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Preface

Organization of and Background for the Workshop

Under the National Earthquake Hazard Reduction Program (NEHRP) established in 1977, the Federal Emergency Management Agency (FEMA), United States Geological Survey (USGS), the National Science Foundation, and the National Institute for Standards and Technology (NIST) are charged with developing methods to reduce loss of life and property damage from earthquakes both through national programs and through the support of and cooperation with state and local programs. Since 1985, FEMA and the USGS have concentrated their efforts in the Puget Sound and Portland areas of Western Washington and Oregon in a cooperative effort with State agencies (Washington Department of Natural Resources, Division of Geology and Earth Resources, (DGER); Washington Division of Emergency Management, (DEM); the Oregon Department of Geology and Mineral Industries (DOGAMI); the Oregon Emergency Management Division (EMD); and the private sector.

USGS efforts to date have primarily concentrated on assessment of the earthquake hazard by scientists within the USGS and by other scientists and engineers supported by NEHRP. USGS has also funded hazard vulnerability studies of lifeline systems in Seattle, Washington and Portland, Oregon and a study of the implementation of earthquake hazard policies in Washington and Oregon. FEMA provides funding to the State emergency management agencies to support the development of continuing state mitigation and preparedness programs, delivers earthquake hazards reduction workshops to specific target audiences (for instance, hospitals), distributes general information on how to reduce earthquake hazards and protect life safety, and carries out research on earthquake hazard reduction strategies.

Annual workshops, like the one held in Seattle on April 17-19, 1990 are an attempt to communicate the information collected by the above agencies and individuals to a larger audience that will use it appropriately to prepare for future large earthquakes in the Pacific Northwest. The 1990 workshop was targeted to the design community of engineers, architects, and planners. Regional seismological and geotechnical considerations important in earthquake resistant design for buildings and lifelines were summarized. Techniques to improve the earthquake resistance of existing buildings were presented followed by a field trip to observe local examples of seismically retrofit buildings. Model earthquake hazard reduction programs in selected lifeline systems in Washington, Oregon, and British Columbia were reviewed.

Unlike previous workshops, which were sponsored by the USGS under contract with FEMA, the 1990 workshop was sponsored by DGER with a grant from FEMA. As in previous years, the meeting was planned by a local steering committee. Committee members were:

Washington

Ray Lasmanis	DGER
Tim Walsh	DGER
Steve Palmer	DGER
Josh Logan	DGER
Carol Martens	DEM
Todd Perbix	Ratti, Swenson, Perbix, Clark
Peter May	University of Washington

Oregon

George Priest	DOGAMI
Ian Madin	DOGAMI

Federal Government

Linda Noson	FEMA
Craig Weaver	USGS
Tom Yelin	USGS
Bill Kockelman	USGS

The steering committee planned the agenda and selected session chairs to arrange for speakers and poster presenters. The session chairs were Stew Smith (University of Washington), Tony Qamar (University of Washington), Todd Perbix (Ratti, Swenson, Perbix, Clark), Don Ballantyne (Kennedy/Jenks/Chilton), Bruce Olsen (Consulting engineer), and Tim Walsh (DGER). Administrative and clerical support by Michelle Davis and Mary Ann Shawver of DGER contributed substantially to the success of the workshop.

This workshop represents the effort to synthesize the wealth of new data gathered under the aegis of the NEHRP program for the Puget Sound and Portland areas and to translate it into engineering practice so as to reduce the risk from future earthquakes in the region. Attendance by more than 300 scientists, engineers, emergency planners, and others attests to the strong interest in understanding regional earthquake hazards and reducing future personal and property losses.

The editors

Fourth Annual Workshop
National Earthquake Hazards Reduction Program
Puget Sound and Portland Area

April 17, 1990

0730 Registration
0815 Welcome addresses: Brian Boyle, Washington State Commissioner of Public Lands; Gary Johnson, Chief of Earthquakes and Natural Hazards Programs Division, FEMA; Ray Williams, Region X Director, FEMA; Craig Weaver, U.S. Geological Survey

Tectonic Framework of the Pacific Northwest Session
Stewart Smith, Chairman

0900 Tectonic overview of the Pacific Northwest: Robert Crosson, University of Washington
0930 Evidence for prehistoric earthquakes in the Pacific Northwest: Curt Petersen, Portland State University
1000 Coffee break
1015 Great subduction zone earthquakes in the Pacific Northwest, fact or fiction: Thomas Heaton, U.S. Geological Survey
1045 Shallow crustal earthquakes (Loma Prieta in our backyard?): Craig Weaver, U.S. Geological Survey
1115 Summary and panel discussion: Stewart W. Smith, University of Washington
1200 Luncheon - Lessons learned for NEHRP from the Loma Prieta Earthquake: Richard Eisner, Director-Bay Area Regional Earthquake Preparedness Project of the Governor's Office of Emergency Services

Earthquake Site Effects Session
Anthony Qamar, Chairman

1330 Site-specific earthquake strong ground motion studies in the Puget Sound and Portland, Oregon areas: Ivan Wong, Woodward-Clyde Consultants
1355 Liquefaction: Stephen Palmer, Washington Division of Geology and Earth Resources
1420 Influence of local geology on amplification/attenuation of seismic shaking: Ralph Archuleta, University of California, Santa Barbara
1445 Coffee break
1455 Probabilistic ground motion model in the Pacific Northwest: S. T. Algermissen, U.S. Geological Survey
1520 Dam safety considerations: Jerald LaVassar, Washington Department of Ecology
1545 Earthquake-induced landslides: Derek Cornforth, Landslide Technology
1610 Seismic philosophy of the Uniform Building Code: John Hooper, Ratti, Swenson, Perbix, & Clark
1635 Surface geology vs. seismic intensity of earthquakes in Washington: Thomas Bodle, University of Washington
1800 No-host bar and complimentary hors d'oeuvres
1830 Poster session

Fourth Annual Workshop National Earthquake Hazards Reduction Program Puget Sound and Portland Area

April 18, 1990

Structural Engineering Session
Todd Perbix, Chairman

0815 Introduction to earthquake engineering of buildings: Todd Perbix, Ratti, Swenson, Perbix, & Clark
0845 Retrofit building types/inventory: Bruce Olsen, consulting engineer
0915 Case study-Heritage Building: NBBJ, Architect; Ratti, Swenson, Perbix, & Clark, Engineer; Thomas Kinsman, Building official
1000 Coffee break
1015 Case History-Union Station: Ratti, Swenson, Perbix, & Clark, Engineer; Thomas Kinsman, Building official
1100 Break for lunch (on your own)
1230 Introduction to the ATC-21 checklist: Todd Perbix, Ratti, Swenson, Perbix, & Clark; Field trip to Heritage Building and Union Station (limited space)
1500 Discussion of field trip
1530 Coffee break
1545 Case history-Franklin High School: Bassetti-Norton-Rekevic, Architect; Mahan & DeSalvo, Engineer; Thomas Kinsman, Building official
1630 Close

Fourth Annual Workshop
National Earthquake Hazards Reduction Program
Puget Sound and Portland Area

April 19, 1990

Lifelines Session
Don Ballantyne, Chairman

0830 Introduction and welcome to lifelines session

0840 Lifeline damage in the Loma Prieta earthquake-slide show with a newscast type summary: Keith Eldridge, KOMO news

0855 Overview of lifeline earthquake engineering: Donald Ballantyne, Kennedy/Jenks/Chilton

0910 Panel discussion

Introduction: Donald Ballantyne, Kennedy/Jenks/Chilton

- Walter Anton, Seattle Water Department
- Allan Walley, Washington Department of Transportation
- J. D. Cattanach, B. C. Hydro
- William Elliott, Portland Water Bureau
- Ken Sullivan, FEMA

The panel will discuss the following four topics:

1) Marketing an earthquake mitigation program to decision makers who control the lifeline's budget
2) Assessing the vulnerabilities of a lifeline system
3) Estimating potential losses to a lifeline system
4) Plans to reduce losses to lifeline systems

0945 Coffee break

1000 Panel (continued)

1130 Panel closing statements

1145 Luncheon- Volcano monitoring and hazards in the Pacific Northwest: Edward Wolfe, U.S. Geological Survey; Scientist-in-charge, Cascades Volcano Observatory

Loss Estimation Session
Bruce Olsen, Chairman

1330 Evaluating the potential extent of earthquake damage: Peter May, University of Washington

1345 Damageability of buildings due to poor soil conditions: W. Paul Grant, Shannon and Wilson

1400 Architectural considerations in the evaluation of potential earthquake losses: Chris Arnold, Building Systems Development

1415 Coffee break

1430 Seismic design and loss estimation in areas of low historical seismicity: Roger McGarrigle, Van Domelen, Looijenga, McGarrigle, and Knauf

1445 Loss estimation vis a vis the insurance industry: Craig Taylor, Dames and Moore

1500 Overview of loss estimation: Bruce Olsen, consulting engineer

1515 Panel; questions from floor

1630 Closing remarks: Ray Lasmanis and Timothy Walsh, Washington Department of Natural Resources, Division of Geology and Earth Resources

Highlights of the Fourth Annual NEHRP Workshop for Puget Sound and Portland Areas*

by

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Division of Geology and Earth Resources
Olympia, Washington 98504

The 1990 National Earthquake Hazards Reduction Program (NEHRP) workshop, sponsored by the Washington Division of Geology and Earth Resources (DGER), Washington Division of Emergency Management, Federal Emergency Management Agency (FEMA), and the American Society of Civil Engineering Technical Council on Lifeline Engineering, was held April 17-19 in Seattle. The workshop consisted of two field trips, 29 oral presentations, two panel discussions, and a poster session. The sessions integrated wide-ranging topics relating to earthquake studies, from seismologic research to sociological studies. Welcoming addresses were delivered by **Brian Boyle**, Washington Commissioner of Public Lands, **Gary Johnson**, Chief of Earthquakes and Natural Hazards Programs Division of FEMA, **Ray Williams**, Region X Director of FEMA, and **Craig Weaver** of the U.S. Geological Survey (USGS).

Keynote speaker **Brian Boyle** noted that earthquake policy implementation was difficult because the issue generates little pressure on political leaders. He questioned the preparedness of Washington, pointing out that the State has no direct expenditure for earthquake readiness—unlike California, which spends more than $3 million annually for geologic studies alone. At the same time, the need for preparedness has been demonstrated: 200,000 children attend schools the Superintendent of Public Instruction thinks would not survive a major earthquake; 65 bridges and freeway ramps are obsolete; and numerous other buildings, such as hospitals, prisons, nursing homes, and office buildings, need seismic evaluation. Boyle further noted that California has 75 state-supported strong motion accelerometers; Washington has only a handful. These instruments provide detailed information about seismic motion, and Boyle suggests this kind of information should be available to every building designer in the state. He stressed the importance of added training and organizational work to coordinate and improve readiness of vital communication links throughout the state.

Tectonic Framework of the Pacific Northwest

The first morning of the workshop opened with a session concerning the tectonic framework of the Pacific Northwest. **Robert Crosson** of the University of Washington provided an overview of seismicity in the Pacific Northwest as it relates to the structure and geometry of local tectonic plates. He suggested that the angle of the subducting plate (deviation from the horizontal) may have profound effects on

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* modified from Washington Geologic Newsletter, 1990, V. 18, no. 3, p. 14-18.

the nature of uplift and on the distribution of earthquakes in the Pacific Northwest. He suggested that the uplift of the Olympic Mountains may be associated with the shallow dip (10° to the east) of the Juan de Fuca plate as it is subducting beneath the North America plate. The low angle is the result of a flexure or "arching" of the slab beneath western Washington noted by previous researchers. (See Weaver and Baker, 1988.) This geometry offers one explanation for the development of the Olympic Mountains. Because the previous large intraplate earthquakes of 1949 and 1965 were located in or on the periphery of arched portion of the slab, other areas overlying arched portions of a subducting slab could be similarly vulnerable to large intraplate earthquakes.

Curt Petersen of Portland State University summarized observations of interlayered deposits of peat and intertidal mud that serve as proxy indicators of tectonic subsidence associated with subduction zone earthquakes in Oregon and Washington. He reviewed his own research, the pioneering work of Brian Atwater, Wendy Grant, Gary Carver and others, stratigraphic evidence for earthquake-generated tsunamis noted by Mary Reinhart and Joanne Bourgeois, as well as archeological research, recent investigations of turbidites near Vancouver Island by John Adams, and geodetic evidence of strain noted by Mike Lisowski and Herb Dragert (using the Global Positioning Satellite to measure across the Strait of Juan de Fuca) and by Paul Vincent (who used first-order levelling near Tillamook, Ore.). Petersen's presentation included an updated compilation of radiocarbon ages associated with the stratigraphic evidence that suggest apparently contemporaneous subsidence in Washington and Oregon.

Thomas Heaton, Scientist in Charge of the USGS seismological laboratory in Pasadena, compared subduction zones and their respective earthquake types in the Pacific Rim to the inferred configuration of the subduction zone in the Pacific Northwest. In particular, he summarized the work of Ruff and Kanamori (1980), who sought correlations between the maximum observed earthquake on a subduction zone interface and various geological and geophysical characteristics of the zone. (See also Heaton and Hartzell, 1987.) Characteristics that can be used in comparing various subduction zones include plate collision velocity, age of the subducting slab, and the presence of back-arc spreading. Using these characteristics, Heaton suggested that the subduction zones in Japan, Colombia, and Mexico are those most similar to the Cascadia subduction zone. Of these, only the Cascadia subduction zone has not experienced moment magnitude (M_w) 8+ subduction zone interface earthquakes in the 20th century. Heaton also discussed the scale and nature of ground motion and response spectra typically associated with great subduction zone earthquakes. These earthquakes cause very strong shaking over a long period (often greater than 2 min). He also noted that ground motions from subduction zone earthquakes seem to persist over longer distances than those from strike-slip earthquakes of a similar magnitude.

Craig Weaver, USGS seismologist, focused his discussion on shallow crustal earthquakes. The occurrence of a damaging magnitude 5.0 crustal earthquake in the Deming area (northwestern Washington) the week before the NEHRP meeting made his choice of topic all the more relevant. He summarized the historic record of shallow-crustal seismicity in the Pacific Northwest (or lack of it in some areas) and related the foci of those earthquakes to geologic structures at depth. Drawing on his previous work, he related the locations and configuration of the northwest-trending St. Helens Seismic Zone (SHZ) and a similarly trending fault zone west of Mount Rainier to the edge of an inferred underlying crustal block at 12 km depth. He speculated that a magnitude 6.3-6.8 earthquake could occur on the SHZ between Elk Lake and Spirit Lake, depending on whether a 7- or 12-km segment of this strike-slip fault were to rupture. Such an earthquake could have significant impact on the Portland area. (See Weaver and Shedlock, 1989.)

Following Weaver's presentation, **Stewart W. Smith** of the University of Washington, who chaired the opening session, led a panel discussion of the participants.

Richard Eisner, Director of the Bay Area Regional Earthquake Preparedness Project (Governor's Office of Emergency Services, California), delivered a luncheon slide presentation showing some of the damage caused by the October 17 Loma Prieta earthquake. He noted that the California Department of Transportation (Caltrans) had spent more than $50 million on seismic retrofitting (phase 1) before the Loma Prieta earthquake, but that $300 million had been required for clean up to date (not including replacement of damaged or destroyed structures). The money invested in phase 1, specifically for the installation of joint restrainers, apparently prevented many bridge spans from collapsing, according to testimony submitted by Caltrans to the Governor's Board of Inquiry to the Loma Prieta earthquake. However, in the case of the ill-fated Cypress structure, the joint restrainers were not enough to prevent collapse, and column retrofitting would have been necessary to mitigate this hazard because column abutments were so far apart. (See Thiel and others, 1990.)

Ivan Wong of Woodward-Clyde Consultants discussed the role of site-specific and regional effects in strong ground shaking. He summarized recent methodologies for predicting strong ground motions and applied them to modeling the spectral response at sites in the Puget Sound and Portland areas for earthquakes of various magnitudes and epicentral distances. These simulation methods are particularly important because of the scarcity of strong-motion data in the region.

Stephen Palmer of DGER reviewed the phenomena of seismically induced liquefaction and ground settlement, using examples from the 1949 and 1965 Puget Sound earthquakes. Liquefaction studies presently being performed by the USGS and other NEHRP-funded researchers in the

Pacific Northwest were described.

Ralph Archuleta of the University of California at Santa Barbara stressed the importance of impedance contrasts and resonance peaks in analysis of amplification of strong ground motion. He summarized work at McGee Creek, Calif., where he and Sandra Seale measured the influences of local geology (glacial drift overlying hornfelsed bedrock) on amplification of seismic waves. Major amplification effects at the surface of their test site were caused by resonance effects brought about by the impedance contrast between soil and bedrock.

S. T. Algermissen of the USGS reviewed the methodology used to generate probabilistic earthquake ground-motion maps. Such maps are often used by engineers to estimate seismic lateral forces during design of buildings, dams, bridges, and other major structures. These maps are revised to "custom-fit" the various parameters used in these analyses to the regional tectonic and seismologic history.

Jerald LaVassar, an engineer with the Dam Safety Section of the Washington Department of Ecology, discussed some of the shortcomings of the probabilistic approach to estimating earthquake ground motions. A major problem in dam safety evaluation is determining the liquefaction potential of older earth-filled dams. Liquefaction depends on both the level of ground acceleration and the duration of strong shaking. Probabilistic acceleration maps provide only an estimate of the maximum probable acceleration and give no indication of the expected duration of shaking. As a rule, larger earthquakes are accompanied by strong shaking of longer duration, which results in a greater potential for liquefaction. Also, it appears that many probabilistic acceleration maps may overemphasize the contribution of earthquakes of smaller magnitude, especially maps for events of long return periods.

Derek Cornforth of Landslide Technology in Portland, Ore., provided an overview of mass movements related to earthquakes. He discussed three broad categories of mass failures: movements from marginally stable slopes, block-slide movements, and movements resulting from liquefaction. Cornforth stressed that liquefaction-induced movements are the most important group of earthquake-induced landslides because they are the most common and cause the most damage to engineered structures.

John Hooper of Ratti, Swenson, Perbix, and Clark in Seattle discussed the philosophy of seismic design of the Uniform Building Code (UBC), including the intent and limitations of its provisions. The basic design philosophy is to insure the safety of the inhabitants. Hooper noted that minimum standards have been set to safeguard structures against major failures and loss of life due to ground shaking. However, no UBC provisions have been made for earth conditions other than basic soil types that are generally not genetically related to geologic parent material. These soil types are used only in characterizing the ground shaking and do not take into account loss of strength of the soil during the shaking. In light of the significance of liquefaction in causing damage to engineered structures, Hooper believes this is an area where improvements to the UBC could be made.

Addressing the importance of geologic parent material, **Thomas Bodle** (University of Washington) related Seattle-area intensities of the 1965 and 1981 Puget Sound earthquakes and 1981 Elk Lake event to surficial geology and elevation. He found significant association between intensity and postglacial surficial deposits at elevations less than 100 ft, and between intensities and all surficial units except till for elevations between 0 and 200 ft. Previous studies had plotted intensities against USDA soils maps; many mapped soil units are only incidentally related to surficial geology.

Evening Poster Session

William J. Perkins and **W. Paul Grant** (Shannon and Wilson, Inc., Seattle) and **T. Leslie Youd** (Brigham Young University) displayed their maps of liquefaction susceptibility for the Seattle North and Seattle South quadrangles. The maps were prepared using estimated thicknesses of units determined to be liquefiable by standard penetration test data.

Mark Holmes presented single-channel seismic reflection data recorded in Elliott Bay by the Department of Oceanography of the University of Washington. These data show that slumping and turbidity flows are the two primary mass-wasting phenomena controlling sediment movement in this area of Puget Sound. These two processes have severely modified the spoils pile deposited in Elliott Bay during Seattle's Denny regrade projects. Also, seismic reflection data clearly show an underwater pipeline that had been uncovered during slumping of overlying mud and evidence of turbidity flows in the deeper channels of Puget Sound.

Stephen P. Palmer and **John A. Shulene** presented their work on liquefaction in the Puyallup valley caused by the 1949 and 1965 Puget Sound earthquakes. Although the 1949 magnitude 7.1 earthquake caused widespread liquefaction in Puyallup, the 1965 magnitude 6.5 earthquake did not produce ground acceleration and duration of strong shaking sufficient to trigger liquefaction.

Ian Madin of the Oregon Department of Geology and Mineral Industries and **Silvio Pezzopane** of the University of Oregon summarized results of recent mapping in which investigators have identified fault offsets in Pliocene-Pleistocene Boring Lavas near Portland. Madin and Pezzopane also documented paleoliquefaction features (clastic dikes and sand blows) in silts and fine sands deposited in the Portland area by catastrophic Pleistocene floods from glacial Lake Missoula. Strong ground shaking during local or distant earthquakes and rapid loading during catastrophic floods provide possible explanations for these features.

Earthquake-hazard maps of the Portland area were also displayed by **Ian Madin**, who used geologic mapping, geotechnical boring, and water-well data to compile the maps. These maps show the distribution of a variety of liquefiable sediments, as well as the locations of numerous faults.

Mark Darienzo and **Curt Petersen** of the Geology Department at Portland State University showed evidence for small-scale (>>1 m) tectonic subsidence in Alsea Bay, Ore., associated with Cascadia subduction zone earthquakes. This subsidence is less than the 1-2 m of sudden subsidence observed in estuaries of northern Oregon and southern Washington. The smaller subsidence could be attributed to Alsea Bay being farther from the axis of subsidence or to separate, smaller magnitude subduction zone earthquakes. The first hypothesis is supported by the position of an offshore fold belt that is seaward of and parallels the subducting trench. Since this fold belt comes onshore near Coos Bay, Darienzo and Peterson suggest evidence of uplift may be found between Coos Bay and Alsea Bay. The latter explanation might imply segmentation of the Cascadia Subduction Zone.

Brian Cohee, Paul Sommerville, and **Norman Abrahamson** of Woodward-Clyde Consultants presented the results of their computer simulation of ground motions from Cascadia subduction zone earthquakes. They computed the ground motion for a hypothesized M_w=8 thrust earthquake on rock and soil sites in the Puget Sound and Portland regions. These computations show that for periods less than 1 sec, the estimated spectral velocities would be as much as twice those recorded during the 1949 Olympia and 1965 Seattle earthquakes, and that the duration of strong shaking would be significantly longer (40-60 sec versus 10-20 sec).

Tammi Baier of the Department of Geography and Regional Planning at Western Washington University summarized her study of the public's attitudes toward and response to earthquake hazards in the Tacoma and Puyallup areas of Washington. Because more than half the replies to her questionnaire were returned on or before the October 17, 1989, Loma Prieta earthquake, she has been able to compare those attitudes with later replies which may have been influenced by the earthquake. Although her statistical analysis of the data is not yet finished, Baier observed that perceptions of dread were noticeably more common among those whose replies were postmarked after the Loma Prieta earthquake. A greater percentage of the post-Loma Prieta replies also indicated persons intended to take more precautions about earthquake hazards.

Harry Halverson, retired vice president and co-founder of Kinemetrics (manufacturer of seismometers), provided a photographic display of Namazu-e, colorful woodblock prints reflecting Japanese folklore that a great subterranean catfish (Namazu) produced earthquakes. The creation of the prints was a result of the Edo

(Tokyo) earthquake of 1855, although the prints contain references to other earthquakes. The Namazu-e reflect conflicting ideas about earthquakes that existed at that time. Earthquakes were seen as punishment of various classes of people in some instances, and at other times, the Namazu were thought to assist carpenters and other laborers who might benefit from the effects of an earthquake. (See Bolt, 1976.)

Structural Engineering Session

For geologists, one of the most rewarding aspects of the NEHRP workshop was the opportunity to interact with engineers, architects, and planners. This session demonstrated that new information on the nature and magnitude of anticipated forces generated by earthquakes must be accounted for in structural design and retrofitting older buildings.

Todd Perbix of Ratti, Swenson, Perbix, and Clark provided an introduction to earthquake engineering of buildings, including a history of design code applicability in the Seattle area. He summarized the typical problem areas: gable failures, parapet failures, and general problems with unreinforced masonry. He showed various examples of anchorage mechanisms and techniques for securing parapets and the installation of ductile frames and diaphragms to allow increased redundant transfer of shear forces.

John Hooper (Ratti, Swenson, Perbix, and Clark) talked about retrofitting the Heritage Building in Pioneer Square. (See field trip below.) This building has a long-aspect ratio problem and had a "soft story" (a lower, open floor not able to accommodate shear forces). The soft story was corrected by adding a center brick shear wall to resist north-south lateral forces, anchoring of the wood floor structure to this wall, and devising drag strut connections. Anchorage rosettes were used to secure brick and sandstone faces of the building. Elsewhere, beam and column tie plates were used, and additional concrete shear walls were constructed.

Thomas Kinsman, building official for the City of Seattle, discussed the nature of the retrofit problem in the Seattle area. He noted that each building is unique, that there are no "cookbook" methods for dealing with retrofitting. Retrofitting regulations are enforced when a hazard is observed, a building is being substantially renovated, there is a change in occupancy, or the building is re-occupied after more than a year of vacancy. A case history of the Franklin High School in South Seattle was presented by **John Desalvo** of Mahan and Desalvo to demonstrate how a local landmark was preserved and expanded as it was upgraded to meet current seismic safety standards. Major shear wall structures in the original building were strengthened by the application of shotcrete veneer and by the addition of steel ties at the roof and floor levels. Additional lateral strength was supplied by the new addition to the original structure.

Field Trips

The day before the meeting's opening, **Robert Bucknam** (USGS) led a field trip to Bainbridge Island (5 km west of Seattle) to show evidence of abrupt uplift in the last 1,700 years. An intertidal platform cut into the Blakeley sandstone has been uplifted about 7 m, and Bucknam postulates that the uplift was caused by an earthquake.

On the second day of the meeting, **Todd Perbix** led a field trip to the Pioneer Square area in downtown Seattle, where the Heritage Building and Union Station were visited. The trip was designed to introduce participants to rapid visual screening of buildings for seismic hazards and to provide an opportunity to view retrofit techniques first hand. Many of the retrofit features described during the Structural Engineering Session were observed during the trip.

Lifelines Session

To begin the final day of the meeting, **Keith Eldridge** of KOMO News narrated a slide presentation showing damage caused by the Loma Prieta earthquake. The presentation was prepared by the Lifeline Session chairman, **Don Ballantyne**. Eldridge was raised in the San Francisco area, and thus had more than casual interest in this earthquake. KOMO News dispatched him to California within hours of the main shock. Ballantyne discussed the importance of insuring that lifeline functions, such as power supply, communications, water and sewer services, can be restored quickly and easily after a major earthquake. He stressed that this can only come about with adequate preparation before the event.

A panel composed of **Walter Anton** (Seattle Water Department), **Allan Walley** (Washington Department of Transportation), **J. D. Cattanach** (B.C. Hydro), **William Elliott** (Portland Water Bureau), and **Ken Sullivan** (FEMA) discussed four issues:

- Marketing an earthquake mitigation program to decision makers who control the lifeline's budget
- Assessing the vulnerabilities of a lifeline system
- Estimating potential losses to a lifeline system
- Planning to reduce losses to a lifeline system

Elliott discussed the inventory and seismic loss estimation model for the Portland, Ore., water and sewer systems. He noted that credible expert opinion is the most important first step in marketing an earthquake mitigation program. Walley noted that 3,000 highway bridges would be in jeopardy in Washington State if a major earthquake were to occur. Of these, 70 would be vulnerable to tension failure, and 85 would have lesser vulnerability. He estimated that $30 million would be needed to retrofit existing bridges to current standards. Anton described the operation of the Seattle water-supply system and said that the most vulnerable part of this system would be the pipeline west of Lake Washington which carries water to Mercer Island. He noted that a $16 million seismic upgrade program has funded a new dam, reservoir lining, improved transition pipeline supports, strengthening of elevated tanks and standpipes, and other improvements. Sullivan explained the broad spectrum of FEMA's earthquake mitigation efforts with regard to five types of lifelines. He noted that FEMA has published volumes relating to each of the categories of lifelines and a final summary volume, *Abatement of seismic hazards to lifelines-An action plan.* The information has been distilled into an agency plan. Sullivan briefly discussed ongoing and planned FEMA projects for lifelines for the upcoming year.

Luncheon Festivities

Professors **Bekhzad Yulgashev** (particle physicist) and **Tursun Rashidov** (seismic engineer) representing the Uzbek Academy of Sciences (USSR) were distinguished guests at the luncheon on the third day of the workshop. State Geologist Ray Lasmanis presented the Uzbeks with a copy of *Engineering Geology in Washington,* published by DGER in 1989. Yulgashev gave a slide presentation about the impacts of the 1988 Armenian earthquake and his work on the design (using seismodynamic theory) of seismic-resistant underground structures.

Edward Wolfe, Scientist-in-Charge of Cascades Volcano Observatory (CVO) in Vancouver, Wash., followed with an assessment of volcanic hazards in the Pacific Northwest, which he had compiled with C. Dan Miller of CVO. Wolfe summarized the postglacial eruptive activity of Cascade Range volcanoes, described the nature of the volcanic processes, and discussed methods of monitoring and volcanic hazards analysis now being conducted by the observatory.

Loss Estimation Session

Bruce Olsen, independent consulting engineer, chaired the Loss Estimation Session. The first speaker, **Peter May** of the University of Washington, differentiated two types of loss estimates: region-wide dollar estimates of prospective losses and vulnerability assessments. He summarized implications and usefulness of these estimates. Noting that there were few examples of loss estimates being put to practical use, May stressed that we should focus more attention on "vulnerabilities of key elements of our physical and social systems than [on] region-wide dollar losses." The result would be an increased emphasis on priorities for upgrading facilities and lifeline systems to reduce their vulnerability. May stated that loss estimates are highly uncertain and that policy actions, such as research priorities and planning monies, linked to them are potentially flawed.

W. Paul Grant of Shannon and Wilson, Inc., discussed the impact of poor soil conditions on building damage during earthquakes. His slides detailed typical damage caused by the 1949 Olympia and 1965 Seattle earthquakes.

Chris Arnold of Building Systems Development focused on the economic aspects of architectural design as they relate to potential earthquake losses. He discussed hidden costs, such as those associated with damage to a new Hyatt hotel by the Loma Prieta earthquake. The hotel sustained $7 million damage outright, but it lost an additional $1 million per month revenue because it could not be opened until repairs were made. He demonstrated how difficult it is to define loss because of the revenue increases in some businesses and the increasing "velocity of money" following an earthquake.

Roger McGarrigle of Van Domelen, Looijenga, McGarrigle, and Knauf discussed seismic design and loss estimation in areas of low historical seismicity. He noted that upgrading structural components is generally a minor component (about 2-5 percent) of the total cost of a new building and that it is much more expensive to retrofit.

Craig Taylor of Dames and Moore in Seattle examined the complicated relation of earthquake losses to lenders and the insurance industry. He analyzed the types of risks that lenders and insurers incur, the nature of loss estimation, and stressed the need for financial models that can quantify the degree to which lenders and insurers, respectively, bear losses, as opposed to losses borne by others.

This Fourth Annual NEHRP Workshop for the Puget Sound and Portland areas was probably one of the more successful in presenting not only geotechnical data, but also engineering information to planners, architects, and emergency response managers and specialists. Likewise, geoscientists were able to learn a great deal about mitigating structural damage, estimating losses, and protecting lifelines. Evidently, not all the aftershocks of the Loma Prieta earthquake were seismic in nature because new data and numerous insights regarding earthquake damage and effects, in a large part related to the recent studies of this event, have provided an impetus for anticipating and solving similar earthquake problems in the Northwest. Mitigation of earthquake hazards will have much greater success with the kind of strong interdisciplinary approach that characterized this meeting.

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WELCOME ADDRESS

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I'm delighted to be here, not least because it's so hard nowadays for elected officials to find anything to be strongly against. It takes a lot of the fun out of being a politician if you can't summon up righteous indignation about something, and today it seems that nearly every practice and substance has a well-organized defender, even broccoli, as President Bush found recently when he came out against it. I am going to take a chance, however, and say that I'm against earthquakes, and I always have been.

Of course, implementation of that policy is the hard part, and, in all seriousness, that's just the point: we can't prevent earthquakes, so we have to rely on mitigation and protection after the fact. But earthquake protection <u>is</u> a remarkably difficult issue for public policy to deal with. Unlike protection from fire or chemical spills, there's really no prevention alternative; and unlike crime protection, earthquake protection typically generates little pressure on political leaders. We don't get reports of slow tectonic movement on the six o'clock news.

It's a sad fact that demands on government always exceed the resources we have and the immediate need almost always carries the day over the possible but not pressing event. Sadder still, much of what happens in the public sphere happens because somebody is getting public credit for it. When we plant a tree, it's an event; when we save a thousand trees through routine fire prevention actions, nobody notices. Opening a new road or bridge is a ribbon cutting ceremony; providing maintenance for that road or bridge is something that happens obscurely in the back office; or maybe it doesn't happen at all.

I'm very much afraid that this is our current situation with respect to earthquake readiness in the state of Washington. Currently, for example, except for some university research funding, and some retrofitting projects there is virtually no direct state expenditure for earthquake readiness. California, in contrast, commits tens of millions of dollars a year on this. Meanwhile, we have <u>200,000</u> children attending schools that the Board of Public Instruction thinks would not survive a major quake. There are sixty-five freeway ramps and bridges in this state that are obsolete and liable to collapse the way the upper deck of the Bay Bridge did during the recent San Francisco earthquake. All throughout the state are buildings housing people for whom the state has special responsibility: not only schools, but hospitals, prisons, nursing homes and office buildings. Whether these people are as safe as we can make them, whether the state government itself has shown an example of readiness is, I think, open to question.

