (types m and l). The cumulative $(E_s)^{1/2}$ released during a 2-week period (a 4-week period in the case of the March 1982 and August 1982 eruptions) around each of the eruptions is shown near the time of each eruption. Light lines represent surface events, and heavy lines represent shallow earthquakes. Each vertical dashed line indicates the most likely time of the beginning of an eruption.

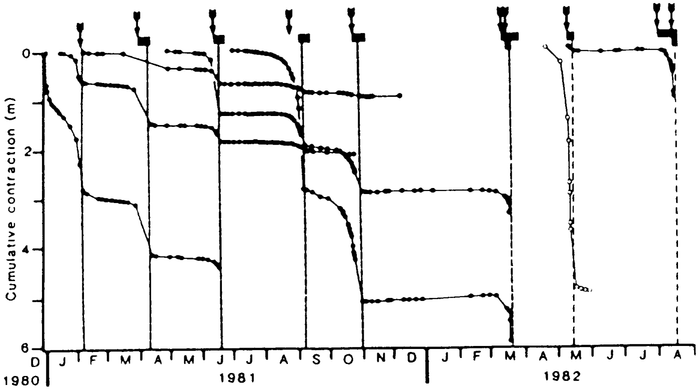


Diagram summarizing the displacement data for selected thrust faults (closed circles) and one target on the west flank of the dome (open circles), dates on which relatively long-term predictions were issued (arrows), predictive window, and dates of the start of eruptions, late December 1980 through August 1982

Fig. 18 Repetitive pattern of seismic energy release (upper) and deformation of the crater floor (lower) at Mount St. Helens, December 1980-November 1982. Contraction across thrust faults and along the one EDM line indicates expansion of the crater floor. Eruptions during this period were dominantly non-explosive dome growth, but a few including that of March 19, 1982 had small explosions at their onset. The repetitive patterns, and comparison of the curvature on these graphs (=acceleration of precursory activity) permitted accurate predictions (arrows and dark blocks) of eruptions (vertical lines).

Upper, from Malone and others (1983); lower, from Swanson and others (1983).

precursors permits one to interpret the causes and hazards implications of any change in the pattern of unrest. For example, eruptions at Mount St. Helens were preceded by several hours or days of unrest in mid-1980, but by several weeks of unrest in 1981. What caused the change, and what are the hazards implications of short vs. long precursors? One possibility is that the change reflected decreasing volatile contents in the magma and hence a progressively more sluggish rise toward the surface. This hypothesis seems to be supported by subsequent activity and analyses, but it still remains a working hypothesis. A corollary is that if in the future precursors should develop quickly, over only a few hours or days, we might see explosive activity once again.

Most geophysical and geochemical changes before eruptions are related. Several types of changes might be the result of a single cause, e.g., seismicity and inflation due to magmatic intrusion. Some precursory processes can also cause or modify other processes, e.g., heating of groundwater might increase pore pressures and cause seismic strain to be released in many small earthquakes rather than in fewer, larger earthquakes. Consideration of the various types of precursory changes together provides more information that the sum of consideration of each change by itself. Precursor data are synergistic. Commonly, one type of data can help to constrain interpretation of another type of data. Geophysical and geochemical data almost always permit several interpretations, and consideration of two or more types of data is necessary to reduce the number of possible interpretations. For example, gravity changes can be due to vertical ground movement or to changes in groundwater levels, but interpretation of gravity data and water well data will show how much (if any) of that variation is due to changes in groundwater, and permit a better estimate of how much uplift or subsidence might have occurred. Similarly, interpretation of whether seismicity at a volcano is due to rising magma can be constrained by ground-deformation data.

The eruption of Mount St. Helens on March 19, 1982 provides an example of how several types of data were combined to make a remarkably accurate forecast. Seismic energy release, outward horizontal displacement of points on the dome, tilt of the crater floor, and the flux rate of SO_2 before this eruption are shown in Fig. 19. The broad pattern of seismicity and ground deformation was like that of other eruptions in 1981 and 1982 (Figs. 18a and 18b), so a dominantly non-explosive eruption was forecast. However, deep earthquakes had signalled the possibility of recharge by more volatile-rich magma, and the absence of any systematic increase in SO_2 flux before the eruption suggested that the high volatile content, if present, was not being released, so an explosive onset to the eruption was also considered possible. Successively more specific predictions based on curve-fitting of both seismic and deformation data narrowed the eruption "window" (the period within which the eruption was expected to begin) to 24 hours. Ten and one-half hours later, an explosion marked the onset of a dominantly non-explosive eruption.

The great increase in what we can learn from several types of data, considered together, is a strong argument for a team approach to volcanic eruption forecasting. Too often, volcano monitoring efforts are fragmented into different specializations (or worse, different institutions), communication between specialists is limited, and much opportunity for accurate forecasts and new knowledge about volcanic processes is lost. A team needs geologists, geophysicists, and geochemists, working closely with each other!

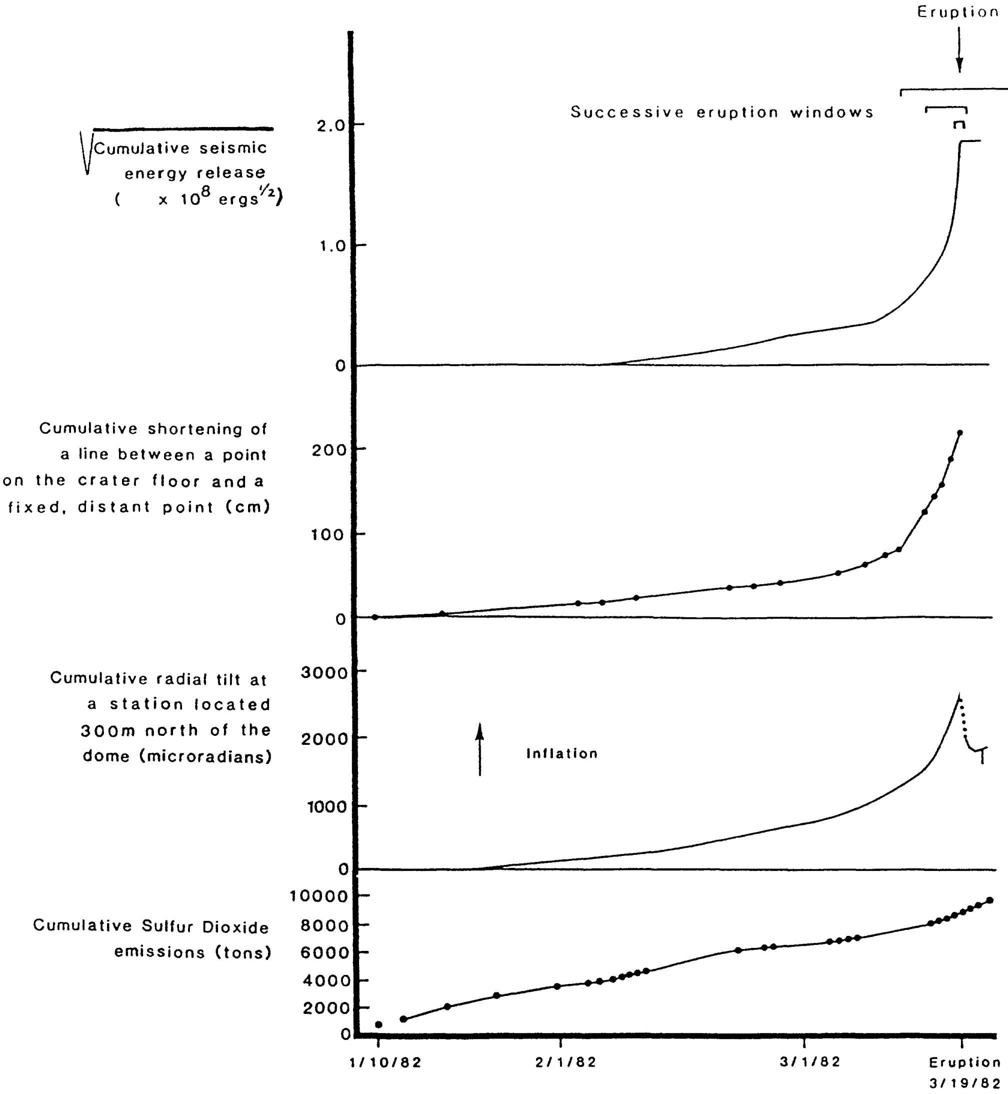


Fig. 19 Seismic, horizontal strain, tilt, and SO_2 precursors to the eruption of Mount St. Helens on March 19, 1982. Plots of several precursors on a common time scale, as in this example, help one to see the time relation of various precursory changes. On cumulative plots such as these, a straight line indicates a constant activity, at a rate defined by the slope of the line. A line that is concave upward indicates accelerating activity.

The first precursor of this eruption was an increased rate of contraction along several EDM lines (including one shown here), followed by increased rates of tilt and seismic energy release, in that order. The three-day average of SO_2, shown here, did not change significantly, but the daily emission rate (not shown and perhaps not significant) doubled from March 18 to March 19. From Swanson and others (1984).

FORECASTING THE AREA TO BE AFFECTED BY, AND SEVERITY OF AN ERUPTION

It is not enough to tell local officials when a volcano will erupt. We must also tell them, as best as we can, what type of eruption will occur, the areas likely to be affected by various eruptive phenomena, and the severity of those effects. This topic requires a blend of information from geologic and historical records, and from monitoring.

A long-term hazards assessment will show the relative frequency with which various types and magnitudes of eruptions have occurred in the past, and areas that have been affected by flowage hazards (pyroclastic flows, pyroclastic surges, mudflows, lava flows, ground-hugging pockets of volcanic gas) and non-flowage hazards (ballistic fragments, airfall tephra). A map will probably show the maximum extent to which various flowage and non-flowage hazards have reached in the historical and geological past, and may indicate smaller areas that have been affected more often. Unless that volcano has been well monitored through previous eruptions, such a map (and the records from which it was constructed) will be principal basis for forecasting the type and size of an impending eruption, and the areas likely to be affected.

A forecast of hazardous areas that is based on the maximum previous extent of hazards will naturally over-estimate hazards for most eruptions of a volcano. If this forecast covers most eruptions adequately, is there any need to try to make it more specific for each eruption? The answer is yes, for two reasons. First, there will occasionally be precursory activity that is of a larger scale than any previously observed at the volcano in question, or analogous to precursors of a particularly hazardous event that occurred at a different volcano. An example, clearer now in retrospect, occurred at Mount St. Helens in March-May 1980. Ground deformation above a viscous dome intrusion (cryptodome) can be unusually pronounced, in some cases measurable in tens or hundreds of meters of uplift or bulging. If such intrusion occurs beneath relatively flat terrain, the intrusion may stop before it reaches the surface or, if it reaches the surface, it may lead to normal, vertically-directed explosive eruptions (e.g., historical episodes of cryptodome intrusion and explosions at Usu Volcano). However, if such intrusion occurs beneath the steep upper flanks of a stratovolcano the intrusion can cause a large landslide, release the confining pressure on the intruding magma or on a hydrothermal system, and cause a major laterally-directed blast (e.g., those at Bezymianny in 1956 and Mount St. Helens in 1980).

The more common need for a revised hazard map arises when precursors or eruptions are smaller and less worrisome than a "worst case" example. If an eruption crisis is disrupting normal economic and social activity, and especially if it is requiring a major evacuation, local authorities will often want to minimize that disruption. It is rarely possible to guarantee that a large, hazardous eruption will not occur, but it is often possible to state that an impending eruption will "most likely" be smaller than the "worst case." Some local officials will ask or expect that volcanologists can forecast that an eruption will be of precisely one type or magnitude; we need to educate those officials to understant that the precise type and magnitude of eruptions cannot yet be predicted. A scientifically honest forecast should note a range of possible activity, and if possible should estimate the relative or absolute likelihood of each type or scale of activity. Local officials at Mount St. Helens initially asked for simple forecasts. Now, they realize the complexity of the volcano and they ask for forecasts that indicate a range of possible events, and the relative likelihood of each event.

How can monitoring data be used to make such revisions? We know relatively little about how to revise forecasts of the type or magnitude of an eruption, but can use the same general principles used to forecast the onset of an eruption. These include:

(1) correlations between specific precursors or patterns of precursors and eruption types, at the volcano in question;

(2) an interpretation of current precursors, even if the current precursors have not been previously recorded at the volcano in question;

(3) a close analogy between current precursors and precursors to a known eruption type at another, similar volcano.

When possible, more reliance should be placed on (1) than on (2) or (3). But (1) is frequently unknown, especially at the start of an eruptive episode, so (2) and (3) become the only available bases for forecasting the type or magnitude of an eruption. Interpretation of precursors (2), discussed in earlier sections, is often complicated by varied effects of the volume of magma, viscosity, volatile content, and repose period since the previous eruption. For example, strong seismic precursors could reflect a large volume of magma, *or* particularly viscous magma, *or* a long repose since the previous eruption (and hence a great amount of work required to re-open a conduit). In order to forecast the type or magnitude of an eruption on the basis of a particular set of precursors, we need several lines of monitoring data, each constraining the interpretation of the other. Reference to activity at another volcano (3) may be more reliable than (1) or (2) if the analogy is based on an unusual, distinctive precursor (e.g., dramatic bulging of the flank of a volcano). Each type of information should be considered to the extent possible, and none should be ignored.

Short-term forecasting of the effects of an eruption requires a forecast about the eruption itself and consideration of non-volcanic factors, such as the wind speed and direction on the particular day of the eruption, or the volume of surface water that might be displaced by or mixed with the eruption products. One can describe the average or prevailing wind directions at any given time of year, but only the wind pattern of the particular day of eruption, at the particular altitudes to which the eruption plume rises, will actually determine the dispersal of a tephra plume. Likewise, the extent of lahars during an eruption is strongly influenced by seasonal or even daily variations in the amount of surface water--snowpack or water in streams and lakes. A relatively small explosive eruption can have far-reaching effects if that explosion melts snow, displaces the contents of a crater lake, or otherwise releases a large volume of water quickly.

FORECASTING THE FURTHER COURSE OF AN ERUPTION

The beginning of an eruption or eruptive sequence is only the beginning of the volcanologist's job of advising local authorities or population. Most natural hazards peak over the course of a few days or less (e.g., the main shock in a tectonic earthquake sequence, or the crest of a flood). In contrast, volcanic eruptions may occur in multiple pulses over a period of weeks, months, or even years, and often peak well after the onset of the eruption.

How can we forecast the further course of an eruption, over future hours, days, weeks, or months? How can we answer the officials who ask "Is the worst already past?" or "How much longer will this eruption continue to be hazardous?" Some information comes from detailed geologic or historical records of previous eruptions, because they suggest eruptive patterns that might be followed again, and models of the relation between that volcano's magma supply, reservoir geometry or "plumbing," and eruptions. If information that is available for a particular volcano is insufficient, one can turn to the record of volcanism at similar volcanoes.

Suppose, for example, that an eruption appears to be following a pattern that has previously occurred at that volcano, e.g., small phreatic explosions, a relatively large magmatic explosion, smaller magmatic explosions, and finally extrusion of lava. We could simply identify the current phase or position in this sequence and forecast the remaining phases, empirically, based on the assumption that previous patterns are likely to be repeated. Or, with monitoring data, we can begin to interpret the processes in the current eruption. In a sequence such as described above, we could hypothesize that gas-rich magma rose through the water table, causing phreatic explosions that evolved into eruptions of gas-rich magma. Subsequent eruptions tapped slightly deeper, gas-depleted portions of the magma reservoir, and finally magma that was so gas-depleted that it erupted non-explosively as lava. If this hypothesis is correct, initial seismicity would be deep and gradually migrate toward the surface, initial deformation would be broad and gradually become more localized, and magmatic gas emissions would increase after the phreatic explosions, peak during the first magmatic explosions, and then begin to decrease gradually. Volatile and silica contents of the eruptive products might decrease slightly from one eruption to the next. If monitoring data support this hypothesis, they lend credence to the empirical forecast. Unless there is a major fresh influx of magma from depth, explosive potential and hazards will be greatest during the first magmatic explosions and decrease thereafter. Historical data can suggest hypotheses about current activity; monitoring data can help to test those hypotheses.

The example cited above is simple and only one of innumerable patterns that are possible. However, the principle applies to simple or complicated patterns: look for patterns in previous and present eruptive activity, formulate several hypotheses to explain their changes, and test those hypotheses using independent (usually monitoring) data. Other changes that might be used to forecast the further course of an eruption include changes in the rate of eruption, changes in vesicularity, crystallinity, and phenocryst content of the eruptive products, and changes in repose periods between phases of an eruption.

FORECASTING THE END OF AN ERUPTION

Though we devote much effort to forecasting the onset of an eruption, and additional phases, one of the most important aspects to civil authorities is the end of an eruption. When eruptive activity declines or stops for a few days, weeks, or months, civil authorities may ask volcanologists to certify that the eruption is over. More often than not, we find it difficult to do so, and a combination of time and social pressures gradually bring life back to normal even without a clear signal from volcanologists that the eruption is "over."

In a sense, we are asked to forecast that there will be no further eruptions. We have learned how to forecast eruptions on the basis of increased seismicity and other indicators, but what can we use to forecast that further eruptions will not occur? Deflation of the volcanic edifice may occur, but is usually in response to a just-concluded eruption or to intrusion of magma into the flank of the volcano, and cannot immediately be construed to indicate that an eruption is over. After an initial few hours or days of deflation, the likelihood of further eruptions probably decreases unless or until re-inflation occurs, but how can we know whether such re-inflation will occur? Indicators such as seismicity, ground deformation, gas emissions or thermal manifestations may return partially to their baseline levels; again, we can say that the likelihood of an eruption has decreased, but has not yet declined to its long-term likelihood.

In still other cases, an unusually long repose time may be our only indication that the eruption might be over. As a rule of thumb, if a volcano has already "completed" what is a typical eruptive episode for that volcano, shows a general decline in precursory activity, and stops

SOME CHALLENGES IN VOLCANIC ERUPTION FORECASTING

Short-term assessments of volcanic hazards have improved markedly in recent decades, but still suffer from inadequate data and uncertainties in how to interpret the data that are available. A few of the challenges for volcanologists are discussed below. The first two challenges relate to the collection of data; four other challenges relate to both collection and interpretation of data.

(1) Optimizing monitoring data when budgets or technical expertise are limited. When budgets are limited, some relatively simple instruments can provide almost as much information as more costly operations. The World Organization of Volcano Observatories (WOVO) has recently suggested an "adequate minimum" of monitoring equipment and information for volcano observatories (Tilling, 1982). The recommended minimum is (a) a seismic network of three stations, with telemetry and an alarm that sounds if seismicity exceeds a certain threshold, and (b) a continuously recording tiltmeter, with telemetry and at least one non-electronic tilt measuring array to serve as a check and calibration. Other recommended but less critical activities are: (c) "periodic re-occupation of a network of 'dry tilt' stations; (d) a modest trilateration (EDM) network and a few levelling lines--to permit at least the establishment of bench marks and initial baseline measurements; (e) observations of physical changes in the vicinity of the volcano--e.g., appearance of new fumaroles and/or changes in known fumarolic areas; ground cracking, minor landslides, and other surface indicators of instability; unaccountable changes or damage of vegetation; (f) collection of information resources, documentation, manuals for instruments or monitoring techniques; (g) study of historical records to

erupting for a period several times longer than the longest repose within the current or other historically recorded eruptive episodes, it is reasonable to say that the eruption is over. However, both we and the civil authorities need to be aware that a volcano can pause for months, years, or even decades and then resume an eruptive episode. Eruptive activity can and often is renewed by a late-arriving influx of hotter, gas-charged magma into a shallow magma reservoir. In many instances it may be advisable to tell civil authorities that the likelihood of an eruption has decreased, and the basis for that conclusion, rather than to tell them categorically that an eruption is over.

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collection of information resources, documentation, manuals for instruments or monitoring techniques; (g) study of historical records to ascertain at least the historic eruptive pattern, (and) geological mapping and other studies to improve understanding of prehistoric activity." (Tilling, 1982, reporting for WOVO).

At the present time, it is more economical in some developing countries to have instruments read by field observers than to have data telemetered to a central location, but technological advances in telemetry (including satellite telemetry), a need for better quality data, and rising costs of maintaining small field stations will eventually make telemetry and centralized recording more economical than large numbers of field observatories. The next few years may see increasing use of microprocessors at field sites to store full traces of a day's or a month's seismic events, and then transmit them rapidly via conventional radio or satellite links to central recording stations. Data processing in central observatories will also become increasingly automated, permitting real-time location of hypocenters and calculation of cumulative seismic energy release in a precursor swarm.

When technical expertise is limited, telemetry can bring real-time data directly to the few scientists in a country who are qualified to interpret it. Satellite and even conventional telemetry are expensive and may require greater electronic skills than may be at hand, but an initial investment in equipment and electronic training can bring a great return in timely and correct data interpretation.

Even though the general trend in monitoring will be toward greater automation and use of telemetry, the value of a local observer should not be underestimated. An local observer who is intimately familiar with a volcano and who checks frequently for a variety of changes may still be the first to recognize some precursors to an eruption. This is especially true if records are not checked frequently at the central recording site, or if electronic problems result in periods when no signals are being telemetered to the central recording site.

(2) Developing instruments for continuous monitoring of deformation and gas emission, to provide records that are comparable to seismograms. Some such instruments already exist, e.g., electronic tiltmeters and H_2 gas probes, but more are needed. Continous monitoring offer the best chance to detect short-term changes and to know the full variability of activity; such monitoring also helps to ensure that critical data can be gathered immediately before and during eruptions without undue hazard to volcanologists.

(3) Recognizing eruption precursors that are subtle or widely separated in space and time. The key to recognizing subtle precursors is to have trained and interested observers, frequent measurements, sensitive instruments near the vent, and careful, regular inspection of the records for subtle changes. The recognition of a relation between widely separated precursors is more difficult, but a general approach is to assume that every reported change within several tens of km of the volcano, or within several years of the time of discussion, may be significant even if that significance is not immediately understood. Too often, changes that are not understood are dismissed and forgotten, only to be seen as significant after an eruption. Embarrassing examples abound.

A related solution to this problem is to encourage communication between all parties who might have pertinent observations--volcanologists, seismologists, airline pilots, surveyors, local residents, and others--so that volcanologists learn of changes near the volcano that they might not normally monitor, and so that each specialist can suggest significance to observations that the others might not have considered.

(4) Improving techniques for forecasting the time of climax of an eruption. Some eruptions reach their climax within minutes or hours of their onset; others do not climax until months or years later. How can we forecast whether an eruption is likely to climax shortly after its onset or much later? If the latter, what geophysical, geochemical, and geological information can we use to forecast exactly when that climax will occur?

(5) Improving techniques for forecasting the type and magnitude of an eruption. Our current limitations have been noted earlier, and new approaches are badly needed. Some promising work is being done to determine pre-eruption volatile contents of magma, and to compare the pre-eruption volatile contents from explosive and non-explosive eruptions (e.g., Melson, 1983). Are there trace elements that might be helpful in this regard? Are there new ways to relate changes in the magma with observed degassing, and with changing patterns of seismicity and ground deformation? A compilation of the precursors to historical eruptions of the world may suggest unrecognized relations between precursors and eruption type. All of these techniques and more are needed.

(6) Minimizing "false alarms." Many potentially precursory changes do not culminate in eruptions, but these are difficult to recognize in advance. The best hope for distinguishing changes that will and will not lead to eruptions is careful, continuous monitoring of a volcano even in times of relative quiet, to learn the full range of possible unrest and its results. This is a process of trial and error, in which the errors can be minimized but probably never eliminated. The problem is particularly common and worrisome where large silicic magma reservoirs can be disturbed by tectonic strains or by an influx of magma from depth, but adjust to that disturbance without an eruption.

Another useful approach to the problem of "false alarms" is to educate public officials and the general public about the nature of the problem, the likelihood of occasional false alarms, and the need to tolerate a few false alarms if they wish to be protected against those increases in activity that do culminate in eruptions. Most laymen have unrealistic expectations about the invincible power of science, and it is better for us to tell them what the science can and cannot do, rather than have them conclude after a few false alarms that scientists can't do anything!

SOME FINAL REMARKS

Advances in eruption forecasting within the past two decades might lead some to conclude that all eruptions can be forecast, and that scientists who fail to forecast eruptions are incompetent. Not so! Despite the advances, most volcanoes have yet to show us their own distinctive precursor patterns, and even a well-established pattern is prone to change as soon as that pattern has been established. The type and magnitude of an eruption still defy short-term forecasting at nearly all volcanoes. Even when a volcano follows a well-established pattern, there may be several supportable explanations for that pattern, each wit different implications regarding volcanic hazards. Diverse scientific opinion and debate are inevitable and healthy, as long as debate re on a professional level and is not played out before public offici the news media who, in general, cannot understand the technical differences and who therefore may reduce the debate to a level of personal credibility. When forecasts are made, the local authorities and the public deserve to know the forecast, its basis, and its uncertainties. In the U.S., we also try to explain any major differences in scientific opinion; this "open" style is appropriate in the U.S. but may not be appropriate in all countries (Fiske, 1984).

Most successful forecasts of recent decades have been the result of hard work, intensive field studies, and full utilization of experience at restless volcanoes and at other similar volcanoes. Most unsuccessful forecasts or failures to forecast despite a capability to do so have been a result of poor communication between scientists, or of some change in a volcano's behavior, as if to keep us alert and humble!

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GEOLOGIC AND HYDROLOGIC HAZARDS IN SOUTHEAST ASIA - AN OVERVIEW

By

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INTRODUCTION

Southeast Asia lies between latitudes 23°N to 12°S and longitudes 90°E to 142°E, stretching for more than 5,600 km in the east-west direction and 4,000 km in the north-south direction (Fig. 1). The mainland portion of Southeast Asia includes Burma, Thailand, Laos, Vietnam, Kampuchea, West Malaysia, parts of Yunnan and Kwangtung Provinces of China, and the Territory of Hong Kong. The rest of the area is fragmented into more than 20,000 islands that comprise Indonesia, Philippines, Brunei and East Malaysia.

There are more than 350 million people in Southeast Asia (Table 1). Millions are clustered in the capital cities of the area. The Indonesian island of Java alone has about 95 million inhabitants.

Southeast Asia comprises one of the greatest concentrations of natural hazards in the world. Here exists the longest known active fault on land (the Great Sumatran or Semangko Fault System) and one of the most active seismic regions of the world. Seven catastrophic earthquakes occurred in Indonesia during 1976 and 1977 in which 6,302 people were killed or missing and 4,158 were injured. More than 500 young volcanoes may be distinguished, 183 of which are active. The greatest volcanic calamity in historical time was the eruption of the Tambora volcano on Sumbawa Island in 1815, which caused the loss of 92,000 lives. The Lahar slides of Mt. Kelud in eastern Java in 1919 caused 5,110 casualties, and 104 villages and towns were destroyed.

The cyclonic storm and associated coastal flooding of November 12-13, 1970, which struck the offshore Islands located to the south of Bangladesh,

Table 1 Southeast Asia: Area, Population and Density, 1976

		Population			
	Area (in sq. km)	1976 (no. in millions)	1967-71 growth rate (% per annum)	1972-76 growth rate (% per annum)	1976 density (persons/sq. km)
Brunei	14,516	0.17	-	3.3	11.6
Burma	678,033	30.83	2.2	2.2	45.5
Indonesia	1,904,345	135.39	2.2	2.4	71.1
Kampuchea	181,035	8.34	3.0	2.8	46.0
Laos	236,800	3.37	2.4	2.0	14.2
Malaysia	329,749	12.30	2.8	2.5	37.3
Philippines	300,000	43.30	3.0	2.7	144.3
Singapore	588	2.28	1.6	1.5	3,877.6
Thailand	514,000	43.60	3.2	3.1	84.8
Vietnam	332,800	46.52	2.5	2.9	139.8
	4,491,866	326.10			

SOURCE: Asian Development Bank, Key Indicators of Developing Member Countries of ADB, April 1977.

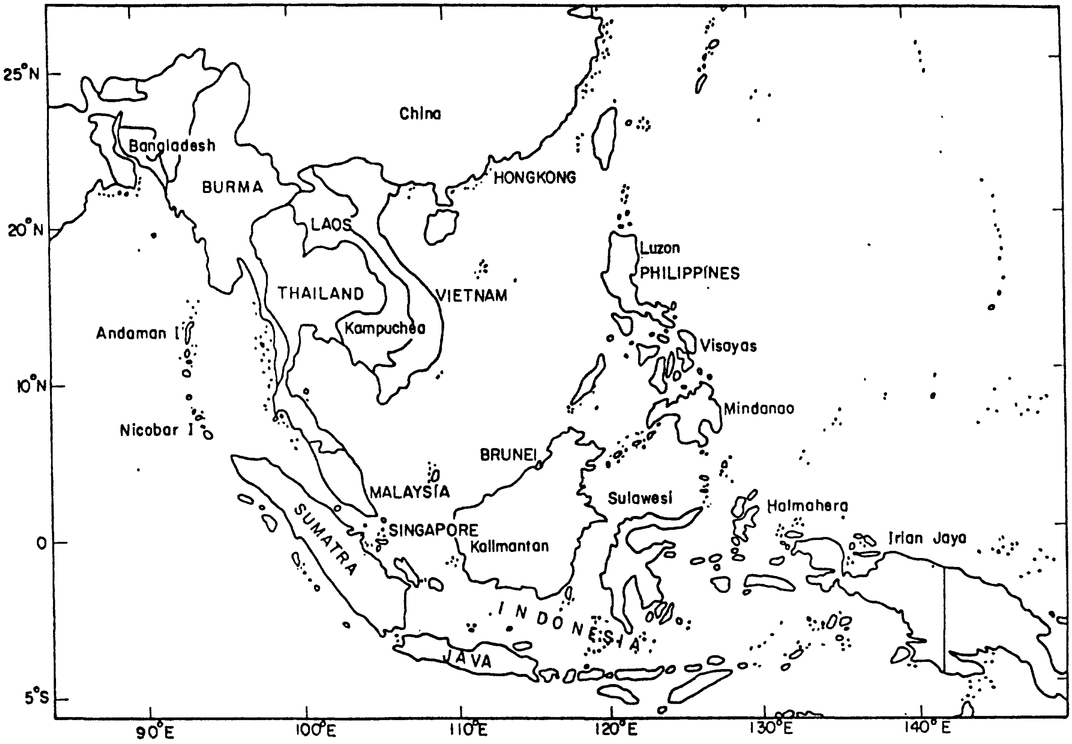

Fig. 1 Map of Southeast Asia

could be regarded as one of the worst in world history in terms of damage caused. More than 300,000 human lives were lost and 400,000 houses damaged in that catastrophe.

The geologic and hydrologic hazards in Southeast Asia can be grouped as follows.

A. Geologic hazards

    1. Earthquake hazards

        a. Ground shaking
        b. Ground failures - ground settlement, liquefaction
        c. Tsunamis

    2. Volcanic hazards

        a. Volcanic explosions
        b. Hot avalanches and mudflows
        c. Tsunamis

    3. Hazards from ground failures

        a. Landslides
        b. Problem soils - expansive soils, dispersive soils and sensitive soils
        c. Land subsidence

B. Hydrologic hazards

    1. Floods
    2. Storm surges
    3. Waterlogging and salinity

The purpose of this paper is to provide an overview of the hazards from volcanic eruptions, earthquakes, ground failures and floods in Southeast

Asia. It attempts to describe their characteristics, identifies some locations, and specifies their impacts.

PHYSICAL SETTING

Geologically, Southeast Asia comprises the Southeast Asian or China plate, surrounded by the Alpine-Himalayan belt which bends toward the south. Three mountain ranges, with their associated river valleys, can be distinguished on the mainland. In the west, the Chin Hills and Arakan Yoma of Burma which continue offshore as the Island arcs: The Andaman-Nicobar Islands, Sumatera and further into the Indonesian and Philippine Islands. The second mountain range forms the spine of the Thai-Malay Peninsula through Singapore to the Islands of Banka and Belitung of Indonesia. The third is the Laos-Vietnam fold belt stretching over 2,000 km toward the Southeast.

The Indonesian archipelago is divided into three parts. The islands of Java, Sumatera and Kalimantan, together with smaller islands, stand on the Sunda Shelf which extends from the coasts of Malaysia and Indochina; there the sea depth never exceeds 200 meters. Irian Jaya on the island of New Guinea and the Aru islands stand on the Sahul shelf which stretches northward from the coast of Australia. The sea depth of the Sahul shelf is similar to that of the Sunda shelf. Between these two shelves are located the group of islands of Nusa Tenggara, Maluku, and Sulawesi, where the sea depths reach 5,000 meters. Together, Indonesia consists of 13,677 islands and islets of which about 6,000 are inhabited.

The 7,107 islands of the Philippines can be divided into three main groups: Luzon in the north; a central cluster collectively known as the Visayas; and in the south, Mindanao and the Sulu archipelago.

Tectonically, the Southeast Asian plate is surrounded by the Indian Ocean, the Australian and the Pacific plates (Fig. 2). Active volcanisms and earthquake belts occur along these plate boundaries as well as along the spreading centers where the plates are formed and move apart (Fig. 3). In Southeast Asia, there are 183 active volcanoes, 137 of which are in Indonesia, 37 in the Philippines, 2 in the Andaman Sea, and 7 in the South China Sea (Fig. 4).

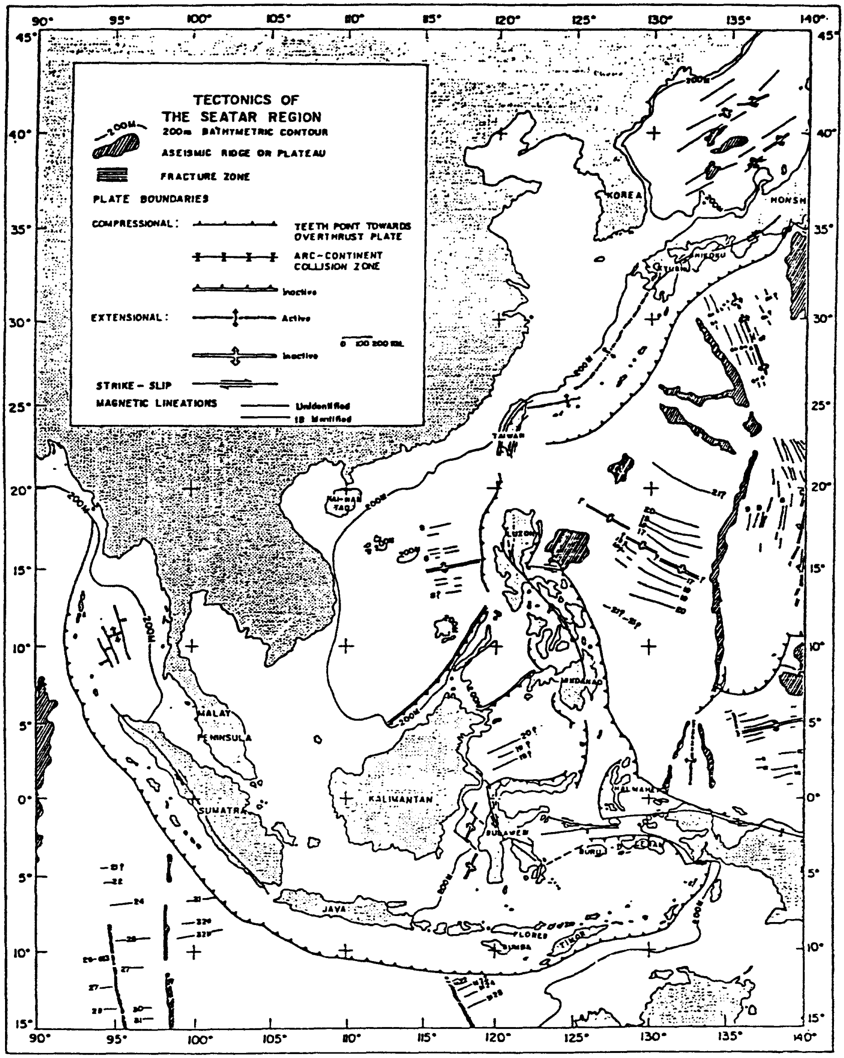

Fig. 2 Tectonics of the Southeast Asian Region (after Hayes & Taylor 1978)

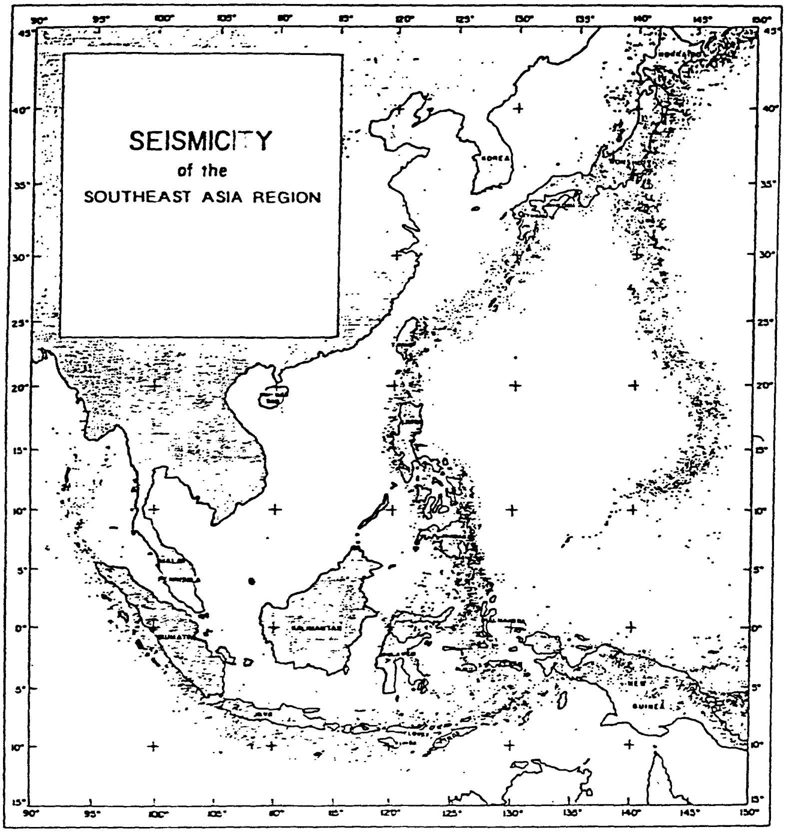

Fig. 3 Seismicity of the Southeast Asia Region (after Hayes & Taylor 1978)

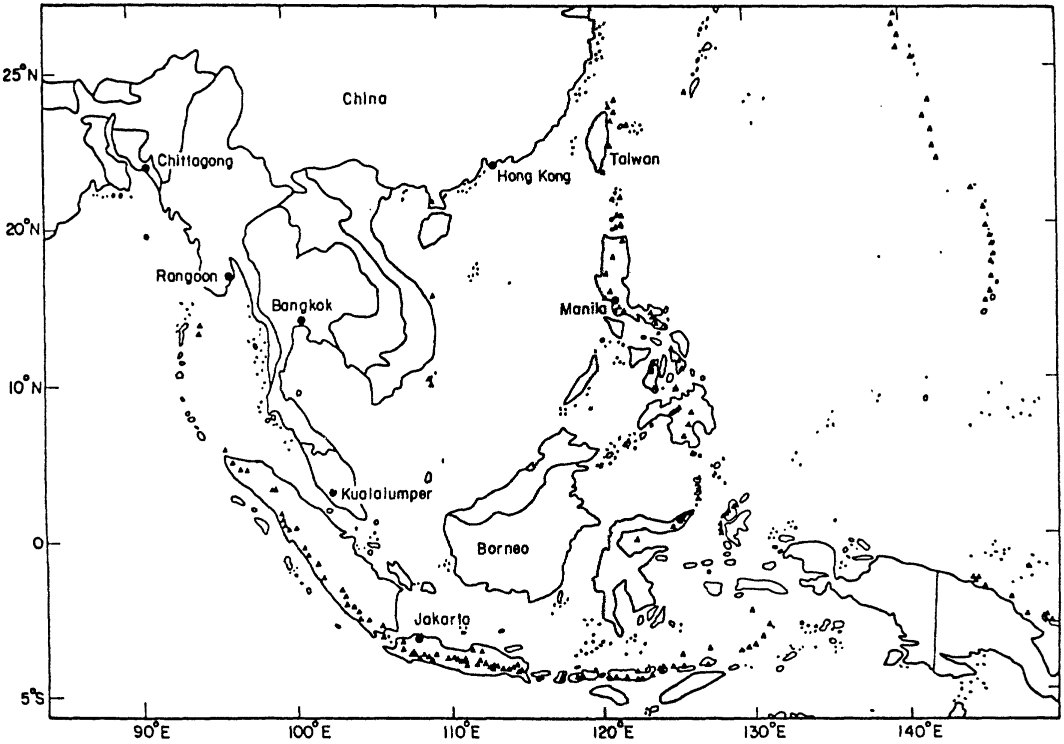

Fig. 4 Active Volcanoes of Southeast Asia (after Macdonald, 1972)

It is useful to note the most important rivers situated in South and Southeast Asia and which influence the topography and economy of the region. Three of Asia's largest rivers: the Ganges (2,500 km), the Brahmaputra (2,700 km); and the Mekong (4,100 km) follow the Himalayan flexture and drain into the Bay of Bengal or the South China Sea. Between the Arakan Hills and the Thai-Malay ranges are the valleys of the Irrawaddy (2,000 km) and the Salween (2,500 km). Other smaller rivers are the Chao Phraya (800 km) of Thailand and the Red River (800 km) of Vietnam.

Southeast Asia is surrounded by the Indian and Pacific Oceans. In the west and south, the Indian Ocean is subdivided into the Bay of Bengal, the Andaman Sea, the Strait of Malacca, and the Timor Sea. In the east the Pacific Ocean is subdivided into the Gulf of Thailand, the South China Sea, the Java Sea, the Banda Sea, the Macassar Strait, the Celebes Sea, and the Sulu Sea.

Heat and rain are the common climatic elements of all Southeast Asia. Equatorial heat varies only a few degrees throughout the year from a mean of about 30^{o}C. Rainfall exceeds 2,000 mm a year in most areas and goes as high as 3,000 to 5,000 mm in others. Some of the rain is seasonal. In some areas the rainfall is rather consistent throughtout the year.

EARTHQUAKE HAZARDS

Southeast Asia, situated on the Southeast Asian or the China plate, is bounded by the Indian Ocean and Australian plates on the west and south, and by the Pacific plate to the east (Fig. 2). The Indian Ocean and Australian plates subduct beneath the China plate along the ocean trenches which stretch from west of the Andaman-Nicobar Islands to southwest of Sumatera to south of and finally bending northward following the Banda Arc belt. The Pacific plate subducts westward along the boundary of the Australian and Philippine Sea plates, from Irian Jaya to Sulawesi to the Mindanao Deep. The interaction of these plates causes frequent occurrence of earthquakes with complicated distribution patterns at various depths. Earthquake hazards include ground shaking, ground failures, and tsunamis. These processes may cause severe damage to structures resulting in death and injury to inhabitants. Numerous historical and recent catastrophic earthquakes have been reported in Burma, Indonesia, and the Philippines.

Earthquake records of Burma date back to 442 B.C. In that year, a great earthquake roared through central Burma; from the barren Myingyan plains southeast of Pagan rose Mount Popa, a 1,518-m high volcano, and the soil on which Sri Ksetra was later to be built rose from the sea floor. Earthquakes in Burma are associated with movements along the active Sagaing fault which underlies the Irrawaddy River valley, and along the Naga-Arakan Yoma subduction zone. Destructive earthquakes along this fault occurred in 442 B.C., A.D. 1482, 1761, 1762, 1838, 1930, 1970, and 1975. (Anderson, 1982). One of the worst earthquakes in Burma occurred on 5 May 1930. The ancient seaport of Pegu was almost entirely destroyed by the earthquake and a sea wave which followed it. The roof of the famous Shwe-Maw-Daw Pagoda, an ancient temple reported to have been entirely covered with gold and very valuable, was wrecked. Hundreds of people were reported killed or injured. Rangoon suffered heavily from the shocks. Many buildings in the European residential district collapsed with reports that 200 people were killed or injured by the earthquakes there. (Seismological Notes, 1930).

In Indonesia more than 400 earthquakes with magnitude of 4 or greater on the Richter scale have been observed annually, of these at least two are destructive. During 1976 and 1977 seven destructive earthquakes occurred in the provinces of Irian Jaya, Bali, West Sumatera and the island of Sumbawa. These earthquakes took a heavy toll of human lives: 6,302 killed or missing and 4,158 injured. (Indonesian Department of Information, 1980). Other destructive earthquakes occurred in Lombok (Lesser Sunda Islands) on 21 March 1979, in Bengkulu on 15 December 1979, in Sukabumi (West Java) on 10 February 1982, and in East Flores December 1982, and in Banda Aceh on 4 April 1983. (Hartono, pers. comm.) The Seismological Observation Network of Indonesia has established seismological and strong-motion accelerograph stations to monitor earthquakes throughout the country; seismic zonal maps and seismic loading codes are being prepared.

The earliest earthquake recorded in the Philippines occurred on 13 July 1589 when the fort in Manila was shaken by strong earthquakes resulting in cracks of more than a finger's breadth (Repetti, 1946). Several earthquakes with magnitudes 7 and 8 on the Richter scale have been recorded in the Philippines. Some of the more destructive ones are listed in Table 2. From Table 2, it can be seen that landslides, ground surface ruptures, ground settlement and spontaneous liquefaction (sand boils and foundation failures) are often associated with large earthquakes in the Philippines.

The most devastating earthquake in the Philippines occurred in Mindanao on 17 August 1976. The epicentral location was offshore in the Moro Gulf, approximately 100 km south of Cotabato City. The damaging effects of the earthquakes and tsunamis which followed were distributed over an area extending as far as Zamboanga City on the west, Davao on the east, and Pagadian on the north. Casualities are given in Table 3 (EERI, 1977). Data from the Regional Disaster Coordination Centers show that outside of Cotabato City, 80% of the casualties were caused by the tsunami. In addition, 39,850 persons were listed as homeless. Of the 6,900 houses damaged, 1,300 were in Cotabato City. Most of the substantial building damage occurred in Cotabato City, with some 30% of the commercial buildings reported to be damaged. Damage to buildings occurred most often where reinforced concrete was a major component. Most of the reinforced concrete buildings were constructed during the 1960's but were not designed for seismic lateral loading. Ground shaking was more severe in certain areas of the city than in others. Significant building damage occurred exclusively on the reclaimed swampy deltaic plain. Foundation settlements were observed at two buildings. Notable ground cracking and soil movements or failures were also seen. Numerous superficial landslides were observed on steep hillsides. A number of small mud boils (60 to 90 cm across) were reported on marshy ground. Differences in damage to structures built on a weathered limestone rise called Colina Hill, and that occurring on the low-lying reclaimed land of the city proper, were pronounced. The former enjoyed essentially complete absence of damage to physical facilities. (EERI, 1977.)

Tsunamis due to the Cotabato City earthquake reportedly reached heights of 4.6 to 6.0 m and travelled inland 1 km. Tsunamis were responsible for 85% of the deaths and 65 percent of the injuries. Of those thought missing, 95% were associated with the tsunamis. Coastal fishing villages took the brunt of the waves. Huts were destroyed, boats were overturned, displaced, or carried out to sea (EERI, 1977).

Table 2.--Significant Earthquakes of the Philippines (after Rapetti, 1946 and EERI, 1977)

Date	Location	Damage
June 21, 1599	Manila	Violent earthquakes with damage to many buildings.
June 16, 1601	Manila	Destroyed and damaged many houses and churches, death of some people, many injured.
December 1621	Panay Island	No one escaped. Many rivers changed their courses. Landslides.
September 1627	Northern Luzon	Mountain was levelled.
1635	Manila	A number of houses demolished. Three thousand persons killed.
November 30, 1645	Manila	Most of the buildings ruined. Many persons were killed.
March 1675	Luzon and Mindanao	Fissures in mountain. The sea entered and flooded an extensive and fertile plain.
1966	Manila	The city was almost entirely destroyed.
July 1787	Panay Island	Only two stone churches remained standing. Fifteen persons killed.
March 22, 1840	Luzon	Seventeen persons killed two hundred injured. The whole neighborhood sank five feet below its former level. Buildings destroyed.
December 8, 1871	Lanao, Cotabato, Davao	Not a single building standing in Cotabato or Pollock.
August 25, 1874	Zamboanga	Damaged to masonry buildings, overturned walls. Large fissures opened near beach.
September 16, 1878	Gulf of Davao	Government House at Davao wrecked.
June 21, 1893	Davao	Bamboo and wooden houses destroyed. Cracks in ground. Subsidence.
June 31, 1917	Cotabato	Mud spouts several meters high. Landslides. 1 m tsunami. Seven killed.
April 1, 1955	Lanao	Four hundred dead. Landslides and seiche on Lake Lanao. Quays and bridges destroyed.
August 17, 1976	Gulf of Moro	4,000 to 8,000 dead, 5 m tsunami, several buildings destroyed.
August 17, 1983	Laoag City	Several buildings wrecked. Sand boils up to 8.5 m diameter. Landslides. Ground cracks and settlements. Several lives were lost.

Table 3 Casualties and Number of Persons Rendered Homeless by the Mindanao Earthquake of 1976 (after EERI, 1977)

Affected Areas	Casualties									Homeless Families
	Dead			Missing			Injured			
	EQ*	TS*	Total	EQ	TS	Total	EQ	TS	Total	
Region IX										
Zamboanga City			153			15			151	97
Zamboanga del Sur			563			521			4,110	818
Zamboanga del Norte			-			-			-	-
Pagadian City			418			29			2,500	3,980
Basilan City			30			6			10	129
Sulu			89			107			15	25
Region XII										
Lanao del Norte			80			162			2	1,488
Lanao del Sur			561			89			273	879
Maguindanao			1,198			429			645	2,761
Sultan Kudarat			305			51			106	1,081
North Cotabato			-			-			1	50
Cotabato City	110	57	167	0	93	93	422	21	443	879
Total 3,564			1,502			8,256			12,183	

*EQ = earthquake; TS = tsunami

Tsunami

Tsumanis are solitary sea waves that are caused by sudden vertical movement of a large area of the sea floor during an undersea earthquake or submarine volcanic eruption. Tsunamis have produced great destruction and loss of life in Southeast Asia. The chronological catalogs of tsunamis in Indonesia and in the Philippines reveal 132 tsunami events in Indonesia since 1629, and 56 events in the Philippines since 1735 (Nakamura, 1979 and 1978). The greatst tsunami ever recorded in the Philippines was 6 m high, associated with the Mindanao earthquake of 21 September 1897. The most destructive tsunami in Southeast Asia, and probably in the world, occurred during the Krakatau catastrophe in 1883; this caused the death of the over 30,000 people living in the coastal areas of South Sumatera and the western part Java. Tsunamis from the Krakatau eruption were reported as far west as the Atlantic Ocean.

The probability of tsunami risk in the Philippines and Indonesia, in relation to wave height and recurrence period, are shown in Fig. 5 (Nakamura, 1978, 1979).

VOLCANIC HAZARDS

Volcanic eruptions have been a major threat to human life and property throughout the histories of Indonesia and the Philippines. Volcanic hazards include volcanic explosions and ash falls (tephras), hot avalances (ladus), mudflows (lahars), and lava flows. In addition volcanic eruptions can produce earthquakes, tsunamis, floods and landslides.

The Volcanological Survey of Indonesia was established in 1922 with its main task to minimize catastrophical effects caused by volcanic eruptions. Its headquarters are in Bandung where it engages 60 volcano observers and 200 other volcanologists and technicians. Hazard maps of volcanic areas of Indonesia have been produced. The Bulletin of the Volcanological Survey of Indonesia is issued annually. The Philippines set up the Commission on Volcanology in 1952. In 1982 the Commission was reorganized and renamed the Philippine Institute of Volcanology. It also has standing teams of volcano watchers to monitor the activity of the 37 active volcanoes.

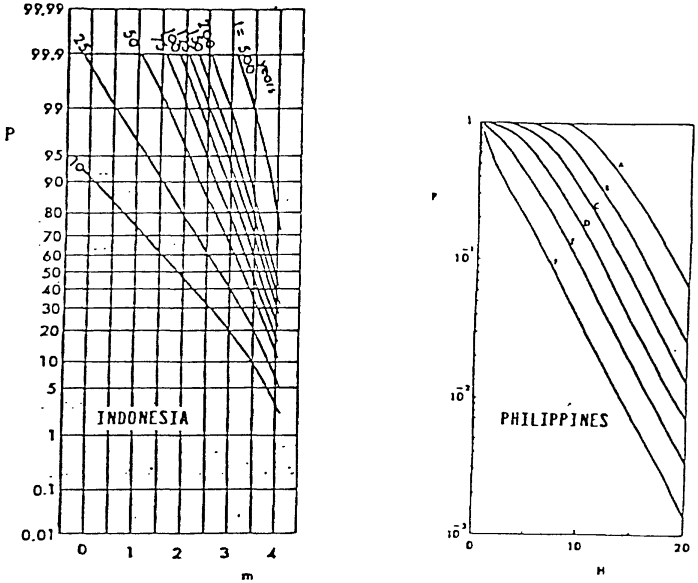

Figure 5. Statistical tsunami risk in Philippines and Indonesia Curves A, B, C, D, E and F are for T = 500, 200, 100, 50, 25 and 10 years respectively. (P and H are exceeding occurrence probability and wave height in meter). (After Nakamura, 1978 and 1979.)

Explosive Eruptions

The greatest volcanic explosion of historical times was the eruption of the Tambora volcano in Sumbawa Island during 5-12 April 1815. The initial explosions were heard at distances up to 1,400 km. During the most intense phase of the eruption, dense ash clouds caused complete darkness for three days on the island of Madura, 500 km away. It was estimated to have thrown up some 100 cu km of volcanic products and cost 92,000 lives, which is more than one-third of the total number of victims of all historic outburts (236,000 as of 1949). Some 10,000 people were killed in the eruption, and an estimated 82,000 died of starvation and disease in the months following the eruption (Van Bemmelen 1970). The climatic effects of the huge quantity of dust ejected into the atmosphere were apparent in many parts of the world. The year of 1816 was unusually cold the world over and commonly referred to as the "year without a summer."

Another great volcanic explosion occurred at Krakatau, an island in the Sunda Strait between Java and Sumatra. On 26 August 1883, a series of tremendous explosions and tsunamis occurred. Ash clouds rose to a height of 80 km and the explosive noise was heared nearly 5,000 km away. After the eruption, two-thirds of the island had disappeared. Tsunamis travelled at a speed exceeding 500 km an hour westward across the Indian Ocean and into the Atlantic. Tidal disturbances were recorded as far as at ports on the Bay of Biscay. Deaths from tsunamis were reported even in Sri Lanka where a woman was tumbled about in 1-m waves and died of her injuries (Furneaux, 1964). After it was all over, a total of 36,417 people had died, 165 villages and towns were destroyed, and another 132 were seriously damaged (Furneaux, 1964).

Recorded eruptions of Indonesia's volcanoes in recent decades are Merapi (1978) and Dempo (1974) in Sumatra; Anak Krakatau (1978, 1979) in Sunda Strait; Butak Petarangan (1979), Semeru (1978, 1979), Raung (1978), Merapi (1976), and Galunggung (1982) in Java; Rokatenda (1978) in Paluweh; Lokon Empung (1978, 1979) in Sulawesi; Karangetang (1978, 1979) in Siau; and Dukono (1978) in Halmahera.

Of 37 active volcanoes and solfataric areas in the Philippines, Mayon volcano is the most active and has had periodic eruptions during the last 300 years. The most violent and destructive eruption occurred on 1 February 1814. A torrent of fire, lava, and large hot chunks of rocks rolled down the southern slopes destroying everything in its path. The towns of Camalig, Cagsaua, Budiao, and half of Albay and Guinobatan were laid in ruins and about 1,200 persons perished (Commission on Volcanology, 1981).

Destruction wrought by Mayon is not limited to eruptions. There are on record great disasters attributed to floods which caused volcanic materials to roll down to the plains and on to the sea after the manner of mudflows, only with greater mobility and rapidity. The first disaster of this kind on record occurred on 23-24 October 1776 when it was estimated that thousands of people perished and much property was destroyed. In November 1875 another flood caused heavy destruction and the number of victims was estimated at 1,500. Other floods that damaged property around the volcano took place on 27 July 1853 and in 1915. Recently, destruction occurred on 30 June 1981 at the height of a typhoon: 40 persons died; nine were injured; and several others were missing. The mudflow laid waste to property worth millions of pesos (Commission on Volcanology, 1981).

Hibok Hibok and Taal volcanoes are two other volcanoes in the Philippines which have caused destruction to villages and the loss of many lives. The Hibok Hibok eruption of 4 December 1951 produced red hot avalanches, clouds of stifling gas, and hot dust which enveloped villages north of Mambajao and took the lives of 500 persons. Houses were burned and bodies of persons and animals were charred and mummified (Macdonald 1972). Ten eruptions of the Taal volcano were listed before the beginning of this century, the largest of which was in 1754 when the old towns of Taal, Lipa, Tanauan and Talisay were all destroyed. On 28-30 January 1911 there was another tremendous explosion hurling up glowing rocks, earth, mud, ashes and gases. There were 1,300 deaths in 13 small villages. On 28 September 1965 another eruption claimed about 200 lives. Other eruptions of Taal took place in 1966, 1969, 1970, and 1976 (Commission on Volcanology, 1981).

Hot Avalances and Mudflows

A lahar is a mudflow containing debris and angular blocks of volcanic rocks. Several types of lahars can be distinguished. Normal lahars (cold mudflows) originate from heavy rainfall on slopes covered with loose material or by earthquakes. Such lahars of the Merapi in Central Java attained a length of 25 to 30 km. One wet lahar of the Raung volcano in Central Java was 40 km long (Neumann van Padang, 1939). The lahars caused by the emptying of a crater lake belong to the volcanic group. The throwing out of large lakes by explosion causes hot mudflows. When incandescent flows of volcanic debris rush down the slopes of volcanic cones, they are called ladus. These ladus are avalanches of lava fragments mixed with sand and dust, originating from active domes or lava flows.

A well-known example is provided by the rock slides and glowing clouds on the flanks of the Merapi volcano in Central Java during 18-19 December 1930. A series of hot avalanches cam down the flanks of the volcano and reached a length of 13.5 km. The glowing clouds caused the death of 1,369 persons and destroyed many villages. Each year several thousand lahar slides, and occasionally a few ladu slides of the Merapi volcano have occurred. Villages have been destroyed and scores of people killed (see Bulletin of the Volcanological Society of Indonesia for the period 1950-1957).

Kelud volcano, also on Java, periodically ejects its crater lake causing mudflows that have taken thousands of lives. In the eruption of 1919, 130 sq km of farmland and 104 villages were destroyed, 5,110 lives were lost (van Bemmelen, 1970).

The Galunggung eruption of May-December 1982 produced 22.27 x 10^6cu m of pyroclastics and 2.98 x 10^6cu m of ash flow deposits and some lava that remained within the crater. At least 50 percent of the loose material will eventually be converted into volcanic mudflows or lahars. Immediately after the first eruptions in April 1982, lahars destroyed several villages at 5 to 8 km from the crater. The lahar danger will still be present for several more years. Embankments up to 7 m high have been built to keep the lahar within desired courses; a few of the embankments have failed and have had to be rebuilt (Tjia, Pers. Comm.).

GROUND FAILURES

Hazards from ground failures include landslides, problem soils, and land subsidence.

Landslides

Catastrophic and destructive movements of rock and soil down a slope are known as landslides. Some landslides involve only the unconsolidated debris lying on bedrock, others involve movement of the bedrock itself.

The Territory of Hong Kong consists of a number of small islands and a portion of Chinese mainland. The population approaches 6 million with an area of only 1,050 sq km. The terrain is rugged with many hills rising steeply from the sea to heights exceeding 400 m. An indication of the scarcity of land is the fact that about one half of the flat terrain has been reclaimed from the sea. Hong Kong has steep slopes of deeply weathered rocks and colluvium mantles which are prone to landslides during the heavy seasonal rain storms. Landslides are a continual problem in Hong Kong, and these have caused heavy loss of life and major property damage. On 12 June 1966 the heaviest rainfall ever recorded fell on Hong Kong Island; serious and extensive flooding ensued. Stream courses and sewers were blocked. Old huts and houses collapsed. A great number of vehicles were washed down hillsides or buried in mud. Altogether more than 500 landslides and washouts were reported. The quantity of earth and debris cleared later amounted to over 100,000 tons. The damage included 64 fatalities, 2,672 people homeless, 8,561 people temporarily evacuated, 407 houses and huts destroyed or damaged, and 31 million dollars worth of damage to roads and other services (Public Works Dept., 1966).

On 16-18 June 1972 another record rain fell on the Island. Two major landslides occurred; 138 lives were lost. Blocked and washed-out roads disrupted communication and isolated a number of areas. In 1982 two major rainstorms occurred in May and August causing over 1,500 landslides. Another major storm in June 1983 caused more than 150 landslides (Brand et al., 1984).

Problem Soils

Problem soils are soils which tend to cause trouble due to changes in moisture content, or due to their having been disturbed. These include dispersive soils, expansive soils, and sensitive soils.

Dispersive soils are prone to rapid dispersion or deflocculation in water. Consequently, they are highly susceptible to erosion or to piping when subjected to slow moving water, or even quiet water, by individual soil particles going into suspension.

Since 1950, a number of earth dams in northeastern Thailand whose embankments were constructed with dispersive soils have suffered serious piping and surface erosion from rainfall; six dams failed during the first season of reservoir filling.

Lam Sam Lai Dam is located in Pak Thongchai District, Nakhon Ratchasima Province. The dam has a maximum height of 14 m, a crest length of 2.5 km, and a capacity of 986,000 cu m. Both the central core zone and the shoulder zones on either side consist of sand, silt, and clay materials. The dam failed by piping during the first filling in 1953, and a section of embankment about 50 m wide was washed out. Since then, the embankment has been subjected to rainfall and very severe tunnel erosion has occurred. Piping holes can be found in the lower portion of both faces and the cavities behind these holes are 3 to 4 m wide. This piping was caused entirely by rain erosion, as the reservoir has remained virtually empty (Fernando, 1979).

Expansive soils are derived from decomposition of shales or volcanic ash which contain clay minerals, such as montmorillonite, that when wet swell to as much as 1.5 to 2 times their original dry volume. The saturation of such soils from rainfall can cause major damage through expansion of soils beneath the structures.

Expansive volcanic soils are found in Indonesia and the Philippines. In Thailand, the expansive soils are derived from weathered shale. A highway section was built across this expansive soil east of Mae Sariang District, northwestern Thailand, in 1966. During every rainy season after the highway was completed, the subsoil swelled and the pavement cracked and heaved to as much as 30 cm. No record of the recurrence was kept, but since 1970 the Thai Highway Department has had a permanent camp with a bulldozer standing by for road clearing operations. X-ray analysis of the subsoil indicated high percentages of montmorillonite and vermiculite (Guo, 1976).

Sensitive soils are clays that undergo a severe loss of strength after slight disturbance of the soil fabric. The sensitivity is expressed as the ratio of the undrained shear strengths of undisturbed and remolded soil. In normal clays the sensitivity is less than 4; moderately sensitive clays have sensitivities of 4 to 8; highly sensitive clays have sensitivities exceeding 8.

Soft Bangkok clay is notorious for its sensitivity and high compressibility. The sensitivity of this clay was not detected until a major gas pipeline project was commenced in 1978. When the trench was excavated in this soil, the soil became liquefied. The pipe lines, which were supposed to be buried, floated upward. Concrete anchors were built to hold down the pipeline, at an extra cost of several million dollars. The sensitivity of this clay varies from 14 to more than 20. Other soft marine clays in Southeast Asia, such as those in Manila and Jakarta, are suspected to exhibit similar characteristics.

Subsidence

Where water soluble rocks such as limestone, dolomite, gypsum or salt are at or near the ground surface, those areas are prone to development of karst and dissolution features from the dissolving action of circulating groundwater. The sink holes and caverns thus developed are potential hazards owing to possible settlement or collapse of the land surface into the underground openings.

The eastern part of the city of Kuala Lumpur was built on the Kuala Lumpur limestone. Several underground cavities were detected during the foundation investigation for large buildings in the city. In August 1983 a section of the main turnpike south of Kuala Lumpur collapsed; subsequent drilling revealed the presence of sinkholes.

A major part of the Khorat plateau of northeastern Thailand is underlain by a rock salt sequence. The underground salt has been commercially extracted by pumping out the groundwater brine through drilled wells and evaporating in solar ponds. In Borabue District, brine extraction commenced in 1967. By 1980 the annual salt production from this underground brine pool reached 100,000 tons. It is estimated that between 1967 and 1980 more than 1,150,000 tons of salt representing a volume of 576,000 cu m of salt was recovered. The effect of brine production resulted in cavities at shallow depth and six sinkholes developed. A total area of 750 sq m was affected by the subsidence (Rau & Nutalaya, 1982). The effect of the salt operations, careless disposal of waste saline water, improper storage of salt, overflowage from evaporation ponds, and leaking casings in the salt wells contributed to an increase in salinity in the nearby fresh-water reservoir and its 200 km downstream channels. In early 1980 the salt concentration in the supposedly fresh-water reservoir reached 25.5 ppt. All varieties of fish (except an imported species *Tilapia*) and other forms of aquatic life disappeared.

The most hazardous land subsidence in Southeast Asia is caused by the excessive withdrawal of groundwater and the consequent decline of piezometric levels in aquifers. Several major cities in Southeast Asia (Bangkok, Jakarta, Surabaya, and Manila) presently face this problem. Bangkok is currently sinking at average rates of 5 to 10 cm per year due to the reduction in pore pressure and consolidation of the clay and sand layers. The subsidence bowl covers a 700 sq km area, the center of which is now 50 cm below mean sea level (Fig. 6). The subsidence has made the drainage problem more acute and flooding more severe. In addition, damage to structures due to differential settlement is prevalent. It is estimated that it will cost at least US $1,000-million to arrest the subsidence and control the flooding (AIT, 1982).

HYDROLOGIC HAZARDS

Severe floods that damage agriculture and human settlements over extensive areas in Southeast Asia are the consequence of heavy rainfalls from tropical storms and cyclonic depressions during the monsoon period. When there is heavy rainfall in the upper catchment areas, river volumes increase correspondingly, and the rivers overflow their banks and water covers the floodplain. The damage in terms of crops, cattle, property and human life can be seen in Table 4.

Table 4 Flood Damage Data of South and Southeast Asia for the Year 1954 to 1957 (after ESCAP, 1977)

Name of country	Area affected		Population affected	Damage to crops			Damage of houses and property		Cattle head lost	Human lives lost	Damage to public utilities
	Sq.km	Sq.miles		Hectarea	Acres	Value	No.	Value			
						(millions)		(millions)			(millions)
Burma	-	-	-	3,600	9,000	-	-	300	400	-	-
Celon	-	-	3,000	181,817	454,543	Rs.54.0	1,390	Rs.0.046	-	6	Rs.60.378
India	-	121,781	59,500,000	-	17,301,000	Rs.1,766.9	3,009.656	Rs.622.2	121,651	2,084	Rs.158.50
Indonesia	-	-	135,677	222,399	502,875	Rupiah54.27	31,376	-	-	266	-
Pakistan and Bangladesh	93,177	36,154	2,541,500	3,132,779	7,854,083	-	449,521	Rs.660	102,916	1,390	Rs.51-56
Philippines	-	52,960	351,615	-	-	-	40,798	-	-	21	Pesos4.8

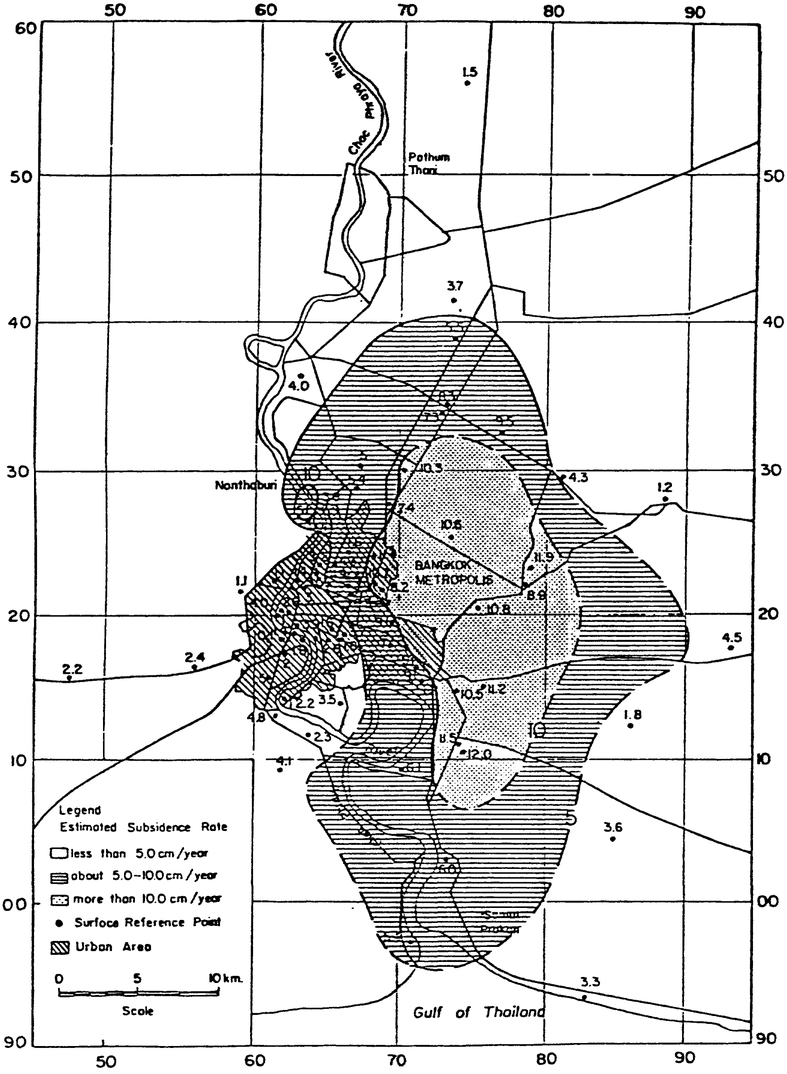

Fig. 6 Subsidence in Bangkok (cm/year)
(after AIT, 1982)

Floods

Bangkok, a city of 5.5 million people, is situated on the floodplain of Chao Phraya River 25 km north of the Gulf of Thailand. The ground elevation of Bangkok is about 0.5 m below to 1.5 m above mean sea level; consequently, heavy monsoon rains falling May and October quickly flood major parts of the urban area. During the last four months of 1983, Bangkok and its environs were devastated by the worst floods in living memory. The true cost in hardship and money is incalculable.

Going through the order of events in newspapers is like reading the same play over and over again with only minor changes in lines. On 4 April 1983 the Bangkok Metropolitan Administration issued statements that steps had been taken to improve the drainage system, dikes had been constructed and pumps installed to pump out the excess water. Assurances were given that Bangkok would not be flooded. On the first weekend in August it rained continuously and extensively. Residents in low lying areas first noted that the water had begun to rise. On 20 August, over 100 mm of rain fell. Water was everywhere in Bangkok and the adjacent provinces. Over 2,000 sq km of the rice paddies were damaged. Stretches of roads and highways were inundated. On the first day of September there was another more deluge. Police in Bangkok said the disruption to traffic was the worst in recent memory. Dozens of roads were under 50 cm or more of water. As days passed, the water kept rising as the rain kept falling; the situation became worse. Thousands of Bangkok workers were rolling up trousers or hitching skirts and getting to work three or four hours late. By 5 September there was a meter of water in the eastern part of Bangkok. Boats were ubiquitous as the only practical means of transport. As October began, the Governor of Samut Prakan, a city south of Bangkok, said: "We are living in a sea of water, and the people have lost hope that the situation will improve." Heavy rains continued and conditions worsened. Train services to the north were disrupted. Twelve highways were impassable in the northeastern and central regions. Public transport in Bangkok became almost non-existent. The list of roads either impassable or dangerous to use was longer and longer. The Minister of Public Health said that about 124,000 Bangkok residents had contacted water-borne diseases since the flooding began. November began with the estimates of flood damage suffered by the private sector as much as US $500-million. High flood waters in the Chao Phraya River prevented many vessels from passing under bridges across the river. On 6 November, high tides brought havoc to the areas near the river, and 30 major roads were affected. Heavy downpour on 11 November kept the floodwater levels high in almost all areas of Bangkok. A bus driver said it took him 12 hours to cover 20 km of distance. On 12 November the Bangkok Post headlined "floods hit back with a vengeance" with most city streets swamped and the main road to the South closed. The full cost of damage from this flood has yet to be assessed, but it is possible to enumerate a few specific items of the damage. A total of 66 persons lost their lives from electrical short circuits or drowning in flood water; 3,000 sq. km of farm land was damaged; 104 government buildings, 125 schools, 108 temples, 507 bridges and 4,798 km of highway were damaged. The cost of loss in production and the loss of potential investment is far too great to be calculated (Bangkok Post, 1983).

Storm Surges

Countries bordering the Indian Ocean are confronted with one of the most serious storm surge problems of any place on the earth. This is particularly true in the northern Bay of Bengal, where an unique combination of a large astronomical tide, a funneling coastal configuration, low flat terrain, and frequent severe tropical storms occasionally produce storm surges that kill thousands of people. This is dramatically illustrated in Table 5 where the number of deaths associated with several noteworthy tropical cyclone or hurricane tragedies are listed.

On 12 November 1970, a tropical cyclone of moderate strength riding the crest of high tide lashed Bangladesh with a 20-ft storm surge which killed approximately 300,000 people (Fig. 7). The official figures show 200,000 confirmed burials and another 50,000 to 100,000 missing.

Aside from the indescribable human misery associated with the horrendous loss of life, the repercussions from this cyclone presented a formidable challenge for the survivors. Table 6 lists damage suffered in several categories; the losses sustained by the fishing industry can be used to indicate the seriousness of post-storm conditions. Nearly 90 percent of the

Table 5 Deaths Associated with Tropical Cyclone in the Bay of Bengal (after Frank and Husain, 1971)

Year	Location	Deaths
1970	Bangladesh	300,000
1737	India	300,000
1897	Bangladesh	175,000
1976	Bangladesh	100,000
1864	India	50,000
1833	India	50,000
1822	Bangladesh	40,000
1839	India	20,000
1789	India	20,000
1965	Bangladesh	19,279
1963	Bangladesh	11,468
1960	Bangladesh	5,149

Table 6 Damage Estimated in the 12 November 1970 Cyclone (after Frank and Husain, 1971)

Population affected	4.7 millions
Crop loss	$63 millions
Loss of cattle	280,000
Loss of poultry	500,000
Houses damaged	400,000
School damaged	3,500
Fishing boats destroyed (marine)	9,000
Fishing boats destroyed (inland water)	90,000

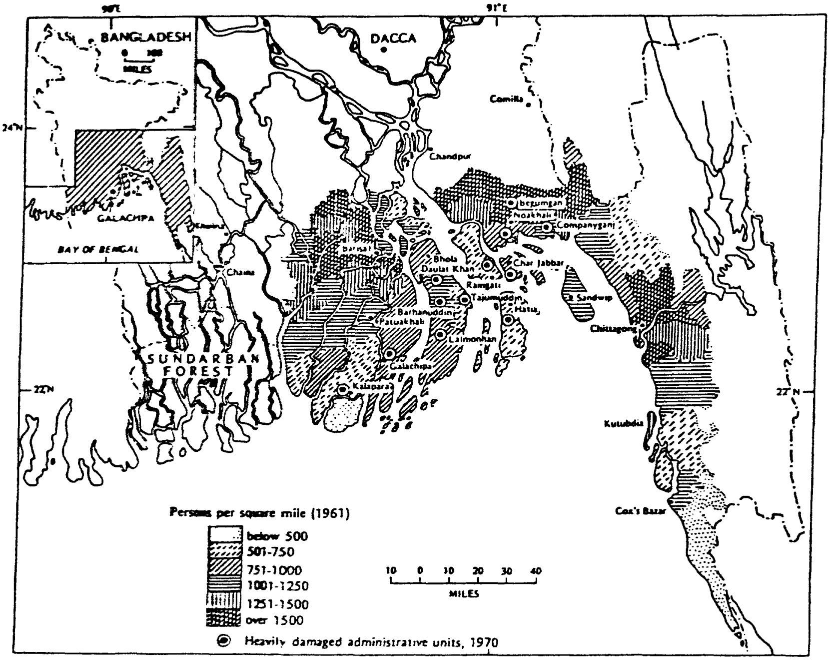

Fig. 7 Coastal areas of Bangladesh. Population Density and Major Cyclone-affected Areas, November 1970 (after Islam, 1974)

marine fisherman suffered heavy losses, including the destruction of 9,000 offshore boats. Of the estimated 77,000 inland fisherman operating in the cyclone-affected region, approximately 46,000 lost their lives, and damage was severe to the surviving 40 percent. It is estimated that approximately 65 percent of the total annual fishing capacity of the coastal region of Bangladesh was destroyed by the storm. The full impact of this loss becomes evident when it is realized that 80 percent of the total per capita intake of animal protein by Bangladesh's 73,000,000 residents comes from fish.

Even though the Government of Bangledesh has improved the Cyclone Warning Service, there remain some critical deficiencies. There is a limitation on the effectiveness of the observing facilities to distinguish between killer and non-killer cyclones. As a result, people have been drastically over-warned, and this breeds apathy. Time after time, coastal residents have been warned of "Great Danger" (the highest degree of danger), then found that conditions were not nearly as bad as expected. It is estimated that over 90 percent of the people in the disaster area knew about the storm; yet less than 1 percent sought refuge in more substantial buildings, and most of those who did were not land or homeowners. (Frank and Husain, 1971).

Waterlogging and Salinity

When groundwater levels have risen sufficiently for the capillary fringe to reach the surface, or for the water table to reach the root zone of agricultural crops, the top soil becomes saline and agricultural productivity begins to decline. This phenomenon is called waterlogging and salinity.

Large agricultural land areas along the Indus Plain of Pakistan and on the Khorat plateau of Thailand are waterlogged as the result of irrigation. It also appears that soil salinity has increased and spread with the growth of waterlogging in the form of the accumulation of salts at the ground surface. The deterioration of land through waterlogging and salinity has resulted in losses in agricultural products, losses of previous investments in irrigation and land development, losses of investment in infrastructure, and ultimately has necessitated new investments to rectify such losses.

The fertile Indus Plain of Pakistan, and the great rivers that water it, are together one of the major natural resources of the world. The surface water of the five rivers that cross the plain amounts to 140 million acre-feet, of which some 83.5 maf are diverted to a highly developed irrigation system, irrigating some 23 million acres of land which represents about 60 percent of Pakistan's cultivatable land (Soomro, 1975). There are 61,000 km of canals and about 60,000 outlets for water courses. Huge quantities of water seeping from the canals and water courses have greatly raised the groundwater levels; large gracts of land are now waterlogged and agricultural production is falling rapidly. Salinity of the soils has also increased and salts accumulate at the ground surface (Fig. 8). Pakistan which once had a surplus in its food production, now has to import hundreds of thousands of tons of grains to feed a population which increases at the rate of 3 percent a year. The Government of Pakistan has already spent about US $540-million in its anti-waterlogging and salinity program during the last decade, but the solution is nowhere near.

CLOSURE

Losses due to geologic and hydrologic hazards can be mitigated. A variety of actions for loss reduction measures including avoidance, stabilization, protection, warning, land-use planning and engineering design can be implemented. For some types of hazards, the development of new and better measures is needed, for others the application of present state-of-the-art procedures to mitigate the losses will suffice. What is of paramount importance in any hazard mitigation, especially in developing countries, is the recognition by the decisionmakers and planners that the hazards do exist and that they can be mitigated. Of equal importance in any endeavor is that there must be an unabated determination to resolve the matter. Once the commitment is made, science and technology can play their roles.

An example of a hazard mitigation measure is given here to demonstrate what is needed to effectively reduce the losses. On 1 February 1953 a spring tide of immense force driven by a North Sea gale engulfed large areas of southwestern Netherlands. About 8 percent of the country was inundated and nearly 2,000 lives were lost. The material loss and the damage inflicted were incalculable. The existing dikes had not been built to withstand such a

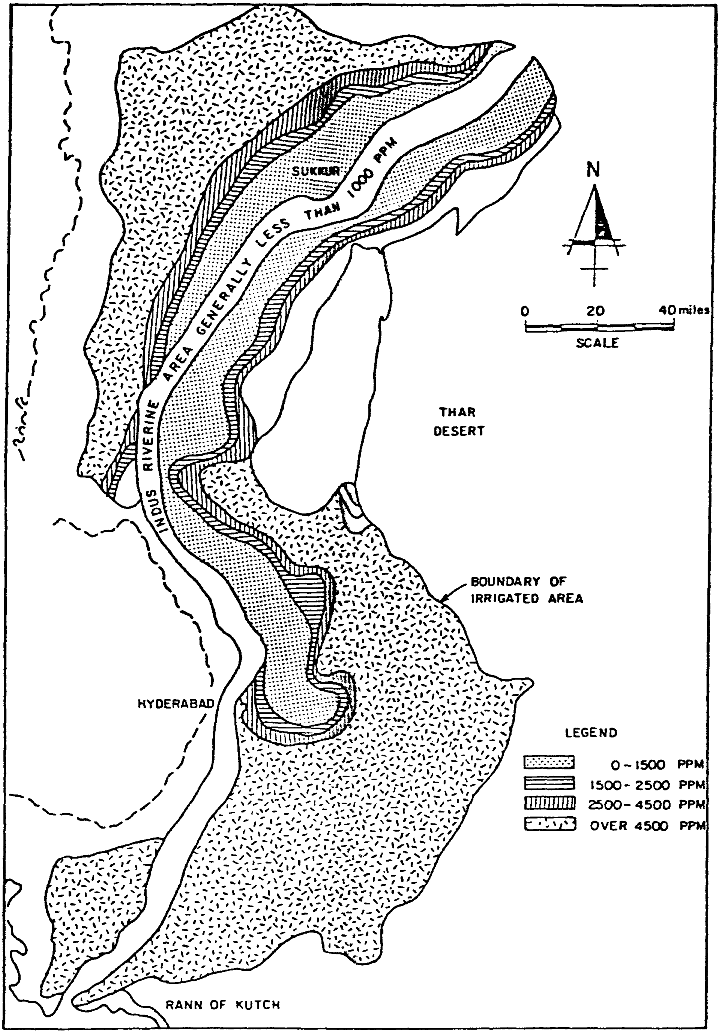

Fig. 8 Sketch Map Showing Groundwater Quality in the Southern Indus Plain. (after BAKIEWICZ, 1969)

disastrous onslaught. The Dutch Government decided to prevent, once and for all, the recurrence of a similar catastrophe. A radical and daring plan was conceived to build huge dams across all tidal estuaries and inlets in the southwest, with the exception of those giving access to the ports. In addition, all coastal defenses in the rest of the country were to be strengthened. In all, 1,000 km long dikes are to be raised and reinforced. Toward the end of this century the dikes will have been made sufficiently high and strong to reduce the probability of flooding to about once in 10,000 year (Voskuil, 1982). The project is now in its 30th year and work continues on schedule.

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MAPPING

OVERVIEW OF NATIONAL MAPPING DIVISION MISSION

By

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A warm welcome from all of us in the National Mapping Division. I sure hope none of your group failed to get to this gathering because the map you were using was inaccurate or incomplete. By the way, if any of you have a map that shows you how to get around in this building, please see me after the program--we would like to get a copy.

As you probably suspected, the National Mapping Division (NMD) is in the business of making maps. The word "map" in itself is not very definitive. If someone told you they had a map showing Denver, about all you could be fairly sure of is that they are referring to a graphic chart that uses some kind of symbols to show the spatial relationships between selected features. The map showing Denver could be a small-scale world map that shows Denver as a speck in its relative position to the continents and oceans. On the other hand, the map could be a large-scale map showing all streets, alleys, and buildings in Denver but with no indication where Denver is relative to the rest of the State, country, or world. Road maps are designed to show just enough detail to tell the user how to get from one place to another. Engineering maps are concerned with near-exact dimensions. A thematic map addresses any subject that can be shown in a spatial relationship to itself or to other spatially related places or features. Maps can be designed to satisfy almost any degree of accuracy or content you desire.

The NMD has been charged with preparing base maps for the United States. The 1:24,000-scale, 7 1/2-minute quadrangle topographic series is scheduled for completion for the conterminous 48 States plus Hawaii during this decade. The 1:63,360-scale base maps for Alaska are also nearing completion. The specifications for these general-purpose base maps represent a compromise that addresses the public need for base map coverage, while falling short of engineering map requirements. The amount of detail that can be shown is limited by the map scale and the desire to produce an easily readable map without excessive clutter. While our 7 1/2 minute quadrangle base maps are the most accurate and complete general-purpose maps available, they would be awkward to use for regional studies that could take dozens of maps. Of course, if you have a very large plain wall in your office on which to assemble the maps, they could be just what you want. Consequently, several derivative maps such as the 50,000-scale; 100,000-scale; and 250,000-scale maps are also prepared to address the needs of larger area studies. Our base maps are designed to be general-purpose maps that will satisfy a large segment of the users. More precise, larger-scale engineering maps are considered the responsibility of private industry or of other agencies requiring site-specific detail.

Our standard base maps include a transportation network, a drainage network, boundary network, public land lines, and a grid coordinate system--any or all of which can be used by map users, like yourself, as a framework to add or plot additional map data. Some data, like the fallout from the Mount St. Helens eruption, can be sketched on a regional map from data collected at a family of test sites. The size and shape of a drainage basin can be determined from topographic base maps. Computer-modeled 100- or 500-year floods can help define the potential for life and property losses. Digital cartography is opening the doors to unlimited scientific modeling and an explosion of knowledge for all spatially related studies. However, very detailed ground surveys may be required to map hazards relating to ground subsidence and cave-ins due to underground mining or local landslides. The National Mapping Division (NMD) has been involved in monitoring ground movements that may signal a pending earthquake or a volcanic eruption, as in the Mono Lake region in California. Both ground surveys and photogrammetric surveys have been used to determine the earth movements resulting from natural and man-induced changes in topography, such as accompanied the Hebgen Dam, Montano and Anchorage, Alaska earthquakes; Houston, Texas, subsidence; and the Mount St. Helens, Washington eruption.

The second, and equally important role of base maps is their use as background for thematic map information, to help the reader visualize the relative location of the theme. An outline showing an area subject to inundation during a 100-year flood would be nearly meaningless without a least some parts of the base map as a reference to establish spacial relationships.

I have mentioned the NMD's 7 1/2-minute base map series and several smaller-scale derivative maps. We also have made orthophotoquads in 7 1/2° units for many areas, especially in the western part of the United States. We are presently making 1:63,360-scale orthophotoquad maps in cooperation with the State of Alaska and other Federal Government agencies there. The orthophotoquad looks like a photo but it can be treated like a map, since the perspective distortions due to relief and camera tilt have been removed. The user can measure distances between any two photo images, just as you can between the symbolized features of a graphic line map. The orthophotoquad provides you a far more complete and detailed framework on which to plot your thematic data, especially in areas where drainage and cultural features are sparse. If you have delineated your theme information on a perspective aerial photograph, the transfer of that information to an orthophotoquad is relatively simple. Transfer of the same information to a graphic line map may require stereo photogrammetric techniques. An orthophotoquad is a far less expensive map product than the conventional line graphic and in many cases, it will probably serve your needs even better. Other photoimage products should also be mentioned. The orthophoto map is an orthophotoquad that has been graphically enhanced to highlight selected features. Our production of orthophoto maps to date has been very limited. You may also encounter a photo map, which is a graphically enhanced photo perspective. The NMD does not make photo maps; they are mentioned here only as a caution that photo maps will not satisfy the normal spacial fidelity expected from map products. The NMD is in the process of establishing a digital cartographic data base that will eventually include all graphic data now shown on our map series in digital form. The primary responsibilitv for including thematic data, such as your hazards data, will rest with the originating or user offices. The advantages of digital cartographic bases and computer modeling will undoubtedly be addressed many times during your meetings here. Much remains to be done toward standardization of data formats, computer programs, coordinate systems, etc., so that data collected for any special purpose can easily be made available to other users.

Geodetic control lists represent some of the earliest digital cartography, starting long before the term "digital cartography" was coined. Now we define points and lines in a raster or vector notation and call them Digital Line Graph (DLG) data. When we define a surface with a gridded array of elevations, we refer to it as Digital Elevation Model (DEM). The term "Digital Terrain Model" is used interchangeable with DEM. Photoimagery may be digitized as a raster array, with intensity, reflectance, density, color, or other distinguishing variables as the pixel value. If we bring the various types of digital cartographic information into a common coordinate system and internally register the map features, we can then write algorithms for the computer to search, compare, and massage the data to create tables, graphs, or other useful reports.

The NMD has also had a role in thematic mapping, primarily for the Geologic Division and Water Resources Division of the U.S. Geological Survey. Our Thematic Mapping Unit provides the cartographic support of preparing the thematic maps for publication. Nearly always, these thematic maps are prepared on NMD base maps or mosaics. Other map products of the NMD worth mentioning include our State base maps, National Park maps, and maps showing bathymetry in our coastal areas.

The specifications for our standard base map series have remained basically unchanged since their inception. With technology continually changing, we have had to find ways of converting these technological advances into economic savings. I'll just touch on some of the major advances. As recently as the 1940's, the Topographic Engineer would go to the field with an alidade, a plane table, a board, and a blank sheet of paper, spending the entire season mapping on one quadrangle. Photogrammetry came along and allowed us to use aerial photography to construct the map in the office after considerable fieldwork had been accomplished to provide the control necessary to set up the stereomodels. In the early to mid-1950's, scribing on scale-stable material replaced the pen-and-ink on paper as the mapping medium. By the late 1950's, electronic distance measuring equipment totally changed our concept of establishing map control. It was no longer necessary to climb the high peaks carrying heavy equipment. Now with helicopters, the remote areas and many mountain peaks can easily be reached. Environmental considerations have limited our utilization of helicopters in some wilderness areas. By the mid-to-late 1960's, computers ushered in the semianalytical aerotriangulation and, later, the fully analytical aerotriangulation eras. Lear jets and U-2's offered the mapping community the option of flying at very high altitudes. This was about the same time that orthophoto differential rectifying equipment became available, which led to the use of quad-centered photography (procurred under the National High-Altitude Photography (NHAP) program.) Remote sensing has been a part of the NMD program since the introduction of photogrammetry. Early black and white photography is being replaced with color and color infrared (I.R.) photography. Filters and special emulsion sensitivities have broadened the options of discriminating remote sensing. Remote sensing from satellite imagery has progressed from Landsat through Thematic Mapper, with the French "spot" imagery just ahead of us. Satellite imagery has not been applied to the larger-scale bases in mapping because of its limited resolution. Satellites have also started playing a role in geodesy. Satellite geodesy has allowed us to refine our geodetic position of islands and continents by its abilitv to span the oceans. Satellite geodesy has made a very important contribution to the 1983 NGS datum adjustment. The continuing development of high-speed computers has opened up the field of digital cartography, the fruits of which are yet to be harvested. Synthetic aperture side-looking radar is a new tool that has some potential uses for the NMD, but it will probably find more applications in the earth sciences. Laser technology is showing up in scanners, printers, distance measuring equipment, and who-knows-what-next. The aerial profile recorder appears to have several applications for the NMD. These are but a few of the technological advances that have impacted the mapping community over the past 40 years and will continue to impact us for many years to come.

I would like to go back to a subject I mentioned earlier--namely, map accuracy. The National Map Accuracy Standards for the 1:24,000-scale map series says that "90 percent of the points tested will fall within 40 feet of their true horizontal position on the earth." Vertical accuracy for contouring is similarly stated: "90 percent of the point tested will fall within one-half contour interval." The standards do not specify how these accuracy standards are to be applied. If all mapping is considered in the test population, the NMD is in great shape, since our mapping procedures are designed to meet the specifications on each quadrangle. Remember, though, that the accuracies apply to scale-stable copy. We cannot be responsible for the natural horizontal distortions in maps printed on paper. The other qualifier for accuracies restricts the testing to distinct features. Linear features intersecting at acute angles are not used in testing, nor are elevations under dense ground cover.

Some people have been confused about the 1983 horizontal datum adjustment. Please think of it only as a new coordinate system that is more precise. Points can be referenced to either system, with only the numeric values of the latitude and longitude being different (just as the coordinates when referenced to the UTM grid are different from the same position referenced to a State grid system). It will be several years before existing base maps and thematic data will be readily available in the 1983 datum.

Map revision has been a problem for the NMD ever since the first quadrangle was completed and someone built a new road or building or made other changes. For several years, we have been trying to find that proper balance between completing the map series, working in more and more remote areas, and revising or updating the maps in urban and high-use areas. We have used three basic approaches to revision. Complete revision involves the full review and field confirmation of map change, with the resulting map being equivalent in all respects to a new map. Partial revision addresses specific changes, such as adding a new interstate highway or reservoir, but ignoring all other changes. Ground truth is obtained by our field parties for the features included in the partial revision. For economic reasons, photorevision has become our most popular type of revision. New photography is obtained, and map changes are based on our ability to photointerpret. All new map features are shown in magenta to signify that the field confirmation was not obtained. Fortunately, newly constructed roads, pipelines, buildings, etc., are relatively easy to photointerpret. Abandoned roads will probably not be detected during photorevision. As we approach the completion of the

7 1/2-minute map series in 1989, map maintenance will take on new meaning. Each map represents an inventory of the conditions that existed at the time the map was prepared. A series of maps prepared for the same area at some interval of time provides many map users with a vital dimension that reflects change. The sequence we used in preparing our original 7 1/2-minute series coverage was based on priorities and resulted in a "shotgun" pattern of quadrangles each year. We hope that a more systematic mapping sequence can be developed for our map maintenance program in the future, which will make the time dimension more useful. Unfortunately, change is still not uniform, and the revision cycle needs will continue to place priority on a scattered pattern of quadrangles.

REVIEW OF TRADITIONAL MAPPING PROCESS

By

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Overview

A brief highlight of our traditional mapping process will be presented so that participants might better recognize what they will see on the tour of the Mapping Center that is to follow this briefing.

Project Authorization

Standard 7 1/2-minute topographic mapping is prioritized from requests made by Federal and State agencies in response to an annual solicitation. The impact of cooperative requests and completion of once-over mapping of the United States by the end of the 1980's will be addressed.

Aerial Photography Flight Planning and Acquisition

Determination of contour interval, type of photography, flight height, and need for supplemental contours is made. All aerial photography is acquired through contract, with inspection and acceptance by USGS. We attempt to fly one or two seasons ahead of field work.

Control Planning

Research is made to locate all existing geodetic control for the project. We plan for the minimal amount of required horizontal and vertical field control, based on proper densities. Project indexes are prepared for subsequent use in both field and office phases.

Field Control Surveys

The National Geodetic Survey (NGS) and the USGS have different roles in control acquisition and in dissemination of data. Electronic-distance measuring (EDM) equipment, theodolites, and satellite-tracking devices are used to establish horizontal control. Vertical map control is established by traverse or precise vertical-angle measurements. We have only a limited requirement for high-order leveling.

Field classification of all cultural features, roads, woodland, and hydrography; location of boundaries and sectionizing; photoidentification of control, etc., are accomplished during the same field season and are plotted on the aerial photography, or other base materials. Geographic names are verified with local residents and officials.

Diapositive Preparation

Film or glass diapositives of the original aerial film are prepared on a Log-E printer for use in subsequent map-making phases.

Aerotriangulation

Aerotriangulation is a method of extending existing horizontal and vertical field control to establish supplemental map control through the geometric relationship of adjacent aerial photographs. Over the years a number of methods to extend field control to control each photo-pair have evolved--long bar, slotted templets, semi-analytical, and fully analytical. All required pass points, strip tie points, horizontal and vertical control points, and test points are selected, marked with a unique numbering system, and pugged (a hole drilled into the emulsion of the diapositive).

Pug points and camera fiducials for each diapositive are read out in x and y on a mono-comparator and the results recorded for computer processing on the Perkin-Elmer computer via the direct geodetic constraint method (DGC). Results are analyzed and once acceptable, all points are plotted on a base sheet using a high-speed automated plotter.

Stereocompilation

Present-day technology permits map compilation exclusively by photogrammetric methods from aerial photography, in lieu of labor-intensive field methods. Type of equipment used depends on topography, flight height (and C-factor), and operator experience. Equipment currently used in the Mapping Center includes Kelsh, PG-2, B-8, and Galileo G-6 plotters.

Map delineation includes compilation of culture, hydrography, contour, lettering, and woodland plates. Compilation rates are about 4-6 hours per square mile. Once compilation is completed, and the manuscript is reviewed, corrected, and certified, boundaries, land-office sectionizing, and names and other information is added, and the hydrography overlay is prepared.

Stereomodel Digitizing

An alternative to conventional stereocompilation methods is the capturing of all data present in the stereomodel in a digital format. Interactive editing of this digital data and the use of automated plotting techniques makes it possible to automatically generate map separates, by color, and avoid labor-intensive hand engraving.

Map Finishing

Final rescribing of compilation manuscripts to tight technical specifications and preparation of areal tint plates are accomplished prior to printing. All materials are closely edited for correctness and proper placement of names and other map-worthy information. The lettering plate is prepared and the map composited for a final review and shipment.

Contracting

Most of our mapmaking is accomplished in-house by Survey personnel. We have, however, contracted with private industry and with small businesses for various phases of our work--scribing, lettering, stereocompilation, aerotriangulation, and on one project, field surveys--all with varying degrees of success. All aerial photography is contracted. Photolab requirements are satisfied by an on-site Government-owned contractor-operated facility.

Provisional Mapping

Two years ago the Survey embarked on an accelerated program to expedite complete 7 1/2 minute map coverage of the conterminous United States by 1989. To accomplish this with existing resources required a compromise with traditional mapping processes and the willingness to accept something less than a finished map appearance. Provisional maps make it possible to meet this time frame. They are prepared to the same National Map Accuracy Standards and contain essentially the same level of information as shown on standard topographic maps, but because many of the map finishing operations have been deferred and minor modifications have been made in field and compilation procedures, the resulting maps have a definite "provisional" appearance.

Tour of Rocky Mountain Mapping Center.

Tour of Western Distribution Branch, Printing and Distribution Center.

Panel Discussion With Workshop Participants.

HYDROLOGIC HAZARDS

AN OVERVIEW OF HYDROLOGIC HAZARDS

by

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INTRODUCTION

Hydrologic hazards are among the most costly natural disasters both in terms of money and suffering. They occur everywhere in some degree: as urban flooding in Florence, Italy, as regional floods in South America, as a chronic regional drought in the Sahel, or as coastal pollution from oil spills in the Gulf of Mexico.

The institutional approach to dealing with hydrologic hazards has several components. At one end it begins with organizations conducting scientific research in the earth sciences in general; it progresses through the applied work of monitoring, data collection, laboratory analysis, mapping, and hazard assessment conducted by meteorological and hydrological services. At the other end, hydrological hazards are dealt with, along with all other hazards, by institutions directly concerned with disaster prevention and mitigation. These institutions make use of the scientific information provided by technical agencies.

Success in the useful application of technical material to disaster management rests on the close cooperation among technical agencies in various disciplines (e.g., meteorology, hydrology, and geology), as well as close coordination among technical agencies and disaster management agencies. Ultimately, the long-term solution to the prevention or the minimization of losses from hydrologic hazards rests with the application of pertinent technical information to land-use planning.

In the United States, hydrologic hazards are considered in terms of extremes of physical, chemical, and biological states of hydrologic systems, and are dealt with by a variety of government agencies. The main technical agencies dealing with hydrological hazards at the Federal level are the U.S. Geological Survey (USGS) (data bases), the National Weather Service (forecasting), the Corps of Engineers (impact assessment and planning), and the Federal Emergency Management Agency (disaster management and mitigation, flood insurance).

The Office of Foreign Disaster Assistance (OFDA) of the Agency for International Development (AID) is the United States governmental agency concerned with natural hazards at the international level.

The origin of the geologic and hydrologic hazard training program introduced in Denver in March 1984 by the USGS, of which this paper is an expression, is in the support that OFDA gave for the development of an international training program that integrated the meteorologic, hydrologic, and geologic aspects of natural hazards.

IDENTIFICATION AND DEFINITION OF HYDROLOGIC HAZARDS

A hydrologic hazard is an extreme physical, chemical, or biological condition involving water in its natural state which is hazardous to humans or to an ecological system.

The causes of hydrologic hazards are meteorological events such as hurricane rainfall leading to floods, human activities such as the pollution of water bodies by industrial, urban, and agricultural developments, and geologic hazards. Even the mineralogy of a rock, as a shale with a high fluoride content, can induce hazardous conditions. Such a rock formation can produce a ground-water supply with a hazardous chemical composition. Table 1 lists types of hazards and their causes.

Floods are the hydrologic hazard most familiar to people. Indeed, often they are taken to comprise the entire spectrum of hydrologic hazards. Floods are classified as various types: flash or riverine floods (depending on the duration of flooding and the quickness of arrival of the flood crest); coastal

Table 1 - Hydrological Hazards and their Causes

CAUSE	TYPE OF HYDROLOGIC HAZARDS			
	FLOOD	DROUGHT	SURFACE AND GROUND WATER POLLUTION	HYDROGEOLOGIC HAZARD
Meteorologic Conditions	• Rainfall • Snowmelt • Ice jams • Coastal waves	• Deficient precipitation	• As fallout from precipitation (e.g. acid rain) • From excessive evaporation (e.g. salt pans)	• Landslides from excessive precipitation • Mudflows • Coastal sedimentation or erosion from wave action and flooding
Human Activity	• Dams • Urbanization • Deforestation	• Excessive land use	• Urban runoff • Desalting • Waste disposal • Spills • Fertilizers • Pesticides • Insecticides • Salt-water intrusion • Mining	• Land subsidence from excessive artesian groundwater withdrawal • Doline karst collapse induced by excessive groundwater withdrawal • Coastal sedimentation or erosion from shoreline developments
Geologic Conditions	• Landslides (e.g. river dam) • Earth movement (e.g. dam failure) • Volcanic eruptions (e.g. river dam) • Tectonic adjustment of crust	• Excessive rock permeability • Volcanic dust(?)	• Rocks' mineralogy (e.g. salt dome) • Rocks' lithology (e.g. excessive sedimentation from badlands) • Volcanic dust • Geothermal sources	• Doline karst collapse • Landslides from water saturated structurally weak rocks • Coastal sedimentation or erosion from geologic processes

and shoreline flooding induced by wave action and tidal effects, backwater flooding induced by a variety of conditions that lead to damming of rivers.

Droughts are extreme hydrologic conditions brought about by deficient rainfall or by excessive land use in naturally arid or semi-arid areas.

Pollution of surface- and ground-water bodies is overwhelmingly a consequence of human activities, although some pollution owes to natural geologic environments, such as, where streams flow over salt beds. The causes of surface- and ground-water pollution shown in Table 1 are classified according to whether pollution orginated from a specific source (point pollution) or to an areal distribution of pollutants (nonpoint pollution).

Hydrogeologic hazards have a geologic as well as a hydrologic component. They comprise the transport, erosion, and sedimentation of rock material. Some examples are mudflows, coastal erosion and sedimentation, landslides of water saturated material, land subsidence due to withdrawal of fluids or land collapse in karst terrain due to ground-water level fluctuations.

All hydrologic hazards can be considered from the point of view of human ability to forecast them. Thus, abrupt hazards of short duration (hours) such as flash floods, landslides, and toxic spills affecting small surface-water systems are difficult to forecast. Forecast is possible for abrupt hazards of intermediate duration (days) as floods on large rivers, toxic spills, and point source pollution of large surface-water systems and permeable ground-water systems. The spreading in time and space of droughts, land subsidence, non-point pollution, and in general, all delayed hazards involving ground-water systems can probably be forecast. These hazards, however, often bring about impacts that are irreversible or extremely costly to reverse.

HYDROMETEOROLOGICAL ASPECTS

Extreme meteorological events are hazards in themselves and nearly always the origin of hydrological hazards as well.

Knowledge of meteorological conditions is prerequisite to hydrometeorological forecasting which, in turn, is of extreme importance to hydrological forecasting in small river basins where the time lag between precipitation and flood peak is short. At the regional scale an understanding of large atmospheric circulation patterns helps greatly in forecasting the onset of flood and drought conditions. An example of this is the recently discovered interaction between the equatorial Pacific circulation pattern and the occurrence of the climatic event off the coast of Ecuador and Peru known as El Nino (1). Which in 1983 brought the most catastrophic floods and droughts of record to most of the American continent.

El Nino

This term, "the child," refers to a warming of coastal waters off the coasts of Ecuador and Peru which takes place—thus explaining the name—at Christmas time. The event is associated with rainfall on arid coastal areas, and it is considered beneficial. It has recently been discovered that El Nino is part of a major atmosphere-ocean interaction affecting the entire equatorial Pacific.

Since the 1920's meteorologists have known that an inverse correlation of high and low barometric pressure existed between the eastern and the western Pacific along the equator. This inverse relation is known as the Southern Oscillation and traditionally the difference in pressure readings at the Tahiti and Darwin (Australia) stations are used as an index, which is called the Southern Oscillation Index (SOI).

In the normal atmospheric circulation pattern in the equatorial Pacific, the eastern trade winds from north and south of the equator converge along the Intertropical Convergence Zone (ITCZ) generating an east to west drag on the surface of the ocean. As part of this pattern a relatively high barometric pressure area exists over the cold Humboldt current off South America, while a semipermanent low pressure area exists over the west Pacific. This east to west drag on the ocean surface is accompanied by a flow of ocean water which lowers sea level in the east and raises it in the west. The flow of water brings about an upswelling of nutrient cold water in the east which sustains major fisheries in the area.

With the seasonal warming of equatorial waters in the east, pressure decreases and a reversal of high and low pressure areas between the east and west, as revealed by the SOI, takes place. Easterlies die down and begin to reverse becoming westerlies dragging ocean surface waters from west to east. Oceanographers believe that a subsurface wave (a Kelvin wave) is generated travelling from west to east at a velocity of a few meters per second and arriving in South America several weeks later. It is believed that the arrival of this wave in South America restricts the upswelling of cold waters and brings about a general warming of waters and an onset of El Nino.

In 1982 the westerly winds brought extreme meteorologic conditions to both South America and eastern Australia and Indonesia. Under normal circumstances the rainfall pattern in the Pacific is usually concentrated in the South Pacific Convergence Zone (SPCZ) which during an El Nino event shifts eastward bringing drought conditions to much of Indonesia and Eastern Australia, while normally dry areas in the central and eastern equatorial Pacific receive rain. In 1982-83 this pattern was repeated in the extreme bringing record droughts to the western equatorial Pacific and record rains to the co-latitudinal areas in the east.

The warming of ocean waters began in May 1982. By June the sea surface temperature (SST) from South America to about 170°E longitude were warmer by 1° to 2°C. This caused a weakening of the easterlies which in turn brought an accelerated warming of the east Pacific. After the collapse of the easterly winds the high level waters in the west Pacific began to move eastward bringing strong El Nino conditions to South America, and in October, early rains to Ecuador and Peru. For the next 9 months these countries suffered from the impacts of their most catastrophic hydrologic hazards on record. By December 1982, while waters were still more than 4°C warmer than normal over most of the Pacific, winds returned to their normal easterly condition. This did not bring, however, a lessening of the SST as El Nino conditions persisted in South America through Spring 1983 and only began to fall in mid-summer, thus ushering in a decline of El Nino conditions.

The hydrologic hazards in the American continent and in the west Pacific consequent to the 1982-83 extreme El Nino is summarized in the following table:

Data Bases

Hazard prevention and mitigation, as well as regional planning within the constraints of hydrologic hazard impacts, can only be conducted with long-term data on the occurrence and the magnitude of extreme hydrologic events.

TABLE 2

Hydrologic Hazards and Impacted Areas from the El Nino of 1982-1983.

D	F	H	S	AREA AFFECTED
	*		*	California, Oregon, and Washington coasts (early '83)
	*			Louisiana and Florida coasts, Cuba (Dec '82 - Apr '83)
*	*	*		Hawaii (Dec '82 - Mar '83, H and F in Nov '82)
*				Mexico (summer '83)
	*			Ecuador, Peru, Bolivia lowlands (Nov '82, Feb '83)
*				Philippines, Indonesia, and Australia
*				Sri Lanka, South India
	*	*		French Polynesia
*				Brazil (summer '83)
	*			Brazil, Uruguay, Paraguay, Argentina (April-June '83)

Explanation: D = drought, F = floods, H = hurricane, and S = storms

The climatological data base on extreme events, often the only available information to assess hydrological hazards in many parts of the world, is obtained from station records and historical reports. Indirect data are obtained from tree ring information, and more remotely, from geomorphic, paleobotanical, and paleontological data.

The hydrometeorological elements most directly related to hydrologic hazards are precipitation and temperature. The record length of direct measurements of these parameters, particularly precipitation, is, except for a few stations, a century at most. Generally, the record consists of once-a-day readings of volumetric catches; estimates of rainfall intensities derived from continuous recordings of rainfall are much rarer. Depths of snow packs are measured with snow boards or snow stakes, and are expressed in water equivalent units. Air temperatures are measured generally with dry bulb thermometers and reported as maxima, minima, and averages. Because a few point data are available in most countries, much remains to be done to improve the station network density that provides the areal values needed for hydrologic hazard analysis.

Information about past extreme meteorological events can be obtained from elderly residents. Historical chronicles are very useful to extend the length of the climatic record of an area. An example of this is the interpretative work of Spanish hydrologists on the religious chronicles associated with irrigation in the Segura River since 1549 (2). Other indirect evidence to assess past climatic records, especially droughts, is through the analysis of tree ring data (3).

Trees grow by adding new tissue to existing tissue under the bark, according to a seasonal pattern - large, thin-walled cells in moisture abundant, early spring, and small, dark thick walled cells, in the moisture deficient late growing season. The dark, late cell growth provides the visible separation between rings, thus permitting easy measurement of the thickness between rings which is related, in part, to the moisture availability during the growing season. Ring widths have been used to reconstruct several thousand years climatic records in arid and semiarid environments of Southwestern United States; work with the temperate climate of the Eastern United States is still in the research stage.

Further removed in time, and more general in their definition of past climate, are archeological and paleontological data.

The hydrometeorological time series are used to develop pertinent statistics which are reported as point values or averaged over areas and portrayed in maps. The point data, e.g. the maximum annual daily rainfall (P_i) for the n years of record are ranked ($m = 1$, $m = 2$ ..., $m = n$) in order of decreasing values and the probability distribution of the ensuing series is analysed by standard statistical methods. The value $R_i = (n + 1)/m$, called return period or recurrence interval, is computed and the paired values (R_i, P_i) are fitted graphically or analytically. The log-normal, extreme value distributions such as Gumbel's and, more recently, the normal or log-normal Pearson Type III distribution are typically used in hydrometeorology and in hydrology (5). For example, from the distribution fitted to the annual daily maxima of rainfall, certain rainfall values with a given return period are computed; those in common use are the 10, 25, 50, and 100 year events, which, in turn are also commonly used in hydrologic hazard analysis.

Examples of hydrometeorological maps routinely prepared are the rainfall intensity maps of the National Weather Service (4), which provide national areal coverage for rainfall intensities for given durations and return periods. These maps supply first estimates of rainfall that can be used in models that simulate rainfall-runoff processes.

Among the many rainfall parameters that are used to estimate extreme floods is the Probable Maximum Precipitation (PMP). The PMP is considered to be the probable maximum amount of precipitation that can physically fall over a given area in a given time. The PMP is usually derived with reference to a specific drainage basin, and often as a means of computing the Probable Maximum Flood (PMF). Although the PMP is occasionally computed from storm models which additionally require upper air data on moisture and wind, the more common approach is to apply maximum values of observed storm precipitation data. The largest storms of record are simulated by increasing their moisture content to a maximum value which, it is assumed, would have produced maximum precipitation. Where data are scarce or lacking, meteorologists transpose storm information from areas with similar meteorologic conditions. Calculating the PMP in snow prone areas entails the computation of snow melt. This computation can be made with theoretical models; in practice, however, it is mostly made by empirical methods such as degree-day factors based on air temperature data (5).

FLOODS

Backwater floods occur when steamflow is impeded, as when an ice jam forms behind a bridge and dams the water until it flows over the bridge, or until the bridge is toppled by the back-water pressure. A second type of flood is an abrupt wave formed as dammed waters are suddenly released from a dam break or are generated as a splash wave by a landslide into the reservoir. Third, and most common, flash or riverine floods develop from concentrated rainfall or melting snow into headwater tributaries, which merge the tributary floods into a single flood wave along the main channel. Coastal flooding occurs as a result of inundation from stream flooding, coastal wave action, or high tides.

If the development of floods is intuitively clear to all, a brief explanation is perhaps necessary for the non-hydrologists to clarify the role played by the stream channel network in concentrating flood waves downstream. Consider a network such as that shown in Figure 1. Let us imagine, for purposes of demonstration, that we can divide the network into reaches of channels for which the flow-travel time from the head to the end of the segment is constant (Figure 2). Let us imagine further that rain falls as a near instantaneous shower uniformily over the basin and let us record the behavior of the streamflow in time periods equal to the travel time of the stream segments in which we have subdivided the stream network. Clearly at the end of the first recording period we only count the flow from the most downstream segment; the water in all the other stream segments flow to their next downstream segment. At the end of the second period we record the arrival of flow from four segments; and at the end of the third period we record flow from seven segments (Figure 3). Continuing in this fashion for up to a number of periods equal to the time it takes for the furthest stream segment to progress to the most downstream segment, we obtain the graph shown in Figure 3. The peak shown in the graph is the flood resulting from the concentration effect of the stream network considered here. Clearly basins of different shapes and, implicitly, different networks, produce different peaks from the same rainfall input. In fact, examination of the linkage distribution of stream segments in a basin is of paramount importance in computing the flood hydrograph. In addition, the pattern of rainfall over the drainage basin and the antecedent climatic conditions are determinant of the peak discharge.

At any point in a basin a flood occurs when the water spreads onto the flood plain. The depth of flooding at a given discharge along a stream is governed by the hydraulic geometry of the channel and of the valley floor which, in turn, are a function of the geologic properties of the basin and the distance from the basin divides.

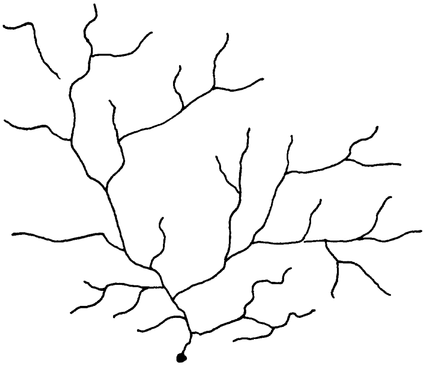

FIGURE 1

Outline of a stream channel network

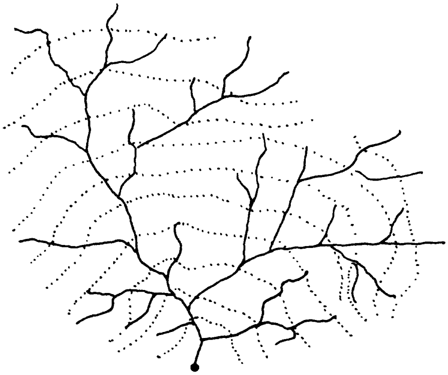

FIGURE 2

Equal travel time contours

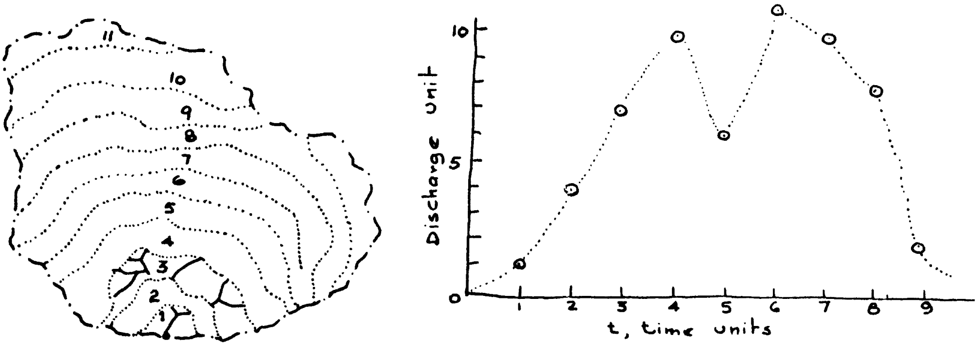

FIGURE 3

Planimetric distribution of areas with given travel time and their hydrographs.

DATA BASE

Flood data consist of river stage and discharge, the latter collected at principal hydrologic stations recording the entire hydrologic regime of the river. Some of the data usually are telemetered, in a variety of communication schemes, to permit hydrologic forecasting and flood agencies to perform their duties. During major flood events, imagery data may also be collected from aircraft and satellites.

For the preparation of flood maps and flood reports, hydrologists collect most elevations of crest after the flood has subsided, by locating the debris that the river has left on the flood plain slopes and the mud markings that designate the river crest on tree trunks or on buildings. A survey of these high-water marks allows the hydrologist to plot the flood boundary on a map peak discharge. In this computation of discharge, the velocity is calculated from hydraulic formulae which relate velocity to the longitudinal slope of the water surface (i.e., from the high water marks) and the frictional affect of the bed channel, banks, and flood plain. It should be noted that the estimation of frictional effects is an art acquired after years of experience in flood computation of many river reaches with different physical characteristics. (7)

Indirect data used to extend in time an existing flood series or to establish a flood series where data are scarce or lacking, are historical records, botanical evidence, areal imagery, and mathematical models. Historical records provide at least some qualitative evidence of the great floods; occasionally reference is available as, for example, in Florence, Italy, where the high water marks of the Arno have been marked in one of the Piazzas since the fourteenth century.

Tree rings provide evidence of past floods in several ways. First, if a tree is toppled by a flood and a sprout grows out of the fallen tree, the age of the sprout indicates the year of the flood. Similarly, when a tree is scarred by debris it is possible to determine the flood year by counting the number of tree rings between the scar and the bark. Partly uprooted trees typically form eccentric rings that likewise can be used as indirect evidence of floods. More recent studies of the wood anatomy of flooded trees have been used to determine the occurrence of summer floods to within two weeks of the event.

Areal imagery obtained during the passage of the flood crest can be used to assess the flood boundaries and thus can be used to prepare flood inundation maps. Even oblique photography can be useful if certain field structure can be located on the photograph and recognized on a map of the area. Remotely sensed data from satellites are routinely used to assess floods on continental sized rivers and on regional flooding of flat lands as in the Chad in Africa. The limitations on the use of satellite imagery concerns the size of the river in relation to the resolution of the image pixels, the degree to which land cover obscures flood boundaries, and the availability of processed imagery on the crest dates.

The types of mathematical models used to estimate floods can be grouped into three categories. First, flood data can be extended in space and time by means of statistical methods which correlate the desired data with data collected at other nearby points with similar hydrometeorological characteristics. These techniques range from simple correlation of discharges at two or more stations during concurrent periods, to regional correlations of flood discharges with geomorphic basin parameters such as drainage area and basin slope. Some progress has been made in relating flood discharges to the channel width measured at critical bench mark points in selected cross sections (8). Second, flood stages and discharges are computed for river channels, as previously indicated, by means of hydraulic formulae. These computations are used, in particular, for routing flood waves through the main channel of river basins as when a flood wave is generated from the concentration of upstream tributary flow and progresses downstream or, hydraulically equivalent, when a flood wave is generated by a dam break (9). The third type of mathematical model uses rainfall data to compute runoff. Some of these models are of the "black box," or statistical type, which require only some concurrent rainfall and runoff data for calibration of the model parameters. Other "rainfall-runoff" models are known as "conceptual"; they simulate the hydrologic regime of a basin by distributing part of the rainfall input to parameters representing infiltration, evaporation, and ground water percolation. The more complex two dimensional models make use of main channel hydraulic routing procedures and snow melt calculations (10).

FLOOD FREQUENCY ANALYSIS

The flood record constitutes a time series from which the probability of occurrence of floods of various magnitudes may be estimated, much as mentioned previously with the hydrometeorological record. The series, usually at least 10 years in length, consist of measured flood stages or discharges above a certain base or of the maximum annual floods during the period of record. These time series, possibly extended in time with historical or other information as previously indicated, are fitted by the log Pearson Type III distribution (9). For the design of large structures or where safety factors are especially significant, the Probable Maximum Flood (PMF) is often computed from the PMP.

After each major flood event hydrological services collect field data and prepare maps of flooded areas particularly where the information is critical for planning. Often an assessment is also made of past events and the resulting map thus provides an estimate of flood frequency. Other types of flood maps show the maximum valley inundation resulting from a dam break. For example, the Mount St. Helens eruption closed off the Spirit Lake outlet with the volcanic debris. A map was prepared of flooded areas downstream from the outlet based on a hypothetical break of the outlet closure and the generation of a sediment-laden flood. (11)

Management and planning flood maps are those prepared for flood insurance or land-use planning. These generally delineate the boundaries of the 100 year flood, this being the design flood most frequently chosen for planning purposes.

DROUGHT

More insidious than floods, drought are often hard to recognize at their early stages. In humid places they become apparent after months wihtout rain. In countries with seasonal rainfall patterns, like India, droughts occur from a delay in the arrival of the monsoonal rains.

In arid and semei-arid regions, like the Sahel, drought is and ever present reality often exacerbated by improper land use.

Different professionals have different views of droughts. To the meterologist, drought is a rainless situation with a small probability of occurrence, to the farmer a drought is lack of moisture for the crops, to the economist a drought is a water shortage affecting the local economy. To the hydrologist, a drought is streamflow and ground-water levels much below the normal state (13).

DROUGHT INDICIES

Several indices have been devised to define drought quantitatively. The U.S. National Weather Service (NWS) uses the Palmer Drought Severity Index (PDSI) (12) to classify droughts. Briefly, according to Palmer (1965) (12), defines a drought according to a water balance procedure derived from antecedent precipitation during a given period, and the duration and magnitude of the abnormal moisture deficiency. The difference between the actual precipitation and that needed to meet the demands of evapotranspiration is considered a direct measure of the departure from normality. Four levels of drought severity are recognized: mild, moderate, severe, and extreme. An example of application of the Palmer index to a report on a nationwide drought is given in (14).

Hydrologic consequences of droughts are low lake levels, low stream flows, and depressed ground-water levels. To the extent that streamflow and lake levels reflect the response of drainage areas, to precipitation also reflect the inertial response of the land phase of the hydrologic cycle to the lack of rain, and may thus lag somewhat behind the meteorological definition of drought. Ground-water levels in water-table aquifers provide another index to drought. These are generally somewhat lagged behind the climatological data and, depending on the extent of the aquifer, surface-water data as well. However, water levels from deep ground-water bodies may lag so greatly that their low points are not recognized as a drought indicator.

Data Base and Frequency Analysis

Traditionally, hydrologists have used low flow frequencies to define a drought (15), based on the lowest annual streamflow during 7 consecutive days. (The 7-day average has been used by hydrologists to smooth out or filter streamflow data from regulated rivers on which the daily flow fluctuates greatly.)

Occasionally, the flow duration curve of daily discharge which shows the percentage of time that flow of a given magnitude is expected to occur. A chosen percentage probability of low flow is used as an index drought low-flow. But hydrologists prefer to use low flow data because it is considered to be sequentially independent in a statistical sense. An annual series of 7-day average minima flows is fitted much as was done with hydrometeorological and flood data, with a frequency of distribution. If the series is short, it may be possible to extend the records by historical means (2), tree ring indices correlated with known streamflow records (16), and correlations with data from nearby long-term stations (15). On the other hand, mathematical models, either conceptual or black box type, have not successfully simulated low flow as well as flood flow. This difficulty arises because the low-flow discharge of a stream is a function of the ground-water storage capacity of the basin which in turn is a function of basin size as well as the lithological composition of underlying rocks and their structure. Quantification of these geologic aspects of basins has proven to be a stubborn problem.

The choice of recurrence interval of low flow is not as standardized as that for floods (for which the 100-year return periods seems to be generally applicable in land-use planning. No doubt the choices depend on the scope of the project for which the data are sought. Choosing an appropriate return period for streamflows to be used in critical operations such as cooling of nuclear power plants will confront the planner more and more in the future (17).

Recession Curves

Streamflow decreases during rainless periods in a manner which can often be expressed mathematically, although the mathematical expression is debated and in any case may require modification when applied to some streams. Nonetheless, recession curves permit the estimation of streamflow in the future, especially where the streamflow originates from snow packs. Thus it is possible to use flow recession curves as forecasting tools, especially where the antecedent streamflow record has permitted the calibration of the recession parameters.

HYDROGEOLOGIC HAZARDS

Under this heading are a variety of hazards which have both a geologic and a hydrologic component (19). Because they are dealt with at length in papers dealing with geologic hazards, they are briefly noted here within their hydrologic context.

Landslides

Landslides involve the downward movement of slope material. The variety of material involved, the mechanisms causing the movement, and the rates of movement, have resulted in the identification of several types of landslides. The hydrologic aspect of landslides is in the increased mass of the material subject to sliding by infiltration of precipitation or other water, or by increased water pressures that induce or hasten sliding. Manmade activities that contribute to these conditions by raising water tables during irrigation or by bank infiltration from reservoir's. A particular hydrologic impact of landslides is a flood wave generated when rock masses fall into a body of water, such as occurred in 1969 at the Vaiont Dam, Italy.

Subsidence

The downward movement of earth material is known as subsidence. It can take place by geologic processes slowly or catastrophically. The hydrologic aspect of subsidence is the settling of the land surface due to compaction of artesian ground-water aquifers or overlying aquitards as a result of ground-water withdrawal. The porosity and the specific yield of the aquifer are reduced, generally permanently. Land surface impacts include changes in gradient, drainage patterns, and development of flooding conditions. Examples of these problems are given in (18).

Karst Terrain Hazards

A specific case of catastrophic land subsidence is the formation of sinkholes in Karst terrain. Limestone, the primary constituent of such terrains, is susceptible to dissolution by rainfall. The dissolution often takes place along specific fractures or fault zones and progresses at times to the collapse of cylindrical rock masses into cavities formed below the land surface. This process is greatly accelerated in areas where ground-water levels oscillate greatly, which causes particular stress when levels are low. Low ground-water levels can be produced by extensive ground-water withdrawals, and thus, human activity is one of the principal causes of Karst hazards (20).

Coastal Hazards

Landslide formation in coastal areas are induced by the carving effects of wave action on the toe of coastal slopes. Land subsidence along sea coasts includes the spreading of salt water inland while simultaneously increasing the probability of flooding. Other hydrologic processes associated with accelerated coastal erosion or deposition arise from decreases or increases in the sediment load of rivers. These often relate to human activities such as deforestation, or the creation of inland reservoirs which produce, respectively, an increase or a decrease of the river-sediment load.

HAZARDOUS POLLUTION

If hydrologic hazards of the physical type such as floods have always been part of the human lot, the problems of chemical and biological hydrologic hazards to the hydrologic regime have come to the forefront with the last generation. Two factors have contributed mainly to their emergence as hydrologic hazards: urbanization and industrialization. Both are typical of industrial countries and result in types of pollutants such as domestic sewage and industrial waste. The former is a major problem where urbanization has proceeded so rapidly as to overrun the available treatment capacity (21); this has occurred most strongly in the developing countries. The latter is a major problem of industrial countries. In these latter countries the focus of the problem is shifting more and more from surface-water conditions to ground water. The problem of acid rain has emerged in the last few years as another source of hazardous pollution affecting surface water and ecological systems.

Industrial waste and domestic sewage are often referred to as conservative and nonconservative pollutants, respectively. Conservative pollutants remain basically unchanged in the environment for long periods, and thus are most effective by dilutions; nonconservative pollutants are generally degraded and hence do not remain as long in the environment. Another classification of hydrological significance is the source of pollutants. Pollutants can be from a point source, as with and industrial plant discharging its waste effluent into a surface-water body by means of a pipe, or from nonpoint source, as with spraying of insecticides over an orchard and the subsequent leaching of toxic residues into the ground-water system. Finally, a note of reference should also be made to radioactive-waste disposal as a source of potential impact on hydrologic systems. At this time the scope and size of this problem are unknown.

Data Bases

Background chemical, biological, and physical data are routinely collected by hydrological services from samples taken at selected surface- and ground-water measuring stations. These data occasionally record trends of developing water-quality hazards, usually by showing a slow increase, with time, of a particular constituent. An example of this would be the increased chloride content of water from some wells near coastal ground-water pumping fields indicating possible sea water intrusion. Most water-quality monitoring stations, however, are established specifically where pollution exists or where it is feared that it will. A particular case of ad hoc monitoring stations are those established after toxic spills.

Because water-quality monitoring is a problem which needs to be evaluated case by case, there are no routine sampling methods. In general, though, in river basins the discharge needs to be measured or estimated at the time the water-quality samples are collected; similarily the water level is measured at ground-water monitoring wells. Occasionally, hydrological services collect travel time information on segments of rivers where the threat of spills exists. This information is obtained by injecting dyes (fluorescent) at some points in the river and measuring the spread of the dye cloud downstream for a certain distance (22). More rarely, similar information is collected in ground-water systems especially in limestone terrains where a tracing substance is injected in a doline and the tracer is measured at nearby springs.

Except for the routine water-quality data collected at long-term hydrometric stations, all chemical and biological data collected from monitoring stations needs to be considered within the context of a drainage basin or an aquifer system or both according to the type of hydrological problem at hand. Use of mathematical models which simulate both the hydraulic and the geochemical aspect of the hydrologic system are becoming an integral part cf hazardous pollution analysis (32). No doubt these hydrologic hazards entail the most complex and multidisciplinary data base for their assessment and evaluation.

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HYDROLOGIC HAZARDS MAPS

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INTRODUCTION

Maps are the natural medium for describing, planimetrically, the two dimensional status of a hydrologic event. Hydrologic maps deal with transient phenomena, nonetheless, they may be used to portray the spatial distribution of hydrologic conditions at a given moment in time or for a specific type of event; in this case an extreme one.

A well prepared map conveys information clearly and simply while integrating a large amount of information often of a complex nature. A map maker interprets and correlates point data with other conditions or point information contained in a base map such as topography. The map obtained may be scientific, the relevance criteria of the map will be, however, its content.

Although several types of maps depicting hydrologic hazards have been prepared routinely by hydrologists, the term "hydrologic hazard map" is almost unknown. A computer search of earth science library files made to retrieve titles and abstracts of documents catalogued as hydrologic-hazard maps, yielded no data. On the other hand, the concept of hydrological hazards as a general term dealing with floods, droughts, pollution, subsidence, landslides, and so forth, is making its way in the literature.

This paper reviews the type of work being done that can be classified as belonging to the field of hydrologic hazard mapping. A case study on a specific type of hazard map is presented and a discussion is made of the role of the hydrologic hazard map as a planning tool.

TYPES AND PURPOSE OF MAPS

Maps can be classified as general purpose (e.g. road maps), educational (e.g. a national atlas), scientific (e.g. geologic map), and management (e.g. a country comprehensive zoning map). The management maps can be further classified as: a) regulatory maps (zoning maps), b) engineering maps (design), or c) planning maps (land use suitability).

The foundation of all maps is information. The progression from scientific maps to management maps involves the transformation of information that is meaningful mainly to scientists to maps that are meaningful to non-scientists. The transformation requires integration and interpretation in order to provide information that is critical to managers. Hydrologic hazard maps, for example, begin with the integration of hydrologic extreme data with other basic topographic, climatic, and geologic data to produce maps which portray the areal extent of the hydrologic hazard or an indication of its magnitude at various locations. These maps can then be combined with socio-economic data to derive maps which show management: 1) the parts of the area that may be affected by a given hydrologic hazard, 2) the damage and economic loss that the occurrence of such a hazard might bring, and 3) and the probability of occurrence of the hazards.

PROBLEMS OF SCALE

The scale of the map is related to its use and a relationship between use and recommended scale is given in Table 1.

Relationships among scale intended use, and size are given in Figure 1 (UNESCO-WMO, 1977) (1).

The actual size of the map, whether folded or unfolded, should be within bounds of comfortable handling. Rules of thumb given by UNESCO-WMO, (1977) are that maximum size should be no more than 70 x 100 cm and preferably no more than 50 x 80 cm. Whenever possible only one sheet should cover the study area.

Table 1: Map scale versus map use (adapted from UNESCO-WMO, 1977 (1))

Map Scale	Map Use
Large Scale: > 1:20,000	OPERATIONAL MAP -- from site to municipal and county level
Medium Scale 1:20,000 to 1:100,000	PLANNING MAPS -- municipal to county level
Small Scale 1:100,000 to 1:500,000	EDUCATIONAL AND PLANNING -- county to State level
Very Small Scale 1:500,000 to 1:1,000,000	EDUCATIONAL -- State to national

MAP PRESENTATION

UNESCO-WMO, (1977), provides some criteria for the presentation of hydrologic maps. These criteria, and others, are presented here within the context of hydrologic hazard maps.

1. The location of all data points should be shown directly on the map by a suitable symbol (or symbols if clusters of data points are involved).

2. The date of the survey or of the data should be indicated in the map or in textual material.

3. The type and reliability of the data should be indicated. In cases of data that vary in time or space, an indication should be given of the limits variability.

4. When maps depict interpreted or interpolated data the methods of interpolation or interpretation and its reliabilities should be indicated.

5. Maps should be critically reviewed and an indication of reviewing criteria should be cited.

TYPES OF MAPS

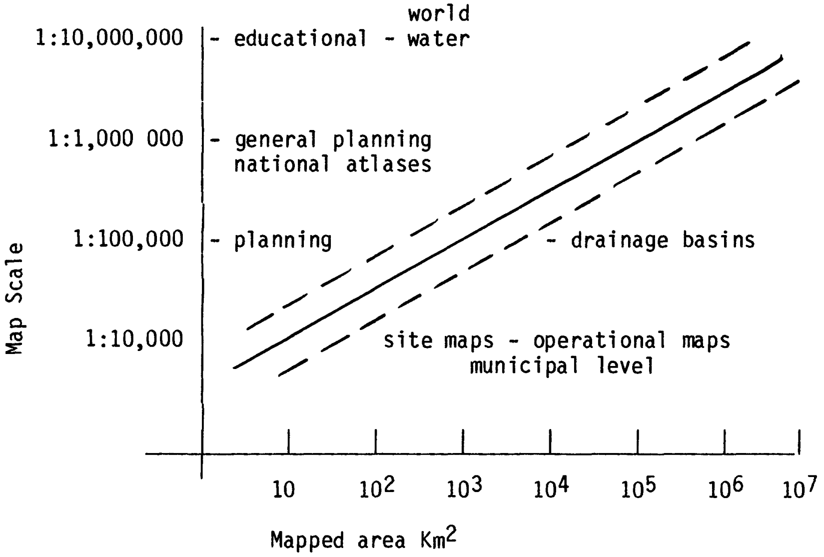

Figure 1.—Relation between map scale and map use.

The map should be graphically well balanced. Avoid information overloading which often result in illegible maps. Where several elements are shown, and one is predominant in importance, it should receive graphical emphasis. The physical size of the map should be within the bounds of its practical handling; when folded, there should be no overlapping folds. Lettering should be minimal and legible; it should serve to identify or categorize map areas or clarify sources of information or reliability. A map that has excessive lettering is like a movie with excessive audio: the message of the primary medium, the visual, is weakened by distraction. Color may serve the same purpose as lettering. However, because color carries specific psychological connotations relevant to hazards, it can be used as such to convey additional information. In hydrologic hazard maps it can be used to portray, qualitatively, degrees of hazardness as shown in Figure 2. The color scheme used is that of traffic lights--green, yellow, and red--and it carries the same cautionary connotation (Hackett and McComas, 1969, 2).

EXISTING HYDROLOGICAL HAZARDS MAPS

The best known hydrologic hazard map, the flood map, is routinely prepared in the United States by the U.S. Geological Survey (USGS) and others to describe areas subject to large floods of national significance. Other maps which relate to hydrologic hazards were prepared by the USGS over a number of years as part of projects dealing with the application of earth science information to regional and urban planning. An annotated list of representative maps prepared during these studies is presented in Table 2. Only one example of maps prepared for each type of hazard mapped or for each element significant for hazard mitigation, e.g. location of sources of emergency water supply, is given from the several available from all study areas; and only maps specifically related to hydrologic hazards were selected.

CASE STUDY ON THE PREPARATION OF URBAN FLOOD MAPS FOR THE COUNTY OF FAIRFAX, VIRGINIA (Anderson, 1970, 16)

This study, conducted in the early sixties (Anderson, 1970), began as a cooperative research project in urban hydrology between the USGS and the County of Fairfax, Virginia. The objective of the project was to devise

Table 2: Annotated list of selected hydrologic hazard maps prepared by the USGS

Ref* No.	Type of Hazard or Element Mapped	Map Scale	Description
SINGLE HAZARD MAP			
3	Flood (Regional and Urban)	$\frac{1}{125,000}$	Map portrays, in blue, the 100 year flood boundary of streams flowing into San Francisco area. The map was derived from field data and site developed stage discharge data by backwater method. Data was interpolated between contours on maps at scale 1:24,000 and transferred to map scale 1:125,000. Map shows potential flooded areas, not actual events.
12	Flood (Coastal)	$\frac{1}{24,000}$	Map (4 colors) shows Gulf Coast areas flooded during the passage of the worst hurricane of record—Frederic-September 12-13, 1979. Textual material describes effects of hurricane, return period of event, and comparison with effects of hurricane Camille (1969). This map is part of a series of 22 maps covering the entire impact area.
7	Flood (Tsunamis)	$\frac{1}{125,000}$	Map (black and white) shows coastal areas in San Francisco Bay flooded from a 20-foot Tsunami wave (200 years recurrence) coming through the Golden Gate bridge. Textual material on attenuation of wave based on 1960 and 1964 data. Runup inland assumed to decrease at a rate inland 1 foot per 100 for given wave height.
13	Snow Avalanche	$\frac{1}{62,500}$	Map shows qualitative probability areas for snow avalaches in Henrys Lake, Idaho, based on land slopes and field observation on past events. Maximum avalanche probability with slope from 30° to 45°. Textual material - one color, two screens.

* The reference numbers given here are those found in the Bibliography at the end of the paper.

Ref* No.	Type of Hazard or Element Mapped	Map Scale	Description
8	High Ground-Water Levels (Urban area)	$\frac{1}{125,000}$	Map (black and white) shows shows artesian zones and ground-water level around San Francisco Bay and artesian zones. Textual material discusses problems: flooding of buildings and underground structures, septic tanks, and underground utilities; subsurface fungal growth and other plant diseases; differential settling of buildings, swelling and settling of soils; liquefaction of granular soils during earthquakes; contamination of potable water.
14	Surface-Water Quality (Coal mine areas)	$\frac{1}{125,000}$	Map (black and white) shows concentrations of dissolved solids concentrations and pH in selected streams in Kentucky. Shading indicates concentration classes of reaches of river; values are also written at sampling points. Only one reach appears troublesome (pH = 4.2, TDS 600 ppm) textual material.
5	Ground-Water Quality (Urban area)	$\frac{1}{125,000}$	Map (black and white) shows for the San Francisco Bay region long-term data on concentrations of nitrate, boron, and dissolved solids in ground water, sampling point locations, variations over time of data, and point symbols and areal shading describing given classes of values. Textual material discusses the data.
6	Solid Waste Disposal Sites	$\frac{1}{500,000}$	Map (black and white) shows solid waste disposal sites in the San Francisco area. Textual material and tables provide data on type: general purpose, special purpose, closed since 1950, proposed; owner: private, government; method of operation: uncontrolled dumping, modified sanitary landfill. An x in the table indicates water pollution problem exists or have existed. Data on site classification: suitable for all but radioactive waste, suitable for some toxic material, suitable for all non-toxic and decomposable material, suitable for non-toxic, insoluble and non-decomposible material.

Ref* No.	Type of Hazard or Element Mapped	Map Scale	Description
9	Emergency Water Supply	$\frac{1}{125,000}$	Map (black and white) shows locations of water wells, storage tanks, springs, and perennial rivers in San Mateo County, California. Textual material provides information on emergency water supply procedures and emergency water supply wells including criteria for selections. Tables summarize data on 60 selected water wells, springs, and perennial streams.
MULTIPLE HAZARD MAPPED			
10	Floods and Droughts	$\frac{1}{125,000}$	Map (black and white) shows extreme points and values only, 100 year or 50 year flood peak discharges, and 7 day low-flow discharges with 10 years frequency for the Greater Pittsburgh area. Textual material.
4	Erosional and Depositional areas	$\frac{1}{125,000}$	Map (black and white) and textual material describe and outline erosional and depositional areas of the San Francisco Bay region. Criteria and methods used to assess sediment yields are discussed.
15	Seiches, Rock-slides, Rockfall, and earthflow	$\frac{1}{62,500}$	Map (4 colors) shows zones subject to landslides, lake-shore areas subject to inundation by seiches, Henrys Lake, Idaho. Probability ranked quantitatively.
11	Subsidence, Landslides, and Flooding	$\frac{1}{50,000}$	Map (black and white) shows areas in Allegheny County, Pennsylvania, where landsliding, flooding, undermining, alone or in combination, pose problems of land use.

* The reference numbers given here is that found in the Bibliography at the end of the paper.

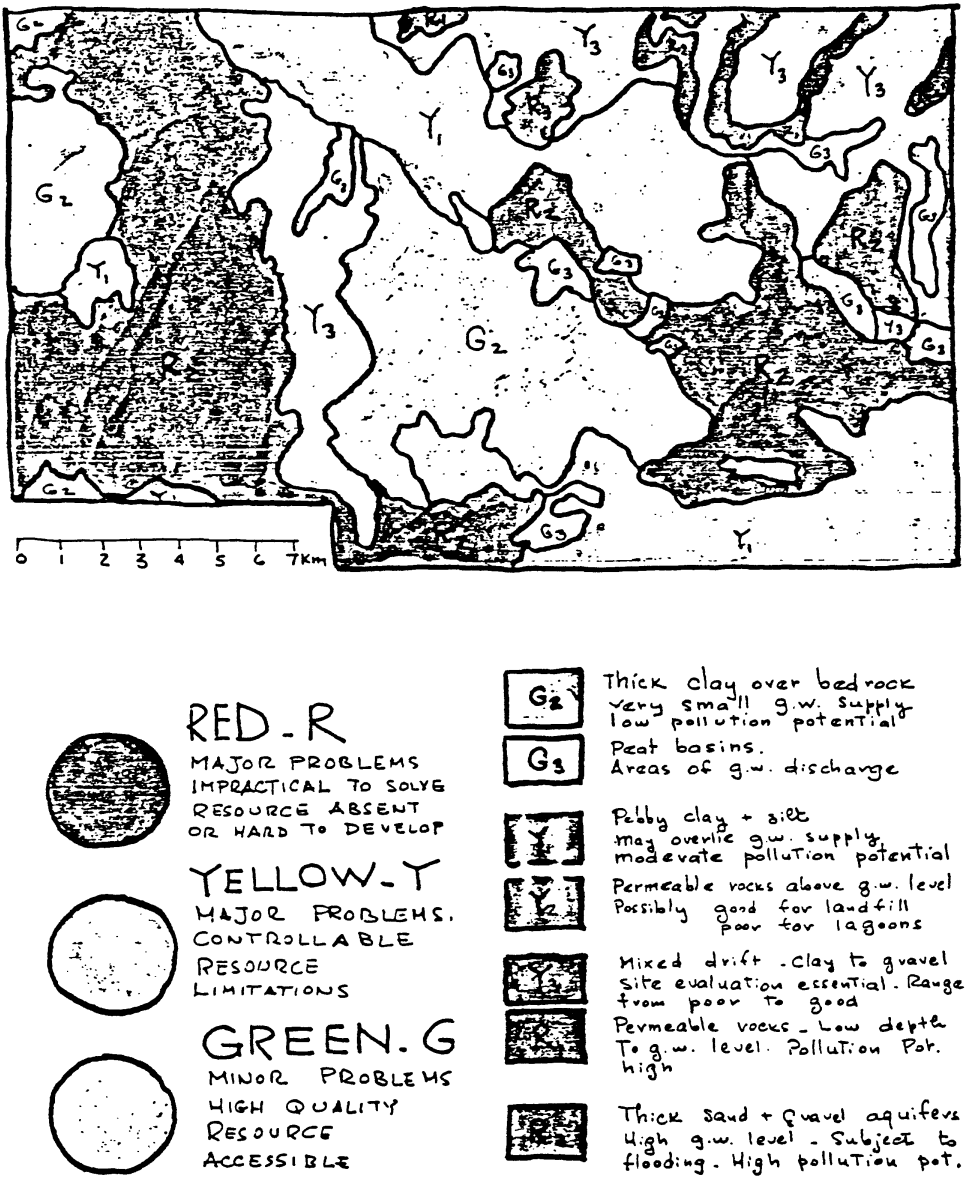

Figure 2.--Example on use of color to highlight hazard severity (adapted from Hackett and McCombs, 1969, as reported in UNESCO-WMO, 1977)

techniques by which maps could be prepared to show the boundaries of flood plain areas in the County which could be inundated at selected frequencies. It was the intention of the County to employ these maps as flood plain zoning tools. It was recognized that urban streams were likely to have higher peaks than rural streams; in fact it was agreed that the entire research aspect of the study was the definition of relationships between peak flow, basin parameters, and degree of urbanization of basins which would permit the estimation of extents of flooding to be expected at ungaged sites. It was agreed that standard open channel hydraulic formulae could be used to compute flood heights from the given discharges and that flood maps could be prepared from the computed flood heights.

DATA COLLECTION

The rainfall and discharge data available in the study area from the Weather Bureau and USGS stations were insufficient for the study. They were supplemented in 1959 with a regular data collection program in Maryland and Virginia. Collectively, data from 61 stations in Maryland and Virginia were available including data from 14 small, impervious basins studied by Johns Hopkins University (Figure 3). Additionally, data from six completely sewered basins in Louisville, Kentucky, were used. Rainfall tipping buckets were attached at the newly installed stream-gaging stations; this permitted obtaining a synchronous record of river stage and rainfall.

METHOD OF ANALYSIS

A first evaluation of basin imperviousness was interpreted from the percentage of the basin area covered with impervious surfaces following a proposal of Carter, (1961, 17). It was assumed that peak flows would result from shorter lag times between rainfall and peak flow in urban streams. Snyder (1958, 18) had found that lag time, defined as the time from the center of mass of rainfall excess to the center of mass of direct runoff, could be estimated from the basin length and slope and the channel roughness. Carter (1961, 17) defined a relation

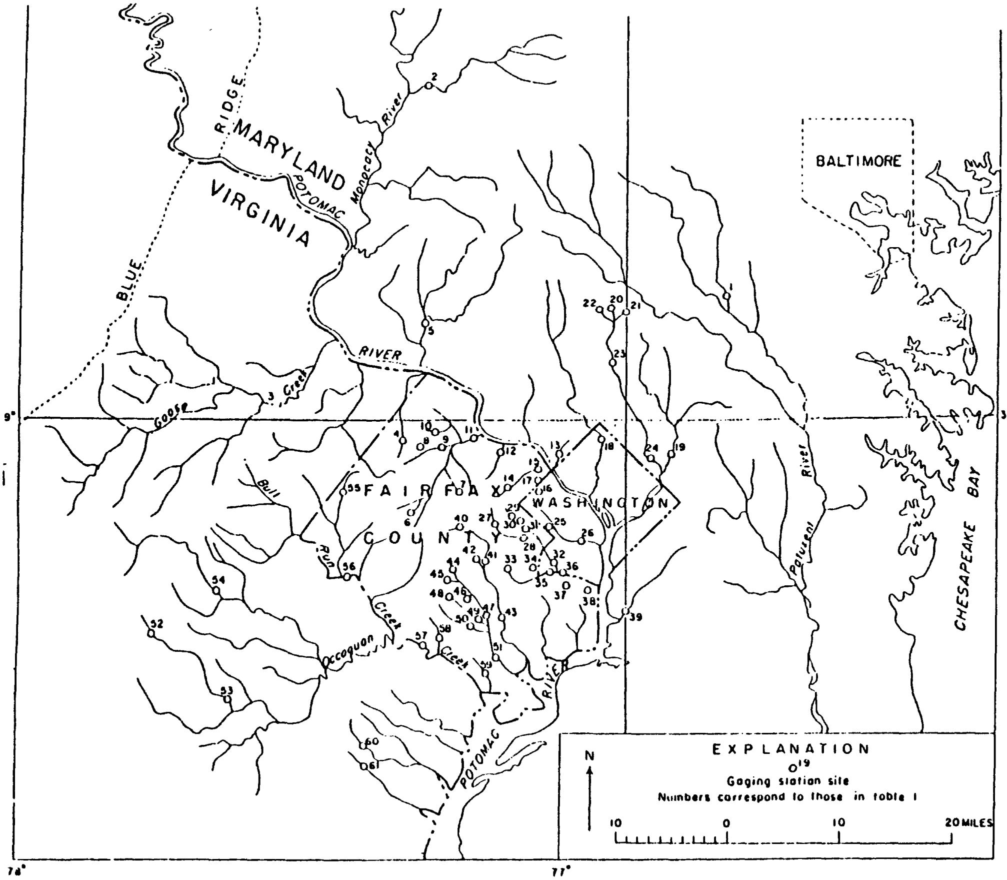

Figure 3.--Location of study area and gaging stations (from Anderson (D. G.), 1970).

$$\overline{Q} / K = f(A, T)$$

where $\overline{Q}$ = mean annual flood, cubic feet per second
K = coefficient of imperviousness, dimensionless
A = drainage areas, miles square
T = lag time, hours

and sought its empirical fit with the data collected in the Fairfax study.

The average flood was determined by graphical interpretation of flood frequency curves prepared with the available record using either the annual series (i.e. a series of the maximum annual peaks) or the partial duration series (i.e. a series of flood peaks above a certain arbitrarily chosen level). Choice of the series was according to length of record: eight or more years of record for annual series, partial duration series for fewer years.

The coefficient of imperviousness K was computed on the basis of empirical findings that whereas about 30 percent of rainfall results in direct runoff in most basins, 75 percent of that which falls in impervious basins runs off directly. The index K was redefined as

$$K = 1 + .015\ I$$

where I is the percentage of the drainage area (A) that is impervious.

Lag time, T, was computed for several basins and related to the ratio $L \cdot S^{-1/2}$ where L is the basin length measure along the longest streamcourse and S is the basin slope measured between 10 and 85 percent of the length as suggested by Benson (1959, 19). Actual lag times were computed from three to six storms for each basin using uniform rainfalls of at least 1/2 inches falling on wet soils. For each storm the volume of rainfall excess was adjusted to equal the volume of direct runoff as presented by Linsley and others (1958, 20). The relationship between T and $L \cdot S^{-1/2}$ led to three empirical equations valid for natural (TN), urban and and partly channelled basins (TU), and completely sewered basins (TS)(Figure 4):

$$T_N = 4.6\ (L \cdot S^{-1/2})^{.42};\ T_U = .9(L \cdot S^{-1/2})^{.50};\ T_S = .6(L \cdot S^{-1/2})^{.52}$$

Drainage area was planimetered directly from maps.

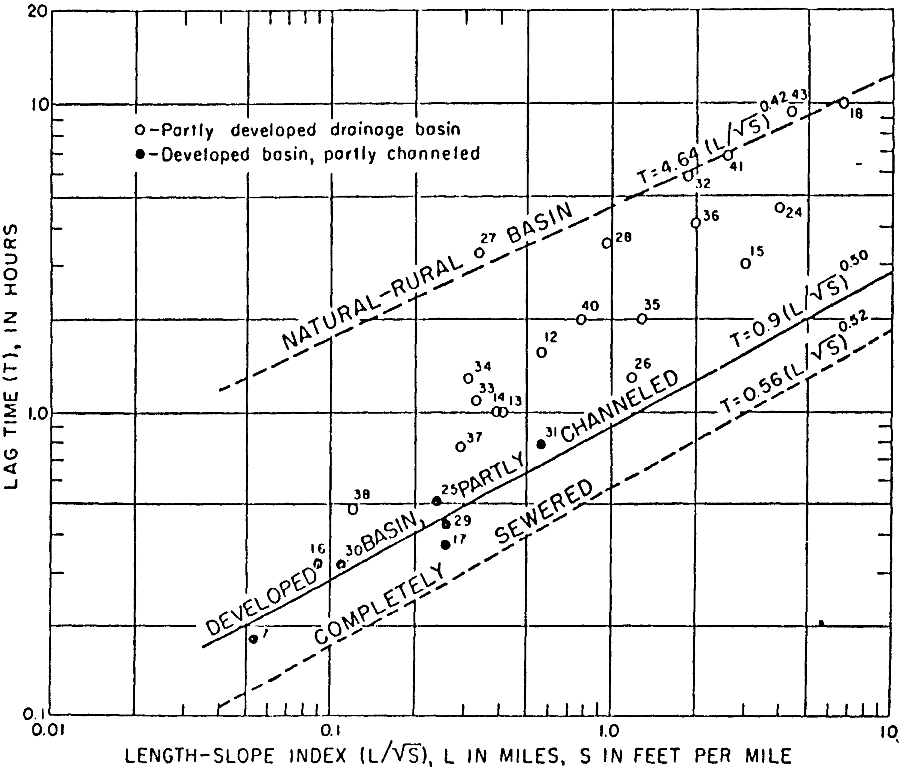

Figure 4.—Relation between lag time and the ratio of streem length to the square foot of the slope ($L \cdot S^{1/2}$)for developed and undeveloped basins (from Anderson (D. G.), 1970).

The analytical fit of the data led to the equation

$$\overline{Q} = 230\ K\ A^{.82} T^{-.48}$$

which had a standard error of + 30% and - 23%.

FREQUENCY

Following the method of Darlymple (1960, 21) a dimensionless regional frequency curve for the area was prepared using seven stations with about 30 years of record. Ratios of floods with 2.33, 10, 25, 50, and 100 years return period (RI), to the mean flood (RI = 2.33) were computed to obtain dimensionless values from which to prepare a regional graph for natural streams. (See upper curve of Figure 5). Because no data were available for impervious basins, it was assumed that for completely impervious basins, the dimensionless ratio would be constant and approach that of rainfall. Thus the lowest curve of Figure 5 was prepared from rainfall data. An empirical abacus was constructed to interpolate data between impervious and natural basins and prepare the other curves of Figure 5.

CONCLUSIONS

This study provided an empirical tool for computing discharge of floods of any frequency in Fairfax County for drainage areas of any imperviousness. The tool was used to compute the 25, 500 and 100 year flood at several sites on every basin in the County. The procedure was as follows:

- The area of each drainage basin (A) was planimetered from topographic maps at the largest scale available.

- The length (L) was measured on the map along the longest watercourse.

- The elevation at 10 and 85 percent of the length was interpolated from the contours and the difference over the length was computed as the slope(s).

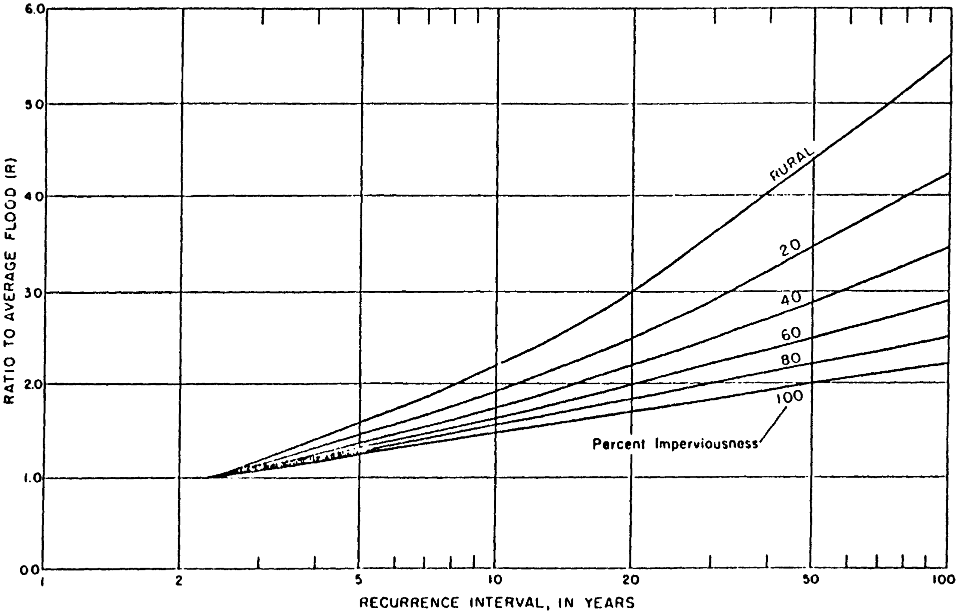

Figure 5.--Dimensionless flood frequency peaks of various frequency to the average flood (from Anderson, D.G., 1970).

- The lag time (T) was computed from the above measured $L \cdot S^{-1/2}$ values employing the formulae appropriate to the degree of imperviousness of the basin.

- The value of K was computed from the ultimate assumed value of imperviousness of the basin as estimated by the regional development plans of the County.

- The flood values were computed from straight application of the estimating equation.

As a general evaluation of the effect of urbanization on flooding, the equation was fitted to several basins of various areas, slopes, basin imperviousness, and storm sewer development. Results indicated that: (a) emplacement of storm sewers with or without regard to basin imperviousness increases peak flows by a factor of 2 or 3 and (b) a completely developed (impervious) basin will exhibit a mean annual flood 2.5 times greater than an equivalent sized natural stream basin, but no difference will exist between the two types of basins for the 100 year flood.

URBAN FLOOD MAP PREPARATION

In order to prepare flood maps of sufficient detail for zoning purposes, a special aereophotogrammetric mission was flown over the county with the objective of obtaining images allowing photogrammetric countouring of elevations at 2 foot intervals. The final maps were prepared at the scale of 1 to 1200.

Field surveys were conducted to measure the channel cross-sectional areas and to estimate the channel roughness coefficients to enable computation of flood peaks by standard hydraulic formulae for open channel flow. Manning's formula and roughness coefficient were generally used, save for bridge and culvert sites where USGS computational techniques of flow through constricted openings were applied.

The County was provided with maps showing computed flood boundaries for 20, 50, and 100 year floods of streams except for the smallest tributaries over the entire County, taking into account projected levels of regional development. The County chose the 100 year flood boundary as the flood plain zoning criterion.

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RISK ASSESSMENT AND HYDROLOGIC HAZARDS

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ABSTRACT

Planning that takes into account the potential impact of hydrologic hazards on water-resource systems is possible. Using methods of Bayesian decision analysis that consider risk and uncertainty, the water resources planner can better evaluate the chance that a project will successfully withstand a disastrous hydrologic event.

An elementary decision problem is developed. Bayes' Theorem is introduced and incorporated into the decision problem. Several measures of hydrologic risk are discussed and a case study water-supply reservoir is summarized.

INTRODUCTION

The water-resources manager usually has a number of conflicting objectives to meet. One of these objectives is to plan or manage a project taking into consideration the potential for catastrophic events such as flood or drought. Since the manager has no choice but to make decisions, it is important that decisions be made that take into account hydrologic hazards as uncertainties.

The paper will show some simple examples of decision analysis, including Bayesian decision analysis which is based on a probability formula called Bayes' Theorem. Several measures of hydrologic risk have been recently proposed (Hashimoto et al., 1982). These will be summarized and a simple case study discussed.

DECISSION ANALYSIS

How do we make decisions when there are risks and uncertainties involved? How do we factor into our decisions limited knowledge and our intuitive notions about risk? The decision maker is often confronted with problems where alternative choices have uncertain outcomes, all of which are risky and none of which are clearly preferable. Decision analysis allows the decision maker to do more than simply flip coins and hope for the best. It provides a rational, structured method to follow in making decisions and, in the process, provides the decision maker with valuable insights into the decision procedure.

In simplified, general terms, a sequence of decisions and events consists of three parts (Figure 1). The first part is the choice of a decision from the set of possible actions the decision maker has available. The second part is the set of conditions or states of the system being acted on which determine the course of subsequent events, and the third part is the set of subsequent events or outcomes of the action. Given the set of outcomes, new decisions are made and the process repeats itself.

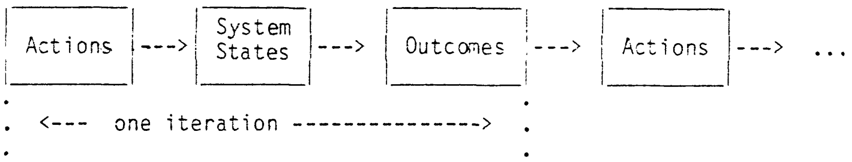

Figure 1. The process of decisions and events.

The decision analysis, in its simplest form, can be done by setting up a decision tree. Given two possible actions, A_1 and A_2, and two possible system states, S_1 and S_2, the tree is constructed as shown in Figure 2. It has four outcomes, which correspond to each possible pair of actions and system states. D denotes a point where a decision is made, C denotes a point where chance is taken into account, and O is the point where the outcome is realized.

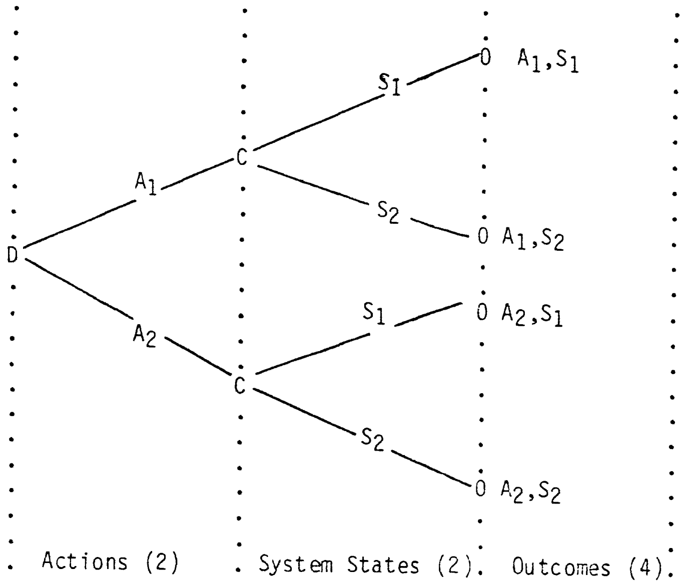

Figure 2. The decision tree.

To complete the analysis, the decision maker must assign likelihoods (probabilities, uncertainties) to the system states at points C and indicate a preference for outcomes (utilities) at points O.

The decision maker would like to have a better estimate of the probabilities of system states before he makes a decision. To do this he decides to obtain some new information. The mechanism used to incorporate the new information into the prior probability estimates of system states is an algebraic probability formula known as Bayes' Theorem. Roughly speaking, Bayes' Theorem states that the probability of the state given the information is proportional to the product of the probability of the information obtained, given the state and the probability of the state. An excellent discussion of

Bayesian decision theory for engineering applications is given by Devore (1982). An example of a decision problem adapted from this text is given in the following section.

A DECISION PROBLEM

There has been no rain for a long time. Streams are drying up and crops are wilting. The government has developed a plan of action for drought and the decision must be made to either (1) implement the plan of action or (declare a drought) or (2) wait. The decision maker is going to base his decision on soil moisture content using the criterion that if the soil moisture of the region is, on average, less than 10%, a drought will be declared and actions will be taken according to the disaster plan.

Recall that in order to do the analysis, the decision maker must assign (1) probabilities to the two ranges of soil moisture (system states) and (2) a preference (utility) to the outcomes. In the example, the decision maker has available the results of soil moisture samples taken at a number of points in the area. Sixty percent of the samples show moisture less than 10%. On this basis a soil moisture probability, (P) is assigned as shown in Table 1.

		Action on Disaster Plan	
		Implement plan	Wait
Average Soil Moisture	<10% P=.6	Correct Decision (0)	Severe Drought Impact (-200)
	>10% 1-P=.4	Unnecessary Implementation (-100)	Correct Decision (0)

Table 1. Outcomes of decision on drought disaster plan
Losses associated with each outcome are shown in parentheses.

The decision maker has also assigned preferences to the outcomes. A correct decision is rated 0. Penalties are assigned of -100 if the disaster plan is implemented and it turns out to have been unnecessary, and -200 if the plan is not implemented and a drought occurs with severe economic loss.

A decision tree can be constructed to illustrate this problem as shown in Figure 3. Beginning at the decision, D, the tree shows the two choices - to act or to wait. At C, chance takes over and reveals the true soil moisture. The decision maker assigns probabilities to soil moistures and numbers to his preference for outcomes. By combining the preferences and probabilites, numbers can be assigned to the two possibile actions that indicate the

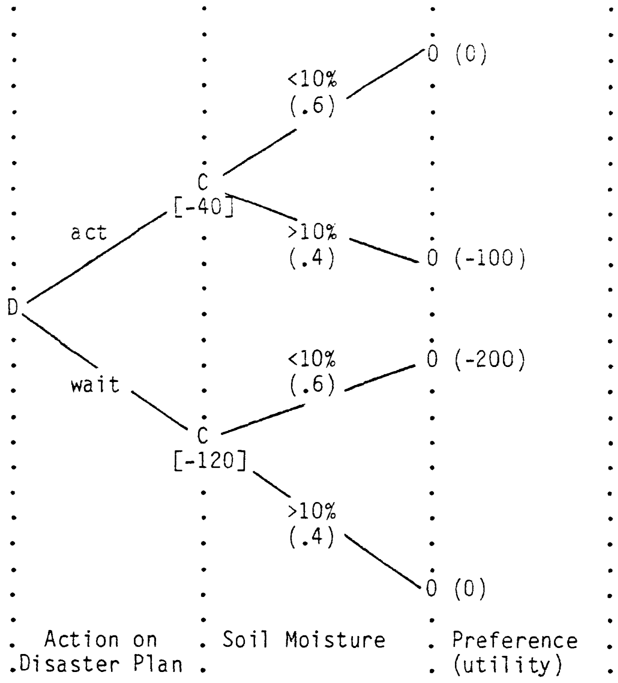

Figure 3. Decision tree for drought disaster plan.

relative gain (or loss) to be expected from the decision. In this example, the expected "losses" associated with the choices of act or wait are -40 and -120 respectively. The obvious choice is the one with the smallest expected loss, which is to implement the disaster plan.

BAYES' THEOREM

Recall that Bayes' Theorem is a probability formula obtained by algebraic manipulation of some elementary probability statements. The use of this formula in decision analysis allows the decision maker to make use of new information such as that gained by experiment or sampling.

For this problem we can suppose that the decision maker does not have much confidence in the soil moisture measurements. He decides to obtain some additional information by airborne remote sensing using a technique based on sensing at two wavelenghts, W_1 and W_2, which depend on soil moisture.

Flights over areas of known soil moisture have yielded performance data for the remote sensing method as shown in Table 2.

		True Soil Moisture State	
		< 10%	> 10%
Observed Wavelength Relation	$W_1 < W_2$	.7	.4
	$W_1 > W_2$	.3	.6

Table 2. Soil moisture probabilities from remote sensing data.

This table is read in the following way: "the probability of W1 being less than W_2, <u>given</u> that soil moisture is less than 10 percent, is equal to .7" or, using symbols, $P(W_1 < W_2 / < 10\%) = .7$. Probabilities for the other three cases are constructed similarly. Note that the probabilities sum to 1 in the columns (.7 + .3 = 1.0, .6 + .4 = 1.0).

The flight takes place and the sensors show values of W_1 greater than W_2 over the survey area. That is, the region does not appear to be as dry as the soil moisture data have suggested. This new information is now used to revise the probabilities of soil moisture state by applying Bayes' Theorem. The calculation yields the "probability of soil moisture being greater than (or less than) 10% given the observation that W_1 is greater than W_2." Or, symbolically, $P(> 10\% / W_1 > W_2)$ and $P(10\% / W_1 > W_2)$. These are referred to as posterior probabilities. Using the values in Table 2, Bayes' Theorem gives posterior probabilities of .43 and .57 for soil moisture values of < 10% and > 10% respectively (the calculations are shown in the Appendix). Substituting these values in the tree in Figure 3 gives penalties of -57 and -86 respectively for the act and wait choices.

Examining this result, we can see that the penalty associated with a decision to wait has decreased significantly (from -120 to -86). On the other hand, the penalty of acting has increased (from -40 to -57).

MEASURES OF HYDROLOGIC RISK

Decision problems in hydrologic applications are much more complex than the illustration just given. An excellent general treatment of risk assessment and decision analysis in the context of water resources systems is given in Loucks et al; (1981). Specific examples of the use of decision analysis are given in papers by Keeney and Wood (1977) and Krzysztofowicz and Duckstein (1979).

Fiering, and more recently, Hashimoto et al., (1982) have proposed three measures of hydrologic risk: reliability, resiliency, and vulnerability. These measures are proposed as criteria to be used to evaluate the possible performance of waterresources systems. They are to be included as alternatives along with the many other objectives considered in a multiple-objective decision analysis.

These measures are summarized below and an example of their use in a reservoir operation problem is given in the next section.

Reliability

The probability that a system will perform without failure is termed its reliability. The system that, on average, fails one time in ten has a reliability of 0.9. The probability that the system will perform satisfactorily is 0.9.

Resiliency

An important quality of a system is how quickly it can recover once it has failed. Resiliency is used to describe the ability to recover quickly; a resilient system is one that recovers quickly when failure occurs. Hashimoto et al. (1982) quantify resiliency as the probability of the system returning to satisfactory operation in a fixed time period, given that it has failed. A resilient system has a high likelihood of returning to satisfactory operation a short time after failure.

Vulnerability

Once failure has occurred, how serious is it likely to be? Vulnerability is used to describe the likelihood of severe failure or a failure of large magnitude.

A Water Supply Reservoir

Hashimoto et al. (1982) choose the problem of a small reservoir, which is to provide water for summer irrigation and to maintain minimum flows, to illustrate the concepts of reliability, resiliency, and vulnerability in choosing a reservoir operating policy.

The standard operating policy is to release the summer demand (the target demand) for irrigation, even if the reservoir must be emptied to do so. Where more water is available than required by the target summer demand, the reservoir is filled. Filling can continue until excess water is spilled. This standard policy is shown in Figure 4, which illustrates a system with a target irrigation demand of four units and a reservoir storage capacity of five units.

The solid line represents the standard operating policy. It shows that all available water is released for summer use until the target demand of four units is reached. At this point the demand is met, the reservoir begins to fill and summer release remains constant at four units. When the reservoir fills beyond the five units of storage, excess water is spilled, which increases summer release beyond four units.

An alternative operating policy is to guard against possible future deficits by never releasing all of the water in storage. That is, summer release may be kept less than the water available in order to store a little water in the reservoir for future needs. An example of this kind of policy is shown in Figure 5. It will be referred to in the following as the saving policy.

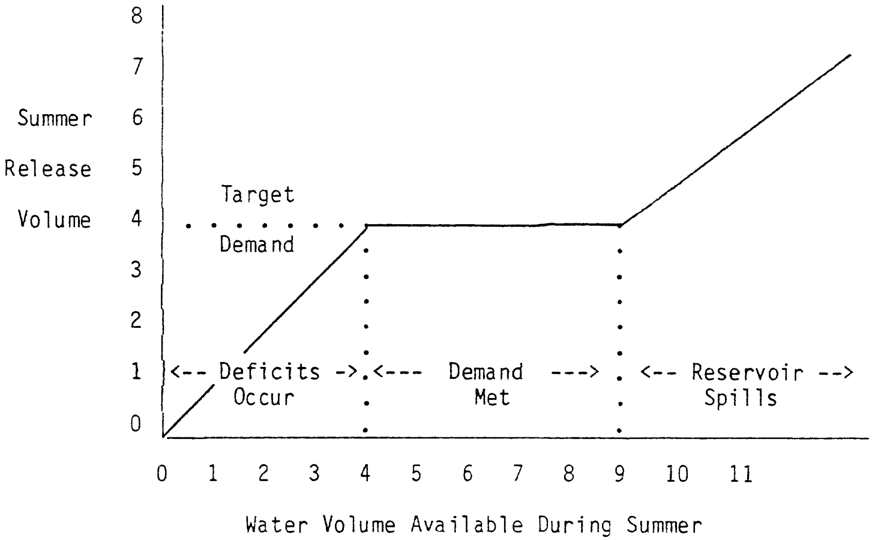

Figure 4. Standard policy for reservoir operation.

In Figure 5, it is assumed that four units of water are stored at the beginning of the operation and that, if no more water is available, summer release will be two units. As more water becomes available, releases are increased until the target is met at about seven units of available water.

The important difference between the saving policy and the standard policy is that saving leads to deficits (summer release for irrigation is less than target) even though the reservoir holds enough to meet the target.

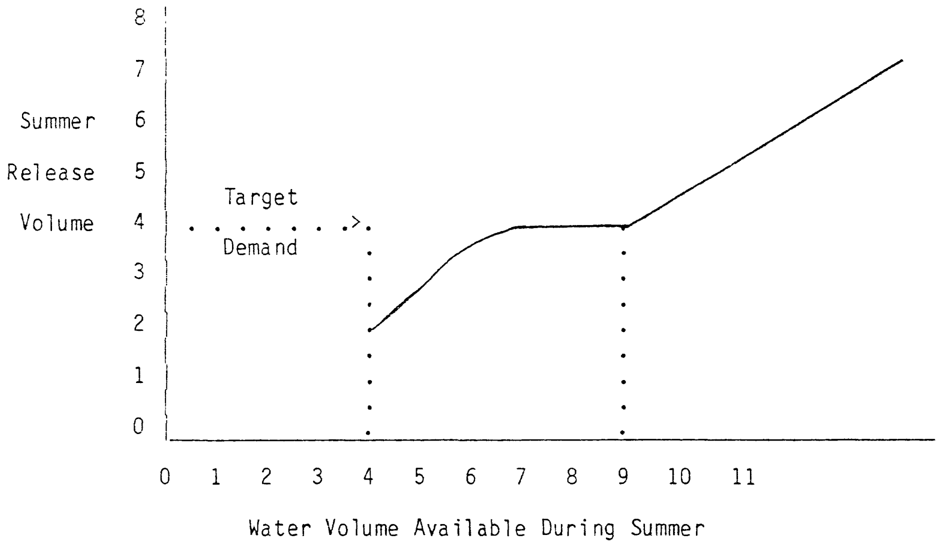

Figure 5. Saving policy for reservoir operation.

Given these two operating policies, it is possible to develop values of reliability, resiliency, and vulnerability by simulation. System failure is defined as not meeting summer target demand. Hashimoto et al. (1982) use a 10,000 year simulation and derive values for a number of operating policies including the ones outlined here.

A qualitative comparison among the two operating policies (adapted from the more detailed treatment in Hashimoto et al., 1982), is shown in Figure 6.

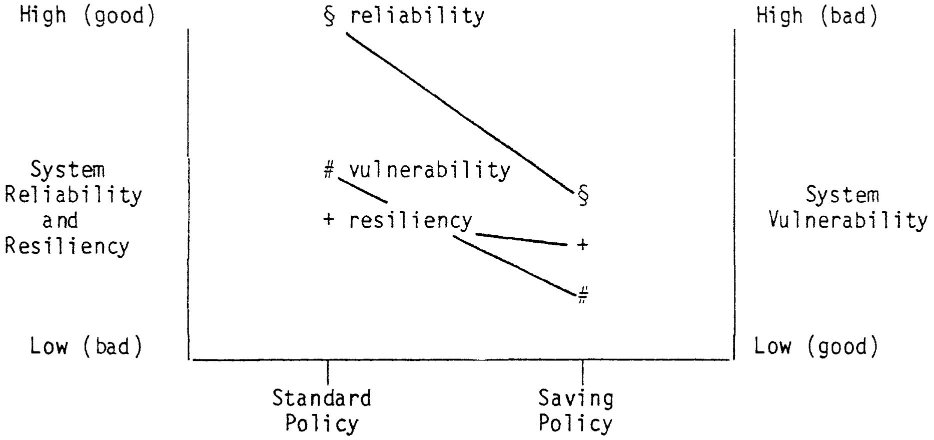

Figure 6. Risk measures and reservoir operating policy.

The saving policy has a significantly lower reliability because the operator is willing to undergo small deficits in order to save. (Recall that a small deficit is just as much a failure as a large one.) Resiliency is somewhat lower for the saving policy, because sequences of summers in which release targets are met will on average be longer.

However, the saving policy is less vulnerable to large deficits than the standard policy. That is, by saving and accepting small deficits the catastrophic failure is avoided.

Taken together, this example of reservoir operation illustrates a tradeoff between reliability and resiliency on the one hand and vulnerability on the other. To have a system that is low in vulnerability, some reliability and resiliency must be sacrificed.

CONCLUSIONS

In the example of the reservoir operation for water supply, it is shown that measures of system uncertainity can be applied to the system and that these measures can be of use in reservoir operation. The vulnerability of the system to catastrophic failure can be decreased by choosing an appropriate operating policy, but the decrease comes at the expense of a decrease in system reliability and resiliency. Put another way, the reservoir operator has to be willing to operate with small deficits, which may tend to be prolonged, in order to avoid the large, catastrophic deficit.

Hydrologic hazards, such as flood and drought, can enter the picture through the streamflow sequences used to simulate reservoir operation. These sequences are then combined with various operating policies and values of the three risk measures are obtained. At this point, these risk values, which have the hydrologic hazard information built into them, are included as objectives (preferences) in the multiple-objective decision problem. In this way, the connection between hydrologic hazard risk assessment and the decision process of the water-resources planner and manager is completed.

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APPENDIX

Example of use of Bayes' Theorem to calculate probabilities for the problem of drought disaster plan implementation.

Need

$P(<10\% \mid W_1 > W_2)$

$P(>10\% \mid W_1 > W_2)$

The decision makers best guess based on information at hand

$P(<10\%) = .6$

$P(>10\%) = .4$

$P(W_1 < W_2 \mid < 10\%) = .7$

$P(W_1 > W_2 \mid < 10\%) = .3$

$P(W_1 < W_2 \mid > 10\%) = .4$

$P(W_1 > W_2 \mid > 10\%) = .6$

The probabilities needed for Bayes' Theorem:

$P(W_1 < W_2 \text{ and } < 10\%) = P(<10\%) \times P(W_1 < W_2 | < 10\%) = .6 \times .7 = .42$

$P(W_1 > W_2 \text{ and } < 10\%) = P(<10\%) \times P(W_1 > W_2 | < 10\%) = .6 \times .3 = .18$

$P(W_1 < W_2 \text{ and } > 10\%) = P(>10\%) \times P(W_1 < W_2 | > 10\%) = .4 \times .4 = .16$

$P(W_1 > W_2 \text{ and } > 10\%) = P(>10\%) \times P(W_1 > W_2 | > 10\%) = .4 \times .6 = .24$

$P(W_1 < W_2) = P(W_1 < W_2 \text{ and } < 10\%) + P(W_1 < W_2 \text{ and } > 10\%) = .42 + .16 = .58$

$P(W_1 > W_2) = P(W_1 > W_2 \text{ and } < 10\%) + P(W_1 > W_2 \text{ and } > 10\%) = .18 + .24 = .42$

Bayes' Theorem for posterior probabilities

$P(<10\% \mid W_1 > W_2) = P(W_1 > W_2 \text{ and } < 10\%)/P(W_1 > W_2) = .18/.42 = .43$

$P(>10\% \mid W_1 > W_2) = P(W_1 > W_2 \text{ and } > 10\%)/P(W_1 > W_2) = .24/.42 = .57$

HYDROMETEOROLOGICAL EVENTS

by

John F. Miller
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INTRODUCTION

The atmospheric part of the water cycle ends when the moisture of the atmosphere returns to the ground as precipitation. Important problems related to natural disasters arise from the variability of precipitation time and space. These problems result from either the excess of water, floods; or from an inadequate amount of precipitation for human needs; droughts. Both of these problems can be equally devastating to mankind. It is the former however to which most attention has been paid by the hydrometeorologists since means of controlling floods are much easier to resolve by engineering solutions than the vastly more complicated problem of changing the atmospheric regime to alleviate droughts. For the flood situation, input information for designing hydrologic structures to control runoff can be provided by the hydrometeorologists. The current state of the art in hydrometeorology only allows us to recognize droughts after they have begun and to describe them and measure their severity. Our knowledge of meteorology is not yet sufficient to enable us to change the broadscale weather regime which result in droughts.

Another problem of concern to hydrometeorologists is the effect of hurricanes. One aspect of hurricanes, extreme rainfall, is considered under the estimation of flood potential. A second aspect, which can be equally devastating, can be the hurricane storm surge. As with the flood situation, where the hydrometeorologists provide information to the hydrologists and engineers, the hydrometeorologists in this case provides meteorological information to the oceanographer to aid in the development of storm surge models both for forecasting and for use by the engineers in the design of coastal protective works.

OBSERVATIONS

All hydrometeorological efforts must begin with the observations of basic data. Estimation of rainfall potential depend upon precipitation measurements. Hurricane surge evaluations require a more comprehensive body of knowledge. Needed are wind observations, pressure observations, speed and direction of movement of hurricanes and observations of surge heights.

Direct Precipitation Meassurements

Hydrometeorological studies require measurements of precipitation amounts both at a point and their distribution in time and space. Many different instruments have been developed for gathering such information on precipitation. Precipitation can be measured in an open container, one with vertical sides that can be easily calibrated to determine the amount of precipitation that fell between measurement intervals is most appropriate. Since the size, shape, and exposure of a gage effect the amount caught by the container, it is desirable to develop a standard gage so observations between different stations may be easily compared. A standard non-recording gage used by the US National Weather Service has a receiver 8 inches in diameter. The rain passes through this collector into a funnel through a cylindrical tube that has a diameter 0.3162 of that of the receiver. This is done to facilitate accurate measuring, and increases the depth of water in the tube by a factor of 10 so that 0.1 in. of rainfall fills the tube to a depth of 1 in. This is the most common gage used in this country and elsewhere.

If a continuous record of precipitation is required so that indications of the time distribution can be obtained, recording rain gages are used. There are three primary types of recording gages: tipping bucket, weighing, and float. In the tipping bucket gage, the water is caught in a collector and funneled into a two-compartment receiver or bucket: 0.01 in. of rain will fill a bucket and over balance it so that it tips and empties into a reservoir, moving the second bucket into the place beneath the funnel. As the bucket tips it activates a pen arm causing it to mark on a moving drum. The weighing rain gage uses a spring or lever balance to weigh the rain or snow that falls into a bucket set on a platform. The increasing weight of the bucket causes a pen arm to rise, tracing a record on a chart on a revolving drum. A float gage has a bucket set on a platform, the float inside the bucket is linked to a pen arm that marks a trace on a chart on a revolving drum.

Since most users of precipitation data need the average depth over some area rather than a single point, precipitation data from several stations, that is a network of gages, is required. The type of storm common to the region and the ultimate use of the data should determine the density of the network. If only larger watersheds are of concern and only general storms are to be studied, a relative coarse network may suffice. If data are required for small areas, or if thunderstorm or convective rainfall are of interest, it will be necessary to have a relative dense network. The World Meteorological Organization (WMO) (1974) has recommended the following as a minimum network of densities for general hydrometeorological practices: 1) for flat regions of temperate, mediterranean and tropical zones, 230-350 mi^2 per station. 2) for mountainous regions of temperate, mediterranean and tropical zones, 40-100 mi^2 per station. 3) for small mountainous islands with irregular precipitation, 10 mi^2 per station, and 4) for arid and polar zones 600-4,000 mi^2 per station.

The error that will result from gage networks must be considered particularly when sparse networks are contemplated. The error tends to decrease with increasing duration and areal size with increasing network density. The chance of a storm center occurring over a regularly established gage in a gage network is dependent on the gage density and the type of storm being investigated. The standard suggested by the WMO are a minimum necessary to insure that the areal variability is sampled adequately. The gages within these networks should be relatively evenly distributed and not concentrated in either heavy or light rainfall regions.

Indirect Precipitation Measurements

Even with the densest rain gage networks that are possible economically, considerable sampling error can be present. These errors are less for average seasonal or monthly precipitation over areas of several thousand mi^2 than for the average depth of a storm that occurs only over a few hundred mi^2 in a duration of less than a day.

The precipitation gages just described measure the amount of rain falling in a particular location on the ground. By use of analysis procedures they can be used to estimate the amount of rainfall over various area sizes. Two indirect methods of measuring rainfall are available, radar and satellite. A radar measures the number of drops falling through a beam of a transmitter of electromagnetic energy. The measurement is the result of a reflection of radar waves by the cloud or precipitation particles and is dependent upon: 1) the drop size distribution, 2) the number of particles per unit volume, 3) whether the particles are solid or liquids, 4) shape of individual elements and 5) if the elements are asymmetrical, their aspect with aspect to the radar. This reflectivity is measured by the average return power. One of the problems with the use of radar data results from the fact the radar measures precipitation in the atmosphere. The radar signal travels in a straight line from the site at which the pulse is transmitted and due to the curvature of the earth, will be at an increasing altitude as the distance from the station increases. The information needed is that on the ground. Another factor is the loss of energy and thus reduction return power due to absorption and scattering during passage through precipitation or clouds. This latter may effect the measurements for more distant rain events. The accuracy of radar measurements of precipitation vary with duration, area, storm type and range. The areal extent of rainfall and to a lesser extent the magnitude, however, may be reliably depicted for ranges up to 125 miles.

The newest technique being used to measure precipitation over areas are based on meteorological satellites. The satellite also does not measure precipitation directly. The satellite primarily measures cloud top temperatures, brightness in the visible range and radiation reflected from Earth. It can also to some degree measure the vertical distribution of moisture. The solution of the measurement of rainfall depends on relating these measurements to the amount of precipitation that reaches the ground. Various techniques have been proposed for doing this. These techniques have been most successful in measuring precipitation over large areas and for the longer duration. As with radar data, it can be most effective in defining the limits of heavy rainfall areas.

Summary

All measurements of rainfall data are to some extent deficient. Rainfall gages, unless pit gages are used, provide a disturbance to the airflow. This disturbance tends to reduce the amount of gage catch. The amount of reduction depends upon the shape and size of the raingage. Gages measure only at particular locations, and determination of areal distribution of rainfall is dependent upon some means of estimating rainfall amounts between gages. Radar and satellite observations do not directly measure rainfall amounts. Radar measures electromagnetic reflectivity, and satellite measures cloud-top temperature, brightness, and radiation. With each of these latter 2 types of observations indirect methods must be used to estimate the rainfall amounts. They do however, provide a better qualitative guide as to the areal distribution of rainfall than do gages. The use of the direct and indirect methods in combination should then generally provide better results than can be obtained than either alone.

HURRICANE DATA

The hydrometeorologist is interested in the tropical cyclone or hurricane, to determine those factors which are important for use in models which are used to estimate storm surge heights. These are the horizontal winds, and, the speed and direction of movement. Since direct measures of the complete wind field are not available, the hydrometeorologist uses available observations of surface winds and the central and peripheral pressure to develop a model wind field. The model wind field together with measures of hurricane intensity and movement is used to compute storm surge heights. The hydrometeorologist is also interested in direct measurements of the damage causing event, the height of the storm surge.

Wind

Reliable wind observations can come from the regularly reporting network of surface observation stations, and from the wind estimates obtained by reconnaissance aircraft as they penetrate the storm circulation. The land station observations are from anemometers and should be considered the most reliable. The wind observation summary from the reconnaissance aircraft are made by experienced observers but rely on primarily on the observations of sea state supplemented by flight aircraft instrumentation. As for the central pressure observations, ship logs from those ships in the vicinity of the hurricane can be used provide supplementary information.

Central Pressure

The most important factor in the model to compute storm surge is the intensity of the hurricane which is directly related to a central pressure. The most reliable observations are the minimum pressures obtained at the regularly reporting stations using mercury barometer or a carefully calibrated aneroid barometer and minimum pressures obtained during the low level reconnaissance aircraft penetrations. The data from land stations must be evaluated in view of the hurricane track to assess if the hurricane did cross the location of the station. If the track of the hurricane did not cross the station location, appropriate techniques must be used to extrapolate the station pressure to the minimum value for the storm center. Somewhat less reliable information, but still valuable, can be obtained from observations taken by private corporations or individual citizens who maintain their own barometers. Another source of pressure information is ship logs from those ships in the vicinity of the hurricane. Not all these sources are available for forecasting purposes, but, they make useful additions in post-storm analysis.

Surge Height

Abnormally high water levels, surge heights, can be obtained from tide gage records and from high-water-mark elevations obtained by post storm surveys. For consistency, all water levels should be related to some common datam. Usually only a limited number of tide gage stations are installed and maintained by governmental agencies. In addition, frequently the storm surge height will exceed the regular tide gage capability (fig. 1). The actual height of the storm surge can only be obtained when the normal astronomical tide is subtracted from the maximum measured height. Post storm surveys are generally required to determine the location and elevation of the highest storm surge in the area. Sometimes these high-water marks will be determined from debris or drift lines, while other measurements may be made inside a house or other structure that can be compared to the stilling well of a tide gage. These latter are the most accurate of the high-water marks. Since the time of high water is not known, the effect of normal astronomical tides can not be subtracted from these high-water elevations.

ANALYSIS OF BASIC DATA

Temporal and Spatial Distribution of Rainfall

Analysis of storm rainfall amounts to be most useful to the hydrometeorologist must determine the complete temporal and spatial distribution of rainfall during the storm period.

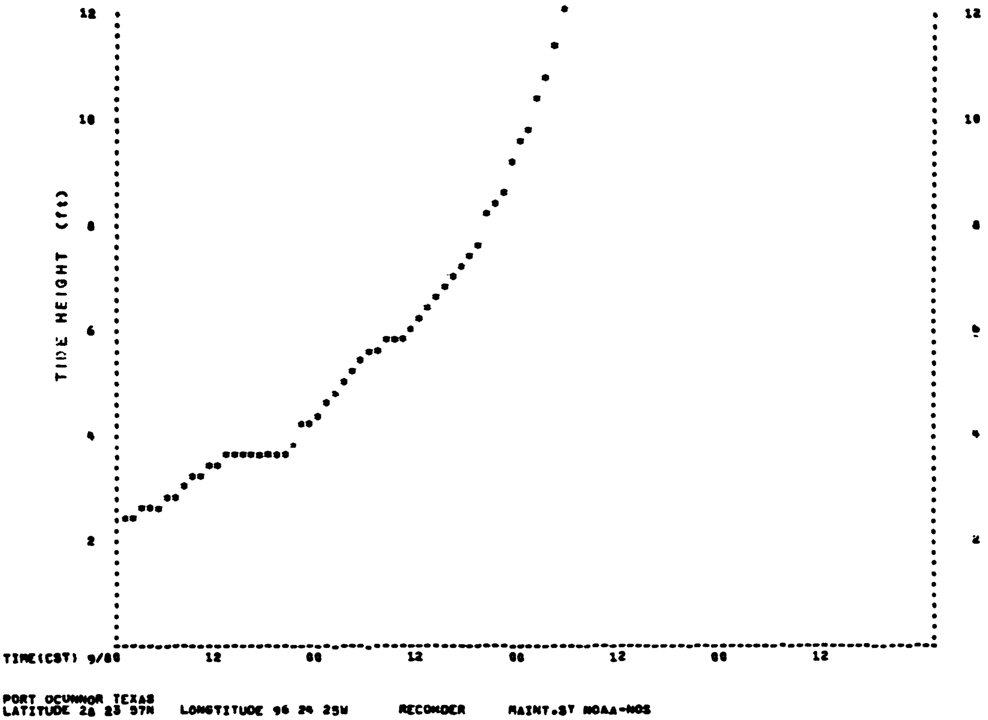
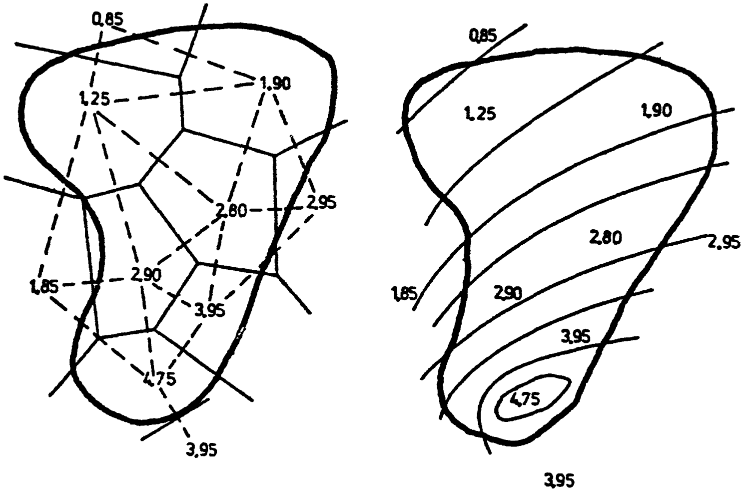


Figure 1.—Tide gage record for Port O'Connor Texas during Hurricane Carla, August 9 and 10, 1961.

Temporal Distribution. The amount of precipitation within various time intervals during the storm can be obtained from recording rain gages. Hourly amounts from these stations are accumulated and used to plot mass curves of rainfall from the beginning of the storm to the end. Accumulated rainfall values from non-recording gages are then plotted at whatever interval they are observed. Data from these latter stations are grouped geographically with data from the nearest recording gages. This latter data are then used to help shape a mass curve of rainfall from the amounts measured at the non-recording gages. The time distribution can then be approximated for all stations within the storm area.

Spatial Distribution. The simpliest technique for developing the average depth over an area is the arithmetic average of the values of the gages within the area for the time period of concern. If gages are relatively uniformly distributed over the area and the values are not greatly different from the average value, this technique will yield reliable results. If gages are not uniformly distributed or if the terrain effects are important, some form of weighting factor must be applied. The Thiessen method (Thiessen 1911) constructs polygons about each station to determine an effective area assumed for each station (fig. 2). Thus a weighted average can be obtained. This is more accurate than simple averaging. A still more accurate procedure is the use of an isohyetal analysis. In this procedure, lines of equal precipitation, isohyets, are drawn through the field of precipitation data. The isohyetal pattern is developed making use of information about the storm movement and topographic variations. The average depth of precipitation for the storm area is found by weighing the average precipitation between successive isohyets by the area between the isohyets and totaling the products.



(a) Thiessen method

(b) Isohyetal analysis

Figure 2.—Methods for determining average depth of rainfall over an area (the numbers shown are storm rainfall depths in inches).

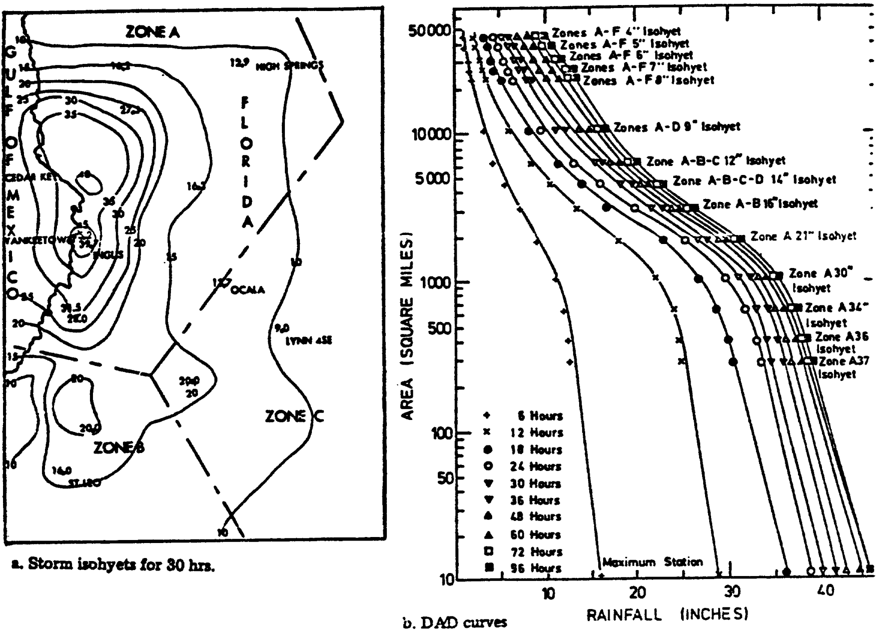
Depth-area-duration Analysis. The mass curves of rainfall and the average depth developed using isohyetal analysis can be used to develop a depth-area-duration analysis of a storm. This type of analysis shows the maximum depth for various durations over various area sizes throughout the storm (fig. 3). This procedure has been standardized and is used in many countries (WMO 1969). These maximum values for various area sizes can be plotted and enveloping lines drawn for each duration. Summaries of such data (fig. 4) then can provide a catalog of a maximum storms over a region.

Temporal Variation of Hurricane Parameters

The development of accurate hurricane wind field models requires an accurate appraisal of various meteorological parameters. The primary ones are central pressure, radius to maximum wind, and speed and direction of movement of the storm.

Central Pressure. For periods when a plentiful number of surface observations are available weather maps are drawn to make an estimate of the central pressure of the storm from the surface observations at all stations. Frequently pressure profile formulas are used to extrapolate inward from the observed data. These estimates of central pressure together with pressure from aircraft reconnaissance are plotted on a time profile and a curve of best fit drawn (fig. 5). The time period should generally extend 36 hr prior to and 24 hr after landfall.

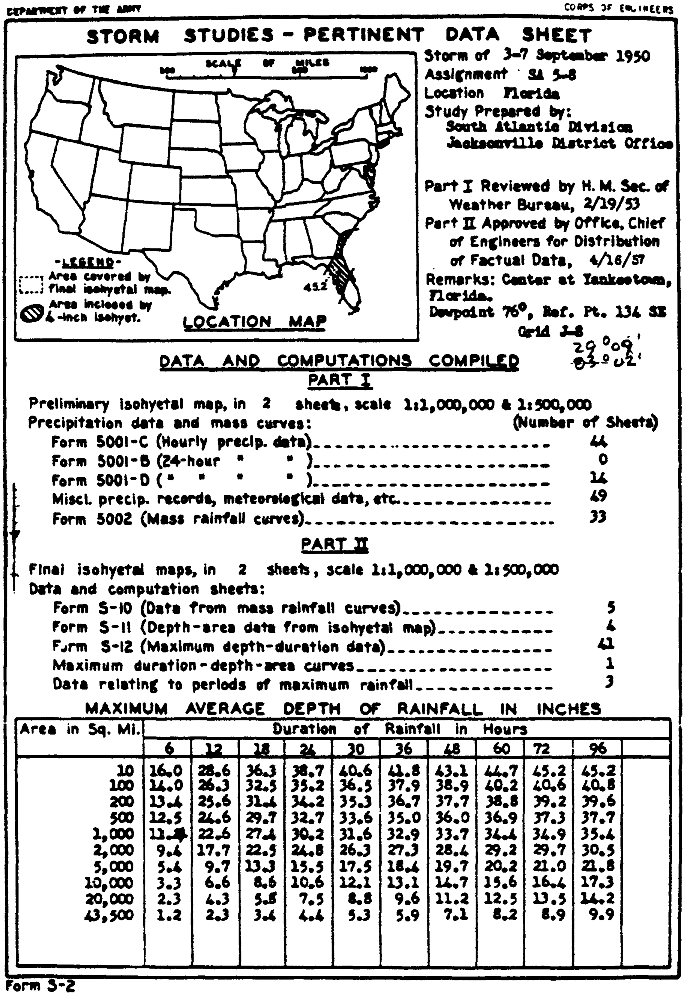
Radius to Maximum Wind. The size of a hurricane is commonly denoted by the distance between the lowest central pressure and the band of highest winds



a. Storm isohyets for 30 hrs.

b. DAD curves

Figure 3.—Maximum depth-area-duration curves for storm of September 3-8, 1950, centered near Yankeetown, Florida.



DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS

STORM STUDIES - PERTINENT DATA SHEET

Storm of 3-7 September 1950
Assignment SA 5-8
Location Florida
Study Prepared by:
South Atlantic Division
Jacksonville District Office

Part I Reviewed by H. M. Sec. of Weather Bureau, 2/19/53
Part II Approved by Office, Chief of Engineers for Distribution of Factual Data, 4/16/57
Remarks: Center at Yankeetown, Florida.
Dewpoint 76°, Ref. Pt. 134 SE
Grid J-8
29°09'
83°02'

-LEGEND-
Area covered by final isohyetal map.
Area inclosed by 4-inch isohyet.

LOCATION MAP

DATA AND COMPUTATIONS COMPILED

PART I

Preliminary isohyetal map, in 2 sheets, scale 1:1,000,000 & 1:500,000

Precipitation data and mass curves:	(Number of Sheets)
Form 5001-C (Hourly precip. data)	44
Form 5001-B (24-hour " ")	0
Form 5001-D (" " " ")	14
Miscl. precip. records, meteorological data, etc.	49
Form 5002 (Mass rainfall curves)	33

PART II

Final isohyetal maps, in 2 sheets, scale 1:1,000,000 & 1:500,000

Data and computation sheets:	
Form S-10 (Data from mass rainfall curves)	5
Form S-11 (Depth-area data from isohyetal map)	4
Form S-12 (Maximum depth-duration data)	41
Maximum duration-depth-area curves	1
Data relating to periods of maximum rainfall	3

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours									
	6	12	18	24	30	36	48	60	72	96
10	16.0	28.6	36.3	38.7	40.6	41.8	43.1	44.7	45.2	45.2
100	14.0	26.3	32.5	35.2	36.5	37.9	38.9	40.2	40.6	40.8
200	13.4	25.6	31.4	34.2	35.3	36.7	37.7	38.8	39.2	39.6
500	12.5	24.6	29.7	32.7	33.6	35.0	36.0	36.9	37.3	37.7
1,000	11.[illegible]	22.6	27.4	30.2	31.6	32.9	33.7	34.4	34.9	35.4
2,000	9.4	17.7	22.5	24.8	26.3	27.3	28.4	29.2	29.7	30.5
5,000	5.4	9.7	13.3	15.5	17.5	18.4	19.7	20.2	21.0	21.8
10,000	3.3	6.6	8.6	10.6	12.1	13.1	14.7	15.6	16.4	17.3
20,000	2.3	4.3	5.8	7.5	8.8	9.6	11.2	12.5	13.5	14.2
43,500	1.2	2.3	3.4	4.4	5.3	5.9	7.1	8.2	8.9	9.9

Form S-2

Figure 4.—Pertinent data sheet from "Storm Rainfall in the United States."

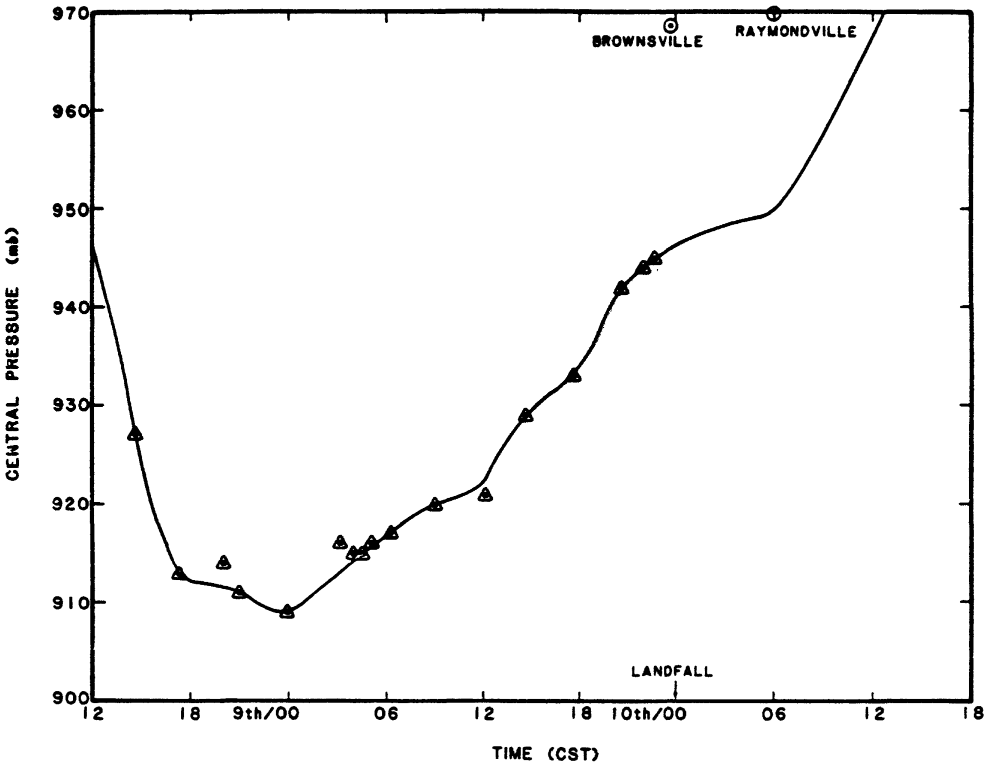


Figure 5.--Variation of central pressure with time for Hurricane Allen, August 8-10, 1980.

around the center, or radius to maximum winds. The wind field can be analyzed in two ways. First, the wind observations at the regular reporting stations are examined. In analyzing the station wind observations, the distance from the storm center and the storm speed of translation must be considered in relation to the storm speed. This will give an estimate of the time of maximum wind speed at the station. This information can be used with other data to define the radius to maximum winds. Next, a streamline analysis of the wind field at 6 hr intervals, or more frequently, should be drawn (fig. 6).

In addition, to the station observations, three different types of observations were available: 1) the maximum flight level winds and estimated surface winds as reported by reconnaissance aircraft, 2) the radar eye diameter reported by reconnaissance aircraft and surface observing stations, and 3) optical reports of the eye wall from reconnaissance aircraft. Each of these sources are used to plot profiles from the center of the storm out to the periphery for a particular time. These profiles can then become composited into a time history of the radius to maximum wind (fig. 7).

Speed and Direction of Movement. The synoptic weather charts and the streamline analysis discussed in the previous sections are used together with center locations reported by reconnaissance aircraft and eye positions determined by radar from surface observing stations to determine the storm center locations at different times. These are all plotted on a chart and a smooth best track determined. The location of the hurricane along this track is determined at frequent intervals (fig. 8). These positions along the track are then used to compute a speed and direction of translation.

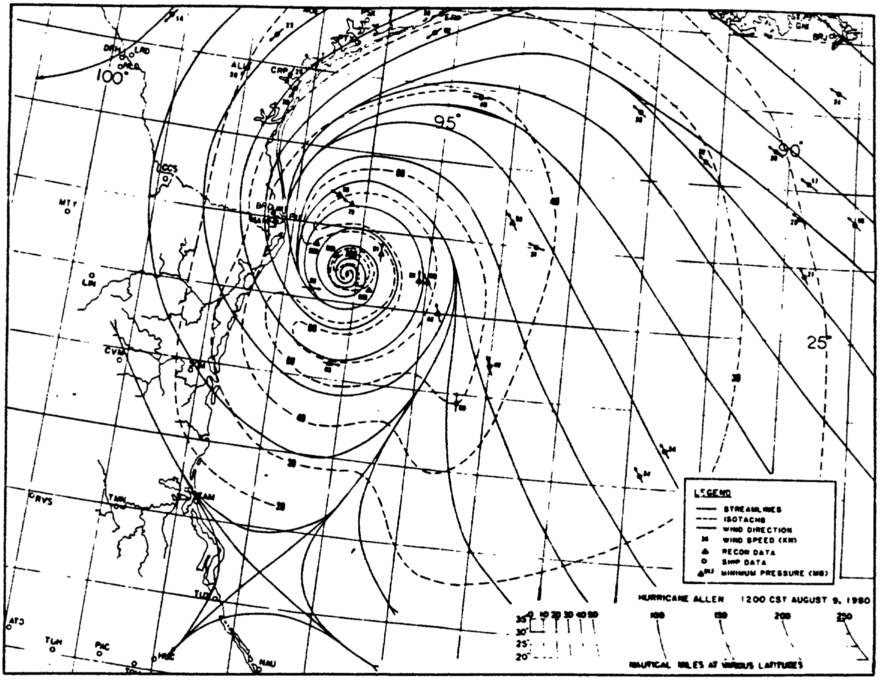


Figure 6.— Streamline analysis, 1200 Central Standard Time, August 9, 1980.

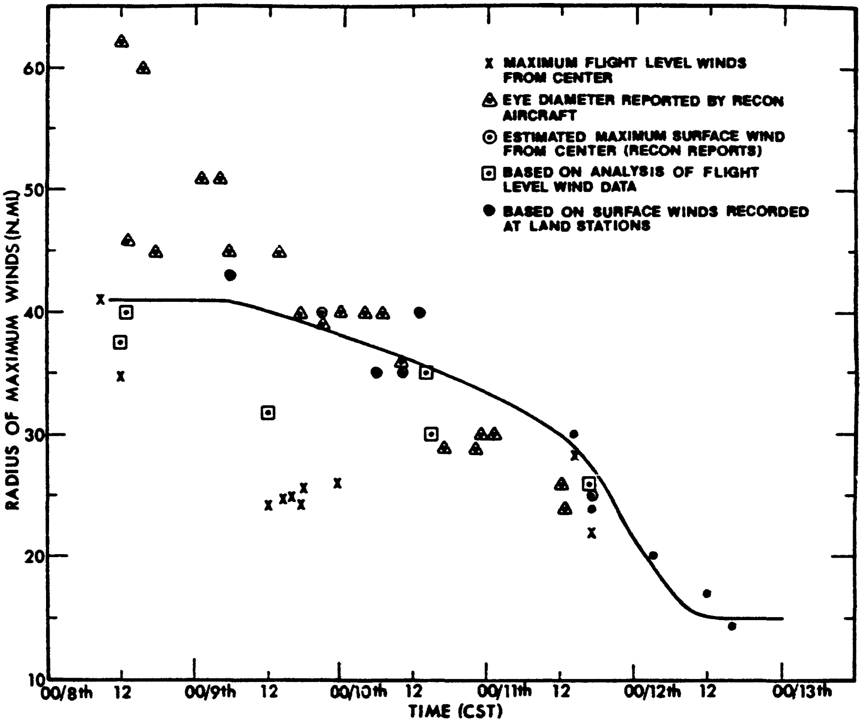


Figure 7.—Variation of radius to maximum winds with time for Hurricane Carla, September 8-12, 1961.

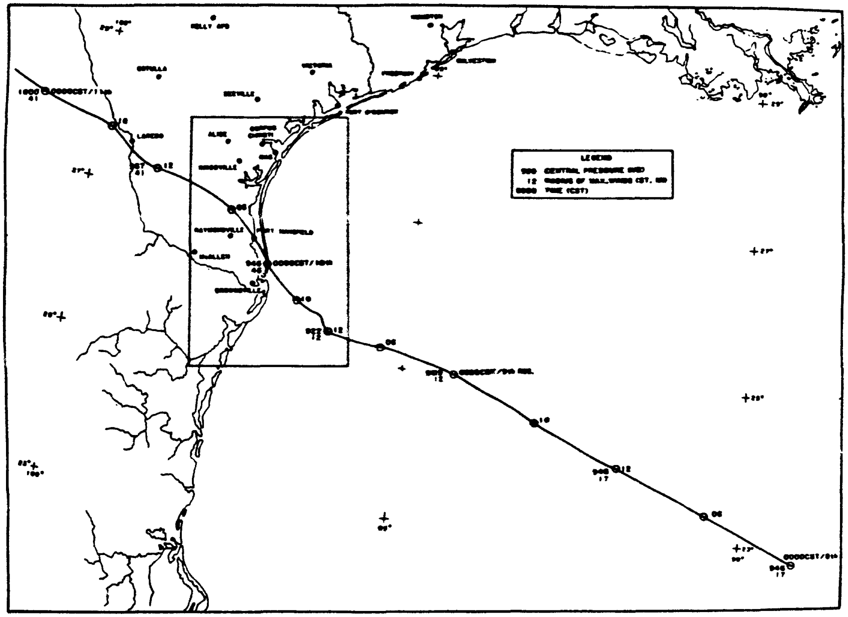


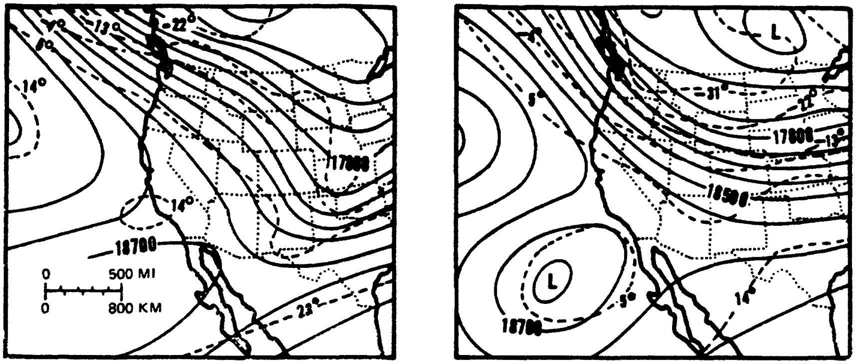
Figure 8.—Hurricane track showing positions every 6 hours from August 8 to 11, with central pressure and radius to maximum winds (st. mi.) plotted at 12-hour intervals.

METEOROLOGICAL SITUATIONS IMPORTANT TO EXTREME RAINFALL

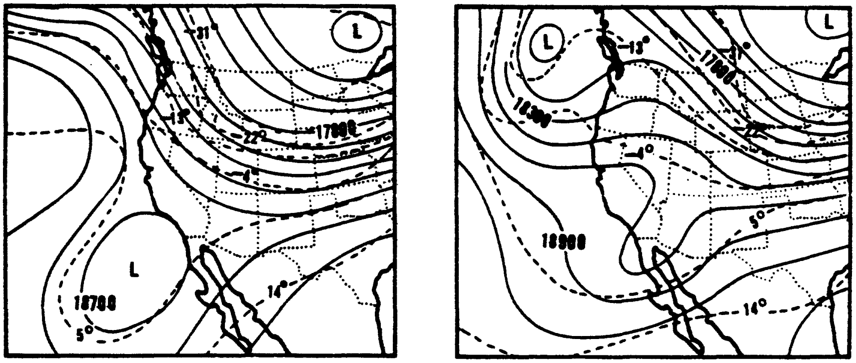
The primary concern to a hydrometeorologist are those meteorological situations which are important to extreme rainfall events. The rainfall events may result from a tropical or extratropical system on the synoptic scale which produces heavy rainfall over extensive areas, or it may be a more localized event as a result of a mesoscale extratropical system.

Synoptic Scale Extratropical Cyclones

The southwestern U.S. is heavily dependent upon an adequate water supply for agriculture and industrial uses. Through this region along the Colorado River and its tributaries many reservoirs have been constructed. These reservoirs have been designed using the results of hydrometeorological studies. A major part of these investigations is a review of an important rainstorms over the regions. Extratropical storms that have produced major rainfall over the southwest have been classified into six groups (Schwarz and Hansen 1981). These are: 1) cut-off lows, 2) high-latitude lows, 3) low-latitude lows, 4) low-latitude lows with breakthrough, 5) mid-latitude lows, and 6) lows using the Gulf of Mexico moisture. The storm of Nov. 24-27, 1967, is an example of a cutoff low situation. At 500 milibars, a cutoff low (fig. 9) was centered above 500 mi southwest of San Diego, California at 0500 Mountain Standard Time (MST) on Nov. 25. By 0500 MST 2 days later, the Low had moved over land and was rapidly decreasing in intensity. The surface weather maps (fig. 10) show that a quasi-stationary front became established by 0500 MST on the morning of Nov. 26 between cool, dry continental air north of a line from southern California and Nevada into central Colorado and warm, moist maritime tropical air to the south. During the following 24 hours the front moved southward to more east-west position extending from near Los

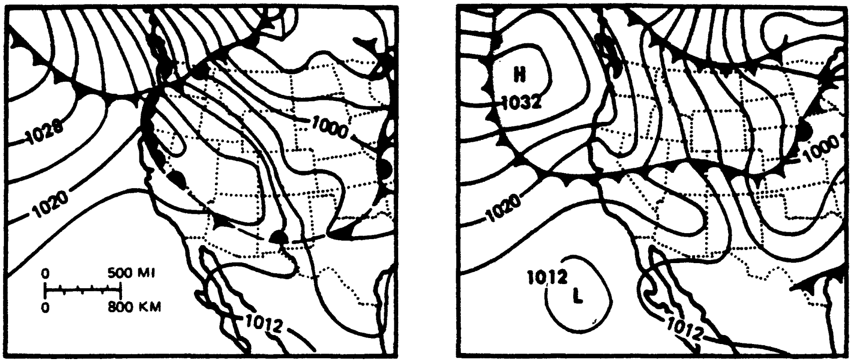


November 24, 1967 (1200 GMT) November 25, 1967 (1200 GMT)



November 26, 1967 (1200 GMT) November 27, 1967 (1200 GMT)

Figure 9.--500-mb charts for November 24-27, 1967.



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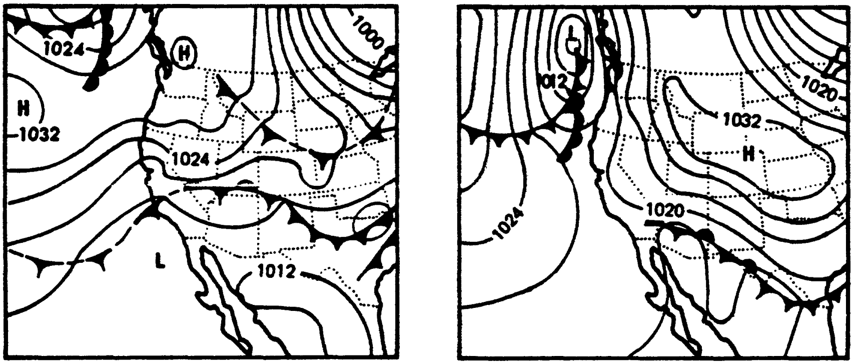


November 26, 1967 (1200 GMT)

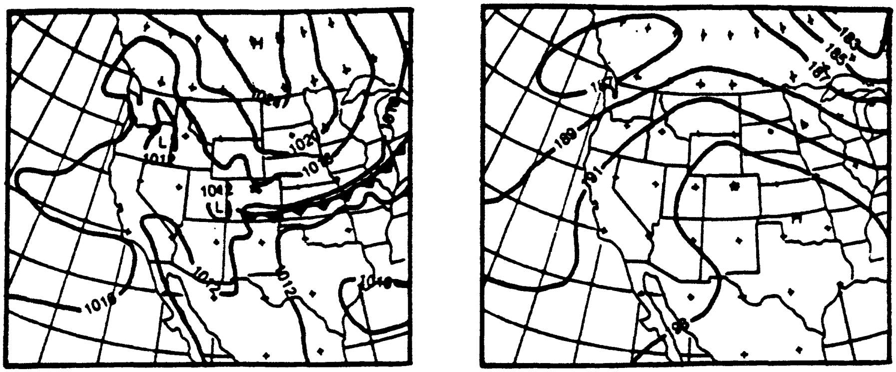
November 27, 1967 (1200 GMT)

Figure 10.—Surface weather maps for November 24-27, 1967.

Angeles to southern New Mexico. This brought it just to the north of the location where the heaviest rain occurred south of Yuma, Arizona over extreme Mexico.

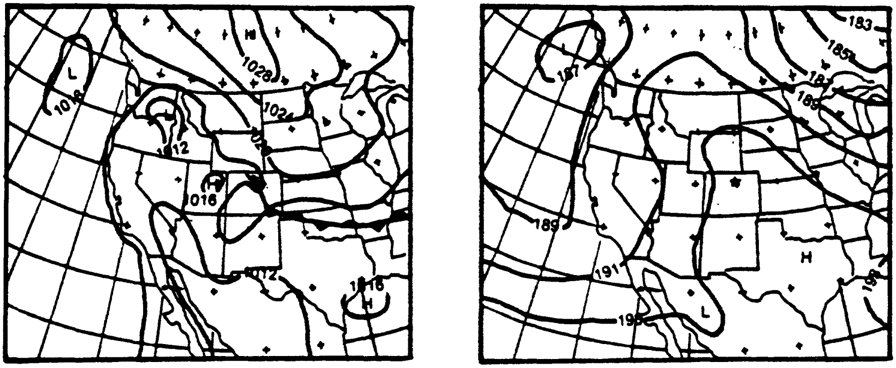
Low level moisture in the vicinity of the mouth of the Colorado River along with unstable air lead to shower activity beginning about 0800 MST on Nov. 26. The regions of showers increased in size, and numerous thunderstorms were reported over northern Mexico by 1700 MST on Nov. 26. Three hours later, moderate, continuous rain was reported over southern Arizona and southeastern California, with lighter rain occurring to the coastal regions of California. In the following 3 hours, the rain generally ceased with only scattered showers occurring over the region for the next 15 hours until the weather system moved out of the region.