And readiness counts. What government does counts: only 67 people died in the Bay area, and vital services were restored in a matter of days. In contrast, the recent Armenian earthquake, where planning and mitigation were essentially absent cost 25,000 lives and brought the economic life of the community to a stop for the better part of a year.

One of the "benefits" (if that's the right word) of the Bay earthquake for us here in Washington, is that it has raised earthquake readiness out of the back office, at least for a while. There is nothing like a good visual aid to stimulate the imagination into action, and San Francisco tilted and burning on the TV looked uncomfortably like Seattle might look like if it were tilted and burning. The meeting called in February by Senators Gorton and Adams to discuss our earthquake preparedness drew over 1400 people, and I think it's fair to say that a lot of them would not have been there without the Bay area example fresh in mind.

So we have a window of opportunity--press and political attention are a lot higher than they would have been without the California shock--but this will fade in time, and with it the understanding that we are far <u>less</u> prepared than San Francisco was.

For example, the Bay area has had in place for decades a dense network of strong-motion accelerometers to measure the strength and direction of tectonic movement. Over the years this enabled authorities to learn a great deal from the numerous small earthquakes that occur in any seismic area, knowledge that was incorporated into damage prevention codes and mitigation planning. We don't have such a network in this state, and there is no funded program to construct one.

Having a base of detailed information about seismic motion would be extremely valuable. As an example, the Department of Natural Resources is currently participating to the construction of a new Natural Resources Building in Olympia. Because we happened to have had recording instruments located a few blocks away from the building site for many years, we were able to provide the building's designers with relatively site-specific technical data, which will eventually allow the more efficient construction of a seismically safer building. What we provided by chance should be available by intent to every building designer in the state. Otherwise we have to rely on uniform building codes, which are designed to prevent catastrophic collapse rather than to allow buildings to continue in use after an earthquake shock.

The Department of Natural Resources supports the establishment of an adequate monitoring system, and our Geology Division is ready to work with other state agencies and the legislature to set one up. Only then can we be sure that each seismic event is an experience from which we can learn what we need to know to survive the inevitable big one.

Structural preparedness is, of course, not the whole story. During the Bay area quake one of the things that broke down first was communications. Outlying areas in the Santa Cruz Mountains, for example, were cut off from the outside world for a considerable time, as wire-based communications systems went down. A number of state agencies, including the Department of Natural Resources, have radio communications systems that could serve as emergency communications links in a stricken area. But I'm not sure that anyone has done the hard training and organizational work necessary to weld these systems into a unit that would survive and serve during a major quake.

And this is just one of the things that should be done while we have a heightened interest in the subject, and, needless to say, before the next big earthquake. What the others are, you, of course, know far better than I.

And with that, let me say that it gives me great pleasure to welcome you all to the Fourth Annual Workshop of the National Earthquake Hazards Reduction Program. You will hear a lot of recommendations over the next few days about what should be done to make us ready; let's hope we accomplish these things before we have to hear them all over again as what we should have done, after it's too late.

SECTION I: GEOSCIENCE INFORMATION

The contributions in this section contain scientific and historical information on various aspects of earthquake hazards in the Pacific Northwest. This information supplements and extends two documents:

1) Hays, W. W., 1989, Proceedings of Conference XLVIII, the 3rd annual workshop on earthquake hazards in the Puget Sound-Portland area: U.S. Geological Survey Open-File Report 89-465, 303 p.

2) Noson, L. L.; Qamar, Anthony; Thorsen, G. W., 1988, Washington State Earthquake Hazards: Washington State Department of Natural Resources Information Circular 85, 77 p.

REGIONAL SEISMICITY AND TECTONICS

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The Pacific Northwest is a subduction zone environment that presents difficulties for earthquake hazard estimation. Although we have made great progress in understanding the patterns and causes of earthquake generation in recent years, there is much work remaining. Since our recorded history is short, we have relatively little observational evidence of large earthquakes. Nevertheless, a number of significant earthquakes have been observed and a pattern may be emerging. Large shallow earthquakes seem to occur in a arcuate band around the Puget Sound region. The only large deep earthquakes (>40 km depth) that we have observed have occurred within the subducted Juan de Fuca plate in the center of this pattern, precisely where large shallow earthquakes have not been observed. Large deep earthquakes may occur beneath western Oregon and western British Columbia although none have been well observed. Further work on structure and plate kinematics may shed light on this problem.

Geologic, seismic, and geometric considerations suggest that the shape of the subducted Juan de Fuca plate exercises important control on the geology, topography, and earthquake generation process in the Pacific Northwest. It has been suggested that the angle of dip of the subducted plate exerts a profound influence on the rate of uplift of rock at the accretionary margin. Theoretical models suggest that uplift cannot occur if the dip exceeds about 10°. Recent evidence indicates that the Juan de Fuca plate is bent into an eastwardly plunging arch, or anticline, beneath the Olympic Mountains. As a result, the angle of subduction is approximately 10° to the east in this region, significantly less than regions to the north and south where the angle of dip is in the range of 15° to 20°. This arch has a natural origin in the plate flexure necessary to accommodate trench geometry off the coast. The Olympic Mountains and the associated arcuate pattern of surface geology surrounding the Olympics, including Puget Sound, apparently result directly from this plate flexure as a consequence of the reduced plate dip. It is also possible that the coastal subsidence in southwest Washington results from a counterflexure of the subducted plate.

The most seismically active part of the subducted Juan de Fuca plate, beneath Puget Sound, lies at the crest of the arch. Its influence may extend to the Cascade front, into British Columbia, and into northern Oregon, affecting the generation of intraplate earthquakes both in the subducted slab and in the overlying North American Plate. It may also be important in influencing interplate earthquakes. The insight that we have gained underlines the importance of further effort to define the plate geometry in Washington, Oregon, and British Columbia.

EARTHQUAKE-HAZARD GEOLOGY MAPS OF THE PORTLAND METROPOLITAN AREA, OREGON

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Abstract

As part of an earthquake hazard reduction program for northwestern Oregon, earthquake-hazard geology maps have been produced for eight 1:24,000 map sheets covering most of the Portland metropolitan area. The maps are based on new and existing geologic mapping and interpretation of several thousand boring logs. The maps depict the distribution and thickness of potentially responsive or liquefiable Quaternary sediments, other Quaternary and bedrock geologic units, faults and contoured depth to basement data. Four units have been identified as potentially responsive or liquefiable. These are, Quaternary catastrophic flood sediments (Qff), Quaternary alluvium (Qal), artificial fill (Qaf) and loess (Ql). Qff and Qal are commonly 30-60 ft thick and sufficiently regular thickness to isopach. Ql and Qaf are locally thick, but have wide variability in thickness and have not been isopached.

Numerous northwest- and northeast-trending faults have been mapped, some of which may cut rocks as young as Pleistocene.

QUATERNARY DEFORMATION IN THE PORTLAND METRO AREA

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Silvio Pezzopane, University of Oregon

Abstract

The Portland Metro area has a modest history of crustal seismicity but lacks any documented example of Quaternary deformation. Recent mapping (Beeson and others 1989, Madin, 1990) has documented Neogene faults in the area for the first time, but has not demonstrated Quaternary seismic activity. Research in progress has identified faulted flows of the Plio-Pleistocene Boring lavas. The flows have a K/Ar age date of 612 +/- 23 ka, and have been offset an unknown amount by several faults. Dating of other faulted flows in the area is pending.

Paleoliquefaction features have been documented in fine sands and silts deposited by Pleistocene catastrophic floods at three sites in the Portland Metro area. The features include sand and silt dikes and sand blows. Sand blows and dikes clearly cut or warp 3 successive paleosols at one site. Strong ground shaking during local or distant earthquakes or rapid loading during catastrophic floods may have induced this liquefaction.

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VOLCANO HAZARDS IN THE PACIFIC NORTHWEST

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Thirteen major volcanic centers and numerous smaller basaltic or basaltic andesite volcanoes occur along the Cascade Range of Washington, Oregon, and California. During the past 12,000 years, Cascade volcanoes have erupted at an average rate of almost two eruptions per century. Most of the major centers have been active during this period, and two (Lassen Peak and Mount St. Helens) have erupted during the present century. The most recent, Mount St. Helens, caused significant loss of life and economic disruption. Future eruptions in the Cascade Range are a virtual certainty.

Assessment and warning of volcanic hazards in the Cascade Range draw upon two complementary types of information: (1) the geologic record of past activity at each potentially active volcano, and (2) the character of processes directly observed at active volcanoes in the Cascade Range or elsewhere. Thus, lessons from recent eruptions such as those of Mount St. Helens or Colombia's Nevado del Ruiz, or from the current eruption at Redoubt Volcano in Alaska, can be applied directly to assessments of hazards and monitoring of volcanoes in the Cascade Range.

Potentially hazardous volcanic phenomena in the Cascade Range include tephra falls, pyroclastic flows and surges, lateral blasts, debris avalanches, lava flows, and debris flows and floods. Debris-flow and flood hazards are particularly enhanced by the large surface areas and volumes of snow and ice that mantle many of the major Cascade volcanoes. Volcanic hazards increasingly threaten human life and human activities as communities and economic and recreational developments expand on the flanks of volcanoes and in their drainageways. Successful mitigation of risk from volcanic hazards requires a continuing program of volcano studies, hazard assessment, and volcano monitoring combined with education of the public and hazard-based planning for land use and emergency management.

Ground Motions from Hypothesized Mw=8 Subduction Earthquakes in the Pacific Northwest

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The amplitude and duration of strong ground motions from hypothesized M_w=8.0 subduction thrust earthquakes in the Puget Sound - Portland region were estimated using a semi-empirical method. The simulation procedure assumes the rupture surface may be represented by a grid of fault elements. Finite difference wave simulation in a detailed two-dimensional velocity structure identifies direct-S and the Moho post-critical reflection as the primary components of the S wave field at the distances considered (30-100 km). Green's functions containing these two arrivals are computed with generalized ray theory in an equivalent one-dimensional structure for each source element - receiver propagation path. Scattering and attenuation structure are empirically modeled by the use of corrected accelerograms from M_w~7 Michoacan, Mexico and Valparaiso, Chile aftershocks as the fault element source functions. Spatial variations in slip on the fault (asperities) are introduced by weighting the fault elements. The technique has been validated for large subduction zone earthquakes by modeling acceleration time histories and response spectra from the 1985 Michoacan (M_w=8.1) and Valparaiso (M_w=7.9) mainshocks.

Fault models for the Puget Sound and the Portland regions and seismic velocity structure models are adapted from regional refraction studies. Uncertainty in the location of the asperities on the fault surface results in a large degree of uncertainty in the simulated ground motions at a given site. If distance is defined as the distance to the closest asperity, then the variability in the ground motions is reduced, indicating that this uncertainty can be lessened by constraining the depth of the asperity. The ground motion estimates are relatively insensitive to the difference in fault dip between the Puget Sound and Portland fault models. For a seismic moment of 1.3 x 10^{28} dyne-cm, the attenuation of peak acceleration with distance r from the fault asperity is given by:

$$\ln(PGA) = 15.5 - 3.33\ln(r + 128) + 0.794s$$

where s is a site term equal to 0 for rock and 1 for soil.

Formal estimates of uncertainty in the calculated ground motions are obtained by estimating both parametric uncertainty (from the range of source models of hypothesized Cascadia subduction earthquakes) and modeling and random uncertainty (from the misfit between recorded and simulated ground motions of the 1985 Michoacan and Valparaiso earthquakes). For periods less than 1 sec, the estimated response spectral velocities in the Seattle - Olympia region are about twice those recorded during the 1949 Olympia and 1965 Seattle earthquakes, and the durations of strong shaking are significantly longer (40-60 sec vs. 10-20 sec).

Evidence for and Implications of Small-Scale (<1m) Tectonic Subsidence in Salt Marshes of Alsea Bay, Oregon, Central Cascadia Margin

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Alsea Bay is a fluvially-dominated estuary in central Oregon. The salt marsh subsurface contains ten buried peat (paleomarsh) layers. The radiocarbon age of the oldest layer is approximately 4500 years B.P. The record of marsh burial in Alsea Bay is important for two reasons. First, a tectonic mechanism (coseismic subsidence) of marsh burial, as opposed to a storm, river flooding or oceanic (i.e. El Niño) mechanism, can be clearly identified. For example, river flooding can be discounted, because 1) sand deposits, directly overlying buried peats, contain a marine sand component, 2) these sand deposits thin up bay, and 3) there is widespread correlation of key stratigraphic horizons.

Second, the estimated amount of subsidence at Alsea Bay (<1 meter) is less than the amount of subsidence (1-2 meters) calculated for buried marshes in estuaries of northern Oregon and southern Washington. Small-scale subsidence at Alsea Bay is based on abrupt transitions from high marsh to lower high marsh or upper low marsh, in contrast to high marsh to tidal flat transitions associated with larger-scale subsidence. This indicates either 1) a greater distance from the axis of subsidence and a closer distance to the zero isobase (where no uplift or subsidence occurs) than northern Oregon/southern Washington estuaries during large magnitude earthquakes or 2) separate events of smaller magnitude, which would argue for segmentation of the Cascadia Subduction Zone. Evidence of small-scale subsidence would place Alsea Bay somewhere between the axes of maximum subsidence and uplift for a large Cascadia Subduction Zone earthquake. The offshore fold belt, that parallels the trench, comes onshore in the Coos Bay area of southern Oregon. Onshore fold belt, rather than megathrust (subduction zone), tectonics might control peat burial as well as their distribution in the Coos Bay area. Therefore, evidence of uplift from a large prehistoric subduction zone earthquake could be found somewhere between Alsea Bay and Coos Bay.

SITE-SPECIFIC EARTHQUAKE STRONG GROUND MOTION STUDIES IN THE PUGET SOUND AND PORTLAND, OREGON METROPOLITAN AREAS

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ABSTRACT

Despite views that the Pacific Northwest does not possess a significant level of seismic hazard based on the relative absence of damaging earthquakes in historic times, recent seismologic and geologic studies suggest the contrary. Realistic site-specific predictions of strong ground shaking that might be generated from future large earthquakes are thus of utmost importance to seismic safety. Until recently, such estimates have not been possible for the Puget Sound and Portland areas nor have they been required based on the perception of low seismic hazard. The only strong motion data available for the Pacific Northwest are a few recordings of the 1949 M 7.1 Olympia, the 1965 M 6.5 Seattle-Tacoma and the 1962 M 5.1 Portland earthquakes. This relative lack of strong ground motion data has historically been a problem for regions outside California. Thus the use of empirical relationships, generally for peak ground acceleration, has generally been the approach taken to estimate potential strong ground shaking at a site. The inability to incorporate site- and region-specific effects, however, severely limits the applicability of such estimates. In particular, actual observations and research have long since recognized the influence of the near-surface geology on strong ground motions, especially for those areas overlain with unconsolidated sediments. Such site effects can often dominate the contributions of the earthquake source and propagation path especially in the frequency range of most engineering concern (approximately 1-10 Hz).

In the past decade, a new strong ground motion methodology incorporating the Band-Limited-White-Noise (BLWN) source model coupled with random vibration theory (RVT) and an equivalent-linear formulation has been developed that appears to successfully predict strong ground motions for both rock and soil sites in a variety of tectonic regimes (Hanks and McGuire, 1981; Boore, 1983; Boore and Atkinson, 1987; Silva and Darragh,

1990; Silva et al., 1990). This approach is attractive in that it utilizes simple source, propagation path, and site properties which are easily determined. Currently, the BLWN-RVT model does not accommodate basin effects which can amplify ground displacements in the period range of 5-10 seconds (Vidale and Helmberger, 1987). At shorter periods (less than 1 sec), basin effects do not appear to exert a controlling influence on ground motions (Seed et al., 1988). Rather, the local soil properties, velocity gradient, damping, and profile thickness appear to be the controlling factors at soil sites where dense strong motion data are available (e.g., SMART-1 array in Taiwan). Strong motion simulation studies for the Puget Sound and Portland regions such as those by Cohee et al. (1990) have been performed to address the details of the rupture process of a potential M 8 Cascadia earthquake. Modifications of the BLWN-RVT methodology are also currently being made to incorporate additional source and path effects.

In a study of strong ground motions in the Puget Sound region employing the BLWN-RVT methodology, we have computed acceleration response spectra for the 1949 and 1965 earthquakes as recorded at the Olympia Highway Test Lab and Seattle Federal Building (Silva et al., 1990). Incorporating site-specific shear wave velocity, density, and Q_s data in a geologic profile for each site and the source parameters of the two events, the predicted strong motions agree quite well with the actual recordings. Acceleration response spectra and time histories for a hypothetical M 8 Cascadia subduction zone earthquake have also been predicted for a hard rock and deep soil site in Seattle at a rupture distance of 70 km. A comparison of the two spectra dramatically points out the influence of near-surface soils and the properties of the underlying rock on the amplitudes and spectral content of strong ground motions. Both amplification and deamplification are evident in the deep soil site response spectra compared to the rock site.

Additionally, strong ground motions resulting from possible moderate to large magnitude earthquakes near the Portland metropolitan area have been estimated for the 28-m-thick soil site of the new State Office Building in Portland (Wong et al., 1990). The earthquakes considered were three crustal events of M 5.5, 6.0 and 6.5 located at an epicentral distance of 10.0 km and a focal depth of 10.0 km and a M 8.0 Cascadia subduction zone event located at a closest distance to the rupture plane of 73 km. Region-specific information on crustal structure and seismic attenuation and a detailed geologic profile of the site were used in the ground motion estimates. The estimated peak ground accelerations ranged from 0.18 to 0.32 g for the crustal earthquakes and 0.20 g for the M 8 Cascadia earthquake. The predicted acceleration response spectra for the site for these events were compared with Uniform Building Code (UBC) design spectra; all but the M 5.5 crustal earthquake exceed the currently recommended UBC zone 2B spectra for the Portland area. This comparison, however, should be viewed in the context of two critical assumptions made in the study: (1) the chosen epicentral distance and focal depth of the crustal earthquakes and (2) the choice of magnitude for the Cascadia event.

Existing geologic and seismologic data cannot preclude the possibility of a crustal earthquake occurring closer to Portland nor a subduction zone earthquake significantly larger than M 8. Thus given the extensive unconsolidated sediments in the Portland metropolitan area (Madin, 1989) and the possible future occurrence of earthquakes of M 6 and larger, strong earthquake ground shaking would appear to pose a potential serious threat to many existing and possibly even to newly constructed buildings in the Portland area.

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EARTHQUAKE-INDUCED LANDSLIDES

by

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INTRODUCTION

This Paper presents a simplified summary of the types of landslides which occur during strong motion earthquakes. Emphasis has been placed on discussing causation, and relevance to geologic conditions in the Puget Sound-Portland region. For discussion purposes, the earthquake-induced landslides have been grouped as follows:

- marginally stable soils
- translational-slide movements in clay soils
- liquefaction of cohesionless soils, especially sands

LANDSLIDES IN MARGINALLY STABLE SOILS

Many slopes have "marginal" stability under normal conditions. In western Oregon and southwest Washington, ancient landslide terrain is encountered in which a stiff clay or colluvium mantles a slope. The ancient slip zone comprises weakened clay at "residual" strength. It often requires little change in the stability relationship (e.g. a road cut or fill) to reactivate movement along the ancient slip; hence the ground has "marginal" stability. Other examples of marginal stability include ocean cliffs, actively eroding river banks, manmade cuts and fills on steep terrain, talus slopes, weathered rock faces, and stratified volcano slopes.

When marginally stable slopes are subjected to the horizontal forces from a strong motion earthquake, failure can occur. Usually these failures are local and fairly small (Chleborad & Schuster, 1989). However, the Olympia earthquake of 1949 (magnitude 7.1) produced a slide in a 300-foot high cliff into the Tacoma Narrows near Fort Nisqually (Noson, Qamar & Thorsen, 1988). The main body of the slide occurred in the Esperance Sand stratum in a slope averaging about 32° to the horizontal. A photograph of the slide indicates a surficial break typical of sands. Although some reviewers have suggested liquefaction may have been partly responsible for the failure, the fact that failure was delayed until three days after the earthquake, and the type of failure, suggest that it can be classified as failure of a marginally stable sand slope. Approximately 50 slope failures were caused by the Olympia earthquake.

Seed and Goodman (1964) and Goodman and Seed (1966) discuss the analysis of these movements during earthquakes in slopes of cohesionless soils with marginal stability. These analyses assume that the slopes are above the water table and will not be subjected to liquefaction during the earthquake.

In clay slopes of ancient landslide terrain, a slope collapse would not occur, but significant movements could damage structures located at the margins of the landslide. Although this specific issue has not been studied extensively, Makdisi and Seed (1978) provide an approximate method for estimating ground movements in clay slopes during a major earthquake. Briefly, the method calculates the horizontal "yield acceleration" k_y needed to bring the factor of safety of the slope below one. During an earthquake the slope is assumed to move during the part of the earthquake-induced acceleration-time graph which exceeds the calculated "yield acceleration". Therefore, significant total movements occur during a large magnitude earthquake in which the duration of strong motions is high. The calculated "yield acceleration" is low for slopes with marginal stability, and thus is likely to be exceeded for longer periods of time (during an earthquake) than in a slope with a higher static factor of safety.

TRANSLATIONAL-SLIDE LANDSLIDES

The second group is block slides in clay slopes which have adequate stability under normal static conditions but can become unstable when subjected to the horizontal forces of a large earthquake. This type of failure is likely to occur when the ground has a plane of weakness in the near-horizontal direction and thus responds to the horizontal forces occurring during an earthquake (Fig. 1). Several major slides of this type occurred in Anchorage, Alaska during the 1964 Alaska earthquake (magnitude 8.4). The clay stratum which sheared along near-horizontal surfaces during the approximately 5 minutes of strong motions is the Bootlegger Cove clay, a blue-gray plastic clay, 200 to 300 feet thick, which is sensitive to remolding and loses strength under cyclic loading (Seed & Wilson, 1967). Silty and sandy beds are found within the clay, especially near the surface of the stratum, and liquefaction pore water pressures within these more permeable beds may have contributed to the failures. However, they are separated here because the main slippage at Anchorage appears to have occurred within the sensitive clay and thus needs to be distinguished from those failures which result from loss of strength within loose sand layers.

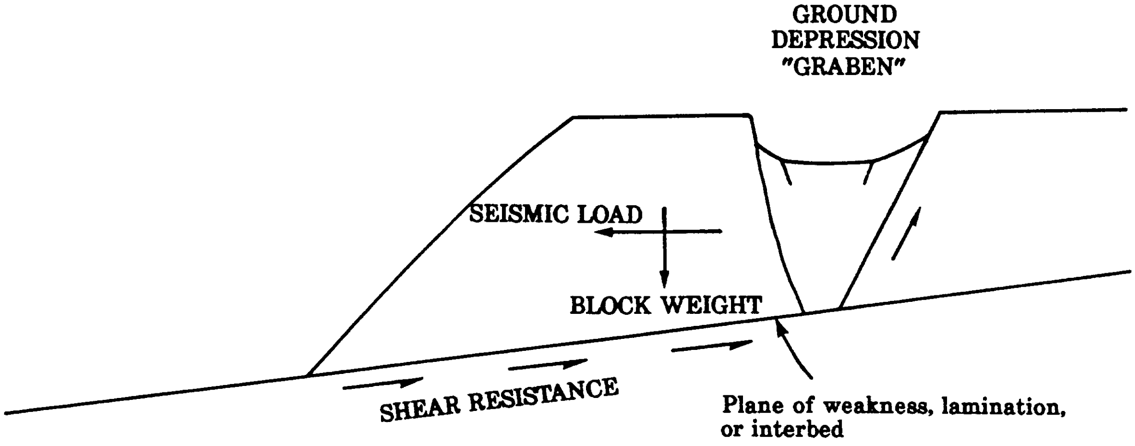


Figure 1. Translational-Slide Landslide During Earthquakes

The failure of the soft sensitive lacustrine clays caused spectacular movements and resultant damage. In Anchorage, the L Street slide moved about 12 to 15 feet horizontally, and the Fourth Avenue slide moved about 10 feet (Long & George, 1967). In each case, the block movement created a ground depression ("graben") at the head of the slide where the unstable block separated from the stable ground (Fig. 1). The length of the Fourth Avenue slide, from headscarp to toe, of about 600 feet has indicated that the half wavelength of the seismic shock may control the breakaway point and has provided one method of making a pseudostatic analysis of the slide (Long & George, 1967). These two slides were stabilized against future major earthquakes by construction of rockfill buttresses at the toe of the block slide.

The Turnagain Heights landslide in Anchorage covered an enormous area: 8,500 feet wide along the coastline and up to 1,200 feet inland (130 acres). The slide moved into the sea for distances of up to 1,200 feet. The slide mass broke up and destroyed 75 homes. Model tests of the Turnaround Heights slide were performed on a shaking table at the University of California (Seed & Wilson, 1967). The results, Figure 2, showed that the failure was retrogressive (i.e. started at the toe of the slide and moved backwards) and the broken up soil in the model had a strong resemblance to the geologic section observed in the detailed site investigations, Figure 3. These results indicate that the size and damage of such landslides depend on the duration of the strong ground motions.

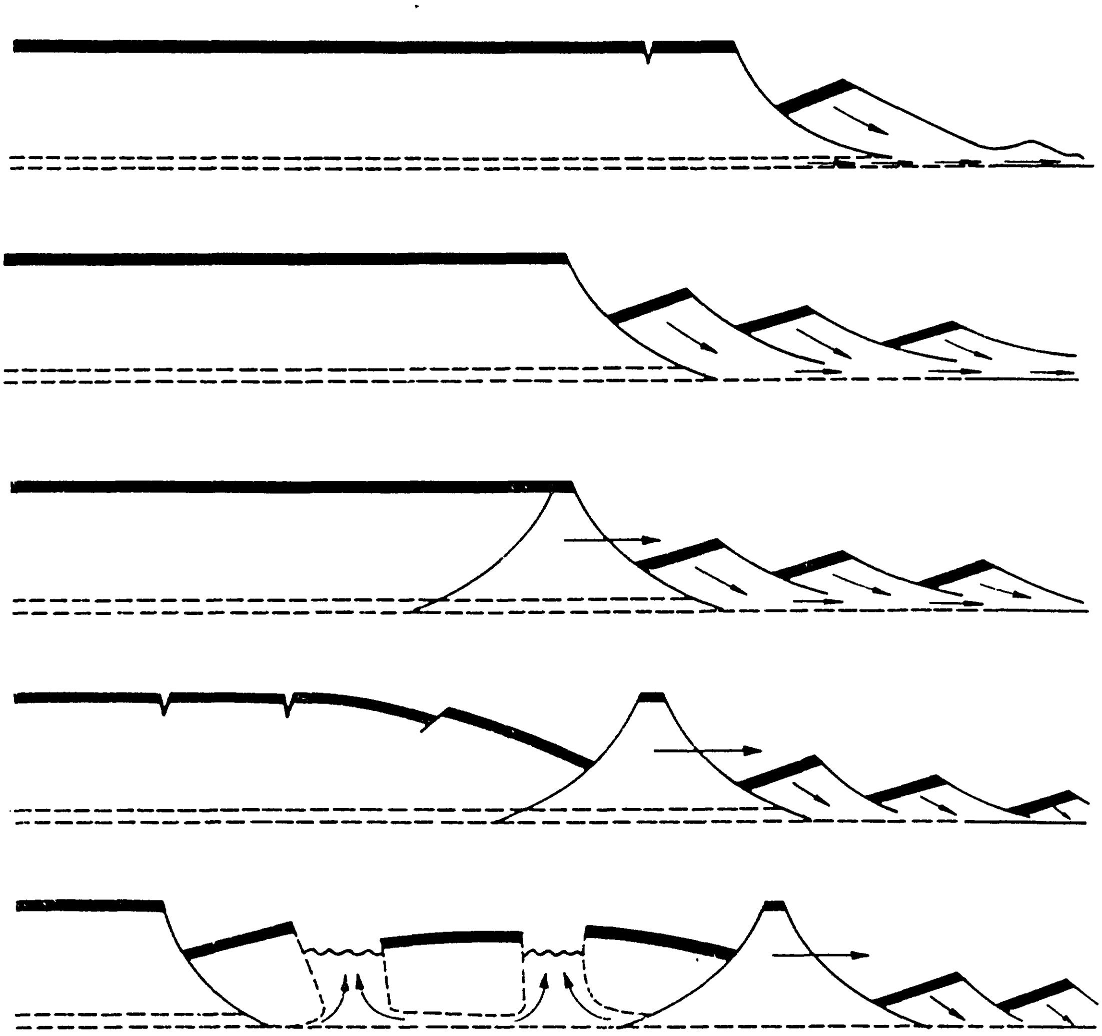


Figure 2. Progressive Failure Mechanism Observed in Model Tests (after Seed & Wilson, 1967)

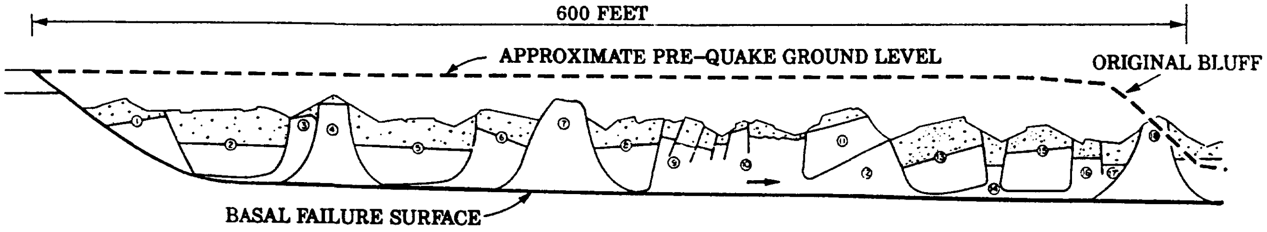
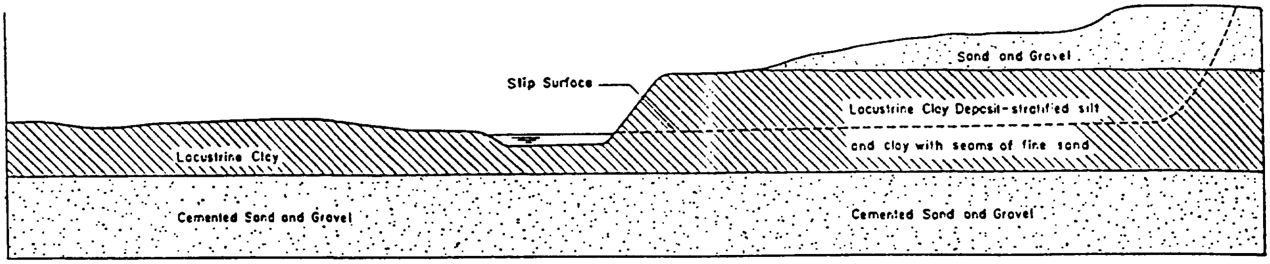


Figure 3. Soil Profile at Turnagain Heights After the Alaska Earthquake (after Seed & Wilson, 1967)

A landslide on the San Pedro River near Lake Rinihue, Chile during the earthquake of 1960 caused shear failure in lacustrine clay with considerable breakup of the ground surface (Fig. 4). This failure extended about 1,700 feet behind the original cliff (Davis & Karzulovic, 1961) and involved 30 million cu.yd. of slide materials.

Before Earthquake



After Earthquake

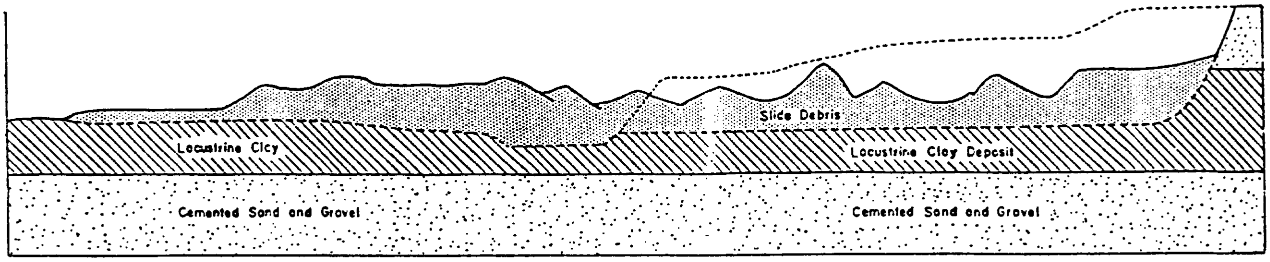


Figure 4. Large Translational Slide Near Lake Rinihue, Chile (1960) (after Davis & Karzulovic, 1961)

Washington State has lacustrine varved clay deposits of silt and clay with near-horizontal bedding planes. Additionally, recently deposited clays and silts in rivers and estuaries are known to drop in strength on remolding. Both types of clays could be susceptible to failure under strong ground motions of sufficient duration. Although there is no past history of such failures, it is possible that clay interbeds between basalt flows could fail under strong ground motions.

LIQUEFACTION-INDUCED LANDSLIDES

The third, and most important, group of landslides results from temporary liquefaction of sands during strong ground motions. Sands (and other cohesionless soils, which include gravels and coarse silts) have point-to-point contact between the grains. When the structure is disturbed by earthquake shaking, the sand grains may go into a fluid state, depending on the compactness of the sand, weight of overburden, duration and severity of shaking, etc. When liquefaction failure occurs, sand "boils" often appear at the ground surface. The surface itself may be broken up, but more often is subjected to extensive cracking as the ground shifts laterally.

Landslides caused by liquefaction are reported in virtually every major earthquake. To liquefy, the sand deposits have to be relatively loose and below groundwater. For a landslide to develop, a slope also has to be present. Such a slope can be an embankment fill on top of the sand deposits, or a river bank, waterfront, etc. Therefore, areas at particular risk of liquefaction include flood plains of rivers, deltas, estuaries, and loosely placed sand fills (including hydraulic sand fills). Highways or railroad fills may fail and bridges may be compressed by ground moving towards the river from one or both sides (Fig. 5). Buried pipelines may be broken or float to the surface.