The region east of the continental divide is another region of considerable hydrometeorological interest. One of the major flood causing rains occurred in this region over the Big Thompson Canyon, Colorado on the night of July 31 - Aug. 1, 1976. The flash floods were the result of a complex system of thunderstorms that had begun to develop over the Colorado New Mexico region on the afternoon of July 31. The storms formed in the humid Gulf of Mexico air that had circulated around a double frontal system extending from eastern Colorado to eastward into Kansas. Weak pressure gradients at the surface probably contributed to the quasi-stationary nature of the fronts in Colorado The surface fronts were very close together and can only be detected by analysis of more detailed than synoptic scale analysis shown here (fig. 11). Dew-point analyses over the south indicated the moist air had moved northward from the Gulf of Mexico and turned with an easterly component of flow over the Plain States. This easterly flow carried the moist air to the front slopes of the Rocky Mountains, in Colorado where it was



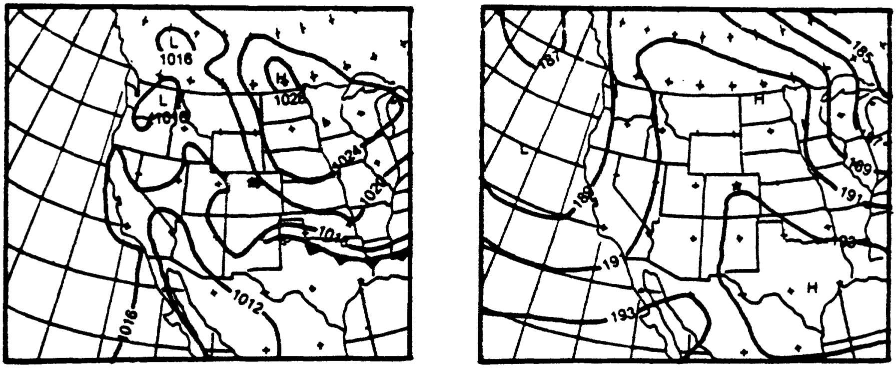
July 31 Surface 0500 MST

July 31 500 MB 0500 MST



August 1 Surface 0500 MST

August 1 500 MB 0500 MST



August 2 Surface 0500 MST

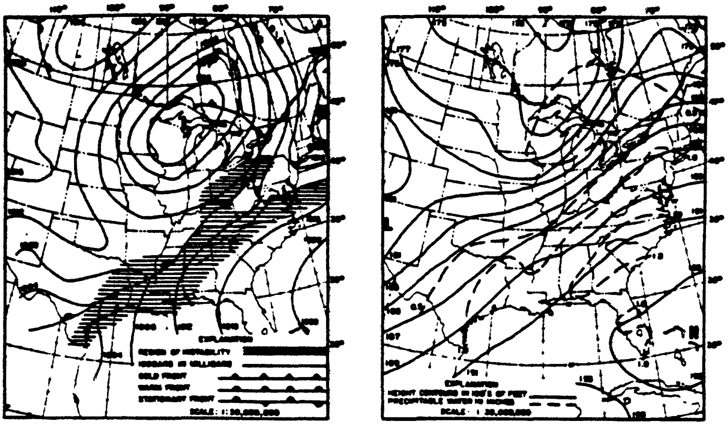
August 2 500 MB 0500 MST

Figure 11.--Surface weather maps and 500-mb charts for July 31-August 2, 1976.

lifted by both terrain and atmospheric processes. During the afternoon of the 31st, thunderstorms formed in several locations along the first upslopes to the Rockies and along the front extending to the east. The timing of the growth indicates that insolation probably played a role in the development of the thunderstorms. Radar summaries and satellite pictures indicate that the thunderstorms were growing rapidly and becoming locally intense in the Big Thompson area by late afternoon.

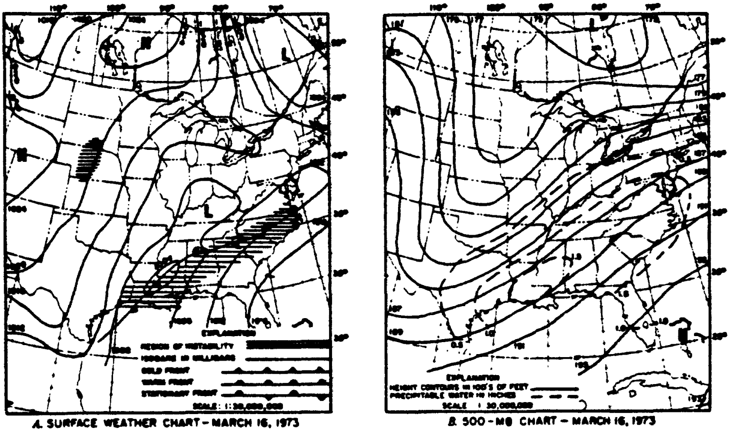
It was about this time that rain started over the Big Thompson Basin. During the next four hours heavy cloudburst-type rains fell in or near the Big Thompson Basin as these severe storms remained nearly stationary over the area. Rainfall was heaviest over the basin from about 1830 until about 2100 MST. This was due to the apparent merging of storm cells over the areas as depicted by radar summaries. Light winds aloft during the thunderstorm period also contributed to the severity of the storms by providing little entrainment of dry air from the surrounding upper levels. Light flow also permitted the storm cells to form and reform over nearly the same location. A short wave trough at 500 mb, moving north along the western edge of the ridge, was also making its way into the storm area during this time. This trough probably enhanced development of the thunderstorms increasing their severity. Rainfall diminished over the Big Thompson after about 2130 MST on the night of July 31. Other heavy rainfall occurred between 2300 and 0300 MST of Aug. 1, in areas to the north-northeast of the Big Thompson basin. These storms were not as severe as those over the Big Thompson basin. The heaviest precipitation occurred in the 10 mile wide band from about 8 mi south-southeast of Estes Park north-northeastward to the Colorado Wyoming Border.

A third example of a synoptic scale storm that caused major precipitation occurred over the southeastern United States from March 14-18, 1973. On the morning of the 13th, a low pressure system at the surface had a large, though not well organized circulation along the eastern slopes of the Rocky Mountains. Aloft the 500-mb trough associated with this system extended southward from British Columbia along the Washington-Idaho border, through eastern Nevada and western Utah, and into Arizona and New Mexico. Although not well defined, a center was present in the southern Utah region. The flow pattern over the eastern United States was still dominated by a ridge that extended northward from the Gulf of Mexico. By the morning of the 14th, the surface low had moved from the eastern slopes of the Rocky Mountains, become more intense, and was located in central Nebraska (fig. 12). The frontal system, well ahead of the Low, lay along the mid-Mississippi Valley, from southern Illinois southward across Louisiana-Arkansas border and then more southwestward to the Texas Gulf coast. At 500 mb, the trough that had been extending south and southeastward split into two separate Lows. One directly associated with the surface system was located over central Nebraska. A second circulation had developed at 500 mb over the California-Nevada-Arizona border. The second Low, as it moved eastward, induced a surface circulation over southern Arkansas toward the end of the week, prolonging the precipitation. The circulation around the surface and upper Low generally induced southerly flow through the entire troposphere over the southern half of the eastern states on the 13th and 14th. The flow in the lowest levels was primarily from the south and south-southwest, becoming more southwesterly with increasing height. The southerly flow from the Gulf of Mexico resulted in a tongue of high moisture in the Mississippi Valley by the morning of the 14th. The region of maximum instability associated with this system extended



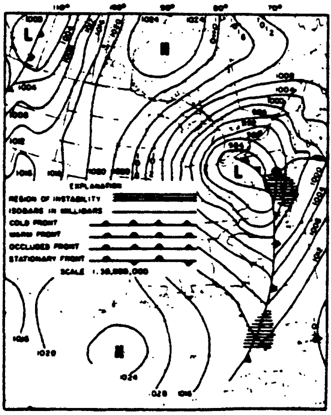
A. SURFACE WEATHER CHART - MARCH 15, 1973

B. 500-MB CHART - MARCH 15, 1973



A. SURFACE WEATHER CHART - MARCH 16, 1973

B. 500-MB CHART - MARCH 16, 1973



A SURFACE WEATHER CHART - MARCH 17, 1973

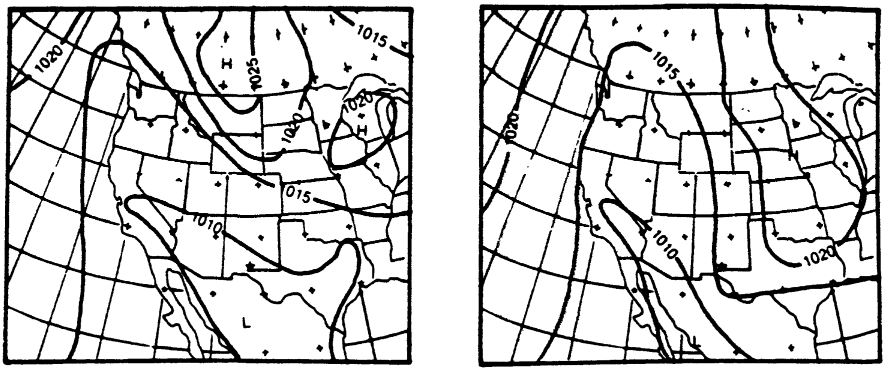
Figure 12.—Surface weather maps and 500-mb charts for March 15-17, 1973.

along a cold front northwestward from the Gulf of Mexico to southern Tennessee, across Iowa, Wisconsin and the Dakotas. It was this region of maximum instability that persisted over the region which resulted in continued rainfall amounts. The low pressure system had been centered over Nebraska on the morning of the 14th continued to move northeastward to over the central Great Lakes by the morning of the 15th; the associated cold front moved eastward much more slowly. On the morning of the 15th the front still extended across central Kentucky, eastern Tennessee, northeastern Mississippi, and northern Louisiana and into the Texas Gulf coast region. As this Low moved northeastward, it slowly began to weaken. At 500 mb, the low center that had developed over the California-Arizona-Nevada border on the 14th moved eastward to just west of the Continental Divide. The movement of this low center retained a trough position over the western part of the United States. This position of the trough caused continued southerly flow across the southern states from the surface up to the 500-level in advance of the weather system and brought a continued supply of moisture through this southeastern part of the country. During the evening of the 15th, a secondary Low developed over Louisiana along the front that extended from the low pressure system over the Great Lakes and southward across the southeastern states. By the morning of the 16th, this secondary Low was centered in central Kentucky. Still another Low developed along the front farther south of Mississippi. East and south of this nearly stationary frontal system across the southeastern states, a continuing southwesterly flow of moisture through depth maintained high moisture and instablility across the southern part of the Gulf States. The rainfall continued during this period until late on the 16th. It was during the 16th that the two Lows that had formed over the lower Mississippi River Valley combined continued to intensify and move the northeast. It was the movement of these Lows and the associated 500-mb trough that changed the circulation over the southeastern States and brought an end to the heavy rainfall.

Mesoscale Systems

Particularly important for smaller drainages are extreme rainfall events associated with mesoscale systems. An example of such a system occurred over Southern New Mexico on the night of August 29-30, 1935. This storm centered over Las Cruces, New Mexico, where 6.4 in. of rainfall was measured between the hours of 2305 MST on August 29 and 0805 on August 30, 1935. This rainfall was measured at a regular reporting station about 2 miles southeast of the center of Las Cruces. Estimates from supplementary precipitation surveys indicated the maximum rainfall for the storm was 10 inches. Synoptic surface weather maps for August 29-30, 1935 (fig. 13), show little in the way of organized weather systems. The chart for the 30th depicts the synoptics surface situation about 2 hours before the end of the storm. Of particular interest are the thermal low pressure over Mexico and the high pressure center over the plain states. These 2 features pumped moisture-rich air from the Gulf of Mexico into New Mexico. The result was an outbreak of storms on the night of the 29th and the early morning of the 30th, the largest of which fell at Las Cruces. There is a complete lack of surface weather fronts or synoptic scale systems over the entire western United States. Although adequate information are not available for detailed mesoscale analysis, similarities between this and other more recent storms indicate it was the result of mesoscale processes.

Another extreme rainfall event that resulted from mesoscale systems occurred at Johnstown, Pennsylvania on July 19-20, 1977. During this period, most of the eastern United States was under a general anticyclonic flow around

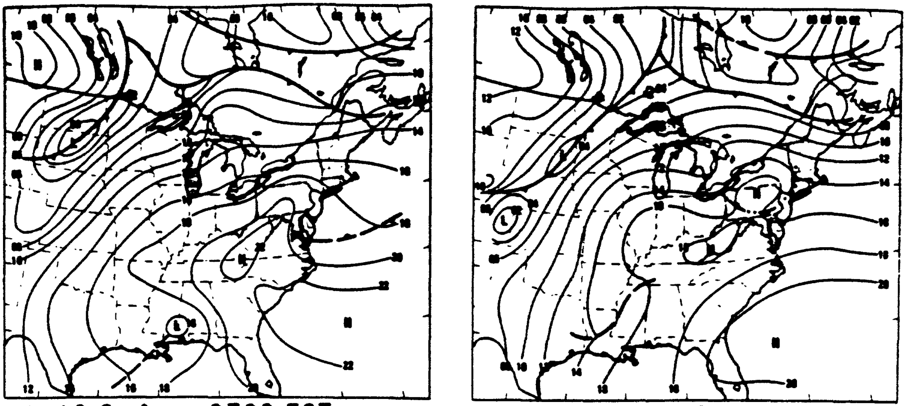


August 29 Surface 0600 MST

August 30 Surface 0600 MST

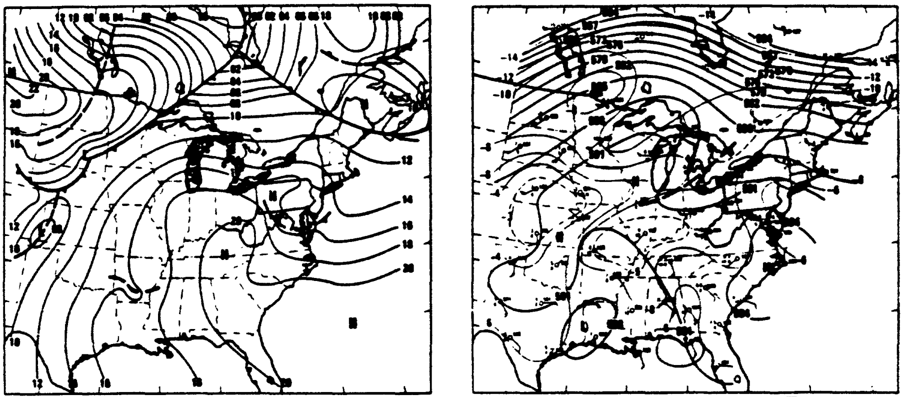
Figure 13.—Surface weather maps for August 29 and 30, 1935.

the large subtropical high located in the western Atlantic—a fairly typical mid summer pattern for the eastern United States. The major storm track was across central Canada with associated frontal systems laying across east-central Canada, southwestward into the northern high plains of the United States. Moisture flow into the Pennsylvania regions was from the southwest. Major thunderstorm outflow boundaries under this regime were generated in the Lake Erie region and moved southeastward across the middle Atlantic States. A mesoscale high pressure system was evident at both 1900 EST on the 19 and 0700 EST on the 20th (fig. 14). The flow pattern at the 500 milibar level was fairly typical for midsummer. At 1900 EST on the July 19, a broad, large-scale ridge existed over the central and eastern United States. A weak, closed, low-pressure area over the southern Mississippi Valley had a short wave trough over the eastern Great Lakes region constituted smaller perturbation in this mean pattern. A moderately strong westerly current extended across southern Canada. The short wave trough over the Lake Huron-Lake Erie region played a key role in the development of the mesoscale system. This trough moved east-southeast at 15 to 20 km across Pennsylvania and southern New York in the next 24 hours as a small 500-mb closed circulation developed over eastern Pennsylvania by the end of the period. Strong, warm advection in the lower troposphere was indicated to the west and northwest of the short wave trough. As warmer air moved under the relatively cool air aloft, the atmosphere became more unstable. Analyses of stability indices computed from standard observations show the development of a very unstable air mass ahead of the advancing short wave trough. An unstable atmosphere, unusually high moisture content, and the slow moving short-wave trough all contributed to the development of the mesoscale system and thunderstorms over a large area on the afternoon of the evening of July 19.



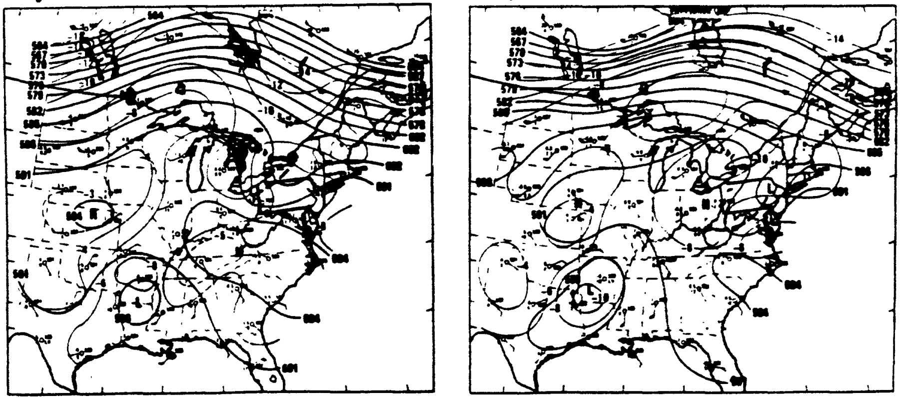
July 19 Surface 0700 EST

July 19 500-mb 0700 EST



July 19 Surface 1900 EST

July 19 500-mb 1900 EST



July 20 Surface 0700 EST

July 20 500-mb 0700 EST

Figure 14.—Surface weather maps and 500-mb charts for July 19-20, 1977.

In western Pennsylvania the boundary separating the rain-cooled air generated by early afternoon thunderstorm activity became quasi-stationary nearly perpendicular to the warm, moist, low-level flow streaming in from Ohio. As this unstable air ascended over the rain-cooled air, new thunderstorms were triggered which then moved southeastward parallel to an outflow boundary. Each thunderstorm cell reinforced the temperature gradients along the outflow boundary, and maintained the triggering mechanism the quasi-stationary position. The rains moved upslope into the western portion of the Appalachian mountains releasing the maximum rainfall to the north and east of Johnstown.

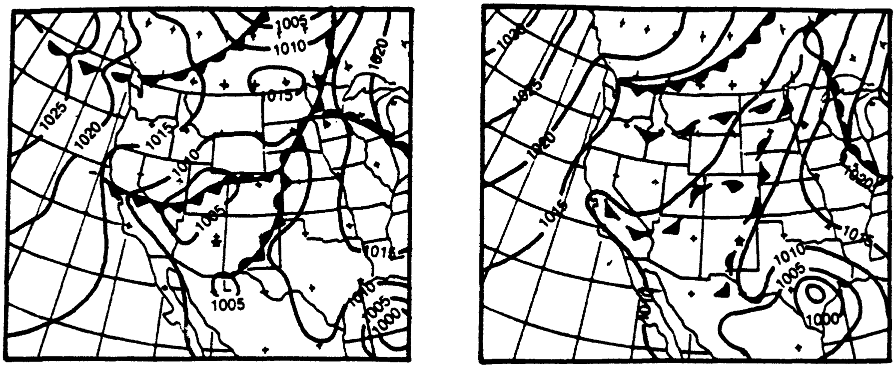
The meteorological conditions leading to extreme rainfall producing mesoscale events around Johnstown are also present in other areas when significant rainfalls have occurred. These mesoscale storms appear to have several very important characteristics: 1) very moist and conditionally unstable conditions with surface to mid troposphere moisture substantially above normal; 2) a weak short-wave trough moving through a large-scale ridge which provides lifting and air mass destabilization; 3) weak wind shear through much of the cloud which contributes to high precipitation efficiency; 4) frontal boundaries, topography, or cool outflow boundaries produced by earlier or existing convection which trigger additional storms in focus of heavy rains over relatively small regions; 5) several cells typically move over the same area and 6) some tendency for the heavy rainfall to occur at night.

TROPICAL EVENTS

Both the storm surge and rainfall produced by tropical cyclones are of interest to the hydrometeorologist. While the storm surge is of concern only along the immediate coastal area, the rainfall from tropical cyclones can have devastating effects at locations quite far removed from the coast. In some cases the extreme rainfall events will occur when the tropical cyclone circulation is no longer evident on the surface weather maps.

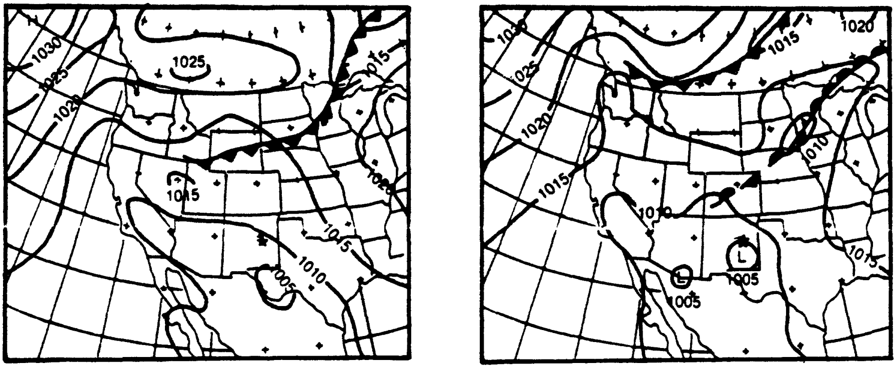
Tropical Storm Rainfall

The rainfall that occurred during the period August 29-Sept 1, 1942, over Rancho Grande, New Mexico was associated with a tropical storm which moved inland from the Gulf of Mexico on the morning of Aug. 30. The circulation from this storm was still identifiable as it entered New Mexico. The storm first originated as a tropical depression in the eastern Caribbean sea near the Gulf of Venezuela on Aug. 21, 1942. It strenghtened while moving westward, and by the evening of Aug. 24 achieved winds of hurricane force. The hurricane continued on a northwestward to west-northwestward course, cross the tip of the Yucatan Peninsula and by the morning of Aug. 29, was approaching the Texas coast. On Aug. 29, a large, maritime tropical airmass covered the eastern United States, while a polar air mass dominated eastern Canada. During the afternoon of the 29th, thunderstorm activity began over eastern New Mexico as tropical air from the Gulf of Mexico was forced upward by the terrain. The hurricane after crossing the Yucatan peninsula continued on the northwestward course and reached the Texas coast near Matagorda slightly before 0530 MST on the morning of the 30th. It continued its northwestward course and travelled toward southeastern New Mexico. The rain area accompanying the storm reached southeastern New Mexico late on the 30th and advanced steadily north for developing most for the lower Pecos Valley by the morning of the 31st. The storm center itself entered New Mexico on the morning of the 31st (fig. 15) and remained nearly stationary south of Roswell, New Mexico with steady moderate rain north of the center. It began moving



August 29 Surface 0530 MST

August 30 Surface 0530 MST



August 31 Surface 0530 MST

September 1 Surface 0530 MST

Figure 15.—Surface weather maps for August 29 - September 1, 1942.

northward again on the following morning with a definite cyclonic circulation still evident. By this time rainfall has spread northward into southeastern Colorado and ended south of the line from Albuquerque, New Mexico to Amarillo, Texas. This storm is evidence that heavy hurricane rainfall can occur quite far removed from the moisture source. The maximum average depth of rainfall over a 1,000 mi^2 area for a 24 hr period was 6.8 inches, while the 2 in. isohyet encompassed an area of over 35,000 mi^2.

Hurricane Agnes was responsible for record breaking rains from North Carolina to New York in June 1972. On June 15, a depression developed off the Yucatan coast (fig. 16). During the next 24 hrs. the system intensified and became tropical storm Agnes. On Sat., the 17th, Agnes began moving north and the following morning hurricane-force winds were found near the center which was about 250 miles west of the Florida Keys. Agnes continued northward and moved ashore as a tropical storm near Panama City late on Monday afternoon the 19th. Agnes throughout history was a minimum hurricane at best. Over the open Gulf maximum sustained surface winds reached only 85 mph. Neither the eye, nor the wall cloud ever became fully developed. However, during Monday destructive tides occurred along the westcoast of Florida. Extreme tides were about 7 ft above normal from about Cedar Key around to Appalachicola. Agnes continued northward into Georgia and weakened and then turned as a large weak depression toward the northeast across Georgia and into South Carolina. Now the principal effect was rain. In Georgia, it was heaviest in the south. In the Carolinas the mountain areas were drenched while in the central and coastal areas rain was light. The system moved northeastward across the Carolinas and crossed the coast into the Atlantic Ocean near Norfolk. Agnes moved off the coast and back out to sea late Wednesday. During the 22nd the tropical cyclone system moved up the east coast, and across Western Long

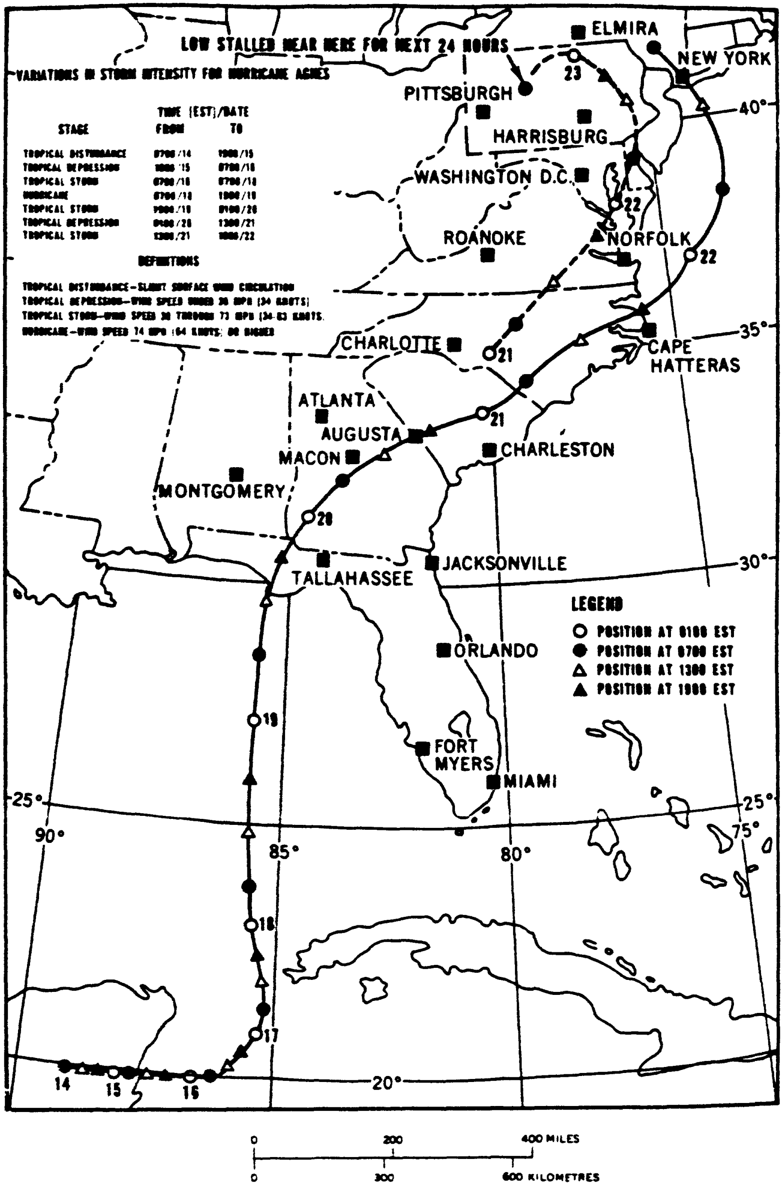


Figure 16.—Best track for Hurricane Agnes June 15-23, 1972.

Island. On Friday, the 23rd, Agnes swung southwestward as it was absorbed into a broad, deep extra tropical low pressure system in central Pennsylvania. This large system continued to dominate the weather over the northeast.

SUMMARY

It is important to recognize in each of these weather situations two primary factors. First, it is necessary to have an abundant supply of moisture available. For large scale wide spread rainfall, the moisture must be available through depth and on a continuing basis from a moisture source. This moisture supply will be above average for the region. The measure of average moisture content can be either based upon surface or upper air observations, but should be based upon at least 25 years of record.

It is not sufficient to have an ample supply of moisture. There needs to be some mechanism to cause the moisture to condense and to droplets large enough to fall as rainfall. Surface heating from sunshine during the day is generally not sufficient to cause extreme rainfall events.

ESTIMATION OF EXTREME EVENTS

The hydrometeorologists estimates the extreme events to provide information for hydrologists and engineers to permit them to design various types of hydrologic structures. These structures may be small culverts under farm to market roads, bridges across major streams, small head water retarding structures in upstream areas or major dams on large rivers. Other types of structures may be involved in coastal protective works such as sea walls, or dykes, breakwaters, berms which extend out to sea to deflect storm surges, or any other type of engineering work which would mitigate the effects of storm surges.

Probability Estimates

For smaller structures where the economic consequence of failure is only moderate and where there is little or no risk to life as a result of failure, the standard in this country is to design on the basis of probability. This probability is usually expressed in the terms of recurrence interval which is the average time interval between the occurrence of events of a equal or greater magnitude. That is if the event is said to have a 5-yr recurrence interval, or return period, it will occur on the average once every 5 yrs. The average interval must be determined over a period of time that is several times longer than the recurrence interval of interest (fig. 17). Since these events may occur successive years or with a lesser time interval in between, it is frequently more descriptive or intelligible to the average individual to express the recurrence interval as its reciprocal that is a percent chance. The 5 yr value is equivalent to a 20 percent chance of the event occurring within the year, a 50 yr value is equivalent to a 2 percent chance occurring within the year.

To determine a return period for a particular event, a particular individual probability distribution is selected. There is no meteorological or hydrological reason to expect precipiation or streamflow extremes to fit any one probability distribution. Investigations by Thom (1966), Hershfield (1962), Hershfield and Kohler (1960), Huff and Neill (1959), have tested different distributions for precipitation events and found inconclusive results. As a result many different distributions have been used to describe precipitation and streamflow distributions. Among the ones that have been used are the normal distribution, gamma distribution, the Poisson distribution, extreme value distributions particularly those of the Fisher-Tippett type and the Pearson types. The Fisher-Tippett type I distribution

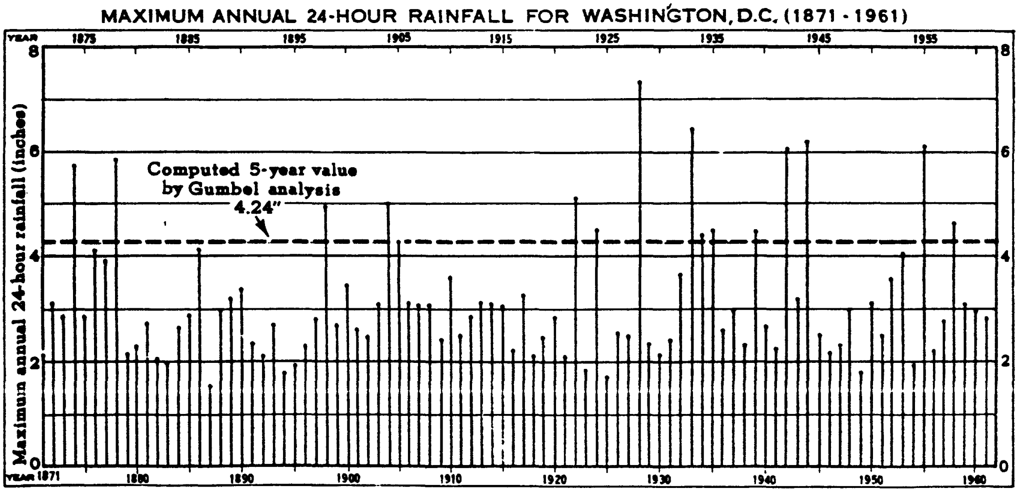


Figure 17.—Chronological distribution of maximum annual 24 hour rainfall amounts at Washington, D.C. (1871 - 1961).

(Fisher & Tippett 1928) has been used by several investigators for frequency analysis of precipitation. Particularly it has been used extensively by the United States National Oceanic & Atmospheric Administration for a series of rainfall frequency publications.

Fisher-Tippett type I is based on the fact that the maximum or minimum values selected from n samples approaches the limiting form as the size of the samples increases. The accumulative distribution function has the form of a double exponential and is given by:

$$F_r (x) = e^{-e^{-y}}$$

where $F_r(x) = Fr(P \leq x)$, e is the base naperian logarithms and y is a reduced variate which is a function of probability. Various investigators; e.g. (Gumbell 1958), (Lieblein 1954), have provided methods of fitting this type of distribution.

Some investigators have also applied the Fisher-Tippett distributions to the analysis of peak streamflow data. Since there is no theoretical basis for determining that streamflow data should fit any particular frequency distribution, the United States Water Resources Council investigated the applicability of various distributions for use in fitting the annual series of peak streamflow events. They have recommended (1982) the use of log Pearson type III distribution for estimating peak flow frequencies for gage locations on unregulated streams. The technique they recommended for fitting a log-Pearson type distribution to observe annual peaks as to compute the base 10 logarithms of the discharge as to select the exceedance probabilities by the equation:

$$\text{Log } Q = \bar{X} + KS$$

where $\bar{X}$ and S are defined below and K is the factor that is a function of the skew coefficient and selected exceedance probability. The mean, standard deviation, and skew coefficient of station data are computed using the following equations:

$$\bar{X} = \frac{\Sigma X}{N}$$

$$S = \left[\frac{\Sigma(X - \bar{X})^2}{(N - 1)}\right]^{0.5} \qquad \text{(a)}$$

$$= \left[\frac{(\Sigma X^2) - (\Sigma X)^2/N}{(N - 1)}\right]^{0.5} \qquad \text{(b)}$$

$$G = \frac{N \; \Sigma (X-\bar{X})^3}{(N - 1)\;(N-2)S^3} \qquad \text{(a)}$$

$$= \frac{N^2(\Sigma X^3) - 3N\,(\Sigma X)\,(\Sigma X^2) + 2(\Sigma X)^3}{N(N-1)\;(N-2)S^3} \qquad \text{(b)}$$

in which:

X = logarithm of annual peak flow

N = number of items in data set

X = mean logarithm

S = standard deviation of logarithms

G = skew coefficient of logarithms

The precision of values computed with the b equations ismore sensitive than with the a equations to the number of significant digits used in their calculation. When the available computation facilities only provide for a limited number of significant digits, the a equations are preferable.

The data used in the frequency analysis are obtained from the streamflow stations and the precipitation stations maintained by the various National Meterologic and Hydrologic Services. The streamflow data required are the peak discharge in cubic ft per second or meters per second. Precipitation observations required are the maximum amount that occurred during some time interval. Precipitation observations are from either a standard raingage where values are obtained by manual measurement at some fixed interval, usually daily, or from a recording gage record with a pen or trace on a chart or punched on paper tape. In either case, the data are usually reduced and published or archived for some fixed interval of time. These intervals are determined by arbitrary clock restrictions and do not usually contain the maximum amount of precipitation that occurred at the indicated time interval. Several studies by the US Weather Bureau (1955, 1956), (Hershfield, 1961), (Miller, 1964) have determined empirical factors that adjust the values obtained from a statistical analysis of fixed interval precipitation data to those obtained from analysis of the true maximum precipitation extremes. These factors are given in table 1.

Table 1. Empirical factors for converting fixed interval to true interval amounts.

Fixed interval duration	Conversion to true interval amount
1 hour	1.13
6 hour	1.02
24 hour	1.01
1 day	1.13
2 day	1.04
3 day	1.03
4 day	1.03
5 day	1.02
6 day	1.02
7 day	1.02
8 day	1.02
9 day	1.01
10 day	1.01

There are two different series that can be used in the analysis of precipitation extremes. The annual series selects the largest value that has occurred in each year of record. The year may be any consecutive 12 month period, i.e., calendar year, water year, etc. The limiting factor is that only one value is accepted for each year. In this series, the second highest from some years will be larger than the largest value in other years. A second series, the partial duration series, selects the largest events in the record without regard for the year of occurrence. The series consists of the N largest events during the period of record. The method recognizes that large events are not calendar-bound and more than 1 event may occur in the time unit used as a year. In practice the number of events selected is at least equal to the number of years of record. The only restriction on the selection of events is that the independence of the individual events be maintained. The two series get equivalent results for occurrence intervals longer than about 10 years, while the partial duration series gives somewhat larger values for the shorter recurrence intervals.

The reliability of results of the analysis of individual station data is influenced by the sampling errors in time and space. These sampling errors result from the chance occurrence of an anomalous storm which has a disproportionate effect on the statistics for one stream gaging or precipitation station but not on those for a nearby station, or by the geographic distribution of stations, placement of stations in regions of abnormally high or abnormally low precipitation. In relatively nonorographic regions, the occurrence of large precipitation events can be considered relatively random over limited geographic regions. Thus, large precipitation event (especially of a convective nature) at an individual station or over a particular basin could just as easily have occurred at a neighboring station or over a neighboring basin. These errors are independent of any possible measurement errors in the observations. One way of assessing these errors is to compute the confidence bands about the individual streamflow or precipitation frequency curves. The World Meteorological Organization and its "Guide to Hydrological Practices" (1974) provides a procedure for estimating the confidence bands about the Gumbel fit to the Fisher-Tippett type I distribution. The United States Water Resources Council in its "Guidelines for Determining Flood Flow Frequency"(1982) provides procedures for determining confidence bands about the log-Pearson type III distribution.

The station analysis still leaves considerable uncertainty about the best estimate for the streamflow or precipitation frequency value for any specific location. This is so because the streamflow and precipitation record present relatively short samples. Therefore, regionalized or generalized streamflow or precipitation frequency studies generally provide the most reliable answers at any particular location.

For streamflow values, studies by Benson (1962) have shown that it is possible to use statistical multiple-correlations techniques that will relate to floods at any defined level of recurrence interval to the hydrologic characteristics of the basin. This procedure eliminates some of the uncertainties. It is probable that different factors may be acting at the separate flood levels or that the same factors may have varying effect at different flood levels or probabilities. The multiple-correlation method when used independently at specific flood levels, allows complete flexibility and does not require making assumptions about the relation of floods of different recurrence intervals. The multiple-correlation methods are used to relate hydrologic characteristics to parameters of an assumed distribution: i.e., mean, standard deviation and skew coefficient; or to the magnitudes of floods for selected percentiles. Among the hydrologic characteristics that have been found most significant are: drainage area, main-channel slope, percentage of the drainage area occupied by lakes or ponds, average annual precipitation or a precipitation-frequency value as an index, an average basin elevation index, watershed relief, drainage density, soil types, and forest cover or other land use characteristics.

An example of such relation is that developed by the United States Geological Survey for the state of Ohio. The general formula is:

$$Q_T = a\ A^b\ Sl^c\ St^d\ E^e\ P^f$$

where: Q_T = peak discharge for the T-year recurrence interval, cfs

a, b, c, d, e, f, = regression constant and coefficients

A = drainage area, square miles

Sl = main-channel slope, feet/mile

St = percentage of the drainage area occupied by lakes, ponds, and swamps, percent plus 1.0

E = average basin elevation index, thousands of feet above mean sea level

P = averge annual precipitation, inches minus 27.0

The state is divided into five hydrologic regions. The parameters, constants, and coefficients vary with each rgion. There are separate formulas for each region to determine Q_2, Q_5, Q_{10}, Q_{25}, Q_{50}, and Q_{100}. The procedure is applicable to watersheds with area sizes between 0.01 and 7,400 square miles.

The procedures developed for generalizing precipitation-frequency values are somewhat different from those developed for streamflow values. For precipitation frequency values, generalized maps are prepared. On these maps, isopluvials are drawn which depict precipitation frequency values at specific locations. The placement of the isopluvial values is dictated by the precipitation frequency values of all stations over a particular region. A common approach to developing such precipitation frequency, maps is to develop maps for a few key durations and to interpolate values for other durations of interest. In selecting durations and return periods for development of the key maps, availability of stations and length of record are primary considerations. For durations of 24 hrs or less it is common to develop maps for the 1 and 24 hr duration and to interpolate for intermediate durations. The recurrence intervals selected should bracket the range of interest in the study. Values for the various other recurrence intervals, between those for the key maps, are determined by interpolation based on the distribution selected for use in individual station analysis. If the longest return period of interest is longer than one for which reliable estimates can be obtained from the station data, a shorter return period would be selected and the values for the longer return intervals determined by extrapolation. The interpolation between values for the various durations are usually determined by empirical relations. Generally, values from a number of long record stations with recording rain gages are analyzed for all durations of interest (fig. 18). Mathematically equations or graphical relations are developed between values for the various durations. In the United States for examples such relations have been developed for durations between 1 hr and 10 days (fig. 19). This relation is based upon data from about 200 stations geographically distributed over the United States.

The variation of station precipitation frequency values over relatively flat terrain is generally determined by various forms of station averaging. The simpliest procedure is a visual space averaging of the values for the individual stations. In this method, the data are plotted on a map along with the length of the record and the width of the confidence band at that recurrence interval. Isopluvials are drawn to the plotted values in a manner very analogous to drawing a set of lines through a field of data in a scatter

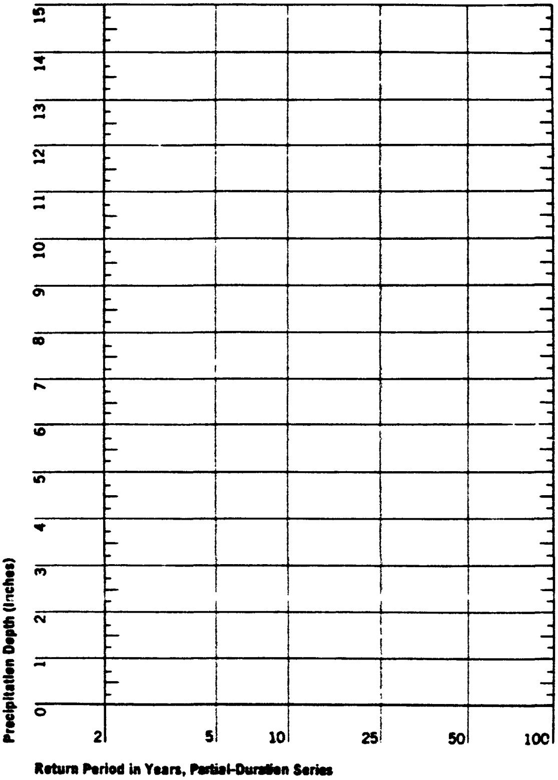


Figure 18.—Return period interpolation diagram.

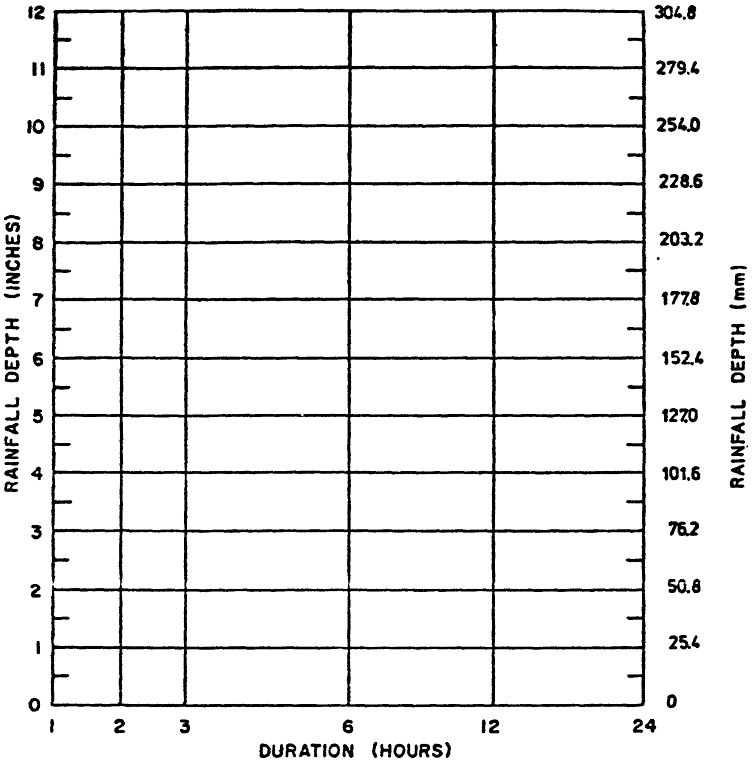


Figure 19.—Duration interpolation diagram.

diagram. As with scatter diagrams, the two criteria of smoothness and closeness of fit to the plotted points must be considered. The isopluvials are drawn with a degree of smoothness that reflects the sampling variability of the data, either drawing through every data point or eliminating all details (fig. 20). Regions of high or low values should be supported by values from several stations.

Computationally explicity objective smoothing techniques have been developed for meteorologically variables, and these techniques have been modified and adopted for analysis of precipitation frequency values (Frederick et al. 1977). This technique does not remove the judgment of the analyst in determining the appropriate degree of smoothing. The size of the area from which station should be selected which influences a particular grid point, the number of repetitions or iterations, and the form of the weighting function which determines the degree of smoothing. Whether grid point values are first computed and the analyst draws the isopluvials or whether the isopluvials are drawn by computer techniques, special care must be given to regions near the border of the study area, or to any region abuting a data void area. In these regions, the number of stations are fewest and since the geographic distribution of stations is unequal, the values may be overly influenced by station values inland from the grid point, since there are no counter-weighting values from outside the grid field.

In orographic regions, the random component of the precipitation frequency values at individual stations is reduced. Differences between stations are influenced by differences in elevation, exposure, aspect, slope and physiographic factors such that spatial averaging techniques are not appropriate. Additionally, the adequacy of the basic network of stations for determining precipitation frequency values is less in mountainous regions than

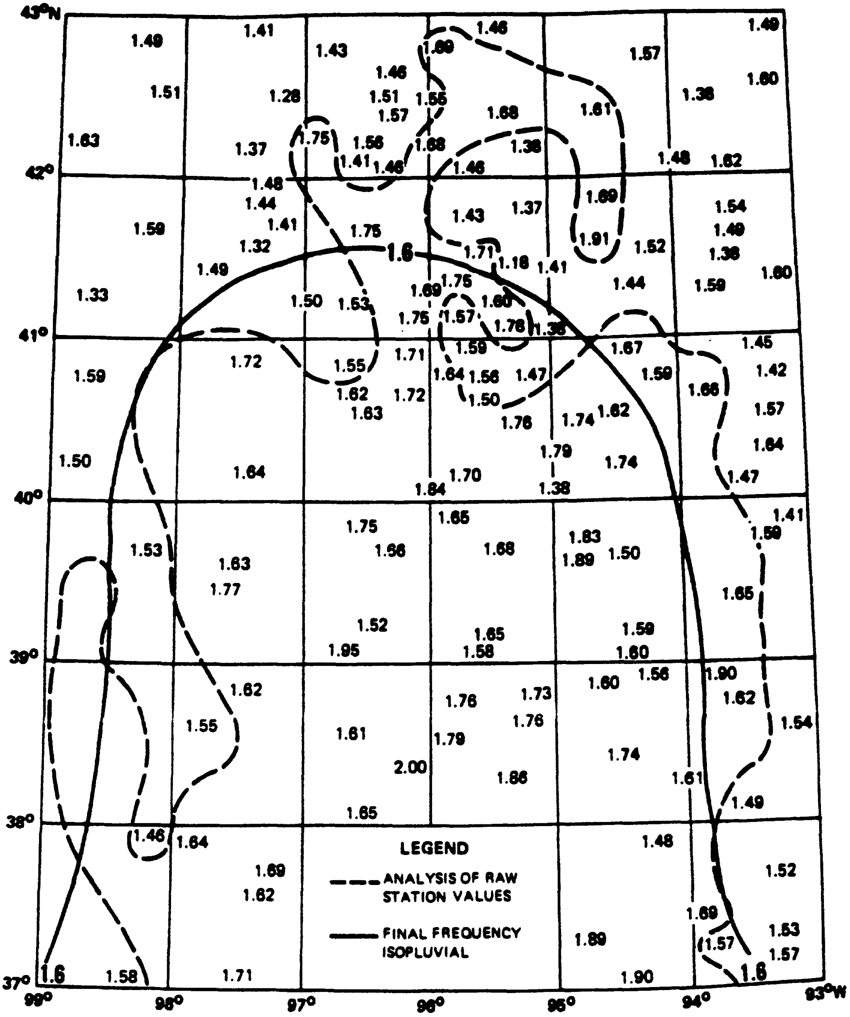


Figure 20.—Example of space averaging of precipitation frequency values in a nonorographic region.

in nonorographic regions. In orographic regions, most of the stations are located in the coastal plains or flat plains adjacent to the mountains, in river valleys, or in the lower foothills of the mountains, following population density. Relatively few stations are located at the higher elevations and on the steep slopes, in sparsely populated areas where access is difficult. For these regions, index relations based on topographic and climatic factors are usually developed to interpolate between stations. A preferred method is to develop multiple correlation relations between precipitation frequency values and various physiographic factors. These relations should be based on topographic variables to avoid transferring the interpolation problem to climatic variables about which additional information is not available. Topographic charts or maps can be prepared from surveys, aerial photographs, or other indirect methods that do not require a series of observations taken at a fixed point over some long period of time. Among topographic factors found useful are: slope of the terrain about the station, distances and direction from moisture sources, distances and directions from barriers between the moisture source and the station location, the number and height of the barrier between the station and the moisture source, roughness of the terrain surrounding the station, and the elevation or effective elevation of the station. The effective elevation is usually some average elevation determined around the circumference of some circle of selected radius. Slopes, roughness, or effective elevation should be measured over scales commensurate with the scale of the storms that produced the large precipitation amounts in the regions. Figure 21 shows an example of the 2-year 24-hour precipitation frequency chart for a portion of Colorado.

The generalized charts developed as a described in the preceeding paragraph provides values for the point for the specific duration of

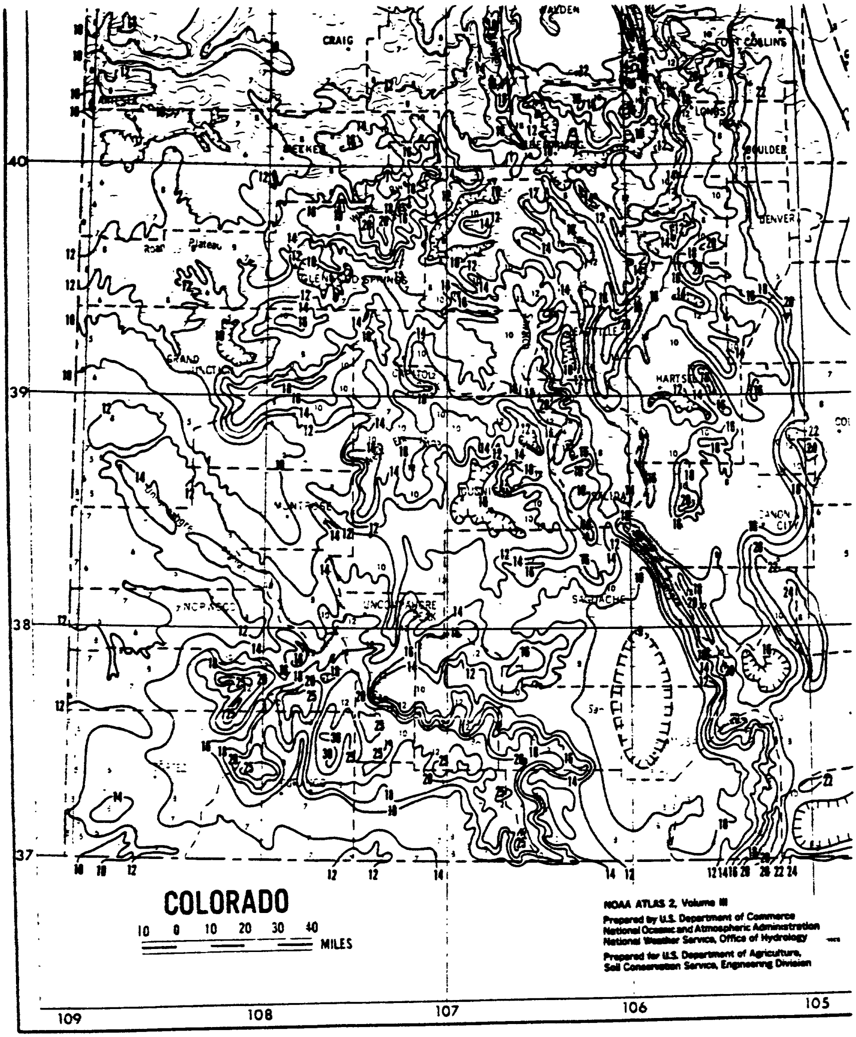


Figure 21.—2-year 24-hour precipitation-frequency maps for southern Colorado, isopluvials in tenths of inches.

interest. Most uses of rainfall frequency data are concerned with the values over some specific area sizes. Depth area curves are required to relate these point values to the values over the area sizes required. There are two different type of depth-area relations (fig. 22). The first are storm centered and relate the values at the maximum rainfall center of the storm precipitation to the average depth over some area surrounding the maximum value. A second type, commonly referred to as fixed area, relate the average point value that has occurred over some predetermined fixed area to the average depth over the area. This is the type which is appropriate for precipitation frequency values.

The fixed area relations are developed using data from networks of recording precipitation gages. The data should be available for at least 15 years of record for all gages in the network and there should be at least 1 gage within each 100 mi^2. Data from the networks with a greater density are desirable. The networks should be in relatively flat terrain so that the results can be applied over larger meterologically or climatologically homogeneous regions surrounding the networks. Ideally ratios should be determined from several different networks, plotted on a scattered diagram and curves of best fit drawn to the data (fig. 23).

The confidence limits about computed frequency curves can give some indication of the reliability of the results obtained by the analysis of the individual station data. Similar measures are not available for the evaluation of the results of generalized analyses. However, some subjective evaluations can be made by consideration of several factors. Most important is the reliability of the individual station values. These data are the basic information used in the development of the generalized charts. The number of the stations for which adequate data are available is also an important

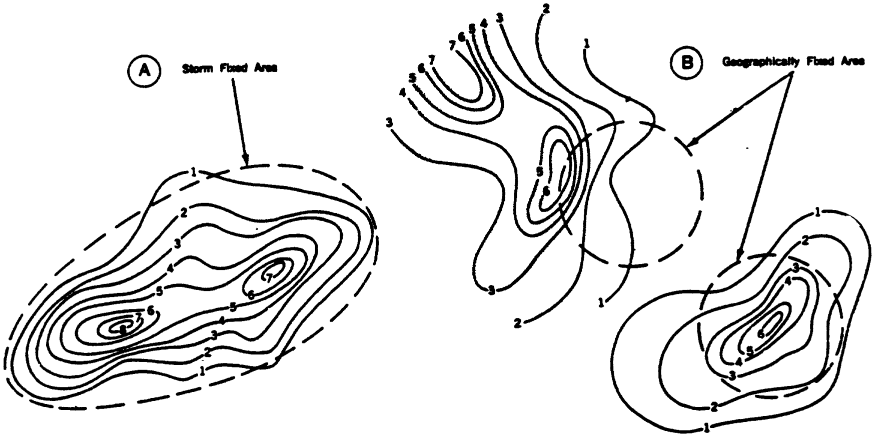


Figure 22.--Illustration of two types of depth-area relations.

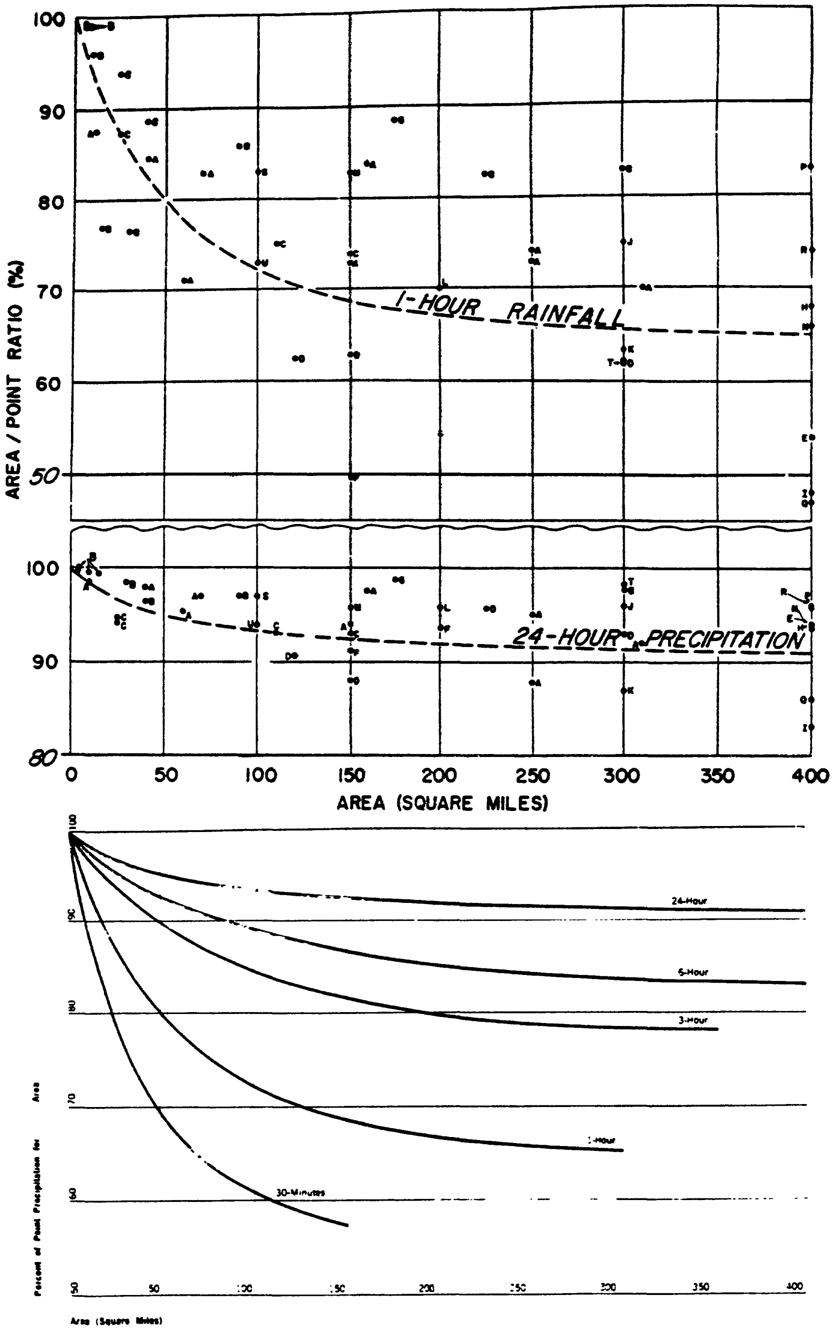


Figure 23.—Depth-area curves developed from dense network values.

factor. In orographic regions if the full range of slopes, elevations, exposures, etc., are not sampled the resulting charts may be deficient in those regions where these factors are beyond the range of values used in developing the interpolation relations. Finally, the degree of smoothness must be considered. This must be appropriate to the sampling of other errors inherent in the daily use in the development of the charts.

DETERMINISTIC PROCEDURES

For hydrologic structures where failure would cause major economic loss or large loss of life or major sociological damage a higher standard should be used. In the United States, India, Great Britain, Australia and other countries such structures are designed for the flood with virtually no risk of exceedance, this is designated the probable maximum flood. It is developed deterministically from precipitation values that are considered to be the upper limit of possible precipitation. Deterministic procedures are used since the phrase "virtually no risk of exceedance" implies a recurrence interval considerably longer than can be estimated with the current data and statistical techniques.

In regions with adequate meteorological data, the probable maximum flood is usually developed from the probable maximum precipitation (PMP) assuming that an antecedent storm occurs three to five days before the design storm. The PMP is normally developed from a study of large storms in and near the watershed. PMP is defined as: "Theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographic location at a certain time of the year."

PMP Estimates for Nonorographic Regions

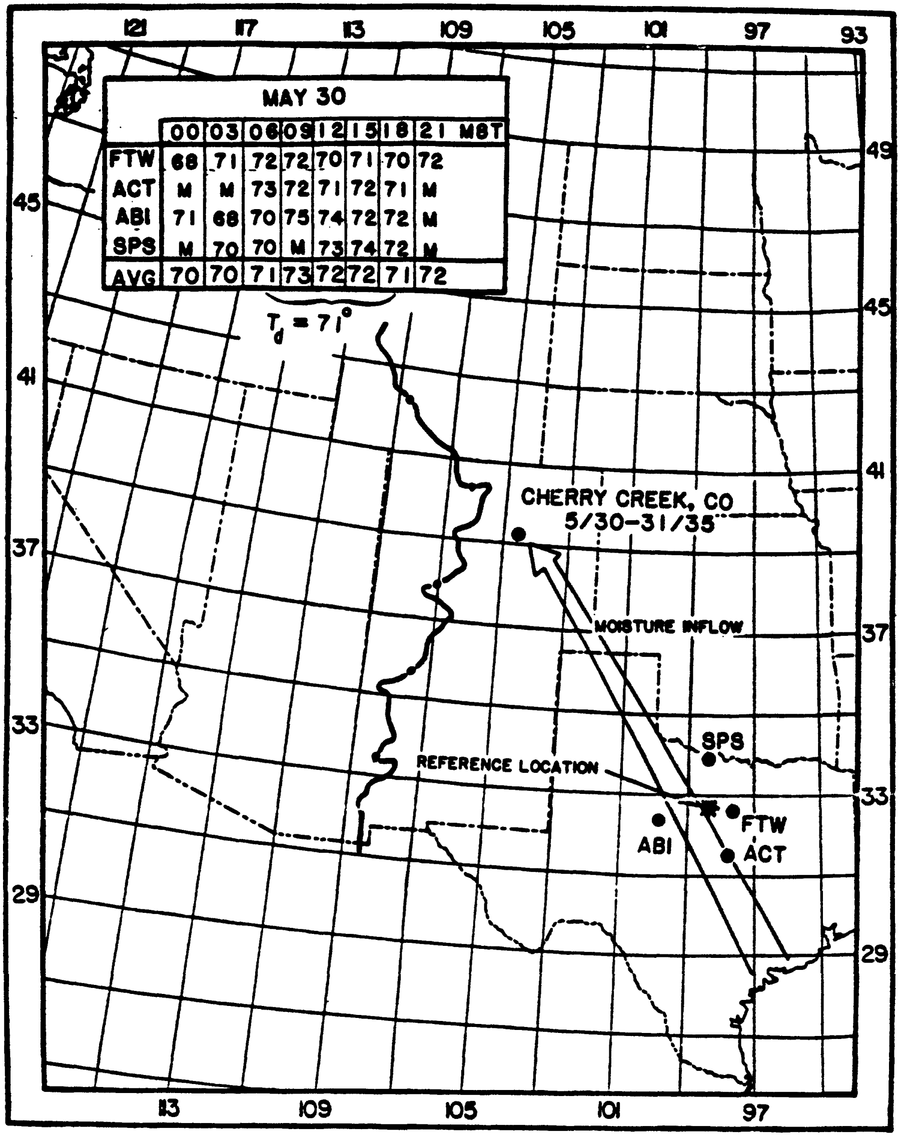
PMP estimates for nonorographic regions are primarily developed by maximization and transposition of major storms of record and enveloping the resulting values. Various approaches may be used in the maximization. The number of magnitude of maximizing step is dependent on the adequacy of the storm sample available. For those basins where a long record is available and many major storms have occurred over the basin or within the region of transposability, frequently the only meteorological factor maximized is moisture. Use of the single maximization assumes that sufficient storms have occurred so that some of them result from an optimum combination of extremes of the other necessary rain-producing factors.

Storm maximization requires at least an implicit model of the precipitation process. These models are based on the interrelation between convergence vertical motion condensation and precipitation. It is the ability to define these models that enable the hydrometeorologists to extrapolate the necessary factor to the upper limit. Applications of any model for a particular study requires that the validity of the model for a particular basin be demonstrated, that measurements have been made of an appropriate model parameters in major storms, that a sufficient length of record of model parameters is available for use in determining appropriate upper limits and that the hydrometeorologists is capable of finding the optimum combination of parameters for producing precipitation. The simpliest model that has been used for estimating PMP separates only moisture from the many other factors important in the precipitation process. This approach requires a minimum number of observations that makes available storms for a much longer period of record since detail upper atmosphere data are not required. Since all parameters except moisture are lumped into a single factor it places less demand on the knowledge of storm structure. When using this procedure, the assumptions made are: 1) the precipitation increases linearly as a moisture supply increases, 2) the storm structure does not change as the moisture supply increases, 3) within the record of major storms some storms have occurred with an optimum combination of other factors. This last assumption is why it is necessary to have a large number of available storms.

For the use of this type of model, measures of atmospheric moisture must be developed that can be consistently applied to storms throughout the period of available record. Observations of moisture through the various levels of the atmosphere are most desirable but they require data from the atmosphere above the surface. This type of information has been available for only approximately the last three decades. Many of the larger storms of record have occurred before that time. Also the observational network of stations with upper air data is not adequate to measure the moisture inflow in many intense small area storms. Thus, we normally we restrict our measurements of moisture to those available at the surface. Fortunately, moisture in the lowest layers is most important in producing precipitation, both because moisture is found in these layers and because in major storm situations moisture is rapidly distributed through the atmosphere. The most commonly used measurement is the surface dewpoint. It is desirable to use a dewpoint measurement that is representative of a persistent inflow over a timeframe that is required to produce large precipitation amounts. A commonly used measure is the highest persisting 12-hour dew point, ie. the dew point value that has been equal or exceeded for a period of 12 or more hours.

The moisture maximization factor is commonly based upon the relationship between a 12-hour persisting dew point observed within a storm situation and the maximum that has been observed in that region for the entire period of record. For each major storm of record, a representative persisting 12-hour dew point is selected in the moisture airflow into the storm. Weather maps are examined and the dew points between the rain area and the moisture source are selected (fig. 24). Observations for several stations are averaged to insure a representative sample of the moisture inflow into the storm. Unless the storm and the stations are at the same elevation, the dew point values should be reduced to a common base, e.g., 1000 mb before averaging. Stations within the rain area are not chosen since these will frequently yield moisture values which are higher than representative of the moisture inflow because of the occurrence of precipitation at the station.

The moisture maximization must be based on an estimate of the moisture associated with the surface dewpoints. The storm model generally used assumes a saturated atmosphere with the psuedo-adiabatic lapse rate. This has been supported by observations in some storms which have produced large precipitation amounts. This information permits the computation of the amount of precipitable water in the atmosphere from surface data. Precipitable water is defined as the amount of precipitation that will cover a unit area if all moisture in the column from the top of the atmosphere to the surface were condensed and released as precipitation. The moisture maximization factor for a storm is defined as the ratio of precipitable water computed from the maximum persisting 12 hour dew point (fig. 25) to that computed from that of the representative storm persisting 12 hr dewpoint. Generally the maximum dewpoint is selected within the period of 15 days between the storm and the warmer season. This is done because it can be assumed that the same storm could occur any time within this two week span. In those cases where there is a relatively unbroken barrier of extended length with a higher elevation than the storm between the rain area and the moisture source, the thickness of the



MAY 30

	00	03	06	09	12	15	18	21 MST
FTW	68	71	72	72	70	71	70	72
ACT	M	M	73	72	71	72	71	M
ABI	71	68	70	75	74	72	72	M
SPS	M	70	70	M	73	74	72	M
AVG	70	70	71	73	72	72	71	72

Figure 24.--Example of selection of representative persisting 12-hour storm dew point for storm of May 30-31, 1935.

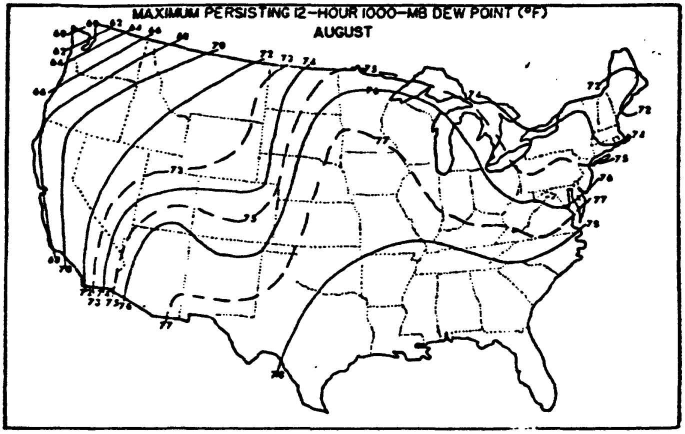


Figure 25.—Isopleths of maximum persisting 12-hour 1000 mb dew point temperatures for the month of August.

column is determined between the height of the barrier and the upper level of the atmosphere rather than between the storm elevation and the top of the atmosphere.

The amount of precipitation is dependent on the total available moisture supply. This is a function of the amount of moisture in the atmosphere and the rate of which the moisture can be supplied to the storm. In areas of limited storm sample, a wind maximization in addition to the moisture maximization is sometimes applied. The wind maximization procedures serves the purpose of the determining the amount of moisture advected into the area by the wind thus increasing the storm efficiency by increasing the moisture supply to the storm. The techniques are generally the same as those used in moisture maximization, wind speeds and directions are determined in the moist air inflow into the storm. Since the technique is used infrequently and since the climatological records of wind vary more widely than the measures of moisture from one part of the world to another, standard procedures for wind maximization have not evolved. Among the wind measurements which have been used are: 1) average wind speed for the layer used for moisture adjustments computed from a representative sounding, 2) average speeds through the moist layer computed from 2 or 3 consecutive soundings over a 12 or 24 hour period, 3) the average surface wind speed or total wind movement every 12 or 24 hour period at a representative station or 4) average wind speed from a group of stations on a series of weather charts over a period of 12 to 24 hours. Whatever measures are used, only those speeds from the wind direction representative of the moist air inflow should be considered. Wind observations used should be free of marked orographic or local influences and storm winds should be selected during the period of heaviest precipitation. The climatological record of course must be searched to find the maximum values of wind speed for those directions used for the representative storm winds.

The storm record over a particular basin is a rather limited sample for estimating upper limits of precipitation. In regions of very limited or no orographic controls, the exact location of a storm is dependent upon atmospheric processes and any individual storm could have occurred at any place within a broad region. In storm transposition, the precipitation from a particular storm is treated as if it had occurred over the basin of interest with an adjustment only for variation in available moisture. The use of storm transposition in PMP estimates requires a detailed study of the three dimensional structure of the major storms that have occurred over the study basin. Since many major storms occurred before a significant amount of information was available above the surface, upper air information may have to be inferred from knowledge of the structure of the more recent storms in the region.

After completing the study of storms producing large precipitation amounts over the basin, storms within a broad geographic region surrounding the basin should be examined. Storms with similar synoptic and dynamic features could have occurred over the basin. It is usually sufficient initially to examine the regular synoptic weather maps routinely produced by the various weather services. This usually is sufficient to identify the storm history appropriate for a particular basin. The next step is to determine transpostion limits, ie. to determine the region over which transposition of a storm is permissible. They should be determined as objectively as possible. If the record of major storms occurring over a comprehensive network of precipitation stations is long, somewhat restrictive limits might be established. If the record is short, generally more liberal limits are established. Figure 26 shows the transposition limits for a storm that occurred in the central United States.

Storm transposition techniques are generally used only in nonorographic regions. In these regions of only broad, gently sloping plains, the only adjustment made to the precipitation amounts for the storms is the difference in the available moisture between the storm location and the basin of interest. The final step in determining the estimated PMP for a particular basin requires the envelopement of the data for all durations. This is a process for selecting the largest value for each duration for various areas. In practice, data are plotted on a suitable graph paper and a smooth curve drawn for the highest points. Two separate plots are usually prepared. One plot, shows transposed maximized values of rainfall of all durations as a function of a duration for a particular area size equal to that of the basin (fig. 27). The second plots all of the data for a particular duration for all area sizes from 10 mi^2, considered representative of point values, up to the size of the basin. These plots are then combined to provide the final estimate for the depth-area-duration curve of the PMP for the basin. The value of the PMP for the total basin or any portion of the basin can be determined from these curves. The envelopment of these smooth curves, data for some durations may fall below the curve. Under our basic assumption for this procedure, the storms in this region for these areas and these durations have not occurred at a maximum efficiency.

PMP Estimates for Orographic Regions

Topographic effects on precipitation have been studied for many years. Many differences between orographic and nonorographic regions can be sited. First there is the increase in precipitation and runoff rates on the windward

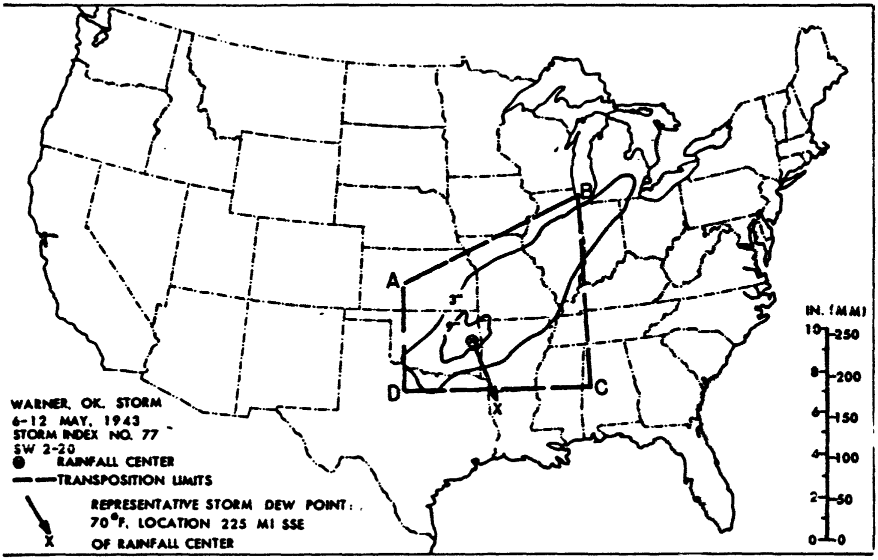
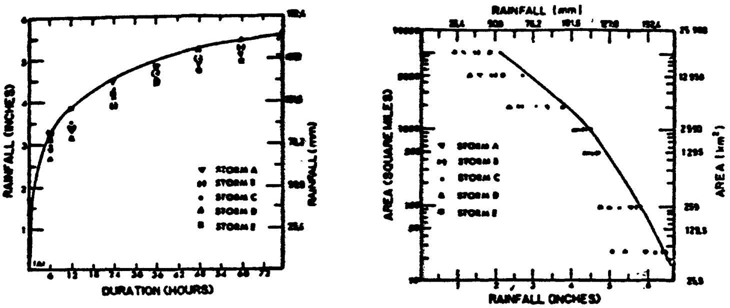


Figure 26.—Transposition limits for storm in central United States.



a. Depth-duration plot of PMP for area of 1000 mi^2

b. Depth-area plot of PMP for 24-hour storm rainfall values

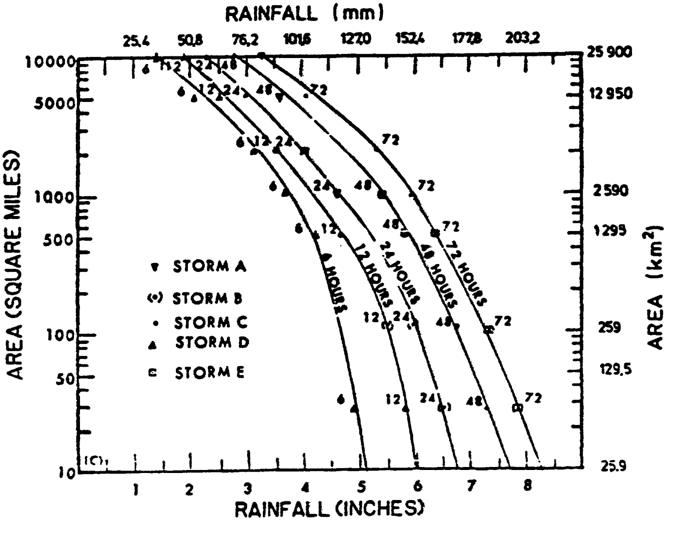


Figure 27.—Depth-area-duration analysis of PMP for a basin with ~ 10,000 mi^2 total area.

slopes of mountains. The magnitude of the effect is a function of many factors including the slope and aspect of the mountain; the strength of the inflow winds; the number of barriers between the slopes under consideration and the moisture source; the number, width, and height of the breaks in the barrier; and the height of the barrier itself. In addition, the mountain slopes "fix" the location of rainbursts and thus rainfall centers in the storms.

Another well known phenomenon is the decrease of precipitation on leeward slopes where the terrain introduces a downward component to the air movement. Immediately adjacent to the ridge or crest line, however, this is not noticed. This region is termed the "spillover zone". It is a region where the forced ascent of the air up the windward slope carries the precipitation drops pass the crest and deposits them on the lee slope. Precipitation in this region can be nearly as great as on the windward slopes near the crest. The geographic extent of the "spillover zone" varies with the type of precipitation. Another feature of orographic precipitation is the simulation effect caused by the first rise or foothills. In many storms, the initial lift imparted by low hills is sufficient to initiate considerable convective activity. This effect is frequently difficult to detect from data in the normal observational network because of the more pronounced effect of the nearby higher barriers. All of these orographic effects are superimposed on the normal meteorological processes that produce rain in nonorographic regions.

There have been several different approaches developed for computation of PMP in orographic regions. These approaches are normally used to prepare generalized charts rather than estimates of individual basins. While occassionally the same approach is used as in nonorographic regions, the transposition of storms must be undertaken with great care. It is generally limited to transposition along the mountain range where the precipitation occurred. Even within this limitation extreme care must be taken to assess the topographic influence and restrict the storm transposition only to basins with the same topographic effect. The second approach attempts to separate the effects of convergence and orographic processes in producing precipitation. One method is to estimate the convergence precipitation and the orographic precipitation separately and combine the results. When combining these two values, some restrictions must be kept in mind to avoid undue maximization. This procedure is commonly termed the "orographic separation method". Another alternative is to estimate the contribution of orography to precipitation in major storms. The convergence portion of the precipitation then can be transposed more liberally than the combined effect. This latter is termed the "storm separation method." It requires a separate evaluation of the orographic effects throughout the region to incorporate these into the convergence precipitation estimates.

Where the orographic and convergence precipitation are estimated separately, the convergence PMP is developed from moisture maximization, transposition and evelopment of the maximum storms of record in the nonorographic portion of the region. The orographic portion of the precipitation is generally computed from a multi-layer, frictionless flow model which is often termed a laminar flow model, using the forced of moist air over a barrier to estimate the amount of precipitation released. A second procedure is to use indirect methods based on other rain indices to estimate the orographic component.

When moist air is forced up and over a long, relatively unbroken barrier, precipitation is released. The air at some great height called the nodal surface is assumed to be essentially undisturbed horizontal flow. Thus, the air in the outflow column has a higher velocity than the inflow column since it must pass through a shallower layer. This model is developed from the storage equation that considers the flow of air at a vertical plane at right angles to a mountain chain or a ridge of long relatively unbroken extent. It is a 2 dimensional model with an x coordinate in the direction of flow and an a coordinate in the vertical (fig. 28). The flow may represent the average over a few miles over tenths of miles in the y, or transverse direction depending on the variations in orographic features. The transverse direction does not appear explicitly in the model. The air at ground level moves along the surface with subsequent streamlines in the vertical having increasingly less slope until they are horizontal at the nodal surface, generally between 400 and 100 mb for a relatively high barrier. In the model, the precipitation is computed for several layers. If the flow across the mountain is assumed to be frictionless and if the variation with height of the horizontal wind, the lift imparted to the air, the levels at which rain or snow are formed, and their drift with the air before striking the ground are known, the resulting orographic precipitation profile can be calculated. These calculations involve construction of snow and rain drops trajectories from the level of their formation to the ground. The final results of these calculations is the precipitation profiles along the slope.

There are many important simplifying assumptions in the development of the model including such things as generalization of mountain slopes, the assumption of frictionless flow, and an essentially 2 dimensional flow over continuous barrier around which the air cannot flow. For these reasons, comparison of the model output with observed precipitation or calibration is desirable.

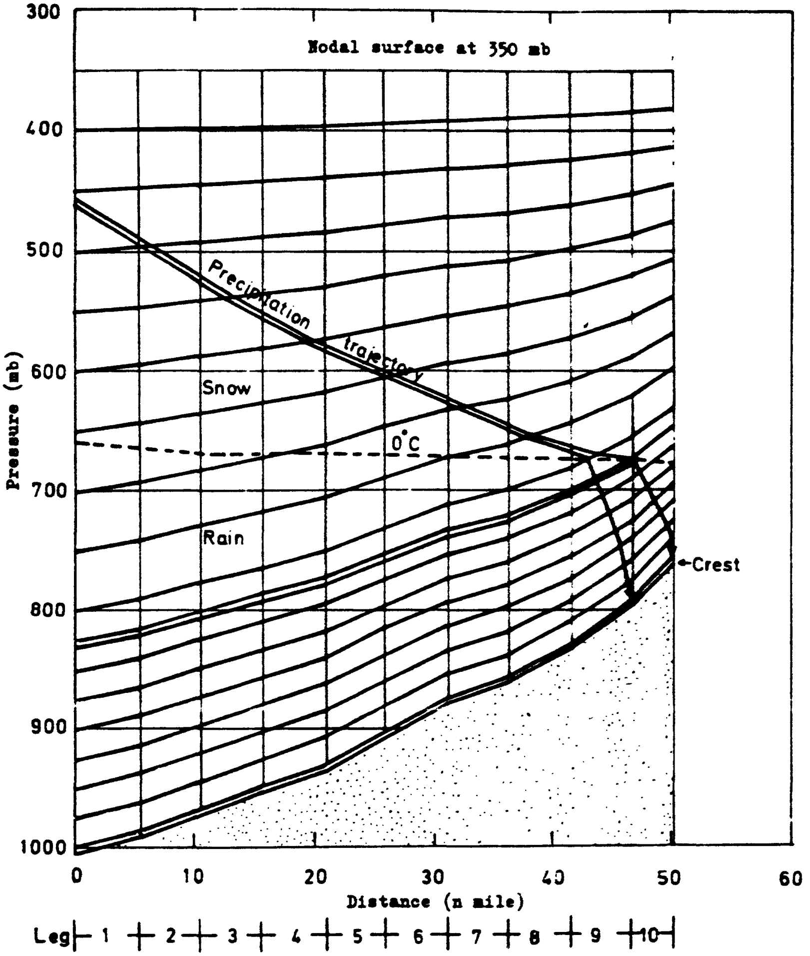


Figure 28.—Schematic of multiple-layer frictionless flow model for orographic regions.

To estimate orographic PMP from the model, the maximum values of moisture for the model are determined either from surface dewpoints as they are for nonorographic regions, or if adequate lengths of record are available from surveys of radiosondes data in the region just upwind of the orographic barrier. Maximum inflow wind should be determined from the record of upper wind observations just upwind of the slope. When such records are not available, winds for use in the model may be estimated from surface winds, surface pressure gradients, upper-level height gradients, or a combination of such sources.

An alternate procedure is to determine the convergence or free atmospheric forced precipitation within all of the major storms in a region, this is the part of the total storm precipitation which remains after the effect of terrain at the storm sites have been estimated and removed from the total storm depth. In nonorographic regions, FAF precipitation would be all the precipitation resulting from the storm. In orographic areas, the method assumes that the basic storm mechanism is unaffected when the terrain feedback is removed. Such an assumption is necessary even though not enough is presently understood on how this feedback affects the way a storm develops or evolves. In this procedure we evaluate observed precipitation data, isohyetal patterns, and all meteorological data and analyses to develop an estimate of the percentage of the precipitation that results from orographic forcing and that which results from free air forcing. Figure 29 shows a schematic of the procedure. The first step is the selection of the area size and duration of interest. The percentage of the storm precipitation which results from topography generally tends to increase with the longer duration and larger area sizes. The initial determination of the percentage is based upon an examination of the observed precipitation amounts. A comparison is made

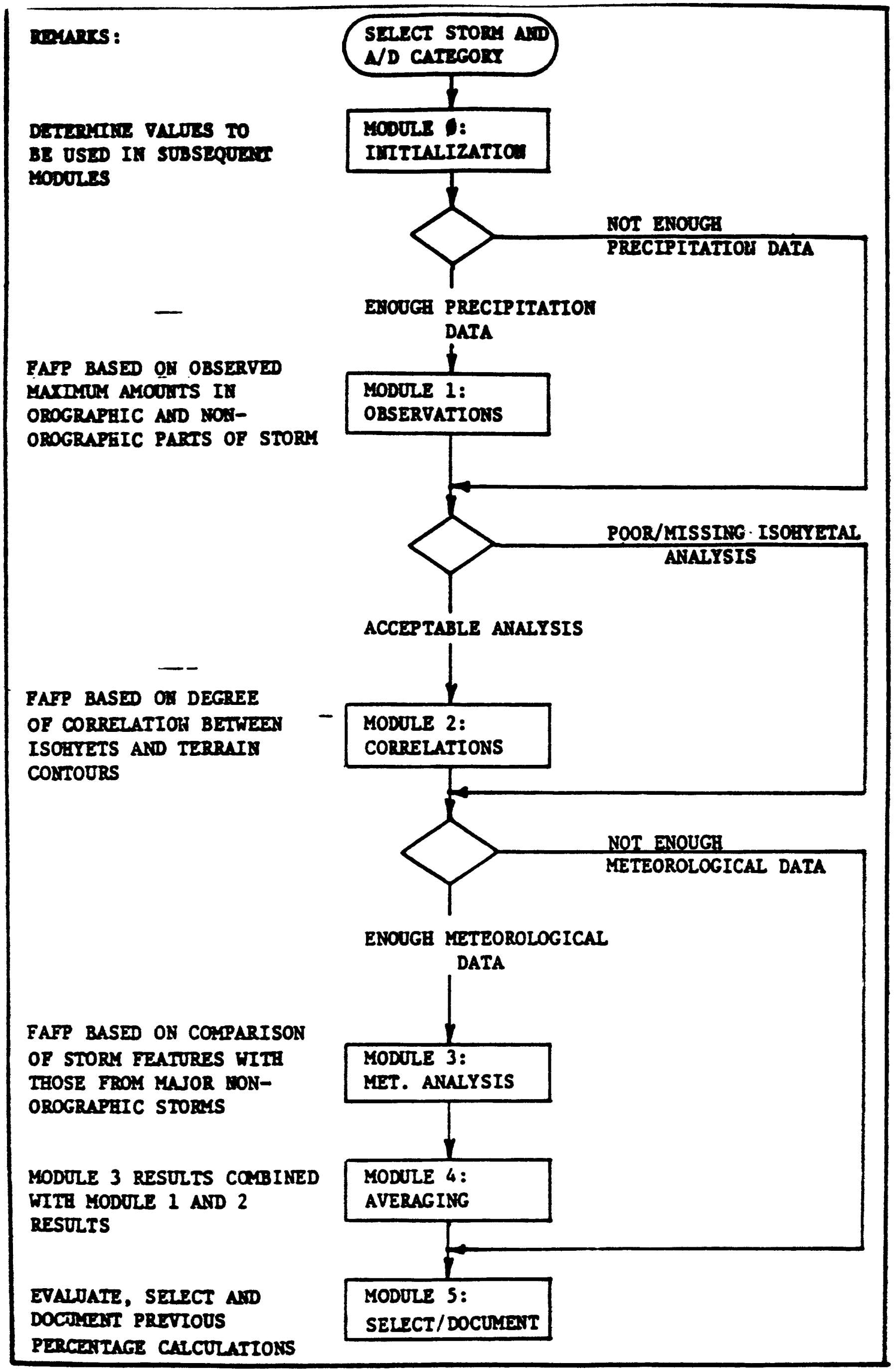


Figure 29.—Schematic for storm separation method.

between the maximum observed amount in the nonorographic region of the storm and the maximum observed amount in the storm, generally in the orographic region. In general as the a difference between these two amounts decreases, the greater the convergence or free air forcing precipitation is in a particular storm. A second evaluation is of the isohyetal pattern. The isohyetal patterns is compared with the terrain contours within the region. In this comparison, the higher the degree of correlation between topographic features and the isohyetal pattern in the region of the isohyet which encompasses the rainfall for the area of interest, the greater the amount of orographic precipitation within the storm. This step in the method might produce suspect results when the isohyetal patterns were drawn based on limited data using mean annual precipitation or terrain features to aid in the analysis. This is so since the isohyetal analysis prepared in this fashion has assumed strong orographic effect.

A third evaluation involves a detailed consideration of all the meteorological factors in producing convergence precipitation. Those present in the storm are compared with those present in the major storms of record in nearby nonorographic regions. Considerable effort should be expended in a detailed reanalysis of surface and upper air charts where these factors may be crucial in estimating the percentages of convergence rainfall in an individual storm.

In this evaluation, two other percentages are obtained by averaging the percentage obtained for the isohyetal analysis with the meteorological analysis, and the analysis of the precipitation observations with the meteorological analysis. When sufficient observational data, isohyetal and meteorological analyses are available, 5 percentages are produced by this procedure. The final percentage used to determine the depth of FAF precipitation is obtained by first quantifying how confident you are that the data assumptions that underline each step are completely appropriate for the candidate storm and for the particular area/duration category selected; and the second, quantifying how well the data available for each step serves the purpose of that step. After determination of the FAF precipitation for all the major storms in the region, the resultant values are moisture maximized and transposed to all of the locations within the region where similar storms have occurred. These limits are determined in a manner similar to those for nonorographic regions. Adjustments to the moisture supply for the storm are made because of differences in elevation and the geographic differences in available moisture at different locations.

It is necessary to adjust this FAF precipitation, convergence precipitation, for the variation in orographic effects that occur over the region. This requires the development of an orographic factor. Frequently a 100-year precipitation-frequency map can be used as a basis for determining an orographic factor. The first step is to determine the nonorographic portion or free air forcing component of the 100 yr amount. This can be done by finding regions on the precipitation-frequency maps of minimal values in plains regions and in broad valleys that were considered to contain the least influence of orographic effects. Smooth isolines of the convergence component then can be drawn through these regions. These isolines should reflect only the slow decrease of convergence potential away from the moisture source. The orographic factor can then be determined by dividing the total 100-year precipitation amount by that amount which is determined to be convergence. This orographic factor can be used to adjust the convergence PMP for variations in orographic effects. In practice, the most intense portion of the storm is usually adjusted by some fraction of the orographic factor while the precipitation during the remainder of the storm is increased by the entire orographic factor.

In some regions it is not possible to use the procedures discussed in the previous paragraphs. In these regions various indirect approaches have been applied to develop and modify nonorographic PMP estimates. In such studies, nonorographic PMP estimates are developed for coastal regions, valleys or other flat areas and extended into the orographic regions. These nonorographic PMP values are based upon moisture maximization and transposition of storms that have occurred within the region. The next step is to prepare generalized methods of evaluating topographic effects throughout the region. One approach is to determine relations between intensification of rainfall by slopes and decreases in rainfall potential by increasing in elevation. In other studies, classification of the broad scale and local topographic effects and increases or decreases in precipitation based upon these classifications adjust the nonorographic PMP estimates. In this approach also orographic effects can be developed in the same manner as for the storm separation method. If maps of precipitation frequency values are not available, maps of normal annual or median annual precipitation frequently are available and may provide useful indices to topographic effects. However, these latter maps must be used with care since they include not only the various rainfall with topography but they also increase frequency of rain on topographic slopes.

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DROUGHTS

By

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INTRODUCTION

Droughts are one of the most costly hydrologic hazards in the United States. This is particularly true in the semiarid Great Plains of the central United States which is one of the major "bread baskets" of the world. Compared to floods and hail storms which are the other common hydrologic hazards in this region, droughts occur throughout a larger area and persist for a longer duration, thus, effecting a greater number of people.

In very general terms, "...drought is a condition of moisture deficit sufficient to have an adverse effect on vegetation, animals, and man over a sizable area" (Warick, 1975). Droughts are a chronic and recurring problem in many parts of the world. They do not occur suddenly but develop gradually during several months or years and have, in fact, been referred to as a "creeping phenomena." All areas of the United States are vulnerable to drought and their durations commonly range from several years in the western states to only a few months in the eastern part of the Nation (fig. 1). They occur to some degree almost every year in parts of the central United States. Prolonged droughts lasting 2 years or longer cause the greatest damage and financial loss.

The regional differences in drought characteristics across the United States are due to various physical processes including orographic lifting of air over mountain ranges, lifting of air by storm fronts, localized temperature differences and temperature convergence, and dynamic lifting of air. Of these processes, the dynamic lifting of air, which is associated with surface cyclones and a broad belt of upper level westerly winds, is considered

Figure 1.--Areas vulnerable to drought (modified from U. S. Geological Survey, 1970).

the most significant process effecting meterological events in the United States, (Perry, 1980).

Even though a drought may be widespread across the nation, many people may not be aware of its severity or extent. Some are not affected because they either live in an arid region where droughts are the norm rather than the exception or they live in areas where local precipitation or stored water supplies remained adequate to meet their needs.

DEFINITIONS OF A DROUGHT

Drought does not have the same meaning or significance to all people. No generally accepted definition is adequate nor is one practical because a drought is the result of many different factors. As a result, it has been measured by many different criteria including the following:

- Precipitation statistics
- Soil-moisture indices
- Ground-water-level and low-flow characteristics
- Overall agricultural production
- Crop yields
- Economic return from crops produced and sold.

The World Meteorlogical Organization (1975) recognizes the following six types of drought:

- Meteorological drought--defined in terms of precipitation deficiencies in absolute quantities for specific durations.

- Climatological drought--defined in terms of precipitation deficiencies, not in specific quantities but as a ratio to mean or normal value.

- Atmospheric drought--definition involves not only precipitation, but possibly temperature, humidity, or wind speed.

- Agricultural drought--definition involves principally the soil moisture and plant behavior, perhaps for a specific crop.

- Hydrologic drought--defined in terms of decrease in streamflows, decrease in lake or reservoir storage and lowering of ground water levels.

- Water-management drought--characterizes water deficiencies resulting from failure of water-management practices or facilities.

Three common definitions are:

- Meteorological drought: "...a period of abnormally dry weather sufficiently prolonged for the lack of water to cause serious hydrologic imbalance in the affected area." (Huschke, 1959).

- Agricultural drought: "...a climatic excursion involving a shortage of precipitation sufficient to adversely affect crop production or range production." (Rosenberg, 1979).

- Hydrologic drought: "...a period of below average water content in streams, reservoirs, groundwater aquifers, lakes and soils." (Yevjevich & others, 1977).

HISTORIC DROUGHTS

Records of historic droughts dating back more than 1,800 years exist in Europe and fairly good drought records dating back more than 1,300 years exist in China. These data do not indicate, however, any apparent regularity in drought occurrence or any clearly defined cycles (Tannehill, 1947). Maximum annual gage heights have been recorded on the Nile River in Egypt since the year 641 A.D. An analysis of this data by Hurst (1950) indicates that the distribution of those annual maximum readings, which fall below the long-term mean annual gage heights, are similar to a random distribution. These data do not, however, confirm that minimum annual discharges, and thus droughts also have the same distribution.

In the United States only limited climatic and hydrologic information is available documenting historic droughts prior to the 1800's. Estimates of the occurrence of droughts in the United States during the past 800 years have been obtained, however, from tree-ring records.

An analysis of 748 years of tree-ring data by Weakly (1962) for central Nebraska showed that 36 percent of the years were sufficiently dry to have adversely affected vegetative growth. The prime reason that Indians in New Mexico and Arizona abandoned their cliff dwellings in about 1276 to 1300 is attributed to an extended drought in that area of the United States during this period (Matthai, 1979).

Troxell's (1957) analysis of tree rings for the past 560 years showed dry periods in southern California ranging from 6 to 40 years--with extended dry periods lasting an average of about 14.5 years.

Weakley (1943) examined 400-year tree-ring records for western Nebraska and identified 13 droughts of 5 or more years duration. The average duration was 12.8 years and the average recurrence interval was 20.6 years--with the longest being 26 years (1539-64).

Mitchell et al (1978) in a tree ring study using data back to about 1600 indicated a statistical association between drought-area changes and the Hale sunspot cycle theory. Their study indicated "rainfall" cycles during the 11-, 22-, 33-, and 80- to 90-year sunspot cycles.

An analysis of tree-ring data in the northern Great Plains dating back to 1640 indicates droughts more severe than the 1930's drought occurred in the 1750's, 1820's and 1860's (Meko, 1982), with the most significant drought occurring in the 1750's.

Historical climatic and hydrologic data collected since the early 1800's show that significant droughts occurred in the United States during the late 1800's, 1910's, 1930's, 1950's, and 1970's and 1980's. A comparison of annual precipitation data in the central United States with solar activity from 1880 to 1977 indicates that major droughts occur at approximately 22-year cycles with less severe droughts occurring midway between the major droughts (Perry, 1980). Some of the more noteworthy droughts experienced in the United States during the past 100 years are described below:

1877-94 - Climatic data collected in the Great Plains of the United States indicate that the first major drought to be recorded in this area began about 1877 and lasted for about 18 years.

1910 - The northern and central Great Plains of the United States had another significant drought in 1910 during which precipitation averaged about 30 percent less than normal for the region.

1931-39 - Oklahoma and large areas of the Midwest had a severe drought which began in 1931 and persisted for 8 years. Nearly one-half of the Great Plains had become a desert climate by 1934. Highwinds over recently plowed fields created the infamous "Dust Bowl" of Oklahoma causing many farmers to move west to California. This drought gave impetus to the National Soil Conservation Service program designed to improve farming practices. It is interesting to note that Soviet Russia had an extraordinarily severe drought during a similar period (1933-40).

1952-56 - The midcontinent again had a prolonged drought during the mid-1950's as a result of a succession of rare drought-producing meteorological events. Long-term precipitation records for the southern Great Plains indicate that a drought of this severity can be expected to occur in this area only once every 140 years on the average.

1961-67 - This drought occurred in the northeastern United States and was the longest and most severe in the history of the region. Many streams in the area had record low flows, salt water from the Atlantic advanced upstream into predominantly freshwater estuaries, and numerous observation wells declined to record low levels. The recurrence interval of this drought is estimated to be about 160 years.

1976-77 - The most severe drought to affect large areas of the entire United States in the 20th Century occurred during these 2-years. The discharge and water levels in many streams and wells throughout the country had record minimums. Index streamflow stations nationwide indicated that monthly streamflow was deficient in some States for 12 or more months during these 2 years. Matthai (1979) in a comparison of precipitation for selected dry years from 1897 to 1977 at San Francisco, California showed that the average precipitation deficiency of this drought was approximately twice that of droughts during the previous 80 years.

1980-81 - The duration and severity of this drought is typical of severe droughts affecting the United States. The combined flow for the Nation's "Big Five" rivers--Mississippi, Columbia, St. Lawrence, Ohio, and Missouri--averaged less than normal for 6 consecutive months during this drought. Record low ground-water levels were measured in at least 20 States, water supplies in several reservoirs in the eastern States became seriously deficient, the salt fronts in estuaries along the Atlantic coast encroached upstream into freshwater areas, and agricultural production was significantly decreased in some areas of the Nation.

EVALUATING LONG-TERM TRENDS IN DROUGHTS

An understanding of the frequency of occurrence of droughts and their severity can be obtained by evaluating at the long-term trends in historic climatic and hydrologic data. Two techniques for evaluating long-term trends are described here: (1) moving averages and (2) cumulative departure from mean.

Moving Averages

A common approach to evaluating long term trends is to compute moving averages of annual precipitation or streamflow data. Moving averages are generally commuted using averages of 3 or more consecutive years. These average values are then plotted (generally at the mid-point of the member of consecutive years) with time for the period of record. The moving averages tend to smooth out the effects of intermittent large and small values and enable the analyst to detect declining or rising trends that may otherwise not be readily apparent. An example of the computation for determining 3-year moving averages using 15 years of average annual Statewide precipiation data for Oklahoma is shown below:

Year	Annual Precipitation (inches)	3-year sum (inches)	3-year average (inches)	Departure from average (inches)	Cumulative departure (inches)
1929	35.4	-----	----	+2.9	2.9
1930	30.7	97.7	32.6	-1.8	1.1
1931	31.6	96.3	32.1	- .9	.2
1932	34.0	96.2	32.1	+1.5	1.7
1933	30.6	92.1	30.7	-1.9	- .2
1934	27.5	94.8	31.6	-5.0	- 5.2
1935	36.7	86.9	29.0	+4.2	- 1.0
1936	22.7	87.6	29.2	-9.8	-10.8
1937	28.2	84.1	28.0	-7.3	-18.1
1938	33.2	88.1	29.4	+ .7	-17.4
1939	26.7	31.2	31.2	-5.8	-23.2
1940	33.8	107.5	35.8	+1.3	-21.9
1941	47.0	120.8	40.3	+14.5	- 7.4
1942	40.0	115.9	38.6	+7.5	.1
1943	28.9	-----	----	-3.6	- 3.5
Average	32.5				

Computations of moving averages also can be made using weighted values which give greater emphasis to the center values in a series of consecutive values. Thus, in the above example, the weighted average for 3 consecutive values is computed as $(P_1+2P_2+P_3)/4$ where P_1, P_2 and P_3 are the annual precipitation values for each of 3 consecutive years. A plot of these annual precipitation values and their 3-year moving averages is shown in Figure 2A.

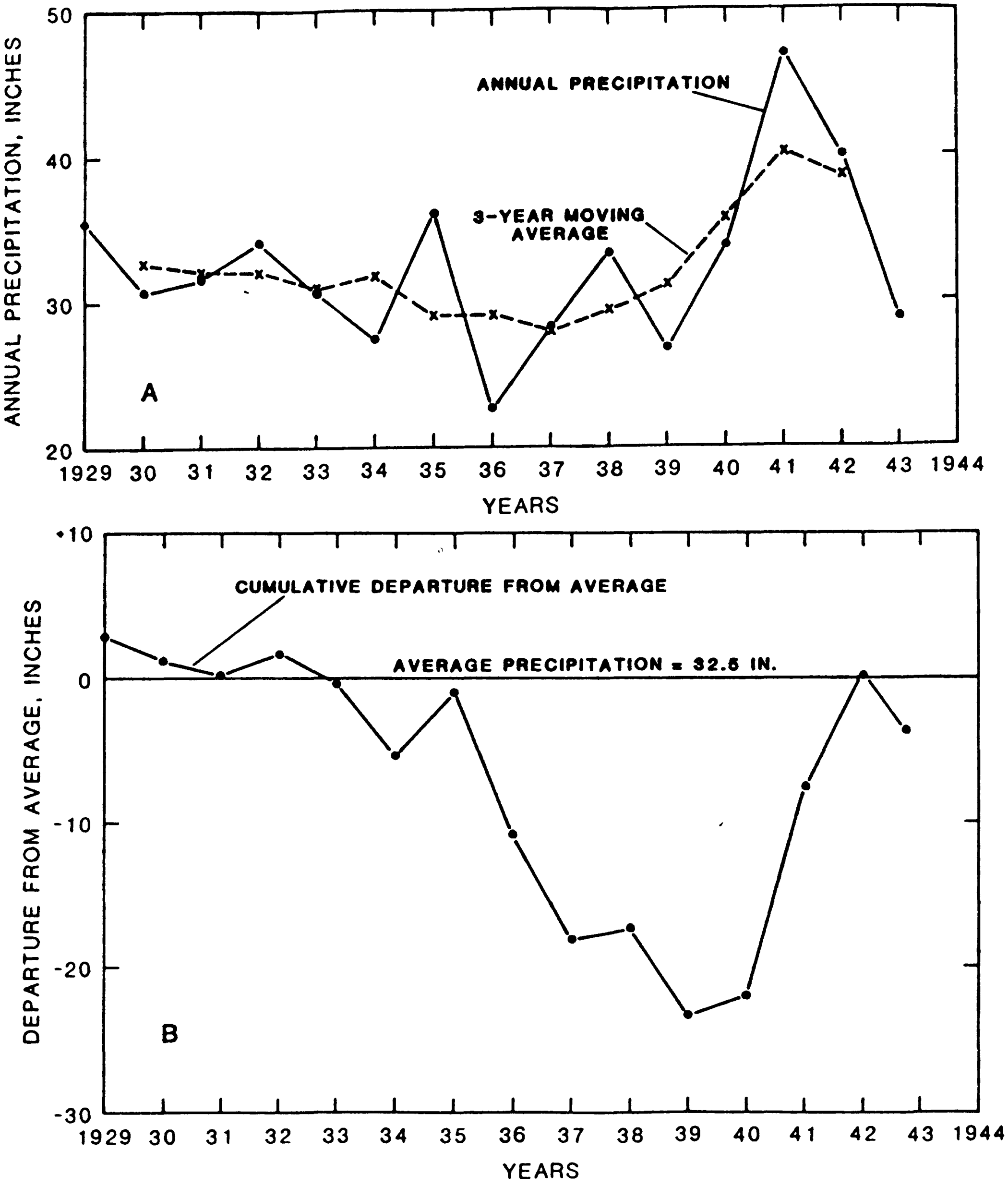


Figure 2.--(A) Annual precipitation and 3-year moving average, and (B) Cumulative departure from average annual precipitation in Oklahoma, 1929-1943.

Cumulative Departure from Mean

Another method for evaluating long-term trends during droughts is to compute the cumulative departure of a climatic or hydrologic parameter from its longterm mean. In this method the departures of the annual values from the long-term mean are algebraically accumulated through the period of record and plotted with time. The table in the previous example includes these departures and their cumulative values. A plot of the cumulative departures is shown in Figure 2B.

Zaporozec (1980) used 100 years of annual precipitation data at a weather station in Wisconsin to investigate periods of prolonged drought. A plot of the annual precipitation in Figure 3 shows considerable variation from year to year with obvious dry years scattered throughout the period of record. The 3-year moving averages of annual precipitation included in the graph emphasizes some short-term droughts lasting a few years. The cumulative-departure plot in Figure 4 indicates a general long-term downward trend in annual precipitation from 1890 to 1963, which is not apparent in Figure 3.

Care needs to be taken in any trend analysis to use data that have not been altered by man's activities during the period of record. Movement of a weather station to a slightly different location, establishment of retention reservoirs upstream from a gaging station or pumpage from a ground-water well during the period of record can cause erroneous results in any trend analysis of the climatic or hydrologic data.

DROUGHT INDEXES

Numerous drought indexes have been developed to provide farmers, water managers, meteorologists, and hydrologists with a measure of the severity of a drought. A description of some of the commonly used indexes are given below:

Precipitation Index: The Oklahoma State Agriculture Department uses monthly precipitation accumulated during the fall and winter rain period (September 1 to February 28) and then again during the subsequent growing season (March 1

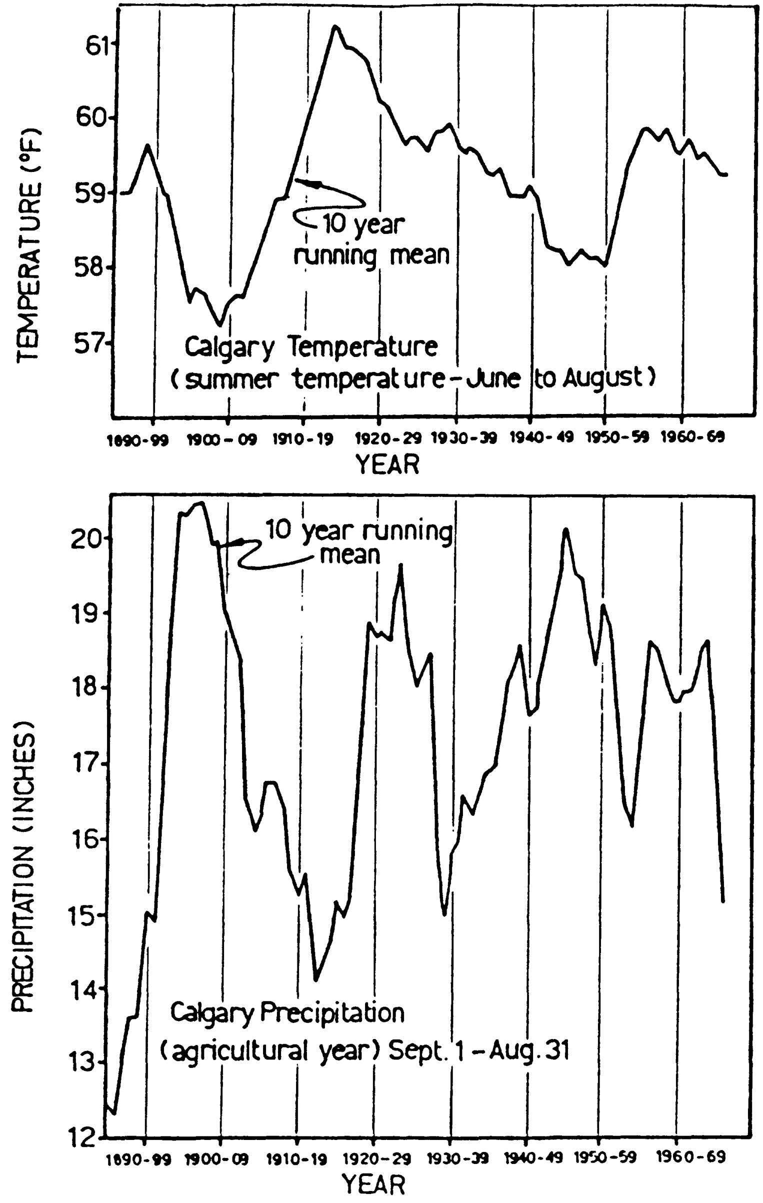


Figure 3.--Annual precipitation and 3-year moving average precipitation in Wisconsin, 1881-1978 (modified from Zaporozec, 1980).