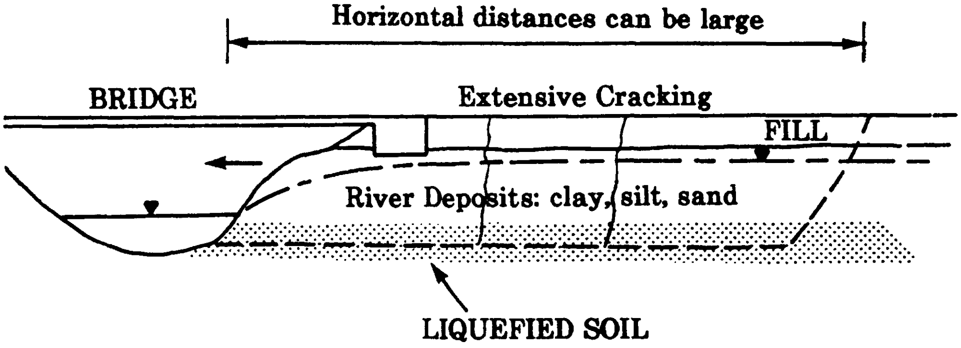


Figure 5. Liquefaction Causing Lateral Slide Movements on Flood Plains

At waterfronts, flow slides may develop. At Valdez, extensive sections of the waterfront were carried away by flow slides during the Alaska earthquake of 1964 (Coulter and Migliaccio, 1966), as shown on the artist's sketch made after the event, Figure 6. Shannon (1966) reported a similar waterfront flow slide at Seward.

It is estimated that 270 bridges were severely damaged by this earthquake, with movements of up to 6½ feet towards the rivers being observed at some locations (McCulloch & Bonilla, 1967). The damage was greatest on the deltas at Whittier and Seward; it was six months before the first train reached Seward after the earthquake. Bridges built with their abutments on rock suffered little damage.

Dames & Moore (1989) examined damage in the San Francisco area immediately after the Loma Prieta earthquake (magnitude 7.1). Although the 10 seconds duration of strong shaking was relatively short, they noted extensive damage to uncontrolled fills. On the other hand, engineered sand fills placed under careful control survived the shaking extremely well.

Figure 6. Artist's Concept of Flow Slide at Valdez (1964) (after Coulter & Migliaccio, 1966)

There has been a considerable advance in knowledge of soil liquefaction over the past 20 years. The impetus to pour effort into research came after the near-failure of the Lower San Fernando Dam in California in 1971 (magnitude 6.6). The 30-foot drop in the crest of this dam, which had been declared safe in an inspection only five years earlier, was attributed to liquefaction of hydraulic sand fill placed in the shoulder of the dam during construction in 1915.

A liquefaction analysis is a fairly complicated study. The study team must first determine the design earthquake for the site, and then determine the probable ground accelerations which will affect the potentially liquefiable soils. These analyses provide the input destabilizing forces, expressed as the "cyclic stress ratio". The resistance of the ground is determined from Standard Penetration Test (SPT) sampling with various corrections for overburden weight, silt content, pre-existing horizontal shear stresses, etc. The lateral continuity of the weak zones is also important. The resistance of the ground is then compared to the input forces of the earthquake to determine the slope stability.

There are several methods of preventative treatments for ground subject to soil liquefaction. They are comparatively expensive although the actual cost will depend on the local circumstances. They include: drainage, soil compaction, remove and replace, grouting, slope support, and relocation of the at-risk facility.

The Puget Sound-Portland region has many waterfront fills on the Sound, Olympic Peninsula, coastline, Columbia River and other tributary rivers. There are also many old fills in the cities and towns where sand is below groundwater levels. The potential for damage to lifeline structures from liquefaction-type landslides during a major earthquake is substantial.

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THE RELATION OF EARTHQUAKE INTENSITY TO SURFACE GEOLOGY AND/OR ELEVATION IN WESTERN WASHINGTON STATE

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Modified Mercalli Intensity values are analyzed to determine their relation with surface geology and/or elevation. The research data for this study consists of approximately 4000 intensity reports derived from the 1965 (M=6.5, depth=59 km.) and 1981 (M=5.5, depth=7 km.) Washington State earthquakes. The intensity reports were collected from locations in western Washington and northern Oregon. As a subset, the Seattle and Olympia areas are the principal focus of my investigation. For each of these areas, the sites of these reports lie in such close proximity that distance from the hypocenter is not a factor in their relative variation. The Seattle area study includes approximately 1600 reports, the Olympia area 250 reports.

In the Olympia area, for both the 1965 and 1981 events, intensity is significantly associated by the Chi-squared test of significance with surface geology. Both sets of intensity data are located 80 to 100 km. from each hypocenter. All the 1981 intensity values are then adjusted by addition of a constant to allow comparison with the larger intensities derived from the greater magnitude 1965 event. The relation of intensity to surface geology is then found to be independent of the type of earthquake (predominantly normal vs. strike-slip), depth (59 vs. 7 km.), location (47.4 and 122.4 vs. 46.4 and 122.2 degrees latitude and longitude), intensity questionnaire, and person assigning intensities from the reports. Cramer's measure of association between intensity and surface geologic units for both earthquakes ranges from .294 to .356 out of a maximum of dependence of 1. The overall results are compared to previous related research in this area.

In the Seattle area, for the 1981 event, intensity is significantly associated with surface geologic classes, although it is not for the 1965 event. Three possible hypotheses are given for the lack of association including deficits in the intensity method and sampling errors. The results from both events are compared with previous related research in this area. Cramer's measure of association between intensity and surface geologic classes for the 1981 results is .137 out of a maximum of 1.

In the Seattle area, for the 1981 event, surface geologic units are classified by site elevation. A significant association is found between intensity, and post glacial surface geologic units located at elevations of 0 to 100 feet in 25 foot increments. Another significant association is found between intensities located on all geologic units except glacial till at elevations of 0 to 200 feet in 100 foot increments.

Evidence of Liquefaction in the Puyallup Valley During the 1949 and 1965 Puget Sound Earthquakes

by

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The city of Puyallup, located 30 miles south of Seattle, is situated on the broad floodplain of the Puyallup River. During the 1949 Olympia earthquake (magnitude 7.1 on the Richter scale) considerable sand blow activity was observed in the vicinity of Puyallup.

The valley's soil profile is varied, but generally the surface layer consists of a foot or more of cohesionless sandy loam. A layer of clean sand has been widely found directly under the loam in many excavations, borings, and wells. A high water table causes this sand to be saturated, thus making an ideal situation for liquefaction of the sand during ground shaking, as in an earthquake. The release of pore pressure from the liquefied sand deposit causes venting of water and sand through the overlying materials, which results in mounds of sand, termed sand blows, on the soil surface.

In May 1989, I placed a short advertisement in the valley newspaper, The Pierce County Herald, requesting information from people who had witnessed sand blows or "gushers" during the 1949 or 1965 Puget Sound earthquakes. I received 27 phone calls from persons who saw these phenomena.

The accompanying photographs were taken immediately after the 1949 earthquake by Richard Six, who was at that time a Tacoma police officer. The street flooding is on 4th Avenue NW, just west of the Puyallup school bus garages. The day of the earthquake, April 13, was sunny and dry. Six stated that although there were some broken water mains after the earthquake, he was aware of no ruptured mains in the area of the flooding. This flooding apparently was a product of the "gushers". A heavy layer of sand that erupted from the sand blows was deposited on lawns.

Memories of many events may dim after 40 years, but an earthquake tends to remain imprinted in the mind. I received some vivid and detailed descriptions from those with whom I spoke. One person told me of a crack in a basement floor that allowed liquefied sand to fill the basement to a depth of about 4 feet. Another said sand filled the basement and floated a furnace. There were descriptions of "gushers" in gardens, in front and back yards, and in crawl spaces, as well as of sandy water venting upward to heights of 6 feet or more. Others told of simple bubbling or small spurts of sandy water.

There were many stories of small hills of sand, 7 to 9 inches in diameter and as much as 6 to 9 inches high. A cluster of as many as 20 such hills appeared on front and back lawns or open fields. One woman described the block where the Puyallup High School gymnasium now stands as an active site of sand blows. A single sand blow north of the Puyallup River in the Firwood area was reported to cover an area of 15 to 20 square feet. The owner states he can grow nothing on that spot.

Most of the reported sites of sand blows are in the northwest part of the city on both sides of the railroad tracks and in the farm lands on the north side of the river toward the city of Fife. I received only one report of a sighting near the fairgrounds in Puyallup, but there were many open fields near that site at the time - and few observers. There was one report from Sumner, and a call from a woman in Orting who reported a "gusher" that seemed a bit "oily". An oily sand blow would be atypical.

The 1965 earthquake (Richter magnitude 6.5) evidently caused very little sand blow activity. The only report I received concerning this earthquake involved a considerable amount of sand and water on the Aylen Junior High School playing field. This field is about 150 yards from the site where Six's photographs were taken in 1949.

The pace of development and population increase continues to increase in the Puyallup area. Liquefaction can disrupt building foundations, and can damage roadways and underground utilities. The location of potentially liquefiable sand deposits can play an important part in mitigating this earthquake hazard if appropriate land use and construction practices are undertaken.

Richard Six's photographs of liquefaction phenomena that occurred during the 1949 earthquake on 4th Avenue NW in Puyallup.

Geotechnical Analysis of Liquefaction in Puyallup During the 1949 and 1965 Puget Sound Earthquakes

by

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Liquefaction is a process in which a water-saturated granular (sandy) soil layer loses strength during the strong vibratory shaking of a large earthquake. A liquefied layer and the overlying soil mass can be subject to large lateral displacements, which may then result in the disturbance of building and road foundations, failure of earth-filled dams and levees, and disruption of underground utilities. Liquefaction of a near-surface soil layer is often expressed by the eruption of a sand-water slurry, termed a sand blow, which forms a conical deposit on the ground around the vent. Lateral spreading occurs when blocks of the soil mass overlying a liquefied stratum slide down shallow slopes (0.5° - 3°) toward a free face, such as a river channel or manmade cut. Fracturing and differential settlement of the moving blocks can severely damage structures or pipelines situated in this material. Sand blows and lateral spreading caused by the 1949 magnitude 7.1 and 1965 magnitude 6.5 earthquakes (Figure 1) were widely reported in the Puget Sound region.

The precise locations of liquefaction phenomena that occurred in the city of Puyallup are now available through the efforts of John Shulene (Shulene, 1990). These locations, shown in Figure 2, provide important data for the evaluation of liquefaction hazards in the Puget Sound area. Geotechnical boring data in the northwestern portion of Puyallup were obtained from Ben Peterson of the City of Puyallup Engineering Division. Three potentially liquefiable soil units, described in Figure 3, were identified in these borings using sample descriptions, sieve analyses, and Standard Penetration Test (SPT) data. Two of these units are described as loose, black, fine to coarse clean sands; this is consistent with many of the descriptions of the material that erupted as sand blows at the Puyallup sites during the 1949 earthquake (Edwards, 1951; John Shulene, 1990).

Liquefaction typically occurs in loosely packed, sandy soils that lie below the ground-water table (and consequently are saturated). The geotechnical properties most important in determining a soil's liquefaction capability are the relative density and the amount of silt-sized fraction. Relative density is a measure of the consolidation of a soil. Soils that have a high relative density are tightly packed and have little capability to liquefy. Soils of low relative density are loose and have a high susceptibility for liquefaction during an earthquake. The SPT is the most commonly used method of measuring *in situ* soil density during drilling of a geotechnical boring. In the SPT, a standardized core barrel (a 2-in. outer diameter split-spoon sampler) is driven into the soil mass by a 140-lb hammer falling 30 in. The sampler is driven 6 in. to be properly seated, and then driven another 12 in. The number of hammer blows to drive the sampler this final 12 in. is counted, and this number is termed the SPT blow count (or N-value). SPT blow counts in the range of 1 to 4 indicate a very loose (low density) soil; N-values over 50 indicate a very dense soil which has little susceptibility to liquefaction. Sandy soils in which the grain size is quite uniform (poorly graded) will liquefy more easily than well-graded sandy soils with equivalent N-values. The grading of a soil is measured by passing a soil sample through sieves of various mesh sizes.

Modified from an article in the Washington Geologic Newsletter, 1990, v. 18, no. 3, p. 3-7.

A cumulative weight percentage of grains passing through each of the sieves is graphed as shown in Figure 4. Typically, sandy soils containing more than 40 percent silt fraction (passing a 200 mesh, or grains smaller than approximately 0.075 mm) are considered to have little potential for liquefaction.

Liquefaction during an earthquake is dependent on the level of ground acceleration, the duration of strong shaking, and the depth to the ground-water table, as well as the relative density and grain-size gradation of the soil. Ground acceleration and duration of strong shaking generally increase with increasing earthquake magnitude and decreasing hypocentral distance. By happenstance, Puyallup was at the same hypocentral distance (65 km ± 2 km) for both the 1949 and 1965 earthquakes (Figure 1). Thus, the difference in acceleration and duration of shaking in Puyallup during these two earthquakes depended, to a first approximation, only on the difference in magnitude of these two events.

The depth to the ground-water table is an important factor in assessing liquefaction susceptibility, as liquefiable soils must be saturated. The elevation of the unconfined ground-water table in the Puyallup valley is governed by seasonal recharge and discharge (Walters and Kimmel, 1968). During April, the month in which both the 1949 and 1965 earthquakes occurred, unconfined ground-water levels are primarily determined by the amount of rainfall during the previous wet season. Precipitation records from the Puyallup Experimental Station indicate that the rainy seasons preceding these earthquakes were similar, and that precipitation was somewhat below normal. Thus, the elevation of the unconfined ground-water table during these two earthquakes probably was quite similar.

I postulate that the difference in earthquake magnitude appears to be the most important parameter controlling the extent of soil liquefaction in Puyallup during the 1949 and 1965 events. Liquefaction-related phenomena were commonly reported only during the 1949 earthquake. This suggests that the smaller magnitude 1965 earthquake did not produce the ground acceleration and duration of strong shaking necessary to cause significant liquefaction in Puyallup.

To verify this interpretation, the critical accelerations required to cause liquefaction for a magnitude 7.1 earthquake were calculated using the method of Seed and others (1983) for each SPT N-value from the borings shown in Figure 2. The method of Seed and others (1983) is an empirically-based analysis that uses SPT N-values and grain-size data to determine the critical stress at which a soil will liquefy during an earthquake of a given magnitude. This critical stress is compared to the stress imparted at the SPT sample depth by the acceleration of the overlying soil mass during this earthquake. If the imparted stress is greater than the SPT-based critical acceleration, then the soil is considered to have liquefied. Critical accelerations were calculated using the SPT N-values from the borings shown in Figures 5a and 5b and soils data summarized in Figure 3 . To account for the seasonal variation of the unconfined aquifer, ground-water elevations in these borings were assumed to have been 5 ft higher during the April earthquakes than during September, 1978, when they were measured.

The above analysis was performed using SPT N-values from profile 1 (Figure 5a) assuming a magnitude 7.1 earthquake. These results indicate that the critical accelerations range from 0.10 g to 0.20 g for soil units 4 and 6, and that no other soil units could liquefy. During the 1965 earthquake a peak ground acceleration (PGA) of 0.08 g was measured at both Tacoma and Seattle, which were at hypocentral distances of 65 km and 67 km, respectively. These measured PGA's are less than the minimum critical acceleration (0.10 g) estimated from the geotechnical boring data and shown in Figure 5a. In addition, the smaller magnitude 1965 earthquake is estimated to have had approximately 75 percent of the number of cycles of strong ground shaking in comparison to the 1949 event (Seed and Idriss, 1982). We conclude that the 1965 earthquake did not produce the ground acceleration and duration of strong shaking necessary to cause liquefaction in Puyallup. Shulene (1990) received only one report of a sand blow during the 1965 event (Figure 2), compared to eighteen reports for the 1949 earthquake.

A preliminary ground motion attenuation relationship specific to Puget Sound moderate depth earthquakes predicts a mean PGA of 0.17 g at Puyallup during the 1949 earthquake (C. B. Crouse, oral commun., 1990). Using this 0.17 g as the PGA in Puyallup during the 1949 earthquake, significant near-surface liquefaction would have occurred only in boring B-9, which has more than 15 ft of soil that is liquefiable at accelerations of 0.17 g or less (Figure 5a). Boring B-9 is located just east of the densest occurrence of reported liquefaction sites during the 1949 event (Figure 2).

In the borings shown in profile 2 (Figure 5b), only soil unit 2 has a significant thickness of potentially liquefiable material. Most soil units along this profile are silts and clays and are consequently not liquefiable. Oddly, this profile is near the reported area of most intense liquefaction during the 1949 earthquake. Trenching during underground utility installation has shown that the continuity of these soil units can be quite variable over short distances (Ben Peterson, oral commun., 1990). Interfingering of these silty/clayey soils with the liquefiable black sands may explain the pattern of liquefaction reports in the vicinity of profile 2. Alternatively, the sand blows reported in the area adjacent to this profile may have resulted from liquefaction of soil unit 2.

This study demonstrates the applicability of the method of Seed and others (1983) to liquefaction assessment of shallow, flat-lying soil sites in the Puget Sound area. Further, plausible bounds on PGA can be estimated at identified liquefaction sites for the 1949 and 1965 Puget Sound earthquakes using this methodology. Geotechnical analysis of other identified sites of liquefaction that occurred during these earthquakes is planned. An on-going sedimentological study of some liquefiable black sand units in Puyallup shows that they are primarily composed of sub-angular to angular lithic and mineral fragments of Mount Rainier andesite, and may represent highly or hyper-concentrated flow deposits derived from Mount Rainier lahars (Pat Pringle, oral commun., 1990).

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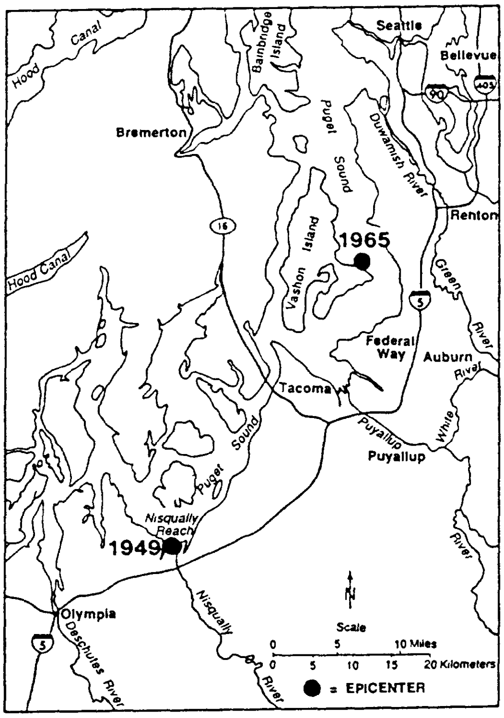


Figure 1. Epicenters of the 1949 magnitude 7.1 and 1965 magnitude 6.5 Puget Sound earthquakes. The hypocentral distance from both of these earthquakes to the city of Puyallup is 65 ± 2 km.

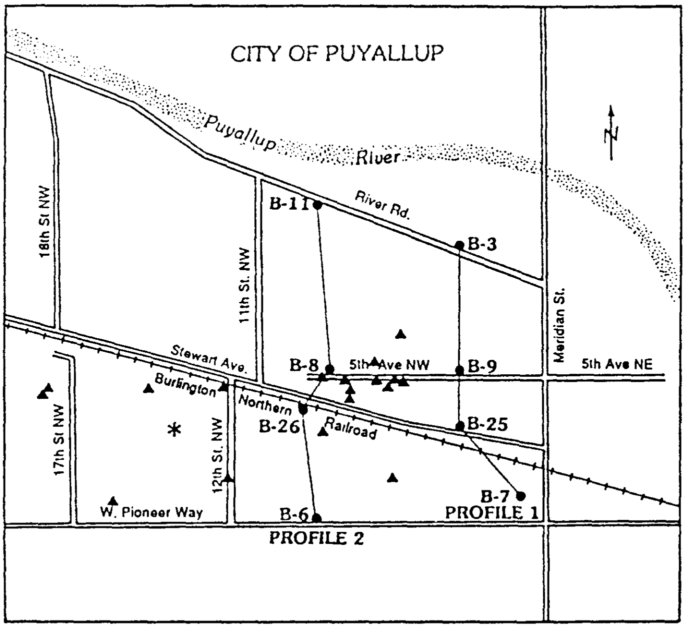


Figure 2. Locations of reported liquefaction phenomena in Puyallup during the 1949 (triangles) and 1965 (asterisk) earthquakes. Most of these reports described sand blows and "gushers" (Shulene, 1990) during the earthquakes. Note that there was only one reported instance of liquefaction during the 1965 event. The geotechnical borings (designated as B-xx) and alignment of profiles 1 and 2 are also shown.

Soil unit	Description	γ (pcf)	γsat (pcf)	% fines	Liquefiable
1	Fill, consisting of dense brown, silty, sandy gravels with scattered cobbles and asphalt blocks	130	140	>35	NO
2	Loose, brown to gray, silty, fine to medium sand with scattered gravel and organic material	90	105	35	YES
3	Very soft to medium stiff, gray-brown, clayey or, in places, sandy silt with organics	100	110	65	NO
4	Very loose to medium dense, black, clean, fine to medium sand with some lenses of silty fine sand and sandy silt	100	120	5	YES
5	Medium dense, gray, silty fine sand and sandy clayey silt with scattered organic material	110	130	>35	NO
6	Loose to medium dense, black, fine to coarse sand with scattered gravel	110	130	5	YES

Figure 3. Descriptions and material properties for the six soil units encountered in the geotechnical borings used in this study. γ is the *in situ* unit weight (bulk density) of the soil unit measured in pounds per cubic foot (pcf); γsat is the saturated unit weight of the soil unit.

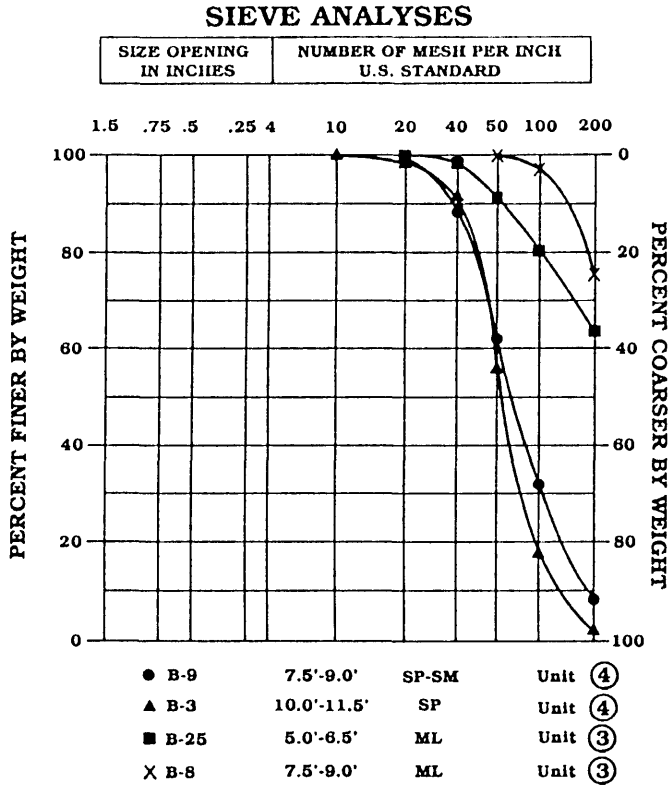
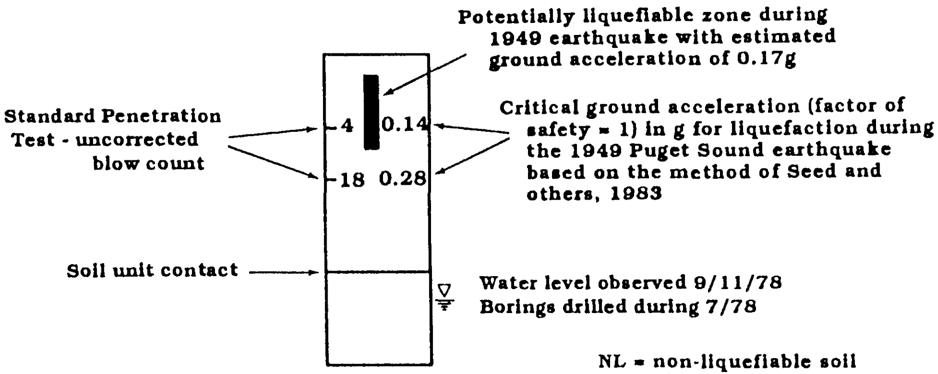


Figure 4. Sieve analysis of four samples from the borings shown in Figure 2. The two samples from soil unit 3 contain more than 60 percent silt fraction and are unlikely to liquefy during an earthquake. The two samples from soil unit 4 are poorly graded and contain less than 10 percent silt fraction. This unit has a high liquefaction potential where SPT blow count data indicate a low relative density.

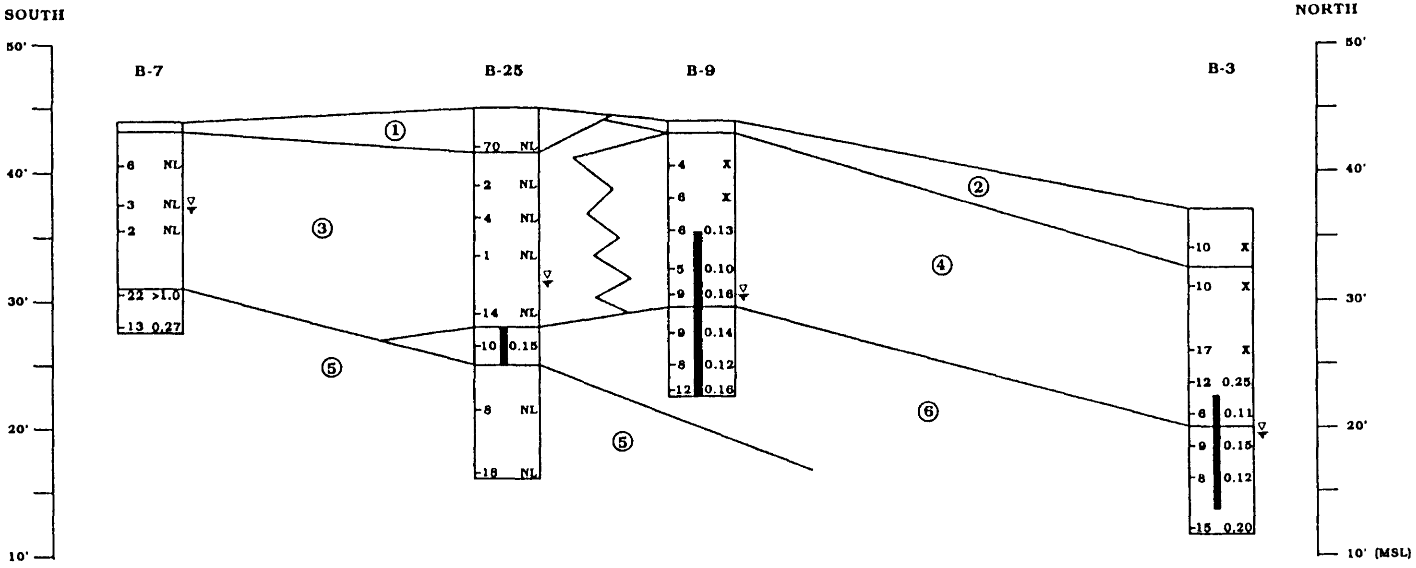
The second column from left shows the depth interval of the soil sample; third column is the soil classification, using the Unified Soil Classification System. In this system, soil types beginning with the letter S are sands (less than 50 percent silt fraction); those beginning with the letter M are silts (greater than 50 percent silt fraction).

Figure 5. (a) **Profile 1** below (location shown in Fig. 2) shows a thick section of liquefiable black sand (units 4 and 6) that is the likely source of the sand blows observed in this area of Puyallup. Note that this profile is slightly east of the most intense area of liquefaction reported during the 1949 earthquake. (b) Most soil units along **profile 2** below are silts and clays, and are consequently not liquefiable, although this profile is in the area of most intense reported liquefaction. Trenching performed during underground utility installation has shown that the continuity of these soil units is varied over short distances. An explanation of boring log data is presented below.

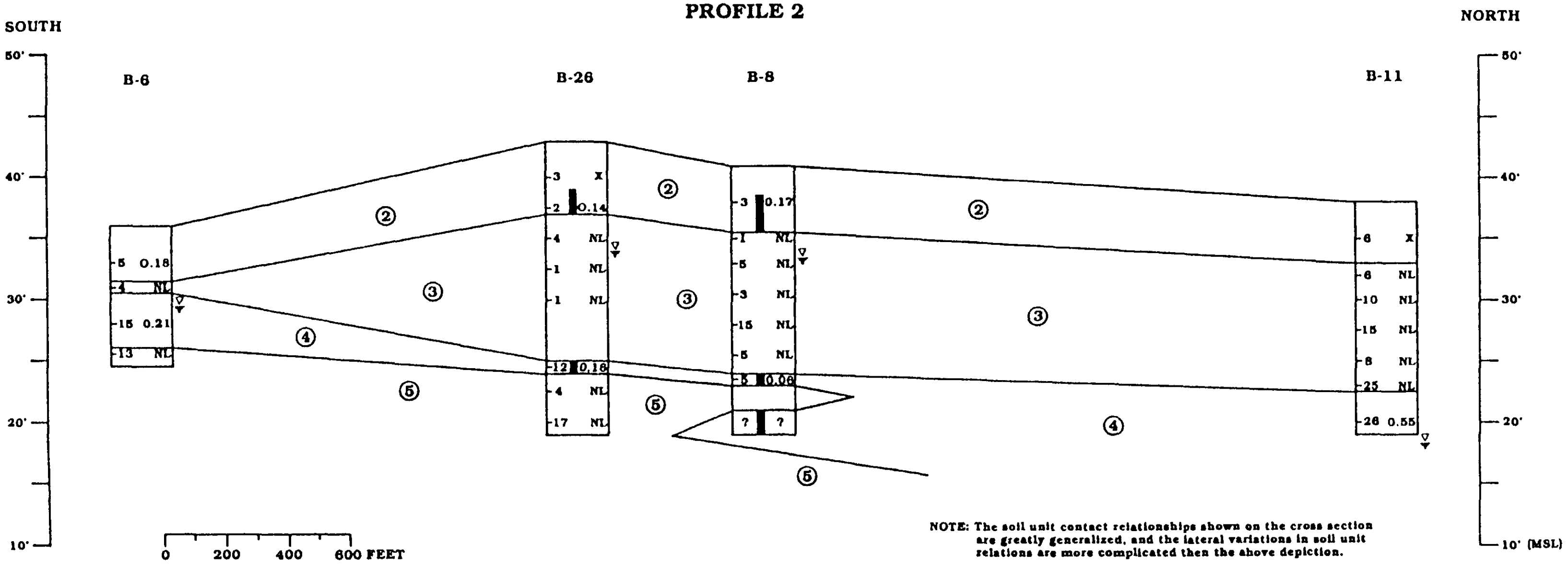


NL = non-liquefiable soil
X = above assumed ground water level during 1949 and 1965 earthquakes

PROFILE 1



PROFILE 2



NOTE: The soil unit contact relationships shown on the cross section are greatly generalized, and the lateral variations in soil unit relations are more complicated then the above depiction.

Liquefaction Susceptibility Maps for the Seattle North and South, Washington Quandrangles

William J. Perkins

W. Paul Grant

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ABSTRACT

To help planners and engineers mitigate future liquefaction damage, we present liquefaction susceptibility maps for the Seattle North and South Quandrangles. The maps delineate areas of different liquefactiion susceptibility.

We collected bore hole data (i.e. SPT blow count) and, using Seed's simplified procedure, determined the cumulative thickness of liquefiable sediment in each boring. Thicknesses were calculated for various scenario earthquakes. We use thickness of liquefiable sediment as a measure of liquefaction susceptibility because damage is likely a function of the thickness. Filled areas along Elliot Bay and the old Duwamish Tide Flats are the most susceptible to liquefaction. Recent lacustrine, alluvial, and beach deposits also have significant susceptibility. Pliestocene or older sediments that have been glacially overridden are not susceptible to liquefaction.

The Standard Penetration Versus Depth Relations of Quaternary Glacial and Nonglacial Deposits in the Southern Seattle Area, Washington: Implications for Studies of Liquefaction Susceptibility

By

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Abstract

The southern Seattle area is underlain by a lithologically complex sequence of glacial and nonglacial gravels, sands, and muds of Quaternary age that show a wide range of physical properties. In this study, we explore the usefulness of one particular property, standard penetration, as an indicator of near-surface liquefaction potential for the water-saturated unconsolidated sediments in theSeattle area. Following Youd and others (1975), we examine the depth relations of standard penetration data for individual, mappable geologic units and compare the slopes of plots of standard penetration values, in blows per foot, versus depth for various lithologic and stratigraphic units in order to determine a relative ranking of liquefaction potential for the units considered.

The standard penetration data are derived from 166 drill-holes in the Seattle South and Duwamish Head 7 1/2 ' quadrangles, and, except where noted, are confined to measurements made while driving a 2-inch outside diameter, split-spoon sampler with a 140 pound hammer dropped 30 inches. Mappable geologic units are grouped into the following stratigraphic scheme: artificial fill (including circa 1900 hydraulic fill), Holocene alluvium, Vashon recessional outwash deposits, Vashon till, Vashon advance outwash deposits, pre-Vashon nonglacial deposits, and pre-Vashon glacial deposits. The units are further subdivided into dominantly fine-grained (muddy) and dominantly coarse-grained (sandy and gravelly) sediment types. Plots of standard penetration, measured in blows per foot, versus depth for samples within 45 feet of the ground surface were prepared for each stratigraphic category, with muddy units plotted separate from sandy and gravelly units.