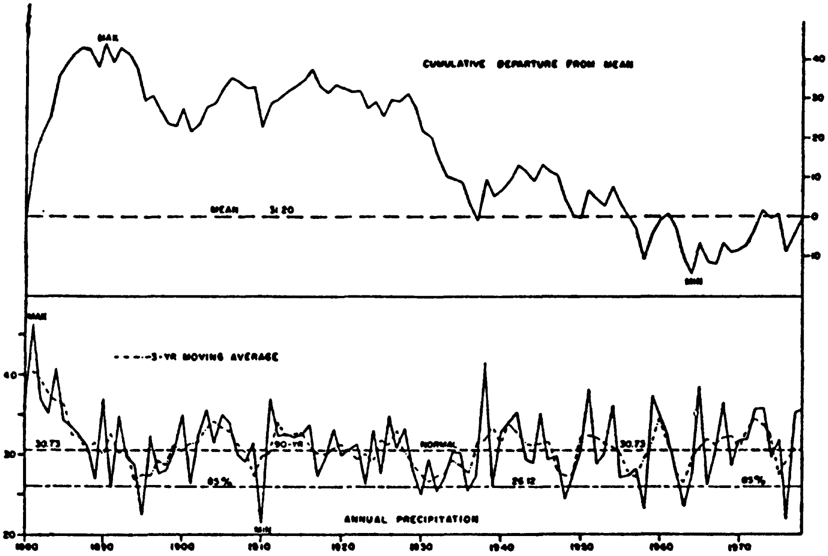


Figure 4.--Cumulative departure from average precipitation in Wisconsin, 1881-1978 (modified from Zaporozec, 1980).

to August 31) to describe the status of moisture conditions for farmers. Actual accumulated precipitation is compared with long-term average (normal) accumulated values to indicate if available moisture is above or below normal at a given time of the year. The following table modified from the August 29, 1983, Oklahoma Crop-Weather Summary shows the cumulative percent of normal precipitation for a significant dry growing season ending August 28, 1983 in each of the nine climatic divisions of Oklahoma:

	Temperature (degrees Fahrenheit) during preceding week			Precipitation inches			
DIVISION	Mean	Max.	Min.	Current Week March 1	Cumulative Since March 1	Cumulative Normal Since Normal	Cumulative percent of
PANHANDLE	85.6	105	64	0.14	11.00	13.56	81
N. CENTRAL	86.4	105	70	.21	18.17	17.53	104
NORTHEAST	87.7	106	69	.13	18.77	23.13	81
W. CENTRAL	86.9	104	73	.00	11.13	16.00	70
CENTRAL	86.2	100	71	.00	17.12	20.43	84
E. CENTRAL	87.6	105	68	.00	17.76	25.49	70
SOUTHWEST	85.9	104	70	.00	11.66	15.88	73
S. CENTRAL	85.9	104	63	.00	20.30	20.91	97
SOUTHEAST	85.3	102	67	.05	25.45	26.10	98

The 1983 summer was a notable drought for Oklahoma as indicated by the persistant below normal percentages for all but one of the divisions. Included in the table are temperature data for the preceding week.

Palmer Drought Index: The National Weather Service uses the Palmer Drought Index (Palmer 1965) to provide a quantitative assessment of periods of prolonged meteorological anomalies. The Palmer Drought Index includes present and antecedent precipitation and the duration and magnitude of the abnormal moisture deficiency. Specific factors used to develop this index include:

1. Monthly temperature
2. Monthly precipitation
3. Monthly potential evaporation - computed from available climatic data
4. Available moisture capacity of soil in the surface and root zones. Values of the derived index indicate departures from the

"normal" climate and are categorized using the following dimensionless values:

More than +2 -- moist condition
+2 to -2 -- normal conditions
-2 to -3 -- moderate drought
-3 to -4 -- severe drought
Less than -4 -- extreme drought

Values of the Palmer Drought Index in Figure 5 illustrate the range and severity in drought conditions throughout the United States for July 2, 1977.

Crop Moisture Index : The Crop-Moisture Index (CMI) was developed by the National Weather Service (1977) to provide information about soil-moisture conditions on a broad scale throughout a large region such as a major wheat-growing area. The CMI is considered to indicate moisture availability in the soil based on the prevailing conditions of precipitation, evapotranspiration, runoff, soil moisture, recharge, and the field capacity of the soil. The CMI does not, however, indicate individual field conditions or the scope, severity, and frequency of prolonged droughts. The CMI applies available information from the Palmer Drought Index and was developed with the objective of presenting a simple index which changes rather quickly as macro-scale moisture conditions change. The CMI is defined as:

$$\mathrm{CMI} = Y + G \tag{1}$$

where Y = index of the evapotranspiration deficit derived using the Palmer Drought Index, and
G = index of excessive moisture (never less than 0).

Index values greater than 0 indicate the degree of "wetness" whereas values less than 0 indicate the degree of "dryness".

The National Weather Service prepares a Crop Moisture Index Map each week during critically dry or wet periods. Shaded areas on the map for the July 1977 drought (Fig. 6) show where the index remained unchanged during the past

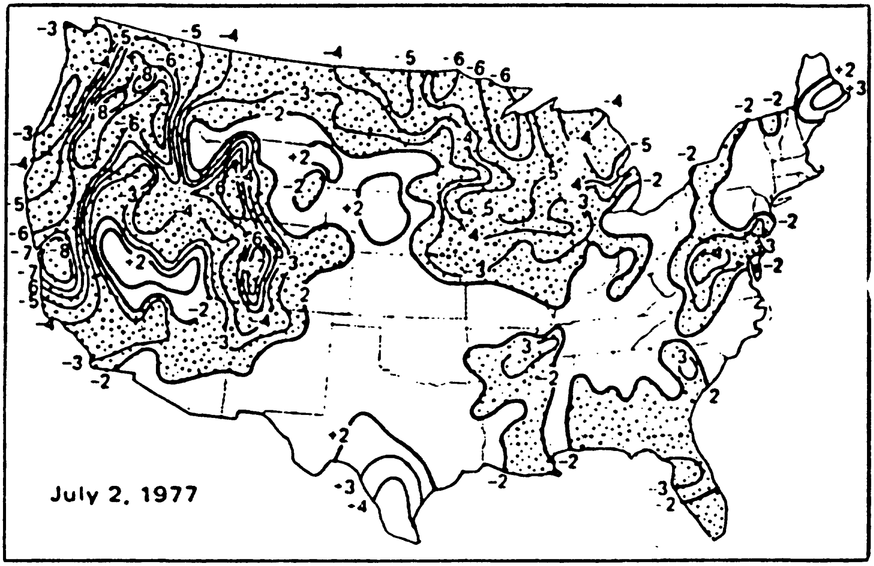


Figure 5.--Values of Palmer Drought Index for July 2, 1977. Values greater than +2 indicate moist conditions; +2 to -2, near normal conditions; -2 to -3, moderate drought; -3 to -4, severe drought, and less than -4, extreme drought (modified from National Weather Service).

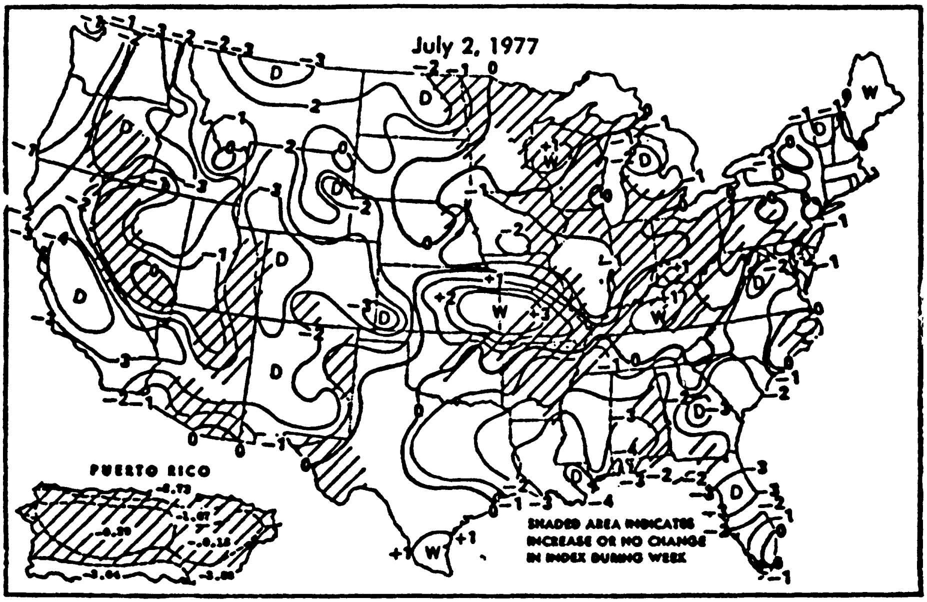


Figure 6.--Values of Crop-Moisture Index for week ending July 2, 1977. Values greater than +3 indicate excessively wet conditions; +3 to +1, generally too wet for favorable crop growth; +1 to -1, favorable growing conditions; -1 to -4, generally too dry for favorable crop growth; and less than -4, extreme dry conditions. Shaded areas indicate an increase or no change in index and unshaded areas indicate a decrease in index during the week (modified from National Weather Service).

week or increased because of precipitation. Unshaded areas indicate that the index decreased during the past week.

Streamflow Indexes: Several statistical measures of streamflow have been used to describe local, regional, and nationwide drought conditions. The more common measures are described below:

National Water Conditions

The U.S. Geological Survey publishes a monthly report of National Water Conditions that shows areas within the United States with above average, average, and below average streamflow based on records at selected long term streamflow-gaging stations located throughout the Nation. A map for March 1981 (Fig. 7), shows the large areas of deficient streamflow across the United States during the nationwide drought of 1980-81. Included in the report are hydrographs for selected streamflow-gaging stations which compare the long-term median discharge with discharge for the past 2 years (Fig. 8A). A comparison of average usable contents of selected reservoirs with contents for the previous 2 years also is given (Fig. 8B).

Daily Flow Duration Curves

A daily flow-duration curve shows the distribution of the daily discharges during the period of record at a streamflow-gaging station. A duration curve is developed by first listing the number of days that the recorded daily discharges are within specified discharge intervals. The histogram in Figure 8A illustrates this distribution of flows during the water years 1911-55 for the James River in Virginia. A daily flow-duration curve can then be drawn by expressing the height of each bar in the histogram in terms of percent of total days and plotting the cumulated percents--starting with the high discharge end of the graph--versus discharge, as shown in Figure 9B. The curve shows the percent of time a daily discharge of any given value was exceeded during the period of record. Thus, flow-duration curves can be used to describe the occurrence of an unusually low flow; they do not, however, describe the duration or severity of a possible drought.

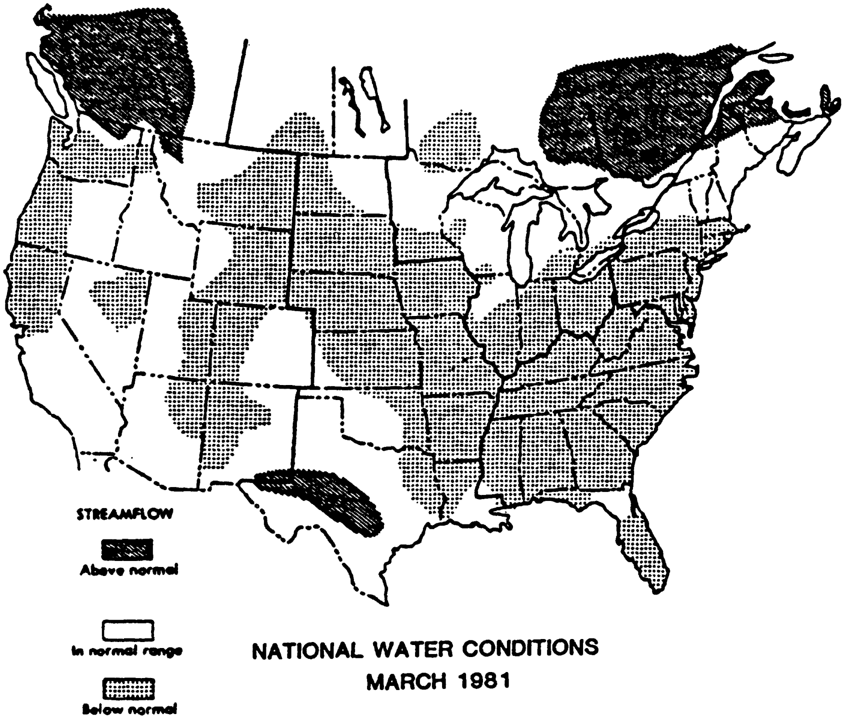
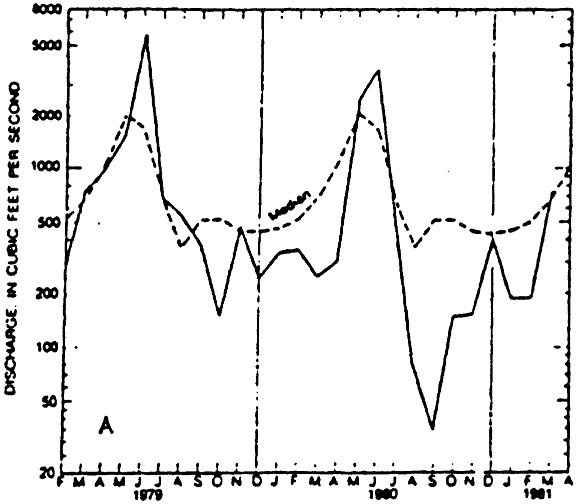
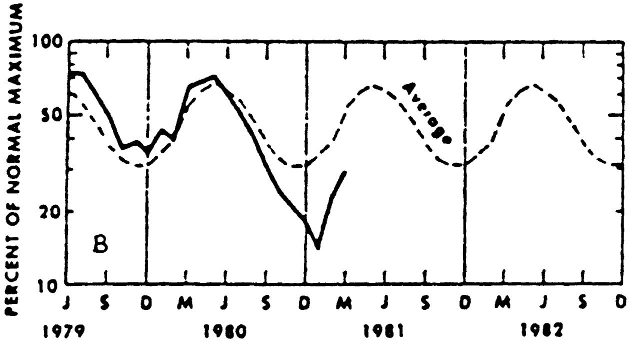


Figure 7. Streamflow conditions in the United States during March 1981 (from U.S. Geological Survey Water Resources Review, March 1981).



MONTHLY MEAN DISCHARGE, WASHITA RIVER NEAR DURWOOD, OKLA.



USABLE CONTENTS, NORRIS AND MELTON HILL LAKES, TENN.

Figure 8.--(A) Monthly mean and median discharges of Washita River near Durwood, Oklahoma, and (B) Useable and average contents of Norris and Melton Hill Lakes, Tenn., March 1981 (modified from U.S. Geological Survey, 1981).

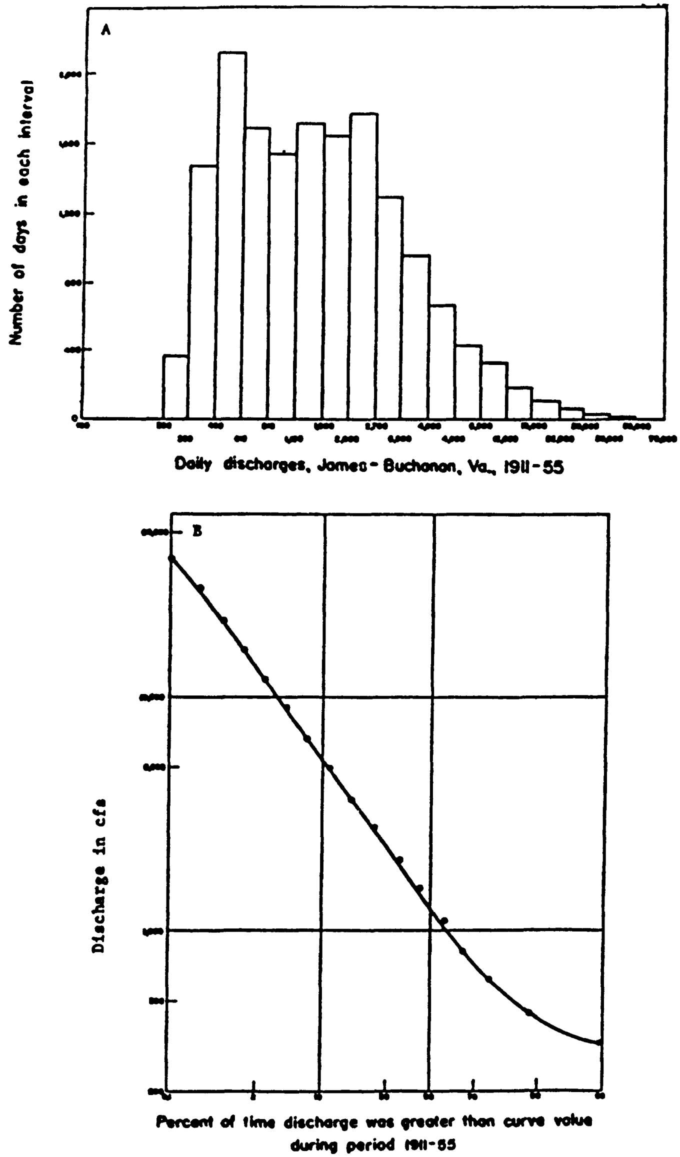


Figure 9. (A) Histogram of daily mean discharges and (B) flow duration curve for James River at Buchanan, VA, 1911-55.

Daily Flow-Duration Hydrographs

Daily flow durations also can be displayed for each day of the year as hydrographs (Fig. 10). Each hydrograph represents a given probability at which the daily discharge can be expected to be exceeded. The lower hydrograph gives an indication of the minimum flow that can be expected to occur for each day of the year.

Low-Flow Frequency Curves

Low-flow frequency curves (Riggs, 1972) generally are developed from the period of record by selecting either the minimum daily flow from each year, or, more commonly, the average 7-day minimum from each year of record. These values are arranged in order of magnitude (smallest value is assigned the number m=1) and their probability of occurrence (P) computed from:

$$P=m/(n+1) \tag{2}$$

where m = order number, and
n = number of years of record.

The reciprocal of P is defined as the recurrence interval (RI). The computations for P and RI using 9 water years (1937-45) of 7-day annual low flows (Q_7) is illustrated below:

Water Year	Q_7 (cubic feet per second)	m	$P=\frac{m}{n+1}$	$RI=\frac{1}{P}$ (years)
1937	111	6	0.6	1.67
1938	112	7	.7	1.43
1939	127	9	.9	1.11
1940	97	4	.4	2.50
1941	71	2	.2	5.00
1942	84	3	.3	3.33
1943	56	1	.1	10.00
1944	108	5	.5	2.00
1945	123	8	.8	1.25

A plot of the relation between Q_7 on the ordinate axis using a log scale and RI, on the abscissa axis using a Gumbel Type I distribution (Fig. 11), shows the average recurrence interval of any given Q_7 value. This curve shows

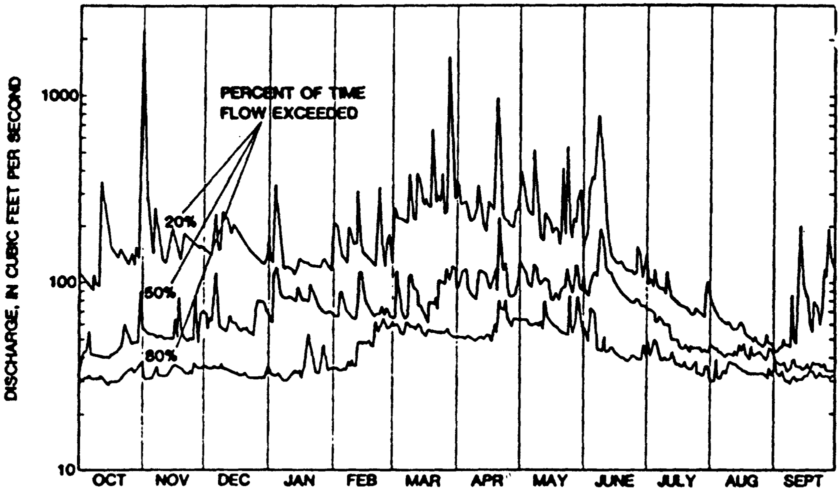


Figure 10.--Flow-duration hydrograph for Blue River at Milburn, Oklahoma, water years 1970-78.

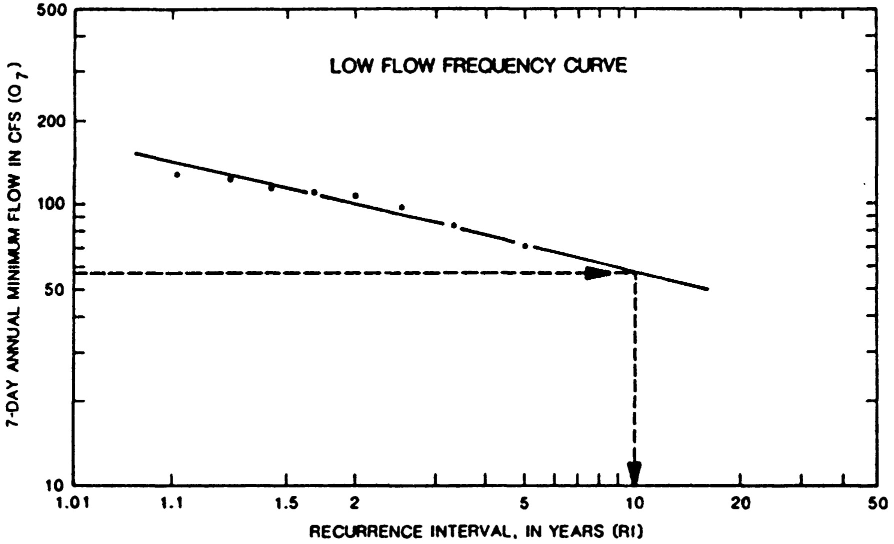


Figure 11.--Low-flow frequency curve of annual minimum 7-day low flow, water years 1937-45.

that an annual 7-day minimum flow of 56 ft^3/s or less would be expected to occur, on the average, only about once every 10 years.

Base Flow Frequency Curves

Base flow is that flow in the stream derived from the ground-water system. For unregulated stream, base flow approximates the total flow in the stream during prolonged dry periods (Fig. 12). Base flow is, therefore, an indication of available ground water in the basin.

Curves defining the frequency of occurrence of base flow at the beginning of the month base flow frequency curves are being developed in a manner similar to the low-flow frequency curves for selected index streams in Oklahoma for the purpose of defining the status of possible drought conditions prior to the normal summer dry-weather season. The base-flow values used to develop the frequency curves were obtained by estimating beginning of the month base flow from under the streamflow hydrographs for the winter and spring months. Beginning of the month base-flow frequency curves were then developed for each month as illustrated in Figure 13 for the Blue River in south-central Oklahoma. For example, a base flow of 25 $ft^{3/5}$ on January 1 can be expected to occur, on the average, only once every 10 years and would, therefore, indicate that drought conditions may exist or may be imminent if greater-than normal rainfall does not occur.

Ground Water Level Indexes

Graphs of monthly ground water levels at long-term sites may show existing or potential drought conditions provided the measured well is not influenced by local pumping or return flows from irrigation. Water levels in deep wells completed in aquifers of low permeability may show a lag of several months or more after a prolonged period of deficient precipitation and resultant decreased recharge to the ground-water system.

The U.S. Geological Survey monthly National Water Conditions publication (described previously) includes up-to-date water-level hydrographs for selected wells in the United States that compare average levels with levels

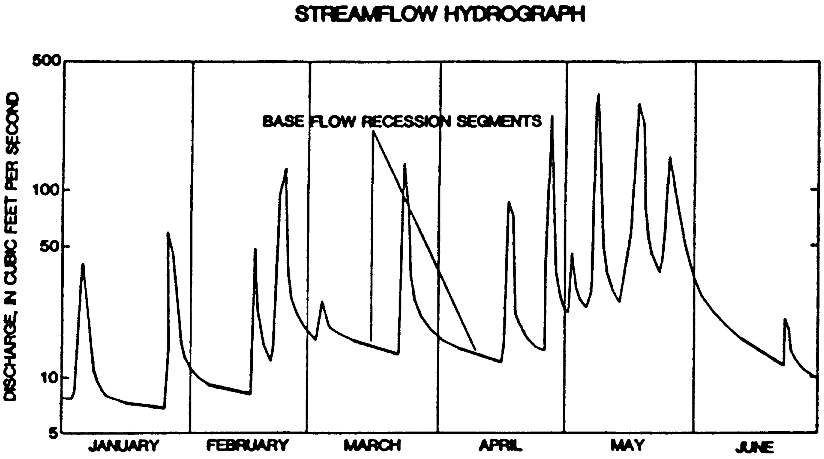


Figure 12.--Total-streamflow hydrograph and base-flow recession segments.

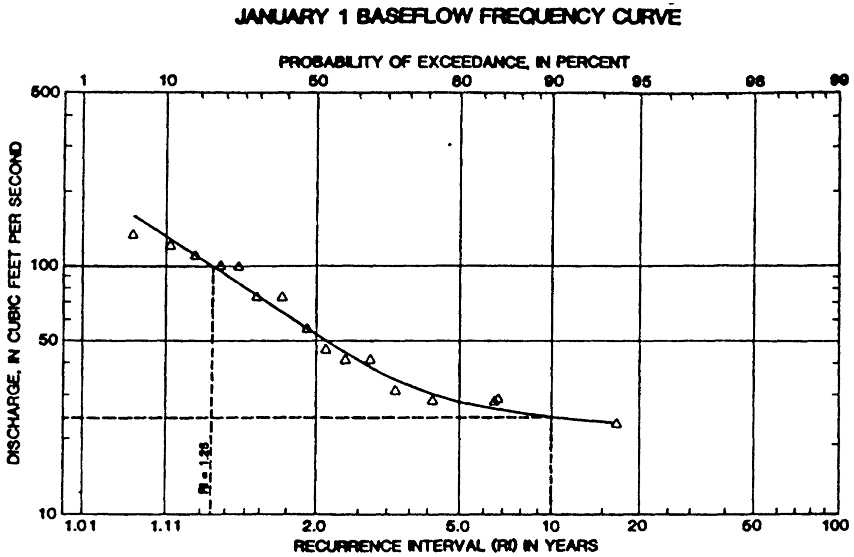


Figure 13.--January 1 base-flow frequency curve for Blue River at Milburn, Oklahoma, water years 1966-81.

recorded during the previous 1 to 2 years (Fig. 14). Even though these hydrographs reflect the status of ground-water conditions for rather limited areas, they do show water-level trends that may indicate existing or potentially significant deficiencies in local ground-water supplies.

The National Water Conditions also includes a map of the northeastern States (Fig. 15) showing areas of: (1) above normal ground-water levels--within highest 25 percent of historic range in levels, (2) normal levels--within 10 percent of average level and (3) below normal levels--within lowest 25 percent of historic range in levels. Arrows are included on the map to show areas of water-level rise and decline during the past month.

DROUGHT PREDICITION MODELS

The various drought indexes described previously in this report describe the severity of an existing or previous drought. There has been little success, however, in developing techniques which adequately predict a future drought. Atmospheric, oceanic, and other related geophysical processes are so complex that effective application of these factors in predictive models have not been successful--particularly when predicting beyond a few weeks.

Knowledge of existing surface- and ground water conditions can, however, be used to make predictions of subsequent water availability which are qualified on the basis of a range of probability. Obviously, if water supplies are less than normal at the beginning of a prediction period, the probability of a future drought occurring is much greater than if supplies are near normal or greater than normal at the start of the prediction period. Predictions of 1 or 2 weeks may be quite reliable, whereas predictions of several weeks or months generally are "educated guesses" based on present and antecedent moisture conditions.

Drought-prediction models developed in recent years have applied such factors as antecedant precipitation, available soil moisture, moisture content of snowpack, seasonal streamflow, base flow, and ground-water levels. Examples of some climatic and hydrologic models that can be used to make drought predictions are described below:

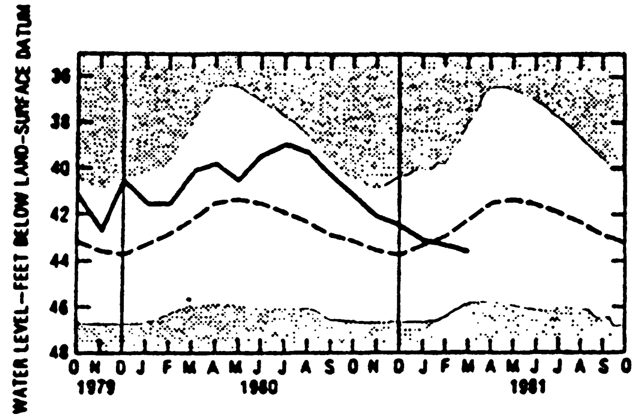


Figure 14.--Month and ground-water levels for well near Chapel Hill, Orange County, N. C. (modified from U.S. Geological Survey, 1981). Unshaded area indicates range between highest and lowest record for the month. Dashed line indicates average of monthly levels for 44 years of record. Solid line indicates level for current period.

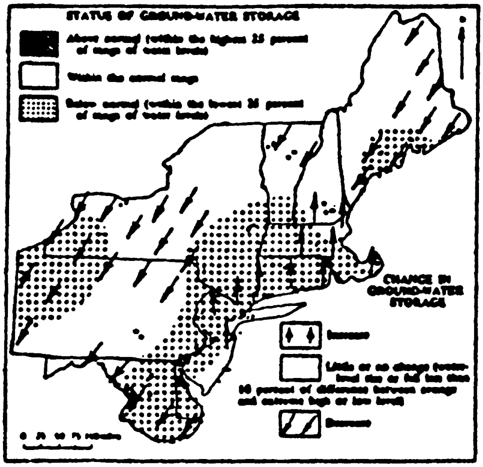


Figure 15.--Ground-water storage near end of March 1981 and change in ground-water storage during March 1981 (modified from U.S. Geological Survey, 1981).

Precipitation Models

Curry (1972) used historical precipitation data to define monthly and annual precipitation quantities associated with various probabilities of occurrence for several long-term weather stations in Oklahoma. Tables were prepared for each county showing the monthly and annual precipitation quantities for pre-selected probabilities ranging from 0.10 to 0.90. Monthly and annual precipitation quantities with corresponding probabilities of occurrence for the Tishomingo weather station in Johnson County in southern Oklahoma are shown below:

			Station		Tishomingo			County	Johnston				
	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	ANNUAL
Probability						Precipitation in inches							
.10	0.32	0.45	0.59	1.61	1.99	1.35	0.27	0.29	0.39	0.20	0.15	0.60	25.80
.20	.56	.78	1.01	2.34	2.78	1.93	.63	.77	1.17	.70	.53	.98	29.62
.30	.80	1.12	1.43	2.99	3.47	2.44	1.01	1.15	1.85	1.20	.89	1.31	32.60
.40	1.06	1.48	1.88	3.64	4.14	2.95	1.44	1.53	2.55	1.75	1.30	1.64	35.29
.50	1.36	1.88	2.39	4.34	4.85	3.48	1.95	1.94	3.31	2.37	1.76	1.98	37.95
.60	1.70	2.36	2.97	5.11	5.63	4.08	2.56	2.41	4.21	3.12	2.31	2.37	40.73
.70	2.13	2.95	3.70	6.04	6.56	4.79	3.22	2.98	5.30	4.06	3.01	2.83	43.86
.80	2.71	3.76	4.69	7.26	7.76	5.73	4.32	3.74	6.79	5.38	3.98	3.43	47.72
.90	3.68	5.10	6.32	9.20	9.66	7.20	6.27	4.97	9.22	7.58	4.76	4.40	53.42

The table shows, for example, that 10 percent of the Januaries at this site have a monthly precipitation of 0.32 in. or less. If meteorological information is available that indicates that less-than- normal precipitation is anticipated for the coming month (or months), selection of precipitation quantities for any probability less than 50 percent represents less-than-normal quantities with the smallest probabilities defining the most severe drought conditions. This technique for making predictions of future drought severity does not include the status of moisture at the beginning of a prediction period. Obviously, the impact of a 10 percent or 20 percent occurrence for each month throughout the year would be much more severe if the year started with less-than- normal water supplies than if available supplies were normal or greater than normal.

Streamflow Models

Seasonal Runoff Predictions Using Winter Precipitation and Runoff

Tangborn and Rasmussen (1976) developed a hydrometerological streamflow model to predict summer (May-September) streamflow from drainage basins in the North Cascade Mountains in Washington.

The model is based on a linear relationship between winter (October-April) precipitation and total annual runoff from the drainage basin. Spring storage, which is assumed to include snow, ice, soil moisture, and ground water, is estimated for the initial day of the prediction period from the relation:

$$\text{Spring storage} = Pw - Rw - Ew \qquad (3)$$

where Pw = winter precipitation, in millimeters
Rw = winter runoff and, in millimeters
Ew = winter evapotranspiration, in millimeters.

The results from this prediction technique were found to compare favorably with regression estimates based on mountain snow survey data that are very costly and time consuming to collect.

Seasonal Low-Flow Predictions Using Snow-Moisture Content Data

The moisture content of the snowpack measured in the headwaters of a drainage basin prior to the spring runoff season can provide a good indication of expected runoff during the subsequent summer low-flow season.

Riggs and Hanson (1969) developed a relation between April 1 snow survey data and the subsequent September monthly mean flow for the Bruneau River, located in the semiarid region of southwestern Idaho. This relation defines: (1) the "assured" mean September flow, representing the lower limit of runoff expected resulting from less-than- normal snowpack and (2) the "probable" mean September flow, representing expected runoff resulting from a normal winter snowpack (Fig. 16). This prediction technique can be quite reliable in arid and semiarid regions where spring and summer rainfall is not significant.

Seasonal Low-Flow Predictions Using Base-Flow Recession Characteristics

Base flow, as previously described, is that flow in the stream that is discharged from the ground water. During extended dry periods, base flow may approximate the total streamflow (Fig. 12).

Riggs and Hanson (1969) developed a family of base-flow recession curves for the Potomac River in Maryland (Fig. 17), using recession segments from recorded streamflow hydrographs (Fig. 12). The recession curves were used to project base flow through the summer after the spring runoff season. The projected base flow assumes that no appreciable precipitation occurs during the forecast period and, therefore, is an estimate of the minimum "assured" summer flows.

Ground-Water Models

Ground water digital models have been developed by the U.S. Geological Survey (Trescott, and others, 1976 and McDonald and Harbough, 1984) primarily for the purpose of making projections several years ahead prior to possible changes in regional ground water levels in an aquifer.

A ground water digital model operates on the principle that a mathematical expression can be used to describe the movement of ground water within specified boundaries of an aquifer. Development of the model requires that the aquifer constants--hydraulic conductivity and specific yield--and the saturated thickness of the aquifer are known or can be estimated.

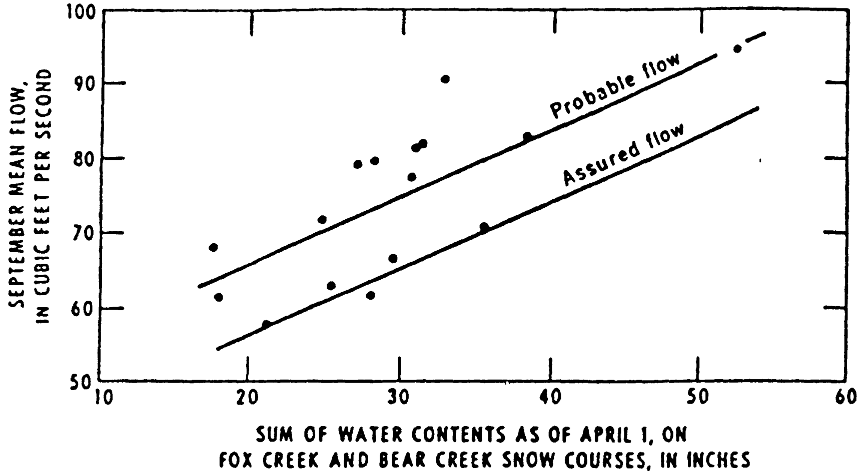


Figure 16.--Predictive relations for Bruneau River near Hot Springs, Idaho (from Riggs and Hanson, 1969).

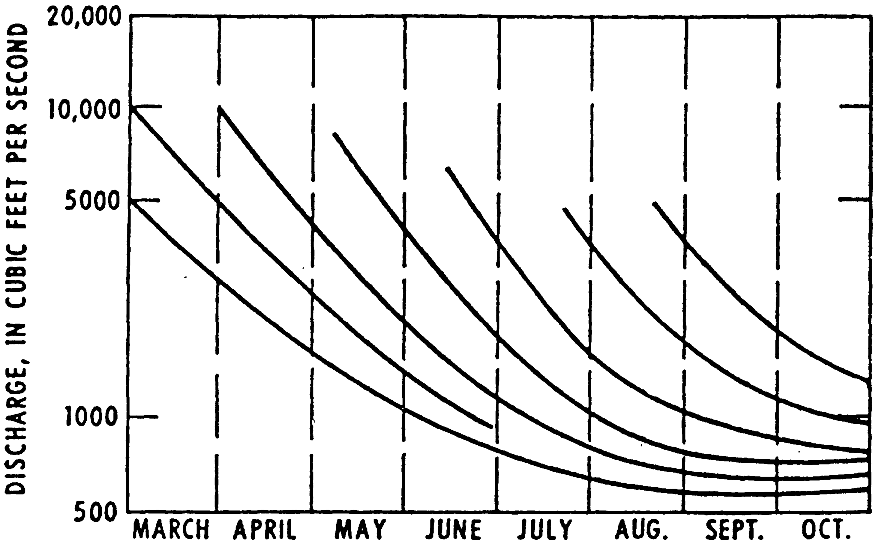


Figure 17.--Base flow recession curves for the Potomac River at Point of Rocks, Md. (modified from Riggs and Hanson, 1969).

To evaluate the effect that an assumed or anticipated prolonged drought may have on an aquifer, the ground-water model may be used to compute the average decline in water level as a result of drought stresses on the aquifer.

A decline in the water level, caused by a drought, can be expected to occur because of two factors: (1) A decrease in recharge to the aquifer as a result of below normal precipitation and (2) an increase in pumping from the aquifer to satisfy irrigation requirements not met by the normal supply from precipitation. Application of the model to predict these drought-causing, water-level declines requires that the decreased recharge and increased pumping rates be estimated or assumed.

HYDROLOGIC AND HYDROGEOLOGIC CONSEQUENCES OF DROUGHT AND THEIR IMPACT

Agriculture usually is the first activity to experience the effects of a drought. The first indication that a drought is developing is the symptom of plant stress as a result of soil moisture deficiency. Next is an awareness of less than normal flow in streams. If the drought continues, reservoir levels will begin declining below normal levels. High temperatures and hot winds will create greater than normal demands for water and water managers may begin rationing water.

A drought lasting several months may begin to cause greater than normal ground water level declines because of decreased recharge from precipitation, an increased pumping stress, and greater than normal evapotranspiration rates. Some wells may dry up or water levels may decline to levels from which it is economically impractical to pump. At this time other sources of water may be sought, such as diversions from nearby streams or the establishment of additional water wells, which impose an added stress on the local ground-water system.

In some parts of the United States, delivery systems, particularly to outlying rural communities, are not large enough to meet water demands during a drought and importation of water from nearby lakes or streams may be necessary to meet basic water needs.

A prolonged drought can have far-reaching effects. For the farmer, crops may produce small yields or be destroyed. As vegetative cover diminishes, soil erosion from wind will remove fertile top soil and bury any short crops.

Significant ground water declines resulting from excessive pumpage can create voids in the unsaturated soil and may eventually cause land subsidance.

The quality of surface waters can be expected to deteriorate as streamflow declines. Fish die when stream temperatures become excessive and birds and other wildlife perish because of lack of water and sufficient food. Along coastal areas, reduced freshwater flow into estuaries allows saltwater to enroach upstream into freshwater zones. Saltwater may even extend into tidal reaches of freshwater tributaries.

An unusually low snow depth in high mountain areas can create an economic disaster for ski resorts and related winter tourist businesses. Reservoirs that are dependent on the annual spring snowmelt runoff for their supply may not be able to meet the subsequent summer water demands after winters of unusually low snowfall--particularly if low snowpacks occur in successive years.

Prolonged periods without rain create a serious fire hazard in the forests and grasslands. After the fire has devastated an area, the eventual rain will cause soil erosion and deposition of sediment in the streams. If the fire danger is too high, forests may be closed to hunting and camping and resort owners can lose an important seasonal business.

The large rivers in the United States such as the Missouri, Arkansas, and Mississippi which provide important navigation also may be affected. The excessive time required to fill locks for upstream traffic can cause delays and unusually low flows may prevent larger boats from passing through some reaches of the river.

U.S. GEOLOGICAL SURVEY ACTIVITIES IN DROUGHT STUDIES

The responsibility for providing climatic and hydrologic information about droughts to the public resides primarily with three Federal agencies:

- The National Weather Service monitors climatic variables related to droughts and provides weather and river forecasts for major rivers.
- The U.S. Soil Conservation Service provides information about soil moisture and western mountain snowpacks.
- The U.S. Geological Survey provides frequency estimates and historical data about streamflow, reservoir, and ground-water conditions.

Significant improvements in short-term weather forecasting of several days have been made in recent years using satellite data. Continued research is needed in this area to provide accurate long-term forecasting for several weeks or even months.

The U.S. Geological Survey, is presently conducting hydrologic studies directed at improving methods of preparing for and coping with droughts. These studies include three major objectives:

1. Provide better information and planning tools to water managers so that they can mitigate a hydrologic drought--that is, collect data from more sites during a short period.
2. Improve the reliability of drought estimates and the extrapolations of probability and severity of droughts.
3. Explore the conjunctive use of ground-water and surface-water supplies, and the potential of new ground-water and surface water supplies.

Man may never be able to prevent droughts, but development of reliable forecasts warning the farmer and water managers several months in advance that a severe drought is pending will be a major step towards decreasing the impact of this hydrologic hazard on mankind.

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ASSESSMENT TECHNIQUES FOR COASTAL HAZARDS

by

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INTRODUCTION

The most common coastal hazards occur as sudden rises in sea level and the subsequent inundation of adjacent populated areas. Strong weather systems often generate extreme high waters in the form of storm tides and high wave conditions. Hurricanes can also produce rapid sea level rises due to storm surges. Since this flooding is due to atmospheric forcing, warnings of these events are presently the responsibility of the National Weather Service (NWS). High water conditions can also be generated by non-meteorological conditions, the most common being the astronomical tide, the tsunamis of the Pacific Ocean, and harbor oscillations.

Other hazardous conditions include beach erosion, which often accompanies severe ocean storms or high astronomical tide conditions, and marine spills of oil or hazardous substances. Dangerous marine conditions can also result from severe coastal fog and from ice-laden waters, although we presently have few standard assessment techniques for these last two.

While it's common to include the shores of the Atlantic and Pacific Oceans and the Gulf of Mexico within the definition of the coastal zone, we also include the beaches of the Great Lakes. These bodies of water are so large that they respond to wind and environmental forces in nearly the same way as the oceans.

METHODS OF PREDICTION

The NWS uses several techniques to develop forecasts of severe marine conditions. Predictive approaches can usually be classified as statistical methods, numerical models, or simulation models.

Statistical Methods

The most common approach is to develop a forecast equation from a statistical analysis of the available data. The prime requirement is the gathering of a large number of observations of the event to be forecast (called the predictand) for each specific location. At the same time, we also need observations of the variables (called the predictors) which we think will be correlated to the predictand. At the NWS we are fortunate to have as predictors the output of numerical atmospheric models. Output variables include atmospheric pressure, wind velocities, and temperatures forecast for several vertical levels at model grid points for 6-hour intervals to at least 24 hours. Quantities predicted by statistical methods include extra-tropical storm surges and severity of beach erosion.

Numerical Models

When the number of observations of the event to be forecast is low, we must rely on the solution of the equations of fluid motion by numerical methods. That is, we construct a grid mesh which incorporates the bathymetry and topography of the area of interest, and solve finite-difference approximations of the equations at all grid points. Such an approach is used for the calculation of hurricane storm surge.

Simulation Models

Simulation models, which have the characteristics of both statistical and numerical models, are used when the physical laws are not adequately known, and when there is too little data for statistical methods to apply. Oil and hazardous substance behavior and some properties of wave generation are usually assessed by simulation techniques.

Each of the hazardous events to be discussed is treated by at least one of the above three methods.

HIGH WATER CONDITIONS

Rapid sea level changes generally result from the strong winds that accompany atmospheric storms over the ocean. Before discussing these we will describe the periodic rise and fall of the sea due to the astronomical tide, which may combine with and amplify the storm-generated tide (the surge).

Astronomical tide

The most frequently observed sea level change is that of the astronomical tide. High astronomical tides may, by themselves, cause flooding, but more severe flooding occurs when storm surges or high waves occur at the time of high tide.

The astronomical tide, which we experience as one or two maximum and one or two minimum water levels a day, occurs because any point on the surface of the globe is moved by the earth's rotation past two great bulges in the ocean's water. In a simple analysis, one of the bulges occurs on the earth in the area closest to the present position of the moon; the other bulge occurs on the opposite side of the earth. The bulges are the result of three forces acting on the water--the gravitational attraction of the earth, the gravitational attraction of the moon, and a centripetal force from the rotation of the earth-moon system about its center of mass.

The actual picture of the tides is complicated by the presence of the sun, which has a gravitational attraction approximately 44% as strong as the moon's, and by the existence of the continents, which block the free flow of water over the globe. Nevertheless, the dominance of astronomical motions in producing the tide has lead to successful prediction by the method of harmonic analysis.

The tide at any point can be represented by the sum of constituent tides with frequencies which are harmonics of orbital frequencies of the sun's and the moon's motions. Following Schureman (1940), we have

$$h = h_o + \sum_i h_i f_i \cos (\omega_i t + E_i - k_i) \tag{1}$$

where h = tidal tide,

h_o = mean height above a datum plane,

h_i = the amplitude of constituent i,

f = the node factor for constituent i,

ω_i = frequency of constituent i,

E_i = equilibrium argument of constituent i at t = 0, and

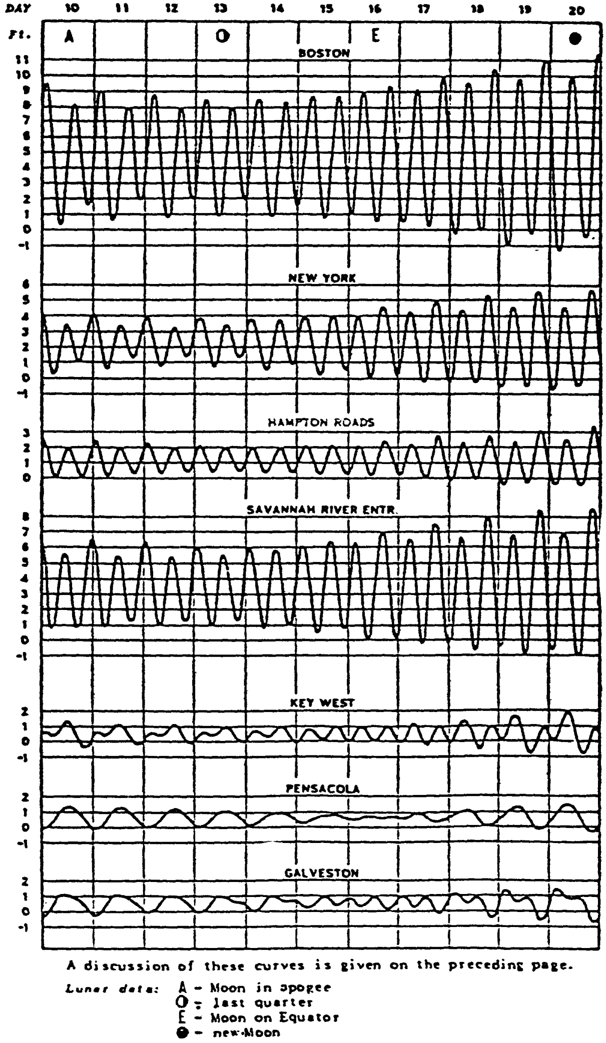
k_i = epoch angle for constituent i.

The values of f_i, ω_i and E_i can be found by theoretical considerations, and the values of h_o, h_i, and k_i are obtained by analysis of hourly data for a single location for a period of one year or more. The harmonic equation is so successful for prediction that tables for high and low waters can be prepared years ahead of time.

The difference between high and low water is called the tide range. The range is not constant, but varies throughout the month. Ranges reach their maximum values near the time of full or new moon, when the sun, moon, and earth are nearly aligned (Figure 1). The uncommonly high tides during these times are called spring tides. Storms which occur at the same time can be especially hazardous.

Wind Waves

The NWS routinely makes forecasts of wind wave heights and periods, and swell heights and periods for the open ocean in the northern hemisphere, and for wave conditions in certain nearshore areas, such as the Great Lakes and Chesapeake Bay. Wave height is defined as the distance from trough to crest, and the period is the time between passage of successive crests. Wind waves are those waves directly associated with the winds in a specific storm, and they move roughly downwind. Swell are the waves that have propagated out of their area of generation. At any point in the ocean, the swell can be coming



A discussion of these curves is given on the preceding page.

Figure 1. Typical tide curves for selected U.S. port cities. Note that the tide range is largest near the time of the new moon. (NOAA Tide Tables)

from several different directions. The time over which a wind blows from the same direction at the same speed is called the duration. Waves in the process of generation propagate downwind, and the distance they travel while experiencing roughly the same wind conditions, is called the fetch.

Forecasts are generally made for the significant wave height, which is the average height of the one-third highest waves. The two common ways of predicting the significant wave height are the singular method and the spectral method.

Singular Wave Prediction

A great simplification to the problem of wave forecasting is the assumption that a constant wind with a fixed duration over a given fetch will produce waves of a single height and a single period. This assumption allows us to produce a simple, empirical relationship for the significant wave height. Over the open ocean, Pore and Richardson (1969) used

$$H = 0.0176\ V^2 + 0.5 \quad \text{and} \tag{2a}$$

$$T = V(0.25 + 0.08D) + 2.0 \tag{2b}$$

where H = significant wave height (ft),

V = wind speed (m/s),

D = duration (hr), and

T = significant wave period (s).

These waves propagate in a direction roughly parallel to the wind direction. When the wind direction changes, the waves continue to propagate as swell but no longer grow. The wave height slowly diminishes and the period lengthens, so that at a later time,

$$T^* = (T^2 + 2.1t)^{1/2} \quad \text{and} \tag{3a}$$

$$H^* = H(T/T^*)^{2.36} \cos\alpha \tag{3b}$$

where T^*, H^* = swell period(s) and height (ft),

t = swell age (hr), and

α = propagation angle relative to the original wind direction.

Waves change greatly as they approach the shore. The group velocity slows, and the waves may focus due to bathymetric influence. Prediction of near-shore and breaking waves is an area of active research.

Alternate methods are needed to forecast waves in confined areas where fetch is limited. Pore (1979) developed equations for the Great Lakes. Data from specific storms are abundant, and the forecasts agree reasonably well with the observations (Figure 2,a).

Spectral wave prediction

The U.S. Navy regularly makes ocean wave forecasts with a Spectral Ocean Wave Model (SOWM) (Pierson, 1982). Rather than assuming a singular significant wave, the SOWM computes a spectrum of wave energies, S_{ij}, for several frequency bands (i), and wave propagation directions (j). At a given frequency, the wave height is related to the spectral density by

$$H_{ij} = [8\ S_{ij} d\sigma]^{1/2} \tag{4}$$

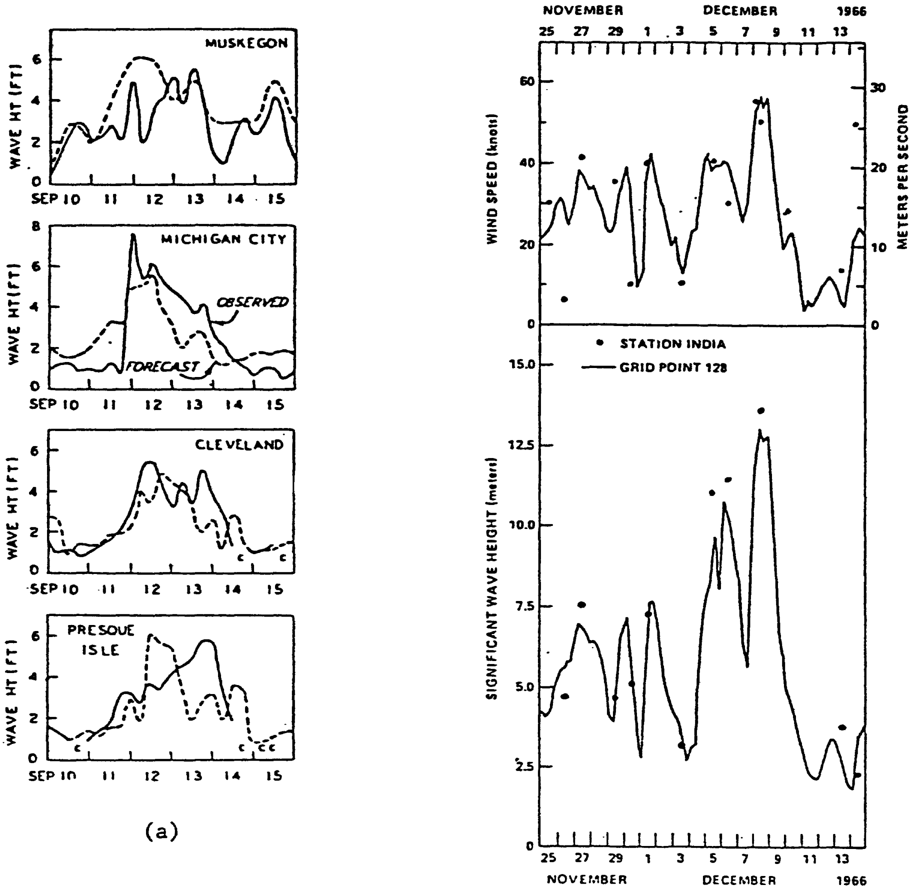
For a fully developed sea,

$$S = 8.1\mathrm{x}10^{-3} g^{2} \sigma^{-5}\ \exp[-0.74(g/V\sigma)^{4}] \tag{5}$$

where $\sigma = 2\pi/T$. The shape of the spectrum changes with time as wind adds energy and as waves dissipate energy when they move out of the generating area. At present, spectral wave forecasts have approximately the same quality as singular wave forecasts (Figure 2,b).

Extra-tropical storm surge

Extra-tropical storms originate in regions north of the tropics (in contrast to hurricanes, which have a tropical origin) and are commonly thought of as strong low-pressure systems. Storms which move out to sea can cause water level rises, called surges, from both the action of the wind on the water and the reduced atmospheric pressure. The surge is defined as the observed water level minus the astronomical tide. The duration of the extra-



(a)

(b)

Figure 2. (a) Observed and forecast significant wave heights (singular method) at cities on Lake Erie for the period 10–15 September 1975 (Adapted from Pore, 1979). (b) Comparison of observed and predicted winds and waves for 20 days in 1966 (spectral method) at Station India, located in the North Atlantic Ocean near 59°N, 19°W (Pierson, 1982).

tropical storm surge is frequently several tidal cycles. Damage is most extensive during those periods when high tides and storm surges occur.

In an extreme simplification of the equations of motion for water, the sea surface slope is balanced by the wind stress, so that

$$\partial h/\partial x = \tau_s/(\rho_a g D), \tag{6}$$

where h = sea level rise,
τ_s = wind stress,
ρ_a = air density,
D = total water depth, and
g = gravitational acceleration.

Since wind stress is generally thought to be proportional to wind speed by

$$\tau_s = \rho_a C_D |\underset{\sim}{V}| \underset{\sim}{V} \tag{7}$$

and the wind speed is proportional to the horizontal pressure gradient, the atmospheric pressure is a prime candidate for a predictor in a statistical method. Also, the pressure is directly involved, since the sea level will rise slightly when the atmospheric pressure is reduced—a phenomenon known as the inverted barometer effect.

Accordingly, Pore (1970) used a statistical technique called screening regression to correlate observed storm surge at Atlantic City, New Jersey and other sites with forecast surface pressures. The pressures were at forecast points of the 6-Layer Primitive Equation atmospheric model (Shuman and Hovermale, 1968), run twice daily by the NWS. The equation developed for Atlantic City was

$$\begin{aligned} SS_o = {} & -129.92 + 0.09\, P^{45}_{0} + 0.47\, P^{1}_{-12} - 0.35\, P^{12}_{-6} \\ & + 0.39\, P^{9}_{-6} - 0.93\, P^{34}_{0} - 0.16\, P^{15}_{-24} + 0.32\, P^{21}_{0}, \end{aligned}$$

where

SS_o = storm surge (tenths of ft) at time 0, and

P_n^m = surface atmospheric pressure (mb) at grid m and time n (hr).

The correlation coefficient, based on the independent data, was 0.91, and the surges verify well with observations. An example of predicted surges for several storms is shown in Figure 3.

Hurricane storm surge

Extremely severe flooding and water damage occurs when hurricanes slam into the coast. The NWS has developed extensive facilities for hurricane tracking and for prediction of the surges due to the these storms. Because of the relative rarity of hurricane events at any particular coastal location, and the consequent paucity of data, numerical models have been regularly used to predict these storm surges.

Hurricanes are strongly developed low pressure systems with warm air centers and central pressures 50-100 mb lower than the surrounding air. By definition, winds in excess of 64 kt (33 m/s) occur. Although a large area of ocean may be covered by hurricane clouds, the region of maximum wind speeds generally is confined to a few tens of kilometers from the storm center (the eye). The circulation around the eye is counterclockwise in the northern hemisphere, and there is some inward flow.

As the hurricane tracks toward the land, a bulge of water rises under it due to the inflow of the wind and the inverse barometer effect. When it meets the coast, the largest surge generally occurs on the right side of the eye, where the onshore winds are the strongest (Figure 4,a). If a storm tracks along the coast, each land station feels the effects of the storm as it passes by.

The solution technique begins with the equations of horizontal motion and mass conservation for water, which are

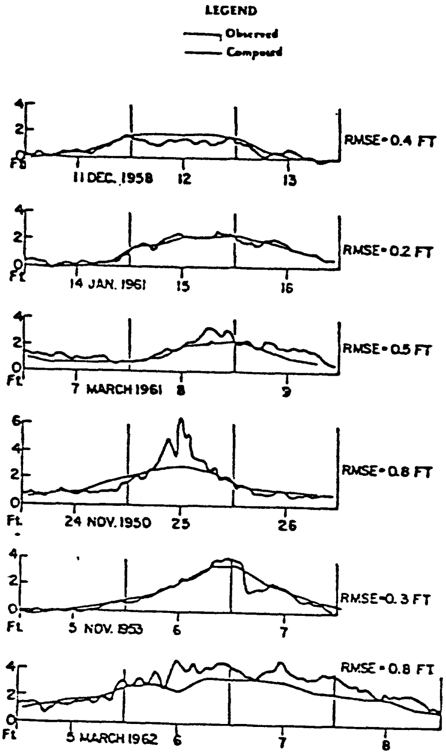


Figure 3. Comparison of the computed and observed extra-tropical storm surge at Atlantic City, New Jersey, for several events (Pore, 1970).

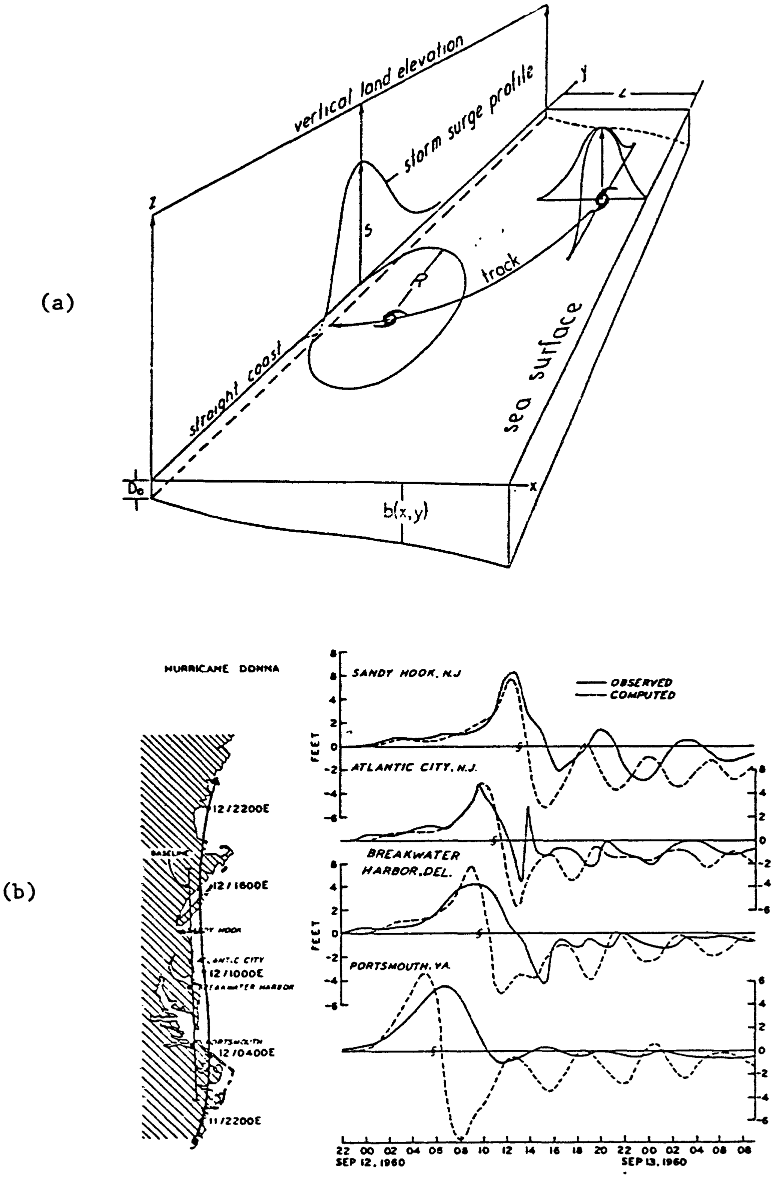


Figure 4. (a) Schematic drawing showing the hurricane storm surge profile to the right of the storm's landfall point. (b) Comparison computed and observed surges for four coastal cities for hurricane Donna (Jelesnianski, 1976).

$$\partial/\partial t(\underset{\sim}{w}) - f\underset{\sim}{k} \times \underset{\sim}{w} = -g\nabla(h+h^*) + (\underset{\sim}{\tau_s} - \underset{\sim}{\tau_b})/(\rho w\ (h+d)) \qquad (9)$$

and

$$\partial/\partial t(h) + \nabla\cdot[\underset{\sim}{w}(h+d)] = 0, \qquad (10)$$

where $\underset{\sim}{w}$ = water velocity,

f = Coriolis parameter,

$\underset{\sim}{k}$ = unit normal in the vertical direction,

h = surge height,

h^* = height equivalent to the inverse barometer effect,

$\underset{\sim}{\tau}_s$ = surface wind stress,

$\underset{\sim}{\tau}_b$ = bottom frictional stress,

ρ_w = water density, and.

d = mean water depth.

Jelesnianski (1976) has solved the equations of storm surge on a set of grid meshs which cover the Atlantic and Gulf of Mexico coasts of the United States. The computer program is in operational use by the National Hurricane Center of the NWS to forecast hurricane storm surge. A sheared grid allows coastline contours to be more closely modeled. Computed surges for the 1960 hurricane Donna are shown in Figure 4,b.

Another model currently in the final stages of implementation is the program for Sea, Lake, and Overland Surges from Hurricanes (SLOSH) by Jelesnianski and Chen (1984). SLOSH uses a polar-stereographic grid mesh, which is then mapped into a rectangular grid mesh. The storm surge equations are then solved in the normal way over oceanic areas, but the model also treats inland flooding and the overtopping of barriers.

Non-meteorological sea level rises

Two important sources of water level rise and inundation that are not directly dependent on atmospheric forcing are tsunami waves and harbor oscillations. Both these phenomena can be thought of as long waves propagating in shallow water.

The tsunami (the Japanese word for tidal wave) is caused by the rapid change in bottom contours that accompany an earthquake. They are especially common in the Pacific Ocean, where a tsunami wave may spread out with wave crests moving at close to 500 mph. With the approach of the wave, a shore station may experience a small water level drop, followed by a gradual rise in sea level. Often large individual waves accompany the rise, and produce extremely high water.

The National Oceanic and Atmospheric Administration's (NOAA's) Pacific Tsunami Warning Center in Honolulu, Hawaii, maintains a monitoring network of tide gages and seismometers to observe the occurrence of tsunamis. Numerical models have been used to simulate the motion of these waves with good results. For example, Huang et al. (1972) use long wave equations like Eq. (10) but without the Coriolis or stress terms. The driving force entered into the continuity equation,

$$\partial/\partial t(h) + \nabla.\ [\underset{\sim}{w}(h+d)] = \partial/\partial t\ (d), \qquad (11)$$

where the term on the right side accounts for differences in the bottom contours.

Very little is done by federal agencies in the area of severe harbor oscillations. The seiching motion in these semi-enclosed basins will generally be at the natural frequency of the enclosure. Harbors of complex shape may have a multitude of seiching motions. The oscillations of some harbors is due to incoming wave trains and, in some cases, to wind forcing from a favorable direction.

BEACH EROSION

The storms that generate high surges and large gravity waves also create the conditions for the erosion of the shoreline. High water allows larger waves to get closer to shore before they break, so the destructive power of the surf is felt directly on the beach and on areas further inland. Beach changes that occur on the seasonal time scale or on the geological time scale, while important, do not present the hazards which accompanies storm-related erosion.

Although there has been much study of sand transport by waves and currents, there are presently no simple relationships that can be used to estimate the size of beach changes. There is enough data for a statistical approach, and Richardson (1978) developed some prognostic equations for erosion intensity. Incidents of erosion or wave damage described in NOAA records were ranked on a scale from 0 to 4; 0 referred to no erosion, 1 to minor, 2 to moderate, 3 to major, and 4 to severe erosion.

Predictors chosen by Richardson (1978) were based on the observed storm surge height and duration, and on the astronomical tide. Two equations derived were

$$I_1 = -1.34 + 0.24\ T + 0.09S^2 + 0.54\ D + 0.12\ D^* \tag{12a}$$

$$I_2 = -0.66 + 0.15\ T + 0.16S + 0.23\ D + 0.23\ D^* \tag{12b}$$

Here I_1 is the intensity for all sites in the states of Maine and Massachusetts, and I_2 is for the sites in Rhode Island, New York, New Jersey, Delaware, Virginia, North Carolina, and South Carolina. Also,

T = maximum height of the sum of the astronomical tide and the storm surge for the event (ft),
S = maximum storm surge height for the event (ft),
D = generalized, non-dimensional storm duration, and
D^* = location-dependent, non-dimensional storm duration.

The storm duration, D, is defined as the number of consecutive times during the event that the storm surge plus the astronomical tide equals or exceeds a critical height. The critical height is defined as 2.5 feet plus half the range of the spring tide at the gage. The variable duration, D*, is defined the same way as the storm duration except that another value, derived for each location, was used in place of the 2.5 feet for critical height.

The forecast intensity was verified on a set of independent data, and two measures of success were computed. The first measure is the fraction of correct forecasts, which was 0.62 for the 12-h forecast, and 0.68 for the 36-h forecast. The second measure is the threat score (TS), which is

TS = correct forecasts/(observations + forecasts
- correct forecast)

Threat scores for large erosion at 12-h and 36-h forecasts are 0.26 and 0.22, respectively, where large erosion is defined as severe, major, or moderate erosion.

SPILLS OF OIL OR HAZARDOUS SUBSTANCES

The U.S. Coast Guard (1983) reported that in 1982, the latest year for which they have complete data, the volume of oil spilled in U.S. coastal waters was 15.4 million gallons (56,700 m^3). Although there were nearly 7,000 individual spill events, 61 percent of the total was due to only two events. In the same year, 154,000 lb (70,000 kg) of hazardous substances were discharged in 72 separate events. Hazardous chemicals include PCB's, pesticides, dry cleaning fluids, and liquid plastics.

Both NOAA and the U.S. Geological Survey (USGS) have developed forecasting models for oil behavior. NOAA's emphasis has been on simulating single events, and USGS's on risk analysis, or multiple event simulation. Since risk analysis actually models many single events, we begin with the NOAA method.

Single event models

Since the dynamics of oil motion are quite complex, numerical models have been rarely used (Hess and Kerr, 1979). The simulation model of Galt and Payton (1983) has been used successfully in several oil spills. The single-event method estimates the oil velocity as

$$\underset{\sim}{U} = c\underset{\sim}{V}_\theta + \underset{\sim}{W}_T + \underset{\sim}{W}_R + \underset{\sim}{W}_B + \underset{\sim}{W}_W \qquad (13)$$

where $\underset{\sim}{U}$ = oil velocity
$\underset{\sim}{V}_\theta$ = wind velocity, rotated clockwise by an angle θ,
c = wind drift factor, and
$\underset{\sim}{W}$ = water velocities for tidal (T), random (R), background (B), and wave (w) currents.

The wind velocity is usually taken from an NWS forecast. The wind drift factor, c, is usually taken to be 0.035, and the wind drift angle, θ, varies from 0° to 20°. The random current can be simulated by the random walk method, so that $\underset{\sim}{W}_R$ has a constant magnitude, q, and a randomly chosen direction. The magnitude q is related to the horizontal diffusion coefficient, D, by

$$D = \frac{1}{2N} q^2, \tag{14}$$

where N is the number of walks, or steps, per unit time. For many areas, tidal currents can be found in publication of the National Ocean Service. Background currents are generally taken to be the long-term average circulations. The wave-drift current is not well known and is frequently neglected.

An oil spill is then modelled as a set of drifting point masses, each with a local velocity determined by Eq. (13). By performing a numerical integration of

$$\underset{\sim}{X} = \underset{\sim}{X}_o + \int_o^t \underset{\sim}{U}\, dt \tag{15}$$

the position of the oil at any time can be forecast. Hindcasts of an oil spill in San Francisco Bay (Hess, 1984) show that only the gross features of a spill can be simulated this way.

Multiple-event models

The USGS has produced a model (Smith, et al., 1982) which assesses the risks to coastal resources of offshore drilling and petroleum transport. Trajectories of particles are calculated with Eqs. (13) and (15), but for

$$\underset{\sim}{W}_T = \underset{\sim}{W}_W = \underset{\sim}{W}_R = 0,$$

and with $c=0.035$ and $\theta = 20°$. The background current is interpolated from monthly averaged observations. The model was verified by simulation of a single event spill with local winds (Figure 5,a).

Wind input is based on local wind records. When a spill is simulated, the wind at the first step is chosen at random from the record. The wind at the next step is then generated by a Markov process, i.e., selected at random, given a distribution of probabilities. The probability of change is derived from the wind record.

A large number of spills (up to 200,000), each with a random starting time, is simulated for a specific release site. Target areas of interest are examined at the end of the run to determine their risk (probability of being hit). An example of the use of this model in the selection of oil-producing tracts in southern California waters was given by Smith et al. (1979). The essential aspects of their study are discussed below.

At each coastal location, i, there is a conditional probability that oil will reach it from production site, or shipping route, j. If that probability is P_{ij}, then the economic impact (cost) of a spill of oil at site j is

$$E_{ij} = C_i P_{ij} V_j \qquad (16)$$

where C_i = economic cost of the damage to coastal location i per unit of oil, and

V_j = volume of oil being pumped at site j.

The problem is then to maximize the total volume of production, V, where

$$V = \sum_j V_j \qquad (17)$$

subject to $V_j \leq V^*_j$

where V^*_j is the maximum production volume allowable from site j. At the same time, environmental costs must be kept small, i.e.,

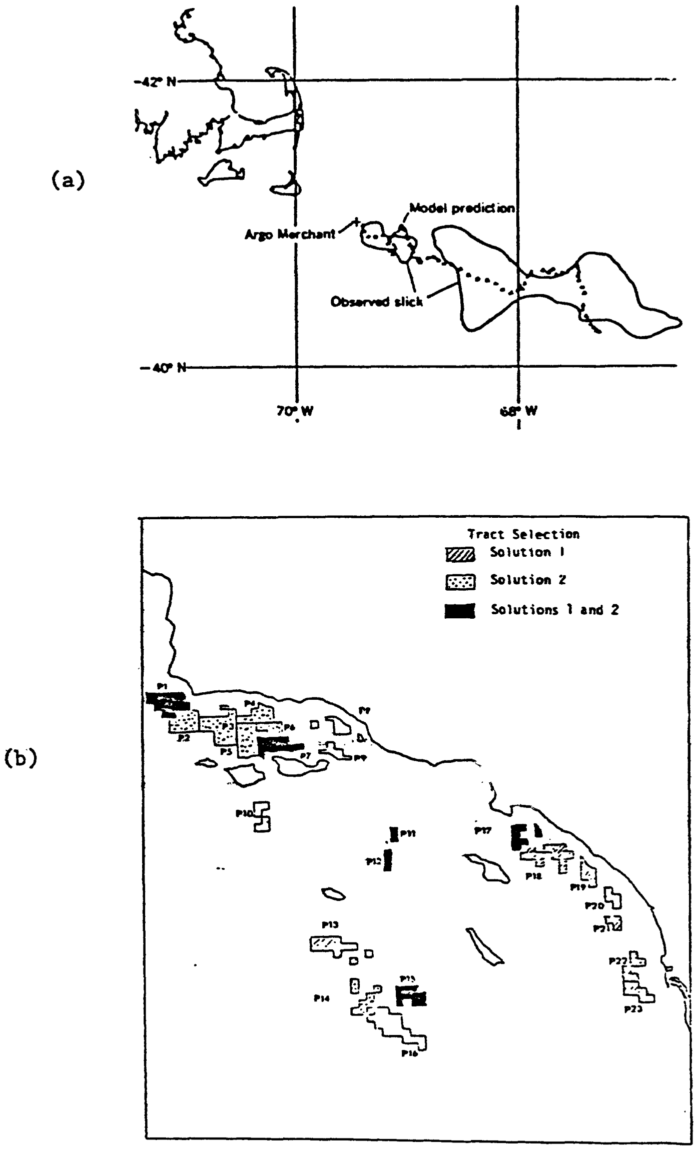


Figure 5. (a) Comparison of observed oil distribution during the Argo Merchant spill and the streakline predicted by the Geological Survey's model (Smith, et al., 1982). (b) Two sets of oil drilling tracts chosen to minimize environmental damage based on two sets of unit costs for resources (Smith, et al., 1979).

$$E_i = \sum_j E_{ij} \leq E^*_i$$

where E^*_i is some acceptable cost of damage for site i.

The unit economic value of the different marine resources, C_i, is likely to be subjective, so that small changes in C_i can produce different production strategies. Smith et al. (1979) show the results of two tract selection procedures that differ only by the doubling of the acceptable cost of damage to sites of commerical shellfish production (Figure 5,b). Clearly, the risk analysis method is a powerful tool for choosing between alternative oil leasing plans, provided the economic costs of environmental damage are determined in an acceptable way.

OTHER COASTAL HAZARDS

Methods of predicting the severity of coastal inundation, beach erosion, and spilled oil impact have been developed by NOAA, USGS, the U.S. Army Corps of Engineers, and others. Two coastal hazards, heavy fog and ice damage, are more difficult to forecast, and little has been done to date in advancing prediction techniques. Fog occurs whenever warm, moist maritime air moves off the water onto colder land. The air cools, and its ability to hold water vapor diminishes. The moisture changes the phase into liquid, forming fog. The extent of ice cover on the Great Lakes and along the coast of Alaska is now monitored by satellites. In ocean waters, wind and currents moving the ice toward land can cause piling up of frozen slabs at the shore. Techniques to assess the severity of ice damage are today in their infancy.

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LECTURE NOTES ON
HYDROGEOLOGIC HAZARDS

By

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INTRODUCTION

Flooding of river valleys as a result of unusual storms and (or) rapid melting of snow, and coastal flooding as a result of unusual storms, are the natural events that cause the greatest loss of human life and property on a year-to-year basis throughout the world. Those types of flooding generally have little, if any, relationship to the local geologic conditions. There are, however, other hazardous conditions that are caused by processes involving both the geologic conditions and the hydrologic conditions. These are often called "hydrologic hazards." Hydrogeologic disasters, often caused by geologic events that resulted in flooding of populated areas by water or mud, have produced some of the largest sudden losses of life, and untold losses of property, during the history of mankind.

Hydrogeologic hazards may be thought of as geologic hazards in which water has a significant part. They include hazardous occurrences in which water creates or adds to the risk or damage, but the trigger is a geologic process or event such as an earthquake or volcanic eruption. Examples can be found of every gradation between mainly hydrologic hazards, such as riverine floods, to mainly geologic hazards, such as landslides due to gravity alone.

In this discussion, emphasis is placed on the recognition of potential hydrogeologic hazards by reviewing places where those hazards have led to disasters, and the conditions that contributed to the disasterous events. Most of the places discussed are in the United States, where the variety of geologic conditions and natural hazards has produced a broad array of dramatic examples.

The types of hazards that are discussed include land instability induced by or related to water, land subsidence resulting from ground-water pumping, surge waves induced by landslides, and floods of mud or water that are not caused directly by storms. A special kind of hazard is natural water that is unusable or that has undesirable chemical characteristics because of mainly natural conditions or processes.

I) Role of water in unstable land

A) Land oversteepened by water erosion (requires only steep land and periodic storms, etc., but man can affect by changing flows of water and (or) sediment)

1) Stream erosion (examples from Pacific Northwest)

2) Wave erosion (Pacific Northwest and California)

B) Ground-water pressure (pore pressure) inducing landslides (Pacific Northwest)

C) Liquifaction (requires saturated unconsolidated deposits, ground shaking)

1) Seismic liquifaction (California, Japan)

2) Surf-tremor liquifaction (Puget Sound area, Washington)

II Land subsidence

A) Depressuring subsidence (requires large decrease in ground-water pressures in a subsurface formation having thick clay) (San Joaquin Valley, California; Las Vegas, Nevada; Houston, Texas; Venice, Italy)

B) Collapse subsidence (requires cavernous calcareous rocks, or manmade caverns/mines, at shallow depths or under large areas of land) (Florida, limestone; Pennsylvania, coal mines)

III <u>Surge waves from landslides</u> (damage can be from direct or reflected waves; more damaging in confined bodies of water)

- A) Surface slides (gravity slides or induced by earthquake)
    - 1) Into lake or reservoir (Vaiont, Italy)
    - 2) Into bay (Lituya Bay, Alaska)
- B) Submarine slides (Valdez, Alaska)

IV Mudflows and nonmeteoric floods (require unusually great amounts of water in unusually short times; most disasterous and fast-moving events start at higher altitudes)

- A) Glacier outbursts (subglacial and englacial water; Mount Rainier, Washington)
- B) Rock and ice avalanches (Mount Huascaran, Peru)
- C) Dam failures (poor construction, seismic shaking, piping or earth failure) Walla Walla, Washington; Van Norman, California; Teton, Idaho)
- D) Volcanic mudflows (lahars) (Mount St. Helens; Mount Rainier, Washington)

V Water-quality hazards (natural only, not pollution)

- A) Natural water, normal conditions (often related to volcanic deposits; Oregon, Idaho)
- B) Natural water, temporary conditions (may be related to geologic event; earthquake effects; Mount St. Helens)
- C) Man-induced changes (salt-water intrusion; Long Island, New York; Long Beach, California)

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POST-FACTO HYDROLOGIC MEASUREMENTS

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INTRODUCTION

In many regions of the world, an unusual geologic or hydrologic event commonly occurs every few years. We have already seen that major geologic events often have major hydrologic consequences. Therefore, anyone who works in hydrology or emergency management can expect to face, perhaps several times in his or her career, the need for hydrologic information and guidance after a major disaster or other natural event.

REASONS AND USES FOR POST-FACTO INFORMATION

There are several possible reasons for hydrologic examinations and data collection after an unusual hydrologic event:

1) to determine whether or where hazardous conditions exist;
2) to determine the extent of the effects of the event;
3) to guide recovery and land-use decisions;
4) to study scientific phenomena;
5) to guide further data collection and scientific studies;
6) to document events of historic significance;
7) to improve predictions of future events.

PREPARATION IS ESSENTIAL TO SUCCESS

The two factors that are most important to a successful program of post-factor data collection (including rapid reconnaissance) are:

1) Planning;
2) preparation;

Most of the essential planning and preparation must be done before the event occurs; the urgency of needs for information and the disruption of normal means of communication and transportation that often follow a major event preclude orderly planning and organization of data-collection efforts after the fact.

The pre-planning should include:

1) Selection of data-collection teams for each type of hydrologic (or hydrologic) event that is likely to occur. The teams should include:

    a) A chief and an assistant chief, who can call the team or teams into action when an event occurs and can start other responses, as needed;
    b) A technical specialist in the particular type of event or hazard that the team is intended for;
    c) Perhaps, a broadly trained hydrologist or other earth scientist.

If the team is all from one agency, then formation, training, and management are much easier. Team make-up may differ for the type of data-collection effort as well as the type of hydrologic event.

2) Interagency coordination, before the event, is essential for all but the least complicated, local, events;

    a) Needed to select, train, and supply interagency data-collection teams;
    b) Needed to agree on agency responsibilities and authority;
    c) Needed to arrange funding;
    d) Needed to reduce duplication of effort or failure to collect essential data;
    e) Needed to help smooth the flow of information;
    f) Requires participation by agency representatives who have decisionmaking authority;
    g) Is complicated by overlap of agency responsibilities or by any interagency jealousy.

h) Interested agencies are scientific (including universities), disaster-relief, emergency-management (usually including the military), mapping, and official information agencies, and local to national political leaders (depending on the scope and impact of the event).

<u>Preparation</u> includes arranging for equipment, supplies, transport, and communication methods, and training of data-collection teams.

1) Equipment and supplies

   a) Essential equipment includes maps (and photographs), notebooks, cameras, <u>much</u> film, and radios.
   b) Equipment should be available for every conceivable need (including emergency needs).
   c) Supplies should be available to allow teams to operate for a few days with no other support, if necessary.

2) Transport may include aircraft, boats, land vehicles, or animals.

   a) Data-collection sites are often in remote areas.
   b) Data-collection teams may be competing with rescue agencies and newsmen for extra transport equipment.
   c) Need <u>prior</u> arrangements for obtaining transport on short notice (contracts, interagency agreements, agents on call).
   d) For rapid reconnaissance, helicopters, airplanes, 4-wheel-drive vehicles, or snowmobiles may be needed (preferred in that order); airplanes may be the best compromise.
   e) Land parties may be started as soon as possible then later directed by radio from aircraft.
   f) Arrangements may be needed for fuel.
   g) Arrangements may be needed for entry permission and (or) police protection of the data-collection team.

3) Training

   a) Team members must know how to use all the equipment, including that of the others.
   b) They need training in survival and first-aid (medical emergency) techniques.
   c) They need training in agency policy and protocol (who to contact under various conditions).
   d) Keep training up to date; train succeeding team members and leaders.

4) Communications

   a) Radios are almost essential for all but the simplest and safest data-colleciton activities; make sure that the correct radio frequencies (channels) are used.
   b) For a complex situation involving more than one field team, a communications center may be needed for dispatching information to teams in the field, and for disseminating information from the teams (helps keep newsmen from hampering field teams; may be best to coordinate this task with other agency or agencies).
   c) A communication center requires at least one person who can translate information from the scientists to the nonscientists, and who can keep the information in the proper perspective (not too sensational or emotional).

TYPES AND SCOPE OF DATA COLLECTION

Post-facto data collection may include rapid reconnaissance, short-term, and (or) longer-term studies:

1) Reconnaissance usually is a desirable first step following all but the most local, simplest events;

   a) is used mainly to detect hazardous conditions, determine extent and scope of problems, and guide further study (reasons 1, 2, and 5, page 1).

b) Time is of the essence; early changes occur quickly (analogy to crime detection).

c) Aerial observation usually is the best method of rapid reconnaissance; helicopter is first choice, but fixed-wing airplanes are more available, cheaper, and usually can be used effectively.

d) Trading information with other reconnaissance groups (such as military) may be fruitful.

2) Short-term data collection:

a) Will probably need to begin soon after the event, but can benefit from the results of reconnaissance.

b) Main reasons usually are to guide recovery and land-use decisions, study scientific phenomena, guide further scientific study, document events of historic significance, and improve predictions of future events (most of the reasons on page 1).

c) Transport might be by either aircraft or land vehicle or water craft.

d) Visit some sites recorded during the reconnaissance, to determine early changes.

e) This may be the only significant study of the entire event, so it should be as good as possible.

3) Long-term data collection:

a) Detection and recording of slower changes, or "recovery" of the environment.

b) Main reasons are research into hydrologic principles and processes, and to improve predictive capabilities (reasons 4 and 6, page 1) but may also guide recovery and land-use decisions (reason 3).

c) Provides opportunity for careful, precise studies, including checking on short-term findings (data from reconnaissance and short-term studies are extremely valuable).

d) Usually involves the installation of data-collection sites for periodic remeasurements.

4) Post-facto hydrologic data collection pertains mainly to surface-water levels, sediment deposition, results of erosion, ground-water conditions, and water quality.

    a) Surface-water levels are needed to determine residual hazards behind impoundments, changes in runoff characteristics, direct measurement of streamflow, and indirect calculation of flood flows.
    b) Data on sediment deposition are needed to determine volumes of impounded water and reduction in channel capacity.
    c) Information on results of erosion is needed to determine changes in channels and floodways.
    d) Ground-water information is needed to assess changes caused by the event (level rises or declines, quality changes).
    e) Water quality (including sediment content and temperature) may be altered by any event that changes rates of runoff or soil conditions in the drainage area, or that results in extensive new deposits, new impoundments, or direct heating or cooling of the water.
    f) Glaciers present special problems and requirements; may be very important but hard to assess, especially in the short term.

DEVELOPMENT OF A DISTRIBUTIVE SYSTEM FOR HANDLING REAL-TIME HYDROLOGIC DATA COLLECTED BY THE U.S. GEOLOGICAL SURVEY

By

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ABSTRACT

The U.S. Geological Survey (USGS) has been collecting hydrologic data using satellite data relay since the early 1970's. Until recently, data transmitted to the satellite from hundreds of automated remote locations have been received, processed, and disseminated from central computers. Although these computers offer advantages such as consolidation of resources and ease of control, the disadvantages due to increased complexity, reduced reliability, and high telephone communications costs have convinced the USGS to develop a system that will permit field offices to receive data directly from the satellite and, following on-site computer processing, distribute the data directly to water data users. Once in place, local USGS offices will, with the exception of the Geostationary Operational Environmental Satellites (GOES), control all phases of the satellite telemetry system from operating remote data collection sensors to providing water data users access to real-time hydrologic data stored in USGS computers.

INTRODUCTION

The USGS is the principal water-resources information agency in the United States, and in cooperation with over 800 other Federal, State, and local agencies, operates an extensive array of hydrologic data collection throughhout the Nation. The data acquired include stage and discharge of streams, lake and reservoir levels, ground water levels, and many indicators of the quality of surface and ground waters.

The data-collection activity has developed since the last part of the 19th Century, and now includes more than 8,000 continuous streamflow stations, 8,000 partial record streamflow stations, 1,000 lake and reservoir stations, 27,000 stations for monitoring ground-water levels, 8,000 surface-water quality stations, and 8,000 ground-water quality stations. The data collection stations are operated by USGS hydrologists and technicians from offices that are located in every State, Puerto Rico, and Guam. Several programs within the Survey provide these offices with technical assistance in the form of computer and data-processing systems, instrumentation support, water-quality laboratory analytical services, and a research program that develops new technologies and methodologies for data collection and analysis.

In recent years, the USGS's instrumentation program has provided technical support and assistance to the field offices enabling installation of a modern remote telemetry capability at approximately 500 continuous streamgaging stations. This automated telemetry capability uses a series of geostationary satellites operated by the National Earth Satellite Data and Information Service (NESDIS) to relay real-time hydrologic data from the streamflow stations to satellite data receiving stations. The USGS's participation in the use of this technology began in the early 1970's with tests of the data collection system on the National Aeronautics and Space Administration's series of Landsat satellites (Paulson, 1974). This activity was followed in the mid to late 1970's with tests of the GOES that are operated by NESDIS (Shope and Paulson, 1981), and tests of a commercial satellite (Forcina and Singh, 1978) for data relay. GOES remote telemetry from USGS streamflow stations are now fully operational and are maintained by USGS field offices in cooperation with other Federal, State, and local agencies that require real-time hydrologic data for fulfillment of their operational missions. The USGS does not usually require real-time data for its interpretive studies or data reports, but the availability of such data to the USGS can be beneficial for automated data collection and for monitoring the status of instruments and data values at hydrologic stations can result in a more efficient scheduling of field personnel for repair of field equipment, collection of water samples and on-site hydrologic measurements.

With the planned integration of new communications and computer technologies into data-collection activities in the 1980's, the USGS will be making a transition from a centralized to a distributive data collection system for acquisition, processing, and providing access to real-time hydrologic data by water data users.

The new computer and communication technologies are beginning to have a profound effect on the way the USGS collects, transmits, stores, and disseminates hydrologic data. This paper provides a description of why and how these technologies are being used to help the USGS move from a centralized system that was developed in the late 1970's and early 1980's, to a more flexible and responsive distributive data collection system that is expected to be in place by the mid to late 1980's. The move to the distributive system will be based on information from tests at several USGS offices. The development and use of the new distributive information systems also holds promise for developing countries that seek to monitor water resources in real-time or collect data from remote locations. This approach, which is based on satellite telemetry and data processing networks, could be used where a conventional communication infrastructure is absent or under-developed.

CENTRALIZED SYSTEM

A system of computers and communications equipment is presently in operation by NESDIS and the USGS in support of real-time satellite data collection. This centralized communications and data processing system was developed from the most cost-effective technologies that were available in the late 1970's. This arrangement conveys hydrologic data from small battery-radios located at remote field stations to USGS field offices and cooperating agencies in real-time, but suffers from problems of reliability and cost. The elements in the system and associated problems are described below.

The USGS began to deploy small battery-powered radios, known as Data Collection Platforms (DCP), at remote sites during the 1970's. These DCPs were interfaced to existing hydrologic sensors and could communicate with one or more satellites. The DCP's were designed with modern electronic technology, and use micro-processors that provide much greater inherent flexibility and reliability when compared to previous instrumentation that rely on electromechanical control. The first DCPs deployed were designed to collect data from the sensor at a fixed frequency, usually every 15 or 30 minutes, accumulate many sets of data, and then at 3-hour intervals, telemeter the data to a GOES satellite as shown in Figure 1.

The National Oceanic and Atmospheric Administration's NESDIS operates the series of GOES satellites that have been placed in geostationary orbits above the Earth's equator. These satellites, which are orbited primarily to image and monitor the Earth's cloud cover and weather systems, can view the entire Earth disc from an altitude o 35,000 kilometers above the equator. As part of an international program of environmental monitoring, the United States maintains operational satellites at 75^{o} and 135^{o} west longitude. Several older satellites, located about midway between these two operational sites, are used for backup in case of a system failure in an operational satellite. Because of their location, these satellites provide an excellent vantage point from which to receive environmental data from DCP's for retransmission to ground receive sites.

Through the 1970's, the USGS relied completely on the NESDIS Command and Data Acquisition facility located at Wallops Island, Virginia, for the reception of data telemetered from USGS hydrologic stations. From the Wallops Island location, NESDIS forwards data from the satellites to a computer located in Camp Springs, Maryland, near Washington, D.C., and distributes the data to users via telephone lines upon request. (Figure 1).

The USGS's Data Relay Project, located at the USGS National Center in Reston, Virginia, provides technical support to USGS offices in the acquisition of real-time data from the NESDIS computer in Camp Springs, Maryland. The Data Relay Project has developed a real-time data computer processing system, known as Hydrecs, that resides on a large general purpose interactive computer located at the National Center. Hydrologic data acquired from NESDIS by a USGS data communications computer are temporarily stored in Hydrecs for interactive retrievals by USGS users and cooperators using asychronous interactive computer terminals. The USGS users can also instruct the Hydrecs system to forward hydrologic data to the USGS's archival data base

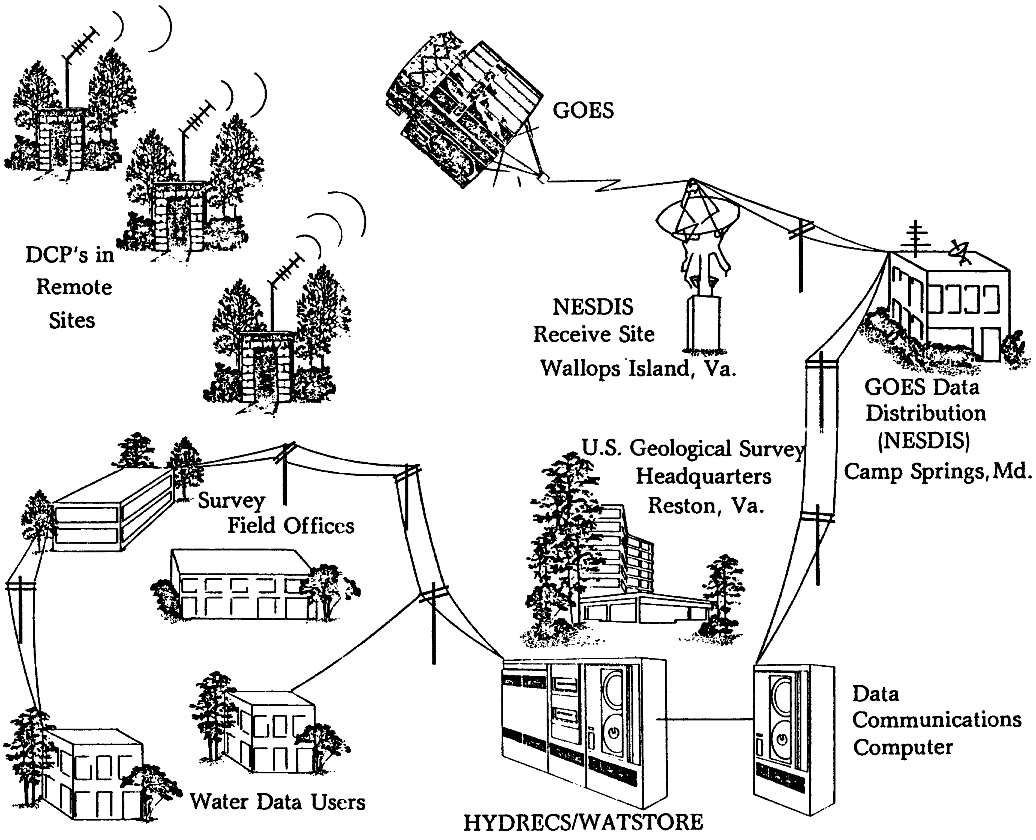
DCP's in
Remote
Sites
GOES
NESDIS
Receive Site
Wallops Island, Va.
GOES Data
Distribution
(NESDIS)
Camp Springs, Md.
U.S. Geological Survey
Headquarters
Reston, Va.
Survey
Field Offices
Water Data Users
HYDRECS/WATSTORE
Data
Communications
Computer

Figure 1

known as the National Water Data Storage and Retrieval System (WATSTORE) (see Showen, 1978) that resides on a large batch computer also located at the National Center. Thus, USGS and cooperator staff, in offices throughout the Nation can employ computer terminals to retrieve real-time hydrologic data for operational purposes, and USGS field personnel can instruct Hydrecs to automatically forward the data to the USGS's National WATSTORE system for nonreal-time report generation, data analysis, and archival. Once the data have been provided to the WATSTORE system, they are available to a wide variety of water data users (also shown on Figure 1), including other Federal, State, and local agencies, consultants, and the public at large.

There are a number of disadvantages and advantages to such a centralized system. The most significant disadvantage to this system is that there is a complex serial system of many computers and communications devices that must operate continuously for data to flow efficiently and reliably to the user. Moreover, elements of this serial system are under the control of different organizations located in different parts of the country and problems of contention, priority, and funding are inevitable. There are terrestrial communication systems operated by regional telephone companies that link Wallops Island, Virginia, Camp Springs, Maryland, Reston, Virginia, and users throughout the Nation. They all must perform reliably for the system to function. In addition, the Hydrecs system operates on a general-purpose computer that supports a wide variety of other applications, and occasionally real-time data processing is interrupted by system failure or through contention for use of the computer by other USGS users. Any single point failure in the complex serial system shown in Figure 1 can often interrupt service to <u>all</u> users of the centralized system. To real-time data users that are a great distance from Reston, the cost of telecommunications from their offices to the Reston computer and the cost of computer use can be quite significant, especially if large numbers of DCPs are in operation. The applications software that is developed for the central computers is complex because it must attempt to serve a wide variety of users who have a wide range of requirements. In many cases specific user requirements cannot be met, which reduces the flexibility and control that any one user may have.

An advantage of the centralized system is that the user of the Hydrecs system need not have expertise in communications or computer operations. Beyond the simple mechanics of dialing up and signing on to the computer, the Hydrecs programs will prompt the user interactively and allow him access to the data with little knowledge of the complexity of data storage and decoding DCP data messages. Furthermore, the user need not invest any significant resources in the development of these computer programs, and is shielded from the necessity of operating Earth receive site stations and computers.

IMPROVED TECHNOLOGY PROGRAM

Beginning in the early 1980's, the USGS began to plan the deplovments of new solid-state, microprocessor-based technology that will have profound impacts on data collection, including a move toward a more decentralized or distributive system for remote telemetry and processing of hydrologic data. Use of the new technology has enabled the USGS to design a new field data acquisition system, deploy and operate direct-readout ground stations for satellite telemetry, and develop a distributive computerized information system. These three systems, which are described below, form the cornerstones for the USGS's real-time data collection network of the 1980's and beyond.

Adaptable Hydrologic Data Acquisition System

During the early 1980's, the USGS began the planning and definition of a new field data acquisition system which will replace much of the outdated data recording and telemetry systems that presently operate out of USGS hydrologic field stations (Paulson, and others, 1982). It will be a microprocessor driven system which will be designed to meet the data collection requirements that are envisioned for the USGS through the remainder of the Twentieth Century. It will provide for onsite storage of data in solid-state electronic memory, or for the remote telemetry of data, either via land line or satellite. This system, which is being cooperatively developed by the USGS and National Weather Service, will provide a more flexible and reliable field system for hydrologic data collection and telemetry. The new field system, which is being designed to interface with the distributive data collection networks through the use of exacting standards, will eliminate the interface problems (electromechanical control, lack of replacement parts, non-standard connections, and others) that plague the present system and will improve the quality of hydrologic data.

Direct-Readout Ground Stations

A Direct-Readout Ground Station (DRGS) is made up of an antenna, radio receive and decoding equipment, and a minicomputer that functions as the system controller. These controllers are powerful enough to control the operation of the radio receiver, decode the DCP messages, flag and disseminate alert messages, store the decoded data in a temporary file, monitor the performance of the DCP transmissions, provide access to the data by multiple users, and forward environmental data to the Distributive Information System minicomputer co-located at the USGS's field offices.

During the early 1980's, the USGS began to process and install satellite DRGS at selected USGS district offices. As opposed to the large, complicated and expensive station operated by NESDIS at Wallops Island, these DRGS's are relatively inexpensive, uncomplicated, and simple to operate. Since these DRGS's do not communicate to the satellite, monitor its health, command it and its sensors, copy imagery data, or position the satellite, they can be designed solely to service the user's requirement for collection of real-time telemetered environmental data. Because the DRGS can now be co-located with the user or data collection network field manager, reliability and responsiveness has been improved, more control has been returned to the field manager, and system flexibility has increased considerably. As a result of these and other factors, the use of these stations has become increasingly attractive to users of the GOES telemetry system.

Distributive Information System

The USGS embarked in the early 1980's on the development of a Distributive Information System (DIS). When fully implemented in 1985, there will be powerful minicomputers in every major USGS office that is involved in hydrologic data collection, analysis and dissemination. These minicomputers will be linked together with a dedicated nationwide communications network.

Each minicomputer will form a node in this network. Software for scientific and administrative applications that now resides on central computers in the USGS's National Headquarters, will be revised and distributed among the nodes of this network. The net result will be that USGS field offices will have powerful onsite capabilities for processing basic data, running hydrologic simulation models, and supporting a variety of administrative and scientific applications. The system of DRGS' that are being deployed in USGS field offices and the new field data acquisition systems will be interfaced with local nodes of the DIS. Real-time hydrologic data acquired by a nearby DRGS will be entered into the DIS and accessed by users at the local, or other nodes within the system. This configuration will provide an excellent opportunity for sharing the resources of a limited number of DRGS's, as well as providing redundant and backup support capabilities for the network of DRGS'. By October of 1984, the USGS expects to have approximately 60 DIS nodes and, 8 DRGS'. By the end of the year, many of the DRGS' will be linked directly to one or more of the nodes.

DISTRIBUTED TELEMETRY

The improved technology that is being developed in the early 1980's will result in a more distributed system for the collection, processing, and distribution of remotely telemetered hydrologic data (see Figure 2).

Under a distributed mode, USGS field offices will continue as with the centralized system to install and operate DCPs at USGS hydrologic stations. The deployment of the new adaptable hydrologic data acquisition systems will have the next major impact on field operations. These new systems will standardize and facilitate the transfer of data via satellite or courier (data retrieved via site visit by field technician) to the DIS network of minicomputers. Under the distributed mode, data retransmitted from the satellite will no longer be acquired from NESDIS, but from local DRGS' that are owned and operated by USGS offices that are directly responsible for the operation of the hydrologic data collection stations.

The processing of data acquired via satellite telemetry in the DIS minicomputer will meet four major requirements. The first requirement will be

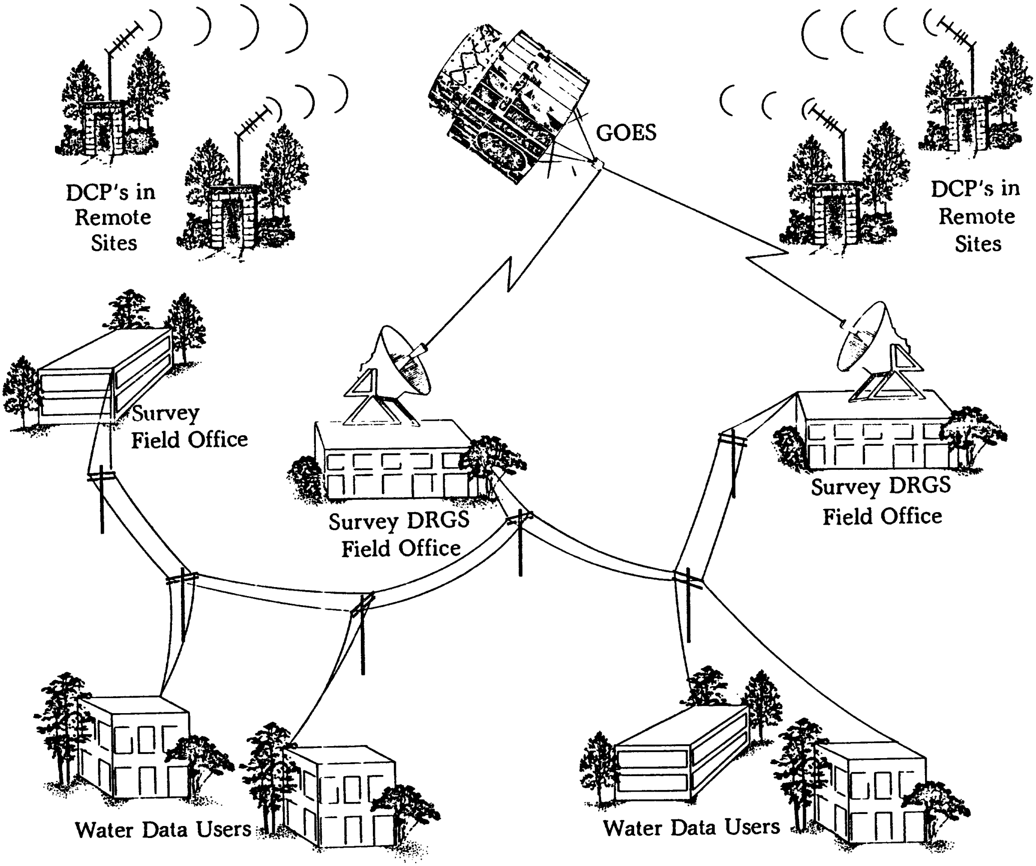
DCP's in Remote Sites
GOES
DCP's in Remote Sites
Survey Field Office
Survey DRGS Field Office
Survey DRGS Field Office
Water Data Users
Water Data Users

Figure 2

to service emergency data alarms. The system will have to service real-time alarms to notify system users that emergency messages have been transmitted from DCPs that have detected extreme hydrologic conditions. The second task will be to attempt to validate the data automatically in real time. There are a number of malfunctions that can occur in the sensor or other links in the system that can cause spurious data to be acquired by the telemetry system. Data validation techniques are being developed to examine absolute values, rates of change, and expected values for hydrologic data from each station, so that the computer can alert the user to data that may require human review and editing. The third requirement will be for report preparation, data analysis, modeling, and long-term storage of the data. An appropriate interface between the real-time data processing system in the minicomputer and the distributed WATSTORE archival system will be defined so that real-time data will be forwarded to the archival system that supports the processing of all USGS hydrologic data. The final major task for the computer will be to support data distribution. The DIS will respond to data requests from water data users that are attached to the minicomputer node servicing the DRGS, as well as users that gain access to the minicomputer from other nodes in the DIS network. A variety of technical and administrative routines will be developed to allow users to access and produce a wide variety of output reports that include both tables and graphs.

The USGS has developed a prototype system to perform most of these functions (not including WATSTORE functions) on a minicomputer that was in place and functioning (April 1983) before most of the DIS minicomputers were installed. This prototype system is located in Phoenix, Arizona, and is linked to the DRGS operated by the USGS's Arizona office. An evaluation of the performance of this system is being used to help define the requirements for further development of the real-time system on the DIS network.

As with the centralized system, there are a set of advantages and disadvantages that exist for the distributed telemetry system. A principal advantage is that the numbers of computers and communications devices that separate the user from the DCP will be drastically reduced. It is axiomatic in such systems that the failure rate exponentially increases with the number of components in the system. Moreover, the components in the distributive system, the receive site and the local computer, are under the control of, and are generally dedicated to, the requirements of the local user and are not contested for by other major users who may have different needs. If the DRGS fails, the user can divert the DIS minicomputer to a backup site at another USGS office. The principal disadvantage will be the capital expense of the computers ($150,000-$250,000) and DRGS ($80,000) and the higher skill level required in the district staffs to operate the computers and DRGS' that are being installed. Fortunately, developments in computer software which will be coordinated from a central office, can shield the user from much of the complexity of data base operation and management.

TECHNOLOGY TRANSFER

The techniques that are being developed by the USGS may be directly applicable to developing countries where the requirements for real-time data exist. Reliable DCPs and relatively inexpensive, simple to operate, and reliable DRGS' can provide a developing country with a communications system that requires none of the normal communications infrastructure that landline or terrestrial radio telemetry systems usually require. Moreover, most developing countries can gain access to geostationary satellites that are operated by the United States, the European Space Agency, and the Japanese Meteorological Agency. Moreover, the experience that the USGS develops with applications software can be shared with other users who seek to use these technologies to meet their requirements.

CONCLUSIONS

As the principal Federal water data collection agency of the United States, the USGS operates data-collection stations in cooperation with a large number of other Federal, State, and local water data users. The operation of this network is influenced by the type and frequency of data that are required by users, and by the technologies that are available to help the USGS meet those requirements cost-effectively and efficiently. Based on the results of prototype tests, new communications and computer technologies are being adopted by the USGS in the early 1980's that will influence the operation of real-time water data collection and processing by the USGS through the end of the century. By the middle of 1984, the USGS will have 8 DRGS' in operation, and by the end of 1984, many of these stations will be linked into the computerized distributive information system.

In addition to the adoption of the new technologies, the movement toward a more decentralized or distributive approach for reception and processing of data will also allow the USGS to more efficiently and effectively provide real-time data for mission requirements of other agencies, and for the monitoring of the USGS's network of automated stations.

Because of the world-wide availability of GOES compatible satellites and the supporting communication equipment and computers, remote telemetry of hydrologic data can also provide developing countries with a viable alternative to other data collection systems (telephone, land-based radio) that cost far more to operate and yet produce no better results.

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HYDROMETEOROLOGICAL FORECASTING

by

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INTRODUCTION

History of Hydrologic Forecasting in the United States

Congressional Resolution H.R. 143 in 1870 established a Weather Bureau with the responsibility of "forecasting the weather, displaying weather and flood signals for the benefit of navigation, commerce and agriculture." A limited attempt was made at river forecasting from Washington, D.C. between 1870 and 1890 but results were less than satisfactory. Poor communications, a lack of data and limited knowledge of hydrologic principles prevented routine beneficial hydrologic forecasts from being made.

In 1893, a change took place so that local Weather Bureau officials were charged with hydrologic responsibilities for their local areas. An attempt was made to provide these officials with whatever data was available concerning past floods in their areas of responsibility. A limited effort was made to install river staff gages. By 1896, the Weather Bureau was making river forecast for 135 locations from 22 Weather Bureau Offices. Hydrologic forecasts began to improve because Weather Bureau staffs began to observe and become familiar with the hydrologic characteristics of rivers and streams within their areas of responsibility.

The next major change in hydrologic forecasting occurred after World War II, in the late 1940's. The Weather Bureau created two offices with the specific purpose of hydrologic forecasting. These offices were located at Cincinnati, Ohio, and Kansas City, Missouri. These offices were called River Forecast Centers (RFC's). The RFC at Cincinnati was charged with forecasting the Ohio River drainage while the RFC at Kansas City had responsibility for the Missouri River drainage and the Upper Mississippi drainage above and including St. Louis, Missouri.

Other RFC's quickly followed. Harrisburg, Pennsylvania, and Hartford, Connecticut, were added in the East. Atlanta, Georgia; Tulsa, Oklahoma; and Ft. Worth, Texas; were added in the South while Portland, Oregon; Sacramento, California; and Salt Lake City, Utah; were added in the West. Anchorage, Alaska; Slidell, Louisiana; and Minneapolis, Minnesota were the last to be added. Thus, thirteen RFC's now exist for the sole purpose of developing hydrologic forecasts for the United States. These thirteen RFC's are staffed by approximately 100 hydrologists and provide a wide variety of hydrologic forecast services.

The RFC's are now undergoing another major change. This is a technological change. There is currently a major revolution in communications, computer and data handling capabilities, and improved hydrologic analysis techniques. The ability to gather, process, and archive data are improving rapidly. Also, the ability to reach the public with timely and accurate forecasts and warnings is also improving. The goal, of course, is to keep loss of life and property damage to a minimum from hydrologic events while, at the same time, making the maximum use of available water resources.