The resulting plots show a great deal of variability through the studied depth range for any given stratigraphic class. The Vashon glacial deposits, including tills and associated outwash, show considerable variability as is typical of standard penetration data derived from gravelly sediments. In general, plots for fill and alluvium display

less variability than do the plots for the glacial sediments, reflecting the more homogeneous nature of those sediments. Using the slope of the standard penetration-depth plot as a measure of a unit's sensitivity yields a crude three-fold classification, with muddy alluvium and muddy fill being most sensitive, muddy recessional and advance outwash, sandy alluvium, and older glacial deposits making up an intermediate category, and sandy recessional and advance outwash, Vashon till , and pre-Vashon nonglacial mud and sand comprising the most stable category. Interestingly, the plot for sandy fill displays a relatively steep slope, perhaps reflecting improved techniques used for emplacing fill in the recent past.

Correcting standard penetration data for overburden pressure introduces considerable scatter into the depth plots and destroys the three-fold classification presented by the uncorrected data. Sandy sediments do show a systematic increase in the slope of the standard penetration-depth plot that agrees with geologic conditions. For example, sandy sediments which have been overlain by Vashon ice are less sensitive than sandy sediments which have accumulated since the withdrawal of Vashon ice. Plots of muddy sediments using corrected data show no such tendency. Corrected blow count values usually decrease slightly or remain unchanged with depth for muddy sediments regardless of stratigraphic situation.

Introduction

Liquefaction of water-saturated sediment during even moderate earthquakes is a commonly observed phenomenon, and has caused much of the destruction during many large earthquakes (National Research Council Committee on Earthquake Engineering, 1985, p. 14-16). The term liquefaction is used in this paper to mean "...the transformation of a granular material from a solid state to a liquefied state as a consequence of increased pore-water pressures" (Youd, 1973, p. 3). The major elements necessary to induce liquefaction during earthquakes are 1) the presence of near-surface, water-saturated, geologic materials with physical properties that enhance the build up of intergranular pore pressure during particle rearrangement and 2) seismic sources sufficiently large or nearby to produce the cyclic accelerations needed to cause particle rearrangement.

Previous Earthquakes and Liquefaction in the Puget Sound Area

Liquefaction has been observed during at least 3 past earthquakes in the Puget Sound area (fig. 1a) and ground failure has taken place (table 1). Numerous cases of geysering, sand boil activity, and settlement of building

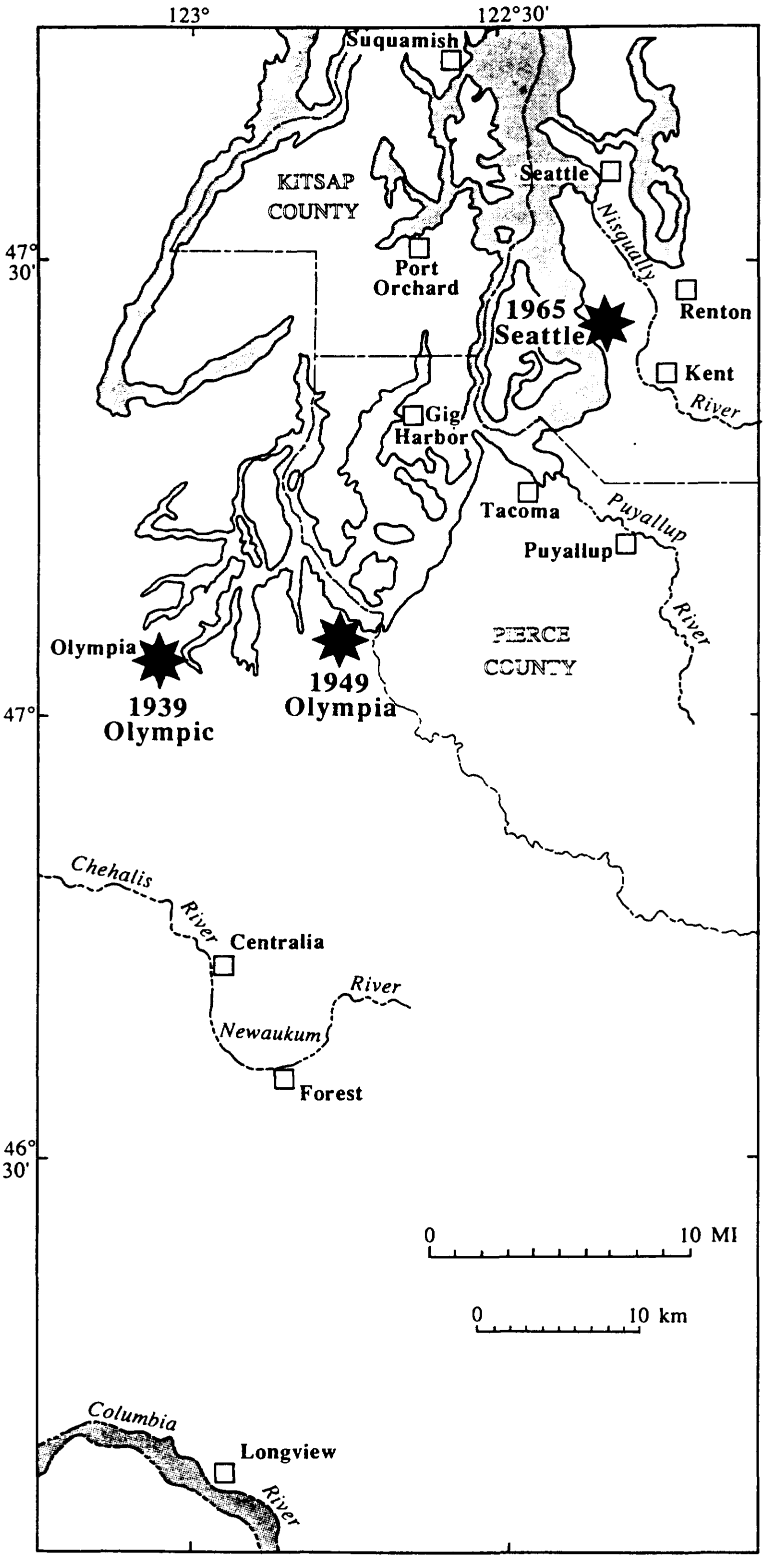

Figure 1A: Locations of past earthquakes in Puget Sound and localities in Puget Sound region.

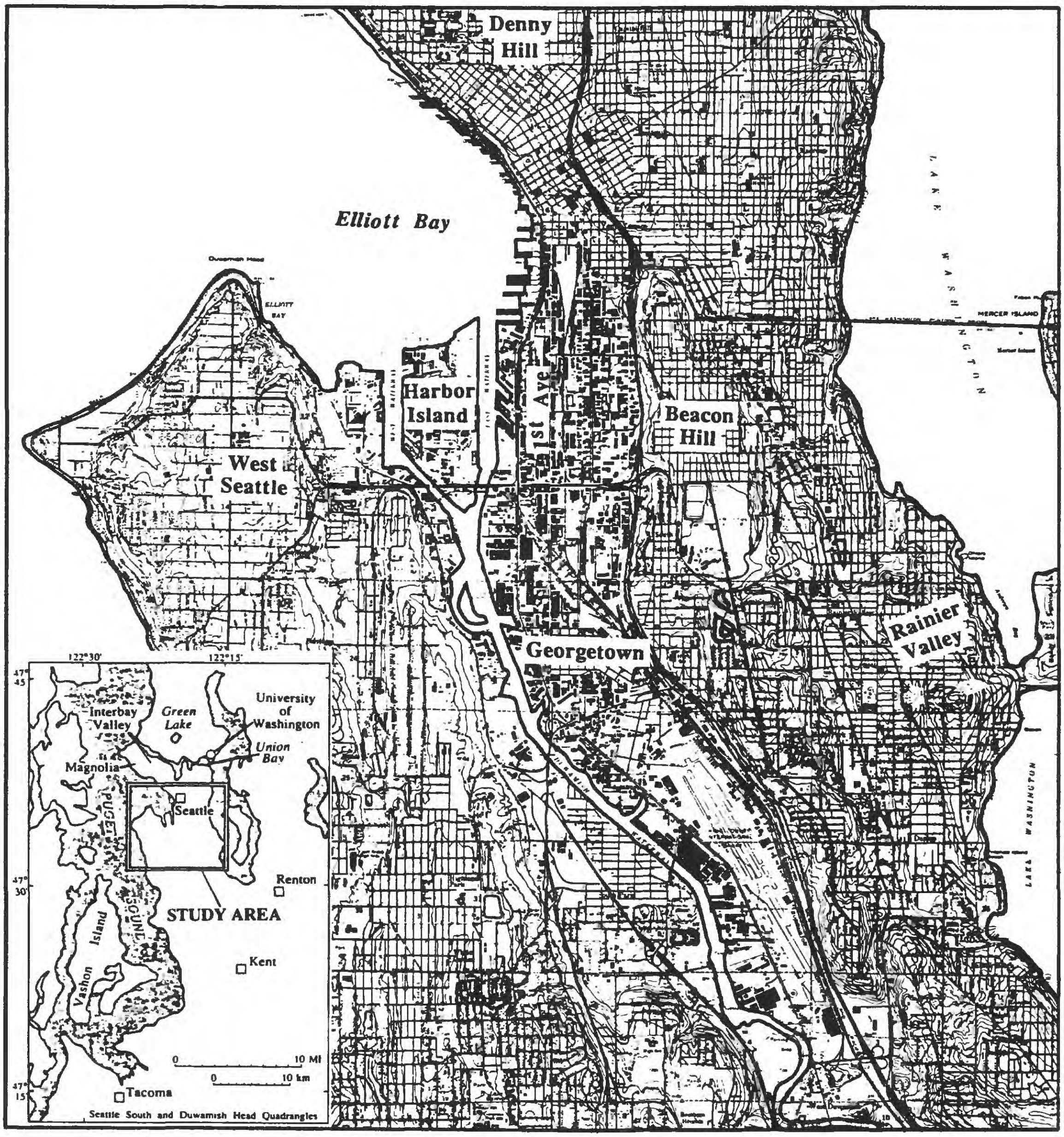

Figure 1B: Location of study area and localities in central Puget Sound and greater Seattle area.

TABLE 1: QUOTATIONS REGARDING LIQUEFACTION AND GROUND SETTLEMENT IN THE PUGET SOUND REGION DURING EARTHQUAKES

(see figure 1 for general locations of areas being discussed; references to figure numbers in quotations refer to original article)

1. 1939, Port Orchard (fig. 1a) "The Washington State Highway Department sent in pictures of a paved highway near Port Orchard, Washington. Investigations of this crack showed it to be on the surface of a fill approximately 260 yards in length and 4 yards thick in the center. A bed of quicksand fed continuously by many springs a short distance away on the uphill side underlies the sand and gravel of the fill."
(Coombs and Barksdale, 1942, p.3-4.)

2. 1949, Centralia (fig. 1a) "Water and sand spouted from the ground."
(Ulrich, 1949, p. 10)

3. 1949, Olympia (fig. 1a) "A large portion of a sandy spit jutting into Puget Sound north of Olympia disappeared during the earthquake."
(Ulrich, 1949, p. 10.)

4. 1949, Puyallup (fig. 1a) "Geysers ejected water and sand."
(Ulrich, 1949, p. 10.)

5. 1949, Seattle (fig. 1a) "Large cracks in filled ground, some cracking of pavement, and water spouted six feet or more from many ground cracks."
(Ulrich, 1949, P. 10.)

6. 1949, Puyallup (fig. 1a) "Geysers of muddy water rose in many yards to heights as much as 3 ft., forming circular deposits of black, sandy clay while in some basements the surging earth pushed the floors up, crushing the furnaces and piping against the joists above."
(Edwards, 1951, p. 6.)

7. 1949, Puyallup (fig. 1a) "During this temporary flotation of the surface areas the horizontal movements of the quake created compression zones in some areas. In parts of Puyallup they were so strong that basement floors were lifted like pistons in a pump as much as 16 in. so that furnaces and pipes were crushed against the joists above and did not recede, while stud or post supports were forced through floors to as much as 8 in. above. These compression areas and adjoining tension areas caused soil movements which pulled apart or broke underground piping or conduit systems."
(Edwards, 1951, p. 7.)

(Table 1 continued)

8. 1949, Seattle (fig. 1a) "The consolidation of the alluvium by the vibratory action of the earthquake freed water which previously had been retained in the spaces between the soil particles. The presence of this released water under the pressure of the surface soil or mat was evidenced by the geysers of water and mud which spurted from the ground reportedly as high as 3 ft., which flowed continuously for as long as 24 hours, and which filled basements in the Sears Roebuck area of 1st Ave. S. (fig. 1b) in Seattle with sand."
(Edwards, 1951, p. 7.)

9. 1949, Frozen waves "Visible waves traveling over the earth's surface (often reported but pooh-poohed or disbelieved by seismologists at the times of other earthquakes) were seen here and in addition left their imprint on the sands and soils of the soft areas in several locations. On the Tacoma lowlands (fig. 1a), definite though slight parallel ridges about 12 ft. apart were left. In a freshly plowed, disked and leveled field near Kent (fig. 1a) definite waves with crests about 6 in. high and 30 ft. apart resulted, and on a black-topped road in Pierce County (fig. 1a), according to the county engineer, troughs were evident afterward extending diagonally across the pavement for 1/4 mile having a crest-to-trough height of 2 to 3 in."
(Edwards, 1951, p. 7.)

10. 1949, Seattle (fig. 1a) "At one building in Seattle the ground settled and was washed from under a footing by escaping ground water. At other buildings, particularly back of bulkheads along and in waterfront structures, substantial settlement occurred, breaking water mains and sewers, pulling electric conduit apart as much as a foot, and causing similar damage to other underground structures."
(Edwards, 1951, p. 7.)

11. 1949, Olympia (fig. 1a) "A large portion of a sandy spit jutting into Puget Sound north of Olympia disappeared during the earthquake."
(Murphy and Ulrich, 1951, p. 20.)
* Repeats 3.

12. 1949, Centralia (fig. 1a) "1 church condemned, continued settling of ground caused extensive damage.... Water and sand spouted from the ground.... Four miles southwest of town, water spouted 18 inches high in middle of field, leaving a very fine sand formation for a considerable space around each hole, the holes varying from 1 to 3 inches in diameter. Water spouted from inch-wide crack 8 or 10 feet long. Caretaker on Newaukum River (fig. 1a) intake noticed gas or air boiling up through water in the river."
(Murphy and Ulrich, 1951, p. 21 [also p.22].)
* 2 repeats a portion of this entry.

(Table 1 continued)

13. 1949, Forest (fig. 1a) "At the Niels Paulsen farm, two springs appeared; the first came in the 1946 temblor and another appeared close by during this shock."
(Murphy and Ulrich, 1951, p. 21.)

14. 1949, Longview (fig. 1a) "Water came through cracks in sizable quantity for about 3 hours after the shock, stopped entirely about 12 hours after the shock. Water and sand spouted from the ground."
(Murphy and Ulrich, 1951, p. 21)

15. 1949, Puyallup (fig. 1a) "Geysers erupted in fields bringing up much sand."
(Murphy and Ulrich, 1951, p. 22.)

16. 1949, Seattle (fig. 1a) "Water spouted 6 feet or more from many ground cracks."
(Murphy and Ulrich, 1951, p. 22.)

17. 1949, Seattle (south section) (fig. 1a) "Water observed spouting 6 feet or more from many ground faults. Blue silt forced up through minor cracks in basement floors. Many basements completely filled with silt, with floors forced upwards until failure resulted."
(Murphy and Ulrich, 1951, p. 22.)

18. 1965, Harbor Island (fig. 1b) "A second instance of damage on Harbor Island occurred at Piers #15 and #16 as shown in figure 26. These piers shifted toward the water by about one foot due to the soil losing much or all of its strength, or partially liquifying, and pushing the dock toward the water."
(Steinbrugge and Cloud, 1965, p. 78.)

19. 1965, Duwamish River (fig. 1b) "The low-lying filled areas along the Duwamish River and its mouth settled and were the locations of considerable building damages."
(von Hake and Cloud, 1967, p. 37)

20. 1965, Harbor Island (fig. 1b) "Piers 15 and 16 on Harbor Island shifted toward the water by about 1 foot due to the soil losing much or all of its strength, or partially liquifying and pushing the dock toward the water."
(von Hake and Cloud, 1967, p. 37.)
*Repeat of entry 18.

21. 1965, Port of Seattle, (fig. 1a) "Pier 5, where construction projects were underway, was hardest hit. The bulkhead and the fill behind it settled, the fill dropping 6 inches to 2 feet for a width of 25 to 40 feet. The bulkhead was reported to be 6 to 8 inches out of line. Several Port piers suffered similar damage. Pier 20 at the East Waterway Terminal settled."
(von Hake and Cloud, 1967, p. 37.)

(Table 1 continued)

22. 1965, University of Washington, (fig. 1b inset) "...a fissure opened in the practice field at the University. Underground pressure from the shock sent sand spurting in a 100-foot-long- zig-zag stretch on the lower football field. Behind the men's pool, areas of the ground dropped as much as a foot. Dirt floor sections in the Hec Edmondson Pavilion also sank slightly."
(von Hake and Cloud, 1967, p. 39.)

23. 1965, Gig Harbor (fig. 1a) "Press reported a part of Crescent Lake Road, west of Gig Harbor, sank out of sight and was covered with water."
(von Hake and Cloud, 1967, p. 41.)

24. 1965, Renton (fig. 1a) "At the Boeing Aircraft Plant,... floors settled away from the foundation piling;"
(von Hake and Cloud, 1967, p. 47.)

25. 1965, Suquamish (fig. 1a) "The press reported the shoreline of Suquamish, in northeast Kitsap County, heaved up 15 feet in places. A 2-story beach house was demolished and trees were uprooted....A nearby resident reported the beach below the bank heaved in a wave-like motion and rolled like a wave toward the bank. The beach close under the bank seemed to sink several feet. 'The earthquake left a high beach , most of which was washed out by the high tide.'"
(von Hake and Cloud, 1967, p. 48.)

26. 1965, Vashon Island "Press reports stated the Burton-Tahlequah Road settled."
(von Hake and Cloud, 1967, p. 49.)
27. 1965, Georgetown (fig. 1b) "About a mile still farther south, however, property loss at the Boeing Company was reported to be high. Much destruction there resulted from subsidence, but numerous broken windows attested to vibration damage as well."
(Mullineaux and others, 1967, p. D188.)

28. 1965, Interbay Valley (fig. 1b inset) "At the northern end of Interbay Valley (fig. 1), broken windows and cracked walls in commercial and industrial buildings probably resulted from vibration, and differential subsidence caused some foundation damage."
(Mullineaux and others, 1967, p. D188)

29. 1965, Green Lake (fig. 1b inset) "Just south of Green Lake (fig. 1), lacustrine sediments overlain by thin fill subsided, apparently as a result of both compaction and lateral movement downslope toward the lake. Here, ground cracks opened as much as 2 inches, breaking the foundation of a small building, fracturing walks and paving, and breaking utility lines."
(Mullineaux and others, 1967, p. D188)

30. 1965, Union Bay (fig. 1b inset) "North of Union Bay, a broad fill over alluvial and lacustrine sediments subsided and exhibited scattered ground cracks and sand mounds."
(Mullineaux and others, 1967, p. D188.)

foundations were reported from the 1949 Olympia (M=7.1, Nuttli, 1952) and 1965 Seattle (M=6.5, Algermissen and Harding, 1965) earthquakes. Lateral spreading most likely developed during these earthquakes (table 1; entries 7, 18, 20, 29) as well as during the 1939 Olympic earthquake (MM Intensity VII, Coombs and Barksdale, 1942) (table 1; entry 1).

Generally, the observed instances of liquefaction and liquefaction-related damage have been confined to areas of fill (Union Bay (fig. 1b inset), Harbor Island (fig. 1b)), young alluvium (Duwamish River Valley (fig. 1b), Interbay Valley (fig. 1b inset), Newaukum River floodplain (fig. 1a), Chehalis River floodplain (fig. 1a)), young lacustrine sediments (Green Lake (fig. 1b inset)), young estuarine deposits (mouths of Duwamish and Puyallup Rivers (fig. 1a)), or young beach deposits (Suquamish (fig 1a)).

Use of Standard Penetration Information in the Prediction of Liquefaction Potential

Many techniques for evaluating the liquefaction hazards associated with earthquakes have been proposed recently (see Chapter 4, National Research Council Committee on Earthquake Engineering, 1985, for a summary). Many of the proposed techniques rely on using data derived from Standard Penetration Tests (SPT) to either 1) assign geologic units to supposed liquefaction susceptibility classes on the basis of their SPT properties (Youd and Perkins, 1978; Youd and others, 1978; Youd and Perkins, 1987) or 2) empirically evaluate liquefaction behavior of materials during actual earthquakes (Seed and Idriss, 1971; Seed and others, 1983)

Problems exist with making and interpreting SPT measurements (Schmertmann and Palacios, 1979; Kovacs and Salomone, 1982) and other material properties (shear wave velocity, cone penetration resistance) may predict liquefaction susceptibility as well or better than SPT measurements. Still, it is attractive, particularly for reconnaissance or large-scale planning studies, to use the large number of SPT tests available in most urban areas throughout the country during evaluations of liquefaction susceptibility.

Purpose of This Study

This study examines the variability of SPT data derived from 166 geotechnical drill-holes through glacial and nonglacial deposits in the southern Seattle area. Standard penetration data (blow counts) are plotted against depth for fine-grained (muddy) and coarse-grained (sandy and gravelly) categories of the various stratigraphic units penetrated in order to evaluate the usefulness of the geologic units as predictors of liquefaction susceptibility. Plots and

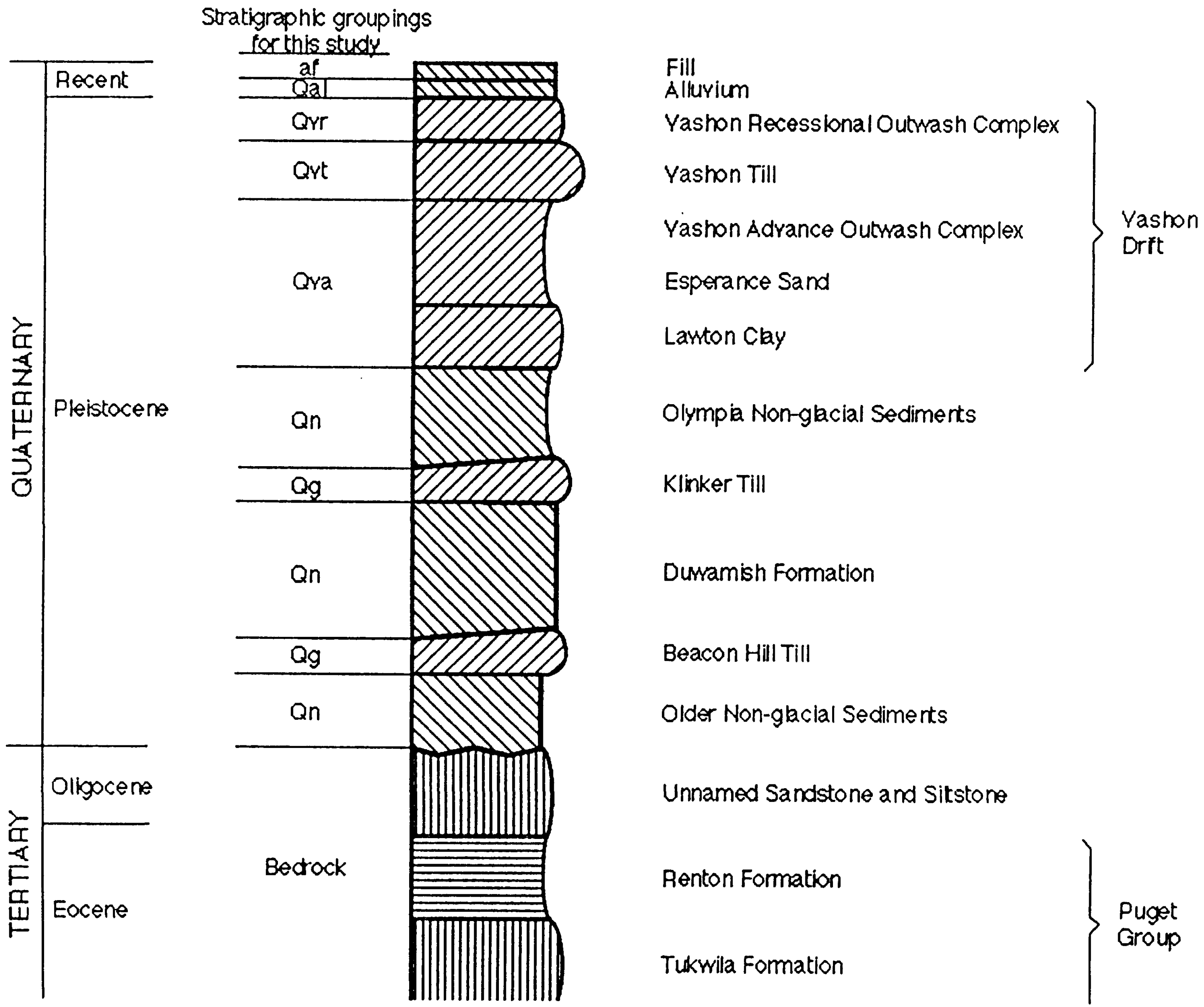

Figure 2: Generalized stratigraphic section, southern Seattle area.

TABLE 2: SUMMARY OF BLOW COUNT VERSUS DEPTH PLOT PARAMETERS

Artificial Fill

	Sand (N[1]=101)		**Mud** (N=14)	
	Uncor.[1]	Cor.[1]	Uncor.	Cor.
Slope[1]	0.16	-0.50	0.07	-0.1
Intercept[1]	12.02	21.9	5.42	8.9
R[1]	0.07	0.15	0.10	0.11

Holocene Alluvium

	Sand (N=161)		**Mud** (N=49)	
	Uncor.	Cor.	Uncor.	Cor.
Slope	0.61	0.22	0.10	-0.06
Intercept	6.08	13.2	5.7	9.12
R	0.40	0.18	0.17	0.11

Vashon Recessional Outwash

	Sand (N=76)		**Mud** (N=29)	
	Uncor.	Cor.	Uncor.	Cor.
Slope	1.2	0.52	0.48	-0.37
Intercept	16.7	28.7	22.2	40.3
R	0.51	0.23	0.29	0.17

Glacial Till

	Vashon (N=82)		**Pre-Vashon** (N=29)	
	Uncor.	Cor.	Uncor.	Cor.
Slope	2.1	0.35	0.49	-0.32
Intercept	39.6	71.3	35.8	51.2
R	0.58	0.09	0.17	0.11

Vashon Advance Outwash

	Sand (N=155)		**Mud** (N=137)	
	Uncor.	Cor.	Uncor.	Cor.
Slope	1.4	0.661	0.57	-0.06
Intercept	16.6	28.9	21.0	33.2
R	0.50	0.24	0.28	0.03

Pre-Vashon Nonglacial

	Sand (N=14)		**Mud** (N=45)	
	Uncor.	Cor.	Uncor.	Cor.
Slope	2.0	1.1	1.3	0.72
Intercept	-5.6	8.2	-2.09	6.9
R	0.81	0.41	0.62	0.46

---

[1] N = number of samples; Cor. = blow count values corrected for overburden pressure; Uncor. = raw blow count values; Slope = slope of regression line for blow count vs. depth plot; Intercept = intercept on blow count axis of regression line for blow count vs. depth plot; R = square root of coefficient of determination (R^2) for blow count vs. depth plot

summary statistics are presented for SPT data that have been corrected for effective overburden stress as well as for uncorrected SPT data.

We would like to thank Les Youd (University of Utah), John Tinsley (USGS), and Paul Grant (Shannon and Wilson, Inc.) for numerous informal discussions and suggestions regarding liquefaction susceptibility and its relationship to geologic materials.

Geology of the Southern Seattle Area

The southern Seattle area is underlain by a sequence of relatively flat-lying Quaternary glacial and nonglacial deposits that is, in turn, underlain by steeply-dipping, faulted volcanic and sedimentary rocks of Tertiary age. Although flat-lying, the Quaternary sediments are separated internally by contacts that often exhibit considerable relief. Over 1000 meters of relief exists on the surface separating the Tertiary rocks from the Quaternary sediments between downtown Seattle and the hills of West Seattle (fig. 1b) or Magnolia (fig. 1b inset) (Yount and others, 1985). As much as 75 meters of Holocene alluvium and estuarine sediment fill the Duwamish River valley near its mouth (Yount, 1983) with the bluffs of Beacon Hill (fig. 1b) standing nearly 100 meters above the floodplain. This relief was sculpted into the underlying sediments as ice overrode the Seattle area during the last (Fraser) glaciation of Puget Sound (Crandell and others, 1965).

Figure 2 depicts the sequence of glacial and nonglacial deposits underlying the southern Seattle area. The volcanic rocks of the Tukwila Formation and arkosic sandstones of the Renton Formation are overlain by arkosic and volcaniclastic Oligocene sandstones and siltstones. A sequence of glacial and nonglacial sediments that predate the deposits of the last glaciation sit unconformably atop the Tertiary bedrock units. Two old tills and associated minor glaciofluvial sediment are interposed between three sequences of nonglacial alluvial and lacustrine sediment. The lowest till, the Beacon Hill Till[1], separates an unnamed underlying nonglacial unit from the overlying Duwamish Formation. The Duwamish Formation contains compact lacustrine and fluvial medium- to fine-grained sand and peaty silt and clay. One cedar branch enclosed in sediments of the Duwamish Formation from a building excavation in downtown Seattle yielded a C-14 age of greater than 42,000

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[1] Unless otherwise indicated, names of the Pleistocene units follow Stark and Mullineaux, 1950 for the pre-Vashon units and Mullineaux and others, 1965 for the Vashon units.

years (Marsters and others, 1969). The Klinker Till overlies the Duwamish Formation in the west wall of the Duwamish River valley. This till is, in turn, overlain by fluvial and lacustrine sands and silts of the Olympia Interglacial interval (Armstrong and others, 1965). Carbon-14 dates from organic debris in Olympia nonglacial sediments range from 15,000 ± 400 to 24,100 ± 900 years BP (Mullineaux and others, 1965).

The Olympia sediments grade upward into sediments that record the advance and retreat of the Cordilleran Icesheet into Puget Sound during the last glaciation. These sediments, termed the Vashon Drift (Mullineaux and others, 1965) consist of fine-grained lacustrine silts and clays of the Lawton Clay Member and well-sorted fine- to medium-grained glaciofluvial sands of the Esperance Sand Member. The Esperance Sand grades upward into coarse-grained sandy and gravelly outwash that accumulated in front of the advancing icesheet. Vashon Till caps the complex of advance proglacial and outwash facies sediments. Recessional outwash deposits, made up of moderately- to poorly-sorted sand and gravel, overlie the Vashon Till in some portions of the southern Seattle area. Minor amounts of fine-grained sediment accumulated in bogs and small lakes after ice retreat. Radiocarbon dates from organic material in such sediments range from 12,300 ± 200 to 14,000 ± 900 years BP (Mullineaux and others, 1965).

Accumulation of alluvial, lacustrine, and estuarine deposits has continued throughout the Holocene in topographic lows in and around Seattle. The greatest thickness of Holocene sediment in southern Seattle occurs in the mouth of the Duwamish River. Rainier Valley (fig. 1b) and Interbay Valley also contain significant thicknesses of alluvial sediment. A large portion of the low-lying areas in the mouth of the Duwamish River and along the shores of Elliot Bay (fig. 1b) have been reclaimed by the addition of fill. Filling began near the turn of the century as street grades were lowered in the Denny Hill (fig. 1b) area north of downtown Seattle and the removed debris was sluiced to the waterfront region. Major construction projects which include emplacement of large amounts of fill have taken place in the vicinity of the Duwamish River mouth, including Harbor Island (fig. 1b).

Groupings of Geologic Units

The stratigraphic sequence penetrated by each drill-hole utilized in the study has been interpreted, to a large degree, from the projection into the subsurface of nearby mapped surface units. Lithologic descriptions and physical properties given in the drill-hole logs also were used to assign the drilled units to a particular stratigraphic interval. A companion report (Yount and others, in press) presents the lithologic and standard penetration data utilized in a standard format for all 166 drill-holes investigated.

To assess standard penetration values in various geologic units the following groupings of stratigraphic units was established (also see figure 2):

Bedrock

All Pre-Vashon Nonglacial Deposits = Qn

All Pre-Vashon Glacial Deposits = Qg

Vashon Advance Outwash Deposits,
including Lawton Clay and Esperance Sand = Qva

Vashon Till = Qvt

Vashon Recessional Deposits = Qvr

Holocene Alluvium and Estuarine Deposits = Qal

Artificial Fill = af

These classes are further subdivided into dominantly fine-grained (muddy) and dominantly coarse-grained (sandy and gravelly) types for the plots of standard penetration versus depth that follow. In past earthquakes the latter two types of deposits (Qal and af) have exhibited liquefaction-related phenomena.

Nature and Distribution of Standard Penetration Data

One hundred and sixty-six drill-holes from the Seattle South and adjacent Duwamish Head 7½′ quadrangle map areas provide the data for this study (Yount and others, in press). Unless otherwise indicated in the original data, all standard penetration values used in this study were obtained with a 2-inch outside diameter split-spoon sampler, driven by a 140 pound hammer dropped 30 inches. Data were grouped by appropriate stratigraphic interval and texture and plotted against depth in the interval from the ground surface to a depth of 45 feet.