Hydrologic Forecasting Organization Within the National Weather Service

The National Weather Service (NWS) has the hydrologic forecasting responsibility for the Nation. The mission of the NWS hydrologic service program is to save lives, reduce property damage, and contribute to the maximum use of the Nation's water resources. The scope is large, current annual flood losses are estimated at approximately 200 deaths and $2 billion in property damage.

The NWS meets its hydrologic responsibilities through the efforts of the thirteen River Forecast Centers (RFC's) located throughout the United States (Figure 1). Each RFC provides hydrologic guidance and expertise to a State network of both National Weather Service Forecast Offices (WSFO's) and

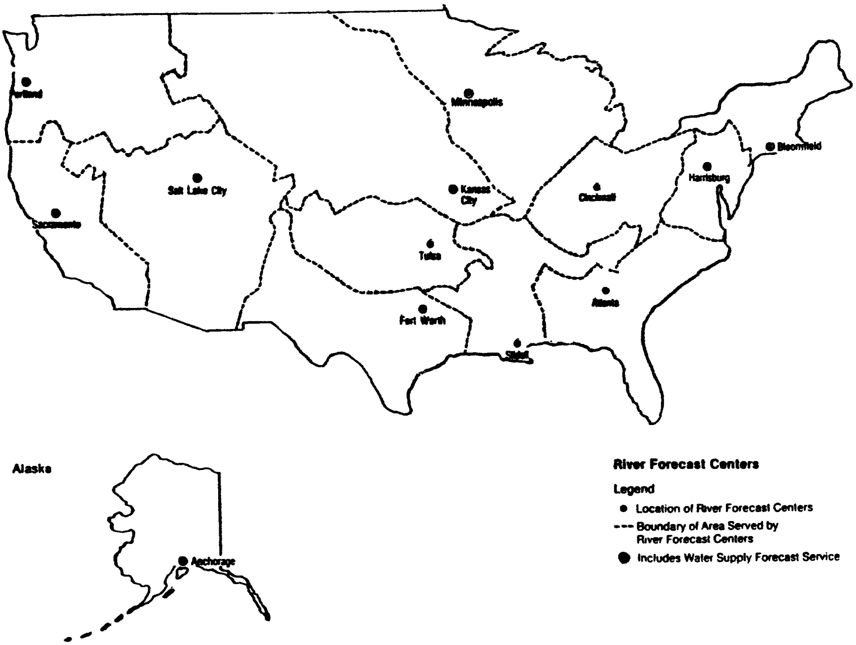


Figure 1.--River Forecast Centers

National Weather Service Offices (WSO's) located within each RFC's area of responsibility. Products generated by the RFC's include flood forecasts, general river forecasts used for navigation and other purposes, reservoir inflow forecasts, water supply outlooks, spring flood outlooks and various types of flash flood guidance.

In addition, RFC's provide a variety of other services. They develop forecast procedures as required and requested, develop and implement new forecast techniques, and remain current in advance hydrologic forecast techniques, computer systems, data handling techniques, and hydrologic related hardware. In addition, the RFC's provide hydrologic expertise on a wide range of hydrologic activities such as dambreak analysis for NWS and other Federal, State, and local agencies. The RFC's also conduct hydrologic training courses and seminars for a wide range of topics and participants.

The RFC boundaries are determined by hydrologic boundaries. The WSFO and WSO boundaries, however, are determined by political boundaries (i.e. State boundaries, county boundaries, etc.). Also, RFC's serve numerous WSFO's. Thus, communication and coordination become a major problem.

Within each State is a primary Weather Service office called a WSFO. The mission of the WSFO is to provide meteorological and hydrological forecasts, watches and warnings to its own State. Within each State, several WSO's, who are each responsible to the WSFO, have similar responsibilities for a smaller area. The WSFO's and WSO's, who are staffed primarily by meteorologists, receive hydrologic guidance from the RFC in whose drainage area they are located.

It is important to note that the NWS takes great care to ensure that hydrologic forecasts come from only one source so as to not confuse the public. The RFC's, therefore, process hydrologic data, generate hydrologic forecasts that are relayed to the appropriate WSFO. The WSFO then releases the forecasts to the public.

The scope of the flooding problem is enormous. About 90 percent of all natural disasters in the United States are flood related. About 30 percent of the population lives on floodplains and about 80,000 people must be evacuated annually because of flooding.

Hydrologic forecasting has wide benefits for many users (Figure 2). The general public is, of course, the primary user. However, other Government Agencies, irrigation interests, power producers, recreation, fish and wildlife, navigation, commerce, etc. all benefit and use hydrologic forecasts. In fact, the benefit/cost ratio for hydrologic forecasts has been established as high as 85:1.

COMMUNICATIONS

Automated Field Operations and Services (AFOS)

The AFOS program is designed to replace manual, semiautomatic equipment and paperwork operations. AFOS is an automated, engineered system providing electronic data handling and graphic display capabilities by means of on-site computers, linked in a nationwide network.

The system increases the timeliness and quality of National Weather Service warning and forecast service. It upgrades the productivity and effectiveness of all operations. Data are displayed on video screens rather than on paper. This eliminates the time consuming process of handling and filing huge volumes of messages previously received on teletype.

The system accomplishes vital improvements in communication. Forecasts arrive on the display consoles in seconds instead of minutes; messages arrive at a rate of 3,000 words per minute instead of 100 words per minute under teletype systems; nationwide, the system is capable of carrying, in less than three hours, the same information that is now carried in 24 hours.

Freed from much of the monitoring, manual data retrieval and raw message composition, hydrologists and meteorologists can provide greatly expanded, faster forecasting and reporting. In severe flooding situations the forecaster can issue warnings sooner, resulting in fewer losses to life and property.

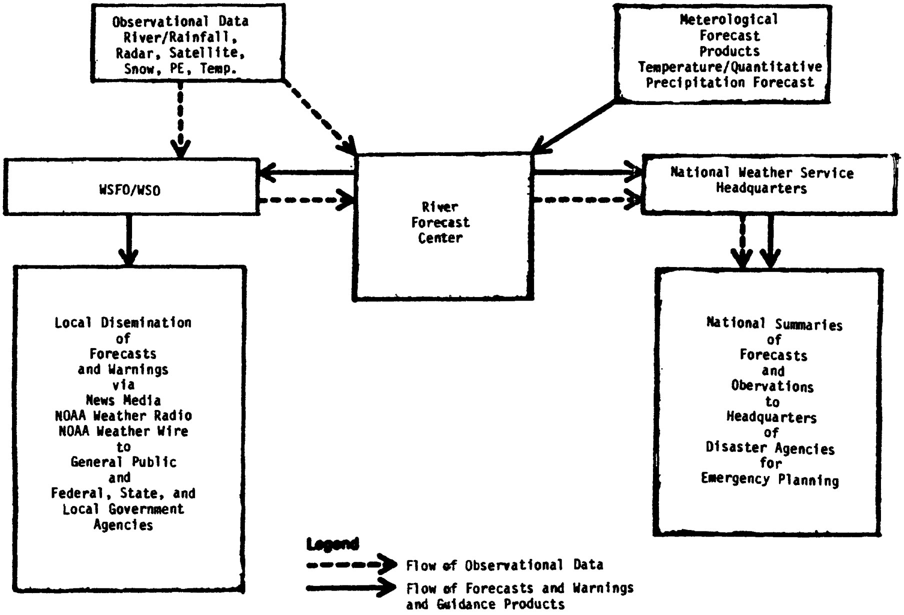


Figure 2.--Hydrologic Forecast System

Using a sophisticated computer system and communications capabilities, data from any point within the distribution circuits are available on a store-and-forward, instant replay basis. Data can be retrieved by alphanumeric and graphic display form on display consoles. If the data requested are not stored in the local system, AFOS can issue requests over the National Distribution Circuit. Within about one minute, the requested data are received. The hydrologist can retain data in storage or purge obsolete data (Figure 3).

Each NWS field office is provided with processors and displays suited to its requirements. Basic meteorological and hydrological data, analyses, forecasts, and warnings are distributed among NWS offices over AFOS. The backbone of the system are the National Distribution Circuits which connect WSFO's and National Centers in 4 regional loops. Radiating from each WSFO are State distribution circuits to the RFC's, WSO's, and WSMO's within a WSFO's area of responsibility.

At offices, alphanumeric (text) and graphic information are presented on TV type displays. These are incorporated into a variety of consoles where the number of displays ranges from one to four. Operational requirements dictate the type and number of consoles provided at each office. The average WSFO has five consoles; RFC's, four; and most WSO's, two.

GOES DATA DISTRIBUTION SYSTEM (GDDS)

The National Environmental Satellite Service (NESS) operates the GOES system for the purpose of collecting a wide range of environmental data. The data are transmitted from data collection platforms (DCP's) on Earth to one of two GOES satellites which, in turn, relay the data back to downlinks on Earth.

About 50 different agencies are associated with the GOES system. About 2500 DCP's are currently functioning throughout the United States. The Federal Government has about 80 percent of the total platforms. Within the Federal Government, Department of Interior is the largest user with about 38 percent of the platforms. Department of Commerce is next with about 18 percent of the total.

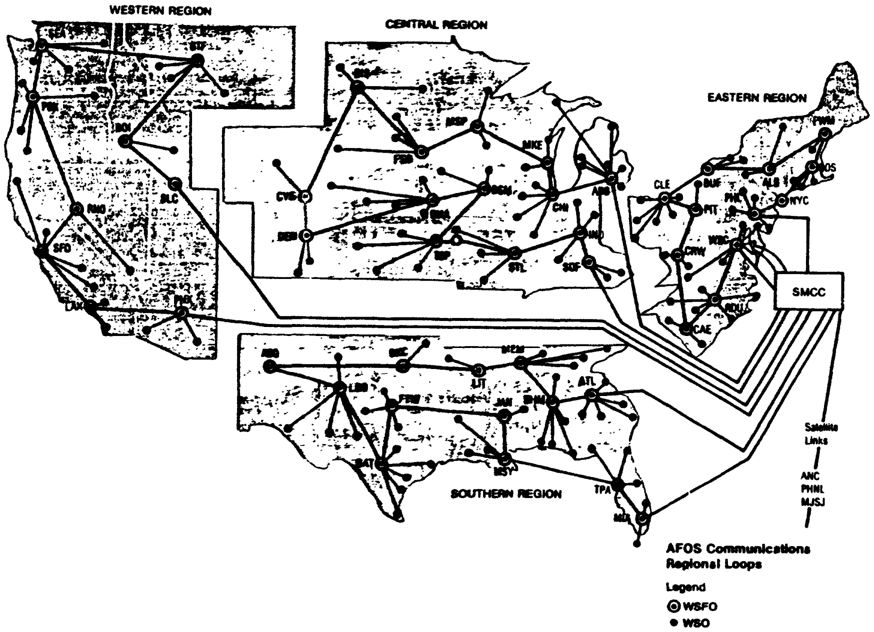


Figure 3. AFOS

Typical parameters available are precipitation, river levels, temperature, wind speed and direction, relative humidity, soil temperatures and radiation. Platforms can be self-timed or interrogated. Within the National Weather Service, data from GOES are available on AFOS to all forecast offices.

Centralized Automated Data Acquisition System (CADAS)

The reporting network has some precipitation and river gages equipped with BDT's (Binary Decimal Transmitter), AHOS (Automated Hydrologic Observing System) and with DARDC's (Device for Automatic Remote Data Collection). The BDT, AHOS and DARDC permit the gage to be interrogated by telephone. CADAS is a system where most of such gages in the United States are interrogated over land lines by a central computer. The data are then placed in AFOS for use by the appropriate forecast office.

DATA

Cooperative Networks

The development and maintenance of a reporting data network is the most important element required to support hydrologic forecasting. Of primary importance are precipitation and river stage data. At times, snow depth, water equivalent and temperature data are vital to generating a river forecast.

Most of the NWS hydrologic network is composed of public-service oriented private citizens who volunteer their time and energies. Generally, they report river and rainfall data to a local Weather Service office on a predetermined schedule, such as 7 a.m. When established criteria are met, however, these observers generally report every six hours. These data are then relayed to the appropriate RFC for analysis and use in the preparation of the hydrologic forecast. The reporting network consists of approximately 6,000 observers nationwide.

The World Meteorological Organizations "Guide to Hydrologic Practices" recommends, as a minimum network for a forecast program one precipitation gage per 600 square kilometers. The current reporting network in the National Weather Service is one gage per 1,200 square kilometers, about half the required density.

Since most of the hydrologic reporting network is volunteer observers, the efficiency is about 30 percent. Thus, the effective network density is about one gage per 3,900 square kilomters. It is clear that high network density and reporting efficiency are vital to a hydrologic forecasting program.

Other types of data are required to make hydrologic forecasts. Reservoir data (e.g. releases and pool elevations), satellite observations, radar, Quantitative Precipitation Forecasts, etc. are all necessary data to fully comprehend a hydrologic situation.

Agency Cooperation

Many Federal, State, and local agencies are involved in water resource activities in the United States. Because of this, cooperation and coordination is essential among all involved in water resource activities, especially in data collection.

At the national level, the Corps of Engineers, Bureau of Reclamation, National Weather Service, U.S. Geological Survey, and Soil Conservation Service all have broad but fairly well defined mandates in water resource activities.

The National Weather Service, from a river and flood forecasting viewpoint, must interface with other Federal Agencies. The Corps of Engineers and Bureau of Reclamation, in particular, are close cooperators and fund a large part of the hydrologic reporting network on a reimbursable basis. These two agencies also operate large dams which are a vital part of the forecast equation.

Automated Local Evaluation in Real Time (ALERT)

The ALERT system consists of event reporting river and precipitation gages which report by radio to a microcomputer base station. The systems are intended, primarily, for flash flood situations where quick action is of the utmost importance. The data generated, however, is of wide use in the total hydrologic forecast program.

These systems are relatively new and fairly expensive. In general, they are installed in high risk areas and in cooperation with local users.

Integrated Flood Observing and Warning Systems (IFLOWS)

IFLOWS is a system older but similar to ALERT. It was developed as a demonstration project in central Appalachia. Originally, the project involved the States of Kentucky, West Virginia, and Virginia. Subsequently, IFLOWS has expanded into parts of Pennsylvania and Tennessee.

IFLOWS was developed as a Federal initiative in cooperation with State Governments. ALERT was a cooperative effort between private industry and the Federal Government.

Again, IFLOWS is event reporting precipitation gages reporting by radio to microcomputer base stations. The equipment and software, while conceptually similar to ALERT, was developed entirely independently. The data, as in the ALERT system, is primarily intended for flash flood situations; however, it is important to the total forecast program.

Remote Observing System Automation (ROSA)

As was mentioned earlier, a large portion of the reporting network is not automated. Most hydrologic data gets to a Weather Service office by an observer calling in by telephone and a staff member answering the telephone and recording the data. The data is then manually entered into AFOS.

ROSA is an experimental program being conducted to automate the observer network. Some observers are being equipped with encoder pads. Instead of calling a Weather Service office and talking to a staff member, the observer dials a microcomputer and, after encoding the data, releases it to the microcomputer. The microcomputer then reformats the data and sends it to the appropriate Weather Service office by AFOS.

This system is currently being evaluated. Should it prove successful, it will give the NWS an opportunity to vastly improve the majority of the network which supports hydrologic forecasting.

Airborne Snow Survey Program (ASSP)

A procedure for remote aerial monitoring of snow is now being used to help develop low cost and more timely estimates of basin snow cover water equivalents. This is accomplished by having an aircraft, with the capability of monitoring natural terrestrial gamma radiation, fly preselected flight lines. The existence of water in the snow cover and in the soil below reduces the amount of gamma flux reaching the airborne detector. By comparing snow and no-snow measurements, an estimate of snow cover water equivalent can be made. This technique provides areal measurements as opposed to point measurements, and provides an additional evaluation of flood potential from melting snows (Figure 4).

Geostationary Observing Environmental Satellite (GOES)

In September of 1961, the United States established a meteorological/hydrological satellite system. GOES operated by the National Oceanic and Atmospheric Administration (NOAA) are a major component of that system. The current NOAA system consists of two operational GOES satellites. GOES-East is located at 75 W and GOES-West is positioned at 135 W.

GOES satellites provide frequent Visible (VIS) and Infrared (IR) images to monitor weather events such as hurricanes and other severe storms and to relay meteorological/hydrologic observation data from surface collection points to a processing center. Data from GOES are also proving to be of

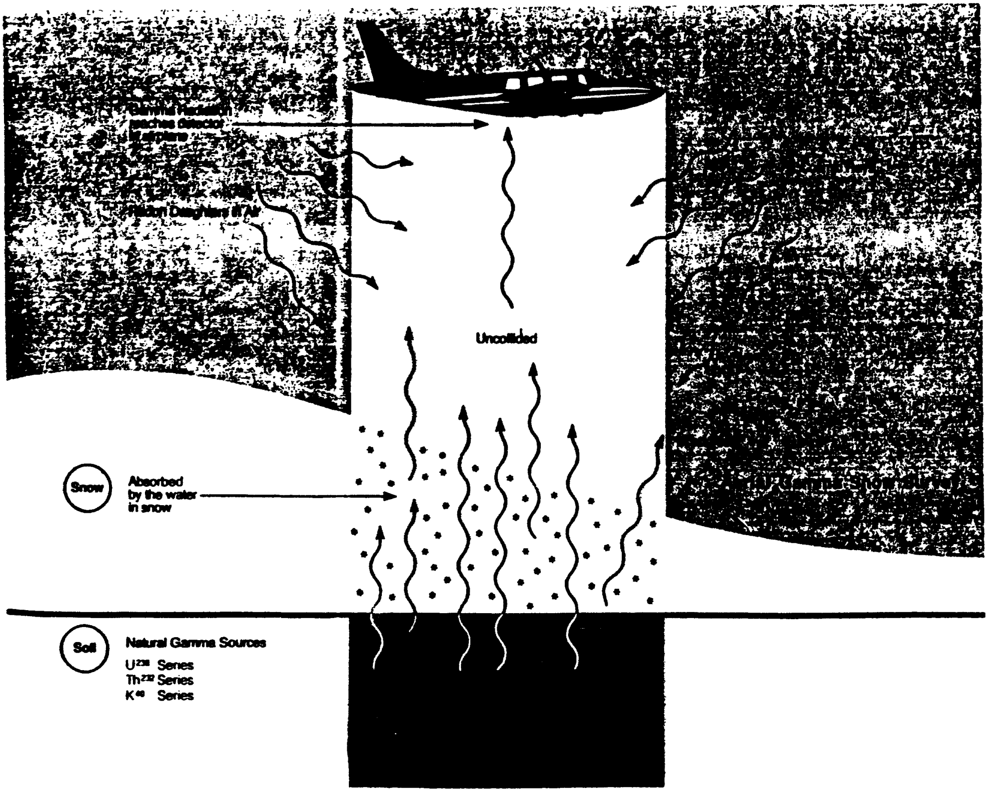


Figure 4: Aerial Gamma Snow Survey

increasing value for other environmental applications such as surface snow mapping.

Onboard the GOES spacecraft, the environmental sensor is a Visible and Infrared Spin Scan Radiometer (VISSR) Atmospheric Sounder (VAS). In its operational mode, VAS scans the Earth and gathers data in the reflected visible light and the thermal IR portion of the spectrum. VAS can also gather IR radiation in any of several spectral bands and produce images in multiple IR spectral bands. This mode is called multispectral imaging (MSI). VAS also has an atmospheric temperature sounding capability for gathering IR radiation data.

FORECASTING MODELS

Antecedent Precipitation Index (API)

The flow of a river is quite irregular when considered over long time periods. It is characterized by rises from rainfall and snowmelt followed by gradually receding flow when neither of these two contributing factors are present. The most elementary river forecasts are concerned with predicting the time and height of stages caused by peak flows. Other forecasts are made for the increasing flows before peaks and for the following recessions. Water supply forecasts attempt to predict the total flow for a season as early as possible during that year. During recent years, the trend has been toward more and more detailed forecasts with continuous flow forecasting required by more and more users.

There are three major steps to forecasting the rise that results from a rainstorm or period of heavy snowmelt:

1. Making an accurate estimate of the volume of water that will run directly off the land surface into the stream. This step utilizes the rainfall runoff relation or snowmelt forecast.

2. Forecasting the distribution of this volume of water with time as it passes a forecast point. This is known as forecasting the

hydrograph. The unitgraph is the tool most commonly used for this purpose.

3. Forecasting the change in shape of the hydrograph as the volume of water moves to points farther downstream. This is known as flow routing.

The quantity of runoff from a given storm is dependent on moisture conditions in the basin prior to the storm, characteristics of the storm itself and, to some extent, season of the year.

A common procedure for determining runoff is based on the assumption that soil moisture decreases logarithmically with time during periods of no precipitation, where I0 is the initial value of the antecedent precipitation index (API) and It is the API value some t days later.

$$I_t = I_0k^t$$

If t equals unity then,

$$I_1 = I_0k$$

The recession factor k can vary from .85 to .98 but is commonly used as .90.

A rainfall runoff relation using the API concept is commonly presented in the form of a coaxial relationship. The basin recharge is considered to be the difference between rainfall and runoff. By plotting API against observed recharge for many storms and labeling each point by week of the year, quadrant 1 of the coaxial relationship can be developed. Using basin recharge and the results from quadrant 1, quadrant 2, and quadrant 3 can be developed (Figure 5).

Using API's and rainfall-runoff relationships, runoff can be calculated. Multiplying runoff times the unit hydrograph determines the forecast hydrograph for the location in question. Unit hydrograph theory assults the hydrographs for given locations reflect the characteristics (e.g. shape, size, slope, etc.) of that unique basin and that similar storms in the basin will produce similar hydrographs.

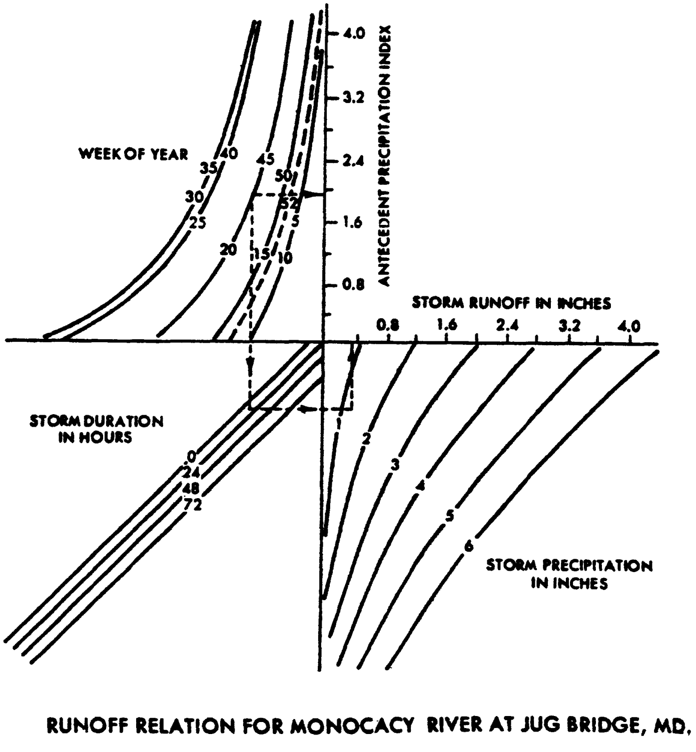


Figure 5. Runoff Relation for Monocacy River at Jug Bridge, Maryland

Soil Moisture Accounting (SMA)

The development of rainfall-runoff relationships has evolved from simple variable runoff graphs to rather complex coaxial graphical relationships. The last few years have seen the development of conceptual hydrologic models where an attempt is made to describe the mechanics of soil moisture flow. The Sacramento Model is typical of these conceptual models.

The goal of the conceptual soil moisture accounting model is to logically distribute available moisture through the soil mantle, have reasonable percolation characteristics and effectively simulate streamflow. The Sacramento SMA Model breaks the soil mantle into upper and lower zones, each with tension and free water components. The capacity of the zones and percolation characteristics between zones are determined by parameters. In all, about twenty parameters control the direct runoff, surface runoff, interflow and base flow.

The utilization of a conceptual model involves some form of calibration on historical rainfall, temperature and streamflow. By varying the values of the model parameters over successive computer runs, an attempt is made to have the model streamflow output simulate observed historical streamflow. The assumption is then made that if the conceptual model can simulate historical hydrographs with the associated precipitation, then the same models and parameter values can be used to forecast future hydrographs based on current precipitation.

The calibration of a conceptual model can be time consuming. Estimates of mean basin precipitation and temperatures must be generated for the calibration period, generally 5 to 10 years. In most cases, the seasonal runoff will also involve snowmelt which requires a separate snow accumulation and ablation model. Once the historical data is prepared for use by the conceptual model, successive computer runs are made. Generally a statistical package within the model computes statistics comparing daily observed with simulated daily streamflow (Figure 6). By manually varying the model parameters to improve statistical output such as simulated means, correlation

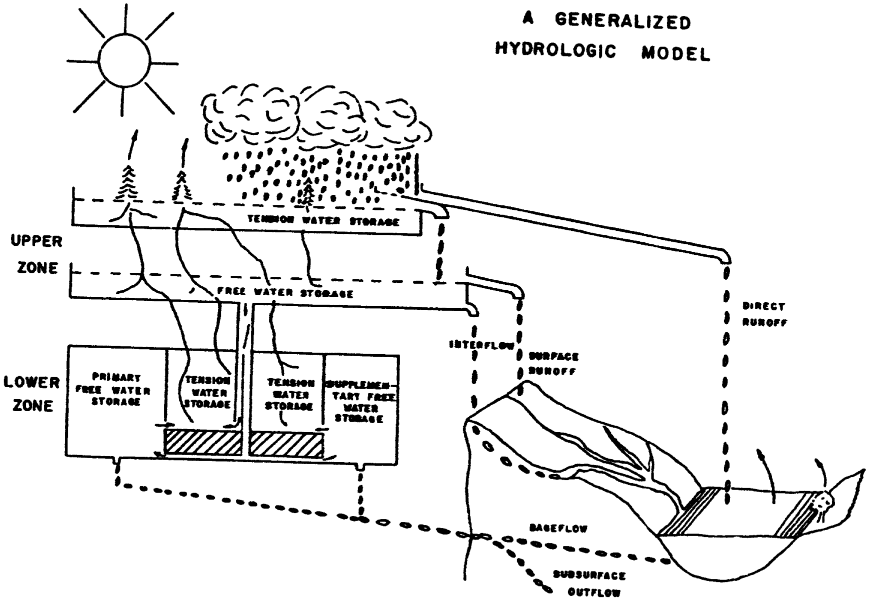


Figure 6.--A Generalized Hydrologic Model

coefficients, bias, etc. the model is slowly "calibrated" over successive runs. The final step in most calibration procedures is some form of automated optimization technique to fine tune the parameter values using an automated technique such as pattern search (Figure 7).

Runoff then is the result of processing precipitation through the mathematical algorithm which represents the soil mantle. The channel routing function can then be determined using standard unit hydrograph techniques. The unit hydrograph can be used to develop a time delay histogram which provides additional options for varying the shape of the routing functions.

Regression Models

Regression type models for hydrological forecasting are most suitable for seasonal volume snowmelt forecasts. In the National Weather Service, regression analysis is used to generate Water Supply Outlooks.

Water Supply Outlooks are seasonal volume forecasts, generally April to September. The forecasts are volumes of flow, generally in acre-feet, which are expected to flow by a given point during a snowmelt season. The users are generally irrigators who need this information for agricultural purposes or reservoir operators who must adjust reservoir storages based on anticipated snowmelt runoff. The multiple regression equations are developed based on statistical relationships between high mountain snowcover, lower elevation monthly precipitation totals and flow parameters reflecting carryover basin moisture conditions.

With this type of forecasting it is possible to include a range of future possible events and determine that impact on anticipated runoff. The NWS and Soil Conservation Service issue Water Supply Outlooks as Most Probable, Reasonable Maximum and Reasonable Minimum. Most Probable is the runoff expected under current conditions and median future precipitation during the remainder of the runoff season. Reasonable Maximum is that runoff expected if subsequent precipitation is exceeded only once in ten years. Reasonable Minimum is that runoff expected if subsequent precipitation is exceeded in nine out of ten years.

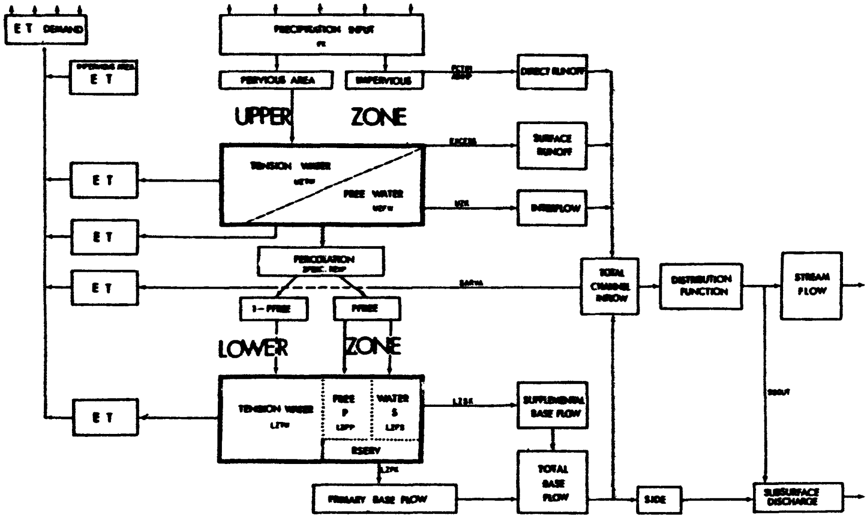


Figure 7.--NWSRFS Catchment Model (Sacramento)

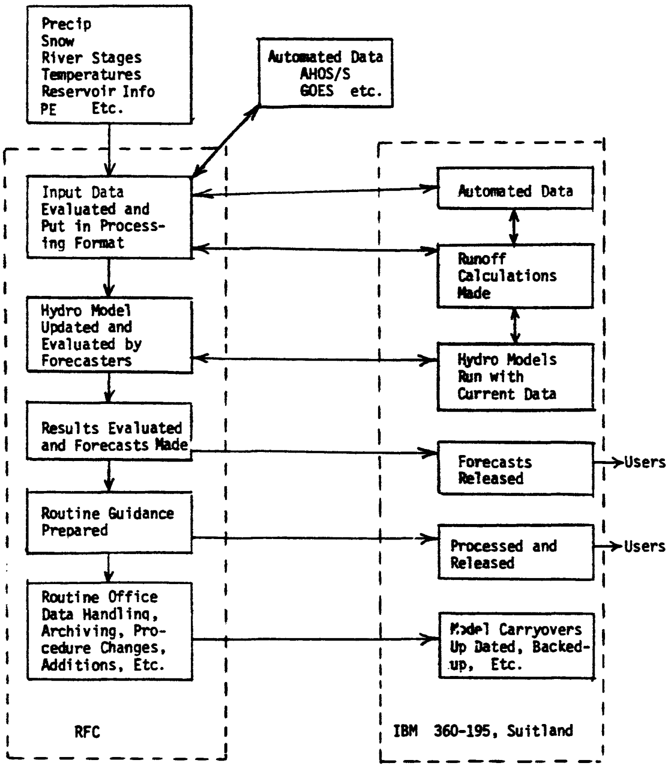
REAL TIME FORECASTING

General Procedures

Hydrologists at the River Forecast Centers analyze and process a wide range of hydrologic data and prepare river, flood, and reservoir forecasts as well as hydrologic outlooks for public distribution. The forecast procedures are based on historical data and account for current hydrologic, hydraulic, and meteorologic factors. The RFC's generate hydrologic forecasts for approximately 1,600 forecast points nationwide and rely on data from a network of 1,800 river gages and 4,000 rainfall stations which represent a large portion of the supporting ground data collection network.

After the necessary hydrologic and meteorologic data are gathered and processed, the river and flood forecast development begins with the determination of available runoff. In the Missouri and Upper Mississippi River basins, the temperature based snow accumulation and ablation model determine the quantity of water from snowmelt and/or rainfall available for runoff. Outflow from the snow pack is then processed to determine runoff using either a soil moisture accounting model or an antecedent precipitation index (API) model. Approximately 700 unit hydrographs are used by the Missouri Basin RFC to generate streamflow hydrographs from calculated runoff. Downstream hydrographs are developed by several routing procedures. These routing procedures account for the change in the shape and timing of a wave as it moves downstream in an ever-changing channel. U.S. Geological Survey rating curves are used to convert forecast discharge to forecast stage (Figure 8).

Spring Flood Outlooks, issued for the upper Midwest, are developed at the RFC's in much the same fashion as river forecasts. The outlooks, however, assume temperature and precipitation patterns for the outlook period. Once the current hydrologic and meteorologic conditions are determined from reported data, predetermined temperature and precipitation patterns are applied to the existing snow cover. A simulation is generated sufficiently far into the future to melt the existing snow cover and to route the resulting hydrographs downstream.



RIVER FORECASTING TECHNIQUE

Figure 8. River Forecasting Technique

The forecasting process begins at the observer level. Observers telephone data into Weather Service Offices early each day. At the same time, automated data is being gathered from a variety of sources. All of this hydrologic data, primarily precipitation, river stages, reservoir data and snow cover data, makes its way to the RFC's, generally over AFOS. By about 9 a.m. each day, the RFC's are in a position to begin hydrologic forecasting.

The processing of the data is accomplished on a large main frame computer. Processing programs calculate mean basin precipitation based on received precipitation reports. Other programs take the basin precipitation and either through API or SMA models, runoff is calculated. Other programs take the calculated runoff, multiply by the appropriate unitgraphs to develop upstream hydrographs. Downstream hydrographs are developed using routing procedures.

A critical point is that throughout the process hydrologists using their experience and expertise monitor all the procedures. Data and results are continually observed for reasonableness. The hydrologists have the ability to intercede in the process to update or modify the models so as to more closely reflect the real world events.

Rainfall/Runoff Relations

The most common rainfall/runoff relationship currently in use in operational forecasting are the API relationships. The API process was discussed previously in the section under Forecasting Models.

As an example of application, the Missouri Basin, with a drainage area of 525,000 square miles has about 20 separate rainfall/runoff relationships. These different relationships reflect different hydrologic regimes within the basin.

The Missouri Basin, for forecast purposes, is divided into 710 runoff zones. Each day, the API value is recomputed based on the recession factor k, which also varies within the basin, and on reported precipitation values.

Thus, runoff within the Missouri Basin in not based on one or a few relationships but on separate daily updated API values applied to one or more of the separate rainfall/runoff relationships over all runoff zones.

Generalized River Forecast Program (RIVALL)

A generalized computer program for hydrologic forecasting, regardless of the type of runoff analysis or routing techniques utilized, becomes quite complex very quickly. It is also the type of software which must evolve, change, and meet current needs of the forecasters and the product users. The program in use in the Missouri Basin River Forecast Center of the NWS is called RIVALL.

A river flows from higher to lower elevation with time through a watershed. In RIVALL, the time scale has 24 equal time periods of 3, 6, 12, or 24 hours each, chosen according to the watershed peaking time and the forecast users needs. The time periods of the first data are reserved for observed input, while the remaining time periods provide forecast flows. For example, a 6-hour time interval permits a five day forecast.

As a river enters any one watershed it is augmented by ground water discharge and some calculated runoff portion of rainfall or snowmelt volume within that watershed. It arrives at the gaged exit of that watershed at some later time. A rather consistent arrival pattern is discerned from historical anaysis in which the upstream, base, and runoff portions are separated by using watershed model concepts, assigned to one of the 24 time periods, and then reconstituted to produce the flow at the gaged exit.

The procedure is repeated for the next downstream watershed by using all 24 flows at that previous watershed gaged exit. Flows prior to the first time period are also used to complete the needed series of inflows. Tributaries having multiple watersheds must be added as they converge on the main stream, either computed with today's input or updated from yesterday's final analysis and carried over with watershed base flows and runoff flows. Detailed hydrologic instructions and conversion tables control sequencing of the generalized instructions in a computer program. The model's forecast for each gaged exit is compared with actual reports for each time period.

If the calculated watershed runoff from the rainfall volume is correct, each observed flow since yesterday morning will agree with the computed sum for the same time period. The forecaster assumes the model input will predict the future flows as well as it has predicted the observed flows. When a calculated runoff fails to predict the observed flows, the hydrologist alters the runoff to match observed flows.

It is not practical to examine each of many watershed runoff computer calculations for accuracy before proceeding downstream. So, flows based on reported stages are projected into the future by merging them toward the computed flows over several time periods. These merged future flows are used in the next watershed downstream to avoid delay.

All components of the computed flows must be presented to let the hydrologist determine whether input needs revision. No simple formula is known for adjusting complex hydrologic interrelationships, so trials are made for alternate timing of runoff and for subsequent or supplementary input, such as:

- Receipt of delayed rainfall and stage reports.
- Correction of erroneous stage reports.
- Correction of transcription and punching errors.
- Predicted rainfall in flood threatened watersheds.
- Predicted temperatures in snow-covered watersheds.
- New reservoir release plans due to new forecast inflow or downstream flood.

Each new input and the resulting merged forecasts will eventually affect all gages and basins downstream. As time permits, new trials must be made before forecasts are issued.

The basic tabular and graphic output are the present and future discharge at a stream or lake gaging point and at the tailwater gage of a reservoir, derived river gage heights and reservoir elevations and contents, and the routed component from upstream observations or computations, including adjusted tributary discharges. The concise output is related to input by printing stages, flows, rain, runoff, measurements, and revisions to carried over data.

Four symbols plot three hydrographs on the same diagram with variable scale to show the observed or adjusted, the computed, and the upstream routed flows. A comparison with the plot of flood stage and flow often aids forecasters.

SPECIALIZED FORECAST SITUATIONS

Snowmelt

A typical continuous snow accumulation and ablation model is a conceptual computer model which represents separately each significant physical process within the snow cover. The snow model simulates the snow pack accumulation, heat exchange at the airsnow interface, areal extent of snow cover, heat storage within the snow pack, liquid water retention and transmission, and heat exchange at the ground-snow interface. Input consists of mean areal air temperature and precipitation data (Figure 9).

The accumulation of the snow cover is based on precipitation and air temperature data. If the air temperature is below $0^{o}C$ (or any temperature selected by the user), the model accumulates snow during precipitation events. If the temperature is equal to or greater than $0^{o}C$, precipitation enters the snow pack as rain. Heat exchange at the air-snow interface is the most critical factor in snow pack ablation. The model assumes melt can occur only if the air temperature is greater than $0^{o}C$. If melt and rain occur simultaneously, the model accounts for net radiative heat transfer, latent heat transfer, sensible heat transfer, and heat transfer by rainwater. If melt occurs without rain, then melt is based on a seasonally varying melt factor (Figure 10).

During cold periods of nonmelt, the snow cover loses heat. This negative heat flow is determined by an antecedent temperature index and a negative melt factor. The negative heat factor varies seasonally and represents a varying

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RULO NE   MISSOURI R                                                           2/DAY STAGE #  57    1
FS   17.0  101279.                        2612 SHIFT TO    37300 CFS MEASURED AT       9.4 FT.
MAX STAGE OF RECORD:   25.6  04/22/52
                       FT   8.6       9.1       9.6       9.9      10.2      10.5      10.9      11.1
57 STAGE  2/DAY    DATE    34000     36000     38000     40000     42000     44000     46000     48000
    9.2   36650 CFS  2 6 AM  I         I= 0      I         I         I         I         I         I
    9.7   38628 CFS  2 6 PM  I         I         I = *     I         I         I         I         I
   10.1   41427 CFS  3 6 AM  I         I         I         I    = *  I         I         I         I
   10.4   43203 CFS  3 6 PM  I         I         I         I         I   = *   I         I         I
   10.5   43494 CFS  4 6 AM  I         I         I         I         I    = *  I         I         I
   10.4   43074 CFS  4 6 PM  I         I         I         I         I  = *    I         I         I
   10.3   42270 CFS  5 6 AM  I         I         I         I        =I*        I         I         I
   10.1   41029 CFS  5 6 PM  I         I         I         I  = *    I         I         I         I
    9.8   39620 CFS  6 6 AM  I         I         I     = * I         I         I         I         I
    9.6   38271 CFS  6 6 PM  I         I        =I*        I         I         I         I         I
    9.4   37218 CFS  7 6 AM  I         I   = *   I         I         I         I         I         I
    9.2   36505 CFS  7 6 PM  I         =  *      I         I         I         I         I         I
    9.1   36085 CFS  8 6 AM  I       = *         I         I         I         I         I         I
    9.0   35638 CFS  8 6 PM  I      = *I         I         I         I         I         I         I
    9.0   35657 CFS  9 6 AM  I     = * I         I         I         I         I         I         I
    9.0   35512 CFS  9 6 PM  I    =  * I         I         I         I         I         I         I
    8.9   35403 CFS 10 6 AM  I    = *  I         I         I         I         I         I         I
    8.9   35331 CFS 10 6 PM  I   =  *  I         I         I         I         I         I         I
    8.9   35281 CFS 11 6 AM  I   = *   I         I         I         I         I         I         I
    8.9   35238 CFS 11 6 PM  I   = *   I         I         I         I         I         I         I
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    8.9   35089 CFS 13 6 AM  I  = *    I         I         I         I         I         I         I
    8.8   35037 CFS 13 6 PM  I  = *    I         I         I         I         I         I         I
    8.8   34989 CFS 14 6 AM  I =  *    I         I         I         I         I         I         I
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Figure 9.--Rivall Forecast Hydrograph Display

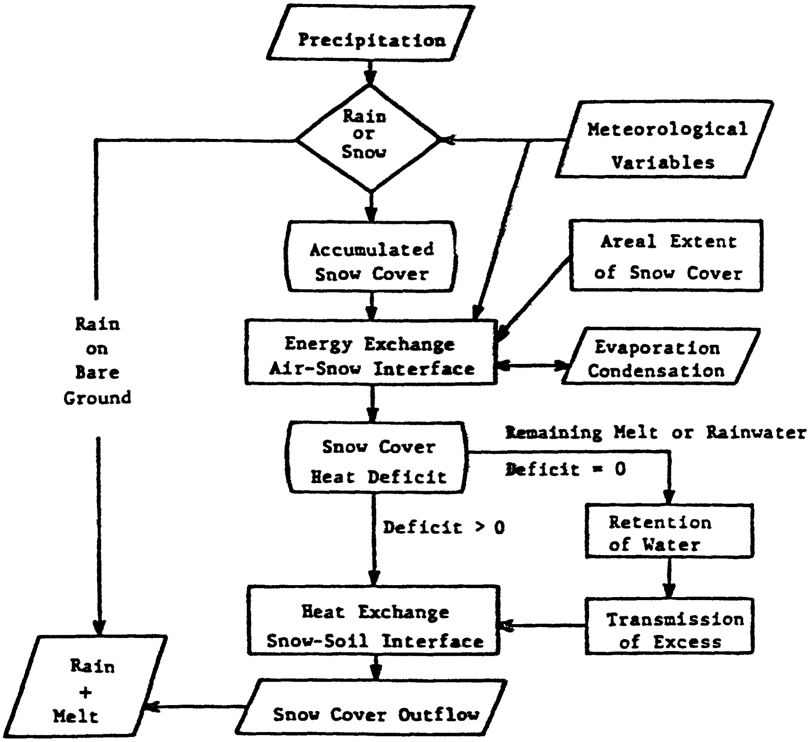


Figure 10.--Flowchart of typical snow accumulation and ablation model

conduction of heat based on changing snow density. The change in heat storage in the snow pack during nonmelt periods is represented as a depth of water equivalent. Also, included in the model is a constant melt rate at the ground-snow interface which represents the geothermal heat flux. The areal extent of snow cover is used to determine the area over which heat exchange is taking place and the areal extent of rain which falls on bare ground. Areal depletion curves of snow cover for a given area have similarities from year to year and are used to relate snow water equivalent to areal extent of snow cover. The snow model maintains a continuous accounting of heat storage of the snow pack. When the pack is isothermal and the liquid water capacity is satisfied, excess liquid water is used as input to API or soil moisture accounting procedures which in turn generate runoff. Prior to exiting the snow pack, the excess water is lagged and attenuated through the pack before being input to a runoff model.

The models are a simplified but effective index approach to determining runoff. Soil moisture accounting procedures, however, have been developed to simulate the process of moisture movement within the soil which contribute to stream flow generation. Rain and snow melt generated by the snow model are input to the soil moisture accounting model and distributed to various conceptual compartments using rational percolation characteristics. In many locations, the permeability of the upper layer of soil can have an important influence on spring snow melt flooding. The most important snow-soil interaction to stream flow simulation is the effect of frozen ground on snow cover runoff. These effects can be quite large where extensive areas of concrete frost have developed over a basin during the winter.

Flash Floods

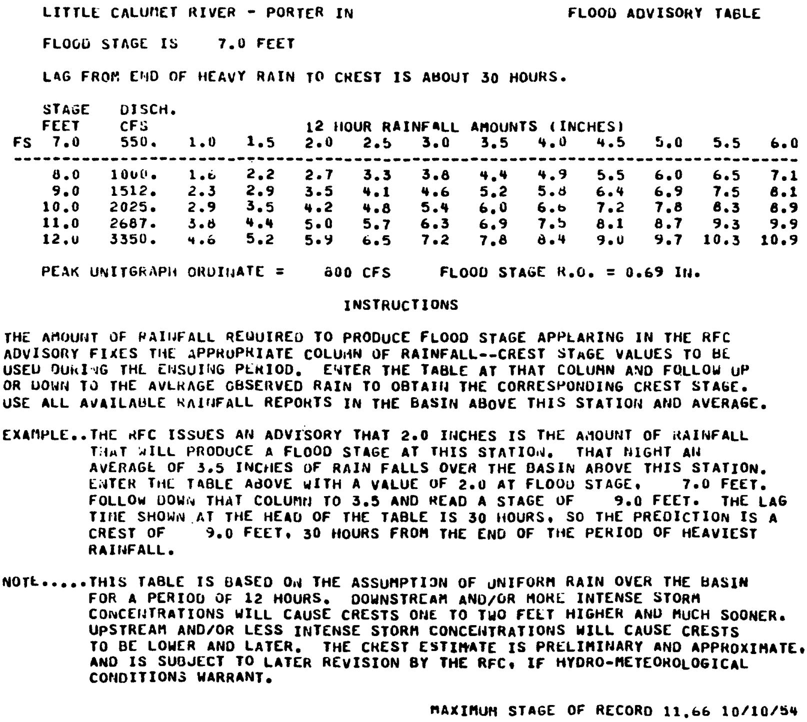
Flash floods are generally considered to be flood events which occcur quickly, generally less than six hours after the causative event. The time frame is so short that it is difficult for the normal forecasting process to be completed quick enough to save lives. For that reason, the National Weather Service has additional tools to handle the flash flood.

The primary procedure for flash floods is for the Weather Service Office to issue Flash Flood Watches or Flash Flood Warnings. The Watch means that conditions are susceptible to rapid occurences of flooding. Probably stream flows are already high, soils wet or saturated and radar or QPF indicates considerable rain is yet to come. The Watch is intended to tell the public to be aware of the situation around them, especially if they live or work near streams susceptible to flooding. The Flash Flood Warning means that flooding is occurring or is about to occur. The emphasis here is on potential danger to life. Everyone in a flash flood warning area should take necessary actions to protect themselves (Figure 11).

Note that in both the Watch and the Warning, specific crest forecasts are not mentioned, only the area or locations where rapid life threatening flooding is possible. Of course, river and flood forecasts follow the Watches and Warnings as quickly as possible. The Watches and Warnings are issued based on Flash Flood Guidance issued by the appropriate RFC to each WSO and WSFO. Each State is broken down into several forecast zones, generally 8-10 per State.

The River Forecast Center keeps estimates of soil moisture (API) values for all runoff zones. By knowing the current API value, and the amount of runoff required for flooding in a particular basin, the amount of rainfall required to bring about flooding in a given basin can be calculated. All of these rainfall values for a State forecast zone are then averaged to provide the flash flood guidance for a given zone.

In addition, the RFC's provide a second form of flash flood guidance. This is called headwater guidance. It is similar to zone flash flood guidance but headwater guidance is for a given specific point. The headwater guidance then will tell the Weather Service Office for a particular time, how much rain will be required to produce flooding at a given location. In addition, the RFC provides a simplifed headwater forecast table so that the WSO or WSFO can make a quick initial hydrologic forecast based only on Headwater Guidance and an estimate of storm rainfall.



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    LITTLE CALUMET RIVER - PORTER IN                            FLOOD ADVISORY TABLE

    FLOOD STAGE IS    7.0 FEET

    LAG FROM END OF HEAVY RAIN TO CREST IS ABOUT 30 HOURS.

    STAGE   DISCH.
    FEET    CFS                  12 HOUR RAINFALL AMOUNTS (INCHES)
 FS  7.0     550.   1.0   1.5   2.0   2.5   3.0   3.5   4.0   4.5   5.0   5.5   6.0
-------------------------------------------------------------------------------------
     8.0    1000.   1.6   2.2   2.7   3.3   3.8   4.4   4.9   5.5   6.0   6.5   7.1
     9.0    1512.   2.3   2.9   3.5   4.1   4.6   5.2   5.8   6.4   6.9   7.5   8.1
    10.0    2025.   2.9   3.5   4.2   4.8   5.4   6.0   6.6   7.2   7.8   8.3   8.9
    11.0    2687.   3.8   4.4   5.0   5.7   6.3   6.9   7.5   8.1   8.7   9.3   9.9
    12.0    3350.   4.6   5.2   5.9   6.5   7.2   7.8   8.4   9.0   9.7  10.3  10.9

    PEAK UNITGRAPH ORDINATE =    800 CFS     FLOOD STAGE R.O. = 0.69 IN.

                                   INSTRUCTIONS

THE AMOUNT OF RAINFALL REQUIRED TO PRODUCE FLOOD STAGE APPEARING IN THE RFC
ADVISORY FIXES THE APPROPRIATE COLUMN OF RAINFALL--CREST STAGE VALUES TO BE
USED DURING THE ENSUING PERIOD.  ENTER THE TABLE AT THAT COLUMN AND FOLLOW UP
OR DOWN TO THE AVERAGE OBSERVED RAIN TO OBTAIN THE CORRESPONDING CREST STAGE.
USE ALL AVAILABLE RAINFALL REPORTS IN THE BASIN ABOVE THIS STATION AND AVERAGE.

EXAMPLE..THE RFC ISSUES AN ADVISORY THAT 2.0 INCHES IS THE AMOUNT OF RAINFALL
         THAT WILL PRODUCE A FLOOD STAGE AT THIS STATION.  THAT NIGHT AN
         AVERAGE OF 3.5 INCHES OF RAIN FALLS OVER THE BASIN ABOVE THIS STATION.
         ENTER THE TABLE ABOVE WITH A VALUE OF 2.0 AT FLOOD STAGE,    7.0 FEET.
         FOLLOW DOWN THAT COLUMN TO 3.5 AND READ A STAGE OF    9.0 FEET.  THE LAG
         TIME SHOWN AT THE HEAD OF THE TABLE IS 30 HOURS, SO THE PREDICTION IS A
         CREST OF    9.0 FEET, 30 HOURS FROM THE END OF THE PERIOD OF HEAVIEST
         RAINFALL.

NOTE.....THIS TABLE IS BASED ON THE ASSUMPTION OF UNIFORM RAIN OVER THE BASIN
         FOR A PERIOD OF 12 HOURS.  DOWNSTREAM AND/OR MORE INTENSE STORM
         CONCENTRATIONS WILL CAUSE CRESTS ONE TO TWO FEET HIGHER AND MUCH SOONER.
         UPSTREAM AND/OR LESS INTENSE STORM CONCENTRATIONS WILL CAUSE CRESTS
         TO BE LOWER AND LATER.  THE CREST ESTIMATE IS PRELIMINARY AND APPROXIMATE,
         AND IS SUBJECT TO LATER REVISION BY THE RFC, IF HYDRO-METEOROLOGICAL
         CONDITIONS WARRANT.

                                             MAXIMUM STAGE OF RECORD 11.66 10/10/54
```

Figure 11.--Flash flood simplified forecast table.

Dam Breaks

Generally, dam breaks occur with little or no warning. Buffalo Creek, Taccoa Dam and the Teton Dam are good examples. All three of these dam failures, and many more which could be cited, have resulted in large losses of life and tremendous property damage. The devastation which occurs downstream of a breached dam is overwhelming. There are at least 20,000 dams in the United States that are located so that failure would result in loss of human life.

Advising the public of flood dangers is the responsibility of the NWS. Because of these responsibilities, the NWS has developed a numerical Dambreak Model to simulate the breach formation, outflow hydrograph and downstream routed hydrographs. In producing a dam break flood forecast, the model first computes the peak outflow at the dam, based on the reservoir size and the temporal and geometrical description of the breach. The computed floodwave and channel properties are used in conjunction with routing curves to determine how the peak flow will be diminished as it moves downstream. Based on this predicted floodwave reduction, the model computes the peak flows at specified downstream points. The model then computes the depth reached by the peak flow based on the channel geometry, slope, and roughness at these downstream points. The model also computes the time required for the peak to reach each forecast point and the time at which that depth is reached as well as when the floodwave recedes below that depth, thus providing a time frame for evacuation.

With an available simplified version of the Dambreak Model, a forecaster may, within minutes, produce forecasts of the dam break floodwave peak discharges, stages, and travel times. It should be noted, however, that the use of the NWS Dambreak Model is not limited to NWS hydrologists. Planners, designers, civil defense officials, and consulting engineers who are concerned with the potential effects of a dam failure and who have limited time, resources, data, computer facilities, and/or experience with unsteady flow models may also wish to employ the model to delineate areas facing danger from a dam break emergency.

Water Supply

Water Supply Outlooks are generated primarily by regression type models (Figure 12). These were previously discussed briefly in the section on Forecasting Models. Water supply forecasts differ from river or flood forecasting in that they forecast seasonal volumes rather than peak flows. While a whole range of probability type volume outloooks are possible, the NWS issues water supply outlooks as Reasonable Maximum, Most Probable, and Reasonable Minimum volumns.

Navigation

Many rivers are large enough to support commercial shipping. The Mississippi, Missouri, Ohio, and other rivers carry commerce of all types including agricultural products and natural resources such as coal.

Barge traffic is greatly affected by river conditions. Increased stages may mean that shippers can load more on each barge. Low stages signal a concern for groundings, therefore, barges cannot be loaded as heavily. Special problems such as extremely high flows or ice conditions may make shipping hazardous. This information is all important to the barge operators.

In addition, a barge can take several weeks to travel from New Orleans upstream to the Upper Mississippi or Ohio Rivers. The shippers want to know as much as possible about the long term trends of the river.

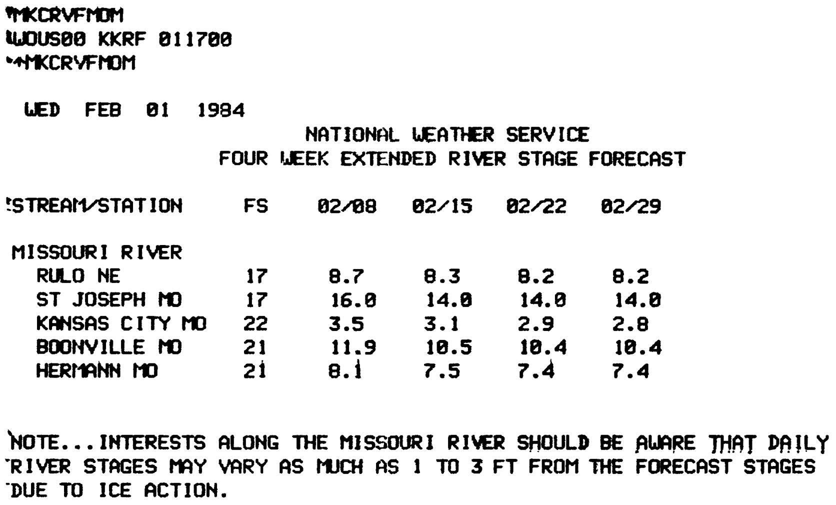
Is the trend up or down and how fast? Where are major flows entering the mainstem? For these reasons, the NWS generates a long term Navigation Forecast.

The long range navigation forecast, in the example case extending out four weeks, is generated by allowing the hydrologic forecast model to "freewheel" into the future (Figure 13). Recession constants are applied to the flows so that the long range forecast is a minimum flow forecast. The purpose is to let the shippers know what the minimum flows will be assuming no precipitation for the future forecast period.

STREAMFLOW FORECASTS

STREAM AND STATION	FORECAST PERIOD	FORECASTS THIS YEAR				20 YEAR (1961-80) AVERAGE RUNOFF (1000 AF)
		MOST PROBABLE (1000AF)	MOST PROBABLE (PERCENT OF AVG.)	REASONABLE MAX. (PERCENT OF AVG.)	REASONABLE MIN. (PERCENT OF AVG.)	
		MISSOURI BASIN				
BIG HOLE RIVER						
MELROSE NR, MT	APR-SEP	750	99	137	66	760
MADISON RIVER						
MCALLISTER NR, MT	APR-SEP	890	105	130	82	848
GALLATIN RIVER						
GALLATIN GATEWAY NR, MT	APR-SEP	555	102	129	83	545
MISSOURI RIVER						
TOSTON, MT	APR-SEP	2663	105	147	69	2545
FORT BENTON, MT	APR-SEP	4023	101	154	61	3980
VIRGELLE, MT	APR-SEP	4573	100	162	59	4570
LANDUSKY NR, MT	APR-SEP	4980	100	163	58	4980
FORT PECK DAM BLO, MT	APR-SEP	4960	100	167	55	4960
LAKE SAKAKAWEA INFLOW, ND	APR-SEP	13000	102			12755
LITTLE MISSOURI RIVER						
WATFORD CITY NR, ND	FEB-SEP	377	82			459
HEART RIVER						
MANDAN, ND	MAR-APR	60.0	58			103
MISSOURI RIVER TRIBUTARIES						
FORT RANDALL DAM ABV, SD	MAR-APR	1510	109			1390
SUN RIVER						
GIBSON RES INFLOW, MT	APR-SEP	442	78	126	54	570
MARIAS RIVER						
SHELBY NR, MT	APR-SEP	405	75	165	44	542
MUSSELSHELL RIVER						
HARLOWTON, MT	APR-SEP	88.1	90	142	47	97.9
MILK RIVER						
WESTERN CROSSING, MT	MAR-SEP	46.3	85	159	54	54.7
EASTERN CROSSING	MAR-SEP	92.7	85	117	70	109
YELLOWSTONE RIVER						
YELLOWSTONE LAKE OUTLET, WY	APR-SEP	890	108			825
CORWIN SPRINGS, MT	APR-SEP	2165	107	125	83	2027
LIVINGSTON NR, MT	APR-SEP	2550	107	129	01	2379
BILLINGS, MT	APR-SEP	4666	104	131	76	4486
MILES CITY, MT	APR-SEP	7070	104	137	69	6787
SIDNEY NR, MT	APR-SEP	7802	104	139	65	7518
CLARKS FORK YELLOWSTONE R						
BELFRY NR, MT	APR-SEP	645	103			628
WIND RIVER						
DUBOIS NR, WY	APR-SEP	113	107			106
BOYSEN DAM BLO, WY	APR-SEP	1221	105	141	70	1163
BIG HORN RIVER						
KANE, WY	APR-SEP	1304	106	150	68	1225
GREYBULL RIVER						
MEETEETSE, WY	APR-SEP	230	107	168	72	214
SHELL CREEK						
SHELL NR, WY	APR-SEP	78.0	100			77.9
SHOSHONE RIVER						
BUFFALO BILL DAM BLO, WY	APR-SEP	922	109	128	80	844
TONGUE RIVER						
DAYTON NR, WY	APR-SEP	144	117			123
CLEAR CREEK						
BUFFALO NR, WY	APR-SEP	37.0	92			40.2
NORTH FORK POWDER RIVER						
HAZELTON NR, WY	APR-SEP	11.1	105			10.6
NORTH PLATTE RIVER						
NORTHGATE NR, CO	APR-SEP	422	161	193	120	262
SINCLAIR NR, WY	APR-SEP	1000	141	172	99	710
GLENDO BLO, WY	APR-SEP	1187	122	164	77	973
GUERNSEY RES BLO, WY	APR-SEP	1236	123	166	78	1001
SWEETWATER RIVER						
ALCOVA, WY	APR-SEP	77.4	105	191	34	73.7
LARAMIE RIVER						
WOODS, WY	APR-SEP	202	153	176	123	132
SOUTH PLATTE RIVER						
LAKE GEORGE NR, CO	APR-SEP	55.1	120	217	83	45.9
CHEESMAN LAKE BLO, CO	APR-SEP	117	123	215	70	95.1
SOUTH PLATTE, CO	APR-SEP	246	125	195	66	197
NORTH FORK SOUTH PLATTE R						
SOUTH PLATTE, CO	APR-SEP	99.6	130	174	63	76.6
BEAR CREEK						

Figure 12.--Typical water supply outlooks



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WOUS00 KKRF 011700
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 WED  FEB  01  1984
                          NATIONAL WEATHER SERVICE
                    FOUR WEEK EXTENDED RIVER STAGE FORECAST

STREAM/STATION     FS    02/08   02/15   02/22   02/29

MISSOURI RIVER
  RULO NE          17     8.7     8.3     8.2     8.2
  ST JOSEPH MO     17    16.0    14.0    14.0    14.0
  KANSAS CITY MO   22     3.5     3.1     2.9     2.8
  BOONVILLE MO     21    11.9    10.5    10.4    10.4
  HERMANN MO       21     8.1     7.5     7.4     7.4

NOTE...INTERESTS ALONG THE MISSOURI RIVER SHOULD BE AWARE THAT DAILY
RIVER STAGES MAY VARY AS MUCH AS 1 TO 3 FT FROM THE FORECAST STAGES
DUE TO ICE ACTION.
```

Figure 13.--Long range (navigation) forecast

Extended Streamflow Prediction (ESP)

Extended streamflow prediction (ESP) combines the needs of Water Supply Outlooks, Spring Flood Outlooks, and Navigation Forecasts into a single forecast process that will meet all needs. ESP is not operational in the NWS yet, but effort is being made in the concept and necessary software required to implement it (Figure 14).

ESP is combined with the current hydrologic forecast model, either API and SMA, to project flows into the future. ESP starts with current conditions as reflected in the forecast model. Then a series of historical data in the form of observed precipitation and temperature patterns are applied to run the forecast model into the future through the snowmelt runoff period. By doing this for many years of historical data, statements can be made concerning probabilities of volumes of runoff which can be anticipated. The ESP concept should provide a management tool which can be utilized to answer "what if" type questions concerning long range flows.

Spring Flood Outlooks are another form of extended streamflow predictions. These outlooks are developed for areas subject to spring snowmelt floods. Over the last thirty years, these forecasts have been very successful and have saved countless lives and millions of dollars in flood damages.

Spring Flood Outlooks start with current conditions as reflected in the API forecast model. Current conditions include actual streamflows, soil moisture conditions and snow cover conditions. A separate model then applies historical melt patterns to the snow cover and runs the forecast model in the future far enough to melt the snowcover. Peak flows are extracted in the process and constitute the Spring Flood Outlook. Two sets of peak flows are provided, peak flows based on current snowcover and peak flows based on current conditions and normal future precipitation.

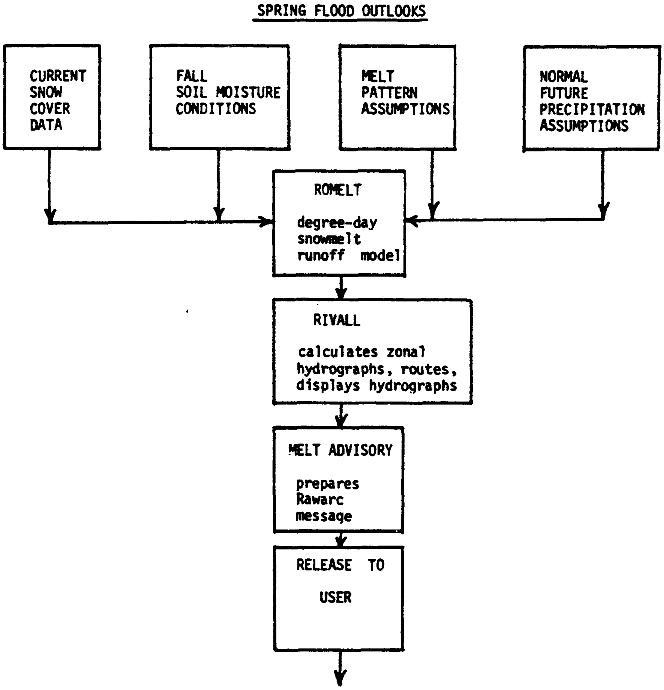

Figure 14. Operational Procedure for spring flood outlooks

CASE HISTORY AND ADDITIONAL FORECAST EXAMPLES

Missouri Basin River Forecast Center (MBRFC)

The Missouri Basin River Forecast Center is typical of the thirteen RFC's in the NWS. The RFC's provide first-echelon guidance for other NWS offices. The RFC's develop and implement hydrologic forecast procedures and techniques, keep current in hydrologic techniques, and computer systems, provide a hydrologic "consulting service" for other Government Agencies and coordinate necessary hydrologic activities with other offices and agencies.

RFC's provide river, flood, mainstem and reservoir inflow forecasts (Figure 15). The RFC's provide water supply, spring flood, and navigation outlooks. Advisories (flash flood guidance) are issued for forecast zones and headwaters (Figure 16). Some of these products are issued daily or more often while others are monthly or even seasonal products. The RFC's provide special services such as dam break analysis, low flow (drought) forecasts, and training programs (Figure 17).

MBRFC has a staff of twelve professionals, primarily civil engineers. They provide about 80,000 hydrologic forecasts annually for 583 forecast points. They process 360,000 pieces of hydrologic information annually and save the public on estimated $83 million annually.

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 MON  FEB  06  1984   NATIONAL WEATHER SERVICE RIVER STAGE FORECASTS

STREAM/STATION        FS   TODAY  02/07   02/08   02/09    ICE REPORTS

MISSOURI RIVER
  SIOUX CITY IA       36   13.8    12.7    12.5    12.4    50% FLOATING
  OMAHA NE            29   14.9    14.8    14.1    14.0
  NEBRASKA CITY NE    18    9.3     9.1     9.0     8.7
  RULO NE             17   E9.8     9.4     9.2     8.9
  ST JOSEPH MO        17   19.7    17.0    11.0    10.0    90% FLOATING
                           CREST NR 22 FT 2/6 NOON
  KANSAS CITY MO      22    4.9     7.5     6.6     5.6
  WAVERLY MO          20   11.7    11.4    14.0    13.1    50% FLOATING
  GLASGOW MO          25   13.7    13.3    14.1    14.7    30% FLOATING
  BOONVILLE MO        21   10.9    10.4    10.9    11.9
  JEFFERSON CITY MO   23   11.8    11.3    11.3    12 0
  HERMANN MO          21   10.9    11.4    12.1    12.0    20% FLOATING
  ST. CHARLES MO      25   15.0    14.7    15.2    15.9
KANSAS RIVER
```

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MKCRVFBBL
WOUS00 KKRF 041600
^MKCRVFBBL

 SAT  FEB  04  1984  NATIONAL WEATHER SERVICE RIVER STAGE FORECASTS

STREAM/STATION        FS  TODAY  FORECAST

BIG BLUE RIVER
  CRETE NE            18  19.9   CRESTED CONTINUE TO FALL
TURKEY CREEK
  WILBER NE           11  13.6   14 FT TODAY 02/04/84
BIG BLUE RIVER
  BEATRICE NE         16  MISG   17 TO 18 FT 02/05/84

 END
```