Correction of Blow Counts

Blow count data were corrected to a standard effective overburden stress of 1 ton/ft^2 as suggested by the National Research Council Committee on Earthquake Engineering (1985, p. 98-101). Correction factors applied to the blow count data were taken from Peck and others, 1974 (fig. 19.6, p. 312) using the effective vertical overburden stress for each sample depth in each drill-hole. The recommended correction of blow count to a standard energy ratio of 60% for the rod energy delivered to the sampler was not performed because we could not be certain that the energy delivery systems were the same for each test used in the study.

Standard Penetration versus Depth Plots

Figures 3 through 8 present blow count versus depth plots for sandy and muddy sediments within each stratigraphic subdivision. Uncorrected and corrected blow count data are plotted separately. Regression equations are given for sandy and muddy sediments on each plot. Table 2 summarizes the regression parameters for each plot.

Examination of the plots of uncorrected blow counts against depth shows a relationship between geologic unit and plot slope that may be useful for grouping units into liquefaction susceptibility categories (fig. 9a). Standard penetration values in muddy artificial fill, muddy Holocene alluvium, and sandy artificial fill increase very little with depth (slopes of .07, .10, and .16 respectively) in the upper 45 feet of the deposits, suggesting that these units have high liquefaction susceptibility. Muddy recessional outwash, pre-Vashon till, muddy advance outwash, and sandy Holocene alluvium (slopes of .48, .49, .57, and .61 respectively) comprise an intermediate category of deposits. Sandy recessional outwash, muddy pre-Vashon nonglacial deposits, sandy advance outwash, sandy pre-Vashon nonglacial deposits, and Vashon till (slopes of 1.2, 1.3, 1.4, 2.0, and 2.1 respectively) show marked increase of penetration resistance with depth and would therefore appear to have a low liquefaction susceptibility.

Corrected standard penetration data yield a poorer grouping of deposits (fig 9b). With the exception of muddy pre-Vashon nonglacial deposits, muddy deposits show no tendency to increase their penetration resistance with depth. In fact, these units show an apparent decrease in resistance with depth. Sandy deposits do show an overall tendency for the older units to have higher resistance at depth than do the younger units, but clear groupings do not emerge.

Discussion

This study of the standard penetration properties of the glacial and nonglacial deposits of the southern Seattle area suggests that SPT data may provide a basis for subdividing geologic units into liquefaction susceptibility categories. This study also shows that plots of blow count versus depth show considerable variability, as may be expected in lithologies as diverse in physical properties as those present in this glaciated region.

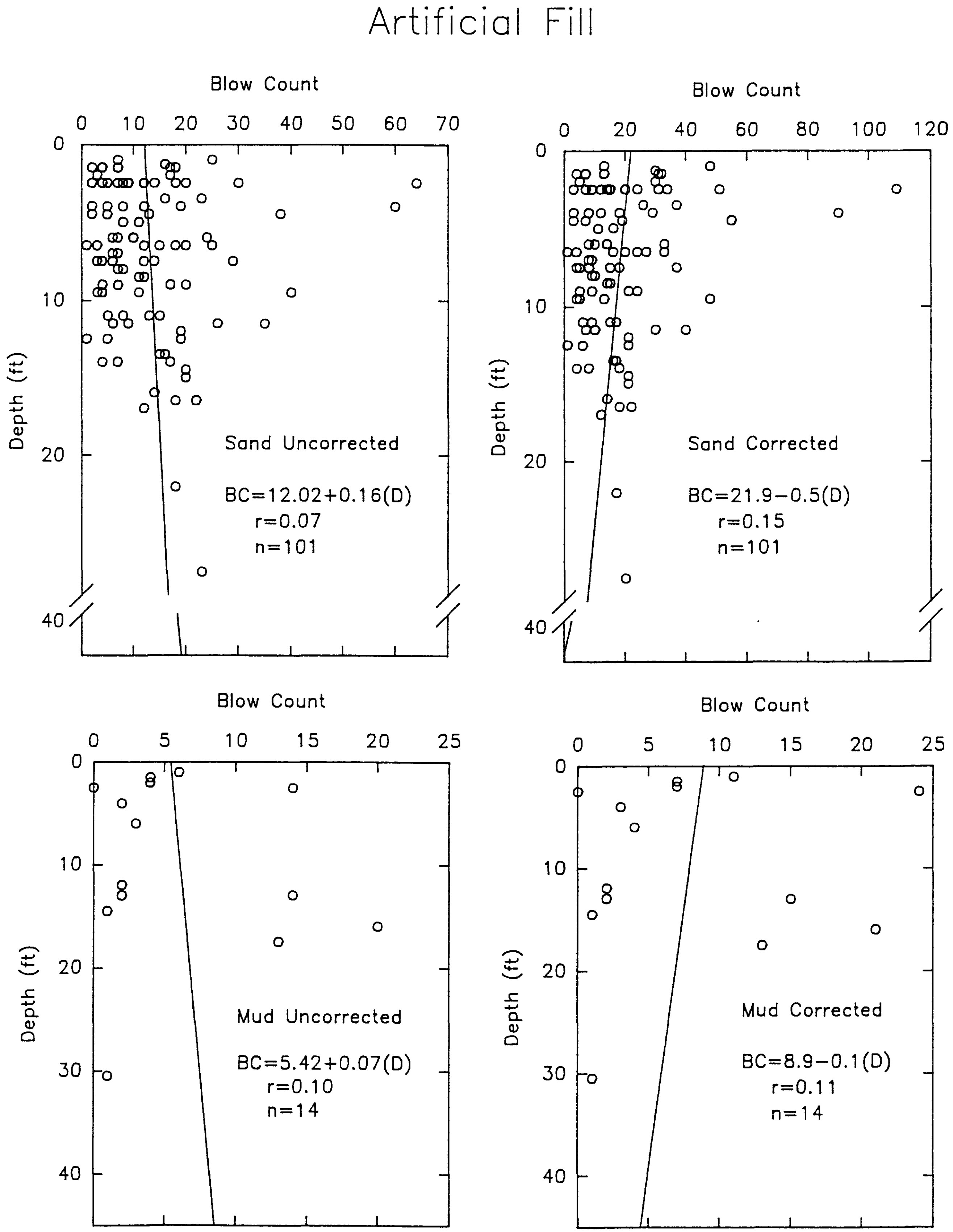
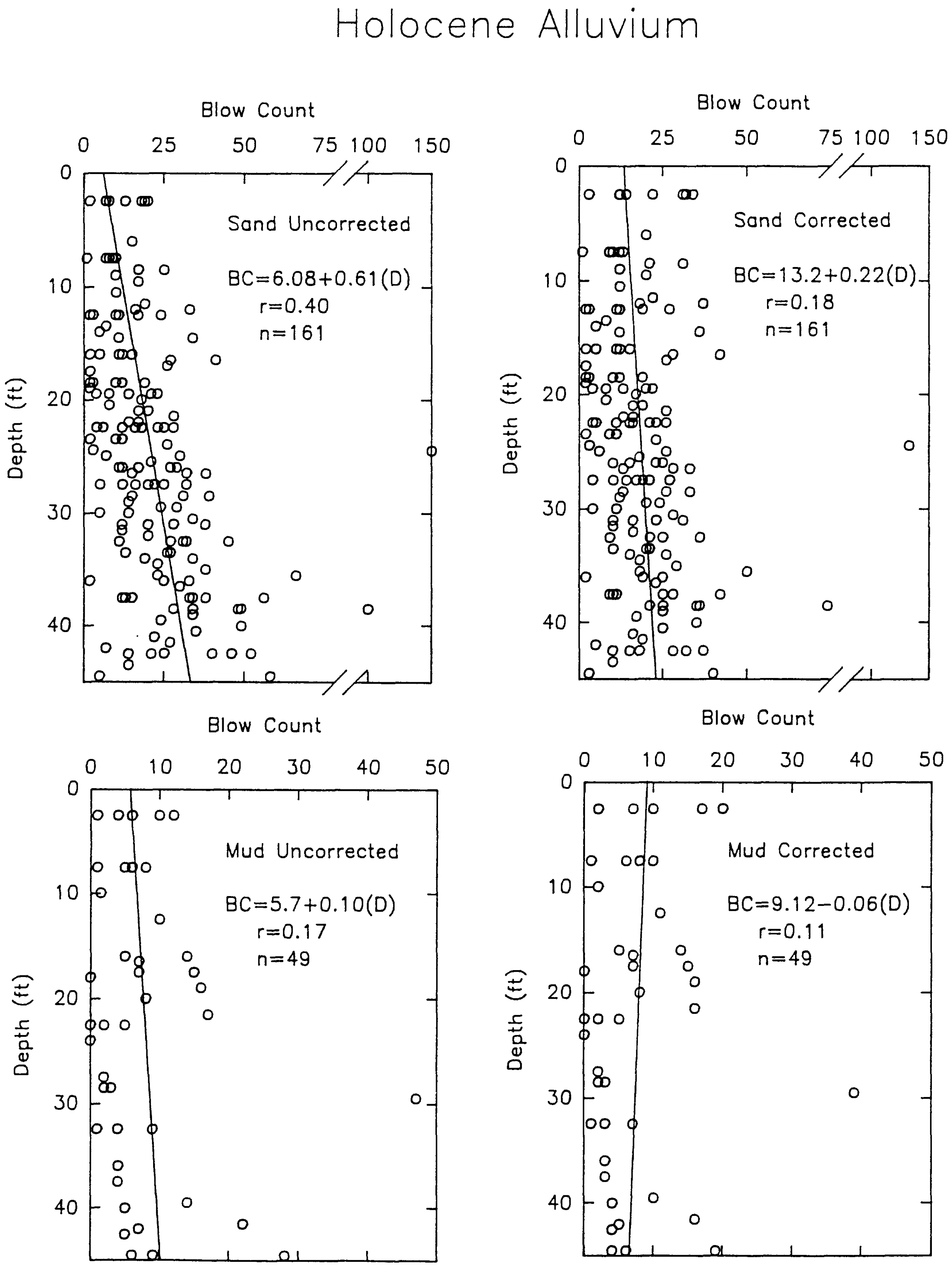


Figure 3: Blow count versus depth plot, artificial fill, mud and sand for uncorrected blow counts and corrected blow counts.



Holocene Alluvium

Figure 4: Blow count versus depth plot, Holocene alluvial mud and sand for uncorrected blow counts and corrected blow counts.

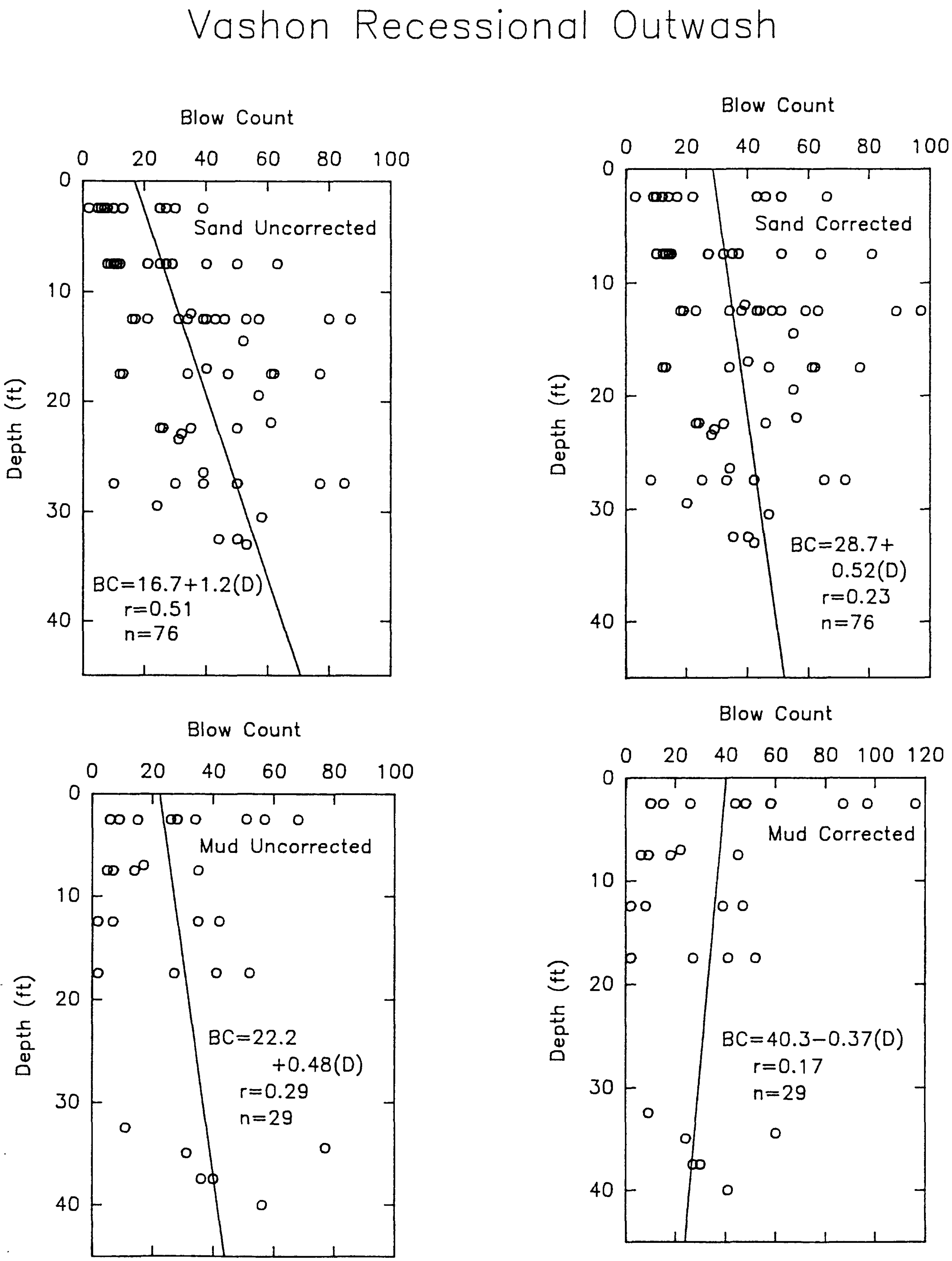


Figure 5: Blow count versus depth plot, Vashon recessional outwash, mud and sand for uncorrected blow counts and corrected blow counts.

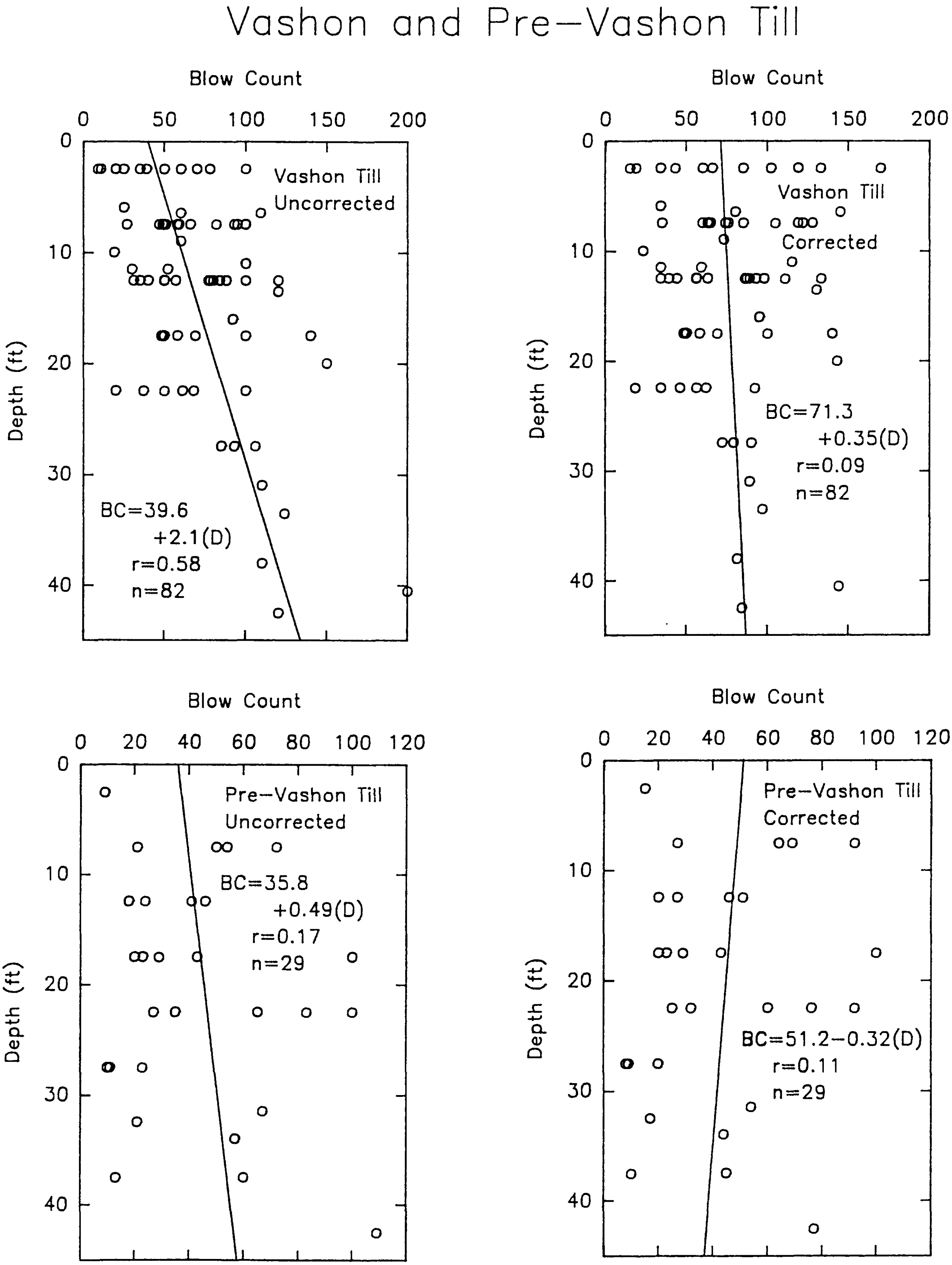


Figure 6: Blow count versus depth plot, Vashon till and pre-Vashon till and gravel for uncorrected blow counts and corrected blow counts.

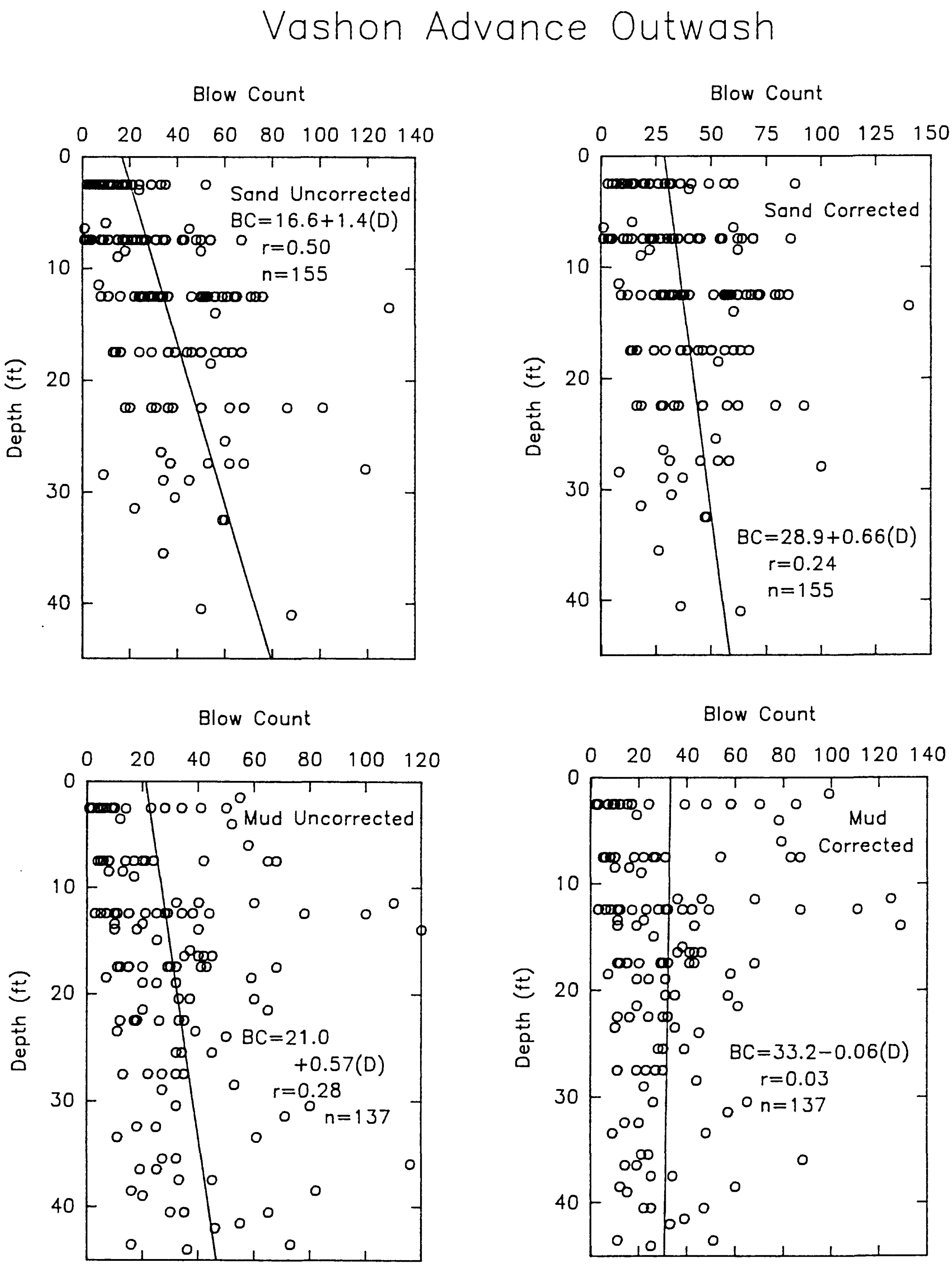


Figure 7 Blow count versus depth plot, Vashon advance outwash, mud and sand for uncorrected blow counts and corrected blow counts.

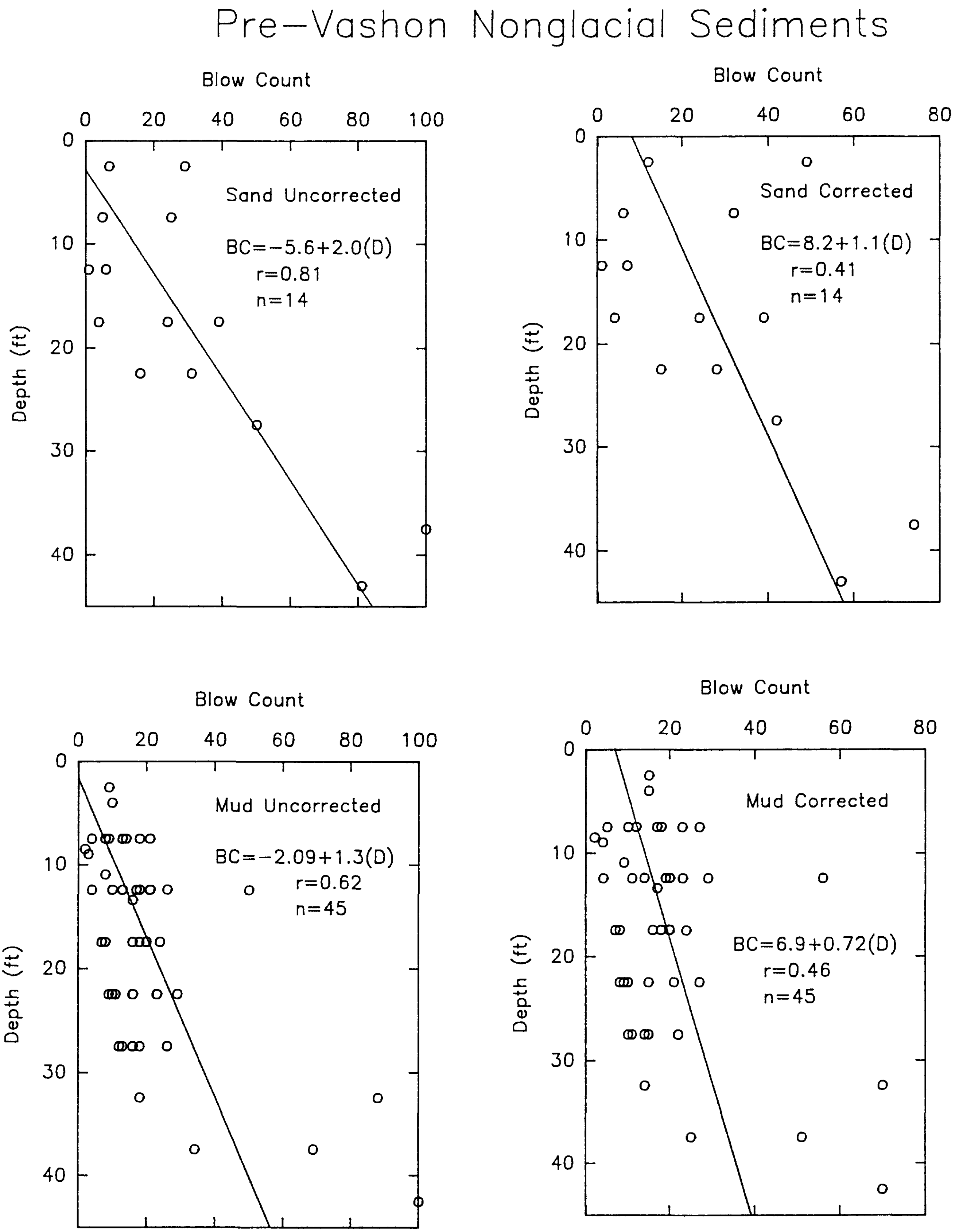


Figure 8: Blow count versus depth plot, Pre-Vashon Quaternary mud and sand for uncorrected blow counts and corrected blow counts.

Summary Regression Plots

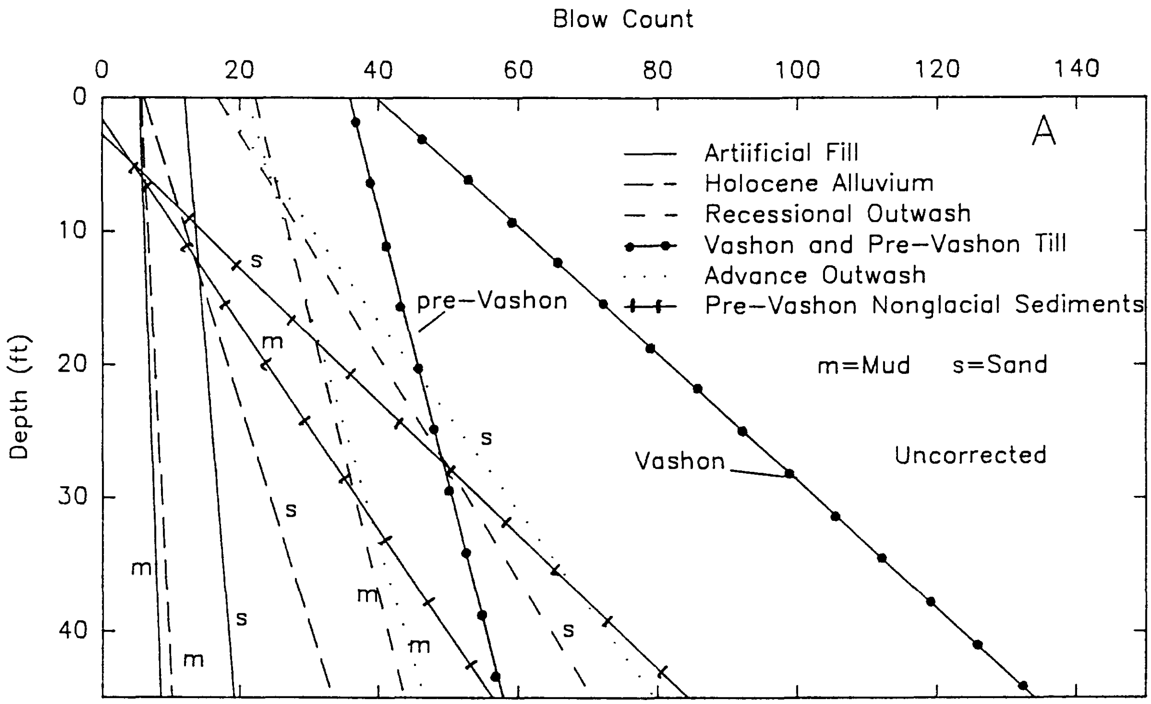


Figure 9A: Blow count data uncorrected for overburden pressure.

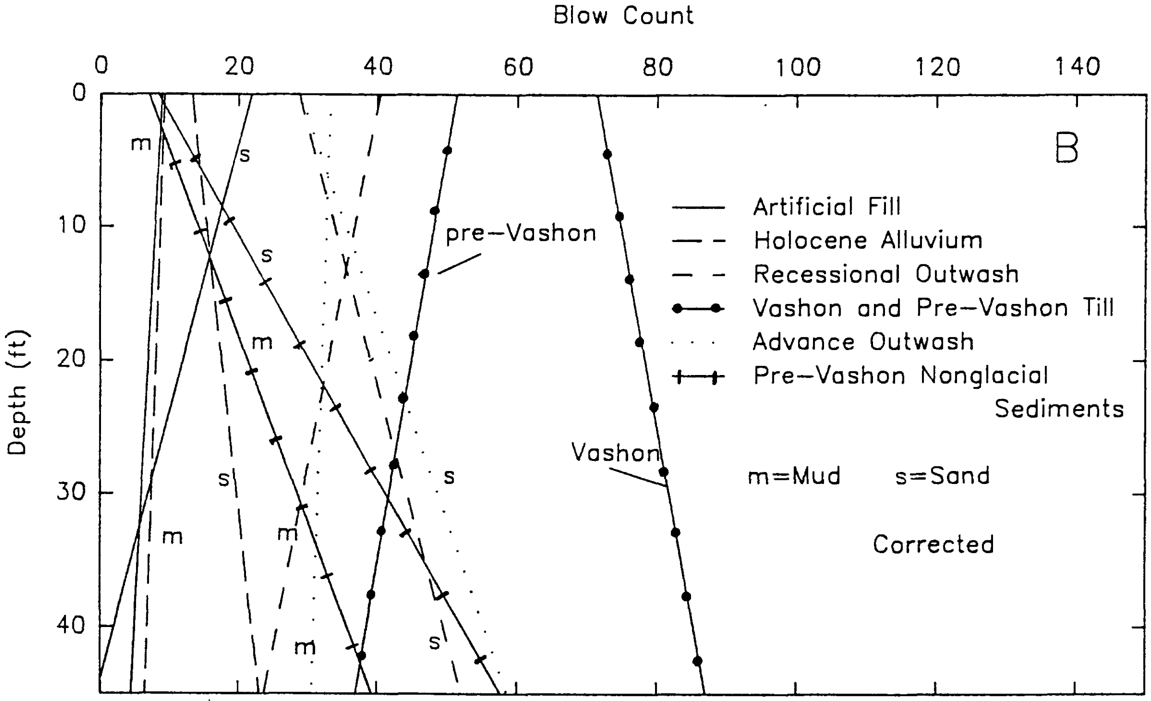


Figure 9B: Blow count data corrected for overburden pressure.

One unresolved issue is the lack of correlation between deposit type and blow count properties when the blow count data are corrected for excess overburden stress. A high standard penetration resistance value at shallow depth will be corrected by a larger amount than will a lower resistance value. That is, applying a correction factor of 1.5 to a blow count of 50 in a near surface test will result in a corrected blow count of 75 blows per foot, while the same correction to a measured blow count of 5 will yield a corrected blow count of 8 blows per foot. Thus, the scatter in values of corrected data versus uncorrected data will appear greater for near surface high resistance units than for near surface low resistance units. This may explain the large scatter in corrected Vashon till data compared to uncorrected data (fig. 6).

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SECTION II: ENGINEERING DESIGN

This section contains information on design and retrofit of structures in seismically active areas, supplementing information found in numerous FEMA publications. A complete list of publications in FEMA's Earthquake Hazard Reduction Series is found in Appendix B.

SEISMIC DESIGN PHILOSOPHY OF THE UNIFORM BUILDING CODE

by John Hooper
Ratti Swenson Perbix Clark
Seattle, Washington 98101

ABSTRACT

The primary function of the seismic design procedures in the Uniform Building Code (UBC) is to provide minimum standards for use in building design regulations to maintain public safety in the extreme earthquakes likely to occur at the site. The intent of these provisions is to safeguard structures against major failures and loss of life due to earthquake ground shaking; no attempt has been made to include provisions that will limit damage due to earth conditions. It is the intent of this presentation to review the seismic information provided by the geotechnical community that helps to establish current seismic design criteria, to discuss additional parameters that will aid in further expanding the near-term state-of-the-art and, finally, to postulate the form of future seismic design criteria.

Abstract for NEHRP meeting held in Seattle, Washington
April 18, 1990

INTRODUCTION TO A SEISMIC RETROFIT OF OLDER BUILDINGS

Todd W. Perbix
Ratti Swenson Perbix & Clark

The object of this talk is a brief introduction to seismic retrofit as an engineering problem as well as its context in the Pacific Northwest. Seismic retrofit is in a time of tremendous change; development of technical methodologies is underway and, of course, a vast amount of research and data collection is taking place as a result of earthquakes and their effects on existing buildings.

Seismic retrofit as a problem has been addressed in the Pacific Northwest, particularly Seattle, for about the last 15 years. In that time it is often proceeded on a singular course, but at present the methods and attitudes employed to make our building stock safer are more and more the result of a national consensus. Historically, seismic retrofit in the Pacific Northwest has focused on building elements; the attempt to integrate, anchor and restrain, individual elements which on the basis of previous experience are hazardous. However, as recently as ten years ago a great deal of effort was put into the global restraint of existing buildings, that is, the installation of reasonably contemporary lateral force restraining systems. Currently, efforts are directed primarily towards understanding the relative risk posed by individual elements or by hazardous building systems and addressing these hazards through techniques which focus on prioritizing the economic question of how much money can or should be spent to make a building which has performed adequately during past earthquakes, more able to resist future ones.

A philosophy has been posed and is generally accepted in the Pacific Northwest. That is, while new building designs and codes provide a substantial degree of life and economic safety and redundancy for all but the most severe seismic events, the retrofit of existing buildings focuses on life safety as its preeminent concern while recognizing that for most existing buildings providing the degree of safety mandated for new building systems is not economically feasible.

The extent to which the technical professions should be involved in societal issues, such as deciding who or what building should be how safe, is a question with which the entire earthquake community continuously grapples. Seismic retrofit represents the most extreme hazard in buildings as well as the largest portion of our building stock. These factors along with the uniqueness of older buildings make them the focus of the larger public policy issues faced by professionals, regulators, and the public. The next phase in retrofit will be the integration of these disciplines into a concept and philosophy which reflects society's values.

HERITAGE BUILDING TOUR

Review of the exterior of the building along both its street and alley elevations, noting wall construction, thickness and fenestration. Also note anchorage rosettes on both the brick and sandstone faces of the building.

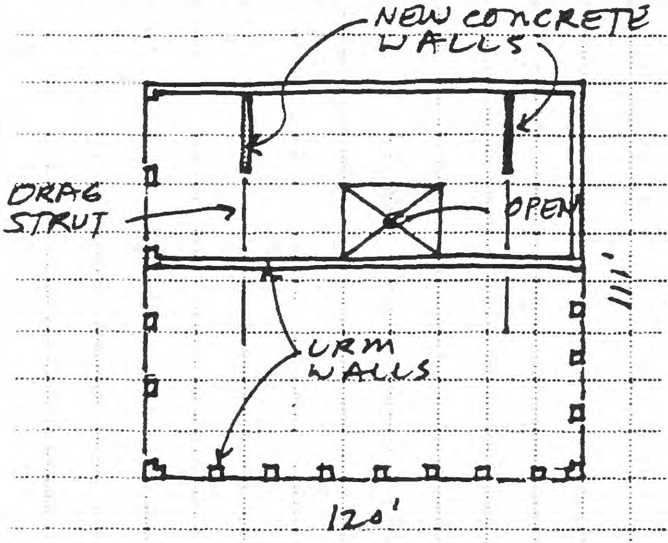
On entering the building, please realize that the Heritage Building is a working office and our visit will take place during working hours, please gather in groups in the gallery space located beyond the lobby, directly behind the grand stairway on the first floor. Note in this area the center brick shear wall which forms a primary load resisting element for north/south lateral forces. Note as well the anchorage of the wood floor structure to this wall and the drag strut connections located over the lobby.

Proceed upstairs via the stair to the second floor lobby. Walking quietly through the large studio space west of the lobby. Note the beam and column tie plates, continuations of drag strut elements and floor to wall anchorage. Also note the concrete shear wall construction in the stairwell directly adjacent to the elevator.

Proceeding again down the staircase, note the large atrium-like openings which make up this stair and which presents some concerns to the structural engineer.

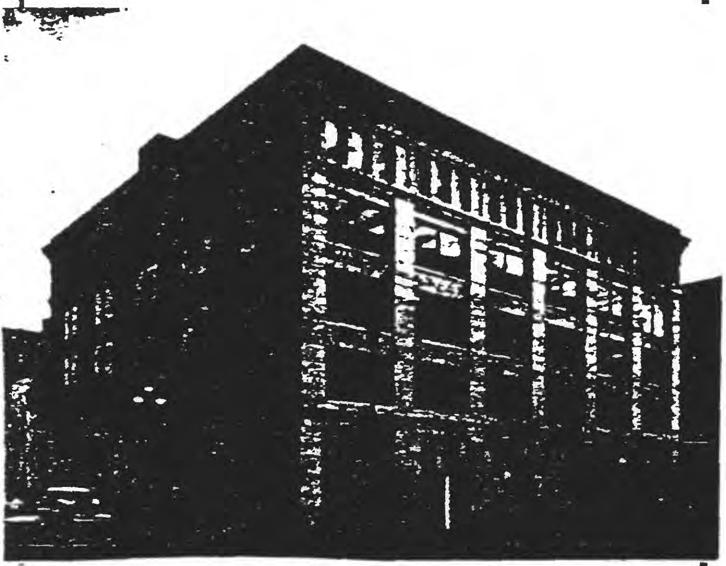
ATC-21/ (NEHRP Map Areas [✓ 5,6,7 High])

Rapid Visual Screening of Seismically Hazardous Buildings



Scale:

Address HERITAGE BLDG
Zip
Other Identifiers
No. Stories 5 Year Built
Inspector Date
Total Floor Area (sq. ft) 65,000 SF
Building Name
Use OFFICE
(Peel-off label)



OCCUPANCY

Residential	No. Persons
Commercial	0–10
[✓ Office]	11–100
Industrial	[✓ 100+]
Pub. Assem.	
School	
Govt. Bldg.	
Emer. Serv.	
Historic Bldg.	

Non Structural Falling Hazard ☐

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data

DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	C1 (MRF)	C2 (SW)	C3/S5 (URM INF)	PC1 (TU)	PC2	RM	URM
Basic Score	4.5	4.5	3.0	5.5	3.5	2.0	3.0	1.5	2.0	1.5	3.0	[✓ 1.0]
High Rise	N/A	-2.0	-1.0	N/A	-1.0	-1.0	-1.0	-0.5	N/A	-0.5	-1.0	[✓ -0.5]
Poor Condition	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Vert. Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-0.5	-0.5	-1.0	-1.0	-0.5	-0.5
Soft Story	-1.0	-2.5	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-2.0	-2.0	-1.0
Torsion	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
Pounding	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
Large Heavy Cladding	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
Short Columns	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
Post Benchmark Year	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	[✓ -0.3]
SL3	-0.6	-0.6	-0.6	-0.8	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.8	-0.6
SL3 & 8 to 20 stories	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8

FINAL SCORE .2

COMMENTS

Detailed Evaluation Required? [✓ YES] NO

SEISMIC RETROFIT OF UNION STATION

Todd W. Perbix
Ratti Swenson Perbix & Clark

Union Station is a complex, concrete frame structure designed in 1910. It is a historic building including large public spaces and detailed in a rich railroad style common to the era. Because of its construction type and lack of a lateral frame system it should be regarded as hazardous during an earthquake. Particularly in view of the large numbers of people which may gather inside. Visual inspection and analysis of Union Station indicated that the large concrete frames which support its gravity loads were inadequate to restrain any substantial lateral load and that while the building has a large and heavy brick in-fill exterior, these bricks were designed and constructed separately from the main building frames. This means that the bricks are poorly connected to the building frame itself as well as not being available to provide substantial damping of loads attracted to the building during an earthquake. Union Station has suffered substantial damage in past earthquakes and was a candidate for interim reoccupation of its great hall. Consequently, recommendations for interim seismic improvement were of two types:

- Anchorage of otherwise unrestrained and unintegrated building elements such as cladding and parapets.
- Installation of a global shear wall system to restrict building movements while carrying shears to the ground.

Due to the interim nature of the improvements, anchorage of exterior cladding elements, parapets, cornices and so on was limited to the public ways. An anchorage of interior elements such as in-fill gypsum walls was limited to those items adjacent to occupied areas.

The second system of improvements, global shear wall systems, were installed in the frame bay between columns and beams using the shotcrete method. These systems were located for the most part adjacent to the great hall, both to increase safety in the area and to allow for the maximum flexibility when the building was fully reoccupied. Shear walls in this area do not extend between the ground floor and the main floor for two reasons. First, overall life safety was best provided by supporting the building above the main floor and there is a substantial system of concrete walls surrounding the building below the main floor.

The retrofit of Union Station is an example of the need to address occupancy risk in the development of seismic retrofit plans. It is also an excellent example of a case where interim improvements substantially increase the safety of the building occupants while addressing the economic needs of the owner and securing the building from further deterioration due to its vacancy.

UNION STATION TOUR

Walk around the exterior, particularly along the Fourth Avenue and Jackson Street elevations. Note the general configuration of Union Station, in particular the large canopy and stone railing above the entrance as well as the fact that at street level you are two floors above grade. Note as well the anchorage locations along Jackson & Fourth and the fact that they do not occur along any other elevation of the building.

On entering through the main entrance, note the great hall in particular since this space was the only intended useful space addressed by the retrofit. Of interest in the hall are the brick archway supports against the far window. All other structural support surrounding this area is visible only from behind on upper floors.

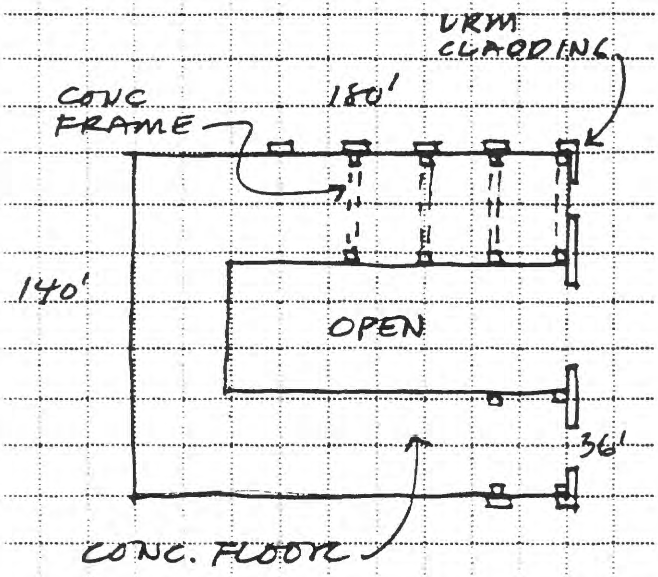
Moving through the lobby to the left proceed up the stairwell to the second floor. Note the steel stud strongbacking as well as its attachment to both the gypsum block and to the upper and lower structures. Note as well the shotcrete shear walls and their placement as between existing beam and column structures. Along outside walls near Jackson Street and Fourth Avenue note the floor to wall anchorage (note wall cavity) and the rail bracing along Jackson Street.

Proceeding to the third floor note the extensive bracing along the street elevations again. In this case the purpose of this work is to restrain the stone sill work which is an ornate element on the building exterior. Note the concrete masonry walls along the corridor which replaced the gypsum walls which collapsed in an earlier earthquake.

If time is available, take the small stair to the fourth floor and note the steel braces directly adjacent to the truss area which extend the lateral force system provided by the concrete shear walls below. Note as well the vault cavity which is of interest because the structural support of the great hall as well as the non-structural support of its plaster finish are clearly evident.

ATC-21/ (NEHRP Map Areas [✓ 5,6,7 High])

Rapid Visual Screening of Seismically Hazardous Buildings



Scale:

Address UNION STATION

Zip

Other Identifiers

No. Stories 5 Year Built 1910

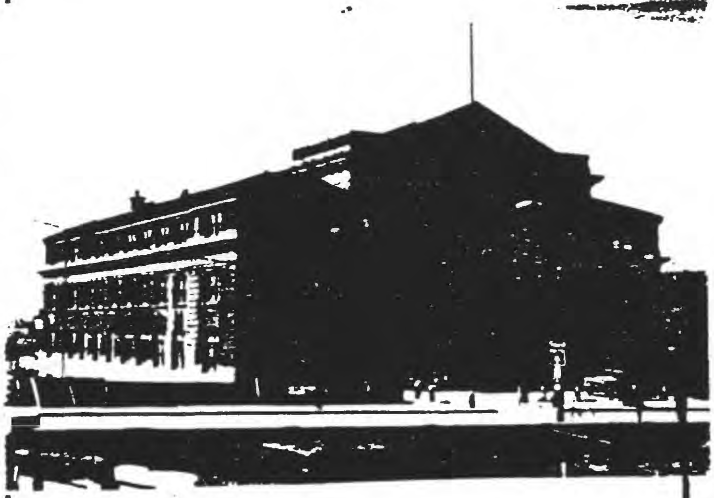
Inspector Date

Total Floor Area (sq. ft)

Building Name

Use VACANT

(Peel-off label)



OCCUPANCY	No. Persons
Residential	0–10
Commercial	11–100
Office	[✓ 100+]
Industrial	
[✓ Pub. Assem.]	
School	
Govt. Bldg.	
Emer. Serv.	
[✓ Historic Bldg.]	

Non Structural Falling Hazard ☑

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data

DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	C1 (MRF)	C2 (SW)	C3/S5 (URM INF)	PC1 (TU)	PC2	RM	URM
Basic Score	**4.5**	**4.5**	**3.0**	**5.5**	**3.5**	**2.0**	**3.0**	**[✓ 1.5]**	**2.0**	**1.5**	**3.0**	**1.0**
High Rise	N/A	-2.0	-1.0	N/A	-1.0	-1.0	-1.0	-0.5	N/A	-0.5	-1.0	-0.5
Poor Condition	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Vert. Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-0.5	-0.5	-1.0	-1.0	-0.5	-0.5
Soft Story	-1.0	-2.5	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-2.0	-2.0	-1.0
Torsion	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	[✓ -0.5]	-1.0	-1.0	-1.0	-1.0
Pounding	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
Large Heavy Cladding	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
Short Columns	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
Post Benchmark Year	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 & 8 to 20 stories	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	[✓ -0.8]	N/A	-0.8	-0.8	-0.8

FINAL SCORE .2

COMMENTS

Detailed Evaluation Required? [✓ YES] NO

PROBLEMS OF UNDERGROUND STRUCTURE SEISMODYNAMICS

T. Rashidov *

Abstract. *The paper gives a brief description of developed in Uzbekistan seismodynamic theory of underground structures, based on wide analysis of data of strong motion damages during recent 30-40 years; results of laboratory and field tests and theoretical study. Comparison results of theoretical and experimental investigation (on the basis of a simple example of plane elastic wave influence on cylinder shell) allow to establish the reliability and the field of practical usage of seismodynamic theory in solving the problems of underground structure seismic resistance. These results are already widely used in construction of underground structures: pipelines, tunnels, metro etc.*

Intensive construction of underground structures in modern cities, functioning of industrial enterprises and load transportation leads to the necessity of rational designing and prediction of the behaviour of structures located in seismoactive areas. Means providing the security and stable work of erected buildings should be economically expedient, that is why the problem of evaluation of the character and level of dynamic influence on structures at possible seismic and shock actions is emphasized. The solution of this problem depends on several factors. It needs the improvement of seismomeasuring instruments and experimental equipment, thorough makroseismozoning, the development of the method of definition of seismic action characteristics and methods of stress-strain state of structure design, and finally, working out of new constructive solutions of seismoresistant structure elements.

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The experience of underground structure exploitation in seismoactive regions and the analysis of earthquake effect on these structures lead to conclusion of special importance of the factor of ground conditions influence. In many cases at relative small mass per length unit (pipes, collectors, tunnels),underground structures have large contact area with surrounding ground. Hence, the effect of non-inertial seismic loads frφm seismostress state of ground prevails inertial ones, wich appeare from presense and distribution of mass of the structure itself. The way of account of seismic loads effect defines the approach to solution of underground structure seismic resistant problems. We may distinquish three approaches used now: quasistatic,seismodynamic and wave one, which takes into account the phenomena of difruction, refraction and reflection of seismic waves.

Wave approach, when equations of motion for underground structures are solved together with equations of ground motion, allows on the basis of mathematical model to follow the dynamics of force change in soil-structure interaction, accounting physical and mechanical features of surrounding ground, the character of dissipating seismic waves and the depth of structure bedding.

Seismodynamic approach to evaluate dynamic effect is based on results of the study of earthquake effect on underground structure and results of experimental tests. It is supposed that any underground structure may branch and in branch zones there are complex (rigid or flexible) massive joints (See Fig. 1). Such structure is shemed as the combination of interacting beam-frame constructions and massive solids having six degrees of freedom. In design motion equations for each element of complex structure are made up; boundary conditions and yeilding and rigid joints are described.

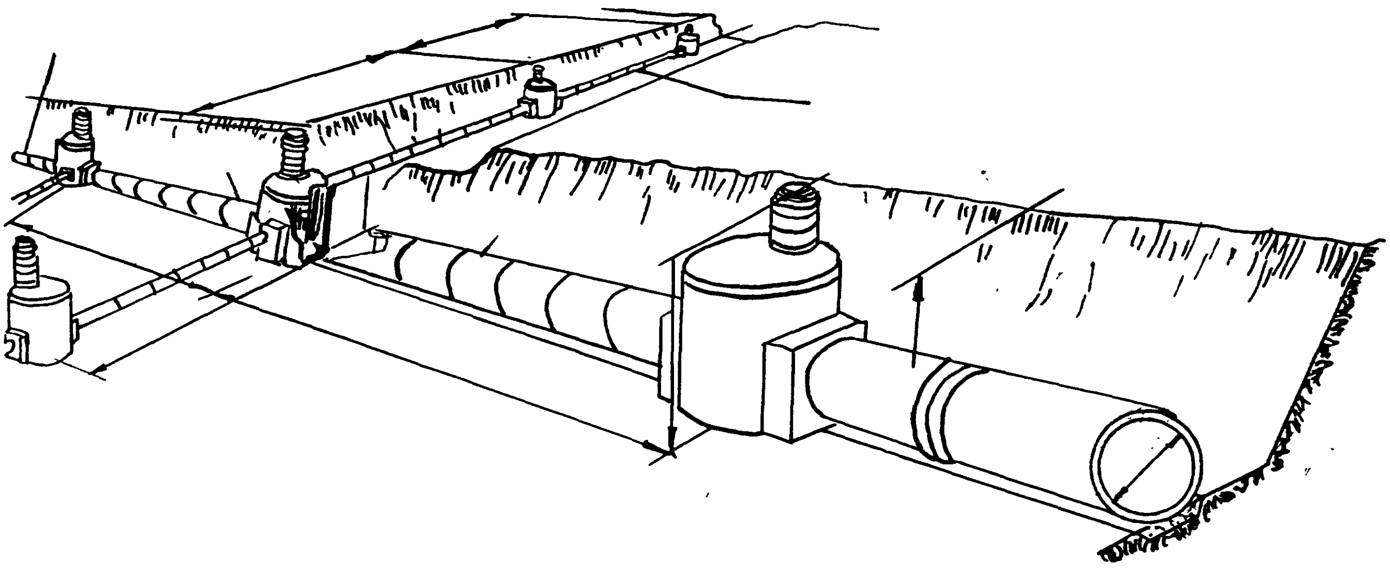

Fig. 1 Variant of complex system of net underground strucrures

Ground conditions, some constructive peculiarities and geometrical dimensions, the depth of bedding and other factors are taken into consideration through coefficient system (in general case operators) called interaction parameters. Experimental tests allow to state change boundaries of these coefficients for different grounds, depth of bedding, geometric dimensions in longitudinal and cross motion, torsion and bending.

Experimental evaluation of interaction parameters was held both in static and dynamic conditions of loading and also under the effect of shock load in field tests. To define the parameters of soil-underground structure interaction, methods of centrifugal modelling were used - practically the only laboratory method providing the preservation of physical nature of phenomena studied. Experimental data, obtained by method of centrifugal modelling give the possibility to measure the distribution of ground structure depending on configuration of its cross-section. It was stated that ground pressure is distributed nonuniformly, maximum pressure value falls on the bottom of the structure, minimum one - on horizontal diameter.

"Loading-unloading" diagrams obtained for rectilinear section of underground pipeline in longitudinal and cross motion, torsion and bending (See Fig.2a-d) show that tangential stress-loading dependence is linear only in initial stage of loading.

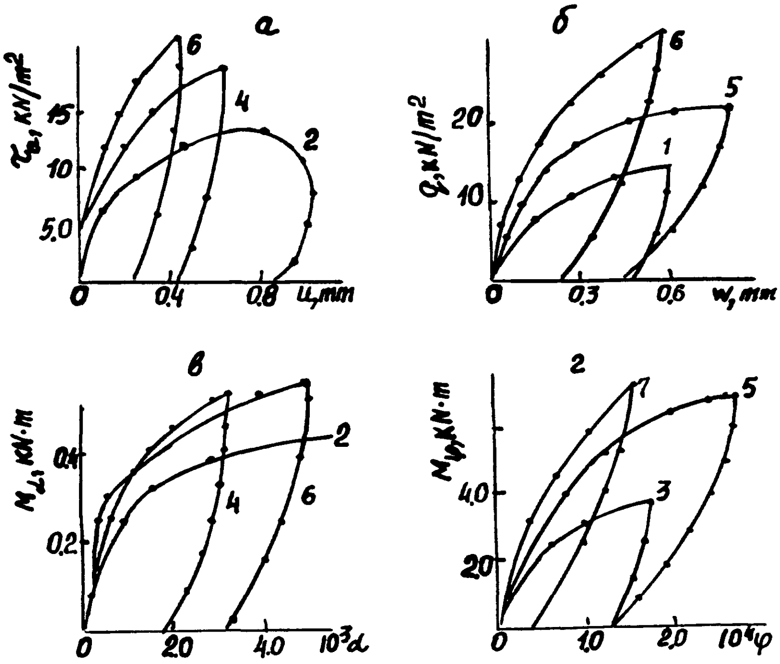

Fig. 2 Diagrams of pipe interaction

Fig. 3 shows diagrams of repeating loading-unloading of interaction. In these graphics in each loop maximum load M_α, τ_α acting on pipelines increases. As it is seen from the figure, even when load values are small residual displacement form considerable part of total displacement. For example in the first loop at M_α = 0.41 kN.m (See Fig.3a) α_6 = 0.6 10^{-3} m that eqals to 50% of total displacement, and at τ_α = 0.04Mpa (Fig.3b) 70%.

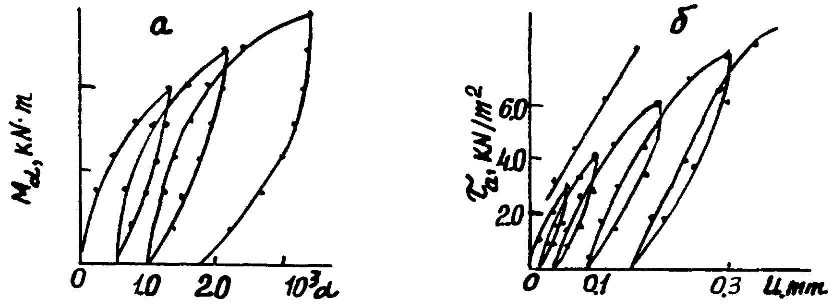

Fig.3 Diagrams of pipe interaction at repeating loading-unloading: a - cast iron (D =0.169m, l =2.0m) pipes with sand; b - asbestos-concrete (D=0.222m, l =3.5m, H =0.7m) pipe with loam

If elasticity limit to consider as a load, when residual part of displacement is no more then 2-3%, in these tests elasticity limit of interaction lies below proportionality limit. So only when load values are small, the interaction submit to the law of deformation reversibility.

The range of linear connection between the loading and displacement is not large. The main range of connection "loading-displacement" is presented by non-linear dependence in the form:

$$q_i = k_i \theta_i \left[1 - \omega(\theta_i)\right] \tag{1}$$

where function $\omega(\theta_i)$ characterizes non-linear (plastic) features of interaction and is obtained from the test. From dependence of the function $\omega(\theta_i)$ on displacement θ_i (See

Fig. 4b) it is shown that its character is similar to one of ordinary elastisity function, which is used in solving the problems of stressed solids plasticity.

In equations of underground structure-soil interaction vibrations the plasticity function obtained experimentally enters in unevident form and it should be approximated by some analytical expression.

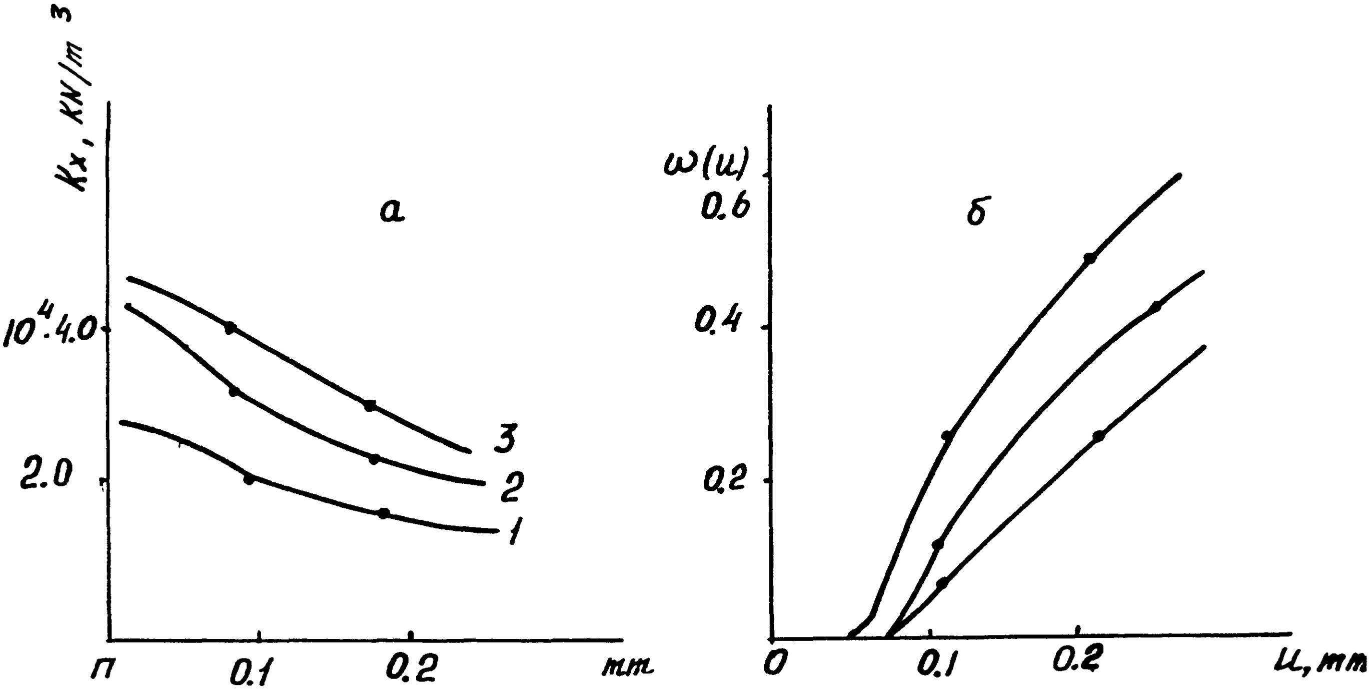

Fig. 4 Cast iron pipe - sand interaction dependences $k_x \sim u$ (a) and $\omega(u) \sim u$ (b) at H =0.2(1); 0.4(2); 0.6(3); 0.8(4)

The difficulty lies in the difference of interaction plasticity functions for different depth of bedding, ground conditions etc., do not submit to a certain laws and in each concrete case should be approximated by different functions. In particular, plasticity function may be successfully approximated by bilinear law of dependence. In solving some problems there rises the necessity of account of rheological features of interaction. To define parameters which characterize the course of rheological processes for underground pipelines several tests on creep and relaxation were held. It was proposed to use results of hereditary theory, writing load-displacement dependence as follows:

$$q_i = - \int_0^t R_i\,(t-\tau)\,d\theta_i\,(\tau)\;, \qquad (2)$$

or

$$q_i = - R_i\,(0)\left[\theta_i\,(t) + \int_0^t \Gamma_i\,(t-\tau)\,\theta_i\,(\tau)\,d\tau\right. , \qquad (3)$$

where $\Gamma_i\;(t) = R_i\,(t) / R_i\;(0)\;,$

$\Gamma_i\,(t)$ - core of creep, $R_i\;(t)$ - core of relaxation.

If to compare the expression (3)at =0 to the dependence for elastic interaction, it becomes evident that interaction coefficient is an operator of the form:

$$k_i = R_i\,(0)\left[1 + \int_0^t \Gamma_i\,(t-\tau)\,d\,\right] \qquad (4)$$

Fig.5 shows curves of creep and relaxation of pipe and complex joint interaction in different grounds depending on time for different depths of bedding, diameter and pipe material. With increasing pipe diameter at equal depth of bedding and load the effect of rheological features on the character of interaction becomes more considerable.

Fig.6 shows the process of graphic description of complex joint of spatial network of underground pipelines.

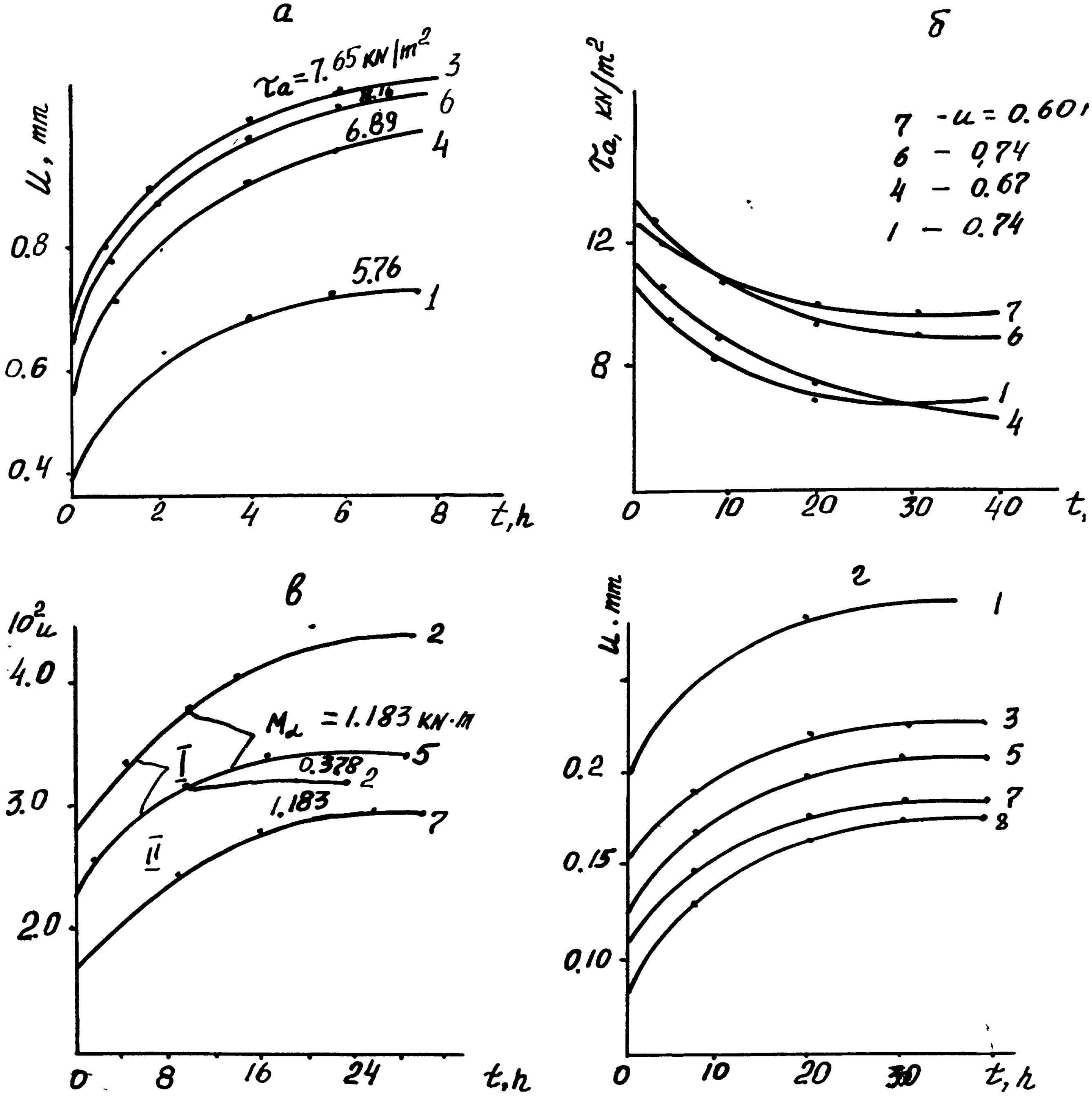

Fig. 5 Curves of creep (a,b,c) and relaxation (d) of steel pipe (a,b), asbestos-concrete pipe (c,1), cast iron pipe (c,2) and complex joint (d) interaction with loam

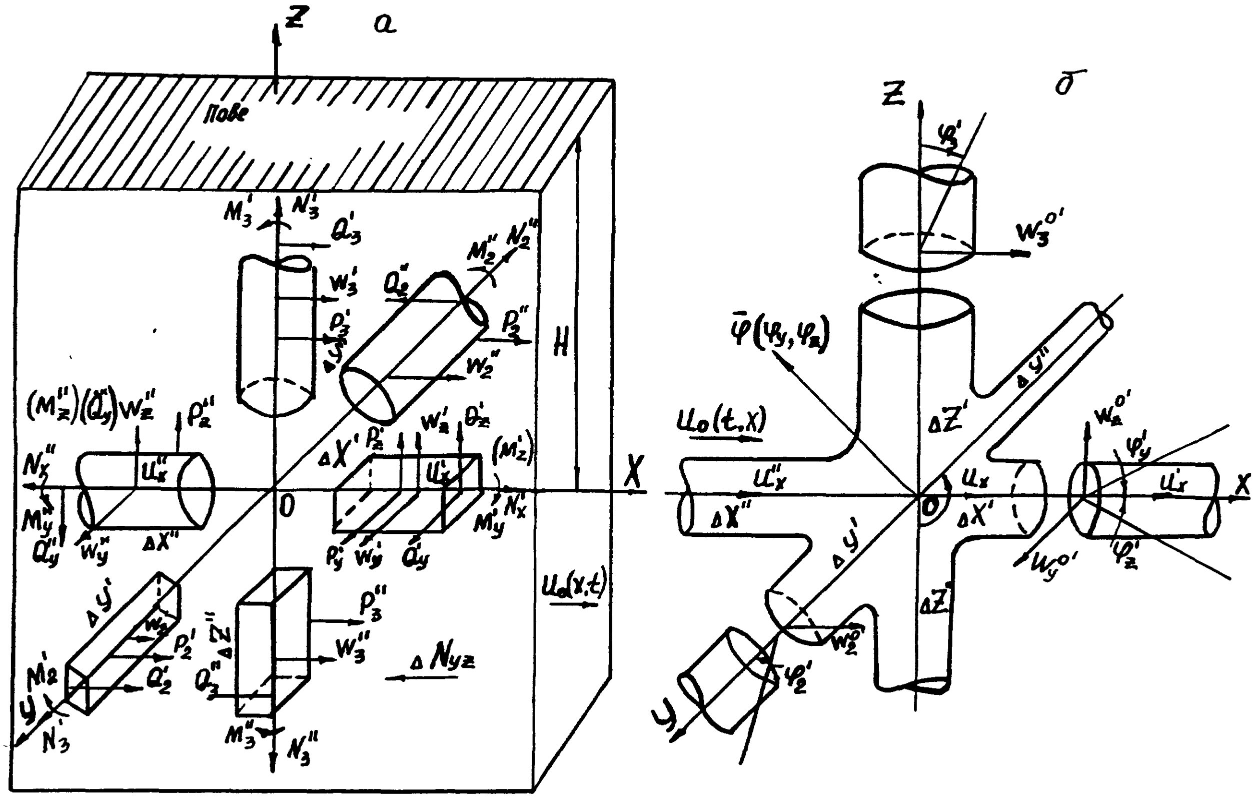

Fig. 6

In selected law of seismic motion of ground (See Fig. 6) pipes located along and axes x , y perform cross motion only, and pipes located along axis z both longitudinal and cross motion (x),(y),(z) have the form:

$$\mathcal{D}_y' \frac{\partial^4 w_2'}{\partial y^4} + |m_y' - m_y'^{\,gr}| \frac{\partial^2 w_2'}{\partial t^2} + p_2' = P_2^1$$

$$\mathcal{D}_z' \frac{\partial^4 w_3'}{\partial z^4} + |m_z' - m_z'^{\,gr}| \frac{\partial^2 w_2'}{\partial t^2} + p_3' = P_3' \qquad (5)$$

$$\mathcal{D}'_{xy}\,\frac{\partial^4 W'_z}{\partial z^4} + /m'_x - m'^{\,gr}_x/\,\frac{\partial^2 W'_z}{\partial t^2} + p'_z = P'_z$$

$$\mathcal{D}'_{xz}\,\frac{\partial^4 W'_y}{\partial y^4} + /m'_x - m'^{\,gr}_x/\,\frac{\partial^2 W'_y}{\partial t^2} + p'_y = P'_y \qquad (5)$$

$$B'_x\,\frac{\partial^2 u'}{\partial x^2} - m'_x\,\frac{\partial^2 u'}{\partial t^2} + p'_x = P'_x \quad .$$

Analogous equation for pipes $(-x)$, $(-y)$, $(-z)$.
Direction of displacement

(W'_1, W''_1), (W'_3, W''_3), (W'_y, W''_y), (W'_z, W''_z), (u', u'') are given in Fig. 6.