Figure 15.--Typical river and flood forecasts

```
[illegible]
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NOUS KMKC 271830
-KRFM7

ALCKT A KRFM7

ATTN ALL WSFOS WSOS SERVED BY KRFM7

AVERAGE 3 HOUR FLASH FLOOD RAINFALL GUIDANCE BY STATE FCST ZONES
RELEASE DATE THU OCT 27 1977

NOTE...GUIDANCE FOR MOUNTAIN AND URBAN AREAS. RAINS OF 1.00 INCH OR
MORE IN ABOUT AN HOUR OR LESS BELOW STEEP TERRAIN AND 2.00 INCHES
OR MORE IN ABOUT 2 HOURS OR LESS IN PLAINS TERRAIN ARE CAPABLE OF
PRODUCING FLASH FLOODING OF STREETS/FREEWAYS/UNDERPASSES AND FLOOD
PRONE LOW LYING AREAS.

COLORADO
  2/3.2   3/3.2   4/3.5   7/4.0
ILLINOIS
  1/2.7   2/1.7   3/2.2   4/2.6   5/2.0   7/2.4   8/2.3   9/2.7
 10/2.6  11/3.2  12/2.8
IOWA
  1/3.1   2/2.4   3/3.0   4/4.6   5/2.7   6/2.6   7/2.1   8/2.3
  9/2.0  10/2.0  11/2.1  12/2.1  13/2.4
KANSAS
  1/3.8   2/4.0   4/4.0   5/4.0   7/4.2   8/3.4  10/3.3  71/3.1
 13/3.2  14/2.6  15/1.8 16/2.1
MICHIGAN
  1/3.8   2/4.7   3/4.9   4/5.1   5/5.2   6/5.4   7/5.3   8/5.5
  9/5.5  10/5.5  11/5.5  12/5.4  13/5.5  14/5.5  15/4.2  16/5.2
 17/2.7  18/2.8  19/3.6  20/3.9  21/2.8  22/4.8
MINNESOTA
  1/3.6   2/3.3   3/3.4   4/3.7   5/2.8   6/3.2   7/3.8   8/3.3
  9/3.3  10/3.5  11/2.9  12/2.8  13/2.5  14/2.7  15/2.5
MISSOURI
  1/1.7   2/2.4   3/2.0   4/1.9   5/3.1   6/3.0   7/2.8   8/3.1
  9/3.6  12/3.7  15/2.2  16/2.9  17/1.6  18/1.4
MONTANA
  3/5.5   5/4.2   6/5.2   7/5.4   8/4.2   9/4.5
NEBRASKA
  1/5.2   2/5.5   3/5.5   4/5.5   5/3.4   6/3.2   7/3.1   8/3.3
  9/2.2  10/2.8  11/2.4  12/1.6  13/1.4  14/2.3
NORTH DAKOTA
  1/4.4   2/4.4   3/4.6   4/5.5   5/5.5   6/4.6   7/4.4   8/4.7
  9/5.0  10/5.0  11/5.5  12/5.5  13/4.5  14/4.3  15/2.9  16/3.3
SOUTH DAKOTA
  1/4.2   2/4.0   3/3.7   4/4.5   5/4.0   6/3.8   7/3.8   8/5.5
  9/5.5  10/4.0  11/4.0  12/3.6
WISCONSIN
  1/4.5   2/3.7   3/4.4   4/3.1   5/3.0   6/3.1   7/3.3   8/3.1
  9/3.4  10/3.4  11/2.9  12/3.9  13/2.9  14/3.4  15/3.8
WYOMING
  2/5.5   3/5.1   4/4.2   5/5.5   6/5.5   7/5.3   8/4.2  11/5.5
 12/3.2

END
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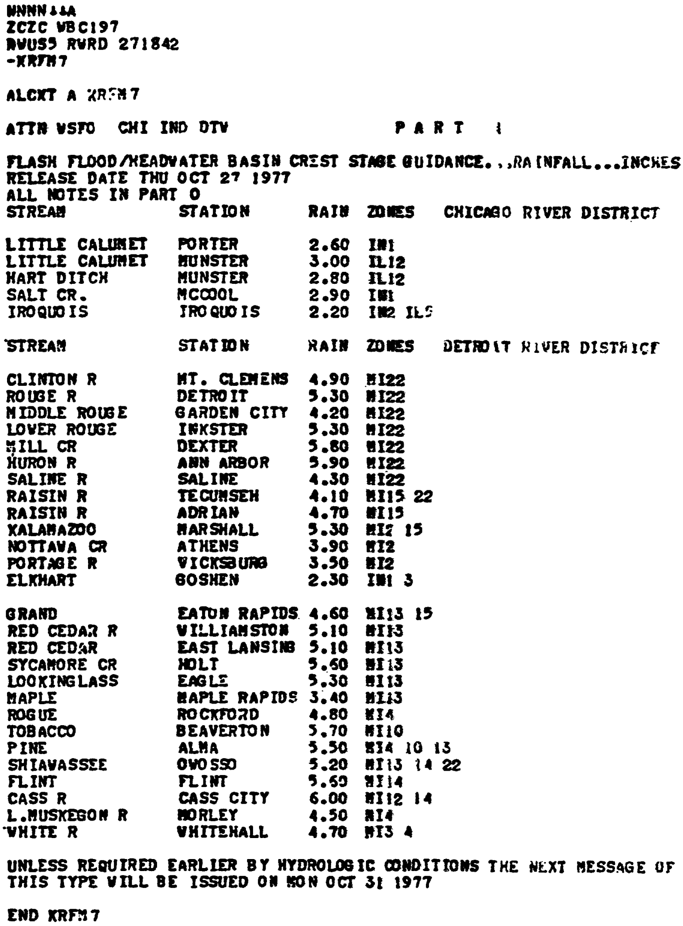
Figure 16.--Typical flash flood zone guidance



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WNNN11A
ZCZC WBC197
RVUS5 RWRD 271842
-KRFM7

ALCXT A KRFM7

ATTN WSFO  CHI IND DTW                  P A R T   1

FLASH FLOOD/HEADWATER BASIN CREST STAGE GUIDANCE...RAINFALL...INCHES
RELEASE DATE THU OCT 27 1977
ALL NOTES IN PART 0
STREAM              STATION        RAIN  ZONES   CHICAGO RIVER DISTRICT

LITTLE CALUMET      PORTER         2.60  IN1
LITTLE CALUMET      MUNSTER        3.00  IL12
HART DITCH          MUNSTER        2.80  IL12
SALT CR.            MCCOOL         2.90  IN1
IROQUOIS            IROQUOIS       2.20  IN2 IL5

STREAM              STATION        RAIN  ZONES   DETROIT RIVER DISTRICT

CLINTON R           MT. CLEMENS    4.90  MI22
ROUGE R             DETROIT        5.30  MI22
MIDDLE ROUGE        GARDEN CITY    4.20  MI22
LOWER ROUGE         INKSTER        5.30  MI22
MILL CR             DEXTER         5.80  MI22
HURON R             ANN ARBOR      5.90  MI22
SALINE R            SALINE         4.30  MI22
RAISIN R            TECUMSEH       4.10  MI15 22
RAISIN R            ADRIAN         4.70  MI15
KALAMAZOO           MARSHALL       5.30  MI2 15
NOTTAWA CR          ATHENS         3.90  MI2
PORTAGE R           VICKSBURG      3.50  MI2
ELKHART             GOSHEN         2.30  IN1 3

GRAND               EATON RAPIDS   4.60  MI13 15
RED CEDAR R         WILLIAMSTON    5.10  MI13
RED CEDAR           EAST LANSING   5.10  MI13
SYCAMORE CR         HOLT           5.60  MI13
LOOKINGLASS         EAGLE          5.30  MI13
MAPLE               MAPLE RAPIDS   3.40  MI13
ROGUE               ROCKFORD       4.80  MI4
TOBACCO             BEAVERTON      5.70  MI10
PINE                ALMA           5.50  MI4 10 13
SHIAWASSEE          OWOSSO         5.20  MI13 14 22
FLINT               FLINT          5.60  MI14
CASS R              CASS CITY      6.00  MI12 14
L.MUSKEGON R        MORLEY         4.50  MI4
WHITE R             WHITEHALL      4.70  MI3 4

UNLESS REQUIRED EARLIER BY HYDROLOGIC CONDITIONS THE NEXT MESSAGE OF
THIS TYPE WILL BE ISSUED ON MON OCT 31 1977

END KRFM7
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Figure 17.--Typical flash flood headwater guidance

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THE ROLE OF REMOTE SENSING AND SATELLITE MONITORING SYSTEMS IN HYDROLOGIC HAZARD MANAGEMENT

By

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U.S. Geological Survey
Reston, Virginia

INTRODUCTION

Aerial photography has been used for decades for mapping hydrologic hazards and monitoring water-related disasters such as floods and droughts. The advent of satellites such as Landsat, which provide small-scale, repetitive, uniform images has allowed mapping and monitoring to be done readily and cost-effectively throughout the world. This brief introduction will show that Landsat images can be used for flood and drought mapping.

Two papers outline the uses of Landsat and other satellites in disaster warning, assessment, and management. The first, "Worldwide disaster warning and assessment with Earth Resources Technology Satellites" was published in 1975 but much of the information in it is still applicable.* The second, "Recommended satellite imagery capabilities for disaster management," published in 1982, outlines the satellite capabilities needed in the future for coping with various types of disasters. It is listed in the bibliography.

INTRODUCTION TO LANDSAT

Landsat satellites (formally called Earth Resources Technology Satellites or ERTS) are used for mapping, inventorying, and monitoring earth features.

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*Robinove, Charles J., Worldwide Disaster Warning and Assessment with Earth Resources Technology Satellites, reprinted from "proceedings of the Tenth International Symposium on Remote Sensing of Environment" Center for Remote Sensing Information and analysis, Environmental Research Institute of Michigan, Ann Arbor, Mich., Oct., 1975, p. 811-820.

Five Landsats have been launched since 1972; the fifth was launched on March 1, 1984.

The major sensing instrument in Landsats is a multispectral scanner that. produces images of the earth in green, red, and two wavelengths of infrared radiation. The products are black-and-white images, and images in digital form that can be processed hy computers.

The basic characteristics of the satellite system and images that are of importance in disaster planning, management, and amelioration are:

1. Each image covers an area 115 statute miles on a side.

2. The size of each resolution element in an image is 1.1 acres or 0.45 nectares.

3. Each wavelength band has specific useful characteristics. For example, Band 7 (long wavelength infrared) always shows water as black, even if it has a high sediment load, and this is most useful for mapping coastlines, lakes, rivers, and floods.

4. The satellites pass over the same point every 16 or 18 days, thus providing repetitive and comparable observations over time. This is particularly important for detection of changes and monitoring of rates and extent of environmental change.

5. Images are available for almost all areas of the world (except within 9 degrees of the north and south poles) and, depending on cloud cover, may be available in repetitive observations for all countries.

Further information on Landsat system capabilities and details of the methods of application of images is well-documented in the "Manual of Remote Sensing" (2nd ed.) published by the American Society of Photogrammetry, 210 Little Falls Street, Falls Church, Virginia 22046, USA.

LANDSAT IMAGES OF FLOODS

Mapping of flooded areas with Landsat images is not difficult. It must be recognized that there are advantages and disadvantages to the methods, but within their specific characteristics and limitations, they can be highly useful.

Several mapping image interpretation criteria are given below and will be followed by a description of the most useful applications and limitations of the images.

Interpretation Criteria

1. Floods are imaged best on Landsat Band 7 (long wavelength infrared) because water is always black in those images and is readily distinguished from lighter toned areas.

2. Landsat color composite images show floods as dark blue, light blue, or bluish white as the concentration of sediment in the water ranges from very low, through medium, to high respectively.

3. Images taken before floods can be overlaid with images taken during floods to readily map the inundated areas as the difference between the images.

4. Digital processing of images may be necessary if the soils in and near flooded areas are very dark and difficult to distinguish from the flood water. This is an unusual situation, however, and is not often encountered.

Limitations of Application

1. Since Landsat images have a resolution of about 1.1 acres (0.45 hectare), details below that size are not readily visible unless they are of high contrast.

2. Because of the resolution, images of floods are best for mapping at scales from 1/1,000,000 to about 1/100,000 but only rarely at scales larger than 1/100,000.

3. Because of those limitations, the images are best used for broad area mapping at small scales and are not highly suitable for mapping in urban areas where delineation of a flood boundary is required at much higher precision.

4. Since Landsat images are taken at regular intervals which cannot be changed, it is not possible to image a flood at its peak, except by chance. It is, however, possible to map areas that have been inundated by a flood up to about 2 weeks after the flood has receded because the wet ground left after flood recession can be mapped.

5. Areas undergoing flooding may often be cloud covered during the time of a Landsat overpass, thus preventing acquisition of usable images.

DROUGHT DETECTION

Droughts are an example of a disaster that occurs slowly and may build to a level of severity over a long period of time, as contrasted to floods which may occur suddenly. Drought detection, therefore, requires repeated observation of land and vegetation conditions over a long period of time to assess the gradual (rather than sudden) changes that take place.

Mapping of land cover changes that occur during droughts can be done with a number of methods of analysis of Landsat images. All of them involve visual or digital comparison of images taken at different times of the same area. Visual methods of comparison are qualitative while digital methods promote quantitative measurements of changes of reflectance of the ground. These changes can be related to increases or decreases in vegetation cover, soil erosion, and soil moisture. Numerous papers on this topic are listed in Walter and Robinove (1981).

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FAMILY AND SOCIETAL RESPONSE TO DISASTERS

by

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INTRODUCTION

During the last three decades the empirical research base focused on social system responses to disaster has grown substantially, both in quantity and quality. One evidence of this is a work that I am revising for publication at the present time. The working title is Human System Responses to Disaster: An Inventory of Sociological Findings. This inventory is based on an analysis of 883 publications--academic journal articles and books. Most were completed by sociologists, although a goodly number of studies by social geographers, psychologists, and economists were covered too. All findings were grouped into one of four phases of emergency management--that is, preparedness, response, recovery, and mitigation. The working draft contains 153 sub-topics, 753 major conclusions, and 1200 specific findings. (Earlier reviews include: Baker and Chapman, 1962; Barton, 1969; Dynes, 1970; Quarantelli and Dynes, 1977; Mileti, Drabek and Haas, 1975; Kreps, in press).

Obviously, a presentation like this short paper can only cover the major highlights from a small segment of this data base. This paper will outline some of the key research findings pertaining to three sub-topics: 1) family responses to disaster warnings; 2) search and rescue activities; and 3) long-term impacts of disaster on families.

Let me stress at the outset, however, that most of this research was conducted within the U.S.A. Thus, it will not generalize to other nations, necessarily. Cross-national studies that are required to specify these ranges of generalization are just now starting to appear in the literature (e.g., Burton, Kates and White, 1978; Bolin and Bolton, 1983).

FAMILY RESPONSES TO DISASTER WARNINGS

Through several studies following both natural disasters, like floods, volcanoes and hurricanes, and various technological hazards, like the threat posed by the Three Mile Island nuclear power plant accident (March, 1979) or transportation accidents involving toxic substances, a general profile of family responses has emerged (Perry, 1983). While events with markedly different characteristics do not evoke uniform responses--for reasons that are still unclear--the modal profiles have been documented. Let's examine five of these.

1. Disbelief, not panic, is the initial response.

Numerous studies have examined the types of responses that are evoked when disaster warnings are issued (e.g., Moore, et al., 1963; Drabek, 1969, 1983b; Perry, Lindell, and Greene, 1981). Typically, people refuse to believe that they are in danger. Of course, this is variable. That is, qualities of the person--like their experience with previous disasters--and of the community skew the modal pattern. Some communities that have reoccuring events on a regularized basis--be it tornadoes or hurricanes, for example--evidence elements of what has come to be called a "disaster subculture". Moore and his colleagues (1964) described this trait initially as they tried to understand hurricane responses by residents of the Gulf Coast region.

Such subcultures are comprised of curious blends of beliefs that contain important inconsistencies. Typically, there are elements that spur protective action. People living in "tornado alley," for example, are more likely to check out a warning siren or seek immediate cover than those living elsewhere. In contrast, however, some aspects of these subcultures act to neutralize perceptions of danger. Thus, Moore and his team discovered that some responded to official advisories by throwing "hurricane parties." They stood in defiance of the potential danger. Neither the extent of these subcultures, nor their causes and consequences are well understood at present (Hannigan and Kueneman, 1978).

2. Primary groups, not isolated individuals are the response systems.

Most disaster warnings are received by groups, not individuals per se. Why? Simply because most of the time, most people are physically near others —be it at work, play, or home.

This social reality has significant consequences. Responses are group based and can not be understood if they are interpreted using an individualist psychology. We know that women and primary school age children tend to take disaster warnings more seriously. Thus, the group response is a dynamic of bargaining. Unless unusual circumstances prevail, families evaluate options with certain givens. The most important of these is that they will remain together physically (Drabek, 1983b; Quarantelli, 1980). Thus, if the threat is of such a nature that they are considering evacuation, the issue becomes one of a total group decision, for if they leave, nearly always it is as a complete unit.

3. Families that are intact--that is, physically together--when warnings are issued typically respond quicker than those who happen to be separated at the time.

Limited research (e.g., Drabek, 1969; Drabek and Stephenson, 1971; Perry, Lindell, and Greene, 1981) has documented this pattern. Families that have members at school, shopping, or elsewhere when warnings are issued, do evidence delaying tactics. If they can not be reunited prior to departure, major efforts are made to account for the missing members. If possible, those not present are advised of the anticipated place of refuge.

In this vein, it is essential to realize that for many American families, pets are defined as "members." Thus, studies like Whyte's (1980) assessment of the response to a toxic chemical threat (Mississauga Canada, 1979), have indicated that about one-half of the households had at least one pet. Typically, dogs were available for quick departure, but many with cats could not locate them. Thus, some families probably delayed leaving as members tried to find the family pet. (Quarantelli, 1980).

4. Evacuation behavior is a complex social process reflecting several alternative patterns.

When the human dynamics of evacuation are examined from a behavioral perspective, it turns out that the decision process is quite complex. Earlier researchers (e.g., see Danzig, Thayer, and Galanter, 1958) tried to assess the process using an overly rational model. Thus, the quest was begun to discover the key piece of information that people received that convinced them. "What did you hear--and from whom--that finally persuaded you to leave?"

Let's be clear here, however. People do receive disaster relevant information--lots of it. Indeed, the snowballing quality of such time periods is remarkable. Families often are confronted with more information than they can process. As they are trying to evaluate one set of data, they are bombarded with additional media updates and telephone calls. Thus, while far from consistent and frequently overtaxing, some families do muddle through the morass of information received and come to a decision--to flee or stay. Within a "bounded" model of rationality (Burton, Kates and White, 1978), they do make a choice.

But for others the process differs (Drabek, 1983b)--the pathways are multiple. Most frequently relatives or friends issue an invitation. That is, during the warning period, they contact the family, often to insure that the message was received. During the interaction, however, an invitation is issued. "Why don't you come over here?" It is the invitation, not a belief that they really are in danger, that is the critical motivant.

Similarly, some families reflect a compromise process. Not all members are convinced of the danger--typically fathers and teenagers. But to "keep the peace" they consent to leave.

For others, the process can only be considered as "default." Example? A family elects to drive to the river to see if the media are exaggerating. Upon reaching it, they come to realize that the threat may jeopardize their home. Perhaps they left the dog or some other "member" there. When they try to return they are stopped by law enforcement personnel who are engaged in security and getting out the last of the stragglers. The circumstances vary greatly, but this process has one key ingredient--these families never really make a decision to evacuate. It just happens.

5. A small percentage of evacuating families will select public shelters as a place of refuge, but most turn to relatives and friends.

When families leave prior to a disaster, they depart as units usually, and rarely seek shelter in a public place. The rates vary, of course, depending upon many specifics--matters like length of forewarning, anticipated scope of destruction, level of community preparedness. Reviewing of numerous evacuations prior to hurricanes, floods, or toxic chemical threats, indicate a range of 3-6 percent (Drabek, 1983b; Quarantelli, 1980). Thus, relatives, friends, and other resources are selected by most.

In specialized circumstances, higher rates have been reported. For example, Perry, Lindell and Greene (1981), discovered that prior to the flooding in Fillmore and Valley--both communities in the state of Washington--much higher percentages of the population turned to public shelters. In both instances officials advised families of shelter locations and urged them to go there. This was especially true in Valley (29% housed in public shelters) since the entire town had to be evacuated for several days--some families couldn't return home for a week.

In short, within the U.S.A., families and other primary groups provide structure and direction to disaster warning responses. In societies with more pervasive kin influence, it is likely that such processes are even more critical.

SEARCH AND RESCUE ACTIONS

In contrast to numerous studies of family responses to disaster warnings, only recently have search and rescue (SAR) activities been scrutinized. Through case studies of single disasters wherein limited observations have been made regarding SAR actions (e.g., Form and Nosow, 1958; Fritz, 1961; Wenger and Parr, 1969; Committee on the Alaska Earthquake, 1970; Taylor, Zurcher and Key, 1970) and one multi-event study (Drabek, 1983a; Drabek, et al., 1981), five major conclusions may be drawn. Let's examine each briefly.

1. Many disasters do not generate high SAR demand levels.

Search and rescue (SAR) activities result when warning systems fail. Of course, there are some events--like explosions and earthquakes--that preclude specific warnings, although in the case of some earthquakes newly developed prediction technologies permit certain types of advance notice (see Mileti, Hutton, and Sorensen, 1981). Also, there are many instances wherein families are warned, refuse to leave, and then have to be "rescued" after impact. Often, many of these "rescues" are really evacuations--although transportation may not be provided by the departing family.

Flash floods, in contrast to the less violent but more frequent riverine flooding, commonly produce SAR demands. So too do large tornadoes. Areas destroyed by these killer winds must be searched quickly or further injuries--even death--will result. When the full spectrum of natural and man-made disasters is considered, however, especially given the disproportionate number resulting from riverine flooding, it becomes apparent that many disasters produce minimal demands for SAR actions. Furthermore, much of the SAR demand generated in these events could have been minimized if local governments had effective warning systems (Leik, et al., 1981). There is a tight degree of interdependence.

2. Regardless of the SAR demand level, unofficial helpers make important, and often life-saving, contributions.

For decades, disaster case studies have documented the quick response by victims and those citizens that circumstance has placed in proximity to disaster impacted areas. Restructuring time is required; the mind has to give meaning to sights never before seen. For most, however, this takes only seconds although a small percentage experience longer periods of daze and bewilderment (Fritz, 1961). Victims who are able and those nearby quickly converge to the most heavily impacted areas and render aid to any they encounter (Fritz and Mathewson, 1957). Thus, even when organizational mobilization is rapid--which it almost always is--most victims are aided initially by those who just happen to be nearby (Drabek, et al, 1981; Drabek, 1968).

One of our case studies documented this reality and its consequence; the response represented an extreme case. It was extreme in the sense that mobilization was instantaneous. That is, when a tornado set down at the northeastern edge of Cheyenne, Wyoming (1979), it barely missed state office buildings housing the highway patrol, highway department, national guard, and emergency preparedness agency. It then moved slowly in an easternly direction, veering slightly to the north as it ripped through tracts of newly built homes and eventually a mobil home park. Midway along the path was a fire substation. It too was spared. Thus, as the funnel cloud danced along, ripping and tearing everything in its path, organizational personnel from numerous emergency agencies moved behind. In some areas of the community, emergency crews preceded the black funnel and alerted residents with sirens.

Thus, rescue personnel arrived onscene instantly. Their presence provided comfort to victims whether they required medical treatment of not. Quickly they moved through the damaged areas, assisting those trapped beneath debris, directing others to move out of the area that now posed a serious fire threat due to downed electrical lines and in some areas leaking propane gas. How many victims were transported to local hospitals by officials? As best we could determine after extensive cross-checking--the answer remained, not one (Drabek, et al., 1981). Neighbors, relatives, friends, even strangers who just happened to be nearby, performed this task.

As in our other case studies, we encountered many heroes there. These were average citizens who rose to the occasion and came to the aid of a friend, neighbor, or relative. And as often as not, the person helped was a complete stranger. These case studies documented that many survivors of the events studied would have perished had this response been curtailed or dampened.

3. In events with high SAR demand levels, SAR is accomplished by emergent multiorganizational networks.

Overlaying this mass assault performed by citizens, are organizational responses. Each agency has a given level of expertise that defines its everyday mission. While there is a degree of interdependence, most conduct their daily affairs with minimal involvement from any of the others. The demand structure created by large-scale disaster requires an alternative design, however. And depending upon the extent and quality of planning, slowly a multiagency structure emerges so that actions taken by those identified with each of these relatively autonomous units can be coordinated—at least a bit (Drabek, 1983a). Thus, as time passes, citizens withdraw or become absorbed into these emergent structures.

4. _SAR networks are complex structures comprised of a diverse array of relatively autonomous units and agencies, reflecting both horizontal and vertical dimensions of a community._

Let's examine one case study in more detail so as to better illustrate this conclusion (Drabek, et al., 1981). The event happened in Kansas—Osage County to be exact. It lies to the south of the state capital, Topeka, where a dozen years earlier the worst destruction in the history of the nation was produced by a violent funnel of death (Drabek and Key, in press). On a hot June night—the seventeenth to be precise-the Showboat Whipporwill departed from shore and began its prearranged course around portions of Lake Pomona, a recently built reservoir. Suddenly, without warning, a funnel cloud converted this placid covering into a threat. It passed near enough to the Showboat to capsize it. Entertainers, passengers, and crew were thrown into the water and sucked deeply downward as the boat came to rest upside down.

As would be predicted by the description above, the initial rescue effort was performed by those who were nearby—campers, boaters, and others. Because of its location inside the boundaries of a state park, mobilization was immediate, both by state agencies and many units of local government. As rescue turned into recovery, more arrived to help. All total, the Osage County Sheriffs Department itemized 78 separate agencies who had personnel at the scene. Selecting only the 20 who performed the most critical aspects of the recovery effort, we proceeded to create a series of social maps that display varied dimensions of the emergent structure. Table 1 lists these 20 and illustrates the range and diversity of agencies that commonly comprise such emergent SAR networks.

5. Typically, there is ambiguity of authority regarding operations decisions.

While any of the cases studied could be used to illustrate this conclusion, let's pursue the night of the Whippoorwill. Here was a dramatic rescue effort, followed by an intensive recovery mission; it was required to locate the last of the 16 who died. During most of the four day life span of this emergent structure, coordination among the component agencies was accomplished. The Osage County Sheriff directed the network and integrated the varied resources each contributed into a whole--at least partially.

But initially there were questions, especially related to the matter of who ought to be in charge. This tragedy occurred at a federal reservoir, that was located within a state park. Yet, because a SAR response was required there was basis for the County Sheriff's claim to control. The Sheriff did assume command and maintained it throughout, although during the initial hours of the emergent network his authority was questioned repeatedly. While complex, this case represented the ultimate in simplicity when contrasted to the multicounty response evoked when Mount St. Helens erupted (May 18, 1980).

When we asked managers from each of the participating organizations about their communications with others, their perceptions of who was directing the entire network--if anybody--and their views regarding the pattern of decision-making, it became very evident that most did not have a comprehensive perspective (Drabek, 1983a). They had rather incomplete knowledge, their perceptions were constrained to segments of the total emergent network. Few, if any, participating managers appeared to grasp the whole network. What they came to realize, and echoed repeatedly, was that cross-agency communications were highly deficient. While accurate, issues regarding network authority and coordination remained submerged. Only among the most perceptive did these matters surface to a conscious level and then only in rather vague ways. None questioned the applicability of the managerial models they were using--

theories rooted in classical bureaucratic frameworks--to the unique demand structure generated by these events (Drabek, 1983a).

LONG-TERM IMPACTS OF DISASTER ON FAMILIES

While the research base on long-term family impacts has grown rapidly during the past decade, the results have not produced a consistent portrait. Many researchers report little or no long-term impacts while a few have produced evidence of highly significant and lasting disruptions to both self and family. To some degree these inconsistent results reflect important differences in measurement and methodologies (Perry, 1979). Those with a clinical orientation--whether they are actually treating victims or not--report more far reaching consequences. Most of these effects are negative--things like sleep disruption, increased use of alcohol, feeling of guilt and increased anxiety. While rare, occasional mention is made of positive change. For example, Palmer (1980) reported that following the Wichita Falls tornado that student nurses indicated higher level of commitment to their studies and future careers. The disaster made nursing skills relevant. The future careers were viewed as being more worthwhile, of greater importance.

Our knowledge base is growing rapidly regarding long-term effects. Thus, within the next decade we will obtain a greatly improved scientific base. But at the present time three general conclusions summarize the current state of our information base.

1. Several qualities have been proposed that may account for impact differences.

To date the comparative work has not been completed that is required to pin down the relative influence of these factors, however. Thus, several have been proposed, but none have been demonstrated to actually account for the differences in findings reported. Most commonly, five factors are discussed: 1) event qualities, e.g., degree of horrification; 2) community characteristics; 3) blame assignation; 4) recovery processes; and 5) special population segment impacted, e.g., social isolates or children.

2. The most severe negative impacts were reported following a flash flood in Buffalo Creek, West Virginia (1972).

What type of negative impacts were attributed to this event? The following selections illustrate a large number of studies.

> "The claim of 'psychic impairment' in the lawsuit against the Pittston Company, whose Buffalo Mining Company subsidary had built the dam, was based on the effects on the survivors' psychological well-being of the flood experience itself as well as the destruction of the community. The lawsuit was settled out of court in August 1974 for 13.5 million dollars, approximately half of which was based on "psychic impairment." ... The psychological impact of the disaster has been so extensive that no one in Buffalo Creek has been unaffected. The overwhelming evidence is that everyone exposed to the Buffalo Creek disaster has experienced some or all of the following manifestations of the general constellation of the survivor (Lifton, 1968, pp. 479-541). Lifton, R. J. Death in Life; Random House, 1968. (Lifton and Olson, 1976:1).
>
> "...the psychic distress experienced by many of the Buffalo Creek victims was crippling to the extent of interfering with effective daily functioning, ... among those followed as long as 4-5 years postdisaster, over 30% continued to suffer debilitating symptoms.... 30% indicated increased alcohol consumption; 44% increased cigarette smoking, and 52% increased use of prescription drugs. As expected, these increases tended to occur in families in which the adults displayed the more severe symptoms, particularly with regard to anxiety and depression, ... Over three-fourths of the respondents admitted having difficulty in getting to sleep or staying asleep during the week preceding the interview, more than 2 years after the disaster. Approximately one-third of the respondents needed to use medication at least 'sometimes' in order to get to sleep, and more than two-thirds 'sometimes' or 'often' had nightmares. These percentages are very much higher than those found in general population surveys" (Gleser, Green and Winget, 1981:141).

Many of these researchers have proposed one or more of the criteria listed above (see point #1) as partial explanation of these impacts. Thus, Erikson (1976) for example, argued that the "second disaster" of recovery may have contributed significantly to the pain experienced by these victims. Similarly, Titchener and Kapp (1976) proposed that the degree of horrification caused by the event itself was intense, hence exacerbating impacts that were worsened even more by the blame assignation processes that followed.

3. Differential adaptations in family functioning have been reported following some disasters.

Data gathered in Topeka, Kansas three years after the 1966 tornado permitted a quasi-experimental design (Drabek and Key, in press; 1976). Thus, pre and post event comparisons were made with both victim and non-victim samples. Analyses suggested that families adapted differentially. Linkages to those social units that had aided victims in the months of clean-up evidenced a slight degree of strengthening. For most these were bonds to relatives. Kin relationships and commitments were stronger. Less so, but still showing signs of intensification were internal family relationships and links to friends. Neighboring was weakened when the entire sample was reviewed, although some families did reveal many forms of stronger ties here too. Also, participation dropped in all forms of secondary associations—PTA, Lions, VFW, and the like. There was one exception, however. Commitment to and participation in religious organizations increased slightly among these tornado victims.

Many of these patterns have been reported following other tornadoes, although no other researchers to date have had the luxury of pre-event data. Events studied include tornadoes in Xenia, Ohio (Taylor, 1977), Omaha, Nebraska (Bell, 1978) and Wichita Falls, Texas (Bolin, 1982; Bolin and Klenow, 1983). While methods differed, results obtained following the eruption of Mount St. Helens by Leik and his research team (1982) had important parallels, especially regarding the increased commitment to religious convictions.

In summary, during the past decade researchers have penetrated warning response behavior so as to provide emergency managers with a solid scientific foundation on which to base procedures and policies. While questions remain, enough has been learned so that implementation is the core problem. In contrast, matters of interorganizational coordination, especially an understanding of the barriers and what can be done about them, requires much further study. What has been documented are the operational problems that can be anticipated when multiorganizational networks lack coordination. And finally, while it is clear that some victims and some families are scarred by disasters with certain qualities, we don't have a clear fix on the extent of this nor its permanence. Thus, long-term impacts has surfaced today as a top priority so that proper intervention strategies and related policies can be formulated, tested, and implemented.

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TABLE 1

LAKE POMONA SAR PARTICIPANTS

Org. No.	Name	Sponsorship	Staff Size Paid	Staff Size Volunteer	Location*	Formalization**
1.	Osage County Sheriff's Dept.	County	10	0	L	2
2.	Osage County Civil Defense Off.	County	½	0	L	4
3.	Osage County Coroner's Office	County	0	1	L	4
4.	Osage County Attorney's Office	County	2	0	L	4
5.	Kansas State Highway Patrol	State	409	0	NL	1
6.	Kansas State Parks & Resources Authority	State	6***	0	L	2
7.	Kansas State Game & Fish Commission	State	NA	NA	L	NA
8.	Kansas State Dept. of Transportation	State	67***	0	NL	1
9.	U.S. Army Corps of Engineers	Federal	9***	0	L	1
10.	U.S. Army Reserve	Federal	NA	NA	NL	NA
11.	Crable Ambulance	Private	2	0	L	2
12.	Franklin County Ambulance	County	9	0	NL	1
13.	Lee's Summit Underwater Rescue Team (Missouri)	Private	NA	NA	NL	NA
14.	Shawnee County CD Underwater Rescue Team	Private	0	30	NL	2
15.	Burlingame Police Department	City	NA	NA	NL	NA
16.	Lyndon Police Department	City	3	12	L	2
17.	American Red Cross	Private	9	900	NL	1
18.	Topeka Fire Department Rescue#1	City	260	0	NL	1
19.	Carbondale Fire Department	City	0	20-30	NL	3
20.	Topeka Radiator and Body Works	Private	26	0	NL	2

*Location = local or non-local. Agencies with permanent offices in the immediate area were considered as being local, e.g., the U.S. Corps of Engineers.

**Questionnaire item: "To what extent are your organization's work rules, procedures and policies in written form?" 1 = to a great extent; 2 = to some extent; 3 = to a small extent; 4 = not at all.

***Number of personnel in local or regional offices only; not for entire state or agency.

Source: Drabek, et al., 1981, p. 39.

PSYCHIATRIC ASPECTS OF COPING WITH DISASTER

By

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From a psychological point of view, disaster is a major Psycho-social Stressor (Ref. 1) or, in more old-fashioned terminology, Psychic Trauma (Ref. 2). Freud defined a Psychic Trauma as an "extensive breach being made in the protective shield against stimuli" (Ref. 2). The unifying factor in these stressors (Table 1) is the fact that they are unexpected and not planned for in the world view of the victims.

Investigators have studied the responses in the survivors, their loved ones, and in the relief workers who respond to the disasters. In general, the pathological or deviant responses have been emphasized and a systematic study of those who cope relatively better has not been done. Recently, however, the field has begun to turn its attention towards more successful adaptation to adversity (Refs. 3,4,5).

After a major disaster, the majority of victims have some form of observable emotional disturbance. Most of these, however, are transient and the victims recover spontaneously. However, after disasters there is an increased rate of psychiatric disturbance in the months and years following the incident.

There are differences between children and adult, both in their psychological response to victimization initially, and in their later attempts to find meaning years after the event (Ref. 6).

There are many variables which are known to affect the psychological outcome, and these include the following: the nature of the stressor itself, the number of killed and injured and extent of mutiliation and burns, the length of exposure to the stress, the accidental or deliberate nature of the

TABLE 1

Psychosocial Stressors	Accidental Man-Made Disasters
- rape	- severe car accidents
- assault	- airplane crashes
- kidnapping	- fires
	- building collapses
Deliberate Man-Made Disasters	Natural Disasters
- terrorist bombing	- earthquakes
- torture	- floods
- death camps	

(DSM III-1980)

stress, and in assault cases, whether the aggressor is known to the victim. Cultural responses and explanations and the response of organized society with respect to warnings, and after the event, organization and relief efforts are also connected to the psychological outcome for the victims. One variable to be stressed in this lecture is the response of surrounding family, friends and community to the victims themselves. Should the victims have known better and avoided the site of the terrorist attack? Did they ask for it--did they somehow invite the crime? It is known that the innocence or accidental nature of the victims' behavior is often resisted by surrounding family and community with the leveling of blame statements such as above (Ref. 7). Surrounding noninvolved family and friends may then accordingly shun the victim and isolate them. There is an additional pressure from the noninvolved upon the victims namely that psychological symptoms are seen as an expression of constitutional weakness in the victims. We do know that no one is immune from psychological ill-effects to overwhelming trauma and, probably in the past, prior individual psychological functioning and family functioning have been evoked too often as the cause of their psychological problems after a massive traumatic event.

We do now have a wealth of descriptions of the psychological responses of adults and children to overwhelming Psycho-social Stressors (Ref. 8). We are in a position to offer a framework with which to appreciate how an individual may or may not adaptively cope with such a stress (Table 2).

Table 2

Trauma Response and Recovery Cycle of Victims in General

Trauma response

- Shock
- Disbelief (attempt to deny reality of the event)
- Realization
- Nonaffective or cognitive survival state

Release or Escape

Recovery Cycle

- Shock (upon being released)
- Depression
- Mood Swings
- Anger
- Philosophical Reflection
- Lying-to-rest

(Ref. 15)

Upon suddenly experiencing a trauma, most victims enter a state of initial shock. This is followed rapidly by a period, during which there is a tendency to disbelieve what is happening in an effort to dispel confusion. "This is not really happening to me." This is shortly followed by a period when the cold reality of the situation becomes obvious to the victim. During this phase there is a flat or frozen affect (emotional state or tone). During this phase victims think thoughts such as, "I may not get through this, I may get killed here." Shortly after this phase, the victim's mind becomes concentrated upon survival. It is during this phase that victims may do relatively antisocial acts in order to preserve their own life. Tyhurst (Ref. 10) found during the first phase of reaction to a massive disaster, which he called the period of impact, that twelve to twenty-five percent of the victims were cool and collected, seventy-five percent were stunned and bewildered with restriction in their field of attention and blunting of emotional expression, and that a further ten to twenty-five percent were confused and displaying extreme anxiety, often crying and screaming. Factors which may influence the type of response experienced include the degree of mutilation of one's own or nearby bodies, the victims' perception that they may have just missed death or more severe mutilation, and in the case of assault and rape, attackers who are know to the victim cause more confusion in the victim than does a stranger attack.

This period of the trauma response is ended when the victim is released or escapes. There then follows a recovery cycle. Initially, there is a period of shock which may last hours or days, and it is during this period, as there is little expressed severe emotion, that the victims may be thought to be "all right." This period is followed by a depressed state during which there may be considerable anxiety, too. Anger is hidden at this stage, and may be particularly prominent in assault and rape cases. Following the initial depressed state, there is a period during which the victim's mood swings from one emotional state to another. Anger is often prominent at this point, and may be displaced onto loved ones, friends, or onto the social system surrounding the victim. It is at this point that minority groups may be scapegoated or a strong desire to blame lack of government organization may be seen. It is also at this point that the victim may turn against the people who are closest to them. Hence, their need for an explanation of the victim's anger at this crucial point. Guilt is another prominent emotion, particularly amongst the survivors who may consider what they could have done to have saved more fellow people, or who now may recollect that their determination to survive involved an act of inconsiderateness with respect to another victim during the trauma response phase. It is at this stage also that victims may connect their guilty feelings with previous acts for which they feel badly.

During this phase also, victims may invoke omens to explain the trauma that has occurred to them.

Also, at this stage, bizarre and macabre humour may be observed in the survivors. This phase is followed by a more reflective stage during which there is an attempt rationally, to put the severe traumatic event into perspective. The person will attempt rationalizations at this stage, and try to form a wider view of the event to which they have been subject. This eventually leads to a state where the disaster is relegated to the status of an ugly memory. This does not imply forgetting but involves an acceptance of personal change incurred as a direct result of the event in question. Those who do not negotiate these stages, may well display Post-traumatic Stress Disorder (Ref. 1). Table 3 summarizes the features of Post-traumatic Stress Disorder.

TABLE 3

Post-traumatic Stress Disorder

Re-experiencing the Trauma

- -recollections
- -recurrent dreams
- -recurrence conviction

Numbing of Responsiveness

- -diminished interest
- -detachment
- -constricted emotional tone

Symptoms

- -hyper-alertness
- -sleep disturbance
- -persistent guilt
- -memory impairment
- -phobic avoidance
- -increased symptoms when events resemble the previous
- -trauma

Children differ in important ways from adults in their responses to massive trauma (Refs. 11,12). In particular, they display more use of omen formation, they do not deny the event to the extent that adults do. They engage in post-traumatic play, which is a phenomena rarely seen in adults. They do not experience true 'flashbacks' in the same way that adults do, nor do they experience the post-traumatic shock with its amnesia and haziness to the extent that adults do. It also appears that children have greater difficulty finding meaning after the event and have a greater struggle to place the world in perspective again after a massive trauma.

When children are followed longitudinally after a massive trauma, their disturbances are seen to be persistent (Ref. 13). On follow-up the children are still disturbed in the ways described above but also have a marked pessimism concerning their future life. This pessimism may be general, concerning the future of society, or may be more specific concerning their own doomed future. In the follow-up studies (Ref. 13) all children had psychological symptoms on follow-up, but symptom severity was related to the child's prior vulnerability and prior family pathology.

From these studies, it would seem wise to intervene early to try to prevent post-traumatic stress disorders in the victims of disaster. It would seem unwise to adopt a 'wait and see' policy (Ref. 14). It would seem that a variety of available psychological approaches should be available in an area where disaster has struck. These approaches should consist of counseling and help to victims to ventilate their anger and to put the event into perspective, as well as more intensive psychotherapy for some victims. It is clear that short-term counseling is not enough for some of the victims. Group treatments have not been adequately evaluated in the period following disasters, and would seem on the face of it and in preliminary attempts, to be an encouraging treatment modality. There is also a need for counseling and support to extend to relatives, friends and community members to prevent the effects of secondary victimization. It is significant that in the period following disasters, frequently help is available for adults but there is less than adequate help available for the children. There is no evidence that children suffer any less after massive disasters, and there is similarly no evidence that children are any less in need of open and honest communication concerning their losses and problems. Research is needed to determine if such an approach with a variety of psychological methods available, could be effective in reducing psychological problems after the event. It is significant that with respect to children, one study which did attempt to monitor the long term outcome (Ref. 13), and which did not show any ameliorating effect of short-term psychotherapy, may have shown such a disappointing outcome because there was a delay in providing help to the child victims for six months after the initial event.

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NATIONAL FLOODPLAIN MANAGEMENT PROGRAMS

By

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INTRODUCTION

The riverine and coastal floodplains of the United States provided important sites for economic development in the past and will continue to offer attractive sites in the future. These sites are subject to sharp competition among potential users due to the relatively limited amount of flat land adjacent to water. However, site development requires encroachment into the floodplain and results in two kinds of costs. Life and property are exposed to serious flood losses and the natural water resource and habitat function served by floodplains are degraded or destroyed.

The conflict between the benefits of economic development and the costs of floodplain occupancy pose a persistent national problem: How can floodplains be used wisely? To fully appreciate the nature of this floodplain management problem four subjects need to be discussed: (1) the variable character of the Nation's floodplains; (2) the division of authority in a Federal System of Government; (3) a unified national program for floodplain management; and (4) the current status of floodplain management in the Nation. Discussion of each of these subjects follows.

FLOODPLAINS IN THE UNITED STATES

The amount of floodplains in the United States represents 7.7 percent of the total nonfederal land. Communities located along rivers contain 106,128 square miles of floodplains which represent 6.4 percent of the total land in the communities. Communities located along the coasts contain 40,898 square miles of floodplains which represent 15.6 percent of the total land in these communities.

There is great diversity among the types of floodplains in the United States. Along the coasts are the broad floodplains of the Atlantic Ocean and Gulf of Mexico, the narrow floodplains of the Pacific Ocean, and the varied shorelines of the Great Lakes. Along the rivers are the broad floodplains of the Mississippi River, the narrow floodplains of Appalachia, and the canyons of the arid mountainous west. Each of these floodplains poses different loss reduction problems and advantages for use.

Use of the floodplain for economic purposes extends across the full range of urban activities (residential, commercial, industrial, and governmental) and nonurban activities (agriculture, forestry, and mining). These economic uses compete with one another to benefit from the supply and quality of water and availability of flat land. These economic uses also compete with the undisturbed floodplain's water resources benefits (natural flood storage and conveyance, water quality maintenance, and ground water recharge), living resources benefits (plant and animal populations and nutrient sources for adjacent land areas) and cultural resources benefits (archeological-historical sites, scientific research, and aesthetics-open space).

Decisions to use the floodplain for economic purposes not only result in loss of environmental benefits but also losses of human life and property. Average annual losses of 80 human lives and property valued at $3.4 billion represent a major national problem and point to the need for floodplain management programs to reduce the level of losses associated with floodplain use.

FLOODPLAINS AND A FEDERAL SYSTEM OF GOVERNMENT

To succeed, floodplain management must involve all parties making decisions affecting use of the Nations' floodplains. Under the Federal system of government in the United States, decisionmaking authority is partitioned among the Federal States and local levels of government, and nongovernmental entities. The Federal constitution partitions authority among the Federal and State government and private entities. The State constitutions partition authority among State and local government and private entities. Local charters partition authority among the local government and private entities. Within the private sector, authority is divided among individual citizens and organizational property owners. Consequently, there is extensive fragmentation of authority and a need for coordination in making decisions regarding use of the Nations' floodplains.

The Federal Government

The Congress has gradually expanded the Federal responsibility for dealing with flood losses from a case by case, post-event responsive role to a systematic preventive, as well as post-event role. Prior to the 1930's, the Federal role was to develop flood control projects subsequent to a major flood event. In the 1930's national flood control programs were authorized. In the 1940's expanded disaster assistance programs were authorized. In the mid-1960's recognition that structural flood control methods were not sufficient led to an emphasis on nonstructural approaches and to authorization of a National Flood Insurance Program. In 1977 Executive Orders were issued to coordinate Federal programs in an effort to prevent unwise use of floodplain and wetlands. A similar sequence of authorizations creating a National Park Service, a Fish and Wildlife Service and a National Environmental Policy Act established a Federal responsibility for protecting against the loss of environmental values.

The consequence of incremental expansion of Federal authority for reduction of flood losses and environmental losses is a multiplicity of Federal Programs. Eleven agencies operate under 44 major legislative authorizations. For example, ten agencies provide technical and planning services, eight agencies carry out floodplain management studies, six agencies construct flood control structures, four agencies operate research programs, three agencies carry out space-environmental preservation programs and one agency is responsible for flood forecasting and warning. Implementation of most of these programs requires a request from a State or local government.

The obvious need for coordination among Federal agencies has led to acceptance of common standards and cooperative interagency activites. By executive order all agencies utilize the one percent chance (100-year) base flood standard and prescribed flood hazard evaluation - avoidance procedures. Interagency technical groups develop standard hydrological procedures and floodplain management cooperative programs. The most recently established coordination mechanism is the interagency post-flood disaster hazard mitigation team which recomends reconstruction activities to reduce future exposure to flood losses.

State Government

Each of the 50 States has their own constitution and enabling laws which permit regulation of floodplains. Prior to 1973 state commitment of resources to floodplain management varied greatly. Since the Natural Flood Insurance Program became operative in 1973, each State has identified a floodplain management coordinator and most have developed expertise and a program to provide technical assistance to local government. Because states have fragmented authority similar to that of the Federal government, technical expertise and leadership may be vested in a technical unit under natural resources, urban affairs, public works or emergency services departments. Coordination among states is fostered by an association of State floodplain managers and state coastal zone managers.

Local Government

Approximately 20,000 communities have been identified as having flood-prone lands. Of these communities, 17,500 participate in the National Flood Insurance Program wherein the Federal government makes available flood hazard maps and flood insurance if a community adopts and carries out floodplain zoning and building codes designed to reduce future losses. Many of these communities also have wetlands or environmental protection laws. The community's power to regulate development is the critical element in national floodplain management.

Since 1973 when a major hurricane led to the Federal requirement that to borrow Federally-insured funds, flood-prone properties must have flood insurance, community awareness and capability has grown tremendously. Community officials know that they must adjust economic development plans to accommodate flood loss reduction.

Nongovernmental - Private Parties

Again, largely because of the National Flood Insurance Program, nongovernmental parties are increasingly aware of the need to carry out floodplain management. Over 2 million flood insurance policies are in force. Plans for the use of floodplains are frequently modified or aborted because of floodplain regulations. The basic conflict between private gains objectives and social cost reduction objectives is recognized by most private parties considering action in or affecting floodplains.

Issues of Loss Reduction in a Federal System

Three pervasive questions are inherent in floodplain management in a Federal system (Figure 1). Given fragmentation of authority, who has the authority for a particular loss reduction activity? The Federal government? The State goverment? The local government? A private entity? Some combination of these?

Once authority has been established, the question becomes: what justification criteria are to be used to determine if the authority is to be exercised? Economic criteria such as benefit/cost ration or cost effectiveness? Social criteria such as preservation of life, health, and safety? Political criteria such as national, regional or local equity?

The final question is often the most difficult: How will payment be made for the loss reduction activity? Who will pay and under what cost sharing ration? Will payment be made before, during, or after the loss reduction activity is provided? Will the form of payment be cash, in-kind services, or some other means?

These questions of authority, justification and payment must be answered before floodplain management can be carried out.

FEDERAL EMERGENCY MANAGEMENT AGENCY

Issues of Loss Reduction in a Federal System of Government

Who has the authority?	**Federal/State/Local/Private?**
What justification criteria?	**Economic/Social/Other?**
What kind of payment?	**Cost Share/Timing/Form?**

Figure 1.

A UNIFIED NATIONAL PROGRAM FOR FLOODPLAIN MANAGEMENT

In enacting the National Flood Insurance Act, the Congress recognized the problem of divided authority at the Federal level and directed the President to develop a Unified National Program for Floodplain Management. The Unified Program (Figure 2) sets forth a conceptual framework based on the promise that the partitioning of authority between and within each level of government and private entities requires acceptance of a common goal and approach to achieve floodplain management. It establishes the goal that use of the Nation's floodplain must be compatible with the risk of loss. It recognizes two constraints on floodplain use: (1) that the risk of human and economic loss from flooding must be mitigated, and (2) that the risk of environmental loss from development must be mitigated. Finally, this conceptual framework asserts that effective loss reduction must be based upon selection of the best mix of loss reduction strategies and tools for any specific floodplain situation; no one strategy or tool is inherently superior to another.

Flood Loss Reduction Strategies and Tools

There are three basic flood loss reduction strategies, each supported by a number of tools (Figure 3). The first strategy is to modify the susceptibility to flood damage and disruption by taking action before flood conditions develop. Regulation of floodplain use and structures through State and local ordinances is the most powerful tool available. Regulation is closely supported by development and redevelopment policies to guide provision of public services and infrastructure away from hazardous locations. Other tools supporting this strategy are floodproofing of exposed structures, establishment of articulated flood forecasting and warning systems and emergency plans, and establishment of disaster preparedness and assistance programs.

The second strategy frequently referred to as the structural approach, is to modify floodwaters so that their level, velocity, or location results in fewer losses. Tools for reducing the runoff of water include dams and reservoirs, which affect stream flow, and land treatment measures and on-site

FEDERAL EMERGENCY MANAGEMENT AGENCY

Unified National Program for Floodplain Management

Premise:

Fragmentation of responsibility requires a common goal and approach to floodplain management

Goal:

Use of the nation's floodplains compatible with the risk of losses

Constraints:

Risk of human and economic loss from flooding

Risk of environmental Loss from development

Approach:

Achieve the best mix of loss reduction tools and strategies for individual floodplains

Figure 2.

FEDERAL EMERGENCY MANAGEMENT AGENCY

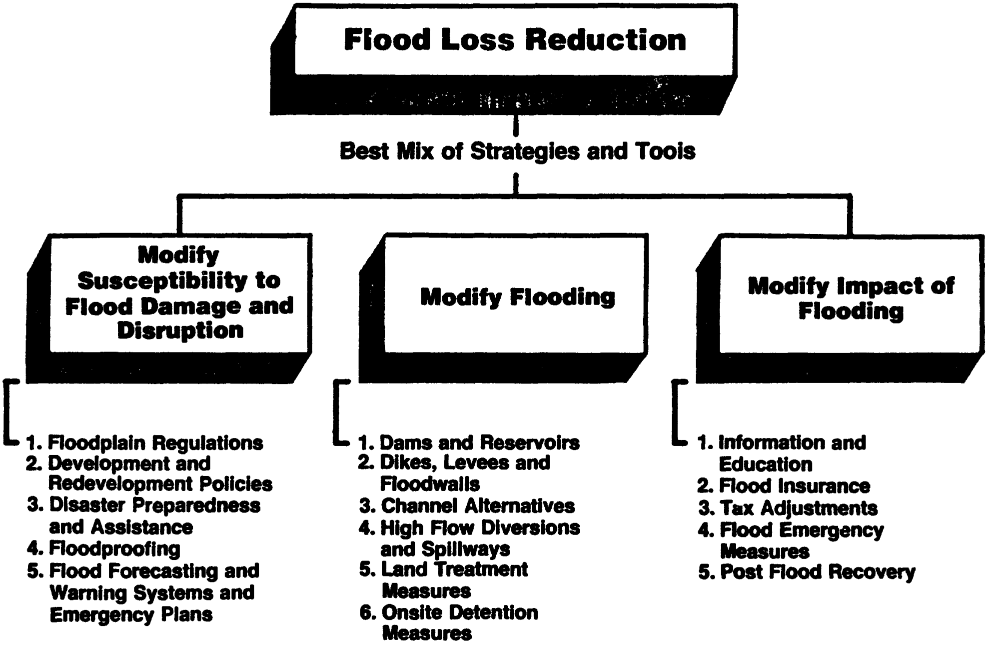

Figure 3.

detention measures which delay water from reaching streams. Dikes, levees and floodwalls hold passing floodwaters away from structures located in the floodplain. Channelization, high-flow diversions and spillways relocate the flow of water away from development in the floodplain.

The third flood loss reduction strategy is to modify the impact of flooding once it occurs. Informed local officials and citizens can take actions to protect lives and property. Flood emergency measures such as flood fighting with sandbags or temporary dikes often prove effective. Once floodwaters recede, flood insurance payments, tax credits, and post-flood recovery measures may spread the burden of losses from individuals to segments of the public and initiate action to mitigate against future losses.

Environmental Loss Reduction Strategies and Tools

The two basic strategies for reducing environmental losses are the preservation of natural values and the restoration of natural values (Figure 4). Natural values are the functions served by floodplains in their natural or relatively undisturbed state. As indicated previously, these are water resource values, living resource values, and cultural resource values. Preservation focuses on the immediate impacts of proposed floodplain actions. Restoration focuses on reestablishment of floodplain conditions degraded or destroyed by prior actions.

Both the preservation and restoration strategies are supported by the same five tools. Floodplain regulations contain many provisions for flood loss reduction that serve environmental loss reduction and in many cases are supplemented by wetland protection and other environmental regulations. Development and redevelopment policies, information and education, and tax adjustments are tools that serve environmental and flood loss reduction equally well. Administrative measures represent opportunities for all levels of government where program and authority permit placing restrictive conditions in contracts or permits, encumbering use of land during conveyance of title, delegating leadership for loss reduction activities to a specific office, and specifying stream and coastal areas most suitable for protection and restoration.

FEDERAL EMERGENCY MANAGEMENT AGENCY

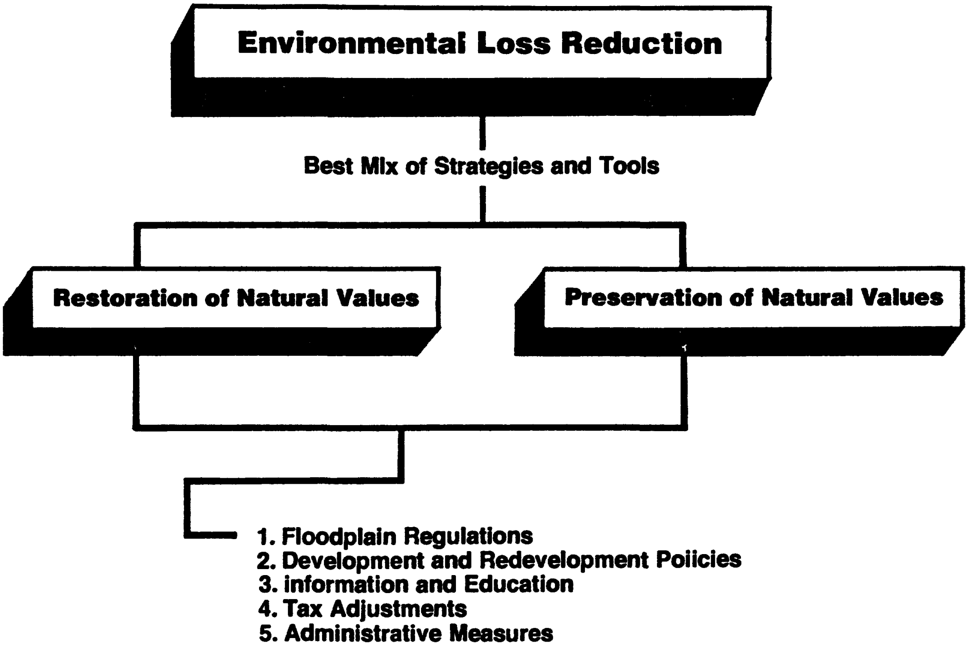

Figure 4.

CURRENT STATUS OF FLOODPLAIN MANAGEMENT

In spite of more than 50 years of Federal programs and the development of Federal leadership in efforts to reduce losses, both the actual and potential flood and environmental losses are unacceptably high. The Nation is faced with potentially catastrophic flood losses, not only from hurricanes along the Gulf and Atlantic coasts but also from flash flooding on the alluvial fans of the western States. Average annual losses from sub-catastrophic floods average $3.4 billion. Similar losses of natural floodplain values are high as indicated by the annual destruction of 450,000 acres of wetlands.

Nevertheless, considerable progress has been made, especially during the last decade. Through the Unified National Program for Floodplain Management a common management goal and concept has been established. Intergovernmental integration of programs has progressed as exemplified by national acceptance of the 100-year base flood standard. The National Flood Insurance Program has brought about establishment of local floodplain regulations in 17,500 communities, coordination mechanisms in all 50 States, and expanded Federal and State technical assistance capability. Program coordination and leadership within the Federal and State governments has been established by floodplain management executive orders and directives. Consequently, proposals to use floodplains are being reviewed to ascertain potential flood and environmental losses. Some proposals are being rejected and others modified to mitigate future losses. While new actions are subject to floodplain management considerations, unwise decisions of the past continue to confront the Nation.

Clearly, important needs remain to be addressed. In general, better implementation of existing programs is needed rather than new legislation. Emphasis needs to be placed upon developing more effective means for balancing short run and long run objectives for floodplain use. Increased resources must be committed to providing the technical assistance needed to strengthen local government programs. The uniquely favorable conditions of public attitude and resource availability associated with post-flood disaster recovery must be utilized to achieve long-term loss mitigation.

In overview, given the diversity of floodplain conditions, the large number of unwise past floodplain use decisions, the context of our Federal system of government, and the progress attained in establishing the concept of a unified program and essential floodplain management programs, it must be concluded that another 20-30 years will be required to achieve full implementation of existing floodplain management tools and the reduction of losses to an acceptable level.

ELEMENTS OF A COMPREHENSIVE FLOODPLAIN MANAGEMENT PROGRAM[1]

By

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INTRODUCTION

Floodplain management can be defined as all those actions taken to reduce or eliminate flood damage potential within the floodplain. A comprehensive floodplain management program should consider all available options for both remedial and preventive activities to insure that the "best" options are implemented. The experiences of the Urban Drainage and Flood Control District, Denver, Colorado, have been drawn upon to form general conclusions as to how to develop and implement a comprehensive floodplain management program.

The National Flood Insurance Program (NFIP) has had a significant impact on floodplain management in the United States, but, as important as the NFIP is, it offers only a partial solution to the floodplain management problem. Local governments, which have the primary responsibility for floodplain management in most states, must be made aware of the other options available to them which, in combination, allow the formulation of a comprehensive floodplain management program which is capable of actually reducing their flood damage potential.

BACKGROUND

The 100-year flood (called the base flood by the NFIP) has become the accepted design standard for floodplain management in the United States. The

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[1] Presented at American Society of Civil Engineers Annual Convention and Exposition, Houston, Texas, October 21, 1983.

100-year flood has a one percent chance of being equalled or exceeded in any year. The 100-year floodplain is the area which would be inundated by the 100-year flood. The 100-year floodplain is the area of primary concern to the NFIP.

The concept of the NFIP is to shift the loss burden from the general public to the occupants of the floodplain while managing new growth to limit the increase in flood damage potential. The NFIP makes flood insurance available within those local governments which enter the program and agree to implement certain minimum land use requirements within the 100-year floodplain. These land use requirements, if properly implemented, can be effective in preventing new flood damage potential from being located within the 100-year floodplain (or at least below the 100-year flood elevation). However, these requirements are not very effective in reducing existing flood damage potential. The NFIP relies on flood insurance payments and a mitigation program following a flood disaster. Local governments which want to reduce the flood damage potential before a flood disaster occurs must expand their floodplain management programs to include remedial actions. The Federal Emergency Management Agency (FEMA) should be consulted for details concerning the NFIP.

OBJECTIVE OF A COMPREHENSIVE PROGRAM

The objective of a comprehensive floodplain management program should be the absolute reduction of flood damage potential. This can only be accomplished by: (1) preventing an increase in flood damage potential resulting from new development in floodplains, and (2) reducing the flood damage potential in already developed floodplains. A program which combines these two actions, preventive and remedial, will be able to reduce the flood damage potential in a community. A community that relies on preventive approaches alone will only be able to reduce its flood damage potential very slowly as re-development of floodplain land occurs. A community that relies only on remedial efforts, without regard to how new development occurs in the floodplain, may find itself with little or no reduction in flood damage potential, even after the expenditure of significant amounts of money. Both approaches must be used if the objective of reduced flood damage potential within a community is to be realized.

PREVENTIVE APPROACHES

Preventive floodplain management approaches usually consist of land use controls, such as floodplain regulations and subdivision regulations, which are applied to the 100-year floodplain. Briefly, the procedure is to define the 100-year water surface elevations, flood outlines and floodway. The floodway is the channel and the portion of the adjacent floodplain required to pass the 100-year flood without significantly increasing the water surface elevation, assuming the remainder of the floodplain is not available to convey flood water. The NFIP criteria, which allows a one foot rise in water surface elevation, is probably the least restrictive (narrowest floodway) standard in common usage. The remainder of the floodplain (often called the flood storage area or flood fringe area) is then available for development. NFIP criteria requires residential structures in the flood storage area to be elevated to the 100-year water surface elevation and non-residential structures to be elevated or flood-proofed to the same elevation. Many local jurisdictions require structures to be elevated or flood-proofed one or two feet above the 100-year water surface elevation.

Once the floodplain and floodway are defined and subject to regulation potential developers have the option of leaving the floodplain in open space (perhaps as public or private park and recreation areas), developing the fringe area as described above, or modifying the floodplain or floodway (using options such as channelization) to remove areas from the floodplain. Development which occurs under any of these options will be essentially free from major flood damage up to and including the 100-year flood event.

It is important to work with developers to make them aware of the flood hazard, the need for addressing the hazard and the options available to them as noted above. Experience has shown that the major developers, after some initial consternation in dealing with land use controls on floodplains, have recognized the need to address the flood hazards and have exercised the options available to them to build quality developments safe from flooding during the 100-year event.

Another preventive approach is acquisition of floodplain land, or at least the development rights to the land. From a practical standpoint, acquisition usually requires a multiple use approach in order to justify the expense of acquisition. Multiple uses can consist of parks and recreation, preservation of open space, preservation of environmentally unique or sensitive areas or other similar uses.

REMEDIAL APPROACHES

In developed floodplain areas, where a high flood damage potential already exists, simply applying land use controls to defined floodplains will not have an immediate impact on the flood damage potential, and additional actions must be implemented if the goal is to reduce a community's flood damage potential. Remedial floodplain management involves the planning, design, construction and maintenance of facilities to reduce the flood damage potential in an already developed floodplain. The remedial options available include construction of flood control works (such as improved channels and detention facilities), retro-fit flood-proofing of existing buildings, flood detection and warning systems, acquisition and relocation or demolition of structures, and public awareness programs. Brief discussions of the most commonly used remedial options, including advantages and disadvantages, are given below.

Channelization

The construction of open channels is a commonly used method of reducing the size of a floodplain or floodway. Channels can be lined with grass, riprap, wire-enclosed rock or concrete. In western areas grass lined channels may be irrigated if the objective is to have an active park usage or native seeded if the usage is less intense. Open channels allow water to enter them at almost any point thus compensating for inadequate tributary collection systems. The design of each type of lined channel should follow generally accepted criteria for depths, velocities, side slopes, and maintenance requirements. It should be recognized that channelization tends to reduce valley or floodplain storage and may therefore increase downstream flood peaks. The design of channels should include continuous adjacent maintenance trails and maintenance access points. These trails can double as hiker/biker facilities, thereby providing a multiple use benefit.

Grass-lined channels are the most aesthetically pleasing type of channel and also provide additional multiple use possibilities such as playgrounds, picnic areas and open space. These channels require the largest amount of right-of-way. They also require, at least in the Denver area, drop structures to control the longitudinal channel slope, and hence the velocity of the flood waters. Grass-lined channels require continuous routine maintenance including mowing the grass, controlling weeds, removing trash and debris and repairing erosion. The design of these channels should include a trickle channel to confine normal dry weather flows to a small area, thus keeping the remainder of the channel dry for maintenance and multiple use opportunities.

Riprap or wire-enclosed, rock-lined channels can withstand higher velocities, thus requiring less right-of-way and fewer drop structures. The higher velocities can pose a greater hazard to anyone caught in them. These channels have limited multiple use potential beyond the maintenance trail. Grass and weeds growing through the voids in the rock must usually be controlled by labor intensive hand work. Trash and debris collect in the voids and the wire enclosures and is harder to remove. Wire enclosures are subject to corrosion, abrasion and vandalism; and loose riprap is also subject to vandalism. These channels are generally less aesthetically pleasing.

Concrete lined channels generally require the least amount of right-of-way and also require the least amount of routine maintenance. They are the least aesthetically pleasing, usually provide no multiple use opportunities beyond the maintenance trail and present the greatest velocity hazard. They usually have to be fenced for safety reasons. Great care must be exercised in the design and construction of these channels.

Detention Facilities

Major on-stream detention facilities have their greatest value when placed immediately above already developed floodplain areas. Detention facilities store flood waters and release them at lower rates, thus reducing or eliminating the need for major downstream flood control facilities, the construction of which would disrupt the developed areas.

Perhaps the greatest disadvantage of detention facilities, assuming a structurally sound facility, is the false sense of security created by the structure among the general public. These facilities are almost never designed to contain the probable maximum flood. Thus they require a spillway to pass discharges in excess of the design flood. Oftentimes the design flood is the 100-year flood, since this frequency has become the standard for floodplain management, as noted earlier. Unfortunately, the public assumes the detention facility has eliminated any flood hazard, and consequently they are totally unprepared for the possibility of a flood which exceeds the design capacity of the facility.

Detention facilities built above undeveloped floodplain areas can actually induce increased flood damages. The 100-year floodplain immediately downstream from such a facility will be quite small and normal floodplain management activities will allow for intense development of the pre-detention floodplain. Then when a flood larger than the design capacity of the facility occurs, extensive damage can result which would not have occurred if the detention had not been built, and the pre-detention floodplain had been managed to control development. It is recommended that in such cases, downstream capacity for the routed discharge from a flood in the 250- to 500-year frequency range be reserved as an intregal part of the detention facility design.

Detention facilities offer many opportunities for multiple use. Normally dry flood pools can be used for athletic fields, playgrounds, tennis courts and open space. A permanent pool water feature is also a possibility. Routine inspection and maintenance is essential to insure the continued safety, stability, and operational capability of the facility.

Floodproofing

Retrofit floodproofing of existing buildings is often a viable solution, depending upon the location of the structure within the floodplain and its structural integrity. There are two basic types of floodproofing: active and passive. Active floodproofing is temporary and requires some positive action on the the part of building owners and/or occupants immediately before a flood event, and will not be effective if personnel are not available to perform the necessary functions when the flood occurs. Passive floodproofing is permanent and does not require any action at the time of the flood.

Active floodproofing requires some type of flood detection and warning system to give time for the personnel to install the floodproofing devices. In flash flood situations the personnel may not be available to respond in time. Therefore active floodproofing is most effective in areas with long warning lead times, and should not be relied upon, if possible, in flash flood areas.

As a practical matter floodproofing should be the financial responsibility of the structure owners. It is hard for government to justify the expenditure of public funds on private properties. It is also difficult for government to maintain floodproofing on individual properties, and government should not construct anything it can't maintain. Floodproofing should be designed by experienced engineers knowledgeable in the hydrostatic, hydrodynamic and bouyancy forces associated with flood waters.

Flood Detection and Warning Systems

Flood detection and warning systems can be effective in reducing loss of life and property damage. In flash flood locations the major benefit will be reduction in loss of life. The short lead times limit the amount of active floodproofing that can be accomplished. In slow rising flood situations major savings in flood damage can be accomplished.

Flood detection systems can range from inexpensive networks of volunteer rainfall and stream stage observers and simple rule curves to sophisticated networks of telemetered gages and computer models. The National Weather Service has a definite role to play in this area and should always be consulted and included in any detection and warning system.

Methods for warning the public should be well thought out, documented, and practiced on an annual basis. Ways to desseminate warnings include the Emergency Broadcast System, NOAA Weather Radio, commercial radio and television, cable television, warning sirens and public address systems. Users of detection and warning systems should be aware that all members of the public will not respond to warnings in the desired manner. An understanding of how and why people respond to warnings is an essential ingredient in any warning system.

Acquisition

Remedial acquisition consists of the acquisition and relocation or demolition of high risk structures in the floodplain. The cost of this alternative is high because full value must be paid for the structure and the land. Sometimes some money can be recouped by selling the structure to someone willing to move it or salvage materials from it. Acquisition can be better justified if the land can be put to some beneficial public use compatible with the flood hazard. In these cases funding assistance from a source such as an open space or recreation fund may be available. Following a flood disaster funding assistance for acquisition of damaged properties may be available through the Federal Emergency Management Agency and the National Flood Insurance Program.

Public Awareness Programs

Public awareness programs should be implemented for the following purposes: (1) to make floodplain occupants and/or owners aware of identified flood hazards; (2) to encourage individuals to take actions to mitigate their flood potential such as purchasing flood insurance, floodproofing and developing escape plans; (3) to make individuals aware of the existence and operation of flood warning plans; and (4) to encourage individuals to keep drainageways clean and to report potential maintenance problems.

Legal counsel for the Urban Drainage and Flood Control District has advised the District that it has an obligation to inform floodplain occupants of known (identified) flood hazards. In response to that advice the District has developed informational brochures which describe the flood hazard (including a map of the 100-year floodplain) and potential mitigative actions individuals can take. These brochures are mailed to every address in every identified floodplain on an annual basis. A study of the effectiveness of these brochures indicated an increase in awareness of the flood hazard as well as an increase in mitigative actions taken. While limited in its potential effectiveness, this approach has significantly increased public awareness of flood hazards.

MASTER PLANS

The basic guide, or road map, which provides local community officials, land owners, and developers the information necessary to manage the floodplain is the master plan. The master plan documents the floodplain, floodway, discharges and proposed preventive and remedial actions to be taken to reduce the flood damage potential.

Each drainageway should be master planned for its entire length through the urban area as well as adjacent areas where future urbanization is anticipated. All local jurisdictions affected by the drainageway should be actively involved in the planning process, and the final product should be one that is acceptable to all jurisdictions. A state or regional agency can be very helpful in coordinating the formulation of a multi-jurisdictional master plan. Each local jurisdiction should contribute financially to the master plan because it has been found that a financial commitment heightens local jurisdictions' interest in the formulation and implementation of master plans.

The master planning process consists of the following steps: (1) obtain good mapping of the study area (mapping at a scale of 1" = 100' with a two foot contour interval has been found to be very useful in most cases), (2) develop hydrology for several frequencies including the 100-year (the other frequencies are used in the benefit cost analysis), (3) delineate floodplains for the frequencies for which hydrology was developed, using existing channel and floodplain conditions (4) estimate flood damages for the various frequencies and develop flood damage-frequency curves and average annual damages (assume that floodplain regulations will prevent new flood damage potential from being introduced into the floodplain), (5) conduct a review of all possible flood damage reduction alternatives; such as dams, channels, conduits, floodproofing, acquisition, etc., to attempt to eliminate from consideration those alternatives which are obviously inappropriate, (6) prepare preliminary designs and cost estimates for the remaining alternatives and delineate residual floodplains for the frequencies being used, (7) determine residual flood damages for each alternative, (8) complete a benefit-cost analysis for each alternative, (9) review each alternative for other factors such as political considerations, multiple use opportunities, environmental factors, etc., (10) select an alternative or combination of alternatives acceptable to each affected jurisdiction, (11) publish a master plan report with sufficient documentation of the above process, a preliminary design of the selected alternative, cost estimates, and the existing and proposed 100-year floodplains.

The hydrology prepared in step (2) above should be based on future basin conditions in terms of development. It is important that master planning be based on future conditions and not be made obsolete as development in the basin takes place that increases runoff peaks and volumes.

Several other factors should be considered in the master planning process. While the federal government puts heavy emphasis on the benefit-cost analysis, local governments are not similarly constrained. Local governments are free to base their alternative selection on other factors including political or environmental considerations, least cost or, conversely, a desire to eliminate a hazard regardless of the cost.

It is also possible to select an alternative for a higher frequency than the 100-year event, such as a 10-year channel. In this case the average annual flood damages will be reduced and frequency of occurrence of flood damage will be lessened, but the residual 100-year floodplain will still have to be regulated or managed. This is a viable solution, particularly when the costs for full 100-year protection are clearly out of reach.

Formulation of a master plan can take from one to two years depending on the complexity of the problems and the number of local jurisdictions involved. On the other hand the delineation of the 100-year floodplain is a straightforward process that can be accomplished fairly quickly, relatively speaking. It is therefore often desirable to publish a flood hazard area delineation report separately as soon as the floodplain and floodway have been defined. Early publication of the floodplain allows local jurisdictions to begin their preventive floodplain management activities before the master plan is complete.

The elements which should be a part of every master plan are land use controls (floodplain regulations) governing the development of the 100-year floodplain, community participation in the National Flood Insurance Program (NFIP) and a public information program.

USING THE MASTER PLAN

Once a master plan and/or a flood hazard area delineation report is completed, it should be adopted by the local governments and should be an integral part of the communities' land use planning and development review processes. Every development proposal in or near the floodplain should be required to conform to the master plan. Utilization of the master plan is further described below.

Preventive Use

Implementation of the preventive aspects of the master plan involves adoption of the floodplain and the imposition of floodplain regulations as described earlier. Then every development proposal within the floodplain must either conform to the requirements of the floodplain regulation (elevating, flood proofing, etc.) or the developer may want to revise the floodplain by structural means, such as channelization. It is very important that the local government either have staff expertise to review development proposals or seek help from a regional or state agency which has the expertise.

If the master plan calls for acquisition of floodplain land, that land will be identified in the plan, and a review of a proposed development in relationship to the master plan will allow for identification of a potential conflict. The local government can then proceed to acquire the land through purchase, donation, or dedication. It is important to remember that a floodplain regulation cannot be used to acquire land. It can only be used to regulate the private use of land. A local government that wants floodplain land for public use must acquire title to that land.

The master plan can also be used to prevent proposed development from creating new obstructions to implementation of remedial works. Here again the review of the proposed development in relationship to the master plan can identify conflicts between the two, and steps can be taken to prevent the conflict from being constructed.

Remedial Use

The master plan forms the basis for the design and construction of remedial facilities to correct existing problems. Once a preliminary design has been agreed upon by all affected local governments, the process of implementation can begin. Unless a detention facility is involved which requires joint funding and construction by two or more local governments, each local government can usually implement the remedial aspects of the master plan at its own pace.

One technique that has worked well in the Denver area is to complete the construction design for the entire project through a local government, and to then divide the project into several schedules that are manageable from a financial standpoint. The schedules are then constructed over a period of years as funding permits. Construction of the initial schedules may not provide the ultimate degree of protection but at least the community is progressing toward the total solution.

OTHER CONSIDERATIONS

While the master plan forms the basis for a comprehensive floodplain management program many other elements must be considered and included if the program is to be successful. These elements include leadership, cooperation with and between local governments, criteria, funding, and technology development. Of great importance is how these elements are put together and what facilitates their interaction. The final product can be a well integrated and functioning program.

Leadership

Every program requires leadership. The question here is who provides that leadership. Effective leadership is not simply the imposition of standards, criteria, or regulations on the public. Rather it is working with the public, it is cooperation between governments and it is the development of sound standards, criteria, and regulations. The trick is to find and/or establish leadership for the floodplain management program.

Leadership can come from the local, regional, state, or federal levels. Generally, the best leadership will come from the level of government closest to the people; consistent with other factors such as size and expertise of the staff, number of local jurisdictions affected by a drainageway, and technical assistance available from other government agencies. For example, in a metropolitan area consisting of many local governments, a county or regional flood control district will probably be in the best position to provide the necessary leadership. The bottom line is that without effective leadership, no floodplain management program will be as effective as it could be.

Cooperation With and Between Local Governments

Usually the general purpose government closest to the people will be the most efficient in managing the floodplain. Local governments have all the tools necessary to implement a floodplain management program; including zoning and other land use controls, subdivision regulations, building codes, land dedication requirements, etc. The responsibility of regional or state agencies is to assist, to provide technical support, to act as a resource, to educate, to monitor critical problems, and to help resolve multi-jurisdictional problems.

As noted earlier, cooperation between local governments is essential in developing master plans. It is also essential in the design, construction and maintenance of flood control facilities and the implementation of floodplain regulations. Again, in metropolitan areas, state or regional agencies can be very important in facilitating cooperation between local governments, and also can step in to prevent irresponsible actions by a local government, particularly when thev affect other jurisdictions.

Criteria

Published criteria establish a consistent basis for implementation of the various floodplain management alternatives. The criteria must be technically sound, and can also establish the philosophical basis to be pursued by the local government. For example, one community may want to encourage preservation and open space uses of the floodplain while another community may opt for structural alternatives due to limited available development areas.

The development of floodplain management criteria that are eventually accepted within the community requires teamwork, salesmanship, and public relations skills. It is important to involve key political figures, decisionmakers, legal professionals, community leaders and technical people. Only through this involvement of a broad spectrum of people will the final criteria gain wide acceptance.

The criteria should be clear, easy to understand, specific, and leave only a limited range for assumptions and interpretation. The criteria may sacrifice room for innovation but it is important to remember that most of the people likely to be using the criteria, both local government staffs and developers, are not specialists in the fine points of hydrology and hydraulics. Consequently these people depend heavily on the technical guidance and requirements contained in clear and concise criteria. Experts who can go beyond the simplified criteria will still be needed for major design projects.

Funding

Funding for floodplain management programs and facilities typically has low priority at the local and state level. Floodplain management must compete for funds from the same limited resources that fund a myriad of other government services. It behooves the professionals in this field to convince the politicians that floodplain management is an integral and important part of community services. As we urbanize, we need roads, electricity, gas, water, and sewers to have a viable community. We also need floodplain management facilities. They need to be planned, designed, constructed, and maintained. If any of these elements are not properly funded, the effectiveness of floodplain management is compromised, often creating greater long term costs to the public sector.

Planning must be funded so that basinwide plans can be developed to guide orderly development or re-development along the major drainageways. In the Denver region it has been found that planning projects work best if they are cooperatively funded by all affected local governments and by the Urban Drainage and Flood Control District, a regional organization. Although the regional organization takes a leading role in these projects, not a single local government is ignored or is left out of the process. Cooperative funding goes a long way in insure mutual cooperation. It also makes the burden easier to bear for each local government.

Floodplain management programs provide the linkage between the planning documents and the actual implementation of the plans. These management programs need funds to develop information on floodplains, to document drainage problems, to monitor development activities, to review development proposals, to meet with involved parties, and to provide all other services that significantly reduce the potential of past mistakes being repeated. Funding for these activities is needed at all levels of government; however, economies of scale can be achieved if technical review assistance and criteria guidance are provided on a regionalized basis, preferable on a voluntary cooperative basis.

The implementation of projects to remedy existing flooding problems requires the commitment of large sums of capital. It is not necessary to solve all problems at the same time. However, there is a need to have an orderly process and firm funding sources that work at solving problems on a continuous basis. Problems in newly developing areas can be prevented by preventive floodplain management and by having the growth areas provide required facilities. Existing problems on the other hand can usually only be solved through public funding. The burden of funding drainage and flood control projects falls primarily on the local, regional and state governments. Federal funding has never been reliable. Unless the projects are large enough to involve the U.S. Army Corps of Engineers or the Soil Conservation Service, the state or its local governments have to come up with the necessary funds. If the drainage problems are clear and citizens demand solutions, the local political process will come up with the money to get the job done.

Another element of floodplain management that requires a funding commitment, but is very often left unfunded, is maintenance. Maintenance seems to be an area where many local governments cut when budgets get tight. Maintenance is too important to be neglected, and funding should be vigorously sought for this purpose. Here again regional agencies can sometimes assist in this area.

Technology Development

When creating a viable floodplain management program we have to include the development of new methods and techniques in the areas of hydrology, hydraulics, public administration, construction, maintenance, quality control, and all other fields and services that are a part of stormwater management. The universities play a major role in the development of technology; however, they need funding and direction from operating agencies. The operating agencies, also need to be directly involved in technology development. In this way universities can draw on the experiences of the operating agencies to direct their research to areas of greatest need. The operating agencies, on the other hand, need to take what is coming out of the universities and try to apply it. By sharing what we are learning at all levels, our technology development progresses efficiently.

The last decade has been a period of tremendous growth in technology. We now have hydrologic and hydraulic computer models that are available to all engineers; the amount of urban rainfall and runoff data has increased by two to three orders of magnitude; we now regularly question the sacred cow institutions of "design storm" and Rational Formula; and are now beginning to explore such topics as spatial rainfall distribution, system effects of random on-site detention, and real time hydrologic data acquisition, and use in operating urban drainage and flood control systems. It is our responsibility to insure that the developing technology is put in a form that makes it available to everyone that wants to use it. We are also now faced with an unprecedented growth in computer hardware and software technology. Computer technology now permits us to model very small discrete elements of the hydrologic and hydraulic systems, yet the data and information needed to accomplish this are not available. As a result, the computational technology development is outracing our ability to use it reliably. Despite the fact that our data base has expanded two to three orders of magnitude, we will need to expand it even more if we are to take full advantage of the growing computing technology.

OWNERSHIP AND MAINTENANCE

Ownership and maintenance responsibility for flood control facilities and floodplains are critical issues that cannot be overlooked without serious consequences. This is particularly true for floodplains and flood control facilities developed by developers. Obviously developers don't want to, and realistically can't, own and maintain these facilities indefinitely. Therefore, they will want to transfer ownership and maintenance responsibility to some other entity; be it local government, a homeowners' association, or some other public or quasi-public body.

Local governments are often reluctant to accept ownership because of the accompanying maintenance costs. In these cases it is not unusual for a home owners' association, which doesn't even exist when the decisions on ownership and maintenance are made and therefore can't participate in the decision, to be given the ownership and maintenance responsibility. Major drainageways function as public utilities, just as water and sanitary sewer systems do, and they should be similarly owned and maintained if at all possible.

When developers propose to reduce the size of the floodplain by constructing flood control facilities, such as channels, they have a tendency to propose facilities that are low in capital cost but high in maintenance cost. Since the developer will eventually leave the facilities in the hands of another entity, great care should be taken by the local government to insure that maintenance costs will not be excessive, even if it means a higher capital cost to the developer.

Local governments which construct or accept flood control facilities have an obligation to maintain these facilities. Legal counsel in the public sector have been advising for a long time that if you do not plan to maintain it, don't build it. Obviously, their opinion is intended to minimize exposure to liability. It is based on the premise that when a publicly-owned facility is responsible for loss of life or causes property damage due to lack of maintenance, the public entity that owns it is liable for the damages. What is equally important is that unmaintained facilities deteriorate, and rebuilding these facilities is much more expensive that the routine maintenance needed to keep them functional. As an example, it is suspected that a major bridge across the Aquafria River in Arizona was lost because the gabion protecting the bridge abutment, damaged a year earlier, had not been repaired. An inexpensive repair of the gabion may have not only saved several million dollars to rebuild the lost bridge, but would have also prevented the loss of three lives.

Another aspect of maintenance that pays back dividends is public relations. Trash and litter are removed, weeds and tall grass are kept in check, drainageways are kept open, and public access is improved. Maintenance access trails can also serve hikers, bikers, and joggers at virtually no additional cost. People become protective of well-maintained facilities that are accessible to them, thereby increasing public support of floodplain management programs. A good maintenance program is one of our greatest allies in the building of support for floodplain management programs.

MULTIPLE USE

Every master plan study should consider multiple use possibilities of flood control alternatives. Channels can double as linear parks, greenbelts and trail systems. Detention facilities can be used as recreation areas. Acquired floodplains can be used for parks, open space, nature areas, land disposal of sewage effluent, etc.

Multiple use provides the opportunity to use the flood control facilities for other activities the majority of the time. Multiple use can also provide more benefits for the money spent and/or can allow for funds to be obtained from more sources. Multiple use facilities provide greater benefits to the community.

SUMMARY

The most important point made is that floodplain management must consist of several actions that when taken in concert effectively reduce flood damage potential. There is a vast array of individual actions that can be taken, which can be categorized as preventive or remedial. The common denominator of any floodplain management program, however, is floodplain regulation and involvement in the NFIP.

It is important to combine both remedial and preventive actions in developing a floodplain management program. The specific actions that are implemented and the approach to their implementation will vary from community to community. It is necessary that each community identifies its needs, resources, and objectives and develops a floodplain management program accordingly.

The basic guide that enables local government to implement a comprehensive floodplain program is a drainageway master plan. A master plan provides documentation for identifying the problems, defining the limits of the floodplain, and outlining proposed preventive and remedial actions to be taken to reduce the flood damage potential. Without a master plan it is difficult to achieve consistency of action and to solve the problems.

TECHNIQUES OF HAZARD MITIGATION

By

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During the past several weeks, you have been lectured intensely on hydrologic hazards, type of floods, droughts, mapping of hazards, coastal hazards, monitoring of hazards, field methods used in analysis, post-facto hydrologic measurements, risk mapping, forecasting, remote sensing, societal response to disasters, and flood plain management programs.

I am going to speak about Techniques of Hazard Mitigation. You have spent the better part of the last week listening to a variety of speakers discuss how to identify, map, plot, assess, and cope with problems. I shall discuss how to solve problems, or how to reduce the impacts associated with natural disasters.

Let me begin by saying that the only limit to developing a sound hazard mitigation technique is your imagination. The more flexible, the more relaxed and the more innovative you are, the better mitigator you will be.

It is always good to start by defining terms, thus let's define Hazard Mitigation.

Hazard Mitigation is a management strategy in which current actions and expenditures spent to reduce the occurrence or severity of potential natural disasters are balanced with potential losses from future events. Please note that I have underscored Management Strategy. In order to be able to develop a management strategy for solving problems, you must understand what management is.

Simply, management is being able to:

1. Plan
2. Organize
3. Direct
4. Control

In order to put together a management strategy or solve a problem, because that is really what we are doing, we must plan, organize, direct and control a process.

Prior to selecting a preferred mitigation strategy, there are fairly specific steps in problem solving that must be followed regardless of the natural event with which we are faced.

Problem Identification: As a professional, you must focus in on what exactly is the cause of the problem. In floods, it is sometimes too easy to say the problem was caused by flood water - be prepared to dig a little deeper - perhaps the real cause was an undersized culvert, perhaps debris from an upstream lumberyard, perhaps a bridge collapse, perhaps a conservation dam released a large quantity of an anticipated flow. In order to solve the problem, you must define the actual specific cause.

Alternative Analysis: Once the problem has been identified, you need to list an array of alternatives available to solve the problem. This list should not initially be limited by any constraints. Let your imagination run wild as to how and what could be done to solve the problem. Within the alternative analysis phase, you begin to list all of the pro's and con's attached to each alternate. There will clearly be alternatives that won't apply either for pure economic reasons, political, or won't work. During this phase, you set your parameters as to what level of protection is reasonable and justified.

Alternate Selection: Having completed the analysis, the recommended alternative is selected based upon the array of objectives you have defined; the objectives are typically keyed to levels of protection to the citizen, environment, structure, coupled with social, economic, and political influence.

Implementation: This phase of problem solving is often where the entire process begins to break down. Often the planning group feels that the job is finished when the alternative is selected. Not so. A carefully designed process addresses implementation answering such questions as, who is responsible for implementing, where does the specific action take place, describing why it is happening, what the action is, and how it is all supposed to happen.

Evaluation: Over the years, I have seen a great many processes fail, and have seen no lessons learned because this final step of evaluation was ignored. Evaluation is simply reviewing after the fact, after implementation, and observing if your solution worked as you had planned. All too many professionals stop their involvement with a project after the act has been implemented. They are so busy, with other projects or whatever, that they don't return to the scene of the incident to truly evaluate, "did the solution work in the field."

Some techniques of hazard mitigation are:

1. Minimizing the probability of occurrence.
   (Restoring damaged dams, levees, flood walls, debris basins)

2. Improvement of structures and facilities at risk.
   (flood proofing, elevating structures, building code modification for flood, earthquake, tornado-prone community)

3. Identification of hazard-prone areas and standards for prohibited or restricted use.
   (floodplain regulations, hazard mitigation plans, structural - non-structural flood proofing, land use planning, zoning, land acquisition disaster contingency planning)

4. Loss recovery and relief.
   (insurance, disaster relief)

5. Hazard warning, forecasting, evacuation.
   (warning assistance, sheltering, relocation and training, emergency public information)

There are really two primary elements that are central to mitigation.

PREVENTIVE OR CORRECTIVE

1. Preventive Actions, are those defined as primarily directed toward vacant, or undeveloped areas of potential risk.

2. Corrective Actions, are typically those actions taken after the at risk area is developed or occupied.

As I am sure we all agree, the best time to mitigate the efforts of the natural hazard is BEFORE the event. As I am also sure we would agree, the opportunity doesn't normally allow this to really happen until AFTER the disaster.

If we all agree with what I've been saying relative to developing a management strategy, identifying the problem and alternatives, characterizing mitigation techniques as far as overall objectives, discussing whether they are preventive or corrective, and agreeing that the best, most efficient mitigating should happen before the disaster, then it follows, as thunder follows lightning, that we ought to be prepared to develop hazard mitigation plans ahead of time so we know what needs to be done both now and later.

The following Tables (A 1-5) were developed for use by the Federal Emergency Management Agency Hazard Mitigation Teams. They are sample guides toward assisting the professional to consider an array of solutions and showing where they may or may not be appropriate.

TABLE A-1 IDENTIFICATION OF OPPORTUNITIES FOR APPLICATION OF NON-STRUCTURAL TECHNIQUES FOR

CRITICAL FACILITIES

	FLOOD EVENT TECHNIQUES	FLOOD WARNING	ACQUISITION-RELOCATION	FLOOD PROOFING	WET FLOOD PROOFING	ELEVATION	SMALL WALLS & LEVEES AROUND STRUCTURES
	REPETITIVE FLOODING	●	●	●	○	●	●
FLOODPLAIN	COASTAL HIGH HAZARD AREA	●	●	○	○	○	○
	FLOODWAY	●	●	○	○	○	○
	FLOODWAY FRINGE	●	●	●	●	●	●
	100-500 YR FLOODPLAIN	●	●	●	●	●	●
DAMAGES	0-50% DAMAGES	●	●	●	●	●	●
	50-100% DAMAGES	●	●	●	○	●	●
DEPTH	LESS THAN 2' OF FLOODING	●	○	●	●	○	●
	MORE THAN 2' OF FLOODING	●	●	○	○	●	○
VELOCITIES	VELOCITIES LESS THAN 3 FT./SEC.	●	●	●	●	●	●
	VELOCITIES MORE THAN 3 FT./SEC.	●	●	○	○	●	○
DURATION	DURATION LESS THAN 1 DAY	●	●	●	●	●	●
	DURATION MORE THAN 1 DAY	●	●	●	○	●	○
STRUCTURE	STRUCTURALLY SOUND	●	●	●	●	●	●
	STRUCTURALLY UNSOUND	●	●	○	●	○	●
	SUB-STANDARD OR NON-CONFORMING USES	●	●	○	○	○	○

● MAY BE APPLICABLE ○ TO BE AVOIDED

TABLE A-2 IDENTIFICATION OF OPPORTUNITIES FOR APPLICATION OF NON-STRUCTURAL TECHNIQUES FOR

FACILITIES

	FLOOD EVENT / TECHNIQUES	FLOOD WARNING	ACQUISITION-RELOCATION	FLOOD PROOFING	WET FLOOD PROOFING	ELEVATION	SMALL WALLS & LEVEES AROUND STRUCTURES
	REPETITIVE FLOODING	●	●	●	●	●	●
FLOODPLAIN	COASTAL HIGH HAZARD AREA	●	●	●	●	●	○
	FLOODWAY	●	●	●	●	●	○
	FLOODWAY FRINGE	●	●	●	●	●	●
	100-500 YR. FLOODPLAIN	●	○	●	●	●	●
DAMAGES	0-50% DAMAGES	●	●	●	●	●	●
	50-100% DAMAGES	●	●	●	○	●	●
DEPTH	LESS THAN 2' OF FLOODING	●	○	●	●	○	●
	MORE THAN 2' OF FLOODING	●	●	●	○	●	○
VELOCITIES	VELOCITIES LESS THAN 3 FT./SEC.	●	●	●	●	●	●
	VELOCITIES MORE THAN 3 FT./SEC.	●	●	●	●	●	○
DURATION	DURATION LESS THAN 1 DAY	●	●	●	●	●	●
	DURATION MORE THAN 1 DAY	●	●	●	●	●	●
STRUCTURE	STRUCTURALLY SOUND	●	●	●	●	●	●
	STRUCTURALLY UNSOUND	●	●	○	●	○	●
	SUB-STANDARD OR NON-CONFORMING USES	●	●	○	●	○	○

● MAY BE APPLICABLE ○ TO BE AVOIDED

TABLE A-3 IDENTIFICATION OF OPPORTUNITIES FOR APPLICATION OF NON-STRUCTURAL TECHNIQUES FOR

SINGLE-FAMILY RESIDENCES

	FLOOD EVENT / TECHNIQUES	FLOOD WARNING	ACQUISITION-RELOCATION	FLOOD PROOFING	WET FLOOD PROOFING	ELEVATION	SMALL WALLS & LEVEES AROUND STRUCTURES
	REPETITIVE FLOODING	●	●	●	○	●	●
FLOODPLAIN	COASTAL HIGH HAZARD AREA	●	●	○	○	●	○
	FLOODWAY	●	●	○	○	○	○
	FLOODWAY FRINGE	●	●	●	●	●	●
	100-500 YR. FLOODPLAIN	●	○	●	●	○	○
DAMAGES	0-50% DAMAGES	●	●	●	●	●	●
	50-100% DAMAGES	●	●	●	○	●	○
DEPTH	LESS THAN 2' OF FLOODING	●	○	●	●	○	●
	MORE THAN 2' OF FLOODING	●	●	○	○	●	○
VELOCITIES	VELOCITIES LESS THAN 3 FT./SEC.	●	●	●	●	●	●
	VELOCITIES MORE THAN 3 FT./SEC.	●	●	○	○	●	●
DURATION	DURATION LESS THAN 1 DAY	●	●	●	●	●	●
	DURATION MORE THAN 1 DAY	●	●	●	○	●	●
STRUCTURE	STRUCTURALLY SOUND	●	●	●	●	●	●
	STRUCTURALLY UNSOUND	●	●	○	○	○	●
	SUB-STANDARD OR NON-CONFORMING USES	●	●	○	○	○	○

● MAY BE APPLICABLE

○ TO BE AVOIDED

TABLE A-4 IDENTIFICATION OF OPPORTUNITIES FOR APPLICATION OF NON-STRUCTURAL TECHNIQUES FOR

LARGE COMMERCIAL STRUCTURES

	FLOOD EVENT / TECHNIQUES	FLOOD WARNING	ACQUISITION-RELOCATION	FLOOD PROOFING	WET FLOOD PROOFING	ELEVATION	SMALL WALLS & LEVEES AROUND STRUCTURES
	REPETITIVE FLOODING	●	●	●	●	○	●
FLOODPLAIN	COASTAL HIGH HAZARD AREA	●	●	○	○	○	○
	FLOODWAY	●	●	○	○	○	○
	FLOODWAY FRINGE	●	●	●	●	○	●
	100-500 YR. FLOODPLAIN	●	○	●	●	○	●
DAMAGES	0-50% DAMAGES	●	●	●	●	○	●
	50-100% DAMAGES	●	●	●	○	○	●
DEPTH	LESS THAN 2' OF FLOODING	●	●	●	●	○	●
	MORE THAN 2' OF FLOODING	●	●	○	○	○	○
VELOCITIES	VELOCITIES LESS THAN 3 FT./SEC.	●	●	●	●	○	●
	VELOCITIES MORE THAN 3 FT./SEC.	●	●	●	○	○	●
DURATION	DURATION LESS THAN 1 DAY	●	●	●	●	○	●
	DURATION MORE THAN 1 DAY	●	●	●	●	○	●
STRUCTURE	STRUCTURALLY SOUND	●	●	●	●	○	●
	STRUCTURALLY UNSOUND	●	●	○	○	○	●
	SUB-STANDARD OR NON-CONFORMING USES	●	●	○	○	○	○

● MAY BE APPLICABLE

○ TO BE AVOIDED

TABLE A-5 IDENTIFICATION OF OPPORTUNITIES FOR APPLICATION OF NON-STRUCTURAL TECHNIQUES FOR

SMALL COMMERCIAL STRUCTURES

	FLOOD EVENT / TECHNIQUES	FLOOD WARNING	ACQUISITION-RELOCATION	FLOOD PROOFING	WET FLOOD PROOFING	ELEVATION	SMALL WALLS & LEVEES AROUND STRUCTURES
	REPETITIVE FLOODING	●	●	●	●	●	●
FLOODPLAIN	COASTAL HIGH HAZARD AREA	●	●	○	○	●	○
	FLOODWAY	●	●	○	○	●	○
	FLOODWAY FRINGE	●	●	●	●	●	●
	100-500 YR. FLOODPLAIN	●	○	○	●	○	○
DAMAGES	0-50% DAMAGES	●	○	●	●	●	●
	50-100% DAMAGES	●	●	●	○	●	●
DEPTH	LESS THAN 2' OF FLOODING	●	○	●	●	○	●
	MORE THAN 2' OF FLOODING	●	●	○	●	●	●
VELOCITIES	VELOCITIES LESS THAN 3 FT./SEC.	●	○	●	●	●	●
	VELOCITIES MORE THAN 3 FT./SEC.	●	●	●	●	●	●
DURATION	DURATION LESS THAN 1 DAY	●	●	●	●	●	●
	DURATION MORE THAN 1 DAY	●	●	●	●	●	●
STRUCTURE	STRUCTURALLY SOUND	●	●	●	●	●	●
	STRUCTURALLY UNSOUND	●	●	○	●	○	●
	SUB-STANDARD OR NON-CONFORMING USES	●	●	○	●	○	○

● MAY BE APPLICABLE ○ TO BE AVOIDED

Tools such as these, draft building code amendments, land use regulations, even a priority ranking of actions needing to be taken can go a long way toward implementing and achieving a sound hazard mitigation program.

There is one warning that I would like to issue. As professionals, we all tend to concentrate on our own area of expertise, hence a hydrologist will only be looking at a flood risk and thus develop a mitigation plan for only floods. The same holds true for a geologist and earthquakes, etc.

As we should be aware, hazards don't always keep themselves apart in neatly, defined areas. You can often be faced with a problem area sitting atop an earthquake fault, having steep slopes and unstable soils, and have severe floodplain problems. Clearly, any plan developed must address the probability of all these risks, and must address how do we rationally proceed while understanding all the risks.

One of the exciting things about being involved in hazard mitigation, is that there aren't a great many rules, what works in one area, may not work in another.

In order to be successful in the field of hazard mitigation, I would like to leave you with two suggestions:

1. When developing alternatives in your problem solving process, let your imagination run wild. Your only limit to alternatives is your imagination.

2. The second suggestion is to follow the KISS formula when developing a plan.

   KEEP IT SIMPLE STUPID.

Table A-1 Identification of opportunities for application of non-structural techniques for critical facilities.

Table A-2 Identification of opportunities for application of non-structural techniques for facilities.

Table A-3 Identication of opportunities for application of non-stuctural techniques for single-family residences.

Table A-4 Identication of opportunities for applicationof non-stuctural techniques for large commercial structures.

Table A-5 Identification of opportunities for application of non-stuctural techniques for small commercial stuctures.

MICROCOMPUTERS IN PLANNING ALTERNATIVE FLOODPLAIN USES

By

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INTRODUCTION

The past decade has seen significant progress in the development of flood simulation technology. The early models were rather simple to construct and solve, at times requiring little more than a desk top calculator. As computer technology evolved and larger capacity machines became more commonplace, model builders were freed to introduce more complex versions of the flood hazard problem. The result has been a proliferation of quantitative techniques, including dynamic system simulation as well as linear, quadratic, and integer programming.

There is little doubt that such tools have proved to be potentially very useful in managing hazardous floodplains. However, at the same time there are a number of dangerous pitfalls. As with all new technologies, these analytical simulation methods are bound to be abused, not just by the untrained, but by the highly skilled analyst as well. The results of analytical studies, particularly those based upon computer routines, tend to take on an aura of reliability far beyond that warranted by the data's accuracy or the model's validity. This is far and away the greatest danger the model builder or user faces.

The wise management of floodplains is at best a difficult task; recent U.S. experiences vividly suggest the complexity of the problem.

It has been learned that knowledge of potential loss through a combination of mapping and simulation may be insufficient to stimulate protective action on the part of those experiencing flood damage risks. There has been little success in heavy reliance upon technological adjustment dams, channels, and levees - as a means of reducing flood loss damage.

The reason that the struggle against flood losses has not been more successful lies in the failure of public agencies to understand the nature of the choices that people make in the face of uncertainty. Economists in particular have been quick to assume that individuals will readily recognize optimal or efficient measures to be employed in reducing flood losses. The optimum in this case refers to that combination of measures, be it proofing, land use planning, engineering works or others, which minimizes the sum of both residual losses and the costs of employing these measures. While the economically optimal solution to the flood problem may become apparent from the results of simulation study, there is little to guarantee that this optimal mix of measures will ever be adopted. Because of this uncertainty only modest claims can be made for the usefulness of simulation in floodplain management. Such studies would have to be bolstered by analyses furthering our understanding as to how and why different damage mitigating measures are adopted. In fact, a number of such studies have already been initiated and are reviewed later.

The task of simulating flood losses is further complicated by the fact that a full complement of flood-related impacts has yet to be adequately identified and measured. Nearly all of the approaches currently in use deal with physical damage only (loss of property), thereby failing to recognize the employment or inflationary effects that a truly catastrophic event can generate. Furthermore, the existing simulation techniques stop short of a multiple objective presentation. In most developed countries, flood adjustments are not selected on the basis of expected benefits and costs (economic efficiency) alone, but are shaped by other socially desirable goals such as: equity in the distribution of benefits and costs, environmental preservation, and avoidance of large scale catastrophe.

Because the majority of the techniques reviewed concentrate on the economic efficiency criterion, it provides a useful point of departure for the class. This will be followed by a sample case study and a brief evaluation of how their technology would fare elsewhere in the world.

OPTIMAL ADJUSTMENT MIX

Just because losses can be avoided, it does not follow that adjustments to the hazard will or should be undertaken. Figure 1 shows how losses and costs respond to any damage mitigating measure. As more protection is undertaken the expected losses decline, but at the same time the cost of adjusting increases. For example, if adjustment in this figure represents the quality of permanent flood proofing adopted by the population at risk, it is to be expected that losses will decline and costs rise as shown. However, it is also entirely possible that no adjustment is warranted. The cost of adjustment is always above the residual losses. Sometimes the best solution to the flood problem is just to bear the loss. The role of simulation in this scheme is to identify the nature of the loss function and to show how incremental adjustments to the hazard reduce these losses. But in order for simulation results to be of any use, these losses must be compared with adjustment costs.

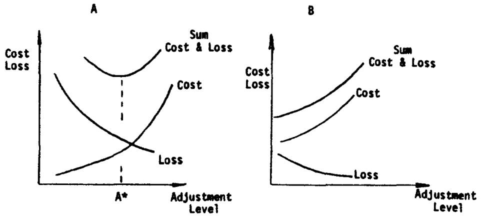

Figure 1. Optimal Adjustment Level

Adjustment Interaction

The desirability of different flood adjustments will be influenced by their interaction. For example, engineering works and a warning system which makes temporary (dynamic) flood proofing possible, are complementary measures. The loss reducing value of warnings depends heavily upon the longer term measures which may be in place. If the floodplain is zoned to exclude building or if channel improvements are adopted, fewer warnings would be required. The problem then is to determine the optimal, or cost minimizing use of both measures. This optimum would embrace a number of cost and loss components:

a. the capital and maintenance cost of the long term measure, such as a flood storage reservoir,

b. the residual losses if the flood storage reservoir's capacity were exceeded,

c. the cost of warning individuals, whether or not the warning turned out to be accurate,

d. the cost of dynamic flood proofing, and

e. the losses incurred if warnings were not made and a flood event occurred that exceeded the capacity of the reservoir.

If forecast accuracy turned out to be extremely poor, then the cost of proofing plus the residual losses could be sufficiently dramatic to push the optimal decision toward the use of the long term measure. If, however, the warning system were reasonably accurate then it might be possible to rely less on longer term options.

The results of flood simulation will be different depending upon how adjustments are modeled - if they are introduced singly or in combination. The value of any single adjustment will be influenced by the availability of competing and complementary measures.

SIMULATION OF AN URBAN FLOODPLAIN

Simulation of urban flood problems begins with an abstract representation of the floodplain. By taking cross-sections at various locations along the river channel, the floodplain can be divided into a set of elevation contours. Each step (j) in a given cross-section is represented by a specific elevation above the river (Ej). Given the flood height S_i the inundation experienced by structures at each elevation contour will be S_j minus E_j. This difference when applied to a depth damage relationship provides some indication as to how destructive any particular flood event turns out to be.

It has long been recognized that reliance upon flood depth alone is a risky means of forecasting potential damages. The ideal depth-damage relationship contains at least four arguments - depth, velocity, sediment carried, and duration of the flooding. The current state of the art with regard to this ingredient of flood simulation is probably less advanced than any of the other components. It should be recognized that these relationships are for a representative flood, and as such they may not be appropriate for specific situations. Changes would have to be made to these relationships in order to incorporate the influence of velocity, flash flooding, sediment and debris carried down from the upstream regions, and so on.

If the validity of the estimated depth damage relationship is accepted, at least in principle, then it is a short step to computing the pecuniary losses resulting from any single event. If D_{kj} represents the percentage damage occurring to building type k for flood height j, then the total damages are given by,

$$(1)\quad TD_j = \sum_{k=1}^{m} \sum_{j=1}^{n} (D_{k,j})\,(N_{k,j})(V_{k,j})\,(sj-Ej)$$

Where: TD_j is the total damage resulting from flood event S_i;$N_{k,j}$ is the number of structures of type k in elevation contour j; Vk,j is the average value of the type k structure in contour j; n is the number of contours; m is the number of structure types.

The total expected damage for all possible floods would be the total damage for each event weighted by its probability of occurrence, i.e.,

$$(2)\quad \begin{array}{l}\text{Expected}\\ \text{Pecuniary} = \displaystyle\sum_{i=1}^{r} P_i TD_j.\\ \text{Loss}\\ \text{(in dollars)}\end{array}$$

All components of (2) have been defined except for the flood frequencies (P_j). These probabilities can be obtained from the climatological records of a region, if available, or through remote sensing as described by Castruccio, 1976.* Given the availability of these data, the expected pecuniary losses are easily calculated by summing damages over all segments of the river basin. Although this procedure appears to be straightforward and requires little additional explanation, the adoption of certain loss mitigating adjustments in any one upstream segment will influence the damages occurring elsewhere downstream. This linkage is addressed in some detail in the section concerning optimal adjustment combinations.

The foregoing description of loss measurement forms the foundation on which the flood hazard simulation is built. The incorporation of alternative adjustment, specifically engineering works, flood proofing and land management, will prove to be a matter of reworking the major components of the model. Flood control reservoirs, diversion channels and levees alter the flood probabilities. Flood proofing truncates the depth damage relationship. And land use management affects the pattern of population growth by shifting the magnitude and mix of floodplain use.

Influence of Engineering Works on Losses and Costs

The fact that stage height is such an important ingredient in the flood problem has led engineers to propose various methods to reduce its magnitude. The more common techniques implemented are levees, channel improvements, increasing channel capacity, and flood control reservoirs. Each measure is designed to alter the way floodwaters pass through the channels. Levees and dikes permit a greater volume of water to flow without spilling over into damageable portions of the floodway. Increased channel capacity acts in a similar way. In contrast to channel improvements and levees, a flood control reservoir alters the time profile of discharge through the absorption of upstream runoff. As a result, flood waters are temporarily stored and released gradually, thereby reducing the volume of water with which downstream reaches must contend.

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*Footnotes refer to References.

The effect of these measures can readily be incorporated into the mathematical description of urban flooding previously provided. First, these works will alter the probability of flooding, virtually eliminating the smaller flood events, but leaving the greater events nearly uneffected. The associated change in the flood probabilities converts to a loss reduction that is a measure of the benefit derived from these adjustments. However, it should be noted that at least a portion of these benefits may be eroded by the fact that failure of engineering works may lead to a much greater loss per dwelling than would have occurred if these works had never been put in place. The example of the broken levee system which induces extensive and enduring flooding needs little additional elaboration.

Land Use Management

Unlike flood control, which channels and diverts floodwater away from occupied areas, land use management seeks to alter the pattern and mix of flood land uses. The effect of this adjustment on losses is easily seen. If the purpose of land use is to restrict growth in the entire floodway or only to permit growth in specified areas, such as above the 50-year return period contour, then losses would be diminished in direct proportion to the number of structures that are excluded from the floodway.

The cost of restricting growth in the floodplain is measured by the foregone benefits potentially available from locating there. Floodplains in some instances provide definite advantages over alternative sites. For example:

a. agricultural productivity will normally be higher within the floodplain;

b. construction costs, clearing and laying foundations, will be less;

c. water supply and waste disposal costs will be less for those industrial activities that require water as an ingredient in the production process; and

d. floodplains are often safer than surrounding areas with regard to such hazards as erosion, landslide and wind.

If the management of floodplain property proceeds in a manner that restricts all potential users, then the costs will be simply the difference between benefits derivable from floodway use less the benefits flowing from the next best alternative site.

It is not difficult to conceptualize those costs, yet their measurement may prove to be extremely complicated. An approach that has been offered by a number of economists is to use land value as a proxy for these floodplain benefits (James, 1964; Lind, 1966).[4,6] If it is assumed that the market value of a piece of land reflects the yearly benefits derived from its use, discounted in perpetuity, then the difference in market value between floodplain and not predicated upon the belief that the full hazard cost is recognized by the buyers and sellers of land. That is, the market for land is nearly perfect and all traders are fully informed. If the potential for direct damage is insufficiently understood by prospective buyers, they may be willing to offer more for the land than it is worth in terms of benefits. Furthermore, land value does not capture the external costs of flooding. The disruption of sensitive production activities could stimulate a substantial set of secondary disruption, the employment effects of which do not enter the market for land. In short, the land value approach may prove to be a weak justification for imputing cost to floodplain lands. A more reasonable approach would be to evaluate different uses of the floodplain separately and determine in a fairly general way the benefits derived from alternative locations both on and off the floodplain.

Flood Proofing

Flood proofing measures belong to one of two basic varieties; permanent or emergency. Permanent measures, such as stilts, earthern platforms, and so on are designed to reduce the damages incurred in relatively minor floods. Rather than apply a normal stage damage relationship, permanent proofing techniques eliminate, or at least substantially reduce, damage up to their design limit.

Dynamic flood proofing, the removal of damageable property or sandbagging, requires some form of warning. Hence, the benefit derived from this adjustment is governed by the accuracy and timeliness of the flood forecast. Similarly, the costs of implementing emergency actions are influenced by forecast error. In short, the expected reduction in losses and adjustment costs may be greatly influenced by the frequency with which individuals at risk in the floodplain are induced to take inappropriate action - removing damageable property when no flood occurred or when unexpected floods damaged property before protective action could be implemented.

Dynamic flood proofing may be incorporated into the model in a manner similar to that already described for the more permanent proofing techniques. Warnings reduce the damages occurring for minor floods, particularly to contents, hence the depth damage relationship is adjusted accordingly. Tai[7], Howe and Cochrane[3], 1975, and Willis and Alkiku[8], 1974, have provided the most recent analyses of this adjustment. Estimates of flood proofing costs have been developed by James[4], 1964 and Day[2], 1970, among others.

Optimal Use of Adjustments

The optimal use of each of these measures taken one at a time is determined by computing the losses for a series of potential floods. These damages then become the base against which adjustments may be compared. By altering the major components of the model to reflect incremental changes in adjustment level, a new set of losses are derived. This set, when subtracted from the base calculations, yields a reduction in loss which is attributable to particular actions. For example, if flood proofing were the specific adjustment under consideration, a set of losses reflecting one-meter, two-meter, three-meter and higher flood proofing levels would be compiled. The loss reduction for these structural improvements can then be contrasted with their costs.

Each of the other competing adjustments would be evaluated in an identical fashion. By so doing the relative efficiency of each could be ascertained.

A MICROCOMPUTER SIMULATION OF ALTERNATIVE FLOODPLAIN MANAGEMENT STRATEGIES

An inexpensive, systematic procedure for reviewing the impacts of alternative designs has been developed to incorporate the Water Resources

Council's guidelines for the computation of Net Economic Benefits. To reduce the cost of exercising the model, computational routines were written for the current generation of microcomputers (IBM-PC).

Components of the Model

The program's mechanics are governed by the Water Resources Council's (WRC) Procedures for Evaluation of National Economic Development and Costs (December 14, 1979). As shown in Figure 2, the economic model, which is the program's foundation, is driven by a set of assumptions reflecting characteristics for both the event and the population at risk. In some instances the term "assumption" is misleading since several of the factors shown can be estimated with some precision or are dictated by the WRC's procedures. The cost of funds, for example, is tied to the government's borrowing rate. FIA will know the amount of insurance coverage currently in force; hence this ingredient too cannot truly be considered an assumption. However, in either instance some judgment is required.

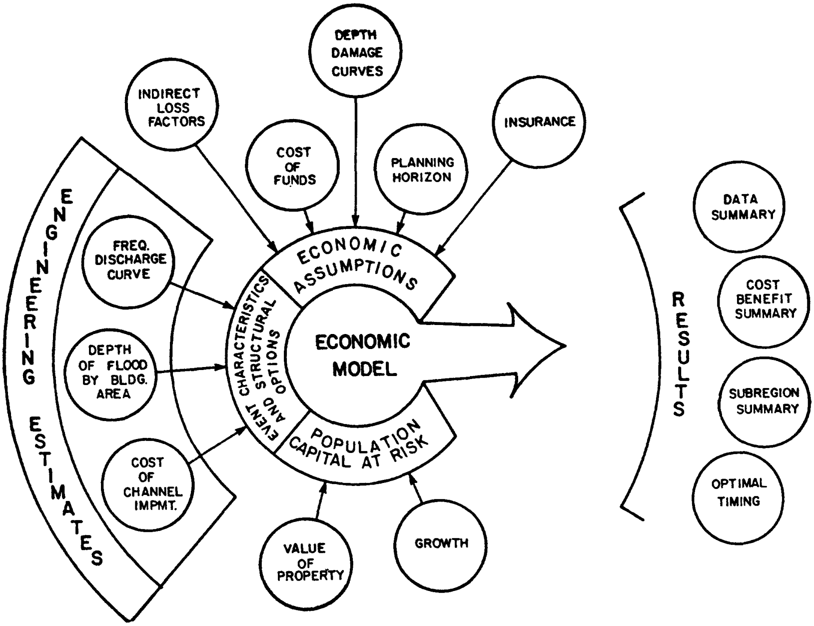


Figure 2.--Diagram of the model's basic elements.

Other "assumptions" required by the model are somewhat less certain. A planning horizon of 50 years is not uncommon. However, high interest rates diminish the sensitivity of this factor. The label "assumption" applies most pointedly to the depth damage curves and indirect loss factors. A review of the literature proves that both are the least researched of all the elements shown in Figure 2.

In contrast, techniques are available that yield reasonably accurate frequency discharge curves and flood depths. Hydrologists and engineers involved in flood hazard studies would certainly utilize such tools. Characteristics of the population at risk, the type and extent of capital subject to flood loss, and their likely rate of change over the foreseeable future can be obtained through aerial photographs and field surveys. Estimating the growth rate is, of course, the most difficult aspect of the problem. Growth is governed by the amount of raw land that is able to be developed and the real rate of appreciation experienced by those who own developed property. Both of these factors are linked to the overall economic viability of the community's economy, something that may be difficult to forecast.

The model computes the benefits derived from alternative floodplain management strategies. Following the WRC's guidelines, these benefits are based on the extent to which each strategy reduces flood damages. The procedure for estimating these potential losses is the model's core.

Annual flood damages for events of varying return interval are obtained by multiplying the depth of flooding, the percentage of a structure's value destroyed, and the total value at risk. The <u>expected</u> loss is directly computed by integrating the area under the loss curve. Indirect losses, i.e., production delays and homeowner inconveniences, are assumed to be a percentage of the direct losses, as are destruction of contents and the damages sustained by public facilities.

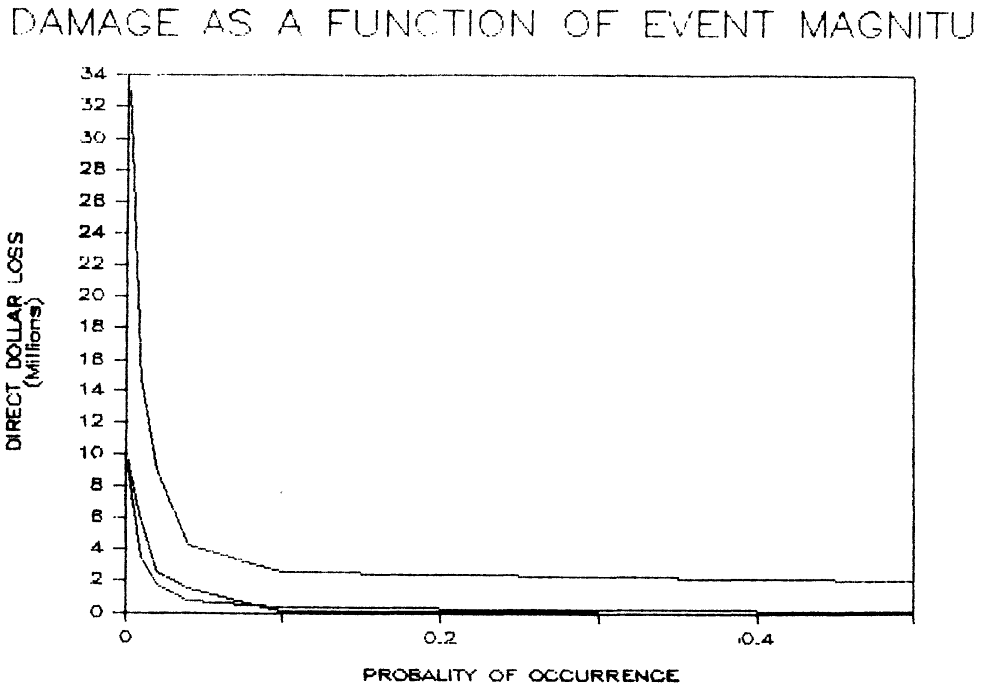
The computation of the benefits is rather tedious for even a small cluster of protective measures. Since the focus of this paper is to demonstrate an economical approach from a universe of possibilities, the hand calculator approach had to be abandoned in favor of a more automated method. The computer model, integrating the components shown in schematic form in Figure 2 was developed to lighten the work load.

A BRIEF DESCRIPTION OF THE PROGRAM

The model was built using an electronic spreadsheet (LOTUS 1-2-3). Tables 1 through 6 illustrate the data requirements. Although only one reach is shown, the model can handle up to five, each containing 15 subareas. The growth rate and the maximum number of structures each subregion can sustain, defines growth in value at risk. Table 6 displays losses that would result in Reach 1 as a result of a two year event. Now shown are the other five reaches or the effects of the 10, 25, 50, 100 or 500 year events. The expected losses and the net benefits of alternative strategies are exhibited on the right side of the table.

Figures 3 through 4 illustrate the graphics capability of the program. Losses can be displayed as a function of subregion or by event. The depth damage curves can be plotted as shown in Figure 5. Figure 6 and 7 illustrate how the results and inputs can be illustrated in three dimensions.

Figure 3



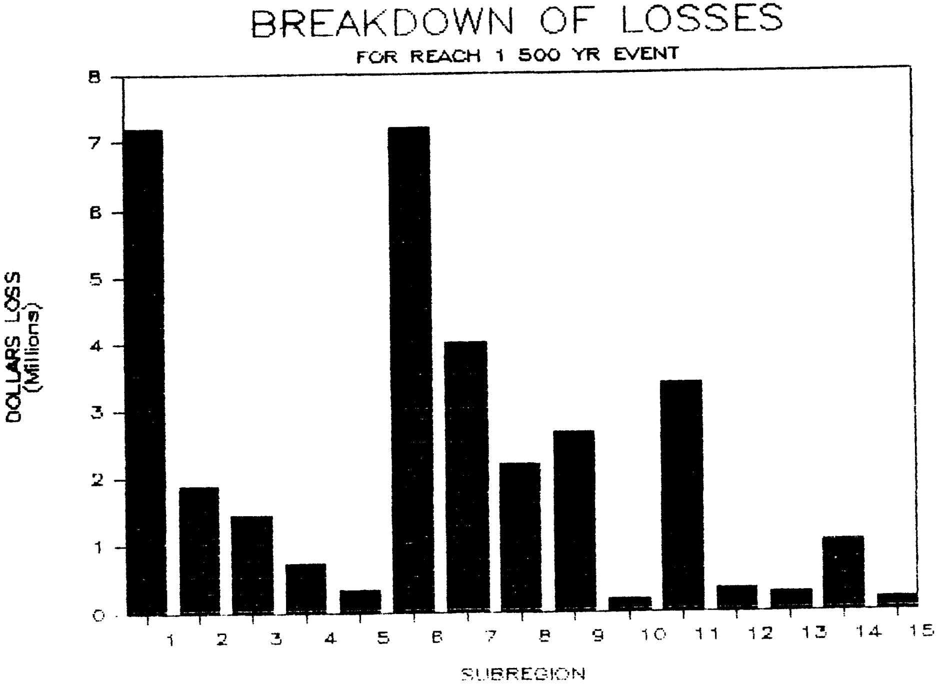


Figure 4.

POTENTIAL APPLICATION IN DEVELOPING COUNTRIES

Is flood loss simulation a tool that could be fruitfully applied to the management of a developing country's floodplains? The answer is neither a firm yes or no. Rather, the desirability of the technique depends upon how it is used. It can be a costly, time consuming, and confusing apparatus if it is employed in a way that tries to capture all of the intricacies of the problem. That is, if precise flood frequencies, depths, depth damage relations, and so on are thought to be necessary, then the task will prove insurmountable. However,

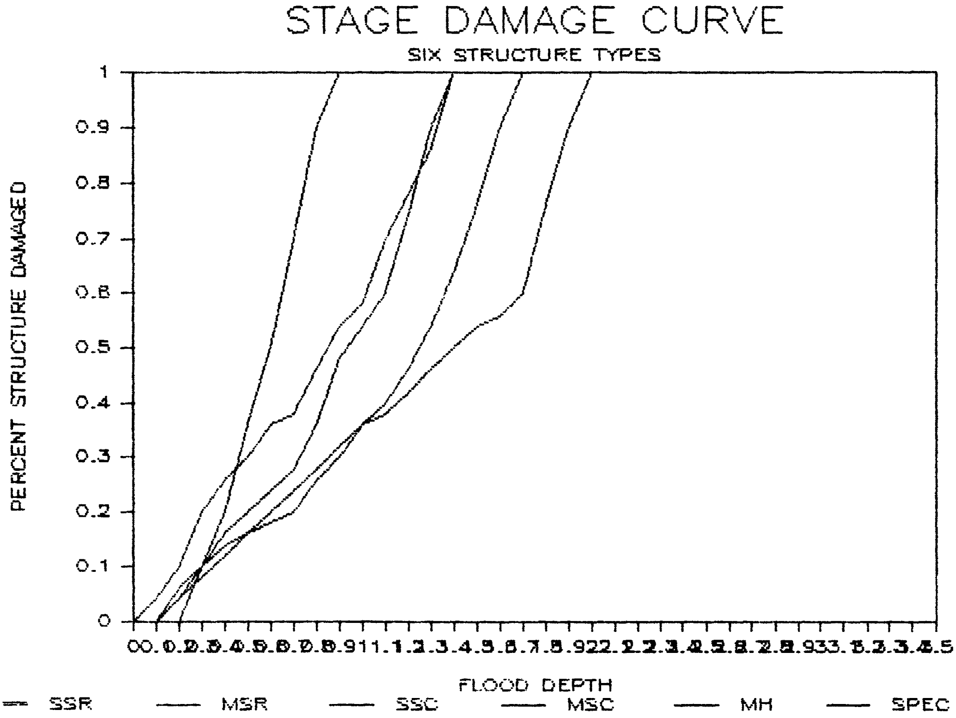

STAGE DAMAGE CURVE
SIX STRUCTURE TYPES
PERCENT STRUCTURE DAMAGED
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
00.0.1.2.3.4.5.6.7.8.911.1.2.3.4.5.6.7.8.922.2.2.3.4.5.6.7.8.933.3.3.3.4.5
FLOOD DEPTH
SSR
MSR
SSC
MSC
MH
SPEC

Figure 5

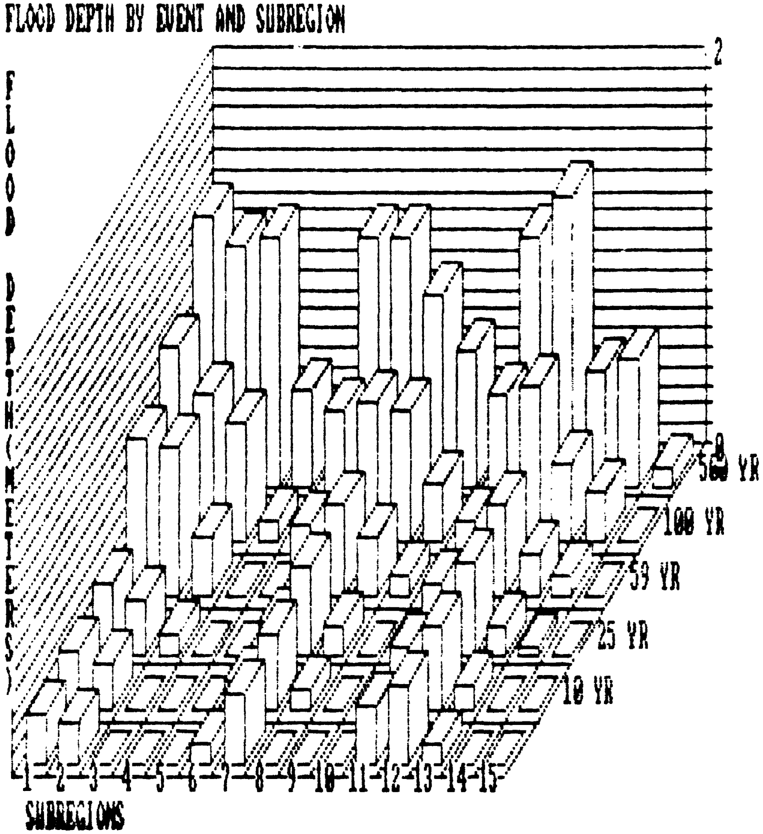

FLOOD DEPTH BY EVENT AND SUBREGION
FLOOD DEPTH(METERS)
2
0
500 YR
100 YR
50 YR
25 YR
10 YR
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
SUBREGIONS

Figure 6

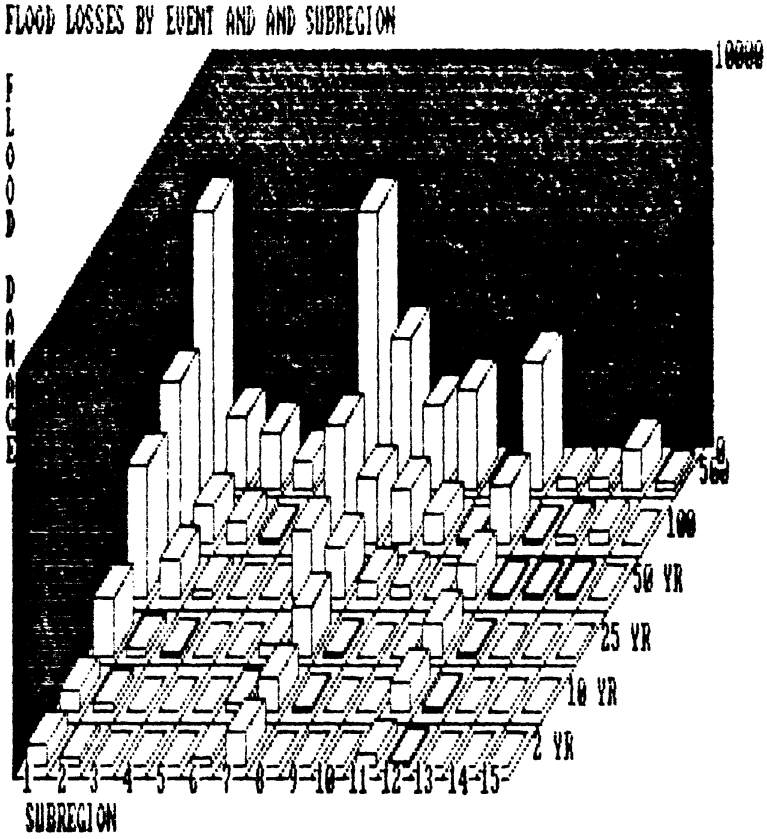

FLOOD LOSSES BY EVENT AND AND SUBREGION
FLOOD DAMAGE
10000
0
500
100
50 YR
25 YR
10 YR
2 YR
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
SUBREGION

Table 7

if at the other extreme, analysts are willing to venture rough approximations concerning the location of the floodplain boundaries and are willing to accept crude relations between depth and damages, then the technique may prove to be extremely valuable in managing flood-prone areas.

This plea for restricting the breadth of any simulation attempt is in keeping with the needs and available resources of many developing countries. It is imperative that countries that have not yet begun to take positive steps to manage their floodplains do so. As discussed above, delays, particularly in an economy that is about to embark on a path of rapid growth, can be quite costly.

This necessity for action becomes all the more apparent when the problem of floodplain management is viewed from the perspective of a developed country. In the United States, for example, planners have found it nearly impossible to roll back growth once it has begun in the flood-prone areas. It appears that as a community's infrastructure is extended to the floodway, further development takes on a dynamic spurt that is almost irreversible. The only dynamic reversal on record is that of Rapid City, South Dakota, where the 1972 flash flood literally removed two-thirds of all structures in the 50-year floodplain. And, rather than rebuilding on the site of the disaster, local planners were successful with the aid of an urban renewal grant from the Federal government, in maintaining the floodway as park land. Examples of such a change are rare and the shift in land use which comes about in such a fashion proves to be extremely costly.

The need to restrict the scope of simulation approaches extends to the analysis of employment effects stemming from flood losses. In many ways these employment effects could be quite significant in a fragile economic setting. Although it is not recommended that initial developing country flood loss simulations delve into the problem as deeply as that suggested previously, some simplified rules should be established. For example, the loss of a textile factory generates twice the income effects as the direct loss in jobs. The loss of a market would generate income effects one half as large as the direct loss. Through such efforts a more realistic, yet inexpensive picture of potential flood losses can be constructed.

Level of Skill and Cost

Much of the flood loss computations can be carried out with a microcomputer, and in developing countries where large scale computer facilities are scarce, this will prove to be a necessity. This strategy forecloses the use of some of the more sophisticated programming techniques described above. At best, the simulation study can accomplish a series of "what if" calculations. For example, what would happen in a specific community if floodplain encroachment was allowed to proceed uninhibited?

What would the time profile of losses look like? How would the loss picture be altered through the adoption of a levee system, static floodproofing, land use regulation, and so on? These computations are quite straightforward, as indicated in the lecture, requiring at most two months per study site, performed by a person with skill equivalent to a master's degree. From the description it appears that the cost of a useful set of results for one site may exceed $3,000 perhaps running as high as $10,000, depending upon the magnitude of the data collection effort.

Observations Pertinent to the Training Course

Experience gathered in installing microcomputers in West Africa (Gambia) has led me to believe that they are well-suited. Such experiences have served to reinforce a previously held conviction that training in sophisticated computer languages was unnecessary and possibly even counterproductive. It was my impression that the Gambian government was,about to send its brightest and most able public servants to study programming and systems design in the United States. The applicability of such skills to the problems of a developing nation should be seriously questioned, in light of the current generation of so-called "user-friendly" software. I suppose at one time it made sense to economize on the former at the expense of the latter. However, memory and external storage are no longer in short supply and the cost of augmenting either is now quite low. New programs have been developed to capitalize on the new situation. LOTUS 1-2-3, for example, is completely self-contained. Whenever the operator feels that help is required, a series of key entries, guided by graphics and text displays, methodically pinpoints the source of confusion and then instructs the operator on the appropriate response. The program purportedly contains 180 equivalent pages of instructional material. Such aids have the potential for dramatically reducing the time to train individuals. It is questionable whether knowledge of FORTRAN, PASCAL, or COBOL will enhance their progress.

Microcomputers, unlike mainframes, are inexpensive enough to be handled by small in-country suppliers. While visiting Bangkok in 1982, I was struck by the ingenuity of the local Apple dealer, which bore a striking similarity to the inventiveness displayed by their American counterparts. I anticipate that the microcomputer revolution will trigger a rapid growth of cottage industries throughout the Third World much the same way it revitalized the entrepeneurial spirit in certain parts of the United States.

Inexpensive memory in conjunction with continued progress in producing ever more compact circuits has fostered significant changes in procedures and skills required to maintain a computing facility. The newest generation of microcomputers comes equipped with self-testing features, enabling technicians to pinpoint the source of trouble without the use of sophisticated auxiliary equipment. Once a failure is detected, it can normally be corrected, often without special tools. The fact that most circuits and peripherals are now quite small lends itself to a mail-order method of restocking backups.

Microminiaturization has tended to reduce power requirements, which in turn is an important ingredient in coping with interruptible power. I was shown two computers that had been previously installed in the Gambia. Each appeared to require substantial power inputs despite a modest amount of core memory (less than 256,000 bytes). The one uninterruptible power supply available at the airport was purchased for approximately $8,000 and consisted of a very large AC/DC converter and a bank of car batteries measuring eight feet by three feet. It had not been operational for three months prior to June due to a lack of parts. A modern unit to service IBM's new microcomputers is slightly larger than a shoebox. The development of liquid crystal displays should enable the next generation of microcomputers to function over a prolonged period without external sources of power. In my judgment the problems of unreliable and "dirty" power will diminish in importance as the microcomputer technology evolves.

A Priority System for Mapping and Simulation

Because of their limited resources, it will be some time before most developing countries will be able to undertake an extensive flood mapping and loss evaluation program. This fact focuses attention on the need to begin work on a priority system for such a program. Given that only a few flood-prone areas can be evaluated in the near future, which ones should they be?

In order to answer this question, it would be important to map and evaluate those areas where urbanization has begun. It is particularly relevant to examine floodplain communities that have significantly sensitive economic activities subject to the risk of flooding. By looking at the economic linkages of the developing economy, it would be possible to pinpoint activities around which the economic base pivots. This information would then direct attention to those flood areas that are of significant importance to the growth and development of the country.

Although this recommendation was formulated in the context of urban growth and industrialization, a similar strategy could be employed for agriculture. For example, the major food-producing regions of a nation could readily be established, and the effects of their disruption determined. The degree of disruption and the economic and social turmoil it generates gives some insight into which flood areas need to be mapped first.

The priority scheme just illustrated should take account of the various flood types, flash, riverine, and coastal. Normal riverine flooding is not likely to do extensive damage to the productive sector. The same cannot be said about flash and coastal flooding which are much more destructive in their impact.

The Need to Understand Choice of Adjustments in Developing Countries

The primary assumption of these recommendations has been that mapping and simulation can provide information upon which water resource planners can act in order to diminish the undesirable consequences of the flood hazard. At least in the United States, this assumption has proved to be less than adequate in planning the use of flood-prone areas. In order for simulation techniques to be of value, they have to be torn from their foundation and revised to incorporate more information as to how individuals at risk do in fact respond to hazardous events. This recommendation calls forth the need to bolster the mechanical parts of flood studies with the efforts of social scientists to understand adjustment choices. Without this supporting role, it is quite likely that the flood areas will simply be mapped and alternative loss mitigating adjustments evaluated, yet no significant shift in land usage or protection from losses instituted. This experience is not uncommon. Using earthquake prediction as an example, physical science research on the technology proceeded for at least a decade before someone asked the question "how would the people respond if in fact we were able to accurately forecast a large event, to which the seismologists responded they didn't know. Lurking behind this apparent ignorance is an assumption similar to that implicit in the flood problem, that individuals in the forecast zone would respond in a rational way. They would undertake certain self-protective actions, reinforcing their homes and businesses and evacuating at the appropriate signal from responsible authorities. Recent social science investigation of this response indicates that, quite to the contrary, individuals will not always act in a way that appears to support their self interest. They may, in fact, ignore the warnings altogether. Assessment of flood losses may meet the same fate unless steps are taken to better understand political, social and psychological obstacles to the adoption of flood mitigating adjustments.

A Realistic Target for a Study on Flood Loss Management

It appears that it is, as yet, too early to actually begin simulation of flood losses. Several difficulties elaborated on above need attention before a sound demonstration project can be launched. Reiterating these points, first, and most important, the potential incompatibility of simulation and dynamic mapping would have to be resolved. Second, care must be exercised to select flood-prone areas in order to yield sufficiently new insight concerning the magnitude of the flood problem. Third, simplified methods for conducting a series of "what if" simulations need be developed. The problem indicated here is not so much in the estimation of physical damages as in the induced social and economic disruption such damage instigates. Finally, some effort has to be devoted to the factors that influence adjustment choices.

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```
                    TABLE OF CONTENTS
     FOR AN INTERACTIVE FLOODPLAIN MANAGEMENT PROGRAM

                           by
                    Dr. Hal Cochrane
                          ANRE
                Colorado State University
-----------------------------------------------------------
 The following program is designed to help floodplain managers
assess the costs and benefits of different strategies for alleviating
losses. It permits the user to enter and edit data,
compute the losses by reach(and subregion within a reach), and
derive the optimal size and time to adopt a project.  The Pg Dn
key provides a menu of options plus instructions on how to access
each of the forementioned features.

              ******* PRESS THE PG DN KEY ********
```

```
                    DATA ENTRY SECTION

The program requires the following data.

     1. Number of structures at risk by building type.
        Building types are described as follows.
            a. Single Story Residential
            b. Multiple Story Residential
            c. Single Story Commercial
            d. Multiple Story Commercial
            e. Mobile Homes

     2. Value of an Individual Structure by Building Class
     3. Growth Rate in the Number of Structures at Risk
     4. Discount Rate
     5. Land Use Management

     ********* PRESS PG DN KEY TO CONTINUE *********
```

```
Select the option you wish to exercise by holding down the Alt key
and then pressing the letter shown in parentheses.

      1. DATA MANAGEMENT               3. VIEW THE RESULTS

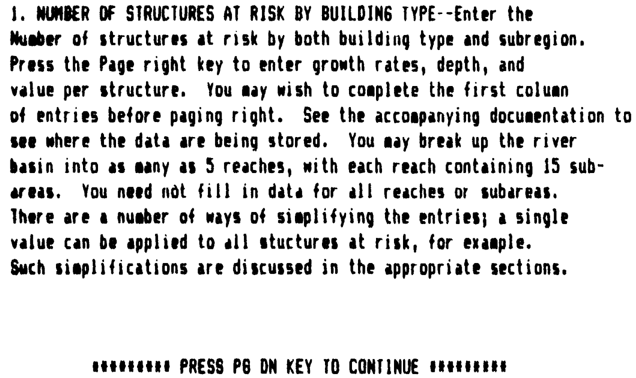
           Alt-(R)ead a File                 Alt-(G)raphics
           Alt-(S)ave a File                 Alt-(N)umerics
           Alt-(E)nter Data
           Alt-(C)hange Data           4. Print Results
           Alt-(J) Read Base Case
           Alt-(L) Save Base Case            Alt-(P)ictures
                                             Alt-Ta(b)les
      2. RUN THE PROGRAM
                                       5. HELP
           Alt-(T)iming Optimum
           Alt-(O)ptimum Size                Alt-(H)elp
           Alt-(V)iew the Results
           Alt-(X) Summary: Benefit Cost, Losses
```

```
     6. Flood Proofing
     7. Channel Improvements
     8. Return Intervals
     9. Flood Depth by Reach and Subregion
     10. Depth Damage Curves by Each Structure Type



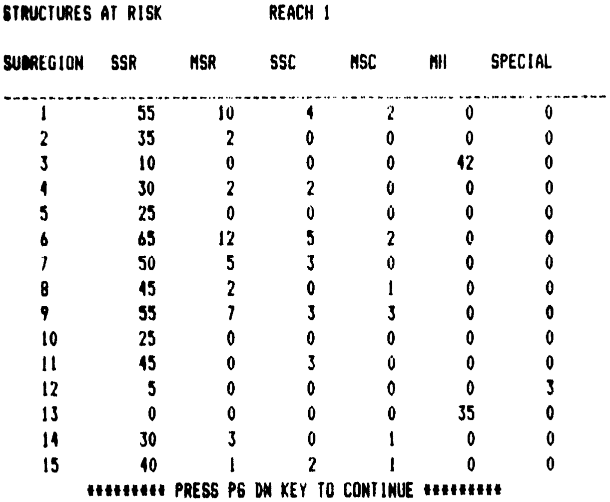
     ********* PRESS PG DN KEY TO CONTINUE *********
```

Tables 1-6



```
1. NUMBER OF STRUCTURES AT RISK BY BUILDING TYPE--Enter the
Number of structures at risk by both building type and subregion.
Press the Page right key to enter growth rates, depth, and
value per structure.  You may wish to complete the first column
of entries before paging right.  See the accompanying documentation to
see where the data are being stored.  You may break up the river
basin into as many as 5 reaches, with each reach containing 15 sub-
areas.  You need not fill in data for all reaches or subareas.
There are a number of ways of simplifying the entries; a single
value can be applied to all stuctures at risk, for example.
Such simplifications are discussed in the appropriate sections.

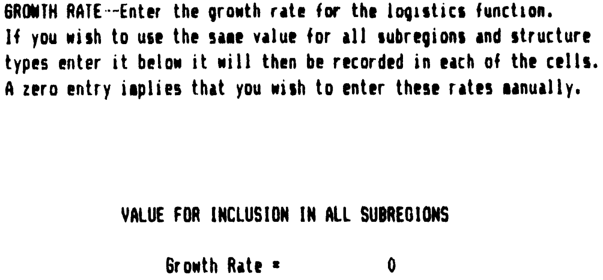

         ********* PRESS PG DN KEY TO CONTINUE *********
```



```
STRUCTURES AT RISK            REACH 1

SUBREGION   SSR      MSR      SSC      MSC      MH     SPECIAL

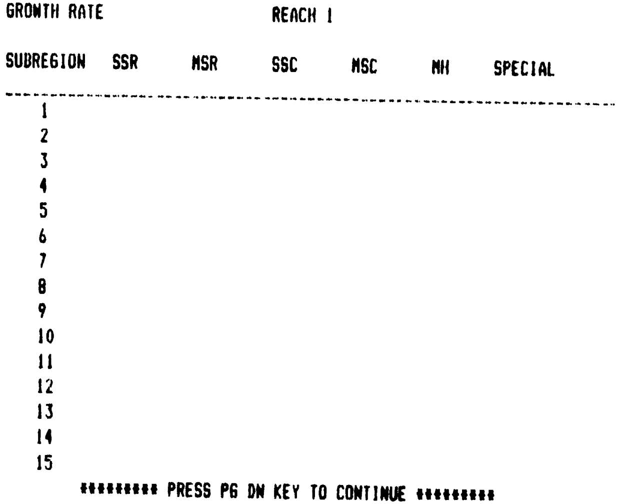
---------------------------------------------------------------
    1        55       10        4        2       0        0
    2        35        2        0        0       0        0
    3        10        0        0        0      42        0
    4        30        2        2        0       0        0
    5        25        0        0        0       0        0
    6        65       12        5        2       0        0
    7        50        5        3        0       0        0
    8        45        2        0        1       0        0
    9        55        7        3        3       0        0
   10        25        0        0        0       0        0
   11        45        0        3        0       0        0
   12         5        0        0        0       0        3
   13         0        0        0        0      35        0
   14        30        3        0        1       0        0
   15        40        1        2        1       0        0
         ********* PRESS PG DN KEY TO CONTINUE *********
```



```
GROWTH RATE--Enter the growth rate for the logistics function.
If you wish to use the same value for all subregions and structure
types enter it below it will then be recorded in each of the cells.
A zero entry implies that you wish to enter these rates manually.


          VALUE FOR INCLUSION IN ALL SUBREGIONS

              Growth Rate =           0
```



```
GROWTH RATE                  REACH 1

SUBREGION   SSR      MSR      SSC      MSC      MH     SPECIAL

-----------------------------------------------------------------
    1
    2
    3
    4
    5
    6
    7
    8
    9
   10
   11
   12
   13
   14
   15
         ********* PRESS PG DN KEY TO CONTINUE *********
```

```
THE MAXIMUM NUMBER OF STRUCTURES AT RISK--It is likely that each sub-
region will be able to support a maximum number of structures.  The
current level of development, the growth rate, and this maximum defines
the path of development.  You may use the following example to observe
how these variables combine to create the path.  Press Alt-E(x)ample.
You may change any of the variables in the table and plot the new
results.  If you wish to utilize an unconstrained growth relationship
enter a "1" in place of the following "0".          0


              EXAMPLE PLOT OF A LOGISTICS FUNCTION

              Growth Rate
              Initial Development
              Upper Limit

              Equation


      ********* PRESS PG DN KEY TO CONTINUE *********
```

```
UPPER BOUND ON STRUCTURES     REACH 1

SUBREGION   SSR     MSR     SSC     MSC     MH     SPECIAL
------------------------------------------------------------
    1        65      10       6       2      0        0
    2        45       4       0       0      0        0
    3        10       1       0       0     47        0
    4        35       2       2       0      0        0
    5        40       0       0       0      0        0
    6        65      12       5       2      0        0
    7        52       6       5       2      0        0
    8        80       5       3       1      0        0
    9        55       7       4       3      0        0
   10        45      10       0       0      0        0
   11        30       2       3       0      0        0
   12         5       0       0       0      0        3
   13         0       0       0       0     42        0
   14        30       5       0       2      0        0
   15        40       3       2       1      0        0
      ********* PRESS PG DN KEY TO CONTINU  *********
```

```
FLOOD DEPTH  Enter the depth of flooding in meters for each subregion
in reaches 1 through 5.  You may use the return intervals given
below or you may opt to change them as desired.


      RETURN INTERVAL STORMS USED IN THE MODEL

                 2 year
                10 year
                25 year
                50 year
               100 year
               500 year



      ********* PRESS PG DN KEY TO CONTINUE *********
```

```
FLOOD DEPTH                    REACH 1
                          RETURN INTERVALS
SUBREGION       2       10       25       50      100      500
-----------------------------------------------------------------
    1        0.25      0.3     0.35      0.8        1      1.4
    2         0.2     0.22    0.275     0.75     0.75     1.25
    3           0        0      0.1      0.3      0.6      1.3
    4           0        0        0        0      0.1      0.5
    5           0        0        0        0        0     0.33
    6         0.1     0.15     0.17     0.35     0.67      1.3
    7        0.35     0.38     0.43     0.45      0.7      1.3
    8           0      0.1     0.15      0.3     0.67        1
    9           0        0        0      0.1      0.3      0.7
   10           0        0        0        0      0.1      0.3
   11         0.3     0.32     0.35     0.37     0.75      1.3
   12         0.4     0.41     0.45     0.45      0.8      1.5
   13         0.1     0.12     0.15      0.2      0.4      0.6
   14           0        0     0.05      0.1     0.25     0.67
   15           0        0        0        0        0      0.1
            ********* PRESS PG DN KEY TO CONTINUE *********
```

```
VALUE OF STRUCTURES BY CLASS BY SUBREGION--Enter the value of each
type of structure.



            ######### PRESS PG DN KEY TO CONTINUE #########


VALUE OF SIRUCURES          REACH 1

SUBREGION   SSR     MSR     SSC     MSC     MH      SPECIAL
-------------------------------------------------------------------
    1      64000   80000  350000  750000   25000
    2      64000   80000  350000  750000   25000
    3      64000   80000  350000  750000   25000
    4      64000   80000  350000  750000   25000
    5      64000   80000  350000  750000   25000
    6      64000   80000  350000  750000   25000
    7      64000   80000  350000  750000   25000
    8      64000   80000  350000  750000   25000
    9      64000   80000  350000  750000   25000
   10      64000   80000  350000  750000   25000
   11      64000   80000  350000  750000   25000
   12      64000   80000  350000  750000   25000
   13      64000   80000  350000  750000   25000
   14      64000   80000  350000  750000   25000
   15      64000   80000  350000  750000   25000
              ######### PRESS PG DN KEY TO CONTINUE #########
```

```
DEPTH DAMAGE CURVES--Enter the percentage of the structure's value
lost as a function of flood depth.

  Press the following key if you wish to look at the shape of the
damage curves entered.  Press any key to return from the graphics
display.



         Alt-(I)  Graphics Display of Stage Damage Curves


         ######### PRESS PG DN KEY TO CONTINUE #########


         DEPTH DAMAGF CURVE --PERCENT OF STRUCTURE'S VALUE LOST
         ######### PRESS PG DN KEY TO CONTINUE #########
   DEPTH     SSR      MSR      SSC      MSC      MH      SPECIAL
--------------------------------------------------------------------
       0        0        0        0        0       0
     0.1     0.04        0        0        0       0
     0.2      0.1     0.04     0.06     0.04       0
     0.3      0.2      0.1      0.1     0.08     0.1
     0.4     0.26     0.14     0.16     0.12     0.2
     0.5      0.3     0.16      0.2     0.16    0.36
     0.6     0.36     0.18     0.24      0.2     0.5
     0.7     0.38      0.2     0.28     0.24     0.7
     0.8     0.46     0.26     0.36     0.28     0.9
     0.9     0.54      0.3     0.48     0.32       1
       1     0.58     0.36     0.54     0.36       1
     1.1      0.7      0.4      0.6     0.38       1
     1.2     0.78     0.46     0.74     0.42       1
     1.3     0.86     0.54      0.9     0.46       1
     1.4        1     0.64        1      0.5       1
     1.5        1     0.76        1     0.54       1
```

```
PROJECT COSTS--Enter the initial cost of the project by subregion
or for the entire reach.  The latter option permits you to skip
the detailed entries.  Follow the same procedure in filling out
the project's maintenance costs.




        ********* PRESS PG DN KEY TO CONTINUE *********



INITIAL COST            0     REACH 1      MAINTENANCE/YR          0

SUBREGION LANDUSE PROOFING          SUBREGION LANDUSE PROOFING
--------------------------          --------------------------
    1                                   1
    2                                   2
    3                                   3
    4                                   4
    5                                   5
    6                                   6
    7                                   7
    8                                   8
    9                                   9
   10                                  10
   11                                  11
   12                                  12
   13                                  13
   14                                  14
   15                                  15
        ********* PRESS PG DN KEY TO CONTINUE *********
```

```
MISCELLANEOUS INPUTS--Enter information concerning project life,
discount rate and other miscellaneous inputs below.  Remember that all
values are entered and computed in real dollars.  Hence, the discount
rate must be real as well.

             FLOOD PROTECTION BY REACH

      ----------------------------------------
        REACH          LEVEL OF PROTECTION

          1                    0
          2                  100
          3                    0
          4                    0
          5                    0


      DISCOUNT RATE                   0.05
      PROJECT LIFE                      50
      INDIRECT LOSS FACTOR             0.2
      DMG. TO PUBLIC CAPITAL          0.25
```

```
COMPUTATIONS OF LOSS--The loss by subregion is computed by multiplying
the number of structures their value, and then applying the percent
loss anticipated as result of the flood depths.  The percentage lost is
is obtained by using a table lookup function.  Page down to view the
losses for each of the five reaches.  Summary statistics are provided
after Reach 5.
```

```
PROPERTY LOSS(2 Yr.)          REACH 1

SUBREGION   SSR      MSR      SSC      MSC       MH     SPECIAL   TOTAL
------------------------------------------------------------------------
    1     352000   32000    84000    60000        0        0   528000
    2     224000    6400        0        0        0        0   230400
    3          0       0        0        0        0        0        0
    4          0       0        0        0        0        0        0
    5          0       0        0        0        0        0        0
    6     166400       0        0        0        0        0   166400
    7     640000   40000   105000        0        0        0   785000
    8          0       0        0        0        0        0        0
    9          0       0        0        0        0        0        0
   10          0       0        0        0        0        0        0
   11     288000       0    63000        0        0        0   351000
   12      64000       0        0        0        0        0    64000
   13          0       0        0        0        0        0        0
   14          0       0        0        0        0        0        0
   15          0       0        0        0        0        0        0
          ######### PRESS PG DN KEY TO CONTINUE #########
```

```
LOSS SUMMARY

                       RETURN INTERVAL STORM (YEARS)

REACH          2        10        25        50       100       500
---------------------------------------------------------------------
  1      2124800   2561200   4351200   9032500  14678000  32973900
  2        38400    362800    781800   1781520   3504000   9153900
  3            0     66000   1529300   2550300   5909780   9653080
  4            0         0         0         0         0         0
  5            0         0         0         0         0         0


         ###### PAGE RIGHT FOR BENEFIT COST SUMMARY ######
```

```
                    NET PRESENT VALUE,
             EXPECTED ANNUAL LOSS AND BENEFITS

-----------------------------------------------------------------------

               BASE    AFTER              PV        PV       NET
               CASE    ADJUST.        BENEFITS    COST        PV
REACH 1   1653516.  1653516.               0         0
REACH 2   235578.2  235578.2               0         0
REACH 3     225713    225713               0         0
REACH 4          0         0               0         0
REACH 5          0         0               0         0
```

GEOLOGIC AND HYDROLOGIC HAZARDS IN THE WESTERN MEDITERRANEAN

By

Luis Alberto Mendes Victor
Lisbon, Portugal

INTRODUCTION

This paper was prepared to provide some general knowledge about meteorological, hydrological and geologic hazards with a view to introduce, on a regional scale, some examples and as far as possible some recommendations which should be used to promote national actions to make a better assessment of the various problems.