$\mathcal{D}$ - bending rigidity of pipes in corresponding plane;

B - tension rigidity of pipes ;

m_x, m_y, m_z - masses of pipe length unit;

m^{gr}_x, m^{gr}_y, m^{gr}_z - substituting masses of ground;

P_x, P_y, P_z, P_2, P_3 - outer loads;

$\rho_x, \rho_y, \rho_z, \rho_2, \rho_3$ - ground response, defined by selected interaction model.

To close the system boundary conditions, conjugated conditions in joints and kinematic conditions characterizing the form of pipe joint are written.

For practical usage the method was worked out; it simplifies dynamic problem of complex system of underground structure vibration and reduces it to a problem of longitudinal vibration of compound long structure with reduced conjugation conditions in complex joint and simple junctions.

The accuracy estimation shows that the difference in numerical values of displacement in both methods does not exceed 10-15% (for soft ground more than for rigid ones); that means that given simplification may be considered possible in solving the problems.

To obtain a reliable estimation of underground structure behaviour it is advisable to take into account wave character of seismic effect.

Detalied analysis of this aspect is based on solution of the class of problem of plane longitudinal wave interaction on linear underground pipelines; distribution front is perpendicular to pipeline axis at different conditions of pipe-soil contact.

Pipelines were simulated by elastic rods; the force acting on side surfaces depends on displacement value of points of the rod relative to moving particles of ground.

It was stated that the behaviour of underground pipelines under the action of seismic wave is considerably effected by the value of M which is the ratio of velocities of wave dissipation in ground media and pipe material α ($M = C_p/\alpha$). At $M \angle 1$ in pipelines prevail high-frequency vibration with spectral composition considerably depending on condition of soil-pipelines contact. It is explained by the fact that pipeline system is under the influence of nonhomogenious wave field, and the wavelendth is less than usual length of a pipeline. With increasing value of M which corresponds to the assumption of instantaneous covering of whole system of pipelines by seismic action, the vibration cheracter approximates to the case of seismic inertia force action, that is in this case, seismic field nonuniformity along pipeline length practically disappears.

The case of a plane wave action on infinite pipeline with its surface interacting with soil according to elasto-plastic law is considered. It is shown that with moving away from the wave front the sections with plastic and elastic features of interaction alternate, and after several full elasto-plastic cycles the interaction may become purely elastic even with invariable intensity of seismic action behind the wave front. With increasing value of the zone of plactic interaction is spreading, it may appear on sufficiently far distance from seismic wave front.

If we consider the case of pipe slipping which is characterized by Coulomb Dry Friction Law we can come to another conclusions. The influence of the shock wave is considered for, the semiinfinite pipe after the front of wave, the velocity of ground fractions is constant. With $M < 1$ the complex wave picture along the pipeline is arranged, it is show that the front of confusion in the pipeline spread with unknown velocity which is defined within the decision. Besides it is revealed that in time along the pipeline arise zones which accomplish combine movement with ground where can arize tension efforts. When the wave influence on the infinite pipeline in the ground with $M > 1$ when distance from wave front over the length of the pipeline the section fixed in the ground and moving with it interchange with the sections, having the relative displacement. With increasing number of

M , the width of the combine movement zone with the ground is decreasing.

In the case of considerable geometrical sizes of the underground structures the modelling of their frame-beam diagrams becomes problematic, besides, it rises the question about admitting in utilization of combine parametrs which have been received within the experimental tests of another size constructions. That's why the theory of shells embedded in the ground, have been utilized for calculation the large-sized underground structures.

For the purpose of definition the main combine parameters of the shells with the ground type structures and estimation the results of the theoretical investigations of the stress-strain state of the structures, large-scale field experimental investigations of the shell-type pipes, embedded into the ground, have been carried out within the dynamical loadings created by the underground explosion.

Large-scale models of underground structures of cylindrical and spherical shell type were used as experimental samples. The underground explosions were conducted at different epicentral distances and with various explosives.

The results of observation were used to define the parameters of seismic waves and their pressure on the underground object, to obtain analytic and emperic dependences for displacement, oscillation rate and stresses in the soil and underground cylindrical structure, to study the stress-strain state of experimental samples of underground structures and to conduct comparison of theoretical and experimental studies.

The records were used to calculate the load to the underground cylindrical structure, relative movements and accelerations of the soil and the studied object. The integral dynamic values of the interaction factor for the cylindrical structures with soil have been determined in the conducted experiments.

Table 1 presents the test results of the underground cylindrical structure made of steel with the didmeter of ϕ =0.72 m and the wall thickness of h =0.008m under the action of seismic loads. The numerical results of the theoretical studies are obtained according to the seismodynamic calculation method with the usage of experimental values of interaction factors K_y =5.09kG/cm , K_z =4,06 kG/cm and seismic loads.

Table 1. Comparison of reults of experimental and theoretical studies

Charge weight Q, kG	Reduced distance R	V_{exp} mm	V_{th} mm	W_{exp} mm	W_{th} mm	σ_k^{exp} kG/cm^2	σ_k^{th} kG/cm^2
5140	14.5	1.0	1.06	2.01	1.23	87.0	67.7
2520	14.7	0.86	0.66	1.06	0.87	81.0	59.3
6540	16.0	0.95	0.89	1.65	1.03	79.3	52.7
2900	24.5	0.86	0.54	0.95	0.63	31.4	23.2
890	36.4	0.31	0.18	0.30	0.21	26.5	19.3

For known values of interaction parameters the problem of theoretical investigation of stress-strain state of underground shell in seismodynamic theory, to define force vector of surrounding soil-shell interaction the following expression is used

$$\vec{P} = [K]\,\vec{\tilde{U}} \qquad (6)$$

where $[K]$ coefficient matrix (in general mode of operators) of surrounding media resistance to displacement components $\vec{\tilde{U}}$

$$\vec{\tilde{U}} = \vec{U}_o - \vec{U} ,$$

where

$$\vec{\tilde{U}}(\tilde{u},\tilde{v},\tilde{w}) \quad , \quad \vec{U}(u,v,w) \ , \ \vec{U}_o(u_o,v_o,w_o)$$

vector of relative and absolute displacement of shell and ground, respectively.

Total equation system of seismodynamics of underground structure of shell-type will be written as

$$\overset{1}{L}_{ij}\tilde{u} + \overset{2}{L}_{ij}\tilde{v} + \overset{3}{L}_{ij}\tilde{w} = f_i + L_i(u_o,v_o,w_o) + p_i, (i=\overline{1,3}) \quad (7)$$

where $\overset{k}{L}_{ij}$, L_i - differential operators, describing stress state of shell with account of its interaction with soil as the result of seismic effect; f_i - function, describing outer loads; p_i - interaction forces.

According to the applied interaction models one can get a system of differential or integrodifferential linear or nonlinear equations systems which describe oscillations of the underground shell during seismic effects.

Using the above approach we have solved certain problems of seismodynamics of underground structure of cylindric or spheric shell type with a closed or open profile with a constant or varying thickness with different boundary conditions at different types of seismic effects.

Comparison results of experimental and theoretical values of displacement and stresses (See Table 1) show some overstating of test data. It is explained by the fact that in theoretical design an elastic model of soil-structure interaction is accepted. When using an elasto-plastic model of interaction results of theoretical study are nearer to experimental ones. An average value of difference of theoretical and experimental results for the examined case equals 35.8%.

The presence of several approaches to underground structure stress state study arises the problem of necessity of correction the areas of practical applicability of different methods and reliability of results obtained. For this purpose the problem of plane elastic wave action on cylinder shell was solved both by the method of wave dynamics and seismodynamic method.

The general statement of the problem for both methods is the same. It is assumed that infinitely long cylindrical shell submerged into the elastic soil is loaded by a plane longitudinal wave acting across the longitudinal shell axis, i.e. in the case the problem of plane deformation is considered. With the approach according to the wave dynamics the solution consists of two parts and each of them is a solution of the wave equation for the shell and the surrounding soil. These solutions consist of stresses and movements connected with the incident wave, excited and reflected waves in the cylinder and the waves spreading in the environment. The solution components are combined under condition that the conditions of continuity for movements and stresses were fulfilled on the external surface of the shell and the internal surface was free of stresses.

According to the selection of the model of the structure interaction with the soil and the load, kinematic and dynamic ratios of the elasticity theory, the equations of the state of the shell and the surrounding soil can have different complex forms whose mathematical integration is a very complicated problem. The system of equation based on a simplified assumption on linear-elastic interaction of the shell with the soil has been obtained according to the above mentioned statement for the given problem realization on seismodynamic technique.

The solution of the equation system obtained by the methods of seismodynamics and wave dynamics was made by using the methods of separation of variable and integral Laplas transformations under stationary (harmonic wave) and non-stationary(step) loads. Here we present the comparison results of the approaches to the solution of the problem of seismic stability of underground structures of the cylindrical shell type.

Fig. 7 shows the comparison results of maximum values $\sigma_{\theta\theta}$ accordingg to the calculation methods depending on the ratio of the length to the structure diameter $\lambda/\mathcal{D}$. It is seen that with the wave length increase $(\lambda/\mathcal{D} > 3)$ (the low frequency range) the stress values for both calculation methods get closer. In this case the nature of the field distribution for movements and stresses along the contour of the underground cylindrical structure is similar and their values according to the seismodynamic method are higher by 20%. The stress attains its maximum value in the range of 1.6 - 2.9 $\lambda/\mathcal{D}$ and then asymptotically approaches the solution of the static problem.

With the wave length decrease which corresponds to the increase in the soil oscillation frequency, the value $\sigma_{\theta\theta}$ is significantly decreased. In this case nature of distribution for the field of movments and stresses along the contour of the underground cylinrical structure according to the results of the method of wave dynamics is different from seismodynamics. This is connected with a complex difraction process occuring during interaction of the high frequency waves with underground objecst. For the underground round shell the resonance frequency (according to the seismodynamic calculation methods) at was equal to $\lambda/\mathcal{D}$=0.35, where an abrupt

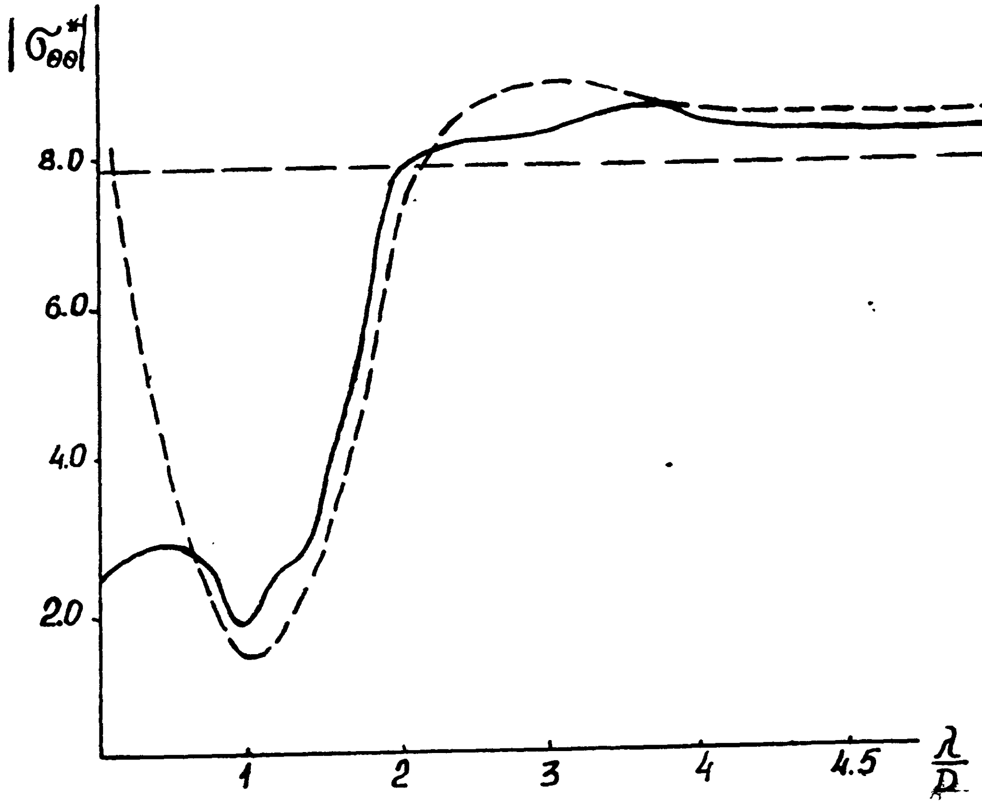

Fig. 7 Comparison of the stress variation
(1 - seismodynamic theory, 2 - wave theory,
3 - static solution)

increase in the stress values is observed.

Having studied the problem of the wave interaction with the round cylindrical shell with a plane statement at equal initial data by two methods we have determined the field of their practical application.

During calculations of cylindrical shell type for the action of seismic waves (the wave front is perpendicular in reference to the cylinrical shell generatrix) the wave difraction at $0 \angle \mathcal{D}/\lambda \angle 1$ is not required. The difraction is important for short waves ($\mathcal{D}/\lambda > 1$).

The results of the seismodynamic theory of the underground earthquake resistance structures found an application, in particular, when solving the problem connected with the securing of the earthquake resistance of the Tashkent metropoliten structures, which are under construction in the region with high seismicity and sedimentary ground. A number of constructive decisions on securing the earthquake resistance of the stage and station tunnels have been worked out. The new earthquake constructions of the stage tunnels and station structures from prefabricated ferro-concrete elements including the new station metropoliten constructions of the column type made from pre-fabricated large-scale volume ferro-concrete.elements have been worked out and inculcated.

The constructions of the stage tunnels of the open excavation in Tashkent metropoliten on the whole have been made from the wholesectional linings which represent right-angled closed (exclusive) ferro concrete prefabricated blocks. According to its constructive decision these constructions meet the earthquake resistance and strength requirements and the modern industrialization building requirements.

The stage tunnel constructions of the close excavation have been made in a form of shell linings made from pre fabricated ferro-concrete blocks monolithicated by specially worked out seismojoints.

The construction of the column type station in comparison with type decision have been considerably procesed taking into account seismic, engineering-geological and climate conditions. Concerning the antiseismic measures the longitudinal and transversal seismobelts have been provided which secure the joint work of the ferro-concrete elements (partition slab, wall blocks, beams, columns). The powerful ferro-concrete slab of the foundation have been provided for the uniform distribution of the loading on the ground. Besides the column type stations, the one-arched stations have been built in Tashkent metropoliten, which present the variable section arch with slide-chut. perceiving the horizontal efforts.

For the mentioned metropoliten types of constructions, on the basis of the seismodynamic theory of the underground structures, the methodology of dynamic earthquake resistance calculation has been worked out, according to the following considerations: the underground structure experience the seismic loading set by the displacement of the ground. Any point of the construction vibrating during the earthquake, get the longitudinal, transversal and tangential displasements.

The difference between the ground and structure displacements gives the relative displacements. The expressions of the displacement and force factors have been received, their numerical value have been calculated and the tension-deformation state of the constructions have been analysed. The calculation diagram having a form of the right-angled frame with rigid connections of elememts , disposed in the ground media, have been picked out from the wholesectional linings for calculating the earthquake resistance of the linings of the stage tunnel. The calculating diagram for investigation the vibrations of the linings of the stage tunnel with circular section with antiseismic joints is given as a closed ring, disposed in the ground, which consist of the separate elements, with elastic connections.

The calculating diagram in the form of the semicylindrical shell in the ground media is accepted for calculating the metropoliten stations of the one-arched type.

In the supposition of the transversal influence of the seismic forces, the vibrations are received and within the following border conditions the expressions for definition the displaceent and force factors, existing in the shell walls, have been received.

Conclusion. Thus, in the given work, the essence of the seismodynamic theory of the underground structures is set. The imformation about the experimental investigations and possible presentation of the parameters, interaction with the ground are brought, which are the initial base for seismodynamic calculation of the underground structures. For the purpose of definition the sphere of practical application and clearing use the reliability of the receivied results, the comparison of the results of simple problem solution of elastic wave influence on the underground shell in the ground, on the basis of seismodynamic and wave approach, have been carried out as well as the comparison of the result with the results of large -scale field investigations.

At present time the followingproblems are confronting us:

- the working out the basis of the theoretico-experimental investigations of the earthquake resistance and shockresistance of the underground spatial structures (metro, depositiry, large diameter pipelines, etc.);
- the working out the method of calculation of the complex underground structures system, taking into account the unlinear properties of interaction the ground with liquids for unstationary accident influence;
- the working out the effective dampers for vibroprotection of the underground structures;
- the experimental investigation of seismodynamic structures, interacted with ground, and creation of the new and improvement of the existing experimental arrangements; having output to the automizing system;
- the establishment of the empirical dependences which characterize the seismodynamic structures for the purpose of securing the theoretical statement of problems with initial information.

SECTION III: EARTHQUAKE HAZARD MITIGATION

This section contains information on vulnerability of systems to earthquake hazards and the application of engineering and geoscience knowledge to mitigation of those hazards. This information supplements and extends the following two documents, as well as numerous publications in FEMA's Earthquake Hazard Reduction Series (Appendix B):

1) Hays, W. W., 1989, Proceedings of Conference XLVIII, the 3rd annual workshop on earthquake hazards in the Puget Sound-Portland area: U.S. Geological Survey Open-File Report 89-465, 303 p.

2) Noson, L. L.; Qamar, Anthony; Thorsen, G. W., 1988, Washington State Earthquake Hazards: Washington State Department of Natural Resources Information Circular 85, 77 p.

Lessons From the October 19, 1989 Loma Prieta Earthquake

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Abstract

The Loma Prieta Earthquake that struck the San Francisco Bay Region on October 17, 1989 was the first *field test* of the efforts undertaken by the State of California and the federal government through the National Earthquake Hazards Reduction Program (NEHRP). State disaster planning, integration of scientific research into disaster preparedness and policy, local preparedness, hazard mitigation programs, and community preparedness and education activities initiated over the last decade changed the context within which Loma Prieta occurred. In evaluating the impact of NEHRP, the Loma Prieta earthquake illustrates that preparedness and hazard mitigation efforts in California changed the outcome of this disaster. This paper will describe the efforts undertaken in California prior to the earthquake, and our initial assessment of their effectiveness.

Lessons From the October 19, 1989 Loma Prieta Earthquake

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Did We Expect The October 17th Earthquake?

While the earthquake that struck northern California at 5:04 pm on October 17, 1989, was a surprise to many, it should have been expected by government officials in the San Francisco Bay Region. The history of northern California over the past 200 years is dotted with damaging seismic events similar to the Loma Prieta earthquake. In 1836 and 1868 major earthquakes stuck on the Hayward fault in the Bay Area. In 1838, and again in 1906, major quakes occurred on the San Andreas fault on the San Francisco Peninsula, damaging structures around the bay. In fact, the October 17 Loma Prieta earthquake is strikingly similar to an 1865 earthquake in the Santa Cruz Mountains that collapsed structures in San Francisco and the East Bay.

Were we expecting the Loma Prieta earthquake? We should have been. In 1983 scientists from the United States Geological Survey identified the Santa Cruz Mountain segment of the San Andreas fault as having a greater than 47% probability of causing a major earthquake within the next 30 years. In their 1988 evaluation of earthquake probabilities, the USGS once again pointed to this segment of the fault as a "seismic gap"--an area in which earthquakes had not occurred for several years, and should be expected.

Although we do not yet have the ability to predict exactly when earthquakes will occur, scientists in California have been closely monitoring activity along high probability fault segments, in an attempt to identify potential precursors to larger events. The Governor's Office of Emergency Services has been working closely with the USGS and the California Division of Mines and Geology, to develop techniques for issuing short-term earthquake advisories in areas of increased seismic risk. In June of 1988, a moderate earthquake occurred near Lake Elsman in the Santa Cruz Mountains. Because scientists felt this area was overdue for a damaging earthquake, an advisory was issued to the local government emergency response officials and the media in the South Bay, warning of the short-term increased probability of larger earthquakes. Again in August of 1989, a moderate earthquake struck the same area. A second advisory was issued, once again warning of the increased probability of larger earthquakes within five days. In both instances, the advisories expired without the larger earthquake occurring; however, local governments receiving the advisories took appropriate actions to ensure their readiness.

While these moderate events permitted short-term advisories and longer-term increased readiness, the days that preceded the October 17 earthquake provided no precursory activity to justify a short-term warning. Because of the previous advisories, however, local governments in the South Bay had increased training and tested local response capability--a capability that was put to the test on October 17.

Overview of the Earthquake

The Loma Prieta Earthquake, with a surface wave magnitude of 7.1, occurred at 5:04 Pacific Daylight Time, October 17, 1989, rupturing a 25-mile segment of the San Andreas Fault. The epicenter was located about 10 miles northeast of Santa Cruz, and 60 miles southeast of San Francisco. The mainshock, lasting 7-10 seconds, initiated at a depth of 11.5 miles (18 kilometers) beneath Mount Loma Prieta in the southern Santa Cruz Mountains. Over 5,000 aftershocks have been recorded in the two months since the earthquake. More than 100 have registered more than M3.0, and several have been greater than M5.0.

The earthquake was felt over a 400,000 square mile area. Damage from the earthquake was reported as far north as Sacramento County and as far south as Monterey County, a distance of approximately 120 miles. Sixty-two people died as a direct result of the earthquake. Most of the fatalities, 42, were caused by the collapse of a two-level elevated highway in Oakland. Approximately 4000 persons were treated for injuries throughout the 10-county disaster area. Over 14,000 persons have been left temporarily homeless by the event. Over 22,000 residential structures and scores of commercial and public buildings have been damaged or destroyed as a result of the earthquake. The regional transportation system in the Bay Area was heavily impacted by the closure of the Bay Bridge, portions of Interstate 880 in Oakland, and Highway 17 which connects the Bay Area to Santa Cruz. Electric power, natrual gas, and water and sewage systems, although heavily impacted in some areas, were restored to most residents within the first few days after the earthquake. However, in several districts, natural gas and water distribution systems are still being repaired.

The physical impact of the Loma Prieta earthquake of October 17 was predictable. Structures that were built on poor soils were more extensively damaged than those on consolidated soils. In the Marina district of San Francisco, both the violence and duration of ground shaking was increased by the poor soils. In the Santa Cruz Mountains, ancient landslides were activated by the shaking, setting in motion massive earth movements, some as large as 3 km long and 2 km wide. In Oakland, the catastrophic collapse of the Cypress viaduct may have resulted from a combination of poor soil conditions and archaic concrete design concepts. Earth scientists had warned us to expect greater damage in these areas; their forecasts have now been validated.

Response of Local Governments

To a great extent, the Loma Prieta earthquake was a series of local disasters. Each affected jurisdiction responded with its own resources, supplemented by the regional and state fire and law mutual aid system. The Governor's Office of Emergency Services activated its regional Emergency Operations Center and State Operations Center, and the Federal Emergency Management Agency established its Disaster Field Office immediately following the earthquake. However, in Watsonville, Hollister, Santa Cruz, San Francisco and Oakland, local emergency responders were able to manage the response without requesting many state or federal resources. Resource requests were primarily for generators, methods of providing potable water, medical supplies, shelter support, feeding support, and engineers. Volunteer engineers were provided by the state to assist local building departments in assessing damaged buildings, and the search and rescue expertise utilized at the collapsed Cypress structure was supplied by the Governor's Office of Emergency Services. However, the earthquake was largely a test of local governments' capabilities to respond. Pressed to the limits of their expertise, most local governments proved themselves up to the task. Training, exercises, and community preparedness programs implemented throughout the Bay region provided the basis for quick response.

At the community level, individual residents of damaged areas spontaneously responded to assist those in need of help. At the Cypress structure in Oakland, in the Marina district of San Francisco, in the central business districts of Watsonville, Hollister, Los Gatos, Santa Cruz, and on the Bay Bridge, spontaneous volunteers risked their lives to pull victims from the debris of collapsed structures. As emergency response professionals arrived at the scenes of damage, they incorporated the volunteers into their efforts to rescue victims. Californians pulled together in response to the Loma Prieta disaster; this positive response was, to a great extent, a result of the state earthquake awareness and preparedness programs. Once again, individual citizens knew what to expect and how to respond.

While the response to the Loma Prieta earthquake was handled as a local emergency, no one should become complacent about the region's ability to respond effectively to a quake of similar size closer to the urbanized center of the Bay region. A larger quake would quickly overwhelm local response. Therefore, both local response capability, and staff management skills in integrating state and federal resources will be necessary for effective response.

Mitigation Programs

Earlier mitigation efforts proved their effectiveness in the Loma Prieta earthquake. After the 1933 Long Beach earthquake, the state initiated a mitigation program with passage of the Field Act to ensure that public schools were earthquake-resistant. The state also mandated seismic building codes in every jurisdiction. It was not until 1980, however, nearly 50 years after the Long Beach disaster, that all public schools were brought into conformance with the Field Act. Although there was slight structural damage to several Field Act schools near the epicentral area, none suffered major damage or collapse.

After each earthquake, seismic codes have been updated and improved, providing an assurance that newer construction is seismically resistant, but we continue to have a significant inventory of existing buildings--both those built before seismic codes and those built to lesser codes--that we now know are inadequate in earthquakes. In the aftermath of the Coalinga earthquake, the state required every jurisdiction in seismic zone 4 (Title 24) to inventory unreinforced masonry wall and infill structures and by January 1, 1990, to develop and implement a program to mitigate these hazardous structures.

It is clear from reviewing the performance of newer structures, and those facilities regulated by the state, that high standards for seismic design, construction and inspection produced structures that were able to withstand the violence of this quake with little or no damage. Older structures, built without seismic resistance or to older seismic codes, did not fare as well. The structures we knew were vulnerable suffered the greatest damage.

Our most recent mitigation efforts in California have been focused in two areas. We have developed, at the local level, a high degree of professionalism and expertise in disaster response. Simultaneously, through the efforts of the California Seismic Safety Commission and the earthquake preparedness projects of OES, we have pressed local governments and other institutions to reduce the hazards in their jurisdictions, particularly hazardous buildings. The programs of OES include support and assistance to local governments through the California Specialized Training Institute and the Southern California and Bay Area Regional Earthquake Preparedness Projects. These programs provide the technical expertise and training necessary to assist local governments in preparing and improving their abilities to respond. Recent evaluations of these two programs and the real test of the Loma Prieta earthquake provide a clear illustration of the effectiveness of the federal and state commitment to educating, training and providing assistance to local governments and business in areas of high seismic risk. BAREPP's work in the Bay Region had a significant impact on the outcome of this event. Local governments knew what to do, many having developed mitigation programs for hazardous structures and training for their staffs; many businesses were prepared for the disruptions in transportation power, and communications; community groups and individuals knew what to do to protect themselves and the respond after the quake. Government support for preparedness and mitigation proved its cost effectiveness by reducing losses and injuries.

Unreinforced Masonry

Unreinforced masonry buildings (URMs), brick structures built without steel reinforcement to resist earthquake forces, collapsed into themselves, onto adjacent structures and onto the sidewalks and streets in Watsonville, Hollister, Santa Cruz, Los Gatos, Oakland and San Francisco. These structures, built before 1933, predominate in most older central business districts in California as they do across the United States. They pose a threat to the lives of occupants, as well as to passersby in quakes as small as Magnitude 6.

This earthquake, like Coalinga (1983) and Whittier (1987) before it, illustrates the folly of investing in urban redevelopment of central business districts through urban design, landscaping, and architectural decoration, *without* seismic strengthening of unreinforced masonry. In Santa Cruz, Los Gatos, Watsonville, and Hollister, older central business districts were rejuvenated through an investment in cosmetic beautification of URMs, many of significant architectural and historical merit. These revitalized communities were once again thriving, drawing business and commerce to their central business districts. Unfortunately, this increased the number of people at risk by raising densities in the most hazardous areas: within URMs, and on the sidewalks and streets in front of them. Many of those who perished in the Loma Prieta quake were drawn to shop and work in rehabilitated structures.

Damage to these older brick buildings around the Bay had many social and economic impacts. URMs provide a vital resource of low-cost housing and commercial space in older central cities. Damage on October 17 resulted in the loss of more than 2,000 single-room occupancy housing units and over 180 business sites, severely affecting lower-income community residents. Displacement of these residents, in communities where the vacancy rate is less than 1%, has posed a housing problem for which we do not currently have a solution. It is critical that jurisdictions provide for the basic housing needs of their residents; if affordable, safe housing is not available, an earthquake will add tens of thousands of families to the ranks of homeless in our communities. The provision of affordable housing continues to be the single greatest challenge to local governments in California.

To reduce damage to their older building stocks, many communities in California now require strengthening of URMs. In these communities, damage to and collapse of URMs has been reduced, but it is still too early to determine which of the many approaches utilized to mitigate URMs proved most effective . It is clear, however, that Bay Area communities which ignored the threat of damage to these structures are now faced with devastated central business districts and business communities struggling to survive. In a form of structural Darwinism, the earthquake destroyed the poorest quality construction, leaving the stronger buildings more or less intact. Many of these surviving URMs await the next test, a larger quake closer to the center of the Bay Area or the Los Angeles Basin. The response of URMs in these future earthquakes is both predictable and avoidable.

Wood Frame Structures

Older wood frame structures--built after the turn of the century on brick foundations, without bolting or bracing of foundation walls--also suffered extensive damage, adding to the loss of low-income housing. In Watsonville, with a prequake vacancy rate of less than one percent, more than eight percent of the housing stock was severely damaged or destroyed. Prevention of this type of damage is both easy and cost-effective. Unfortunately, while the cost of mitigating one structure was in the hundreds of dollars before the earthquake, the cost of repairing damage to the same structure following the quake may well be in the tens of thousands of dollars. These structures can readily be strengthened to withstand earthquake forces if local governments join mortgage lenders and insurance companies, all of whom have a stake in preventing damage, in requiring foundation bracing and ties in all wooden structures. If these financial institutions were as concerned with potential earthquake damage as they are with termites, the problem of damage to wooden structures would be solved by the private sector, without government intervention.

Non-Ductile Concrete Frame

The collapse of a single structure, the Cypress structure of Interstate 880, resulted in the single greatest loss of life on October 17. The catastrophic collapse of more than a mile of this non-ductile (brittle) concrete frame structure points up the greatest challenge to Californians: we have thousands of structures of this type, both in our freeway infrastructure and also in our more dense urban centers, where non-ductile concrete was utilized for midrise office, commercial and residential occupancies though the 1960s. The collapse of the Cypress presented a sobering view of the potential impact of future earthquakes, where the loss of life in the collapse of a single structure could eclipse the losses from all structural collapses in earthquakes during this century. The challenge before us at this time is to move forward quickly to inventory and strengthen these structures to ensure their seismic resistance.