The Western Mediterranean area has been largely affected by natural hazards whose quantifications have to be established in order to promote a realistic of disaster preparedness from a scientific and technical well defined background.

In Part A we introduce some definitions generally adopted for the most important hazards and in Part B we present some applications and strategy for specific cases, and we try to find out the best framework for more detailed studies in selected areas.

PART A

METEROLOGICAL AND HYDROLOGICAL HAZARDS

Tropical Cyclones

The small, intense depressions of tropical latitudes are called tro-cyclones (or typhoons or hurricanes, depending on the region in which they occur).

Their fundamental characteristics are:

- Frequency of occurrence, intensity, speed and direction of movement, etc.;
- Wind and rainfall distribution;
- Storm surges--frequency/height distribution, and relationship to meteorological parameters.

The hydrological component of tropical cyclones is concerned mainly with the following subjects:

- Hydrometeorological aspects of tropical cyclone rainfall--depth/duration/frequency relationships;
- Use of hydrological models for estimating probabilities of flood river discharges associated with rainfall or given characteristics.

Tornadoes

A number of dangerous meteorological phenomena--tornadoes, thunderstorms, lightning and hail--are conveniently classified within the discription "severe local storms." These storms have a relatively short life cycle and affect small areas rather than large regions. Although any of these phenomena can be a serious threat to life and property, the tornado is the most dangerous of all, and is capable of bringing total devastation to settlements and developments lying in its path. The mechanism involved is not yet sufficiently understood. Nevertheless an analysis of the meteorological elements which determine the vertical structure of the atmosphere will give frequencies of occurrence of conditions which might result in the formation of a tornado hazard in a given area. For the purpose of a realistic assessment of tornado hazard, these statistics should be used in conjunction with records of actual occasions when tornadoes were experienced in the area.

River Floods

Excessive rainfall is the basic cause of a river flood but there are simultaneously other contributory factors. These may include structural failures such as the collapse of the walls of a reservoir or the embankment of a river proving insufficiently robust to contain the strong flow of water. When rainfall is of very high intensity, the resulting flood may be of sudden onset, usually described as a flash-flood. This phenomenon is particularly dangerous because it leaves very little time for any adequate warning or evacuation. If a river flood takes place near the coast, the hazard may be enhanced if, at the same time, strong onshore winds cause a storm surge. In order to describe river flood hazards, hydrologists undertake the preparation of two basic maps. One map delineates areas liable to flooding on average once every 10 years; the other map shows corresponding areas for 100 years flood cycles. A flood event which may be equalled or exceeded only in 100 years would inundate large areas of the flood plains. Where as a 10 year flood would cover a much smaller area, mainly in the neighborhood of the river banks. The methods employed by hydrologists can readily be applied for the preparation of maps for return periods other than those mentioned.

Storm Surges

Storm surge is caused by strong winds and low barometric pressures (usually generated by a cyclone) blowing a large sea surface. Water is thereby lifted and driven towards the coast. Where the depth is shallow, the return flow is retarded by friction at the sea bed, and the excess water piles up on the shore line until it eventually invades the hinterland. The originating phenomenon will probably be accompanied by heavy rains. Thus, the sum total of destruction may prove to be exceptionally high because of the contribution of three major factors--storm surge, heavy rainfall and increased discharge, if not actual flooding from rivers.

The most vulnerable parts of a coastline lying in the path of tropical cyclones are bays and shallow estuaries. To assess the hazard it is necessary to make a frequency analysis of storm surge heights along different sections of vulnerable coastline, and to consider, in addition, the possible combined effects of the meteorological surge and the tides.

Landslides

Landslides are also associated with earthquakes. However, consideration should also be given to the possibility of landslides where heavy rains and floods may occur.

Landslide hazard is difficult to estimate as an independent phenomenon. It seems appropriate, therefore, to associate landslides with other hazards such as tropical cyclones, severe local storms and river floods. This consideration is clearly observed in countries which, for the purpose of preventing or mitigating flood damage, also adopt measures to prevent hillside erosion and landslides.

Drought

It is not an easy task to define a drought because it depends strongly on the social impact. From the meteorological points of view, if for a certain period the precipitation is less than the normal average for the area or less than a percentage of that normal the drought occurs, but from the hydrological point of view it is necessary to consider the run-off for the area.

From the socio-economic side we will have drought when the water demand exceeds the water supply. This implies a definition of a critical value for each critical area. This critical value must reflect the needs of the social community, at least. Consider the Figure 1. If we are able to plot flow discharge of water needs against time and if the water demand is Q_p we will have a deficit from t_i (beginning) until t_f (end).

For any time t_x the deficit is $Dx=Q_p-Q_x$ and maximum deficit is reached when $D_M=Q_p-Q_m$.

The severity of the drought is estimated from the shadow are $\underline{D}$ and the total availability is Q x T-D. The index I=D/T can be an indicator of the intensity of the drought.

The knowledge of the distribution function of the precipitation is again fundamental for the estimation of this natural disaster, triggered by the level of water availability.

Assuming normal distribution of the precipitation it is possible for a certain region to compute the return periods of the drought and its intensity (long series of observations are necessary). Although both aridity and drought are identified by a lack of water, a distinction has to be made between the two concepts. Aridity is an average climatic characteristic where as drought is an extreme phenomenon.

It is rather difficult to identify the beginning and the duration of a drought as well as the total water deficit during it, because the water demand implies not only use of the surface storage capacity but also groundwater storage capacity. transfers between river basins or within the same basin, evaporation and evapotranspiration control, use of soil conservation measures and recirculation of used waters.

A suitable management of water resources under drought conditions implies the existence of the technical and institutional instruments necessary to carry out water resources management policy under normal conditions.

GEOLOGIC HAZARDS

Seismic Aspects

Seismic hazard is defind as the probability F(r) that a certain ground motion parameter will be exceeded in a period of (T) years. It is essential that the users of primary seismological data, or of hazard figures should be aware of the inaccuracies inherent in the data and of possible errors in the determination of individual parameters. Users should always ask seismologists and geologists for an assessment of the accuracy and confidence in the information and advice which they provide.

The procedures which provide probabilistic values of seismic hazard cannot always be followed in practice. Simple approaches may be used if the data required are not available, or if a rough estimate of hazard would be acceptable. In many countries, the largest macro-seismic intensitites that have been observed so far are regarded as defining the level of the hazard and, using this technique, maps can be constructed. However, such an approach may result in dangerous gaps since earthquakes may occur in places where no activity had previously been reported. Efforts are therefore being made, using geotectonic evidence, to improve the maps by extending the zones of largest macroseismic intensity.

Faults

Using geological, seismological and historical data it is often possible to assess the relative activity of a geological fault and to classify it into one of the following categories:

- Active
- Potentially active
- Uncertain activity
- Inactive

Active faults are marked by historical or recent surface faulting associated with damaging earthquakes; by tectonic fault creep or geodetic indication of fault movement; by geologically young deposits being displaced or cut by faulting; by fresh geomorphic features characteristics of active fault zones present along the fault trace; by physical ground water barriers in geologically young deposits; by stratigraphic displacement of quaternary deposits by faulting; by offset streams. Seismologically, earthquake epicenters are associated with individual faults with a high degree of confidence.

Earthquake-Related Hazards

While an earthquake is in progress, major hazards may arise as a result of the particular geological materials present in the localities where seismic shocks are taking place. Some notes on this important aspect are given in the following subsections:

Fractured Bedrock on Steep Slopes

Large masses of fractured rock forming the walls of valleys may be dislodged by a strong seismic shock. If the difference in elevation between the potentially unstable mass and the valley floor is sufficient for the mass to gain high momentum, the mode of movement will change from sliding and tumbling to an extremely rapid and destructive flow of rock fragments.

Loose Surface Materials on Steep Slopes

Steep slopes of coherent bedrock often have a surface covering of weathered material or soil a few metres thick. This material is often wet or saturated by rain or snow melt and the contact with underlying firm material forms a surface of low strength. Earthquakes can cause this layer to fail and descend rapidly, destroying farms on the slope and villages in the valley below. Such failures have resulted in heavy casualities in many areas of the world particularly in mountainous regions in tropical and temperature climate. The situation is aggravated where slash and burn agricultural practice has destroyed natural vegetation cover.

Liquefaction of Loose Flat-Lying Sedimentary Deposits

Some deposits in flat alluvial valleys have a very loose structure that is disturbed by seismic vibration. Inconsequence the component particles of a "sensitive" clay of fine sand, for example, assume a closer packing and smaller bulk volume. If the layer is initially saturated, the load from material above would not be carried by solid-to-solid grain contact but by the interstitial water. A soil in such a condition has effectively zero shear strength, and thus the sediments above are free to move under gravity forces towards any free face. The whole of the material above the liquefied layer may then spread laterally and break up into smaller units. Moreover, if buildings are founded upon a layer which is subject to liquefaction they may subside, break up, or tip over. Standard penetration tests and mechanical analysis of soils may be used as a first estimate for determining liquefaction potential.

Landslides

The term "landslides" is here used in its broad sense to include downward and outward movement of slope-forming materials--either natural rock and soil or artificial fill- by falling, toppling, true sliding along a surface or surfaces of shear failure, or by distributed movements involving lateral spreading or flowing.

Although individual slope failures generally are not so spectacular or so costly as some other natural catastrophes, they are more widespread and the total financial loss due to slope failures probably is greater than that for any other single geologic hazard to mankind. Moreover, much of the loss of life and damage occurring in conjunction with earthquakes and heavy rainfall are due to landslides triggered by shaking or by water.

Risk management requires knowledge of the specific areas which are subject to the hazard and, if possible, the ability to predict the time of occurrence. In this context, landslides are a type of hazard that is susceptible to a considerable degree of rational management. The kinds of geological and topographic environments that lead to high incidence of slope failures and the triggering agents that precipitate failure are relatively well known.

Mapping of areas subject to slope movements and delineation of the degree of hazard are now being successfully pursued in many parts of the world and the techniques used can be widely applied at various levels of detail and sophistication.

VOLCANOES

Volcanic emergencies differ from other types of large-scale emergency such earthquakes and hurricanes in that it is possible to delineate very specific and relatively small danger areas of generally less than 100 km^2 where devastation may be nearly total. In a situation where an eruption threatens to become violent, evacuation becomes a logical and necessary step. Precursors to a possible violent eruption of a known volcano may develop over a period as long as many months before the eruptive climax, and this poses the problem of deciding upon the point at which the evacuation of population becomes necessary and also the point at which the evacuation should be ended.

In other regions where volcanic activity occurs at locations over a wide area without clear correlation with previous craters, the interpretation of possible precursors is more difficult. Decisions involving mitigation of risk after the outbreak of an eruption should, however, be based on experience of the character and course of previous eruptions at the better-known volcanoes in the region. There are numerous different types of volcanic activity which present substantially different hazards. For example, glowing avalanches may descend the flanks of a volcano at speeds in excess of 100 km/h, whilst lava flows generally advance at no more than a few tens of metres per hour.

Information on volcanic hazard is best presented in the form of hazard zoning maps. Such maps must be based on the records of each volcano's history, using all historical data and extended back by stratigraphic studies. The products of each eruption should be identified their areal distribution and volume measured, and the type of eruption established. It is also worthwhile monitoring the chemical composition of the materials emitted during the course of a prolonged eruption, because systematic change in composition can in some cases be correlated with the type and violence of volcanic activity. Such zoning maps show the nature and frequency of specific volcanic hazards, and hence the risk to life and property.

Where the type of volcanic activity is capable of causing total devastation, the vulnerability is 100 percent and the risk is directly proportional to the hazard. A numerical assessment of the hazard should be made by vulcanologists after they have carried out a systematic and comprehensive study of relevant historical precedents. The nature and violence of most volcanic phenomena make it practically impossible to reduce the vulnerability of human life and property to below 100 percent. The only way to mitigate risk is therefore to reduce the elements at risk either, on a long-term basis, by restricting human settlement and investment in hazardous zones or, in a short-term basis, by evacuating populations and movable goods from such zones during periods of increased hazard (i.e. periods of actual or predicted eruptive activity).

There nevertheless remain certain possibilities of reducing vulnerability to some volcanic phenomena such as ash falls, lapilli showers, etc. Sloping roofs are less liable than flat roofs to collapse under layers of ash; windows of houses may be boarded up to reduce the risk of fires started by incandescent lava bombs, etc. Some agricultural crops are less vulnerable to ash falls than others.

SEISMIC VULNERABILITY ANALYSIS

General Comments

An earthquake, even of moderate intensity, releases an enormous amount of energy and usually triggers a complex chain of events, the effects of which are potentially hazardous to man and his environment.

The analysis of these events is rather difficult because of their interrelationships. Nevertheless, one can roughly define three categories of chain effects generated by an earthquake:

- the direct effects of the earthquake which are related to its magnitude, fault displacement and ground-shaking;
- modifications of ground motions on unstable geological features or structures. Ground motions can be either amplified or reduced by local geological conditions. A given level of ground-shaking may induce liquefaction in the presence of loose sand layers;
- the reaction of man-made structures to earthquake forces (which in fact reveal the specific vulnerability of these structures).

For urban planning purposes, the formulation of preventive measures requires rather detailed information which cannot always be drawn exclusively from historical data:

- the knowledge of epicentre zones from past records cannot generally differentiate risks within an urban area whatever its size. These records can only provide basic information for estimating probabilities of occurrence of various magnitudes over the whole area.
- the factors which may differentiate risks within the area are geological conditions whenever their amplifying effect is uneven.

Numerous studies throughout the world have dealt with the amplifying effects of the subsoil and their consequent effects on structures. During an earthquake, two kinds of phenomena liable to cause disasters can be defined:

- ground response as determined by ground acceleration and soil composition in a given place.
- resonance which relates the amplitude of ground vibrations with the natural period of built structures.

The latter is related especially to the height of buildings, where as the former phenomenon is related to the thickness of sediment. In determining vulnerability due to resonance, soil analyses are required down to bedrock level.

The following items will therefore be analysed:

- physical factors related to ground response,
- physical factors related to resonance,
- a comprehensive seismic vulnerability index,
- land-use and building constraints,
- a comprehensive vulnerability mapping system.

Physical Factors Related to Ground Response

Geological data are the basis for determining ground response. However, it is usually not possible to find standard geological maps containing sufficient data for this purpose, particularly in areas underlain by unconsolidated sedimentary deposits where, according to historic records of large earthquakes, some of the greatest loss of life and structural damage have occurred. Such sedimentary soils are found in flat low-lying areas which are often densely populated, and even highly developed. Using the findings of geotechnical studies, a ground response index (I) can be built up from the following components:

- a basic component (i) related to subsoil composition and the volume of sedimentary layers.
- other components related to the specific characteristics of the area, such as the presence of faults (f), loose sand layer(s)....

STUDY OF FLOOD VULNERABILITY

General Comments

The determination of flood risks is in general less complex than of earthquake risks. Usually a flood has two direct means of impact: the flood level and its velocity. The frequency and even the intensity of flooding can fairly easily be estimated, given the wide availability of modern hydrological methods and techniques. Moreover, the relative simplicity of the flooding process, compared to the complex chain of events triggered by seismic shaking, allows one to estimate with a fair degree of accuracy the vulnerability of a given location.

The present study for integrating flood risk in urban planning covers:

- description of the physical factors related to flood risk,
- the impact of existing flood control measures,
- the determination of land-use and building constraints,
- the synthesis map of constraints related to floods.

The characteristics of flood are closely related to their origin. In the present context the following may be noted:

Flooding Due to Heavy Rainfall

Generally, this kind of flood is very sudden and rapid (flash flood), and can be triggered in the hours immediately following heavy rainfall. The frequency of occurrence is of course closely linked to local climatic conditions, the topography and the operating conditions of the drainage system.

Flooding Due to River Bank Overflow

While this type of flood is not as sudden or fast moving as the former, it nevertheless presents the highest risk within its area of likely impact. It is generally the most damaging type of flooding and is almost always caused by topographic conditions (such as shallow river beds, incapable of containing high rates or levels of low). But this type of flooding is often aggravated by human intervention, reducing out-flow.

Physical Factors Related to Flood Risk

The frequency and intensity of floods are related; usually, the more frequency the flood, the less intensive it is. The two major parameters of intensity are:

- the height or level of flooding, for any kind of flood
- the flow velocity, for river over-banking and erosion

Height of Flood (Flood Level) and Frequency

For a given location the estimation of the flood level is related to its frequency as indicated above. It is normal to consider 10 year, and 100 year flood levels (these are also known as the "return periods").

Maps of flood-prone areas can be drawn up marking, in the form of contour lines, the differences between flood levels and ground elevation.

Flow Velocity

Flow velocity has a dynamic effect usually characterized by evidence of soil erosion and the destruction of man-made structures (buildings, bridges, etc.).

Criteria for Flood Vulnerability

In order to define flood vulnerability one must first establish a typology of damage related to flood risk, namely:

- Loss of lives
- Capital losses (damage to buildings and civil engineering)
- Operating losses (disruption of economic activities and services).

Frequent floods are characterized by high operating damages, but cause little if any loss of life or capital loss.

Infrequent floods are characterized by important losses of life and high capital losses, operating losses being minimal by comparison. In discount values over a long period, it is clear that frequent floods can provide a fairly good index for measuring operating losses, while infrequent floods are clearly indicative of capital losses.

Therefore, for a given area, vulnerability is a function of the heights (or levels) of frequent and infrequent flooding.

PART B

According to the introduction given above, the hydrologic hazard must be defined taking into account not only the total amount of precipitation received by each of the considered basins throughout the studied area but also the intensity and the duration. A flood has at least two direct impacts: flood level and its velocity. Thus the vulnerability of a given location seems to be simpler than the seismic one to be established.

The integration of flood risk in urban planning is made taking into consideration:

- a description of the physical factors related to flood risk
- the impact of existing flood control measures
- the determination of the vulnerability factors
- the determination of land-use and building constraints
- the synthesis map of constraints related to floods

<u>Flooding Due to Heavy Rainfall</u> (an example)

The heavy precipitation needs to be known all over the region, and particularly for short periods when the infiltration intensity is reduced and the normal run-off exceeded. In order to find an easy estimation of maximum precipitation intensities, we present this simple exercise.

Over an area of about 100.000 km^2 more than 500 rainguages are providing 24 hour values but only from 20 we have available data for shorter periods. With a complete and homogeneous series we can use the Gumbel distribution function. For the whole sample the extreme of precipitation extreme, in order to define the necessary prevention of the flood and confinement according to the statistics used.

TABLE 1

AREA (Km^2)	MAXIMUM FLOOD DISCHARGE FOR THE PERIOD OF OCCURRENCE (m^3/s)						
	5y	10y	25y	50y	100y	500y	1000y
50	43,83	56,46	67,79	75,98	91,14	114,91	134,13
100	73,46	94,63	113,61	127,34	152,75	192,59	224,79
150	99,37	128,01	153,68	172,24	206,61	260.50	304,07
200	123,12	158,60	190,41	213,41	255,99	322,77	376,74
250	145,39	187,29	224,84	252,01	302,30	381,14	444,88
300	166,54	214,53	257,56	288,67	346,28	346,59	509,61
350	186,81	240,64	288,90	323,80	388,42	489,73	571,62
400	206,35	265,81	319,12	357,67	429,05	540,95	631,41
450	225,28	290,19	348,38	390,47	468,40	590,56	689,32
500	243,67	313,83	376,83	422,36	506,64	638,79	745,61

Rainfall Prediction and Flood Phenomenon in Small Basins or in Urban Areas

(Strategy established in the National Centre of Weather Analysis
and Forecast-Lisbon-Portugal)

Bearing in mind the features of the flood in small hydrographic basins or in urban areas, namely its short concentration time, the meteorological disturbance from which it originates should be dealt with on the space-time scale that best fits those features.

The size of the basins (tens of square kilometers) and the concentration time (some minutes, or at most one or two hours) require that the meteorological disturbance should be dealt with on a subsynoptic scale of the same space-time magnitude.

Next Sections include a brief discussion on the present knowledge and value average $\underline{M}$ corresponds to a certain return period Teva. For each of the reference (20) sites we compute the extreme values corresponding to the 100 years return period (M100) and the ratios:

$$R^{0}_{4} = \frac{M\ (6\ \text{hours})}{M(24\ \text{hours})}\ ; \quad R^{0}_{24} = \frac{M\ (60\ \text{minutes})}{M\ (24\ H)}\ ; \quad R^{0}_{144} = \frac{M\ (10\ m)}{M\ (24\ H)}$$

and

$$R^{100}_{4} = \frac{M_{100}\ (6\ H)}{M_{100}\ (24\ H)}\ ; \quad R^{100}_{24} = \frac{M_{100}\ (60\ M)}{M_{100}\ (24\ H)}\ ; \quad R^{100}_{144} = \frac{M_{100}\ (10\ M)}{M_{100}\ (24\ H)}$$

With these values for the 20 sites irregularly spaced over the area we contour maps for each one of the ratios and return period. We assume that the contouring is covering the area where the other observation stations are. The estimation for return periods other than Teva or T_{100} can be obtained using the extreme probability paper, from the straight line defined by M and M_{100}. For durations up to 24 hours we can use some graphical techniques such as that mentioned above.

Example: (Portugal-figures 2 to 7)

In one station--Alcobaca--we need to estimate the maximum annual extreme of rainfall for 6 hours, 1 hour and 10 minutes for return periods of 5, 10, 25, 50 and 100 years. The extreme value average for that station in 24 hours (30 years of observations) is M (24) = 49.0 mm (Teva-2.33 years) and for the return period of 100 years M_{100} (24H) = 99.0 mm.

From the contour maps prepared we get for that station

Return period / Duration	2.33	100
10 M	20%	26%
60 M	40%	50%
6 H	69%	73%

Therefore:

Duration / Intensity (mm)	10 M	60 M	6 H
M (24 H)	9.8	19.6	33.8
M_{100} (24 H)	25.7	49.5	72.3

Figure 1

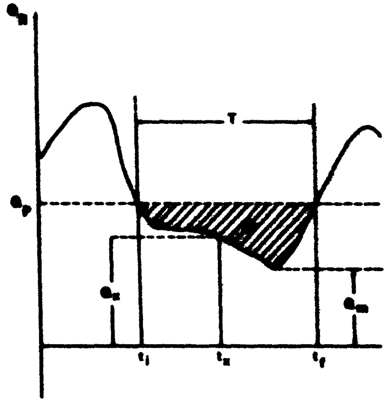
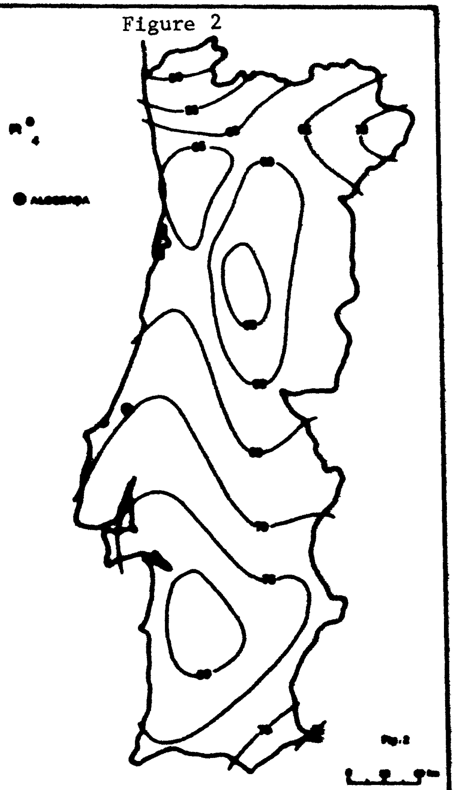


Figure 2



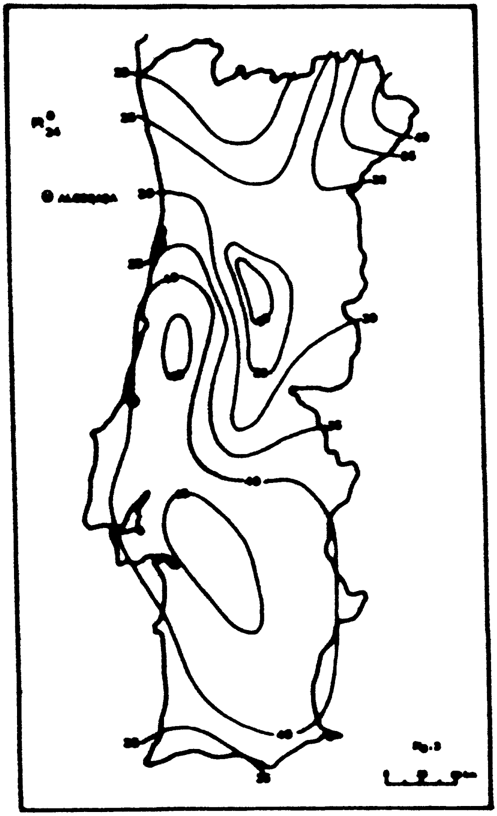


Figure 3

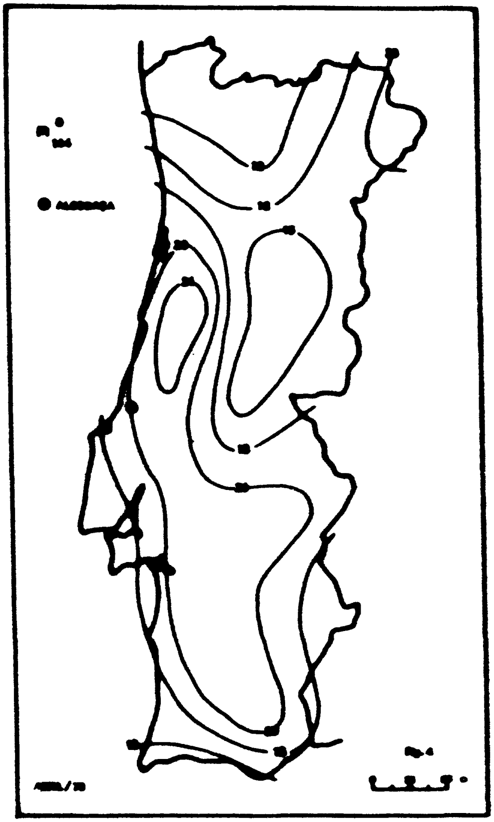


Figure 4

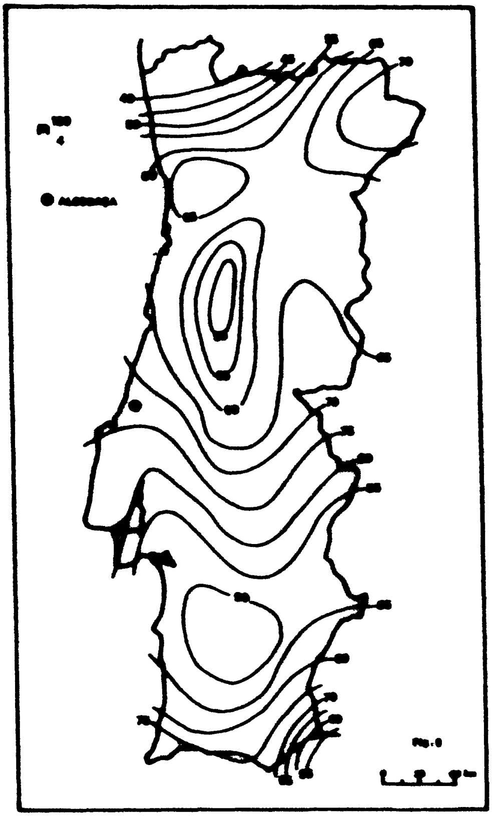


Figure 5

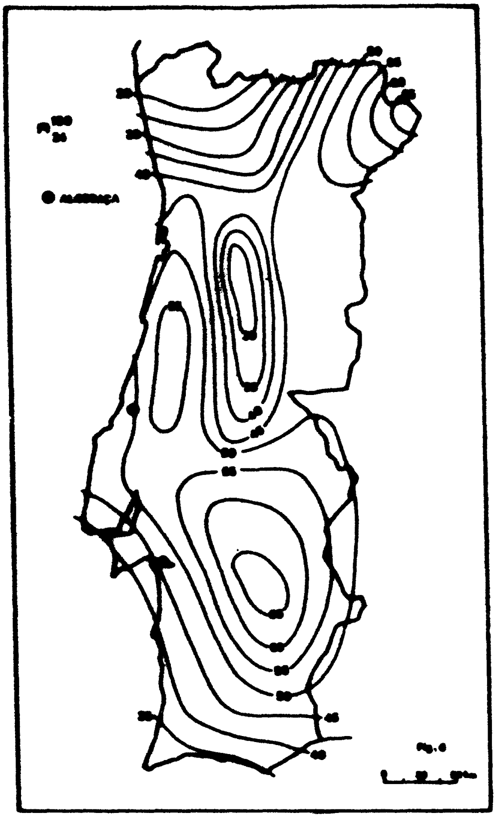


Figure 6

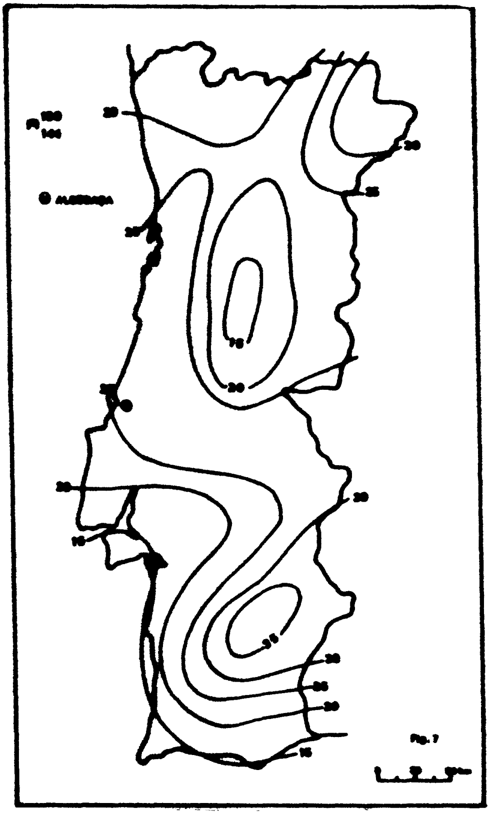


Figure 7

Using the extreme probability paper we obtain the extreme values for the return periods we have been looking for, (fig. 8). With the values estimated by this way we can establish the curves defining the full range behavior (10 M to 360 M) for return periods of 5, 10, 25, 50 and 100 years, (fig. 9). The knowledge of these extremes is fundamental for the computation of hydrologic risk, and it has been proved for a surface coverage better than 1900 km/station (44 km grid).

Maximum Flood Discharge

In order to attempt to establish some empirical formula which will conduct to the peak flow estimation for different river basins it is necessary to take into account the characteristics of each one and the occurrence period too, covering a long series of observations (homogeneous and complete series of the annual extreme values of flow discharge). The extreme values are sometimes lacking for long series and the application of the Gumbel distribution function can lead to erroneous conclusions. If we plot in the extreme probability paper the values corresponding to each station against the areas of the river basin, avoiding physical and geological configurations we will get important dispersion. The experience has shown that taking that into account it is possible to define a general area behavior for the desired return periods expressed by a law of the type:

$$Q_P = C.A^Z$$

Where Q_P is the peak flow of the flood associated to the return period of $\underline{T}$ years for the river basin of $\underline{A}$ km surface, $\underline{C}$ is the parameter related to the return period T and Z is a factor associated with the configurations of that basin.

Example: (Guadiana River Basins)
$Z = 0.745$

	Return Periods (years)						
	5	10	25	50	100	500	1000
C	2.377	3.062	3.676	4.120	4.943	6.232	7.274

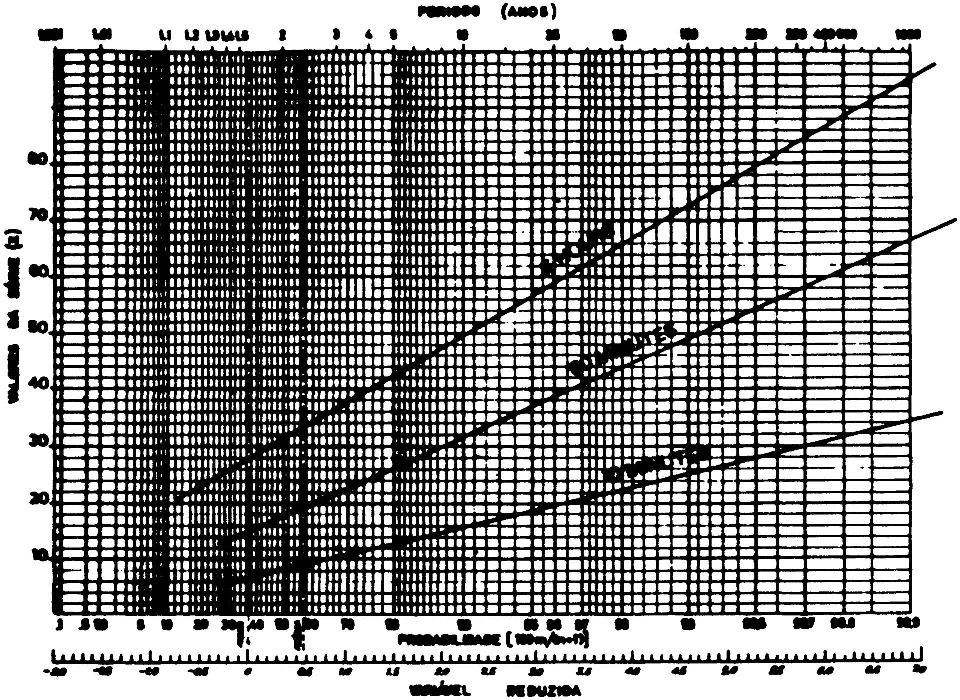
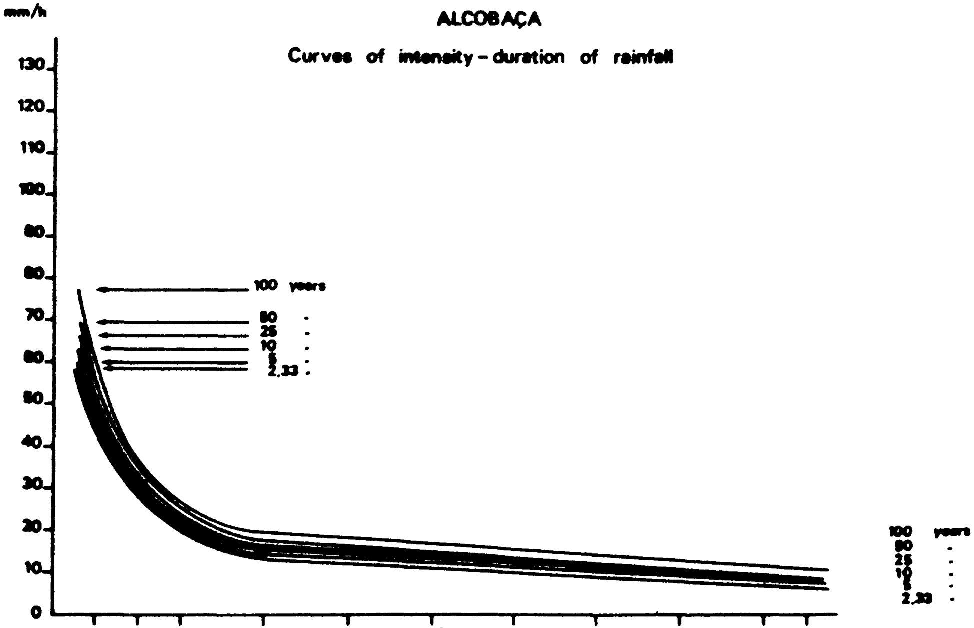

PERÍODO (ANOS)
1.1
1.2
2
3
4
5
10
25
100
1000
VALORES DA SÉRIE (x)
10
20
30
40
50
60
70
80
PROBABILIDADE [100 m/(n+1)]
VARIÁVEL REDUZIDA
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0

Figure 9


mm/h
ALCOBAÇA
Curves of intensity-duration of rainfall
130
120
110
100
90
80
70
60
50
40
30
20
10
0
100 years
50 .
25 .
10 .
5 .
2,33 .
20
40
60
100
140
180
220
260
300
340
380
minuts
100 years
50 .
25 .
10 .
5 .
2,33 .

With these estimations it is possible to establish an easy interpretation using the Table I and the Figures 10 and 11. Thus, if the data is available, it is possible to evaluate the most damaging flood. We must remember that the more frequent the flood, the less intensive it is. Nevertheless it is very important to monitor the height of the level of flooding for anyone using the technique meteorological technology, at the national level, concerning analysis and forecast of meteorological disturbance on that scale.

PRESENT STATE OF FORECAST

Quantitative forecasts of the precipitation amount, available at the National Institute of Meteorology and Geophysics (NIMG), result in the use of highly sophisticated physico-mathematical models at global level and with a space resolution on the synoptic scale and a time resolution on the subsynoptic scale.

This product, obtained from the model now in use at the European Centre for Middle Range Weather Forecasts (of which Portugal is a Member State), is still in its experimental stage and must, therefore, be considered with the utmost caution.

Due to the great space-time variability of the precipitation and to the strong influence of local topography, the accuracy of global and synoptic models is relatively limited. The predictability depends very much on the place, and generally does not exceed 72 hours. The best results achieved refer to predictions of precipitation amounts equal or less than 5 mm in 24 hours.

In general, the values predicted are above the ones observed, and the deviation is greater for a larger number hours covered by the forecast. It should, however, be noted that for very high values, the deviation is reversed.

These results were obtained by the ECMWF towns in Europe. However, the tests already carried out in the mainland territory lead to opposite results concerning the sense of the error in forecasts between 5 and 20 mm in 24 hours.

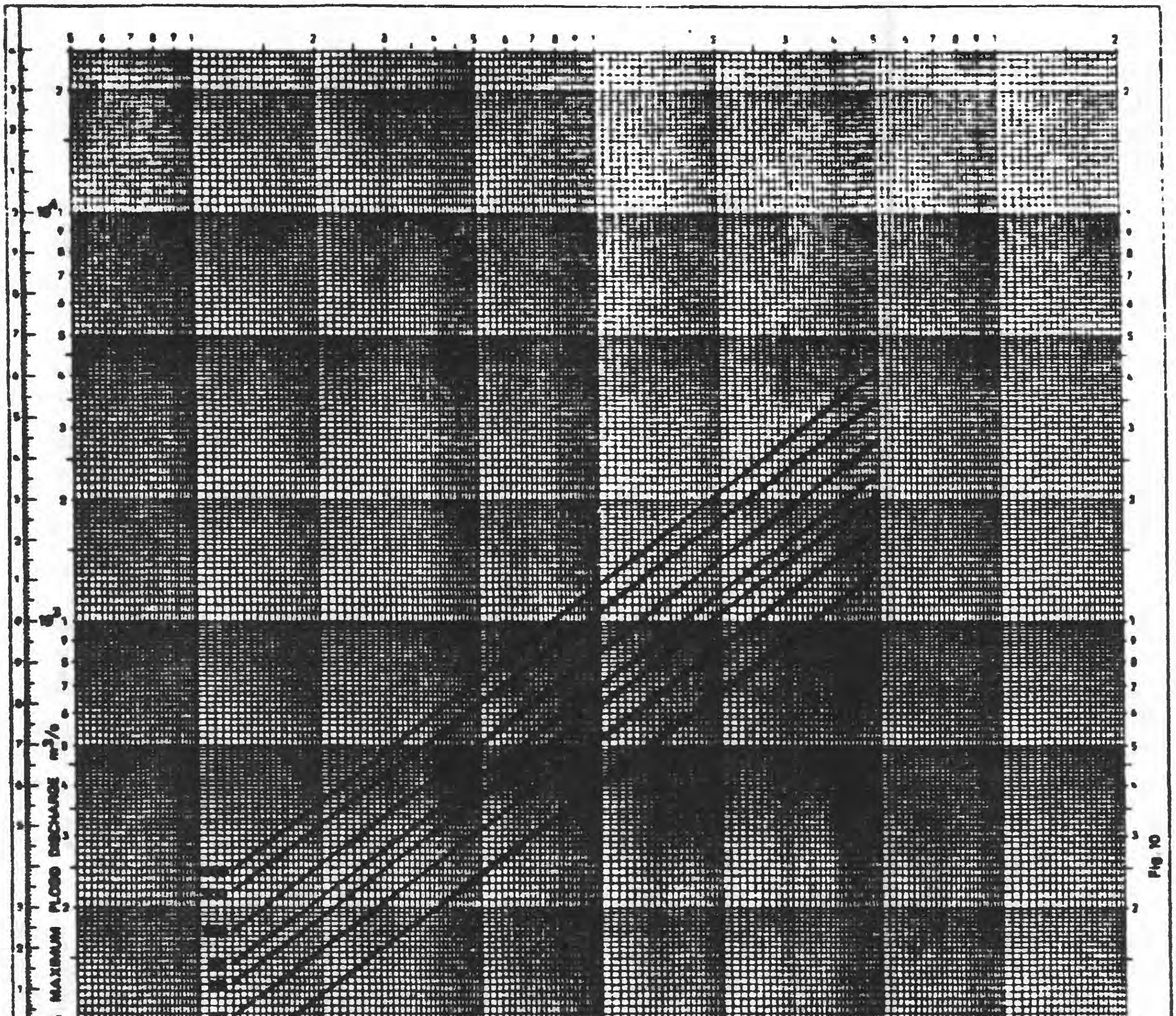


Figure 10

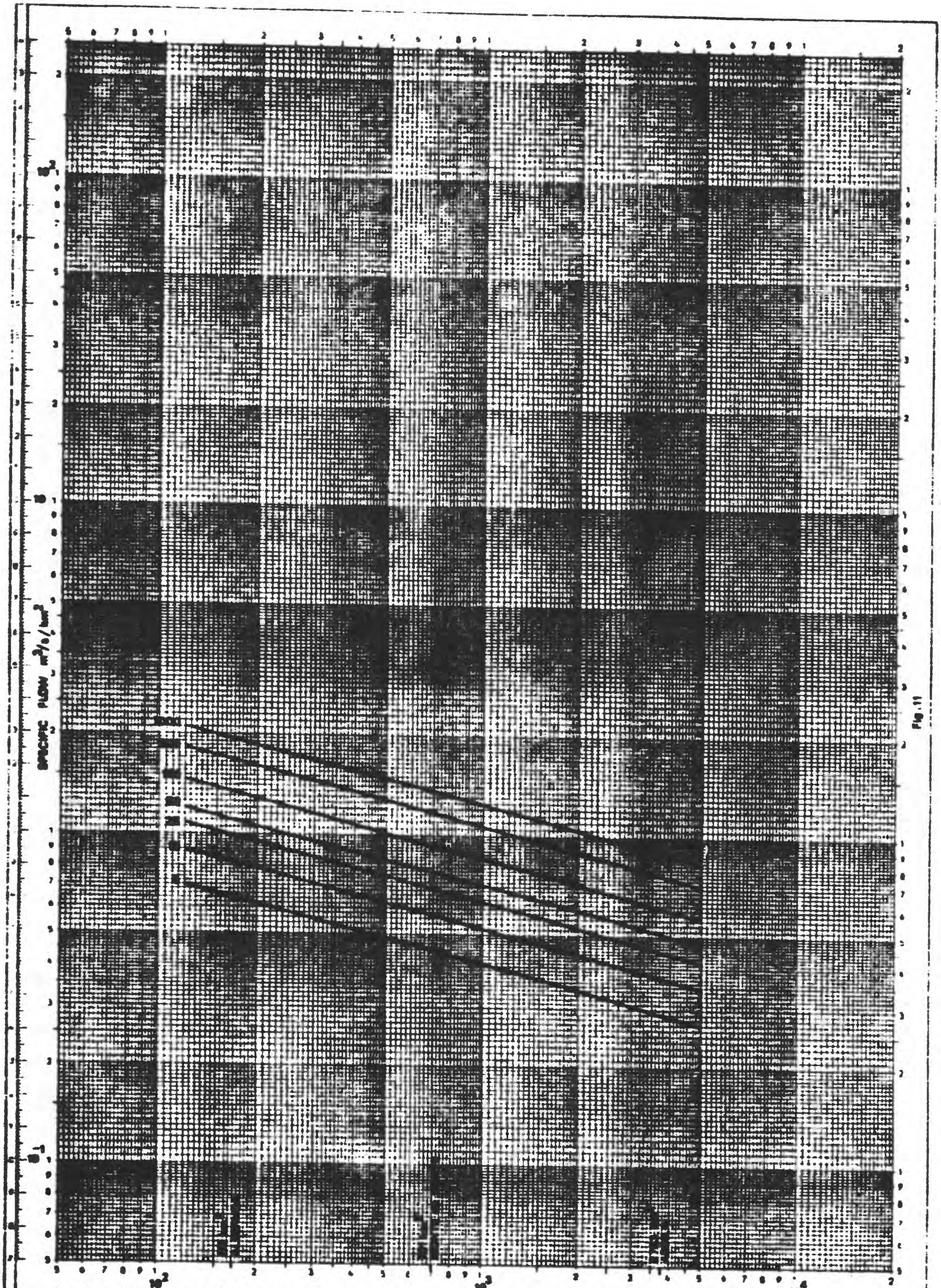


Figure 11

On the basis of the tests already carried out for the mainland it can also be admitted that, for large basins, such as the one of the Tagus, the values predicted are a good indication, better than persistence and climatology.

These are, in short, the features of the precipitation amount forecasts daily available at the NIMG.

Thus it can be conclued that, for this problem, the use of these forecasts is very limited, although they are very important as a forecasting element for other purposes.

WEATHER WATCH

Due to the above mentioned limitations, and while more sophisticated models of finer space-time resolution have not been developed, the means available and used in meteorology to solve this problem are the observation and tracking of the disturbance in high space-time resolution, a true permanent weather watch.

This watch is carried out at the National Centre of Weather Analysis and Forecast of the NIMG, especially through three kinds of observation:

- Observation of the precipitation amounts occurred in the NIMG network of surface meteorological stations over 6 hour periods, which in case of need can be reduced to 3 hours or less.
- Observation of cloud patterns using imagery from meteorological satellites, visible and infrared, at six hour intervals that can be reduced up to 30 minutes, if necessary.
- Observation of cloud patterns and precipitation intensity using a meteorological radar device in Lisbon, covering a circle with a 200 km radius.

These observations are made every 6 hours, or even permanently if necessary.

This watch is based on the tridimensional thermodynamic analysis of the troposphere every 12 hours and on surface analysis every 6 hours, with monitoring of the meteorological information from the national network every 3 hours.

This watch enables tracking the meteorological disturbances that can affect the weather in the area Azores-Madeira-Mainland and taking three kinds of steps:

- Change or adjustment of short range forecasts (24 to 72 hours)
- Storm warnings to the Maritime Authorities, Fire-Brigade, etc.
- Alert warning to the National Service of Civil Protection in cases of extremely violent disturbance.

In this case the National Centre of Weather Analysis and Forecast of the NIMG sends on alert warning to the NSCP whenever the precipitation amount over 6 hours in urban areas is equal or over about one forth of the 24 hours maxima with a return period of 50 years and the rainfall is expected to continue (Fig. 12 and 13).

SEISMIC HAZARDS

Tectonic Environment of the Western Mediterranean Area

It is generally assumed that at the triassic only one supercontinent would exist (Pangea) or two (Gondwana and Laurasia) integrating the total existing Lythosphere. Bullard et al (1965) were followed by Smith (1971) in that successful reconstitution taking the 500 fathoms line.

For the North Atlantic Region the most interesting composition, Fig. 14, was discussed by Le Pichon, Biju Duval and others in 1976 taking the continental plateau limit line (2000 to 3000 m) and the structural features. Some other reconstitutions have been undertaken by other scientists such as Tapponier (1976), Fig. 15.

The global movement is understood as a left lateral displacement of Africa in respect to Europe following the Azores-Gibraltar-Sicilia transform fault with 35° rotation of the Iberian Peninsula.

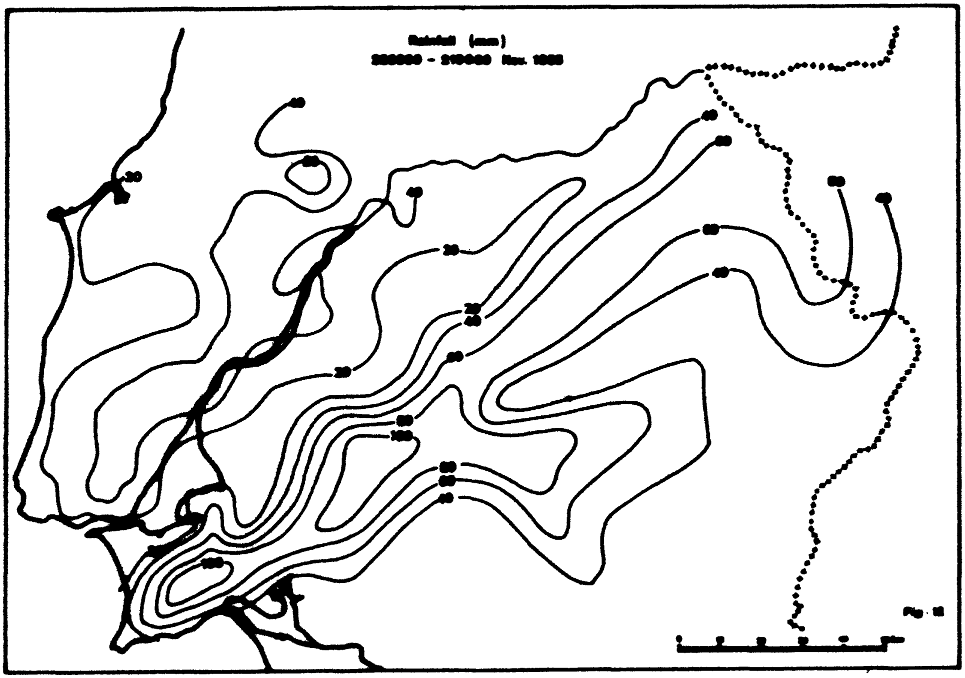


Figure 12

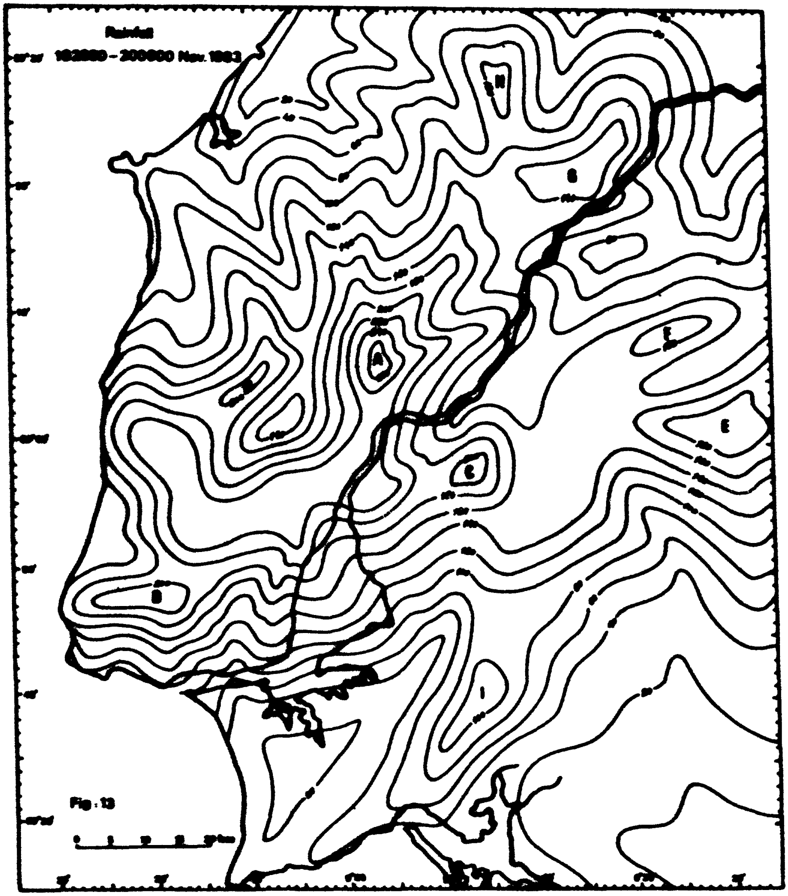


Figure 13

Fig. 14 - Reconstitution of the plates Europe, Africa and South America before the opening of the Atlantic Ocean (After Biju Duval and others, 1976)

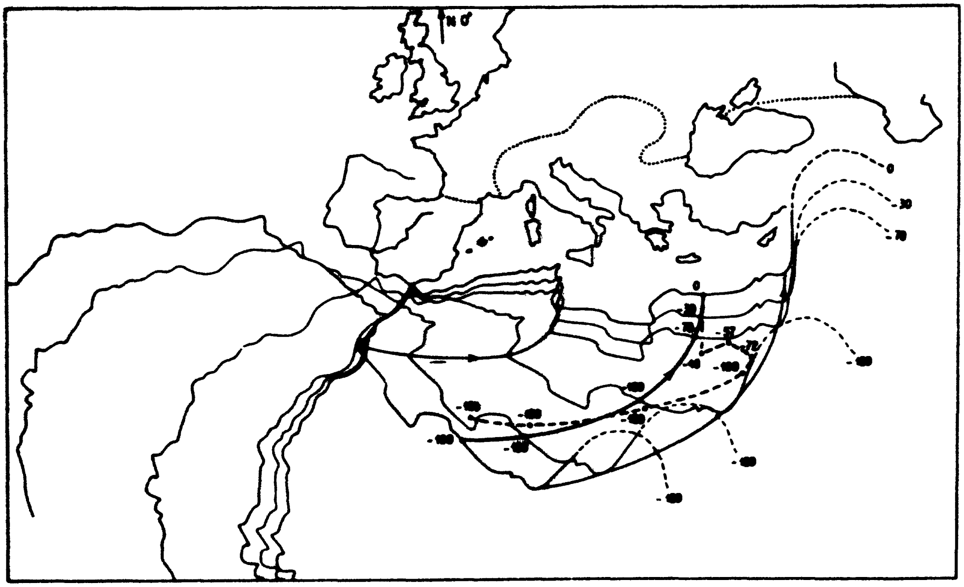


Fig. 15 - Displacement of Africa and Iberia in respect to Europe (Tapponier, 1976)

From 180 to 80 My a large displacement (1,800 km) left lateral took place followed by a right one less important (500 km) at the same Gibraltar level and compression from 50 my ago. The rotation finished 70 my ago.

After that evolution the tectonics today are mostly monitored by the earthquakes felt and studied in this century. In the Figure 16 we present a sketch for the Mediterranean area with the location of the main shocks, but going down in the magnitude of the event, the picture will be composed differently.

In general we can accept that the seismic hazards in the Western Mediterranean area are located close to the picture presented by Hatzfeld (1976) (Fig. 17 and 18).

General Mechanism

Klaus Meyer and others from Uppsala, Sweden, in 1983, tried to establish the migration patterns of the large observed seismicity from 1901 to 1981, concentrating the investigation to the region above, i.e., the western segment of the Azorean-Iran belt with limits 25--50^{o}N in latitudes, and 40^{o}N--60^{o}E in longitudes. But, in complicated patterns, where a considerable number of sub-plates are present, they try to explain the situation mechanism, taking only the longitude of the epicentres.

The foreshocks and the aftershocks were excluded (an event is considered a foreshock if it has occurred less than one month before or after and less than one degree from the main shock).

The cut-off magnitude (body-wave magnitude was 6.4. A series of 161 earthquakes was considered homogeneous and complete from $m > 6.4$ without consideration of their focal depth.

Available epicentral longitudes were plotted as a function of time occurrence of the respective earthquakes. They identified a prevailing eastward progression of large earthquakes with propagation velocities for individual probable migration sequences from 3500 to 5300 km/yr. and established that large

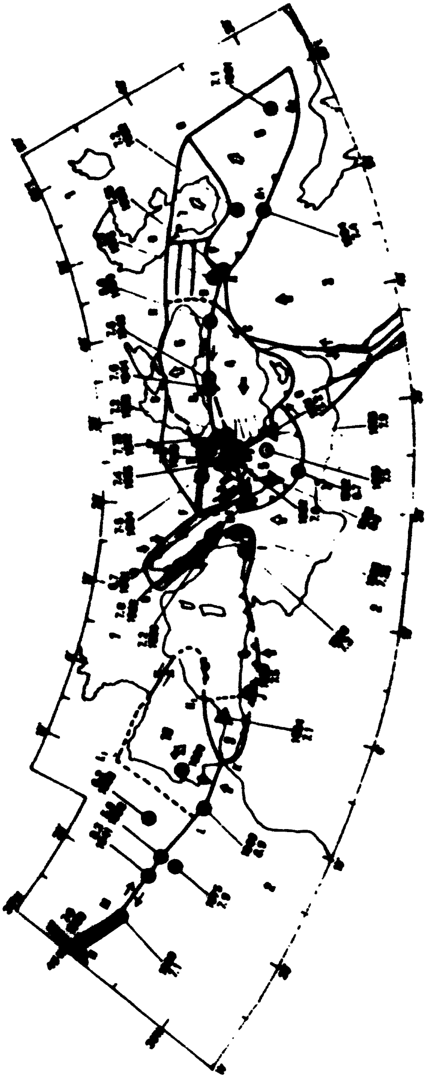


Figure 16. Seismotectonic sketch of the Mediterranean Region

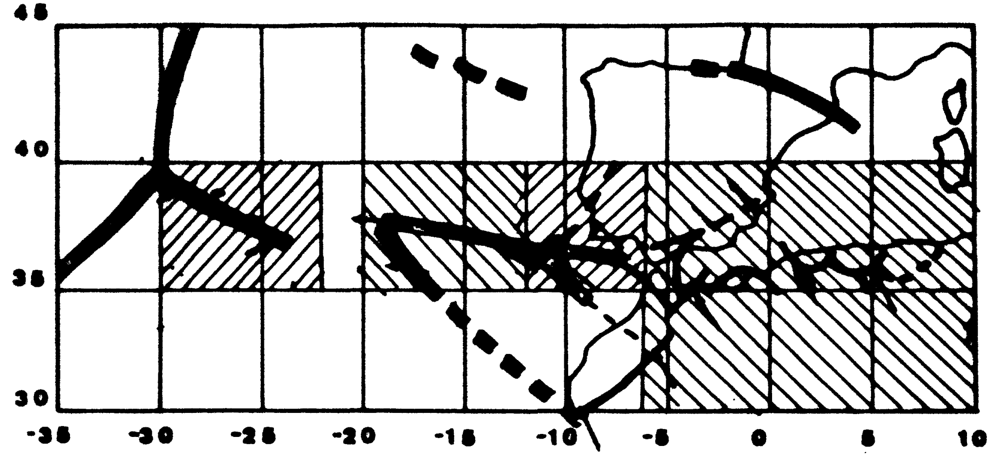


Fig. 17 - Seismicity of the Azores-Sicilian area. The arrows area showing the movement of Africa respect to Europe

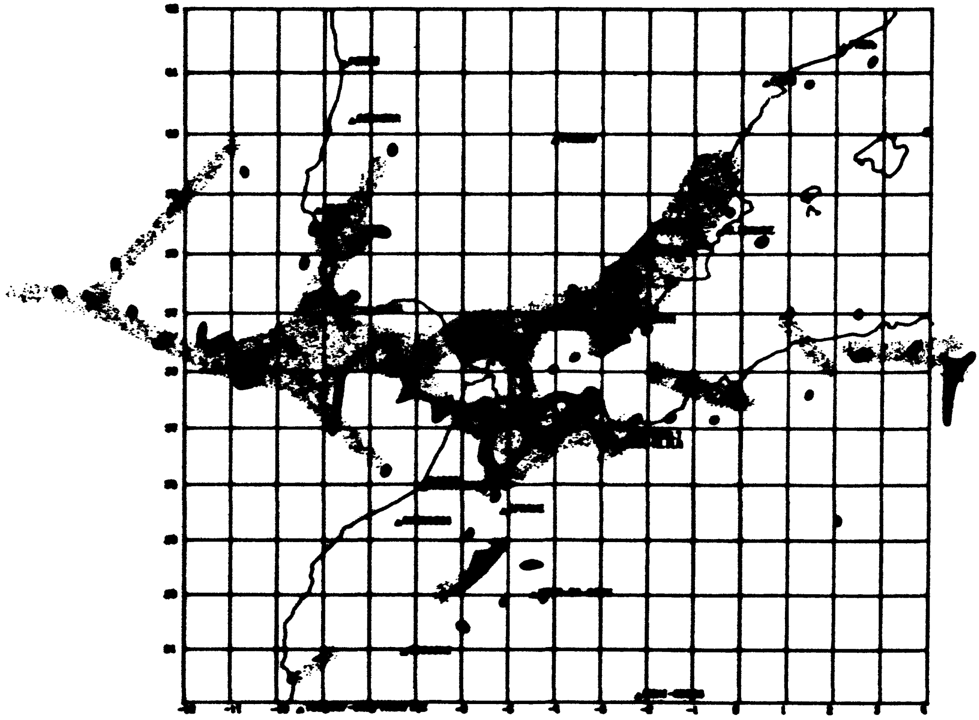


Fig. 18 - Seismic hazards in the Gibraltar arc (black areas are locations of epicenters within 25 km distance, gray are subjective interpretation). After Hatzfeld (1976)

events west of $3^{o}W$ reoccur with almost constant return period of 5 yr. But, we cannot accept this mechanism as a final achievement.

Seismic Microzonation Gradients

Seismic hazard analysis provides probabilities of occurrence of exceeding of a certain ground motion or derived parameter related to a reference.

For planning or construction on the scale of a town, the hazard estimates will contain more parameters, determining the seismic microzonation. The results may change the average hazard figures.

The scientific uncertainties which beset the microzoning techniques demand that extreme caution be taken when trying to carry out microzoning of recent alluvial deposits.

In order to introduce the better guidance for seismic microzonation in a capital area like Lisboa we develop recently a cooperative program. Some results will be presented at Eighth World Conference on Earthquake Engineering (S. Francisco, USA, 21-28 July, 84).

This very important exercise was conducted by an interdisciplinary group that was able to prepare the articulations of the different tasks envolved in that program as it is presented here (Fig. 19).

FINAL REMARKS CONCERNING NEW FACILITIES

The disaster management community should be prepared to use the newly-available high-resolution satellite data by organizing itself to provide for:

- participation in simulation studies,
- identification of major, disaster-prone areas for early data acquisition and archiving,
- rapid identification of disaster-stricken areas and issuance of requests for data acquisition,
- interpretation of data,

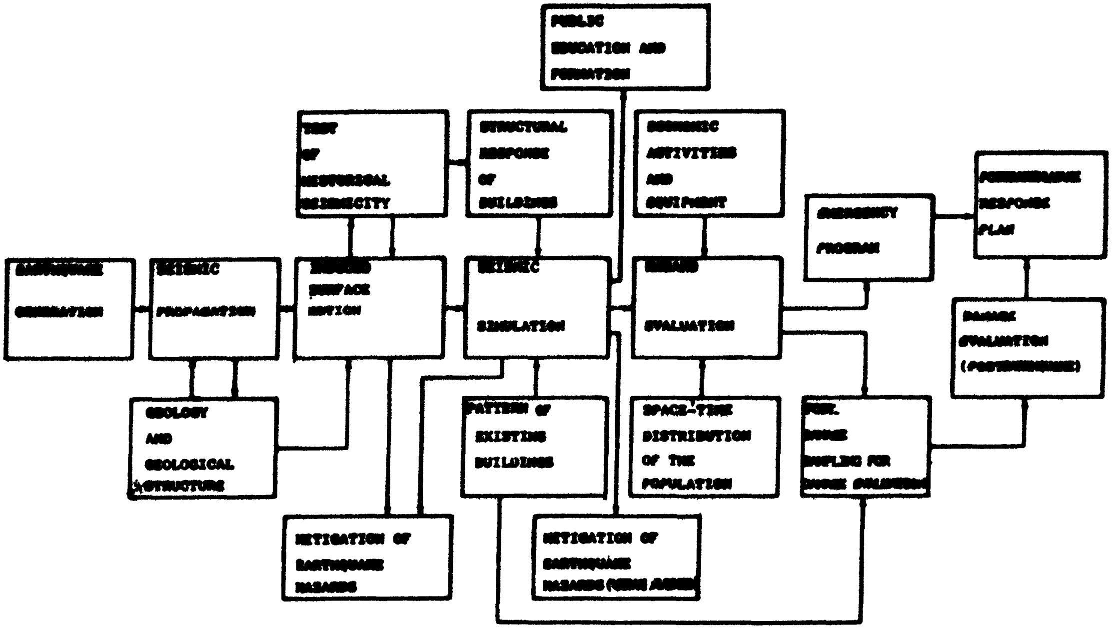


Fig. 19 - Seismic microzonation guidelines

- analysis of results with respect to damage assessment and integration of analyses into general descriptions of disaster situations,
- dissemination of results as part of normal distribution of information on disaster situations.

Large efforts are needed to inform the situation in order to establish:

- The spatial and temporal resolution and spectral bands required in a land observing satellite to allow the extraction of the information needed for some appropriate and significant problems in disaster management.
- The feasibility and desirability of a communication system using polar orbiting satellites, such as the NOAA spacecraft, which could provide twice a day bi-directional communication of a short (less than 100 words) message from a small man pack carried by a field disaster manager to and from a central communication facility.
- The feasibility and desirability of a system that could provide a few voice channels via a dedicated segment of a satellite using low frequencies.
- The utility of soil moisture data, such as could be acquired by a microwave sensor (10 km resolution, global coverage every 3 to 5 days) to drought and crop monitoring activities in disaster management.
- The most desirable and feasible augmentations of the direct broadcast meteorological data (APT and HRPT) from polar orbiting satellites to help countries concerned with improving their warning services for tropical cyclones and floods in remote areas and to provide additional data for drought and crop monitoring.

The radar and spatial observations will have considerable impact on:

Meteorology and Hydrology

- improved accuracy and/or timeliness of tropical cyclones forecasts;
- improved (and predictive) climate models;
- global flood alert systems;
- implementation of national/regional flood warning systems;
- improved surface level wind velocity measurements (esp., as already demonstrated, over oceans);

- improved rainfall measurements over land for flood warnings (If these measurements cannot be performed synoptically, they would still be useful for calibrating surrogate monitoring techniques such as the estimation of rainfall through cloud mapping);
- soil moisture measurements (for flood warnings and crop disaster warnings).

Geology and Geophysics:

- timely acquisition of high-resolution images for disaster management, in particular damage assessment;
- archiving of imagery of disaster-prone areas for change detection;
- precise, continuous contemporary determinations of rates of movements of seismically-active plate boundaries and faults.

Hazard Mapping: (the development of base maps indicating hazard distribution by type, intensity and frequency). Imagery from space can be used in mapping various types of hazards though it generally must be employed together with data from other sources. The most notable applications are for tectonic and flood plain maps. Tectonic maps are used in combination with other Geological and Geophysical data in the determination of earthquake hazard to supplement missing information on seismic history in combination with other geological and geophysical data. Space imagery can be used directly in geomorphological mapping. In constructing a tectonic map, one looks for signs of faulting, the surface manifestation of which is often a lineament. Such lineaments are alignments (of topography, river valleys, vegetation, etc.) which are often easily mapable on synoptic imagery though they may be undiscernable on the ground or even from aircraft. Many seismic hazard maps have been constructed on the basis of such lineament information.

In addition, small horizontal and vertical movements of the tectonic plates or blocks which may indicate neo-tectonic activity are also monitored. This requires geodetic measurements using sophisticated instrumentation carried out at either the local or regional scales.

There are several approaches to the hazard and risk mapping of flood plains but the end objective is the same: to plot the boundaries of various flood levels in terms of their recurrence probability.

Land Use Mapping: another input required for vulnerability and risk analyses is a land use map of appropriate scale. Remote sensing satellites can provide such information, though perhaps not always at the level of detail required. These data can be used to distinguish among broad categories of land use: agricultural, semirural and urban. Within these categories, some sub-categories may be distinguished (e. g., transport centers, commercial centers, grazing land, etc.).

Vulnerability Analysis: since, as explained above, satellite imagery provides a cartographically accurate data base combining some aspects of hazard, vulnerability and land use, these elements can be readily and effectively integrated in performing risk analyses. For example, the spatial relationships between tectonic lineaments and urban development can easily be perceived in images containing both types of information.

DEFINITION OF UNDRO TERMS[1]

Natural hazard, meaning the probability of occurrence, within a specific period of time in a given area, of a potentially damaging phenomenon.

Vulnerability, meaning the degree of a loss to a given element at risk, or set of such elements, resulting from the occurrence of a natural phenomenon of a given magnitude, and expressed on a scale from 0 (no damage) to 1 (total loss).

Elements at risk, meaning the population, buildings and civil engineering works, economic activities, public services, utilities and infrastructure, etc... at risk in a given area.

Specific risk, meaning the expected degree of a loss due to a particular natural phenomenon and as a function of both natural hazard and vulnerability.

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[1] Natural disasters and vulnerability analysis, report of expert group meeting, UNDRO, 1979.

Risk, meaning the expected number of lives lost, persons injured, damage to property and disruption of economic activity due to a particular phenomenon, and consequently the product of specific risk and elements at risk.

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IMPLEMENTATION FOR THE NATIONAL FLOOD INSURANCE PROGRAM AT THE REGIONAL AND LOCAL GOVERNMENT LEVEL

By

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INTRODUCTION

Much of the discussion during the past month has been on the scientific identification of the geologic or hydrologic hazards. I would now like to describe a program which is unique in getting hazard information implemented at the local level of government. The National Flood Insurance Program, in a very real sense, forces the transfer of technology from scientists and engineers into use at the local level of government and in the private business sector. It accomplishes this through sanctions or requirements placed upon local governments and the private banking industry.

I realize that in some of your countries the banking and market systems may be quite different from those in the United States. The Free Market System and the decentralization of decisionmaking down to the private businessman poses a particular challenge in trying to implement a national program of flood plain management in this country. I believe, however, that you will find the National Flood Insurance Program interesting from the viewpoint that it is an example of "reinforced decisionmaking." In other words, the decisions which are made in the local city hall are complemented or reinforced by the independent decisionmaking of the private businessman. I will show you this relationship later in the presentation.

There is also another important fact about the National Flood Insurance Program which you should know. The philosophy of the program is to "Keep people away from the water." In other words, to regulate the activities of man in relationship to the hazard area. We often refer to this as the nonstructural approach to flood plain management. The structural approach refers to the construction of dams and levees or "Keep the water away from the people." Obviously in practice, we have a combination of schemes or solutions which blend both the structural and nonstructural approaches.

The National Flood Insurance Program was created by the Congress in 1968 with two principal objectives. First, the states and local governments were encouraged to guide future development away from flood hazard areas. Secondly, flood insurance was to be made available, at affordable rates, to the general public. Of course, there was a long history preceeding this in which billions of dollars was spent constructing dams and levees while annual flood losses continued to rise. Prior to 1968, flood insurance was not available to the general public. In the few instances in which flood insurance was available from private industry, the premiums were very high reflecting both the risk and the fact that few policies were sold.

Thus a program was constructed, beginning in 1969, which contains three functional areas. The first was the identification of flood hazard areas through hydrological engineering. The second was the development of Federal minimum standards for the regulation of the flood hazard areas and thirdly, was the development of a flood insurance program which made both subsidized and actuarial flood insurance available. The challenge which faced the administrators was to develop an insurance program which provided both a subsidy for those currently occupying the floodplains and an incentive toward intelligent use of the flood hazard areas in the future. The three functional areas of engineering, regulation and insurance, are obviously, complementary to each other.

The daily operations of the National Flood Insurance Program, however, become quite complex since there are also three categories of actors in the Program. We will discuss these in greater depth, but in brief they are as follows: 1) the Federal government provides the engineering data and the economic incentives, 2) the local governments regulate the developments and land use and, 3) the private banking industry provides the financing to the land developers and individuals purchasing homes and businesses.

FEDERAL GOVERNMENT ROLE

The first principal responsibility of the Federal Emergency Management Agency (FEMA), Federal Insurance Administration, is to provide an identification of the flood hazard areas. The agency conducts hydrologic engineering studies which it calls flood insurance studies, for the purposes of regulation and insurance rating.

The studies contain two levels of engineering detail. Local governments are consulted regularly during the study process to determine the history of flooding and the areas to be evaluated. Approximate study areas only provide the estimated 100-year flood plain delineation on the Flood Insurance Rate Maps. Detailed study areas, however, are the result of extensive hydrologic investigations which include cross-section ground surveys, flood discharge estimates, computerized back-water analyses, and mapping. Four flood profiles are developed which identify the mean sea level elevations at given points for the floods with recurrence intervals of 10, 50, 100 and 500 years. Maps are developed on a scale of 1"=400' (one inch equals four hundred feet) for developed areas. The 100-year and 500-year floods are delineated on the Flood Insurance Rate Maps. Flood insurance rating zones are also included on the maps for the benefit of the insurance industry. The detailed studies are generally conducted in floodplain areas where development currently exists or is projected in the next five years.

The second principal responsibility of FEMA is to provide guidance to the local governments in the development of flood plain management regulations. The local regulations must meet the minimum criteria for land management and use established by the Federal Insurance Administration. There are a number of performance standards established within the Federal criteria, but the most important is the requirement to control new development so that the first floor of buildings is at or above the 100-year water surface elevation. Local governments are also expected to utilize the engineering data in the flood insurance studies and enforce their regulations. Consequently, the agency also has the responsibility to monitor the enforcement activities of local government.

The Federal government does not have the authority to control land use; that power is reserved for the States. The States in turn have delegated most land use authorities to the local levels of government; the cities, towns, and counties. FEMA, however, by Federal statute establishes the criteria by which a local government may participate in the National Flood Insurance Program. Currently, over 17,000 local governments participate in the National Flood Insurance Program.

The third major responsibility of FEMA, Federal Insurance Administration, is to make flood insurance available to the general public. Flood insurance is not generally available through the private sector in the United States. The Federal flood insurance may only be purchased on buildings and their contents located in communities participating in the National Flood Insurance Program. In other words, flood insurance is a benefit program from Federal government which is available only in communities that have established floodplain management regulations.

Federal flood insurance policies are issued directly by FEMA. However, the servicing of the insurance policies and claims is contracted out to a private corporation. Flood insurance policies are sold by over 30,000 fire and casualty insurance agents located in the communities. Currently over 1,901,000 flood insurance policies are in effect with coverage in excess of $113,287,000,000.

The insurance rating structure is divided into two categories: subsidized and actuarial or true risk. In the construction of the program, it was necessary to make flood insurance available at reduced or subsidized rates to those individuals occupying structures built prior to the completion of the flood insurance studies.

Congress reasoned that the floodplain occupants should become less dependent upon the Federal Government for disaster assistance and that they should share in the cost of occupying hazardous areas. A typical subsidized flood insurance policy will cost forty-five cents (45¢) for each one hundred dollars ($100.00) of coverage for a dwelling or $90.00 per year for a $20,000 policy. Obviously, subsidized flood insurance rates could only be provided for structures built prior to the specific identification of the hazard in the flood insurance study.

In order to avoid stimulating the development of the floodplain, it was necessary to charge actuarial insurance rates on all new structures built after the completion of the flood insurance study. The actuarial flood insurance rates are keyed to the 100-year water surface elevations determined in the rate study. The actuarial insurance rates also reinforce the floodplain decisionmaking of the local government since their permits would require the elevation of new structures to the 100-year level. A typical actuarial flood insurance policy for a new dwelling located with the first floor at the 100-year water elevation would cost $50.00 per year. The same structure built improperly, several feet below the 100-year water surface elevation, could result in a flood insurance policy costing several hundred or thousand dollars per year.

In 1973, Congress amended the National Flood Insurance Program in order to encourage the purchase of flood insurance and to stimulate greater participation in the program by local governments. Much of this was accomplished through what is known as the Mandatory Flood Insurance Requirements. Congress reasoned that the availability of Federal grants, loans, and guaranties for construction and acquisition made it possible to develop flood hazard areas. In addition, Congress found that the private savings and loan associations and commercial banks, which are regulated and insured by Federal instrumentalities, also made it possible to develop the floodplains. Thus, the law stipulated that flood insurance was a condition of financial assistance on structures located within identified flood hazard areas after certain dates. The effects of the mandatory purchase requirements were profound. The numbers of communities participating in the Flood Insurance Program and the numbers of flood insurance policies soared. Perhaps most importantly, the private sector became aware of and concerned about flood hazard identifications.

STATE AND LOCAL GOVERNMENT ROLE

The role of local government is critical to the success of the National Flood Insurance Program. In the United States, the local government typically has full authority to regulate the use of land and the construction of buildings. Whether they exercise those authorities is another question. The floodplain management criteria established under the National Flood Insurance Program are general enough, however, to permit participation by nearly all communities.

There are several basic floodplain management elements that should be considered in any program. First, the local government must require permits for new construction in order to determine whether the development is proposed for a flood-prone area. A review must be conducted of the proposed project relative to the known flood hazards. Very specific performance standards can be developed for construction in flood hazard areas. I believe, however, that the two most useful criteria are either to elevate the first floor of the structure above the regulatory flood (the 100-year flood) or prohibit encroachments into the floodplain. The National Flood Insurance Program emphasizes prohibiting encroachments into areas identified as floodways. These would typically be high-hazard areas characterized by greater water depths or velocities. In fact, we've found over the years that the easiest form of floodplain management is to simply stay out of the known flood hazard area. The specific Federal criteria for local flood plain management regulations are included in your packet of materials. Two speakers who follow me will provide you with specific techniques and approaches to flood plain management and hazard mitigation as seen from the local level.

PRIVATE SECTOR ROLE

Each day millions of decisions are made by individuals and private corporations to develop or acquire real estate. For the most part the decisionmaking process of the market sector is external to the affairs of local government. Only when a decision has been made to subdivide the land or to seek a building permit, does the private sector contact the local government. Typically, the most knowledgeable people about flood hazards in a community would be the local government planners and engineers.

The 1973 amendments to the National Flood Insurance Program changed much of that. The Federal Government identifies (map) all flood-prone communities in a very short period and secondly, the Act required that lending officers in

Federal agencies and in the private banking sector condition future loans in identified flood hazard areas upon the purchase of flood insurance. Thus, the lending sector was given the information on the location of flood hazard areas and they were required to make judgments upon it.

In my opinion, the involvement of the private sector in the decionmaking loop is equal in importance to getting the local governments to adopt floodplain management regulations. The bankers and developers are concerned about the economic feasibility and marketability of their projects. In regard to economic feasibility, the actuarial flood insurance rates on new buildings penalize construction below the 100-year regulatory water surface elevation. An improperly constructed building could result in annual flood insurance premiums in the hundreds if not thousands of dollars, thus disqualifying and discouraging potential buyers. The existence of a flood hazard identification or a new development could also diminish the marketability of the project. Consequently, bankers and developers are anxious to respond to the hazard identification on the Flood Insurance Rate Map. The response comes in a variety of ways. It could include utilizing the hazard area as open space, as an amenity to the community, or the developer might respond by filling the floodplain or constructing a channel or levee to remove the flood threat. When the hazard has been modified, the Federal Emergency Management Agency responds by reviewing the engineering and issuing amendments to the Flood Insurance Rate Map and removing the floodplain delineation and flood insurance purchase requirement. Of course, these actions result in a closer working relationship between the private sector and the local government.

SUMMARY

In summary, the National Flood Insurance Program represents a shared decisionmaking process among: the Federal Government, which identifies the flood hazards and provides the incentives for the use of the engineering data; the local governments, which regulate the floodplains; and the private sector, which makes the rational use of the flood hazard area possible. Each actor has distinct responsibilities, yet the program is designed to link the decisionmaking at critical points, reinforcing the national objective of guiding future development away from flood hazard areas.

EARTHQUAKE PREPAREDNESS PLANNING AT THE LOCAL GOVERNMENT LEVEL

By

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LOCAL GOVERNMENT EARTHQUAKE PREPAREDNESS PLANNING

Initiating a "local" earthquake preparedness planning effort in a large metropolitan area (Los Angeles, Lima, Peru, Mexico City, etc.) is a complex process that requires ample time to develop. Local government planning, nevertheless is a prerequisite in effectively dealing with seismic hazards and associated risks.

The most direct effects, both social and economic, of damaging earthquakes are local in nature. If the occurrence of a major seismic event is centered in a heavily populated area, the first to bear the burden of its effects will be the "local" governmental or political subdivision(s) and the residents and commercial establishments therein. There are cases where such a subdivision is of major national significance in the overall governmental structure and/or economic system; a nation's capitol for example. In such cases, the subdivision's public safety and welfare may be the responsibility of a higher level of government, but the actual effects of a damaging earthquake on that center of population will remain to be local in nature.

The geophysical effects of earthquakes will vary within a geographic area and the centers of population contained therein may be affected differently with varying degrees and types of damage and losses. Yet each center will have to respond to those immediate effects with the resources and capabilities <u>locally</u> available. Beyond the immediate post-earthquake environment, that situation will change as "outside" assistance and aid become available. But even so, the optimum deployment and allocation of outside resources will depend on how well the locale was able to respond to the immediate emergency.

These may all be obvious points, but the fact remains that efforts to strengthen local government/political subdivisions' capabilities to respond to any or all major disasters are not so obvious.

Planning Principles

How does one go about strengthening local response capability during major disaster? It is through a pro-active planning process that is based on a set of sound planning principles. The end-results of a pro-active planning process are dynamic in substance and adaptable to changing environments. The best way to define pro-active planning is by enumerating the principles upon which the process is based. These principles include the following:

1) The planning processs must begin at the local level. It must address the needs of both public and private entities. This particular principle provides the essential framework for higher levels of planning. That is, local planning serves as a base to subsequent planning that must be done by higher levels of government to guide their assistance to local governments; be it for mitigation, preparedness, emergency response or relief and recovery. Basically, it is a bottom-up approach to planning. The principle relies less on compliance-type planning without hindering efforts to promote standard approaches.

2) The planning process must help achieve a stronger social, political, and economic organizational capability to manage the consequences of a damaging earthquake at the local level.

3) The planning process must be comprehensive. That is, the full range of potential consequences of a damaging earthquake or its prediction--legal, public safety, warning and information, application of hazard reduction techniques, readiness of emergency services, outreach and self-help programs--must be addressed.

4) The planning process must be coordinated with all who need to be involved. Both in and out of the area, all sectors of society likely to be affected by or to have responsibility for responding to a damaging earthquake should be included.

5) An independent process for evaluation must be included to analyze lessons learned in the planning and to facilitate their incorporation in subsequent planning efforts and plan maintenance. Part of this evaluation process should be a continuous program of emergency response tests and exercises involving all elements of society.

Metropolitan Regional Planning

The application of the planning principles enumerated above is most appropriate at the metropolitan regional level. A metropolitan regions may be defined as a large continuous urban area or center. (Some metropolitan regions will have rural or undeveloped areas within or adjacent to it. This is important to recognize if efforts are undertaken to manage growth away from high seismic risk areas that are yet undeveloped.)

Most metropolitan regions throughout the world consist of complex social (including governmental and political), economic, and physical infrastructures. For example, the Los Angeles Metropolitan Region consists of virtually hundreds of independent cities (self-governing), several counties, hundreds of special districts (for education, water/sewage, and other services), major corporations with world headquarters in the region, large utility companies that service millions of people and manage complex lifeline systems, and a diverse social and economic society. And within the region there is the strong presence of higher levels of government.

Earthquake preparedness planning at the metropolitan regional level must attempt to deal with these complexities. It is a difficult undertaking, and in most cases the resources available to actually do the planning will be minimal i.e., few staff and financial resources will be made available to carry-out an enormous planning effort. The principles presented above, when applied properly, can assist in the design of a planning effort that can make the best use of the resources allocated to it.

Because of the complexities of a metropolitan region, one cannot expect the initial results from the planning effort to be a unified regional earthquake preparedness plan. But there are some reasonable objectives that a regional planning effort can accomplish and eventually lead to an overall improvement in seismic safety in a metropolitan region.

One crucial objective is to promote standard approaches to earthquake preparedness planning. These standard approaches to planning are best promoted if they are based on sound technical data and directly involve users in their development. Too many times, planning techniques are developed but never applied because they are found to be impractical or foreign to the user. A program of "planning partnerships" is one way that a small planning staff can work within the complexities of a metropolitan region and achieve potentially positive results in efforts to standardize planning.

For the last three years, the Southern California Earthquake Preparedness (SCEP) Project has been working within the framework of this planning partnership concept. The SCEP Project defines this key working relationship as follows:

> "Consistent with established selection criteria and available resources, a limited number of representative local planning partners from the Los Angeles Metropolitan Region, (a county, cities of varying size, a corporation, school districts, utility companies, etc.) plus appropriate state and federal agencies, are selected to work with the Project staff in developing transferable comprehensive earthquake preparedness planning guidelines. Each partner must have both the commitment and resources for successful participation. Where appropriate, each partner will form working committees to insure discussion of common problems and solutions, share technical information (provided by the Project), and develop independent but interrelated plans."

The criteria by which to evaluate the results of this concept will of course be the level of "receptivity" and application of the results by non-planning partners.

A second objective is to promote the development and use of a common regional data base for quantifying the effects of potential damaging earthquakes. One major problem encountered in local earthquake preparedness planning is the lack of useful data for planning purposes. There are two aspects to this problem that should be noted. One of these is the lack of standard or total absence of methods for forecasting earthquake damage. Past and current advances in the structural engineering community are making this aspect less problematic. Another is the dependence on local government structural inventories. This situation poses both a problem and an opportunity. If local governments can directly participate in the development of a regional data base by contributing essential information, that is a major first step toward regional cooperation. But, even if local governments agreed to participate in such a program, their structural inventories and other applicable records are often in various forms and levels of maintenance; there are no current standards by which the information is collected, filed, and maintained. In order for the information to be used effectively, a tedious effort for geographically-coding the information is involved that can be costly. Techniques for surveying structures and utilizing aerial photography do help in simplifying the process but are not as reliable as well maintained local government structural files and records.

These are not insurmountable problems that innovation cannot take care of. One of the best examples in the application of innovative techniques, to the development of regional data base for earthquake preparedness planning purposes, is the work of Kuroiwa and others in the Lima Metropolitan Region, Peru. This is a data base that was created with little financing but has been widely used in the development of mitigation and preparedness planning programs in that metropolitan region.

The development of a regional data base is an objective of paramount importance in earthquake preparedness planning. After all, seismic safety planning is no different than other metropolitan regional problems such as transportation, air and water quality and environmental issues that are virtually impossible to address adequately without a good information base with which to define and analyze their effects on our urban infrastructures.

The need for metropolitan regional planning is best defined by the consequences of a catastrophic earthquake occurrence in a large urban center. A 1981 assessment of the consequences for a catastrophic earthquake occurrence in the Los Angeles Metropolitan region, conducted by the United States Federal Emergency Management from analyses carried out by the National Security Council, comes to some important conclusions that bear out this need.

> "...The Nation is essentially unprepared for the catastrophic earthquake (with a probability greater than 50 percent) that must be expected in California in the next three decades. While current response plans and preparedness measures may be adequate for moderate earthquakes, Federal, State and local officials damage and casualties from a catastrophic earthquake, and with disruptions in communications, social fabric, and governmental structure that may follow. Because of the large concentration of population and industry (in a metropolitan region), the impacts of such an earthquake would surpass those of any natural disaster thus far experienced by the Nation."

The most effective means to expand and accelerate the quality of preparedness measures to counter the effects of this catastrophic earthquake cannot be developed at the National level, nor strictly at the local level, nor at some intermediate level of government. These have to be developed at the metropolitan regional level, where all levels of government already interact effectively on a day-to-day basis and private industry is defined by metropolitan region economics.

The Role of Earthquake Prediction and Warning Systems

The objective to implement operational systems for predicting damaging earthquakes, in areas of high seismic risk, carries with it some important implications for local government earthquake preparedness planning. This objective is based on the premise that "earthquake prediction is a significant and primary means for enhancing the effectiveness of preparedness activities and for mitigating the effects of great earthquakes." Although the premise is a sound proposition, it is dependent on one important factor: preparedness and mitigation planning to effectively respond to and utilize earthquake prediction information should proceed in parallel with the development of operational prediction networks and systems. Given the primary role of local government/political subdivisions in preparedness and mitigation efforts, earthquake prediction capability poses some unique planning problems for a metropolitan region.

The same aspects of earthquake prediction that pose planning problems for local government/political subdivisions are actually the same ones that enhance the effectiveness of preparedness and mitigation activities. These aspects are related to time frames and probabilities. (A valid scientifically-based prediction should provide information on both of these aspects.)

Crucial to any type of planning effort is the time frame within which planned activities are to be implemented. Defining appropriate time frames for earthquake preparedness planning purposes becomes problematic if the prediction research community cannot provide this information in terms that are applicable to planning. At the same time, preparedness planners need to recognize the uncertainties related to formulating earthquake predictions and current lack of capability to specify tight time frames. The solution to this problem is the development of a common set of terms for "expressing" earthquake potential and/or predictions that can be utilized effectively by both the prediction research community and preparedness planners. In facing this problem in southern California, a set of terms were developed (Wallace, McNally, David, et. al) that were acceptable to local planners and some members of the prediction research community.

This set of terms (see Table 1) provided local planners with sufficient time frame definitions to work with in planning appropriate response activities. The time frames proposed by this terminology enhances the effectiveness of preparedness and mitigation activities in two important ways:

1) prediction, and associated time frames, communicate a sense of increased likelihood (probability) of a damaging earthquake occurring within a metropolitan region. A recent technical report for the

TABLE I
SUGGESTED TERMS

	Time Window
A. LONG-TERM EARTHQUAKE POTENTIAL	No specific time window. Can refer to decades, centuries or millennia.
B. EARTHQUAKE PREDICTION	Any specific time window shorter than a few decades.
1. Long-term prediction	Few years to a few decades.
a. Forecast	
b. Watch	
2. Intermediate-term prediction (no subdivisions suggested at present)	Few weeks to few years
3. Short-term prediction	Up to a few weeks.
a. Alert	Three days to a few weeks.
b. Imminent alert	Up to 3 days.

(Terms for Expressing Earthquake Potential, Prediction and Probability by Davis, McNally and Wallace, 6/3/83)

United States Geological Survey concludes that, "increased likelihood motivates a decisionmaker to take mitigating actions, such as structural retrofitting, relocation, etc..."

2) The time frames accompanying a prediction also enhances preparedness and mitigation in that any activity planned must have some correlation to the time it will take to implement it. Thus, the varying time frames proposed in the terminology will result in appropriate actions to be taken within each time frame. Depending on the time frames, response actions may vary from long-term retrofit programs, requiring years to decades to carry-out, to the emergency evacuation of persons from hazardous buildings which must be done in a matter of hours or days before the earthquake occurrence.

The planning process proposed here consists of five phases. It is designed to meet the requirements of an important planning principle presented above i.e., the planning process must be comprehensive. A brief and simple description of each phase is presented to provide a general idea of the technical and/or political functions that are involved in each.

Phase I: Hazard Identification and Risk Assessment

The planning processes begin with an identification of the earthquake hazard facing the community. This identification may include:

- Fault-mapping
- Recurrence probability
- Potential earthquake-induced effects (ground shaking, landslides, liquefaction, etc.)

An assessment of earthquake risk, based on the identified hazard, is also essential. This assessment may include projections of potential:

- Structural damage
- Dam inundation
- Fires

- Hazardous material spills
- Homeless caseloads
- etc.

The results will be a highly useful data base for setting policy and scenario development for planning purposes. (This data base, coupled with an operational earthquake prediction network and data processing, can be an effective earthquake warning system.)

Phase II: Development of Seismic Goals and Objectives

Based on the data base developed in Phase I, community policy, in response to potential scenarios, should be stated as a set of goals and objectives to guide the rest of the planning process.

- Goal
  Goals should reflect a desired state of preparedness
- Objectives
  Objectives should provide concrete steps to reaching these goals

EXAMPLE

Goal: Maintain a comprehensive public information program that will promote preparedness measures in the home and safeguard the family.

Objectives:

- To create a public information network for earthquake preparedness
- To develop regional programs to motivate preparedness activities
- To develop a system to measure the level of public preparedness.

Phase III: Design of Potential Mitigation and Preparedness Strategies

The application of the seismic safety goals and objectives of the community should consider social, economic and political constraints.

- Prime consideration
- Measures that could save the greatest number of lives and reduce risks in a feasible, cost-effective manner.
- Strategies

  Strategies for land use, emergency operation procedures, warning systems, structural design criteria, and similar measures are developed in this phase of the planning process.

Phase IV: Program Development

The effective application of mitigation and preparedness strategies depends upon the development of interrelated seismic safety programs - programs that involve a cross-section of society to increase preparedness in the community. These programs can include:

- Determining the spatial distribution of risk
- Guiding future land use
- Upgrading the design construction process
- Planning prediction response and warning systems
- Preparing emergency operations and recovery procedures
- Conducting tests and exercises
- Planning for reconstruction

Because these programs are interrelated, they should be pursued logically and sequentially (programs and policy) and must be implemented in a logical, sequential manner. This is because each program forms an essential stepping-stone for those that follow. To do this, the community should consider a multi-year plan approach.

Phase V: Design of Plan Format and Evaluation Methods

Most important in the design of a plan format is that it be practical, that is, the format should reflect the plan's purpose and uses.

- Content

  The plan format should outline specific actions and make assignments for their implementation. These actions can be grouped into functions that correspond to the general program categories developed in Phase IV.

- Implementation

  The plan format should be designed to allow for an organized implementation process. Actions identified in the plan will be numerous and varied but should provide a base from which to set priorities.

- Evaluation

  Because the plan is dynamic, ongoing evaluation should continue throughout the implementation process.

The earthquake preparedness planning process is circular, figuratively speaking. Basically, this means that the planning is "never-ending," but should significantly improve overall preparedness with each cycle. Using the circular anology once again, this means that the planning process is an upward-bound spiral.

FIGURE

This concept is important in that our knowledge of seismic phenomena and associated risks is constantly growing. Proper use of this information will improve and refine planning efforts by providing more specific information on earthquake potential and effects. As this information becomes available, it can improve the quality of the results of the planning process. Using the model presented above, for example, one can see how new techniques in hazard identification and analyses (Phase I), that improve our capability to predict damaging earthquakes and/or their effects on our urban centers, will also improve the quality of the results obtained in the subsequent phases (Phase II-V) of the planning process.

(The upward-bound spiral (planning process) does need a constant driving force to make it "never-ending." The regional data base discussed above, no matter how rudimentary can provide, in part, such a force.)

The most cost-effective means of implementing this planning process is at the metropolitan regional level. The obvious involvement of many local governments in the region as well as the higher levels of government and the private sector makes a strong case for the process to be implemented at the metropolitan regional level.

At the local government level, technical assistance and planning guidance for the integration of the planning process in day-to-day functions and responsibilities is important. The development of a local government "user's guide" to the planning process, that provides basic recommendations on how to proceed and organize personnel to be involved in the planning and is based on actual experience (the planning partnership concept discussed above) can prove to be an effective tool for initiating such an effort at the local government level. At the metropolitan regional level, the integration of the various local governments plans can better succeed if a standard approach was used by all of them.

Planning Elements and Functions

Phase IV of the planning process discussed in the prior section of this presentation, is the design of a plan format that is practical and can reflect the plan's purpose and uses. The "plan" referred to is a comprehensive earthquake preparedness plan and deals with a variety of mitigation, preparedness and relief and recovery activities. The format of such a plan must be designed in a way that can take full advantages of existing plans and programs. It must also remain flexible enough so that the integration of future activities will not require a totally new effort of design or documentation. Given the current status of earthquake preparedness planning in the Los Angeles Metropolitan Region, the Southern California Earthquake Preparedness Project recommends to local government the use of a plan format based on several planning elements and corresponding functions. The format recommended is referred to as a "planning guide" and describes functions for responding to a predicted or unpredicted damaging earthquake. It specifies actions that can be carried out under each function by local governmental agencies and/or nongovernment and private entities that have unique and specialized capabilities applicable to earthquake response activities. The functions and corresponding actions are organized into four plan elements; each relating to a specific time period. The four plan elements are:

- Long-Term Prediction Response
  (A few years to a few decades before the earthquake occurrence)
- Short-Term Prediction Response
  (A few days to a few weeks before the earthquake occurrence)
- Emergency Response
  (The first 72 hours to a few weeks afer the earthquake occurrence)
- Short-Term Recovery
  (The first 1-2 months after the earthquake occurrence)

The first two plan elements are based on SCEP Project's report referred to above, Terms for Expressing Earthquake Potential, Prediction, and Probability by Wallace, McNally, and Davis. The planning guide does not specify any functions or actions for responding to an "intermediate-term prediction" (A few weeks to a few years before the earthquake occurrence), though the SCEP Project is currently working in "partnership" with the City of Los Angeles in the development of those functions and actions.

The third plan element suggested, Emergency Response, is one that is very familiar to most local government entities. In general, most governmental entities currently have an emergency operations plan in place to deal with a variety of emergencies. In some cases, this plan may have an annex that deals specifically with earthquakes. The functions and actions contained in this element supplements any planning the governmental entity may have already developed for responding to a catastrophic earthquake event. The time-frame within which this plan element will operate is the first 72 hours to a few weeks after the earthquake occurrence.

The fourth plan element proposed relates to short-term recovery; the time-frame for its operation will be the first 1-2 months after the earthquake occurrence. It is important that the functions and actions identified in this element be implemented as soon after the earthquake event as possible. The implementation of these functions and actions will set the framework for the governmental entity's long-term (from a few years to possibly a decade) recovery efforts.

Long-term recovery from a catastrophic earthquake, impacting numerous local jurisdictions, will require a regional planning approach. It is important to note that the short-term recovery actions will have an important impact on long-term recovery options.

Table II summarizes the functions prescribed for each of the four planning elements outlined here and are described in much greater detail in the SCEPP Project's Comprehensive Earthquake Preparedness Planning Guidelines, (for Counties, Cities and Corporations), 1983.

AWARENESS-PREPAREDNESS PLANNING RELATIONSHIIP

There is a direct relationship between the level of community earthquake awareness and the effectiveness of earthquake preparedness planning at the local government level. But what is earthquake awareness? It is by definition, being mindful or heedful of the causes and effects of earthquakes. It is knowing about earthquakes by means of information, education and/or experience. Awareness can also imply being alert, that is, having both knowledge about earthquake effects and the capability for a swift and apt response to those effects. From this definition, one can immediately acknowledge the important relationship between awareness and preparedness planning.

The Development of Earthquake Awareness: A Major First Step in Preparedness Planning

In recognizing the relationship between earthquake awareness and preparedness planning, it is also important to recognize that awareness is developed. And if it is to be used to positively effect preparedness planning and more importantly, implementation activities, we must first understand its dynamics and then strive towards guiding its further development.

First we must understand that there are varying degrees of earthquake awareness, each developed through some source of information or experience. Each degree of awareness can have a purpose and impact on the overall preparedness planning process. Some examples follow:

1) The occurrence of historical earthquakes in California has led to a general level of awareness in the United States that this state is "earthquake country."

2) The identification of California's fault system (principally the San Andreas Fault) has led to the awareness that most of the state's urban centers are at a relative high level of exposure to earthquake effects in that area.

3) Past and current engineering research on structural performance has led to the awareness that there are safe versus hazardous building types.

4) Research on the performance of nonstructural elements has led to the awareness that home and worksite preparedness is important but too often neglected.

5) Research on the performance of lifeline systems has led to the awareness that after a great earthquake there will be electrical power outages, water shortages/contamination and subsequently to the importance of self-help concepts.

We must also know who has the key roles in the development of earthquake awareness. There are four main groups of players that do or can have major roles.

1) The scientific community and its efforts to understand the cause and effect of the seismic phenomena is all important to the development of earthquake awareness. In general this group is comprised of geologists, geophysicists, seismologists, and engineers. This group provides the basic information that often can have the most dramatic

effect on earthquake awareness. But what often happens with the research findings of the scientific community is that its findings do not get translated into terms that are understandable to those that must be aware of such studies; i.e., government and private industry officials and the general public. Greater dissemination of scientific knowledge, in comprehensible terms, is crucial to the development of earthquake awareness.

2) The media, both print and electronic, has a powerful role in the development of earthquake awareness. Part of this role fits in with their normal day-to-day function in society which is to report the news. Damaging earthquakes are news and the media does an excellent job in reporting such events. This in itself will have an impact on the public's level of awareness. But the media can have a wider role in developing earthquake awareness by reporting this type of news with less emphasis on sensationalism and more emphasis on education.

   The media and its associated professionals are beginning to recognize their potential role, not only in awareness development but also in the post-disaster period.

3) Business and industry, in many countries the most sophisticated sector of society, can directly influence the development of awareness except for the fact that its leaders are often not as aware themselves as they should be.

   As part of their own awareness development process, this sector's leaders should realize their responsibility to inform and educate their employees, be informed themselves on potential economic losses due to the interruption of business operations, and have the foresight to realize the cost-benefit of earthquake preparedness planning, as they are advised by the scientists.

4) Government, particularly local government, has the most important role in earthquake awareness. Government can provide the framework for an alliance between it and the three other groups discussed above through well-designed programs.

The Role of Education/Information Programs

The Earthquake Hazards Reduction Act of 1977 (PL 95-124) recognized the general threat of earthquakes and the need to develop measures to reduce the enormous 'loss of life, injury, destruction of property, and economic and social disruption' caused by earthquakes. One of the areas of concern in that Act was 'public education and involvement program'."

Following passage of PL 95-124, the first step in preparing an implementation plan was a report issued in 1978 by the Working Group on Earthquake Hazards Reduction of the Office of Science and Technology Policy of the Executive Office of the President. That report defined the general threat of earthquakes as "the largest single-event natural hazard faced by the nation," and stressed the need for education for preparedness, especially the immediate targeting of key government decisionmakers, representatives of mass communications media, spokesman and opinion leaders, "people who will be interpreting the program and communicating their interpretations to the general public." Their recommendations on education and communication include:

1) Hazard information to key audiences, especially the community leaders noted above, disseminated in a careful sequence with two-way communication.

2) Face-to-face discussion of preparedness issues to supplement print material, and effective communication between procedures and the users of information. Careful "translation" of scientific and technical information to promote better understanding.

3) Inclusion of earthquake data in programs of information about all other hazards and in science education programs.

4) A program to help people understand earthquakes, thereby promoting preparedness.

5) Development of education programs and inclusion of those programs in already existing institutional curricula (public and private schools, professional schools such as engineering, architectural, and law schools).

6) A national center for information dissemination with state centers in earthquake-prone areas.

7) Improvement in Federal communication and education program to "set an example and set of standards" for other programs.

Development of Tools and Resources for Earthquake Awareness

Implementation of the recommendations discussed in the prior section of this presentation has been an on-going objective of the SCEPP Project. A key strategy in achieving this objective has been the integration of earthquake awareness efforts with preparedness planning.

(Audio-visual presentation as a demonstration).

SHELTER AFTER DISASTER: THE UNDRO VIEW

by
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It is evident that in the past decade the understanding of disasters and their consequences has improved. In the face of the mounting social and economic costs of natural disasters in the third world, the international community (donors and recipients of aid alike) have made considerable efforts to improve the quality of disaster relief, preparedness and prevention: to improve our understanding of natural hazards; to estimate the risks resulting therefrom more accurately; and to take adequate precautionary or preventive measures ahead of disasters. Progress has, nevertheless, been slow: population growth, rapid and uncontrolled urbanization, degradation of the environment, economic recession, and poorly coordinated development planning have, together, conspired to outstrip progress in the control of disasters. It is certain that disasters are not merely "acts of God" but are aggravated by human error and lack of foresight; that disaster relief can be made ever more effective through systematized planning and management; and that predisaster planning does help, at least, to reduce some of the harshest effects of disasters. Therefore, whatever the difficulties, efforts to improve disaster relief and predisaster planning must continue unabated.

It can be said with some assurance that relief management in the fields of medicine, health, and nutrition has, nevertheless, significantly improved over the last decade. The benefits of the lessons learned from major disasters during the 1970's and early 1980's are beginning to show. However, there remains one particular sector in which too little progress has been made, and in which many conservative and obsolescent attitudes survive, that is: emergency shelter, and shelter after disaster in a more general sense. Perhaps the core of the problem lies in the fact that, although housing is one of the most complex and intractable problems of development, it is also one upon which everyone has his or her personal opinion, thus, creating much confusion between objective and subjective evaluations. The least understood of all issues is that a house is merely the end-product of a long chain of social, economic, technological, environmental, political and other interactions. In some countries the housing issue is not "the house," but land and utilities (water, electricity, roads, transport, etc.). In others, the poorest, housing has a lower priority than employment and nutrition. In no more than a handful of countries can the house as a product be said to be of primary concern. Until it is fully and widely understood that shelter is a "process" rather than a "product," many housing programs, however well-meaning, will fall short of expectations--especially in the developing countries. The foregoing reasoning is as true for the shelter aspects of disasters as for the "normal" housing process.

The emergency shelter problem in developing countries is fundamentally different from that in industrialized societies, for in the third world the question of emergency shelter cannot be dissociated from the prevailing housing problems as a whole. This finding alone influences every other consideration one may hold about the subject.

The process of rapid and uncontrolled urbanization in developing countries has resulted in the proliferation of vast slums and squatter settlements. These account, on the average, for more than 70 per cent of urban development. In such areas, and therefore, for the majority of urban populations, the concept of temporary shelter in times of emergency is somewhat equivocal when, under "normal" conditions, urban dwellers are permanently lodged in housing which the authorities do not recognize, or which they consider as temporary to start with. Furthermore, in conditions of chronic housing shortages, overcrowding, unsanitary conditions and high rents, the investment of scarce capital resources in prefabricated temporary or emergency shelters, specifically designed to be stockpiled and used only in case of natural disasters, can only create additional obstacles to the provisions even of minimal housing.

In rural areas, tradition dies hard, and cultural resistance to donor emergency shelters often provokes frustration and misunderstanding among all concerned. So-called "temporary" or "emergency" shelters are often inappropriate, but at the same time become permanent, only to create fresh sets of problems.

Emergency shelters, especially those donated by the international community and imported into disaster-striken areas, can serve to upset a delicate socioeconomic balance by raising expectations, which, in most cases, neither the local, nor the national, nor indeed the international authorities have the means to satisfy. The importation of shelters can, furthermore, play a negative role by stifling local and even national initiative, especially when they comprise prefabricated systems invariably posing problems of appropriateness, assembly, and cost-effectiveness.

In the several major natural disasters throughout the developing world over the last decade, it has been shown that imported donor shelters have never produced the impact that most relief agencies would have desired. Shelters often arrive in insufficient numbers, or too late to be of value during the emergency phase. Their unit cost is nearly always disproportionate vis-a-vis the recipient economy, and if one adds the cost of transport they are seen to be quite uneconomical. For this reason alone, the emergency shelter policies of the donor community at large need to be re-examined.

An important fact to consider is that the problem of emergency shelters is less one of product, design, or manufacture, than one of planning, managment, and the mobilization of local resources. The problems posed are not, as a priority, technological (as is so widely believed), but are functions of development policies themselves, and of the changing relationships between donors and the developing countries. Relief agencies and international organizations should encourage disaster-prone developing countries to build up their own state of preparedness, notably in the emergency shelter field, by mobilizing local material and technical resources, and to encourage self-help schemes for this purpose. It is essential to link donor assistance to local initiative and effort.

The spontaneous reconstruction of housing begins extremely rapidly after a disaster, and often during the emergency phase inself. All actions to discourage this process should be avoided, except in cases of extreme danger. Assisting groups who support rapid reconstruction policies are likely to obtain the most positive and far-reaching results. However, the assisting groups themself require education and training on how to assist and manage post-disaster housing programs within a risk reduction framework: they require education on what is the housing process as a whole in developing countries, on appropriate building technology, on financing and management, and on the socio-eoconomic aspects of low-income housing.

The key to success ultimately lies in the participation of the local community--the survivors--in reconstruction. Assisting groups, and those they help, must be accountable to each other in order to ensure social satisfaction, economically viable housing, technically sound buildings, and a safer environment. Accountability is, therefore, a key criterion of assistance to survivors, especially those in the developing countries.

Linked to the question of accountability is that of rising expectations among all people in the developing countries. Rising expectations are frequently the source of conflict and confusion in post-disaster housing policies and programs, and a lack of awareness of the phenomenon can compromise, not only post-disaster housing but the entire housing policy of a country. In the final analysis social, economic, and cultural obstacles are far more difficult to overcome than purely technical, material problems.

Lastly, guidelines on emergency shelter and post-disaster housing for individual communites must be drawn up at the local level itself. The design of local guidelines cannot, therefore, be incorporated in a global study of this nature.

In conclusion, a number of guiding principles emerge. These are:

Resource of survivors--The primary resources in the provision of post-disaster shelter is the grass-roots motivation of survivors, their friends and families. Assisting groups can help, but they must avoid duplicating anything best undertaken by survivors themselves.

Allocation of roles for assisting groups--The success of a relief and rehabilitation operation depends on the correct and logical distribution of roles. Ideally, this allocation should be undertaken by the local authorities who are best qualified to decide who should do what, when and where. However, if the local administration is too weak to assume this responsiblity, the priority must be to strengthen it.

The assessment of needs--The accurate assessment of survivors's needs is, in the short term, more important than a detailed assessment of damage to houses and property. Partial or inaccurate assessments or human needs by assisting groups have been a frequent cause of past failure of relief efforts.

Evacuation of survivors--The compulsory evacuation of disaster survivors can retard the recovery process and cause resentment. The voluntary movement of survivors, when their choice of venue and return is timed by their own needs, can be a positive asset. (In the normal course of events some surviving families may seek shelter for the emergency period with friends and relatives living outside the affected area).

The role of emergency shelter--Assisting groups tend to attribute too high a priority to the need for important shelter as a result of mistaken assumptions regarding the nature, and in some cases, relevance of emergency shelter.

Shelter strategies--Between emergency shelter provisions and permanent reconstruction lies a range of intermediate options. However, the earlier the reconstruction process begins, the lower the ultimate social, economic, and capital costs of the disaster.

Contingency planning (preparedness)--Post-disaster needs, including shelter requirements, can be anticipated with some accuracy. Effective contingency planning can help to reduce distress and homelessness.

Reconstruction: the opportunity for risk reduction and reform--A disaster offers opportunities to reduce the risk of future disasters by introducing improved land-use planning, building methods, and building regulations. These preventive measures should be based on hazard, vulnerability and risk analyses, and should be extensively applied to all hazardous areas across the national territory.

Relocation of settlements--Despite frequent intentions to move entire villages, towns, and cities at risk to safe locations, such plans are rarely feasible. However, at the local level a disaster will reveal the most hazardous site (i.e., earthquakes faults, areas subject to repeated flooding, etc.). Partial relocation within the town or city may, therefore, be both possible and essential.

Land-use and land-tenure--Success in reconstruction is closely linked to the question of land-tenure, government land policy, and all aspects of land-use and infrastructure planning.

Financing shelter--One of the most important components of a post-disaster shelter program is its financing system. Outright cash grants are effective in the short term only, and can create a dependancy relationship between survivor and assisting groups. It is far more advantages for both the individual and the community to participate in the financing of their own shelter programs, especially permanent reconstruction.

Rising expectations--Apart from the tendency of prefabricated temporary housing to become permanent because of its high initial cost, and in spite of its frequency rejection on socio-cultural grounds, temporary shelter, nevertheless, frequently accelerates the desire for permanent modern housing, well beyond reasonable expectation. It is important for assisting groups not to exacerbate social and economic tensions by such provisions where there are widespread and chronic housing shortages among low-income and marginal populations.

Accountability of donors to recipients of aid--Since the most effective relief and reconstruction policies result from the participation of survivors in determining and planning their own needs, the successful performance of assisting groups is dependent on their accountability to the recipients of their aid.

Guidelines for the local level--Guidelines on emergency shelter and post-disaster housing for individual communities can only be formulated by qualified, local personnel, in the light of the prevailing local conditions (type of hazard, building traditions, economic base, social system, etc.). Such guidelines can, however, be modeled on the structure of this study.

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TRANSFERRING HAZARD INFORMATION FROM THE SCIENTIST TO THE POTENTIAL USER

by

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INTRODUCTION

The avoidance or mitigation of geologic or hydrologic hazards depends upon the accuracy with which the hazard has been identified and located; the precision with which its magnitude and risk of occurrence can be calculated, and; the speed and skill that are employed in effectively conveying that information to the threatened population.

The subject of this paper is the latter phase of the process, that of conveying or transferring hazard information from the scientist to the potential user. This phase may be delineated into the following steps:

1) Translation of the scientific data into language and graphics that are comprehensible to the lay reader.

2) Manipulation and aggregation of this and other information into reports and graphics that clearly explain the potential hazard and weigh its risks.

3) Utilization of all available organizations and information vehicles to publicize, explain and to convey the hazard information products to the potential user.

TRANSLATION OF THE SCIENTIFIC DATA

Any reader of "Science" knows that the subject of a paper on molecular biology is incomprehensible to most earth scientists. No wonder that the layman cannot be expected to understand the precise terminology of the earth scientist concerning a natural hazard. Nor should the layman be expected to translate that information for his use. The responsibility of this translation is clearly the domain of the earth scientist, who is best qualified to distinguish between subtleties in meaning and precision of terms. He is the ultimate arbiter of what, where and when concerning the hazard. Even the interjection of a science writer is to be discouraged because of the potential for the introduction of imprecision to the translation. It is frequently useful, however, to have the draft translation reviewed by another earth scientist experienced in this procedure.

The Glossary of Geology (1980) as well as other specialized glossaries and dictionaries are useful reference points for the translation of scientific terms and words. The translation of concepts and the symbols of earth science present a far more complicated aspect of this process.

Major faults, volcanoes, landslides, flood scars and other fossils of natural disasters, or the loci of potential hazards, are mapped together with a multitude of map units and symbols. One or more hazards may occur in the same map area, or only one hazard may bear any immediate or potential risk. The scientist may extract only the hazard areas of immediate risk, or extract all hazards and prioritize them. He may then aggregate, prioritize and present these on a regional basis.

The translated hazard map should meet the same rigorous standards of logic and internal consistency that are applied to geologic maps (Varnes, 1974). In addition to simplifying the map by reducing the number of hazard categories and eliminating esoteric terms and symbols, the earth scientist must consider the graphic response or receptivity of his products.

Regardless of map scale, microscopic units can not be effectively dealt with. If they are critical, then the scale of the map should be enlarged. Monochrome graphics require a skillful application of screens and patterns to clearly distinguish among many units, and 9 to 12 units are generally considered a maximum effective number. The introduction of color helps to distinguish between simultaneous events or time periods, to organize information into logical groups, or to highlight important data (Murch, 1983). We associate yellow with warning, and red for a critical situation. Skillful combination of patterns and colors can increase the number of distinguishable map units, but the speed of recognition and the impact of the map may suffer.

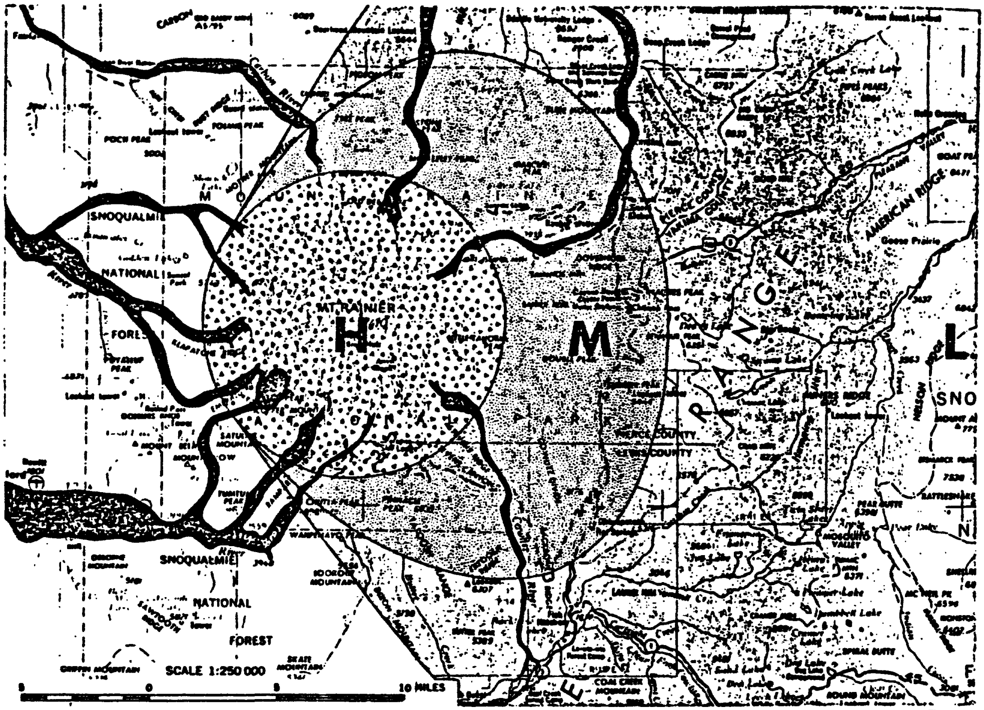
The hazard map must attempt to localize the area of maximum risk, or delineate the structures at maximum risk. Generally, a second category of moderate risk is delineated on the basis of distance, structure and compositional differences from the high risk zones, or a consideration of its activity through time. A relatively safe or inactive area may occupy a location furthest from the perceived hazard.

The simple and effective graphics are then combined with the translated explanation to constitute a map or report that is readily understood and therefore applicable to the needs of the lay user. This product can be used directly as hard copy or placed in a computer with the graphics digitized for further manipulation and combination with other information.

Some earth scientists are able to translate and to synthesize their data directly into a coherent hazard map (Figure 1, Crandell, 1973) and report. Frequently, however, further manipulation and aggregation is required of the basic hazard data with other information.

Witkind's two maps of the Henry's Lake Quadrangle (1972) depict the faults and ground-breakage hazards (Figure 2) and the areas vulnerable to seiche, rockslide, rockfall and earthflow hazards (Figure 3). The text is simple yet explicit and the map units sharply delineated. Earthquake risk zones are considered at much larger scale in Figure 4. The authors first grouped the geologic materials into five zones on the basis of similar physical properties and then they postulated the geologic effects for each zone based on an earthquake of approximately magnitude 8 with an epicenter in the northern San Francisco Bay area.

Land subsidence in the Santa Clara Valley (Figure 5) is a very costly natural hazard that could have been avoided with proper information, but can



VOLCANIC HAZARDS

MUDFLOW AND FLOOD HAZARD ZONES

▨

High risk

Valley-floor areas in which there could be a high degree of danger from relatively small but frequent mudflows and floods

▨

Moderate risk

Valley-floor areas which could be covered by a mudflow as large as the Electron Mudflow, in valleys in which mudflows have been relatively frequent

▨

Low risk

Valley-floor areas which could be covered by a mudflow as large as the Electron Mudflow, in valleys in which mudflows have been relatively infrequent, and areas in the other valleys which might be subject to flooding

TEPHRA (airborne rock debris) HAZARD ZONES

H

High risk

Areas in which there could be a high degree of danger to human life and property from asphyxiation, falling rock fragments, and accumulation of tephra

M

Moderate risk

Areas in which there could be a low degree of danger to human life and a moderate degree of danger to property from falling rock fragments and accumulation of tephra

L

Low risk

Areas in which there could be a low risk of damage to property from accumulation of tephra, but virtually no direct risk to human life

INTRODUCTION

Mount Rainier is a dormant volcano which might endanger the lives of thousands of people when it again becomes active. The last significant eruptive activity of the volcano was between 120 and 150 years ago, and several minor eruptions were reported during the late 1800's. Mount Rainier will almost surely erupt again some time within the next few hundred years, but at present there is no way to predict when. It is possible, however, to forecast some of the effects an eruption will have if it is assumed that future volcanism will be of the same type and on the same scale as that which has occurred repeatedly during the last 7,000 years. This sample period of 7,000 years was chosen because the volcano's behavior during this period is better known than is its earlier history.

Conceivably, an eruption of Mount Rainier could occur which would be of a different type and on a vastly more violent scale than any of those of the recent past, and which could cause a major disaster in the surrounding region. Such a catastrophic eruption might resemble the one at Mount Mazama in southern Oregon, which preceded the collapse of the volcano to form the basin of Crater Lake about 6,600 years ago. During that eruption, hot masses of pumice rushed down valleys to distances of as much as 40 miles, and pumice erupted high above the volcano was carried by wind over a region of more than half a million square miles. If this kind of eruption occurred at Mount Rainier, property damage and loss of life could be high. However, the possibility that this kind of eruption will occur at Mount Rainier within the next few centuries is so remote that the extreme cost and major changes in land use that would be required to protect the public from such an event seem unwarranted. It is more practical, instead, to plan for the kinds of eruptions and other geologic events which have occurred in the past 7,000 years and which almost surely will occur again.

The chief hazards from such future eruptions will be mudflows and floods moving along valley floors and fallout of tephra (airborne volcanic-rock debris) in areas downwind from the volcano. Future lava flows probably will not extend beyond the flanks of the volcano, but mudflows and floods may be caused by rapid melting of snow and ice by lava. Avalanches of rock debris could bury valley floors, but these probably would reach only a few miles beyond the base of Mount Rainier.

VOLCANO MONITORING

A volcano can be monitored in several ways in order to detect the upward movement of molten rock that precedes an eruption. This molten rock causes earthquakes within the volcano which can be detected by instruments; at other volcanoes, an increase in the number and strength of quakes has been followed by an eruption. The instruments now being used to detect and record earthquakes at Mount Rainier include standard seismometers and a seismic-event counter which records only quakes that are stronger than a predetermined magnitude. Data from these instruments are relayed by radio to the University of Washington for recording and interpretation.

The thermal state of a volcano can be monitored by temperature-measuring devices (thermistors) placed in steam vents and areas of hot rock, and infrared radiometers which detect heat.

Figure 1. Map showing potential hazards from future eruptions of Mt. Rainier, Washington (from Crandell, D. R., 1973).

This map is one in a series that depicts the geologic events, such as landsliding and seiches (periodic oscillations of standing bodies of water such as lakes), which will occur if the area is shaken by a major local earthquake. Such a possibility is very real, for on the Seismic Map of the United States (Algermissen, 1969) the Henrys Lake quadrangle is wholly within seismic risk zone 3, defined as an area in which "major destructive earthquakes may occur."

▨ ZONE 1 – Area in which major ground breakage may occur during a major earthquake

▨ ZONE 2 – Area in which moderate ground breakage may occur during a major earthquake

☐ ZONE 3 – Area in which minor ground breakage may occur during a major earthquake

Of the many faults known in the Henrys Lake quadrangle, only those considered to be active are shown; all others have been omitted from this map.

Two types of ground breakage can be expected in this area if there is any renewed movement on the faults shown. The first type of ground breakage would occur along the trace of a reactivated fault and would consist of scarps, some conceivably as much as 20 feet high. Parallel to and near these major scarps would be other somewhat smaller ones. Both types of fault scarps would probably be confined to a zone about half a mile wide astride and along the trace of the reactivated fault.

The second type of breakage would result from the shaking the area underwent during the earthquake. Most of this breakage would occur in the loose unconsolidated sand and gravel deposits which form the floor of the Henrys Lake basin. Both open fractures and scarps would form. All, however, would be short and irregular. These would be randomly distributed and would reflect more the degree of compaction produced by shaking and vibration than any other factor. The scarps would probably be a foot or two high, and the open fractures would probably range from less than an inch to as much as 6-8 inches wide at the top and be closed at depths of a few feet.

Minor ground breakage would also be expected in semiconsolidated and unconsolidated deposits on steep slopes everywhere. There would also be multiple breakage and slump in the unconsolidated materials which form lakeshores and the banks of streams and rivers.

It has long been known that manmade structures built on loose or partly consolidated surficial materials are more likely to be damaged by vibration during a major earthquake than are comparable structures built on bedrock.

To a great extent the intensity of structural damage that may occur in this area (if there were movement on one of the shown faults) would have a pattern similar to that of the severity of ground breakage. Obviously, structures within areas of major ground breakage (Zone 1) are likely to be severely damaged by the abrupt displacement of ground during fault movement. Zone 2, areas of moderate ground breakage, delineates areas underlain mainly by unconsolidated sand and gravel. Structures built on these materials are likely to be shaken more, and therefore damaged more, than similar structures built in Zone 3 – areas of minor ground breakage – chiefly because Zone 3 includes all areas underlain by bedrock.

Fault – Dashed where inferred or concealed. U, upthrown side; D, downthrown side

SCALE 1:62500

CONTOUR INTERVAL 80 FEET
DATUM IS MEAN SEA LEVEL

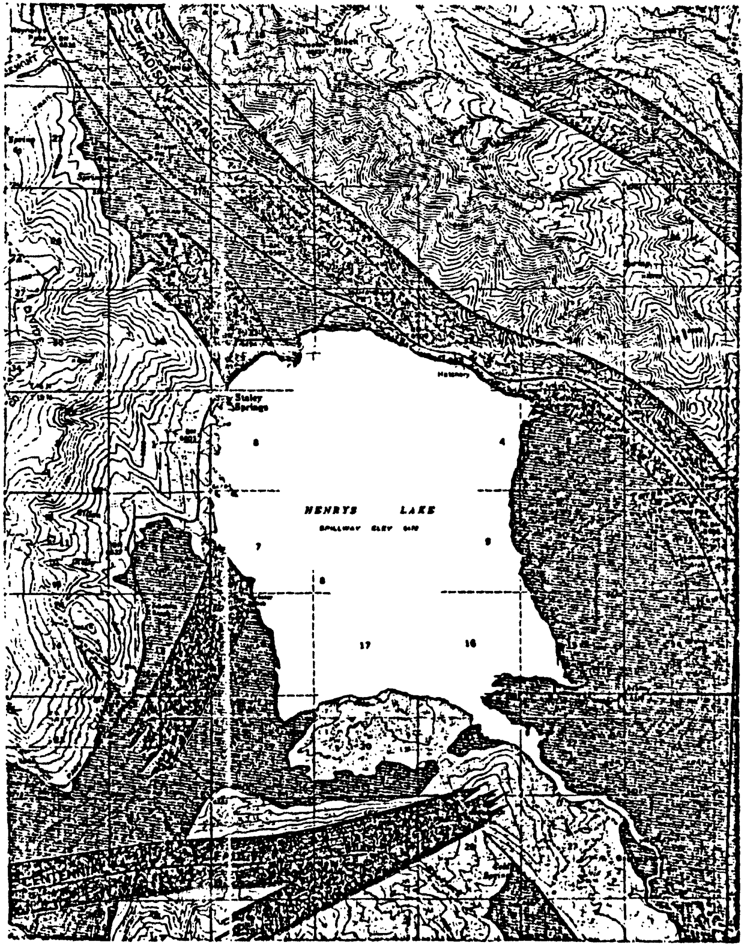


Figure 2. Map showing faults and ground-breakage hazards in the Henrys Lake Quadrangle, Idaho and Montana (from Witkind, I. J., 1972).

This map is one in a series that depicts the geologic events, such as ground breakage, landsliding, and seiches (periodic oscillations of standing bodies of water such as lakes), which will occur if the area is shaken by a major local earthquake. Such a possibility is very real, for on the Seismic Map of the United States (Algermissen, 1969) the Henrys Lake quadrangle is wholly within seismic risk zone 3, defined as an area in which "major destructive earthquakes may occur."

▨ Area that will probably be flooded by seiche

A *seiche* is a periodic oscillation of a body of water in a containing basin. If this area were shaken by a major local earthquake the waters of both Henrys Lake and Hebgen Lake would almost certainly be thrown into periodic oscillations during which the lake shores would be successively inundated for brief periods of time. (It is much as if a person carrying a basin of water were inadvertently jostled. The water would slosh back and forth, and might even top the basin's sides.) During the seiche it is likely that some waters would top Henrys Lake outlet dam and flow southeastward, following the Henrys Lake outlet, and spread across the basin floor. Henrys Fork might also swamp parts of its flood plain for brief periods.

If the outlet dam were to fail – a remote possibility – it is estimated that about 80,000 acre-feet of water would fan out across the southern end of the Henrys Lake basin. Although some water would seep into the porous sands and gravels which floor the basin, much of the water would escape downstream following Henrys Fork. Ultimately these waters would discharge into the Island Park reservoir.

Estimates of the shape and extent of those areas likely to be flooded by seiches are based on three assumptions. First, the earthquake will be a major local one; second, the lakes will be at near capacity when the earthquake occurs; and third, during the first surges of the seiches the lakes will flood their shores to a level about 30 feet higher (as measured vertically, not horizontally) than their pre-earthquake levels. So, for Henrys Lake, the surface of which is at an altitude of about 6,472 feet when the lake is full, the waters may extend as far inland as the 6,500-foot contour (6,472 feet + 30 feet = 6,502 feet). For Hebgen Lake, whose surface is at an altitude of 6,544 feet when the lake is near capacity, the waters may flood all ground below the 6,575-foot contour (6,544 feet + 30 feet = 6,574 feet).

Judging by the action of Hebgen Lake during the 1959 earthquake, once the seiche has subsided, Henrys Lake will reflect the tilting of the block of the earth's crust on which it rests. Parts of its shoreline will be flooded even as the lake floor on the opposite side of the lake is exposed. The basin (in the example above) will have been tilted, and the surface of the water as it comes to rest after sloshing back and forth will act as a level to indicate the amount of tilt.

The periodic flooding that occurs during a seiche is short-lived and gradually subsides; by contrast, the lake shores that are submerged when the seiche ends will be permanently flooded. A block of the earth's crust containing the lake within its confines and possibly measuring hundreds of square miles in area will have shifted and it cannot rebound to its original position.

■ ZONE 1 – Area in which rockslides, rockfalls, or earthflows are very likely to occur if area is shaken by a major local earthquake

▓ ZONE 2 – Area in which rockslides, rockfalls, or earthflows may occur if area is shaken by a major local earthquake

░ ZONE 3 – Area in which rockslides, rockfalls, or earthflows are unlikely to occur if area is shaken by a major local earthquake

⬭ Slump block – Arrows indicate direction of movement

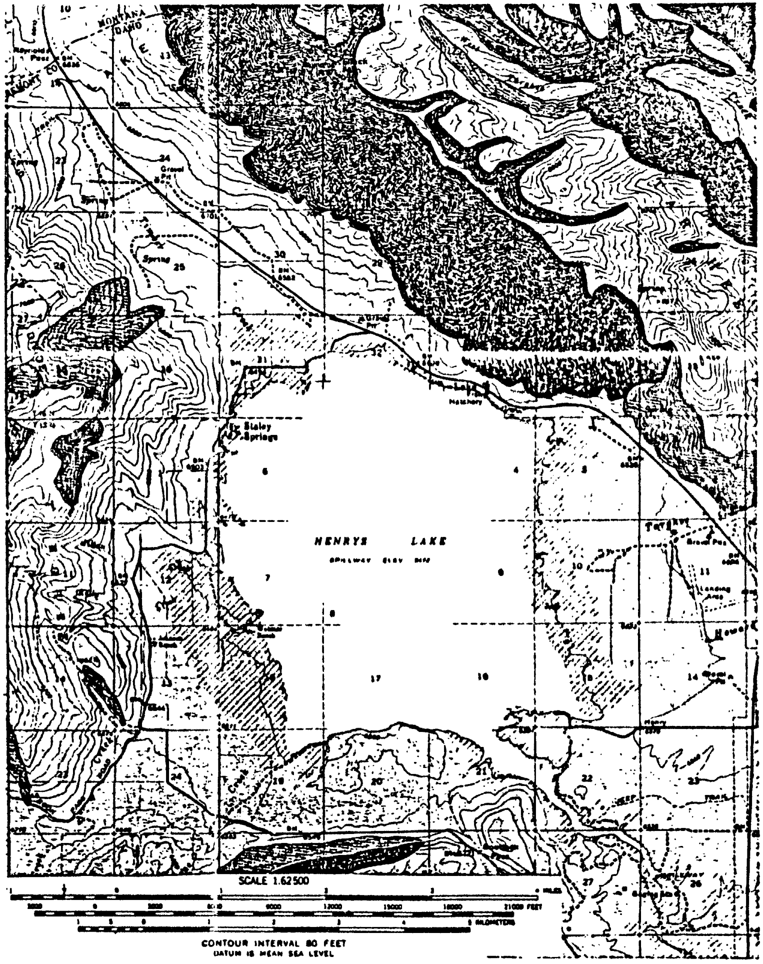


Figure 3. Map showing seiche, rockslide, rockfall, and earthflow hazards in the Henrys Lake Quadrangle, Idaho and Montana, (from Witkind, I. J., 1972).

EARTHQUAKE RISK ZONES

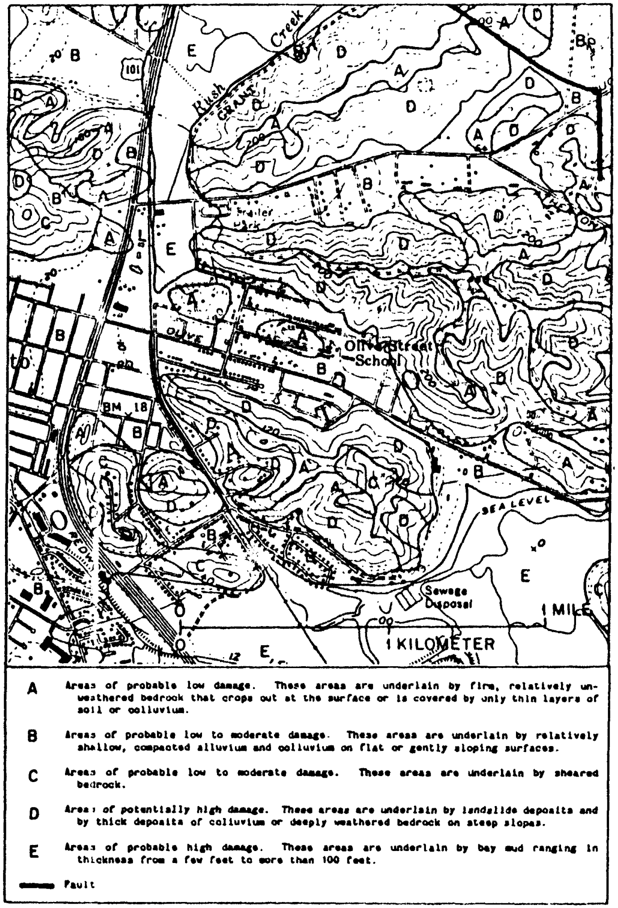
Five "earthquake risk" zones are developed and mapped at a scale of 1:12,000 (fig.   ) for the Novato area by the California Division of Mines and Geology in cooperation with the County of Marin and the City of Novato, California. Ground shaking, surface rupture, ground failure, and landslides are considered as having the highest potential for damage, whereas tsunamis are considered not to be significant potential hazards (Rice, 1975). An earthquake of approximately magnitude 8 with an epicenter in the northern San Francisco Bay area is postulated (Rice, 1975).

Geologic materials are grouped into five zones on the basis of similar physical properties; for example, "firm, relatively unweathered bedrock" in zone A, "relatively shallow compacated alluvium and colluvium" in zone B, "deep upslope landslide deposits" in zone D, and "bay mud" in zone E. In addition, a symbol is placed over the trace of the potentially active Burdell Mountain fault (Rice, 1975).

Some geologic effects of the postulated earthquake are predicted for each zone; for example, "subject to relatively high frequency vibrations" in zone A, "may be threatened by landsliding" in zone B, and "rapid differential settlement" in zone E. In addition, each zone is rated from probable low to probable high damage. The geologic materials, geologic effects, and damage ratings are combined and mapped by zone in figure

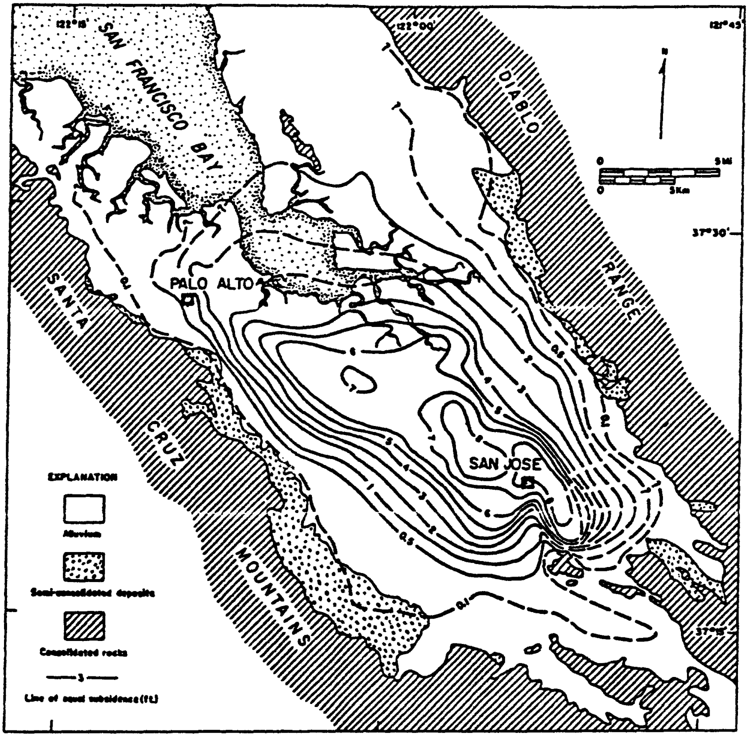
Recommendations related to the risk zones, such as "large public structures... not be located on any demonstrated fault trace, "residential developments on fill may not be fitting use" in zone E, and "Engineering geology reports, based on detailed geological mapping, (should) be required" in zone D, are made (Rice, 1975).

The "earthquake risk" zones and recommendations have been considered by the City of Novato and the County of Marin and used as a basis for their seismic safety plan elements.



A	Areas of probable low damage. These areas are underlain by firm, relatively unweathered bedrock that crops out at the surface or is covered by only thin layers of soil or colluvium.
B	Areas of probable low to moderate damage. These areas are underlain by relatively shallow, compacted alluvium and colluvium on flat or gently sloping surfaces.
C	Areas of probable low to moderate damage. These areas are underlain by sheared bedrock.
D	Areas of potentially high damage. These areas are underlain by landslide deposits and by thick deposits of colluvium or deeply weathered bedrock on steep slopes.
E	Areas of probable high damage. These areas are underlain by bay mud ranging in thickness from a few feet to more than 100 feet.
▬	Fault

Figure 4. Earthquake risk zones in the Novato area, Marin County, California (Kockelman and Brabb, 1979; from Rice, 1975; in Brabb, 1979).



Land subsidence in the Santa Clara Valley, a large metropolitan area on the south end of San Francisco Bay, California was recognized after releveling in 1932. This subsidence ultimately affected more than 595 km^2 (230 mi^2) of land (Poland and Green., 1962) and locally attained a maximum of 3.9 m (12.9 ft) in the City of San Jose (Poland, 1977). Approximately 260 km^2 (100 mi^2) of land subsided more than 1 m (3 ft) from 1934 to 1967 (Poland, 1969). Because most of the shoreline area consists of marshlands that constrained early residential and commercial development except for salt evaporation ponds, the economic impact from the subsidence was smaller than it might have been. Nevertheless, subsidence of land adjacent to the bay did force considerable construction and raising of levees. Poland (1977) estimated that 44 km^2 (17 mi^2) of land was spared from inundation by this construction. Approximately $9 million was spent on levees at the ends of depressed stream channels, and an unknown amount was spent by a major salt company on levees to preserve 78 km^2 (30 mi^2) of salt evaporation ponds along the bay margins (Poland, 1977). In addition to costs associated with loss of elevation, more than $4 million was spent to replace or repair water wells whose casings had been damaged by compaction in the aquifer system (Roll, 1967). The gross costs of subsidence probably have been $15-20 million (Poland, 1977), although Fowler (1981) estimated that direct costs attributable to subsidence totaled more than $130 million when brought to 1979 values. His estimate was dominated by an estimated $103 million that would be required to construct a new levee system to improve protection of bayfront lands from salt-water flooding.

Figure 5. Subsidence, in feet 1934 to 1967 in Santa Clara Valley, California (Poland and Ireland, 1968 from Holzer, 1983, in press).

certainly be mitigated. This simple map conveys the essential information effectively.

Glacier dammed lakes and outburst flood courses in the Cook Inlet area, Alaska, are delineated in Figure 6. Just a few symbols and colors are effective in highlighting this particular hazard. Often the simplest type of map (Figure 7) is all that is required to depict the relative range of a particular risk. The addition of three colors, keyed to three selected zones of increased numbers of iceberg sightings, can enhance the map and draw attention to the zones of highest risk.

MANIPULATION AND AGGREGATION OF HAZARD AND OTHER INFORMATION

Maps containing hazard and other information may be combined or aggregated by overlaying them on a light table and drawing boundaries on a clear overlay around the new or aggregated units. Each of the maps must be in categories and in numbers of units that lend themselves to aggregation. Otherwise, the process becomes difficult and the resultant map contains a proliferation of poorly defined units which are not readily applicable to the needs of the user. Aggregated maps of earth science data can in turn be combined with other aggregated maps as may be required. Logic and simplicity are the two important guides in the preparation of aggregated maps, or basic geologic maps.

As the number of factors or attributes increases in the preparation of an aggregated map, the overlay technique must move from the light table to the computer. Other considerations affecting this decision are the intended frequency of using these data and the cost of digitizing the map information. This latter technique significantly increases the speed of map aggregation, provides an electronic file for numerous other aggregations, and provides a method for immediate statistical analysis of the map and its units.

The earthquake hazard map of the Henry's Lake Quadrangle by Witkind, (1972, Figure 8) is an overlay aggregate of his two other related maps (Figures 2, 3) of the same area. This map is a good synthesis of all earthquake-related hazards in that area, and it provides a useful graphic guide for planning, construction, and hazard avoidance and mitigation.

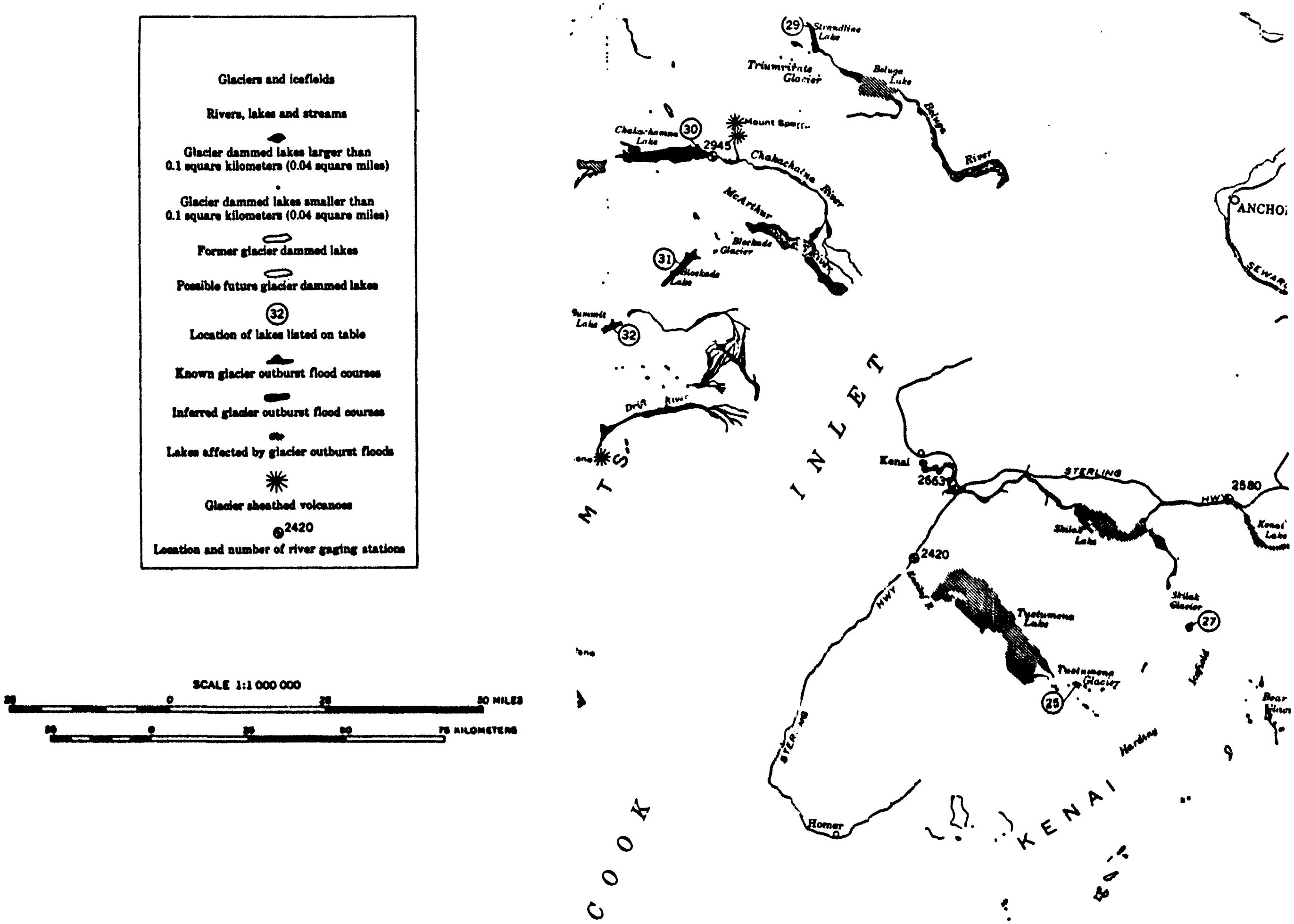
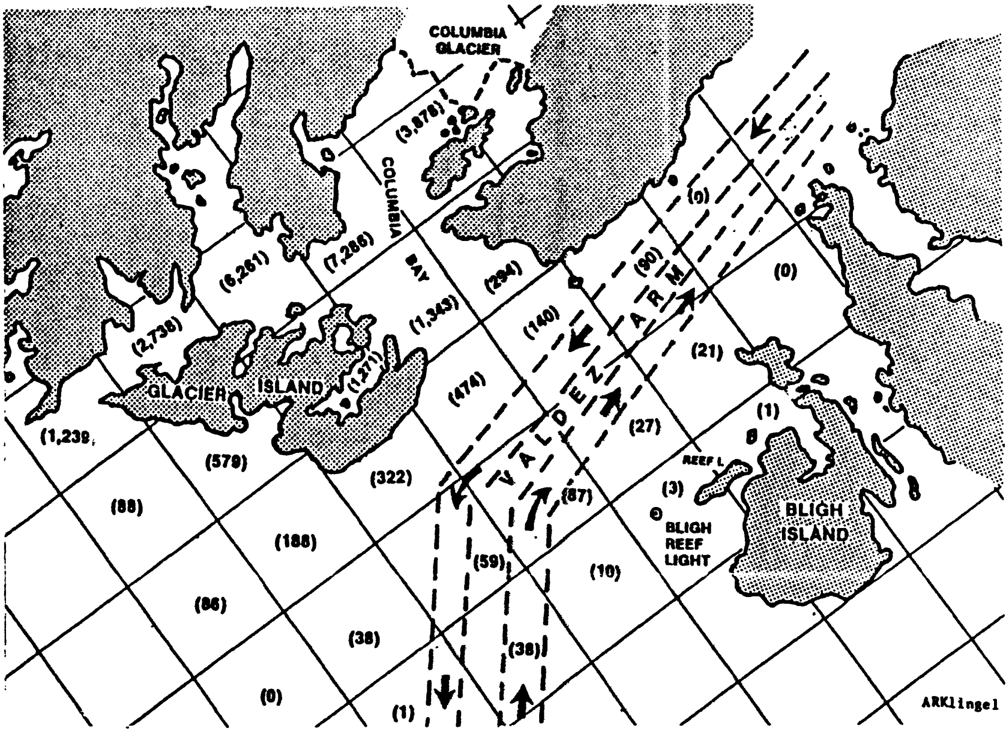


Figure 6. Glacier dammed lakes and outburst floods in Alaska (from Post, A. and Mayo, L. R., 1971).



Columbia Glacier in south-central Alaska appears to be unstable and on the verge of a drastic retreat that over the next several years could release many large icebergs into the important oil shipping lanes of Prince William Sound near Port Valdez. The USGS predicts that the glacier's rate of retreat will accelerate during the next two to three years. As a result, the annual discharge of icebergs will increase to a peak of about 20 to 27 million tons per day during the period of 1982 and 1985, or about six to eight times the discharge during 1978.

Because an increased flow of large icebergs could become a problem to tankers transporting oil from the southern terminal of the Trans-Alaska Pipeline at Port Valdez, about 110 miles east of Anchorage, the USGS has been monitoring the 425-square-mile glacier since 1974. The program has been intensified in recent years to determine the probable rate of retreat and rate of discharge of the resulting icebergs. During 1979, tankers carried over 1 million barrels of oil a day through Prince William Sound from the pipeline terminal for consumption in the lower 48 states.

Dr. Mark F. Meier, chief of the USGS glaciology program, Tacoma, Washington, said, "Give or take a year, we expect Columbia Glacier to have retreated about 5 miles by 1986. This will release about 10 cubic miles of ice, some of which will be discharged into the narrow Valdez Arm of Prince William Sound in sufficient size to be of concern to shipping. "As the glacier retreats at a speed of a mile or more per year, as much as 50 cubic miles of icebergs could be discharged into Prince William Sound over the next 30 to 50 years," Meier said.

Figure 7. Map of reported iceberg sightings in Valdez Arm during the period 1977-1982. The numbers of iceberg sightings within each grid are relative approximations because on many days no trips for sightings were made; or, sometimes two sighting trips were made on the same day; and, during some years many more trips were made than during others (from Klingel, 1983).

The Henrys Lake quadrangle is wholly within seismic risk zone 3, defined on the Seismic Map of the United States (Algermissen, 1969) as an area in which "major destructive earthquakes may occur."

If there were renewed movement along one of the active faults in the quadrangle, and an earthquake resulted, parts of the area would be more or less severely marred in one way or another. Manmade structures within these areas would be damaged to varying degrees. Some parts of the area would be inundated by oscillatory waves (seiches) set up in Henrys Lake and Hebgen Lake; ground breakage of varying degrees of intensity would occur in other parts of the area; and rockslides, rockfalls, and earthflows would be triggered locally. Two other maps, (1) Seiche, rockslide, rockfall, and earthflow hazards, Map I-781-C, and (2) Faults and ground-breakage hazards, Map I-781-D, depict the possible extent of these various geologic hazards. This map, derived from these other maps, divides the quadrangle into four zones of earthquake hazard. It shows those specific areas likely to be damaged by several of the geologic hazards listed above; in a sense it emphasizes the potential cumulative damage which may occur in any one locality.

Although some parts of the quadrangle may be unaffected during a major local earthquake, other parts may be severely marred as a result of the cumulative effects of several kinds of geologic events. Thus, in places the floor of the Henrys Lake basin may be practically undamaged, being broken here and there only by minor fractures. By contrast, some of the nearby mountainous parts of the area may be severely damaged by major ground breakage as well as by rockslides, rockfalls, and earthflows. Obviously, some parts of the area are much more hazardous than others. This map arbitrarily divides the quadrangle into four zones of earthquake hazard based in part on the kinds of geologic hazards likely to be triggered in any one area, and in part on the differing intensity or certainty of these events.

EARTHQUAKE HAZARD ZONE A – Double major hazard. Areas in which two kinds of major hazard are present. For example, includes areas most likely to undergo major ground breakage and inundation by a seiche.

EARTHQUAKE HAZARD ZONE B – Single major hazard. Areas in which two kinds of hazard are present, one of which is major. For example, includes areas in which rockslides, rockfalls, or earthflows are very likely but in which only minor ground breakage is likely.

EARTHQUAKE HAZARD ZONE C – Moderate hazard. Areas in which moderate or lesser hazards are likely to be present. For example, includes areas in which both moderate ground breakage and rockslides, rockfalls, or earthflows may occur.

EARTHQUAKE HAZARD ZONE D – Low hazard. Areas in which one or more lesser hazards are likely to be present. For example, includes areas in which only minor ground breakage is likely.

For most of the area the selected zones represent possible rather than certain damage. But, locally, a few areas are almost sure to be damaged; for example, the broad low flats which flank Henrys Lake. These lowlands are virtually certain to be repeatedly inundated by the back and forth sloshing of the lake that will occur during a major local earthquake. What is uncertain is the amount and degree of damage that will result from such oscillations.

VIBRATION DAMAGE – It has long been known that manmade structures built on unconsolidated or semiconsolidated materials are more likely to be damaged by the vibrations set up during an earthquake than comparable structures built on bedrock. Such damage can occur far from the epicenter[1]; what is important is the type of material on which the structures are built. Bedrock is preferable to unconsolidated detritus.

[1] An imaginary point on the earth's surface directly above the focus point (the position where the rocks first ruptured) of the earthquake.

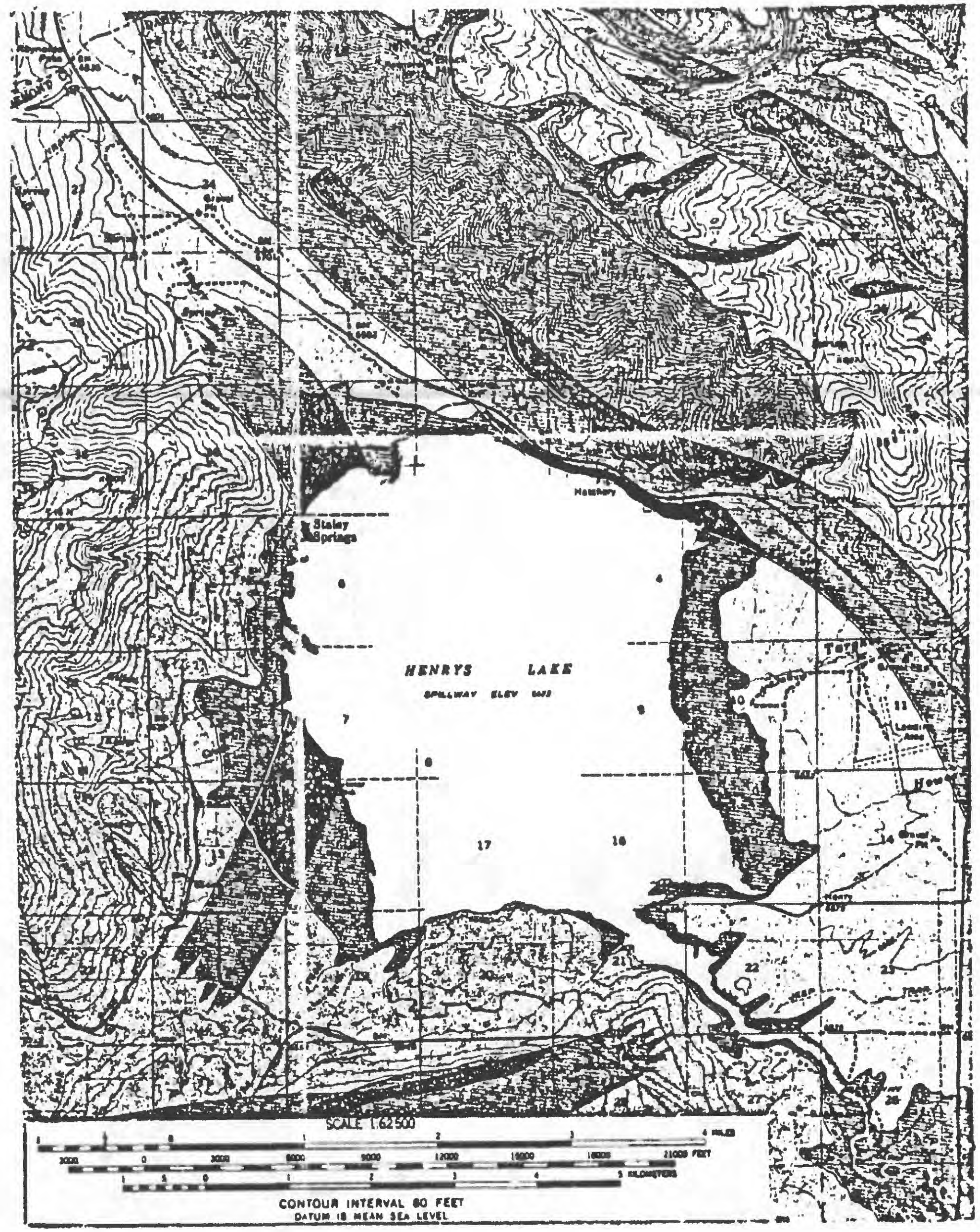


Figure 8. Earthquake hazard map of the Henrys Lake Quadrangle, Idaho and Montana (from Witkind, I. J., 1972).

Miller (1973, Figure 9) combines slope and geologic materials to construct a relative slope stability map of part of West-Central King County, Washington. His particularly lucid explanation greatly contributes to the value of the map as a planning tool, and the small number of map units places attention and emphasis on the critical areas.

A more complex illustration of the preparation of a relative slope stability map of part of Northern Contra Costa and Southern Solano Counties, California, is provided by the following series of maps: (1) a generalized slope map of that area (Figure 10) is combined with (2) a generalized photointerretive map of landslide deposits (Figure 11) to produce (3) a preliminary relative slope stability map (Figure 12) which is then combined with (4) a map of bedrock and surficial geologic units (Figure 13) considered to be especially susceptible to slope failures. The resultant relative slope stability map (Figure 14) contains seven map units which are clearly delineated and appropriate to the needs of regional planners. A much larger scale would be required at the municipal level or by builders, because this resultant map has become too detailed for its small scale. Any further aggregation could best be accomplished by digitizing the individual maps and combining them in a computer.

The basic procedure for aggregating or compositing maps by computer has been well explained and illustrated by Van Driel (1980, Figure 15). Currently, source maps can be read by an optical scanner for automatic roster digitization of up to 12 colors at once. The digitized map can be edited on a color video screen at any magnification, and the editor can add, delete, combine, or reformat elements of the map with electronic tools and software designed for cartographic functions. The edited video screen images can be printed, stored in a digital file or transmitted. Figure 16 is an example of a computer map processed in a unique way. Color separation and screening for four-color process printing was done on a large format laser plotter using dot screens and angles developed on that equipment and plotted directly onto the color separations as data were read from the digital file. Note that a complete statistical analysis of the map unit areas are also available.

PURPOSE

This map classifies part of west-central King County, Washington into areas of differing relative slope stability. Slope stability is a variable natural condition that may pose problems and should be evaluated before land use is changed. Knowledge of the distribution of areas of differing slope stability can help land planners and decision makers. Urbanization engulfs areas of different natural characteristics; failure to identify areas having potential problems can result in physical or financial difficulty.

This part of the Puget Sound lowland is separated into three classes of stability: terrain virtually free of stability problems (class 1), terrain almost certain to be susceptible to such problems (class 3), and terrain that is stable under natural conditions, but which may become unstable owing to man's activities, and which should be examined in detail before changes in land use are adopted (class 2).

The boundaries shown on the map are arbitrary lines based on available information. Areas of different classes too small to identify at this map scale, or changes in steepness of slope confined to the zone between contours so that these changes do not show on the map, are included within each classified area.

The map was compiled from data collected by others (see references). Their work and published descriptions of slope failures in the Seattle area has indicated that the presence or absence of tight silt or clay beds beneath sand beds is a principal geologic criterion for potential slope instability.

The selection of 15% slopes to separate classes 1 and 2 is also based on recorded experience of others.

The validity of this three-fold slope-stability classification has been checked briefly in the field.

DESCRIPTION OF SLOPE STABILITY CLASSES

In this report, slope stability refers to the resistance to gravitational forces of earth materials underlying an inclined surface; instability results in outward and downward movement of these underlying materials.

Class 1: characterized by slopes less than 15%, and by near-surface materials that probably do not include silty or clayey beds. Includes some hard glacial drift on gently rolling uplands. Slopes tend to be stable unless deeply excavated; subsequent instability generally is confined to excavation walls or embankments of artificial fill.

Class 2: characterized by slopes that are steeper than 15% and by absence of silty or clayey beds near the surface. Includes some hard glacial drift that generally forms steep slopes along bluffs and ravines, and caps small hills and ridges within the upland. Tends to be stable under natural conditions.

Class 3: characterized by slopes steeper than 15% and by silty or clayey beds near the surface, beneath sand. Includes glacial drift that generally forms steep slopes along bluffs and ravines. Although most slopes in class 3 are stable, all are believed to be potentially unstable and ready to slide from sufficient natural or man-made causes.

UNDERSTANDING SLOPE STABILITY

Stability of slopes is influenced by two general kinds of conditions: constant and changing. The constant factors, such as the general steepness of the land surface and the types and interrelation of geologic materials, tend to determine the location of slope instability. Changing factors, such as erosion, man's activities, and addition of moisture tend to determine the timing and frequency of landslides. Of these changing factors, the addition of excess moisture, most commonly by rainfall, is probably the most crucial in the Puget Sound region. As a result, most landslides occur during the rainy season.

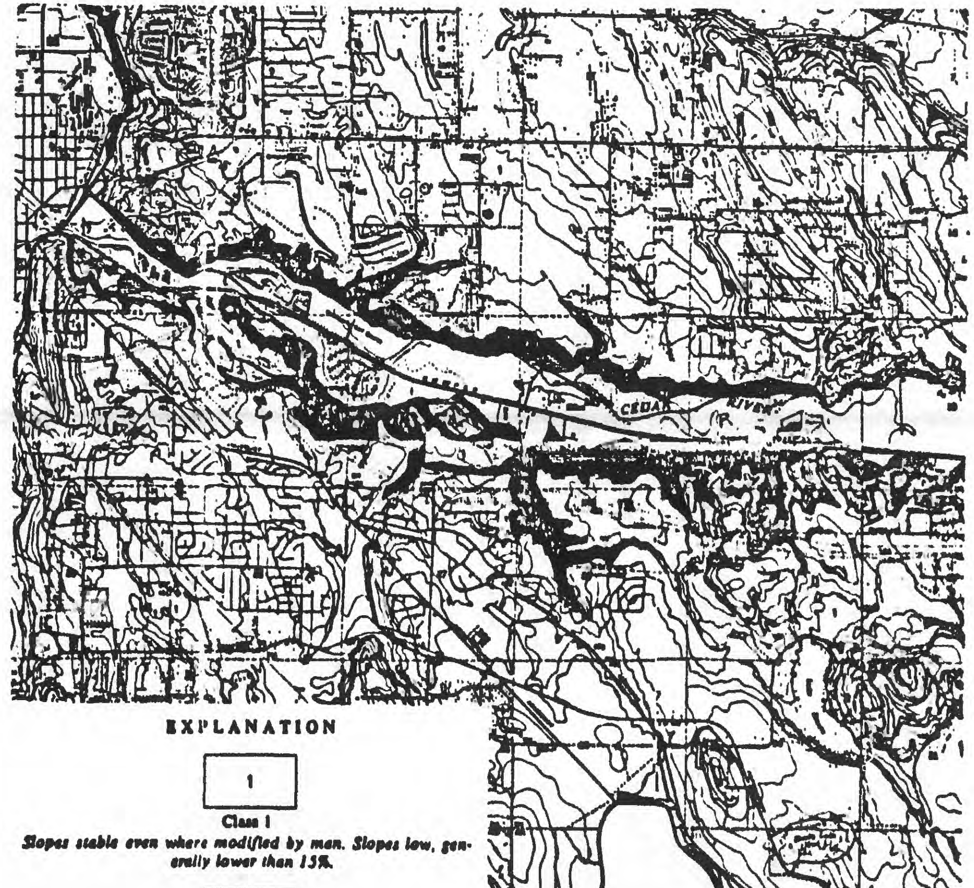
Geologic conditions may strongly influence relative stability on slopes of similar steepness. Most of the geologic materials in the map area are flat-lying and relatively weak glacial or alluvial deposits. The composition and physical characteristics of such deposits vary greatly, both laterally and vertically. Of particular importance is the occurrence of relatively impermeable materials, such as tight silt or clay, beneath or within more permeable materials, such as sand or gravel materials. Water moving through earth materials can saturate those which are permeable (sand or gravel) and is impeded by beds which are impermeable (silt or clay). The result may be to cause springs to flow from a hillside near the contact between the sand and clay, or to saturate the thin overlying soil materials that blanket the slopes, or to saturate the sand above the silt or clay; such saturation may cause the materials to lose cohesion and move downslope.

Other kinds of geologic materials also are important to the stability of a slope. Hard rock forms steep, normally stable slopes along some of the bluffs bounding mainland stream valleys; it also forms slopes of variable steepness in the mountains in the eastern part of the mapped area. In most places, slopes on hard rocks are stable. However, soil and slope debris overlying the rock may be unstable. Commonly, the thin soil and debris are highly permeable but the underlying rock is not. When saturated, the soil and debris move downward and the surface of the rock acts as a slip surface.

Two aspects of slope stability are not evaluated on this map. No attempt is made to determine or depict the effect of landslide debris encroaching upon flat, stable surfaces from unstable slopes uphill; sites along the base of class 3 slopes should be evaluated for this potential risk. Also, no assessment is made of the stability of man-made cuts and fills.

SCALE 1:48 000

1 MILES ½ 0 1



EXPLANATION

1 — Class 1
Slopes stable even where modified by man. Slopes low, generally lower than 15%.

2 — Class 2
Slopes normally stable, but may become unstable due to man's activities. Slopes moderate, generally steeper than 15%. Not underlain by clay beds, but may include ancient inactive landslides.

Class 3
Slopes readily become unstable due to natural events and to man's activities. Slopes moderate to steep, generally steeper than 15% and in places steeper than 40%. Probably underlain by tight silt or clay beds.

Modified land
Slopes not classified. Large excavations and fills made after the topographic map was prepared, so that map contours cannot be used to classify slope stability.

Old landslide deposits
A single deposit may include areas of different slope stability classes. Apparently stable now, but stability could change with changes in climate or slope.

Figure 9. Map showing relative slope stability in part of west-central King County, Washington (from Miller, R. D., 1973).

Figure 10. Generalized slope map of part of northern Contra Costa and southern Solano Counties, California (from Nilsen et al., 1979).

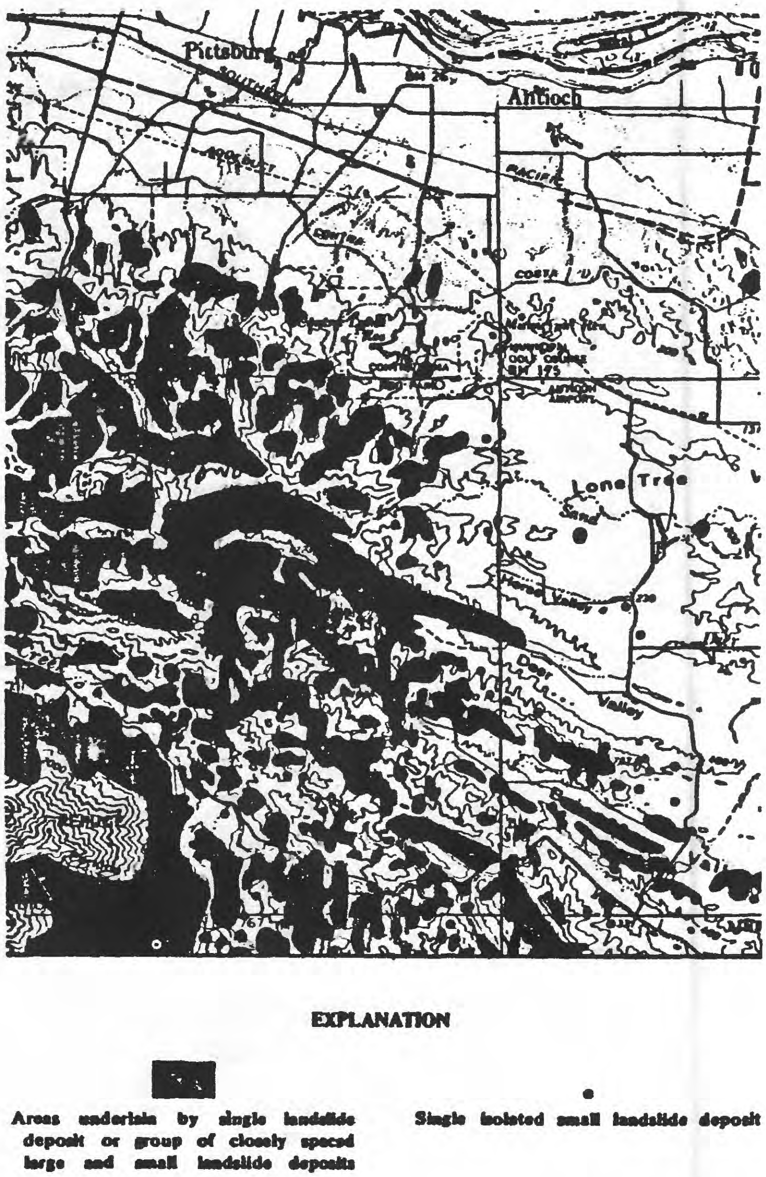


Figure 11. Generalized photointerpretive map of landslide deposits in part of northern Contra Costa and southern Solano Counties, California (from Nilsen et al., 1979).

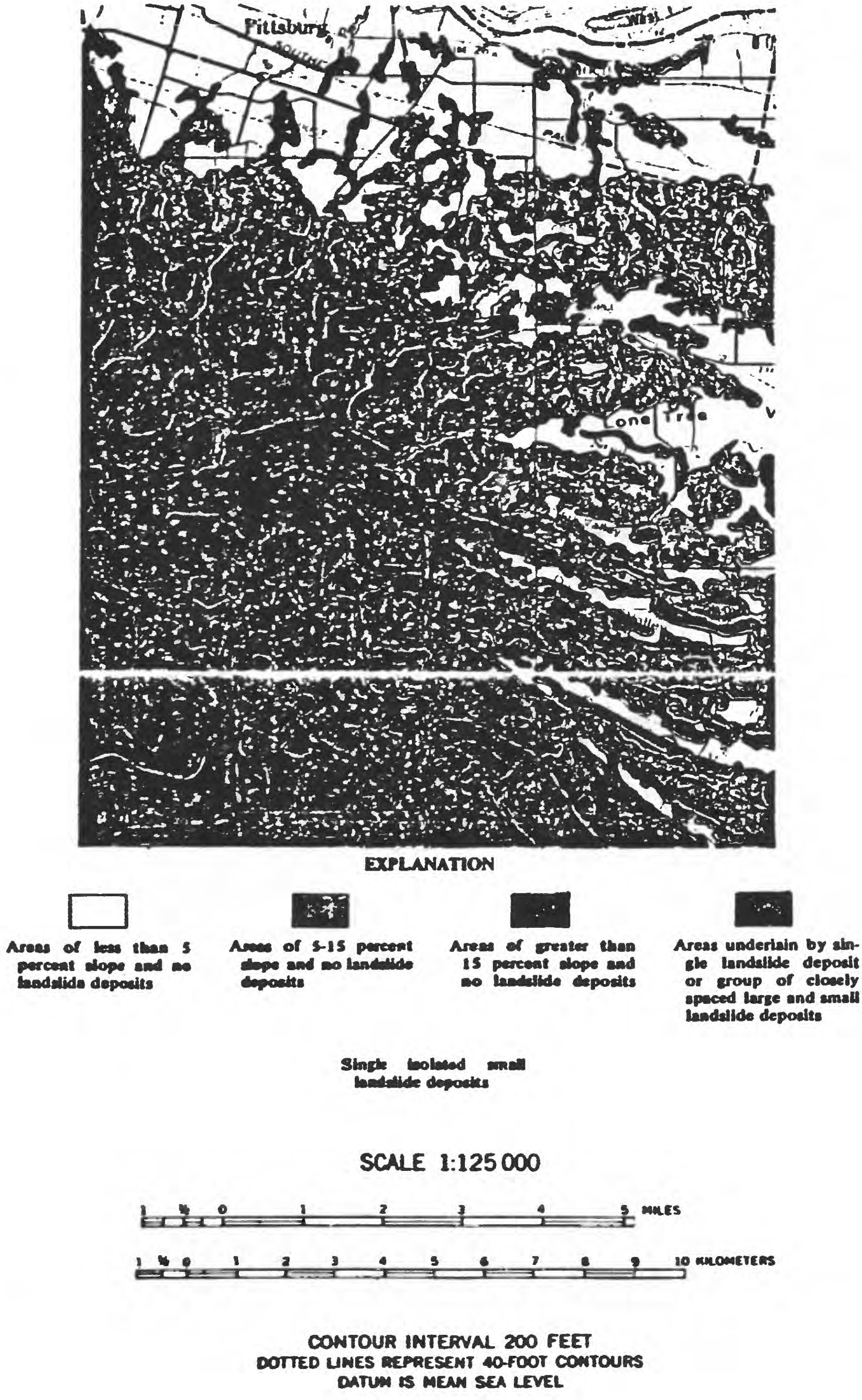


Figure 12. Preliminary relative slope stability map of part of northern Contra Costa and southern Solano Counties derived by combining the generalized slope map (figure 10) and generalized map of landslide deposits (figure 11) (from Nilsen et al., 1979).

Figure 13. Distribution of bedrock and surficial geologic units condidered to be especially susceptible to slope failuresin part of northern Cjontra Costa and southern Solano Counties (modified from Brabb and others, 1971, Nichols and Wright, 1971, and Sims and others, 1973).

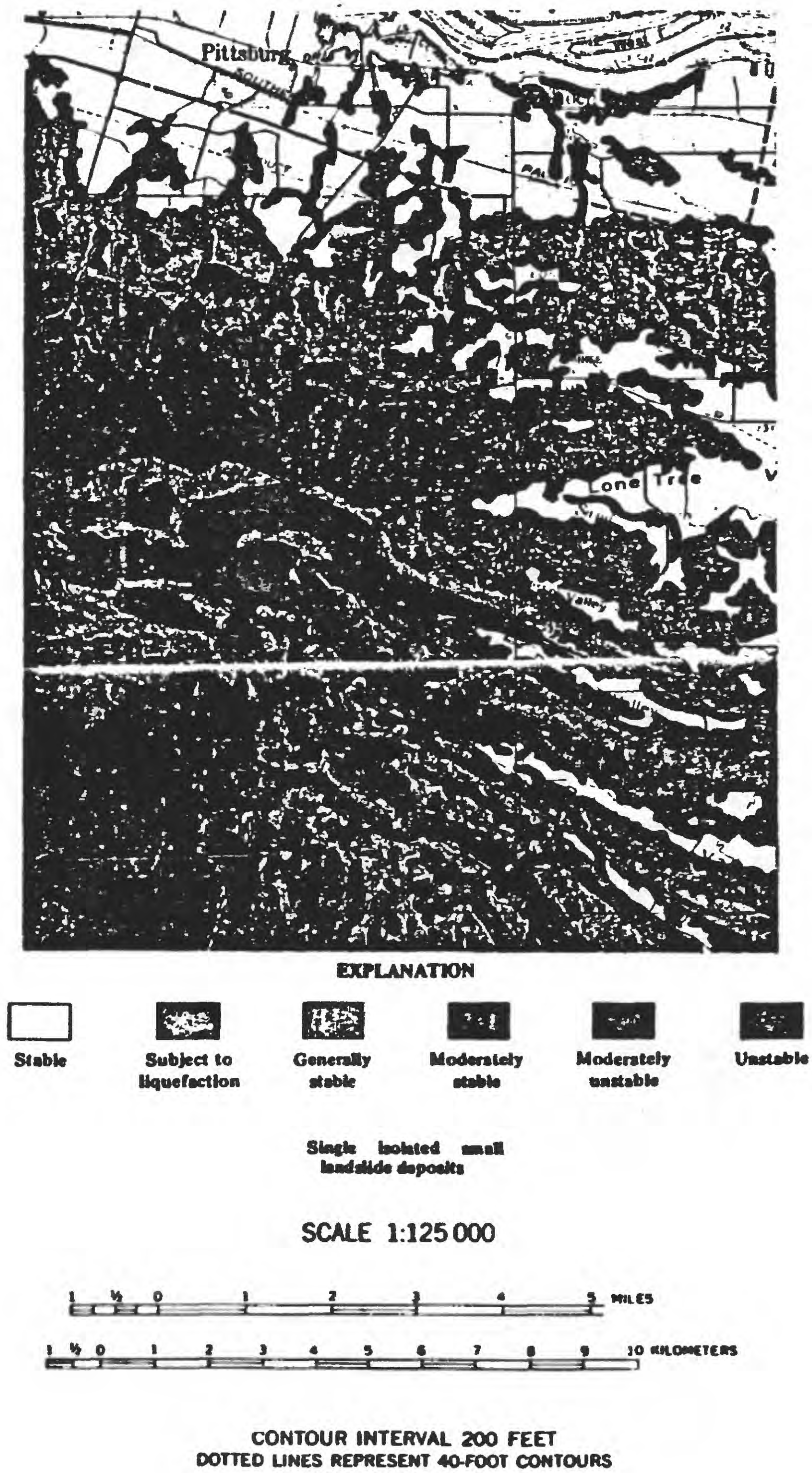


Figure 14. Relative slope stability map of part of northern Contra Costa and southern Solano Counties derived by combining the preliminary relaive slope stability map (figure 12) and map of bedrock and surficial geologic units considered to be especially susceptible to slope failures (figure 13).

The general procedure for producing composite maps by computer is illustrated in Fig. . In order to combine source maps with the computer, the mapped information must be converted to numerical form. Fast, economical computer processing is made possible when this information is stored in small subdivisions of the source map, called cells. In a process known as digitizing, a sampling grid is superimposed on the source map, and the latitude-longitude address and numerical code for the map unit are recorded for each cell. These data are put on punchcards for ease in updating and correcting and are stored on magnetic tape or disk for processing. The program user specifies, through a set of control cards, the way in which the digitized source maps are to be processed.

A composite map is prepared on a cell-by-cell basis by summing the numerical values that occur in each cell when several factors are combined. The resulting composite map, which represents the synthesis of all factors, is produced by a line printer. A detailed example of the compositing process is shown in Fig. , in which four single-factor maps are combined.

The source maps in Fig. a have been divided into cells, and the digitized version of each map is represented by Fig. b. Although letters are used here for clarity, a four-digit numerical code is actually recorded and stored wherever a factor is present in a cell of a source map being digitized. In the example (Fig.), the mapping program user considers it desirable to distinguish, on the composite map, each of the factors that contributes to every combination. In general, the number of possible composite map units, including each of the factors occurring alone, is given by the expression $2^n - 1$, where n is the number of factors. For a composite map of four factors, therefore, 15 map units are possible from the factors and their combinations. The expression may be simplified to 2^n if the null set (no factors occurring in a cell) is added to the total. By designating importance values 1, 2, 4, and 8 for the factors in this example (Fig. a, a unique score from 0 through 15 is calculated by the computer for each combination of factors as shown in Fig. d. The computer prints a user-selected symbol for each score level to produce the composite map. These symbols may be alphabetical, including special symbols; numerical; or combinations produced by overprinting as in the symbol for score 10, which is formed by combinig "T" and "V" (Fig. e).

The method of compositing that produces a unique score for every combination is very useful in understanding the interrelationships of large numbers of spatial factors. However, the method does have limitations. The 64 map units produced by compositing six factors may be fascinating to the program user but barely comprehensible to a subsequent user of the composite map. Several alternatives exist, all of which reduce the composite map complexity by sacrificing detail or the capability of identifying factors within combinations. The most straightforward technique for combining six or more factors is the assignment of equal importance values to all the factors. If six factors are combined, for example, and each factor is assigned the value one, the highest possible score is six, and the score indicates the number of factors that occur in each cell of the composite map. This method is particularly useful in analyses that involve a large number of factors of similar importance and type, where it is sufficient to show the occurrence, rather than the kinds, of factors in combination. If the factors do not have the same relative importance, a third technique may be used. Each of the factors is assigned a value that reflects its importance in the analysis, and the resulting scores are grouped by the program user. Each group consists of a range of scores the magnitudes of which will have similar meaning when the results of the composite map are used in making a decision. Although similar to the "unique combinations" method (Fig.), which provides for the assignment of different importances, this grouping of scores allows the program user to combine more factors in a composite map that is easier for users to read and understand.

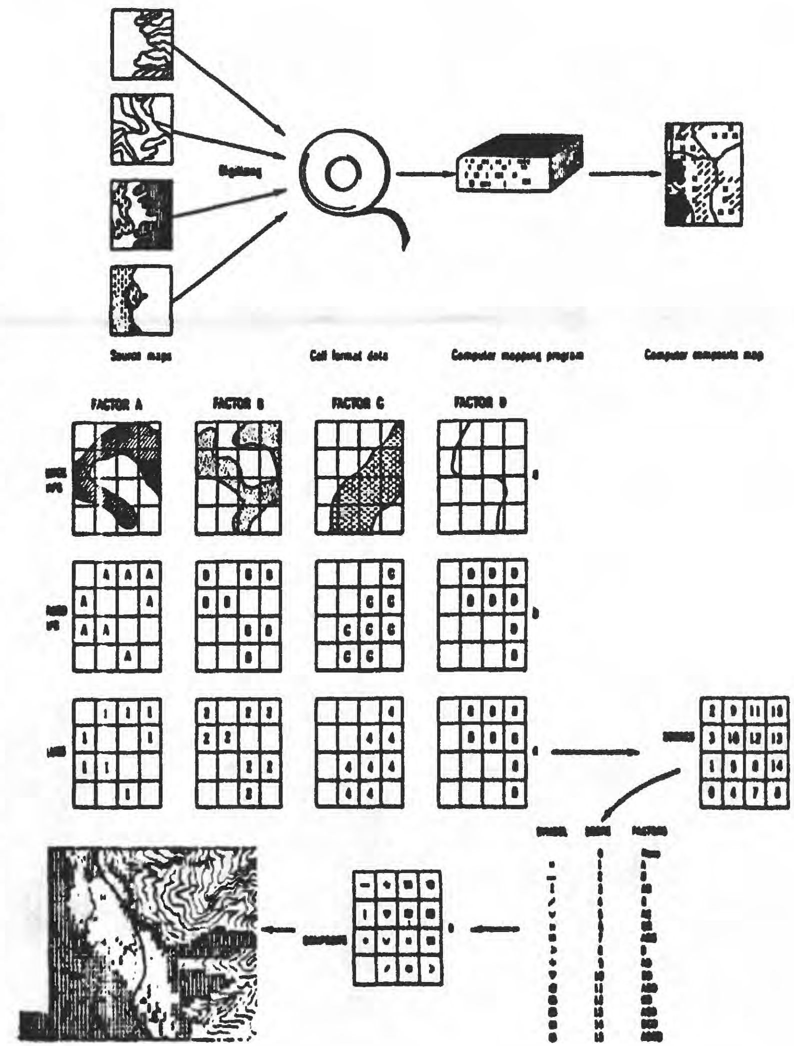
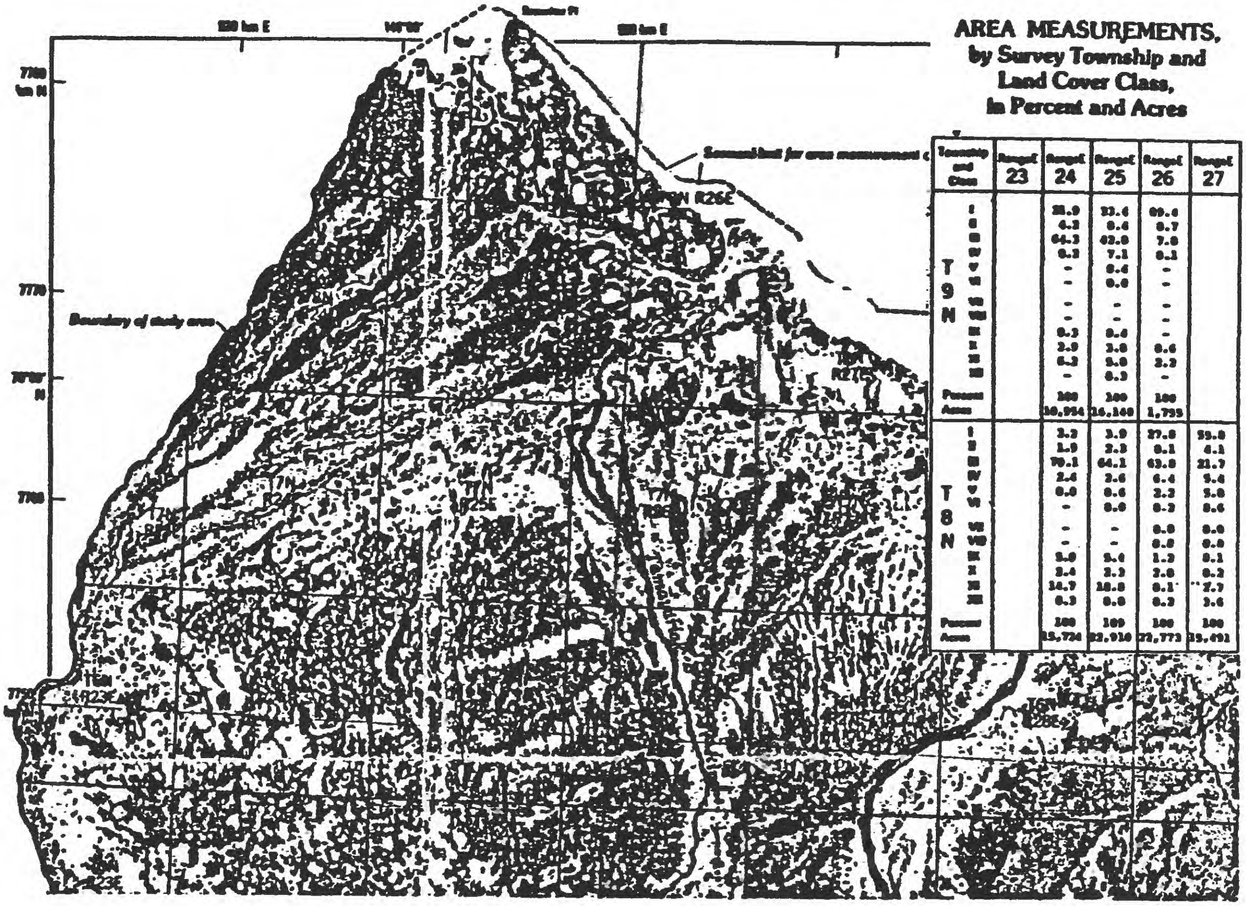


Figure 15. Example of computer-composite mapping a) source maps; b) digitized version of each source map; c) importance values of each factor; d) computer-scores for each combination F factors; e) symbols used in map construction (from Van Driel, 1980).



AREA MEASUREMENTS, by Survey Township and Land Cover Class, in Percent and Acres

Township and Class		Range 23	Range 24	Range 25	Range 26	Range 27
T 9 N	I		21.0	33.6	89.6	
	II		4.2	0.4	0.7	
	III		64.3	42.0	7.0	
	IV		0.3	7.1	0.1	
	V		-	0.4	-	
	VI		-	0.0	-	
	VII		-	-	-	
	VIII		-	-	-	
	IX		0.3	0.4	-	
	X		2.9	3.0	0.6	
	XI		6.3	3.0	2.3	
	XII		-	0.3	-	
	Percent		100	100	100	
	Acres		30,954	16,160	1,755	
T 8 N	I		3.2	3.9	27.8	53.0
	II		1.9	2.3	0.1	4.1
	III		70.1	64.1	63.8	21.7
	IV		2.4	2.6	6.4	5.4
	V		0.0	0.6	2.3	5.0
	VI		-	0.0	0.3	0.6
	VII		-	-	0.0	0.0
	VIII		-	-	0.0	0.0
	IX		3.0	3.4	1.3	0.1
	X		3.4	3.3	2.0	0.2
	XI		14.7	10.0	0.1	2.7
	XII		0.3	0.0	0.3	3.6
	Percent		100	100	100	100
	Acres		15,734	22,910	22,773	15,491

Land cover class, dominant vegetation, and map surface area in acres, hectares, and percent

The total surface area shown is 1,640,627 acres (2,563 sq. mi.), or 663,952 hectares (6,640 sq. km.).

I Water.—Ocean surfaces, and lake and river surfaces larger than 1 acre. (Area measurement excludes water surface outside seaward limit of study area.) .... 101,355 a (41,018 ha) (6.2 pct).

II Pond/Sedge Tundra Complex; Aquatic Tundra; or shallow water.—Very wet tundra areas with ponds and/or emergent communities of *Carex* spp. or *Arctophila*; and up to 50% moist or wet tundra...16,964 a (6,865 ha) (1.0 pct).

III Wet Sedge Tundra.—Wet tundra with little standing water or with up to half of surface area water-covered or emergent vegetation, or coastal areas periodically covered with salt water ..... 260,057 a (105,244 ha) (15.8 pct).

IV Moist/Wet Sedge Tundra Complex; or Dry Prostrate Shrub, Forb Tundra (*Dryas* river Terraces).—Moist sedge tundra with up to 40% wet sedge tundra; or dense prostrate mat of *Dryas* on river terraces .... 270,565 a (109,496 ha) (16.5 pct).

V Moist Sedge, Prostrate Shrub Tundra; or Moist Sedge/Barren Tundra Complex (frost-scar tundra).—Better-drained areas on rolling terrain sometimes with tussocks; or sparsely vegetated frost-scar tundra .... 434,512 a (175,845 ha) (26.5 pct).

VI Moist Sedge Tussock, Dwarf Shrub Tundra.—Well-drained upland tussock tundra in foothills with high percentage of cottongrass tussocks and dwarf or prostrate shrubs .... 414,550 a (167,766 ha) (25.3 pct).

VII Moist Dwarf Shrub, Sedge Tussock Tundra; or Moist Sedge Tussock, Dwarf Shrub/Wet Dwarf Shrub Complex (water track complex).—Upland tundra with shrubs to 50 cm high; or upland tussock tundra with shrubs in water tracks... 51,148 a (20,699 ha) (3.2 pct).

VIII Shrub-Tundra.—South-facing slopes in foothills or subalpine, with willow, birch, alder to 2m; or dense shrubs in water tracks .... 3,142 a (1,272 ha) (0.2 pct).

IX Partially vegetated areas.—Diverse habitats including river bars, alpine tundra and moss mats with barren rock and talus, lichen-covered, sorted stone-nets and beach or mud flats .... 27,678 a (11,201 ha) (1.7 pct).

X Barren gravel or rock.—Bare light-colored river gravel, gravel and sand spits, alpine barrens (especially dolomite), and cultural barrens (road or runway), often with rich but sparse flora ..... 27,642 a (11,186 ha) (1.7 pct).

XI Wet gravel or mud.—Extensive barren mud in river deltas and wet or dark-colored gravel on beaches or river beds, or dark-colored barren rock in mountains .... 28,402 a (11,494 ha) (1.7 pct).

XII Ice.—River icings in the braided stream channels of most larger streams .... 4,612 a (1,866 ha) (0.3 pct).

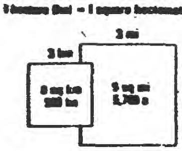
Introduction

This map is produced in conjunction with an environmental impact statement assembled by the U.S. Fish and Wildlife Service (USFWS) in anticipation of oil exploration on the Arctic National Wildlife Refuge coastal plain. Classification of vegetation and land cover is derived from digital multispectral data comprising Landsat scenes indexed in the margin. For location control and area measurement, land cover data are assigned to 50 x 50 m cells in UTM Zones 6 and 7, then merged into Zone 6 (extended) for map printing.

Vegetation classification and field review are by William Acevedo (Technicolor Graphic Services) and Donald Walker (Institute of Arctic and Alpine Research). Design of map and statistical products is by Leonard Gaydos and James R. Wray (USGS). Color separation and screening for four-color process printing are done on a large-format laser plotter using dot screens and angles developed on that equipment and plotted directly onto the separations as data are read from the digital file. The resulting colors replicate those otherwise achieved at USGS using open-window plate negatives, conventional mechanical screens, and the same process ink colors.

Area scales 1:250,000

1 ha = 2.47 a   1 a = 0.405 ha
1 hectare (ha) = 1 square hectometer



Distance scales 1:250,000

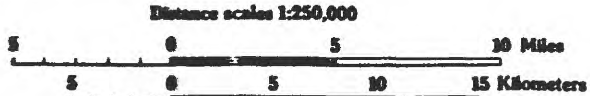


Figure 16. Vegetation and land cover Arctic National Wildlife Refuge Coastal Plain, Alaska (from Acevedo, W., Walker, D., Geydos, L., and Wray, J., 1982).

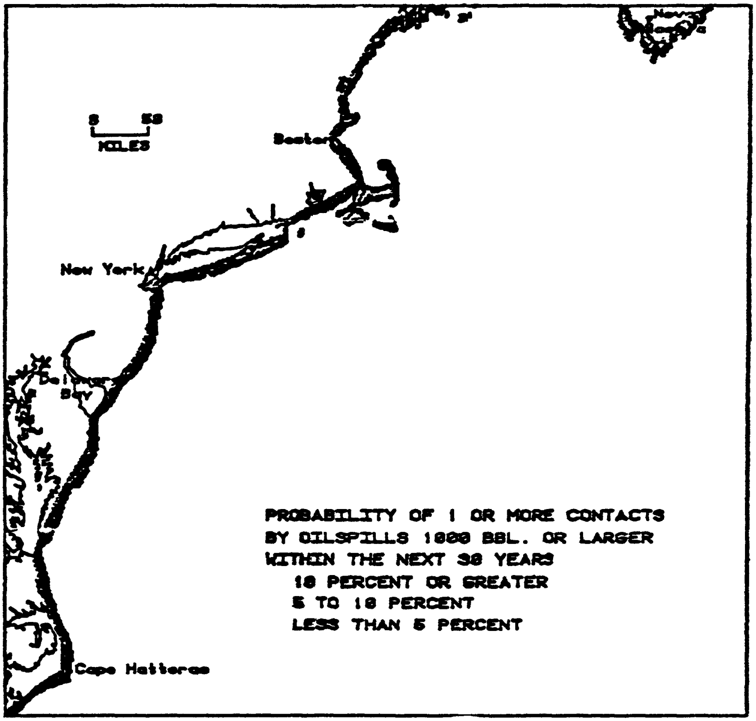
Offshore oil development with its attendant oilspill risks provided the need to estimate these risks by developing models capable of handling many dynamic factors such as fish and bird migration, winds, currents, tanker routes and the paths of spills, with a sample of 500 spills per season modeled in Monte Carlo fashion. Figure 17 is a map illustrating the probability of one or more contacts by oilspills 1,000 bbl. or larger within the next 30 years off the mid-Atlantic coast.

EXPLAINING AND CONVEYING THE HAZARD INFORMATION PRODUCTS TO THE POTENTIAL USERS

There are two aspects of effectively transferring hazard information products to the potential users. The first is to make sure that the potential user actually sees, hears or reads the product. The second aspect, often more difficult, is to convince the user that the hazard is real and potentially dangerous to both his property and perhaps to his life. In spite of repeated floods in the Mississippi Valley, or repeated eruptions at Mt. Aetna, many of the residents through some powerful pull to the land, accept the risk and refuse to leave.

The risks of an earthquake in the Anchorage, Alaska, area were detailed in a USGS publication (Miller and Dobrovolny, 1959) five years before the 1964 earthquake. In spite of its documented risks, expensive homes were built on top of the unstable Bootlegger Cove clay in "Turnagain by the Sea," the site of major damage. More publicity and warnings should have been issued, and there should have been actual restrictions placed on building loans in that area.

In the case of Mt. Saint Helens, tremendous publicity and effective information transfer had occurred prior to the May 18, 1980 eruption. The 60 lives lost belonged to those who refused to accept that the advertised risk was real. During this period the U.S. Geological Survey strengthened its capacity to anticipate and to assess volcanic hazardous events and to ensure that effective emergency response planning was done prior to the next such event. This was accomplished by the preparation of a "U.S. Geological Survey Interim Emergency Response Plan for Volcanic Hazards in the United States"



The U.S. Geological Survey has developed an oilspill risk analysis model to aid in estimating the environmental hazards of developing oil resources in Outer Continental Shelf lease areas (Smith and others, 1982). The large, computerized model analyzes the probability of spill occurrence, as well as the likely paths or trajectories of spills in relation to the locations of environmental resources which may be vulnerable.

The probability of spill occurrence is estimated from information on the anticipated level of oil production and method and route of transport. Spill movement is modeled in Monte Carlo fashion with a sample of 500 spills per season, each transported by monthly surface current vectors and wind velocities sampled from 3-hour wind transition matrices. Model output includes probability distributions for oilspills occurring and contacting resources such as the shoreline within preselected time limits.

The model provides the U.S. Department of the Interior with a method for realistically assessing oilspill risks associated with Outer Continental Shelf development, and for comparing them to existing risks such as importing oil. A recent analysis by Amstutz and others (1983) estimated oilspill risks to the Atlantic coast between Cape Hatteras and Nova Scotia. All tanker routes and all imports of oil over the next 30 years were considered, as well as anticipated oil and gas development. Figure shows the probabilities that specific parts of the shoreline will be contacted by one or more oilspills 1,000 barrels or larger within the next 30 years. The portions of the shoreline most at risk are those nearest busy shipping lanes and ports. This type of map would be useful for oilspill contingency planning and for studying alternatives to reduce risks considered unacceptable.

Figure 17. Oilspill probability map of the Mid-Atlantic Coast (from Amstutz et al., 1983).

(Krinsley, 1980). A similar plan was prepared for earthquake hazards (Krinsley, D. B., and others, 1981).

The USGS issues hazard warnings (Federal Register, 1977, 1984) when appropriate, to a communications network involving all organizations involved or responsible for the area affected. The USGS issues press releases and other forms of publicity for its publications, especially those that are deemed critical to all or parts of the population. Films and speakers are always available to educate the public concerning natural hazards, and long-term hazards are monitored by local or regional offices in constant communication with the civilian authorities. The USGS works very closely with all the state geological surveys and with the Federal Emergency Management Agency (FEMA) which is responsible for the avoidance or mitigation of natural disasters.

The earth scientist concerned with natural hazards has a responsibility to translate his data into language and graphics that are readily and effectively understood by planners, decisionmakers and the general public, in order to avoid or mitigate the effects of geologic and hydrologic hazards.

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EVALUATION

EVALUATION OF THE USGS GEOLOGIC AND HYDROLOGIC TRAINING PROGRAM

By

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University of Colorado
Boulder, Colorado

The Geologic and Hydrologic Training Program was a unique attempt to provide scientific information to emergency managers, scientists and government advisors throughout the world. The course was designed to develop hazard-abatement expertise necessary to save lives and reduce economic losses in countries where geologic and hydrologic hazards are prevalent. Individuals with emergency management responsibilities from 28 countries representing such diverse disciplines as hydrology, seismology, volcanology, landslides, and emergency management took part. The training program was intended to acquaint participants with techniques for evaluating and reducing the impacts of geologic and hydrologic hazards, and thereby enable them to develop a national hazard mitigation program in their own countries.

This evaluation was undertaken by the Natural Hazards Research and Applications Information Center to determine to what extent the month-long program met its stated objectives. Written evaluations submitted by participants at the conclusion of each of the program sections, and at the conclusion of the four-week program, make it possible to analyze what participants thought about the quality of presentations, materials, and expected utility of the information to which they were exposed. An evaluation of long-term objectives--dealing with actual application of new information and possible initiation of a hazard mitigation program--may be undertaken only after sufficient time has elapsed to allow participants to integrate the new knowledge into their government or research roles.

Forms were completed by participants following the seismic, landslide, volcano, hydrologic, and emergency management segments. In no case, however, did all of the participants complete the evaluation. Responses ranged from a high of 41 for the first session, dealing with seismic hazard, to a low of 21 responses for the hydrologic section. Thirty-three participants completed the landslide evaluation, 28 the volcanic hazard section, and 26 the emergency management and overall course evaluation. Where percentages are provided below, they are based upon the total responses for a particular section, and not upon the total number of course participants.

Evaluations

A review of all the evaluations shows that the program was well received. Participants spent four weeks listening to dozens of experts in fields touching on each of the major geologic and hydrologic hazards and emergency management practices. At the conclusion, respondents called it a success. Nearly 100 percent expressed interest in taking part in future similar training programs. All felt that similar courses would be valuable if given in their own countries, and when asked to evaluate the substantive content of the four-week program, 65 percent felt that their own geologic or hydrologic management concerns or research interests were adequately addressed. A space was provided on the form for participants to suggest changes, new materials and additional or different subjects for future courses. Many suggestions were put forward and will be discussed in some detail below.

Tables 1 through 5 provide numerical summaries of the responses for each of the program sections. Tables 6 through 10 contain responses as percentages. Responses were elicited on a five-point scale, 1 and 2 representing lowest level of agreement, 3 moderate agreement, and 4 and 5 highest agreement. A glance at the tables indicates that participants evaluated each section of the program favorably. Approximately 75 percent of the respondents gave each of the sections high marks for quality. The vast majority found the sections extremely useful in defining procedures for evaluating a particular hazard. Each increased their understanding of the physical causes of the hazard, and was successful in describing possible applications and recommending subjects for future research. More positive than those were responses to the question regarding how well the training program benefitted participants and their organizations. Even in countries where a particular hazard was not a problem, participants found that the program gave them new information, ideas and perspectives, and it established a much better understanding of how scientists, planners and engineers evaluated that hazard. This was particularly noticeable for the segment of the program dealing with volcano hazard (Tables 3 and 8). Although volcanoes are a problem for only a few countries, this portion of the program generated generally positive evaluations from all respondents.

Participants were asked to evaluate the content and style of the lectures. These, too, were regarded favorably. The weakest program component seems to have been the first, the seismic section (see Tables 1 and 6). This is the only part of the course to receive a number of low marks dealing with the presentations themselves. In evaluating the written materials, respondents were nearly unanimous in finding them extremely useful.

Where respondents offered a less positive evaluation of the program, often the reason put forward was a lack of knowledge or interest in a particular hazard because it simply was not a factor in the participant's country. This is reflected in the response to the volcano component (see Tables 3 and 8). This is the only section that received a significant number of negative comments to questions pertaining to participation in similar future programs. Approximately 20 percent of the respondents would decline to participate in similar programs should they be offered in the future because volcanoes are not present in their countries.

Certainly the most telling criteria by which to judge the success or failure of a training program is its ability not only to inform, but to influence behavior, in this case to bring about the incorporation of the new information into a hazard management program, or the development of a hazard mitigation program itself. Although it is too soon to evaluate the extent to which the new information will actually be incorporated into management practices, participants were asked to indicate their intended uses of the information. They were unanimous in stating their intentions to share the information with colleagues and generally to use it in their professional activities. A few, however, went beyond these general responses and specified plans to use new information to develop a program on natural hazards in Honduras; to establish a hazard mapping program in Nepal; to develop a microzonation plan in Chile; to incorporate information in the hydrologic hazard program in Paraguay; to use materials in television education in Taiwan; and to adopt criteria in the guidelines for preparation of the landslide hazard mapping program in Nepal.

It is desirable to determine to what extent participants will be successful in carrying out these and other applications stimulated by their participation in the training program. It is recommended here that an attempt be made to contact participants six months after the training program to get a better sense of the impact of the course on actual management and research activities.

Recommendations

At the conclusion of the course, participants were given an opportunity to recommend changes for future training programs. Although the majority of participants were satisfied with the level of information, a number of the physical scientists called for more technical information in their area of specialization. The most common requests were for more practical information, with more emphasis on application. Opportunities for field visits with their counterparts in government agencies or research institutes during the course to exchange information on a one-to-one basis were lacking. Since most of the participants, whether scientists or emergency planners, play advisory roles to members of their governments, it would have been useful for the information to have been more specific to management problems, with management techniques highlighted, and to have taken into account typically limited technical resources and paucity of highly trained personnel in many of the participating countries.

Other observations concerned the tendency for lecturers to provide examples based almost exclusively on U.S. experience. This often confounded application to problems in other countries. Further complicating this situation was the fact that most speakers seemed not to be available outside the classroom to meet informally with the students to discuss specific management problems.

Perhaps the most serious oversight was the failure to provide an opportunity for participants to share information concerning their management problems and programs with their fellow classmates. Participants perceived that lecturers seemed unaware of the nature of many of the problems outside the United States, and regretted the lack of opportunities to learn about innovative techniques employed successfully by their classmates in other countries.

Future Programs

Providing greater opportunities in future programs for participation by members of the class may be worth considering. It would help to broaden the understanding not only among other participants, but among the training staff of the nature and extent of problems facing those in various countries. It would serve to define the range of techniques currently being employed in other countries and the types of resources that can be brought to bear on pressing problems. It would not be inappropriate to require each participant to prepare a written statement prior to attending a similar course that summarized research problems or management issues in his or her country. If time for formal presentations were not available, at least these written statements could be given to participants and faculty at the beginning of the program.

Participants were asked to comment on the multi-hazard approach that was used. In spite of the fact that for most participants this meant sitting through presentations that were not directly relevant to their concerns, they overwhelmingly approved of this tactic for the first program. Nearly 100 percent responded positively to information on issues that were outside their own area of specialization. However, they noted their desire to participate in courses that would build upon this broad base, and provide more specific and technical information with a narrower focus. Many suggested that future meetings be defined along disciplinary or management lines. This would attract a more homogeneous audience and permit discussions of greater depth and sophistication.

Although materials provided to participants were well received, a number of suggestions were put forward urging that future materials be made available sufficiently prior to class presentation to enable those with English language difficulty to preview the presentation. Participants noted occasional frustration in their inability to obtain slides and other visuals used by the speakers; frequently, figures, charts and graphs were included in the oral presentation but not in materials that were distributed.

Conclusions

Based on responses provided during the course and at the conclusion of the program, it can be said that the training program was well received and successful in providing the participants with a great deal of new and potentially useful information. The distributed materials were judged to be extremely valuable. Before it can be determined whether the program stimulated participants to apply the new information, it will be necessary to do a follow-up evaluation. Participants should be contacted and invited to give examples of applications which could, in turn, be compiled and sent to other members of the class. This procedure could initiate an exchange of information between group members, serve to maintain linkages developed in Denver, and encourage the exchange of information not only between the U.S. and other countries, but among each of the participating countries as well.

An informal newsletter could be established to carry information through this world-wide network. Participants could be invited to provide examples of applications or management problems to OFDA and USGS for dissemination.

Consideration might be given to presenting a modified version of this training program in the future to an entirely new set of participants. This would broaden the potential hazard mitigation activities in an increasing number of countries. For those participants who have already been through the training program, future programs might be more individualized. Rather than developing specific, highly focused, technical training programs in each of the disciplinary areas, it might be useful to consider establishing field internships through which participants would spend time each year or two working side-by-side with counterparts in the U.S. or other countries in which innovative hazard mitigation activities were being applied.

TABLE 1
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
SEISMIC COMPONENT

March 5-12, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating seismic hazards?	—	8	32
b. increasing understanding of the physical causes of seismic hazards?	6	10	25
c. describing possible applications in your country?	1	10	30
d. recommending subjects for future research?	1	7	32
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?	—	6	35
b. establishing better understanding of how geologists, seismologists, planners, and engineers evaluate seismic hazards?	—	6	35
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?	—	4	37
b. well illustrated?	1	9	31
c. clearly presented?	1	5	35
d. reinforced by the handout material?	3	8	30
e. relevant to your particular needs?	3	11	27

4. Are the written course materials useful to you?............ Yes 40 No 0

5. In what ways could they be improved?

6. Are you interested in participating in future training programs similar to this?................................ Yes 37 No 2

7. Would a similar hazards training course be useful if given in your country/area of the world?.................. Yes 37 No 4

*Responses received from 41 participants

TABLE 2
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM LANDSLIDE COMPONENT

March13-16, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating landslide hazards?	—	1	32
b. increasing understanding of the physical causes of landslide hazards?	—	4	29
c. describing possible applications in your country?	—	7	25
d. recommending subjects for future research?	—	3	27
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?	—	—	33
b. establishing better understanding of how geologists, seismologists, planners, and engineers evaluate landslide hazards?	—	—	33
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?	—	—	33
b. well illustrated?	—	—	33
c. clearly presented?	—	—	31
d. reinforced by the handout material?	—	—	31
e. relevant to your particular needs?	—	—	25

4. Are the written course materials useful to you?............ Yes 33 No 0

5. In what ways could they be improved?

6. Are you interested in participating in future training programs similar to this?................................ Yes 32 No 1

7. Would a similar hazards training course be useful if given in your country/area of the world?.................. Yes 33 No 0

*Responses received from 33 participants

TABLE 3
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
VOLCANO COMPONENT

March 19-21, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating volcano hazards?....	—	—	28
b. increasing understanding of the physical causes of volcano hazards?....................................	—	—	28
c. describing possible applications in your country?......	4	6	14
d. recommending subjects for future research?............	2	2	21
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?....	—	—	28
b. establishing better understanding of how geologists, seismologists, planners, and engineers evaluate volcanic hazards?....................................	1	2	25
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?....................................	—	1	27
b. well illustrated?......................................	—	—	28
c. clearly presented?.....................................	—	—	28
d. reinforced by the handout material?...................	—	2	26
e. relevant to your particular needs?.....................	2	3	20

4. Are the written course materials useful to you?............ Yes 27 No 1

5. In what ways could they be improved? ______________________________

6. Are you interested in participating in future training programs similar to this?.................................... Yes 23 No 5

7. Would a similar hazards training course be useful if given in your country/area of the world?.................. Yes 21 No 6

*Responses received from 28 participants

TABLE 4
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
HYDROLOGIC HAZARD COMPONENT

March 21-27, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating hydrologic hazards?	—	—	21
b. increasing understanding of the physical causes of hydrologic hazards?	—	4	17
c. describing possible applications in your country?	—	6	15
d. recommending subjects for future research?	—	5	16
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?	—	1	20
b. establishing better understanding of how geologists, hydrologists, planners, and engineers evaluate hydrologic hazards?	—	1	20
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?	—	1	20
b. well illustrated?	—	2	19
c. clearly presented?	—	2	19
d. reinforced by the handout material?	—	2	19
e. relevant to your particular needs?	1	6	14

4. Are the written course materials useful to you?............ Yes 21 No 0

5. In what ways could they be improved?

6. Are you interested in participating in future training programs similar to this?.................................. Yes 21 No 0

7. Would a similar hazards training course be useful if given in your country/area of the world?.................. Yes 21 No 0

*Responses received from 21 participants.

TABLE 5
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
EMERGENCY PREPAREDNESS COMPONENT

March 28-30 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. providing information on individual and family response to disasters?................................	2	7	17
b. increasing your understanding of flood plain management in the U.S.?...	—	4	22
c. increasing your knowledge of flood management techniques?...	1	6	19
d. increasing your knowledge of earthquake preparedness planning?...	1	6	18
e. describing possible applications in your country?......	1	5	19
f. recommending subjects for future research?............	1	4	21
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas and perspectives?.....	—	3	23
b. establishing better understanding of how emergency managers and planners develop hazard mitigation programs?...	—	5	21
3. Did you find the presentations by the speakers to be:			
a. factual and useful?..................................	—	2	24
b. well illustrated?....................................	—	2	24
c. clearly presented?....................................	—	2	24
d. reinforced by the handout material?..................	—	5	21
e. relevant to your particular needs?.....................	1	13	12

4. Are the written course materials dealing with emergency preparedness useful to you?................................ Yes 26 No 0

5. In what ways could these materials be improved? ____________________

*Responses received from 26 participants.

TABLE 6
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
SEISMIC COMPONENT

Responses as Percentages

March 5-12, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating seismic hazards?	—	20%	78%
b. increasing understanding of the physical causes of seismic hazards?	15%	24%	61%
c. describing possible applications in your country?	2%	24%	73%
d. recommending subjects for future research?	2%	17%	78%
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?	—	15%	85%
b. establishing better understanding of how geologists, seismologists, planners, and engineers evaluate seismic hazards?	—	15%	85%
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?	—	10%	90%
b. well illustrated?	2%	21%	76%
c. clearly presented?	2%	12%	85%
d. reinforced by the handout material?	7%	20%	73%
e. relevant to your particular needs?	7%	27%	66%

4. Are the written course materials useful to you?............ Yes 100% No 0

5. In what ways could they be improved?____________________

6. Are you interested in participating in future training programs similar to this?.................................. Yes 90% No 5%

7. Would a similar hazards training course be useful if given in your country/area of the world?.................... Yes 90% No 10%

*Responses received from 41 participants

TABLE 7
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
LANDSLIDE COMPONENT

Responses as Percentages

March 13-16, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating landslide hazards?..	—	3%	96%
b. increasing understanding of the physical causes of landslide hazards?....................................	—	12%	88%
c. describing possible applications in your country?......	—	21%	77%
d. recommending subjects for future research?............	—	9%	82%
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?....	—	—	100%
b. establishing better understanding of how geologists, seismologists, planners, and engineers evaluate landslide hazards?....................................	—	—	100%
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?....................................	—	—	100%
b. well illustrated?......................................	—	—	100%
c. clearly presented?.....................................	—	6%	94%
d. reinforced by the handout material?....................	—	3%	94%
e. relevant to your particular needs?.....................	—	15%	77%

4. Are the written course materials useful to you?........... Yes 100% No 0

5. In what ways could they be improved? ______

6. Are you interested in participating in future training programs similar to this?.................................. Yes 97% No 0

7. Would a similar hazards training course be useful if given in your country/area of the world?.................. Yes 97% No 0

*Responses received from 33 participants

TABLE 8
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
VOLCANO COMPONENT

Responsesas Percentages

March 19-21, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating volcano hazards?	—	—	100%
b. increasing understanding of the physical causes of volcano hazards?	—	—	100%
c. describing possible applications in your country?	14%	21%	50%
d. recommending subjects for future research?	7%	7%	75%
2 Did the training program benefit you or your organization by:			
a. providing new information, ideas, and perspectives?	—	—	100%
b. establishing better understanding of how geologists, seismologists, planners, and engineers evaluate volcanic hazards?	3.5%	7%	80%
3. Overall, did you find the presentation by the speakers to be:			
a. factual and useful?	—	3.5%	96%
b. well illustrated?	—	—	100%
c. clearly presented?	—	—	100%
d. reinforced by the handout material?	—	7%	93%
e. relevant to your particular needs?	7%	11%	71%

4. Are the written course materials useful to you?............ Yes96% No3.5%

5. In what ways could they be improved? ______________________________

6. Are you interested in participating in future training programs similar to this?................................ Yes82% No 18%

7. Would a similar hazards training course be useful if given in your country/area of the world?................... Yes75% No21%

*Responses received from 28 participants

TABLE 9
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
HYDROLOGIC HAZARDS

Responses as Percentages

March 21-27, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. defining procedures for evaluating hydrologic hazards?	—	—	100%
b. increasing understanding of the physical causes of hydrologic hazards?	—	19%	80%
c. describing possible applications in your country?	—	29%	71%
d. recommending subjects for future research?	—	24%	76%
2. Did the training program benefit you or your organization by			
a. providing new information, ideas, and perspectives?	—	5%	95%
b. establishing better understanding of how geologists, hydrologists, planners, and engineers evaluate hydrologic hazards?	—	5%	95%
3. Overall, did you find the presentation by the speakers to be			
a factual and useful?	—	5%	95%
b. well illustrated?	—	10%	90%
c. clearly presented?	—	10%	90%
d. reinforced by the handout material?	—	10%	90%
e. relevant to your particular needs?	5%	29%	66%

4. Are the written course materials useful to you?............ Yes100% No 0

5 In what ways could they be improved?

6. Are you interested in participating in future training programs similar to this?.................................. Yes100%No 0

7. Would a similar hazards training course be useful if given in your country/area of the world?.................. Yes100 No 0

*Responses received from 21 participants.

TABLE 10
EVALUATION

GEOLOGIC AND HYDROLOGIC HAZARDS TRAINING PROGRAM
EMERGENCY PREPAREDNESS

Responses as Percentages

March 28-30, 1984
Denver, Colorado

Please circle the number that best represents your response to the following questions. Number 5 represents the highest rating and number 1, the lowest. Your answers will help us evaluate this training program.

	Low	Medium	High *
1. Did you find the training program useful for:			
a. providing information on individual and family response to disasters?	8%	27%	65%
b. increasing your understanding of flood plain management in the U.S.?	—	15%	85%
c. increasing your knowledge of flood management techniques?	4%	23%	73%
d. increasing your knowledge of earthquake preparedness planning?	4%	23%	69%
e. describing possible applications in your country?	4%	19%	73%
f. recommending subjects for future research?	4%	15%	81%
2. Did the training program benefit you or your organization by:			
a. providing new information, ideas and perspectives?	—	11%	88%
b. establishing better understanding of how emergency managers and planners develop hazard mitigation programs?	—	19%	81%
3 Did you find the presentations by the speakers to be:			
a. factual and useful?	—	8%	92%
b. well illustrated?	—	8%	92%
c. clearly presented?	—	8%	92%
d. reinforced by the handout material?	—	19%	81%
e. relevant to your particular needs?	4%	50%	46%

4. Are the written course materials dealing with emergency preparedness useful to you?................................ Yes 100% No 0

5. In what ways could these materials be improved? ____________________

*Responses received from 26 participants.