It should be noted, however, that the general mitigation and response planning efforts of CalTrans and the public utilities proved effective during this earthquake. Excepting the collapsed Cypress structure and failed Bay Bridge connections--both older structures built to older seismic codes on poor soils--the retrofit of the freeway system proved its value: structures remained operational after the earthquake. Similarly, except where massive ground failure occurred, the public utilities continued to provide service with only scattered interruptions. Response capability developed within the utilities reestablished service within days to most areas. The monumental task of repairing the damaged Bay Bridge was completed in less than 30 days. In the interim, planning by the transit operators provided transbay travel on BART (Bay Area Rapid Transit), ferries, and in van pools.

Lessons for California Cities

The Loma Prieta Earthquake presents a graphic lesson to those of us in hazard mitigation and emergency response planning. First, and most important, seismic design pays off. Structures designed and built to the most recent seismic design provisions of the Uniform Building Code withstood the forces of the Loma Prieta earthquake with little or no damage. Structures built to lesser code provisions, with no seismic resistance, suffered extensive damage.

The earthquake also reinforced the notion that site conditions can be a dominant factor in determining damage to structures. We can no longer feel reassured that structures are safe if they are built outside of the Alquist-Priolo Special Studies (Fault) Zones, because poor soil conditions can make a structure vulnerable despite a relatively earthquake "safe" location. In this earthquake, damage was most severe in the Marina, in downtown Santa Cruz, and in Oakland--outside the Special Studies Zones! It is therefore important to ensure that seismic design provisions adequately address site factors. Risk mapping of existing development on poor soils can assist both emergency responders and redevelopment planners in understanding the extent of local risk.

Unreinforced masonry buildings continue to be a life threat in even moderate earthquakes. These structures can be cost-effectively strengthened to resist seismic forces. Because URMs provide a vital source of low-cost housing and commercial space in older central business districts, their strengthening and preservation is essential to the economic and social fabric of our communities.

As in the 1971 San Fernando and 1979 El Centro earthquakes, Loma Prieta illustrated the danger of non-ductile concrete structures, designed and built before the 1973 revisions to the Uniform Building Code. These structures can collapse in moderate and larger earthquakes, and have the potential for causing a catastrophic loss of life. We must inventory and retrofit these structures to withstand seismic forces.

The performance of schools, buildings housing essential services, and hospitals was ensured through enforcement of strict seismic design and construction provisions administered by the state. These facilities are essential to our communities. The regulation of their construction in California by state agencies provided a greater measure of safety and performance in this earthquake; however, it has taken almost 50 years to bring all of California's public schools into conformance with the Field Act. The outcome of Loma Prieta would have been different if the programs had not been initiated so long ago!

A seismically safe environment can not be created overnight. Therefore, emergency response capability--including urban search and rescue, rapid damage assessment, and disaster management--must be in place. With the assistance of FEMA, California has put in place a comprehensive program of training, support for local government preparedness and mitigation, and procedures to integrate state and federal resources to respond to the needs at the local level. These systems and programs would be essential in the response to an earthquake in the Los Angeles basin or on the Hayward or San Andreas faults in the Bay region, where hundreds of jurisdictions will be hit simultaneously. The capability to utilize these resources and manage the next earthquake disaster will require a commitment from each community's leadership. It will also require training and regular exercise of all critical city and county departments to ensure that plans are understood and procedures can be effectively implemented during a crisis.

The task of making our communities safe is one of making sure that new construction is safe and, more important, of abating the hazards that decades of non-seismic design have produced. A seismically safe environment can not be achieved without a political and financial commitment.

Earthquakes are unique as natural hazards. Unlike hurricanes, tornados, or floods, they most often occur without warning and complete their destruction within seconds. The outcome of an earthquake can be determined before the ground starts shaking: our planning and development decisions, the quality of our building design and construction, and our ability to respond quickly make all the difference. We believe we have changed the outcome of earthquake events through mitigation and preparedness.

Preparedness pays!

Japanese NAMAZU-E Woodcuts

Harry T. Halverson

World folklore has many fanciful concepts as to the cause of earthquakes. One of the more intriguing is that of the Japanese who attributed the many devastating earthquakes in that country to a NAMAZU (giant catfish) living underground under Hitachi province. An earthquake would be caused if this NAMAZU was permitted to thrash around. Control of the NAMAZU was the responsibility of the Kashima damyojin (Japanese god) who placed a pivot stone on its head to restrain it, but on occasion their control would falter, and the NAMAZU would lift the pivot stone and produce an earthquake.

This poster exhibit displays six photocopies of the original woodcuts dating from the 1855 EDO (Tokyo) earthquake, and discusses briefly the cultural connotations of the NAMAZU earthquake concept.

FOURTH ANNUAL WORKSHOP

NATIONAL EARTHQUAKE HAZARDS REDUCTION PROGRAM

LIFELINES SESSION

Donald Ballantyne, P.E.
Kennedy/Jenks/Chilton
206-874-0555
Moderator

Lifeline definition

1. Geographically distributed over a large area with varying seismic hazards.
2. Society is dependent on these systems for survival: life and function of institutions.
3. Network systems with interaction between components.

Systems typically considered lifelines

1. Transportation - highways, airports, ports, trains
2. Electric Power
3. Water and sewer
4. Communications - telephone, TV, radio, electronic
5. Gas and liquid fuels

First we would like to define the problem using photographs of damage to lifelines in the Loma Prieta earthquake, 17 October 1989, near San Francisco. The Modified Mercalli Intensities of 7, 8, and 9 experienced in that event and the related damage are similar to what might be expected her in the Pacific Northwest.

Here to present those photographs is Keith Eldridge with KOMO News 4. Keith serves as weekend anchor for KOMO. Before joining KOMO in 1983, he worked in Denver, and closer to earthquake country in both Fresno and San Luis Obispo California. Keith provided coverage from San Francisco following the Loma Prieta earthquake last fall.

We have seen how earthquakes can impact lifeline systems. How can we mitigate their impact on our lifeline systems here in the Pacific Northwest?

We have with us today 5 panelists who will address that question. Panelists represent lifeline systems from Vancouver, British Columbia, Seattle, Olympia, and Portland as well as a national perspective from the federal government.

Speakers today come from backgrounds in water supply, power, and highway systems.

I will pose four questions to the panel members which cover a broad base of the considerations in lifeline mitigation programs:

1. We are all aware of the potential impact an earthquake can have on our lifeline systems. How can we get a program initiated?

   Describe your experience with marketing an earthquake mitigation program to decision makers controlling your lifeline budget?

2. Once a program is initiated, how can we assess the earthquake vulnerability to our lifeline system components?

   Describe your plans or experience in assessing the earthquake vulnerability of your lifeline system? Focus on the vulnerability of system components.

3. Once system component vulnerability has been identified, how will their performance impact the overall function of the system network?

   Describe your plans or experience in estimating potential earthquake losses to your system? Discuss both dollar losses and lifeline "system network" function loss.

4. And finally, component and system weaknesses have been identified. What can be done to mitigate earthquake impact?

   Describe your plans or experience with measures which will result in directly reducing the impact of an earthquake on your system? Such measures could include upgrade of physical structures or equipment, or emergency planning.

Now I would like to present our distinguished panel members:

1. Mr. Walter Anton - Chief Engineer, Seattle Water Department

2. Mr. Allan Walley - Bridge Engineer, Washington State DOT

3. Mr. J. D. Cattanach - B.C. Hydro

4. Mr. William Elliott - Portland Water Bureau

5. Mr. Ken Sullivan - Federal Emergency Management Agency

LIFELINE DAMAGE FROM THE LOMA PRIETA EARTHQUAKE

NEHRP Workshop, 19 April 1990

Presented by Keith Eldridge, KOMO
Prepared by Don Ballantyne, Kennedy/Jenks/Chilton

On 17 October, 1989, over 60 people were killed in the magnitude 7.1 Loma Prieta earthquake. The earthquake, whose epicenter was located in the Santa Cruz Mountains between San Jose and Santa Cruz, caused over six billion dollars damage.

The Cypress viaduct collapse on highway 880 resulted in two thirds of the earthquake's deaths. Collapse of a section of the Oakland Bay Bridge cut off 200,000 commuters daily from passing between Oakland and San Francisco. Repair only took one month. Slides on Highway 17 between San Jose and Santa Cruz closed the road with complete renovation taking over 30 days. A bridge crossing Struves Slough on Highway 1 north of Watsonville was lifted of its supporting pile foundation and came to rest after moving laterally. That resulted in the piles puncturing the road section.

The earthquake fault, with an offset of 7.5 feet, occurred between the North American and Pacific plates at a depth of 11.5 miles below the earth's surface. A Modified Mercalli Intensity of 8 resulted at the epicenter with intensities as high as 9 occurring in the San Francisco Marina District and Oakland.

Power was lost regionally with some areas being blacked out for four days. Damage to a 500-kilovolt switch yard at this Pacific Gas and Electric Moss Landing power plant was one of the primary causes. A raw water tank at that site split at the bottom emptying its contents. The resulting vacuum imploded the tank roof. Engine-generator sets such as this one at a Pajaro Water District installation south of Watsonville, and this one at the Palo Alto wastewater treatment plant were critical in maintaining lifeline system function.

Liquefaction at the Oakland Airport's main 10,000 foot-long runway resulted in its closure. Upon reopening it was limited to 7,000 feet of usable length. One window in the San Francisco airport control tower fell out which, along with other disruption, caused airport closure. Resumption of full service took 2 days.

Liquefaction and settlement at the Port of Oakland's Seventh Street facility made container crane operation impossible. An estimated $75 million in damage was estimated.

Over 100 water main breaks in San Francisco's Marina District resulted in fire control problems. The fire control problem was compounded by natural gas pipeline damage resulting in fire. Ground motion amplified by soft soils damaged building structures. Water continued to be in great demand. Water pipeline break locations correspond closely to fill areas. Liquefaction and permanent ground deformation was the primary pipe damage mechanism. Even the City's Auxiliary Water Supply System built specifically to be earthquake resistant did not function as planned because of operational error. An emergency operation center was set up in a neighborhood school. While pipeline damage was repaired, water shortage continued to impact the community. The same level of damage is expected to be identified in the city sanitary sewer system when its investigation is complete.

Closer to the epicenter, the Santa Clara Valley Water District's Rinconada water treatment plant was devastated by sloshing water. Nearly half the radial launders were ripped from their mounts putting the 80 MGD plant out of service. Repair is expected to cost $1.5 million. At the same plant proper anchorage of gas cylinders kept them in place. A chlorine cylinder owned by another water purveyor broke loose, severing piping and releasing a chlorine plume. The area was evacuated. Emergency battery racks remained intact having incorporated earthquake resistant design. Most lab chemicals were kept on the shelf using innovative designs.

Sludge digester cover guides were damaged from sloshing sludge at San Jose, East Bay Municipal Utility District in Oakland, and as far away as Sacramento. Sloshing sewage damaged clarifier baffles at two wastewater treatment plants in the south bay area. Scum troughs broke and fell into clarifiers at Palo Alto. Sloshing sewage in Palo Alto's primary clarifiers pushed aluminum hatches out of their frame. Plant operators then had to go fishing to pull them from the bottom of the tank.

The East Bay Municipal Utility District's Sobrante Water Treatment Plant was shut down for four days as a result of failure of a 60 inch prestressed concrete cylinder raw water line. Stored water provided adequate service until the line was repaired. Compression failure in a 20 inch cast iron pipe under 20 feet of fill also impacted the District.

A 1.1 million gallon post tensioned concrete tank owned by the Parissima Water District split 4 inches at a seam discharging its contents. The release ran towards Interstate 280 through four finely landscaped yards, collapsing a garage, and carrying mud into a house.

A bolted steel tank owned by a small water utility in the Santa Cruz Mountains buckled at the base.

A water tank on the roof of the Amfac Hotel near the San Francisco Airport fell through the elevator shaft, requiring closure of the hotel. No one was injured.

At the Richmond petroleum terminal, an unanchored tank rocked resulting in elephant's foot buckling.

Over 100 dams were within a 60 mile radius of the epicenter. Of the 12 subjected to heavy shaking, 9 showed evidence of damage. The Newell Dam, holding the city of Santa Cruz water supply in Loch Lomond had small fissures open on the face.

In the Santa Cruz Mountains, the area hardest hit by the earthquake, the Redwood Estates water system, with 400 services, was not completely restored for five months. Portable showers and water tanks were brought in to meet people's need.

In Scott's Valley, home of Seagate computer disk drives, a 1 million gallon water tank connection snapped when the tank rocked, releasing its contents.
The bedding around pipelines leading to the Santa Cruz wastewater treatment plant liquefied. The pipelines are suspect for damage. A raw sewage force main broke in Santa Cruz releasing raw sewage into the San Lorenzo River. In Santa Cruz, permanent ground deformation resulted in over 100 water line breaks putting two city hospitals without water. Several fires erupted. The levee along the San Lorenzo River slumped. Building damage was extensive including the Santa Cruz Police headquarters and the downtown mall. Businesses such as Zoccolli's delicatessen relocated to stay in business.

Liquefaction and slumping along the Pajaro River, south of Watsonville, opened a fissure, pulling apart a water line and reversing the flow in a gravity sewer main.

Finally at Moss Landing, extensive liquefaction and slumping resulted in offsetting the top half of this sewage pump station.

OVERVIEW OF LIFELINE EARTHQUAKE ENGINEERING
PANEL DISCUSSION

ABSTRACT OF RESPONSES

by WALTER F. ANTON, CHIEF ENGINEER
SEATTLE WATER DEPARTMENT

During the past ten years, the Seattle Water Department has carried out a seismic reliability evaluation of its many water system facilities. While the water system facilities are generally expected to remain operable in the event of a major earthquake, some damage and leakage is expected which could reduce the capability of the Department to provide water in sufficient quantity and pressure for drinking, fire fighting and sanitation throughout its service area. City officials have supported the Department's $16 million seismic upgrade program, which includes a new dam, reservoir lining, improved transmission pipeline supports, strengthening of elevated tanks and standpipes, and structural strengthening and more secure anchorages at many of the operating facilities. The Department's Emergency Response Plan is being modified to improve the Department's response readiness and repair capability.

EA13.61.1

OVERVIEW OF LIFELINE EARTHQUAKE ENGINEERING

PANEL DISCUSSION
RESPONSES TO FOUR TOPICS

by WALTER F. ANTON, CHIEF ENGINEER
SEATTLE WATER DEPARTMENT

1. MARKETING AN EARTHQUAKE MITIGATION PROGRAM

The marketing of Seattle Water Department's earthquake mitigation program has not been particularly difficult as the program has evolved over the past ten years.

Initially, the Water Department and City Council readily budgeted the necessary funds to comply with the requirements of the National Dam Safety Program to evaluate the earthquake vulnerability of the large and small dams in the water system and to carry out the seismic strengthening work identified. The last of this remedial work is in this year's capital improvement program budget.

The next major effort involved the comprehensive review of the seismic reliability of all other water system facilities, including the review of the Department's emergency response plan. The City Council authorized the $192,000 consultant study in the Water Department's 1988 budget.

This seismic reliability study was carried out under Water Department management by Cygna - a San Francisco area engineering firm that was augmented by Seattle area subconsultants.

The next major step was getting budget authority for the addition of the $9 million Seismic Upgrade Program in the Water Department's 1990 Capital Improvement Program based on the initial results of Cygna's seismic evaluation. Recent media coverage accompanying the publication of the final results of Cygna's evaluation has now acquainted the public with the need for the upgrade work -- now a $12 million program to be carried out during the next five to seven years. As a result, we expect positive support from the City Council as we carry out the necessary work.

2. ASSESSING EARTHQUAKE VULNERABILITY OF SEATTLE WATER SYSTEM

The Seattle Water Department puts top priority on public safety and is constantly looking at ways to improve the water system and reduce risks to the population and our drinking water. To this end, the Seattle Water Department engaged Cygna (a San Francisco area engineering firm augmented by Seattle subconsultants) in 1988 to perform a comprehensive review of the reliability of water system facilities in the event of a major earthquake.

This detailed analysis of major pipelines, pumping stations, treatment plants, control centers, and tank type distribution storage reservoirs (tank type) augmented earlier studies of the Cedar Falls Dam, Tolt Dam, and embankment type reservoirs.

The evaluation considered 6.5 magnitude and 7.5 magnitude earthquake events (with one-in-a-hundred and one-in-five-hundred years recurrence intervals, respectively) centered under the facility studied. The study also considered a larger magnitude subduction event with an epicenter located near the coast of Washington.

Overall, the water system facilities are generally expected to remain operable in the event of a major earthquake; damage sustained would be repaired on a priority basis. Specific results of the study are as follows:

- The Water Operations Control Center (Administrative Building) -- the nerve-center of the water supply system which houses the equipment that directs water delivery functions -- was determined to be adequately designed to withstand a major earthquake.

- Cedar Falls Dam and Tolt Dam and all but one embankment type distribution reservoir are expected to survive a severe earthquake without serious damage.

  The earth dam at Lake Youngs (a key regulating reservoir) in the Maple Valley area and the earth embankment at Maple Leaf distribution reservoir in north Seattle were determined to be subject to liquifaction damage.

- Seven elevated water tanks and nine standpipes located throughout the Seattle area are among the more vulnerable facilities in the system. All but three of these structures were found to need additional support and anchorage to adequately withstand a major earthquake. Out of sixteen storage structures in the City's distribution area, thirteen structures may not be operational after a 7.5 earthquake and nine of these structures may not be operable in the event of a magnitude 6.5 event. Although these reservoirs would be damaged and may leak, it is unlikely that they would collapse during an earthquake.

- Throughout the system, the anchorage of control panels, electrical equipment, consoles and roof-to-wall connections need to be strengthened.

- The supports of some water transmission lines need to be improved. This work will include improving the support of the major single feed supply line to Mercer Island.

- Some damage will occur to distribution system watermains at locations where soil conditions are sandy, wet and unstable -- similar to cast iron pipeline breaks that occurred during past major earthquakes in Western Washington.

3. ESTIMATING POTENTIAL EARTHQUAKE LOSSES TO WATER SYSTEM

Following a large earthquake, the Seattle Water Department expects to be able to provide water from existing sources to a majority of its service area. However, damage to water mains, some tanks and operating facilities would reduce the Department's ability to provide the normal level of pressure, flow and reserve water.

Severe water use restrictions may have to go into effect to provide water for drinking, fire fighting and sanitation.

Localized water main and sewer pipe breaks could result in severe loss of pressure and possible contamination. In the event of an earthquake, SWD would issue an immediate "boil water" order through the media until water quality is assured.

Since it is the Water Department's obligation to protect the public; to minimize property damage, and to provide water for drinking, fire fighting and sanitation following a major earthquake, there was little question in the mind of Water Department management that it was prudent and sound utilities management to embark on the $12 million upgrade program recommended by Cygna and to continue with the $4 million seismic upgrade work previously underway. Any effort to assign a dollar value to public safety or potential property damage resulting from the escape of stored contents from a storage structure is difficult at best. It was obvious that even the most modest monetary estimate of potential public risk would likely far exceed the renovation cost for the specific elevated storage tank or standpipe even if complete replacement of the structure is necessary.

4. PLANS FOR REDUCING IMPACT OF EARTHQUAKE ON WATER SYSTEM

The Seattle Water Department has already embarked on a $16 million program to strengthen its facilities to resist a severe earthquake. The program includes:

- Constructing a new embankment dam immediately downstream of the vulnerable Lake Youngs Dam (underway with completion by mid 1990).
- Lining Maple Leaf Reservoir to prevent leakage water from causing a liquifaction type foundation failure (underway with completion by fall 1990).
- Pipeline tie-down measures and improving the support of the Mercer Island supply line.
- Augmenting the bracing, connections, and anchorage of eight elevated water tanks (design to begin in 1990 with strengthening between 1991 and 1995). Three of the tanks that need upgrading are being left empty during low water use months to reduce damage potential during a possible earthquake.

- Improving the anchorage or replacement of seven standpipes (design to begin in 1992 with construction between 1993 and 1995).
- Improving anchorage of equipment and structure modification of vulnerable pumping and treatment facilities. This work is being initiated on a priority basis.
- Although the Water Department's Emergency Response Plan already includes most of the basic elements normally included in an earthquake prepardness plan, some modifications will be made to improve the Department's response readiness and repair capability. Planned additions include formal plans to assess earthquake damage, prioritizing repair work after an earthquake, additional stockpiling of repair parts and equipment, and formalizing mutual aid agreements with other utilities.

EA13.62.2

Ken Sullivan
Federal Emergency Management Agency

Q.1 Describe your experience with marketing an earthquake mitigation program to decision-makers controlling your lifelines budget.

A.1 The decision-makers for FEMA are the members of Congress and the President. We have to convince them (and the American people) of the importance of earthquake hazard mitigation for the whole United States.

Lifelines comprise:

(1) Transportation (airports, highways, waterways, railroads, oceans, tunnels, subways, ports, etc.)

(2) Communications (telephone lines overhead and underground, radio towers, communication disks [satellite], TV towers, and stations, etc.)

(3) Electric Power (towers & lines, nuclear power plants, dams, solar power, coal plants, oil-fired plants, etc.)

(4) Water and Sewage (distribution lines, water towers, treatment plants, etc.)

(5) Fuel Transmission Facilities (oil, natural gas, steam, propane tanks, fuel lines [above and below ground], etc.).

The key to the lifeline mitigation program is:

(1) Inventory all existing lifeline systems in the United States.

(2) Assess which of those lifelines are the most vulnerable.

(3) Implement a program to strengthen the most vulnerable lifelines.

Marketing this program involves making people aware of the danger and then convincing the decision-makers of the nation as to how much of their resources can be used to mitigate the future danger in all areas of the country. I will cover how we "reach out" to the decision-makers later in this presentation.

Q.2 Describe your plans or experience in assessing the earthquake vulnerability of your lifeline systems. Focus on vulnerability components.

A.2 FEMA's concerns cross the spectrum of lifeline systems. Thus, our plan includes all lifelines in the United States.

Our past experiences have involved establishing contacts with Building Seismic Safety Council (BSSC) to establish an over-all plan for lifelines. With FEMA funding, a workshop was convened in Denver, Colorado in 1986. The workshop attracted professionals from all over the United States and resulted in six published volumes of material – one for each of the five lifelines and one covering political, social, economic, legal, and regulatory issues. A final volume: *Abatement of Seismic Hazards to Lifelines – An Action Plan* was produced to summarize all of the workshop's six volumes. The FEMA published documents are contained in FEMA 135-143.

FEMA then had a panel of experts convened by National Institute of Building Sciences (NIBS) in 1989 to review and distill all the data into one national plan – *Strategies and Approaches for Implementing a Comprehensive Program to Mitigate the Risks to Lifelines from Earthquakes and other Natural Hazards.*

FEMA is the lead agency for this program, but they can't do it all. Therefore, FEMA has developed an Agency plan – *An Evaluation and Planning Report, the Lifelines Segment of the FEMA Earthquake Program.*

The objectives of the Lifelines Segment of the FEMA Earthquake Program are (1) to reduce the vulnerability of lifeline systems in the United States to seismic hazards in order to save lives and property and (2) to avoid catastrophic national disruptions of lifeline services when earthquakes occur.

The goals of this program are to:

- Increase the awareness of lifelines systems providers, designers, builders, managers, operators, and users of potential seismic hazards to their systems and what can be done to reduce vulnerability to these hazards.
- Expand the knowledge base of vulnerability of lifelines systems to seismic hazards.
- Increase the availability of technical publications, guidelines, design criteria, standards, and model codes needed for improving the seismic resistance of lifeline systems.

Q.3 Describe your plans or experience in estimating potential earthquake losses to your system. Discuss dollar losses and lifeline "system network" functional loss.

A.3 Again, – FEMA's program addresses <u>all</u> lifelines throughout the United States. Lifeline systems in the United States have been estimated by NIBS to cost $92.9 Billion.

The cost of rebuilding the lifeline systems depends on where the earthquake occurs, how close the community is to the epicenter, and how well the vulnerable area is prepared to meet the threat.

The lifeline loss potential includes (but is not limited to):

(1) Lives

(2) Injuries

(3) Assets (fuel transmission facilities, transportation facilities, water and sewerage facilities, electric power, and telecommunication facilities)

(4) Work Productivity (lost time)

(5) Economic Markets.

(A good book on this subject is FEMA's *Estimating Losses from Future Earthquakes – A Panel Report from the National Research Council,* 1989.)

Q.4 Describe your plans or experience with measures which will results in directly reducing the impact of an earthquake on your system; such measures could include upgrade of physical structures or equipment, or emergency planning.

A.4 Currently FEMA has the following ongoing or planned projects for lifelines this year:

(1) Vulnerability Assessment and Impact of Disruptions to Lifeline Systems

This FEMA project is with the Applied Technology Council for a study to (1) develop a national overview evaluation of the overall extent and distribution of lifeline systems and the potential consequences of earthquake damage and disruptions to major lifeline systems, and (2) develop and verify a practical approach for assessing the vulnerability of lifelines and the impact of failure and disruption for at least one selected lifeline system in a selected site (region) that is high prone to earthquake damage and recommend priority steps for reducing the impact. The high-prone system selected is the water supply system for San Francisco, California.

(2) Study of Existing Federal Practices on Lifeline Systems

Studies are conducted on Lifeline Systems where the Federal Government owns, operates, leases, regulates, and finances these facilities. The first report, due shortly, will be on Electric Power and Telecommunications. The next reports, individually, will be on:

- Fuel Transmission
- Water and Sewer Systems
- Transportation Systems

(3) Cajon Pass study of the Risks Posed by the Current Placement of Lifeline Systems in the San Bernardino, California Region

The purpose of this project is to conduct a vulnerability study of the current placement of lifeline systems in the Cajon Pass. The study would address the risks that lifelines pose from earthquakes, including multiple and serial events, The study will provide recommendations on means to mitigate the risks identified. The means may involve lifeline separation, new design, or new construction methods and materials. The recommendations will be applicable to locations beyond the Cajon Pass area. The Cajon Pass lifelines accident in May of 1989 will be used to provide "lessons learned" and insights for mitigation techniques that can be developed and applied there and elsewhere in the United States.

(4) Coordination and Guidance of Lifeline Seismic Hazard Reduction

Because "lifelines" embraces such a broad spectrum of organizational and technical activities, comprehensive coordination and guidance are essential.

Subject to availability of funds, we plan to initiate a lifelines safety council similar to the BSSC so as to bring together all the diverse factions in the country on lifelines. This council would:

- Stimulate the various sectors of the lifelines community to support and participate in the lifelines hazard reduction program, including the areas of design, construction, and maintenance.
- Recommend actions to improve the lifelines awareness, dissemination, and application activities.
- Assess the adequacy of educational and technical materials and recommend projects for development of materials needed to fill gaps.
- Evaluate dissemination and application activities and develop distribution techniques and networks useful to all of the projects in the Lifelines Segment.
- Develop means to keep the lifelines community informed on a regular basis of the progress of projects underway, of their results, and of the availability of published products – through existing (or where needed, new) newsletters, periodicals, training publications, or other appropriate channels.

DAM SAFETY CONSIDERATIONS

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ABSTRACT

The principal dam safety concerns arising from earthquake loadings on earthen embankments are:

1. The assessment of liquefaction potential, and
2. The estimation of seismic induced deformations.

The peak bedrock acceleration values at a site by themselves are insufficient to assess the above issues. It is necessary to know the number of cycles expected for differing "equivalent acceleration" levels. An "equivalent acceleration" is the uniform cyclic acceleration level that produces approximately the same effect as the varying acceleration levels of a typical earthquake time history.

In conducting periodic inspections of dams where past seismic analyses are inadequate, the Dam Safety Section does a simplified assessment of the seismic stability. This is essentially a coarse screening to determine whether the projects warrant more detailed analysis by the Owners' consulting engineers. This screening involves selecting an appropriate time history for the bedrock motion and scaling it to an appropriate peak acceleration level for the site. This scaled time history is then used as the bedrock motion in the SHAKE program to model the amplification of bedrock motion in the soil column.

In the past the Dam Safety Section has used USGS Open-File Report 80-471 to appropriately scale the peak acceleration of the bedrock motion at a project site. It was assumed that in the Puget Sound Lowlands the mapped accelerations in this report were controlled by the large magnitude earthquakes in the subducting Juan de Fuca Plate. However, it has recently been learned that the USGS maps do not appropriately reflect the impact of seismicity in the deeper, subducting Juan de Fuca Plate. For annual exceedance probabilities of .01 or less (return period of 100 years or more), the accelerations generated by shallow, smaller magnitude earthquakes are larger than those generated by higher magnitude, deep-seated earthquakes. The deeper large magnitude earthquakes produce considerably greater durations of strong ground motion than the shallow, lower magnitude earthquakes in the upper crust. A smaller peak acceleration for a longer duration of strong ground motion poses potentially greater ground failure problems than a few cycles of larger acceleration produced by shallow, lower magnitude earthquakes. A number of models are being used in practice that provide information on the peak bedrock motions associated with both deep-seated as well as near-field, small earthquakes.

The USGS maps of peak accelerations used the Schnabel and Seed attenuation relationships that largely reflect data from California. The appropriateness of this relationship to the deeper seismicity of the subducting Juan de Fuca Plate is questionable. However, these attenuation relationships were deemed of sufficient accuracy for screening projects for gross inadequacies.

In the Puget Sound Lowlands the Dam Safety Section currently accepts designs based on ground motions produced by deep-seated earthquakes (M_s 7.5) in the subducting Juan de Fuca Plate and shallow (M_s 6.5) events in the over-riding North American Plate. Project proponents are informed of the controversy regarding the interface events and encouraged to consider designing to accommodate the greater ground motions potentially associated with these very large earthquakes. At this time the Dam Safety Section does not require projects to be designed for large interface events.

To conduct a coarse screening of projects, the Dam Safety Section has assumed that the deconvoluted accelerogram for the M_s 7.1 1949 Olympia earthquake will have an acceleration spectra representative of a M_s 7.5 event. This deconvoluted bedrock accelerogram was assumed to produce within the soil column and embankment a reasonable approximation of the maximum stresses under a M_s 7.5 earthquake. However, it seemed unlikely that the use of this accelerogram would give a reasonable approximation of the effects of the longer duration of strong ground motion associated with greater magnitude earthquakes. In practice, the equivalent stress at a particular point in the embankment cross-section from SHAKE is used. But, the time-history of stresses from SHAKE is not used to determine the equivalent number of cycles for a representative uniform stress. Instead, an empirical approach has been taken that a M_s 7.5 event would produce 15 cycles of the equivalent stress.

The maps of peak acceleration values potentially could provide a very helpful tool in performing initial screening of seismic hazards. However, simply mapping peak accelerations without regard to the associated duration of strong bedrock motion severely restricts the utility of these reports. Ideally, future acceleration maps will address this issue.

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Development of Inventory and Seismic Loss Estimation Model for Portland, Oregon Water and Sewer Systems

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Overview

This presentation is based on a study of expected earthquake damage to water and sewer systems in Portland, Oregon. This work is supported by a grant from the National Earthquake Hazards Reduction Program (NEHRP) of the United States Geological Survey (USGS). The study addresses two sewage drainage basins to demonstrate the applicability of the approach. The findings from this research will help guide actions that can be taken to inventory and display the effects that earthquakes would have on the water and sewer lifelines in an urban setting.

Objective

The objectives have been to develop methodologies for determining damage estimates and to develop loss estimating algorithms so that the importance of damage can be better displayed to decision makers. Objectives further are to utilize readily available technology (personal computers) and to discuss how these approaches can be used for mitigation planning and in actual disaster situations.

Scope

Two demonstration areas were chosen in Portland, each with different characteristics and different mixes of water and sewer facilities. The demonstration areas are as follows: the Tanner Basin includes important water delivery and storage facilities and sewage collection and pumping facilities serving a portion of the central business district; the Fiske Basin is a densely developed residential area and includes the principal sewage treatment plant for Portland, the Columbia Boulevard plant.

Project Status

To date, field inventories have been completed and geographic information systems (GIS) have been created. In particular the following are in place:

- Dr. Wang of Old Dominion University in Norfolk, Virginia has developed a graphical information system and prepared the overlays including transportation system, parcels, water facilities, sewer facilities, geology, seismology, ground shaking intensity, and the relationships of earthquake intensity to distributed facilities (buried pipelines).

- Concentrated facilities have been field reviewed and inventoried by Don Ballantyne of Kennedy-Jenks-Chilton Engineers. Inventory formats for buildings, equipment, and interior piping systems have been developed and used. Loss experience research has been reviewed. Replacement costs for facilities and their components and loss algorithms for water and sewage facilities have been developed.

- The City of Portland has reviewed the Federal Disaster Relief and Restoration system and prepared a flow chart of activities for display. The relationships between earthquake magnitude and ground shaking intensity have been described so that nonscientific readers can better understand the cause and effect relationship.

Limitations

With regard to distributed facilities (pipelines), since the two study areas represent only 9 percent of the served area (2,500 acres of 27,500 acres) and each basin has unique water or sewer facilities, no attempt has been made to extrapolate and make a direct correlation to city-wide damage.

With regard to concentrated facilities, some system wide locations have been reviewed in detail, but not all operating facilities are represented in the reviews. The demonstration project looked in detail at the Columbia Blvd. Waste Water Treatment Plant, the Headworks and Bull Run source intake facilities, and the Groundwater well and pumping facility. All of these facilities serve the entire city. Evaluations also look at key structures and storage facilities that serve the test areas and that are an integral part of the service to or from the test areas. Thereby, the damages and losses portrayed are not inclusive of the entire water or sewer systems and serve only to demonstrate the approaches and their feasibility.

With regard to earthquake magnitude and ground shaking from an earthquake, the study used the best available information on geology and seismicity to develop two earthquake scenarios. Since there is very little information on ground shaking intensity, the study utilized published information on local effects following the May, 1968 Portland earthquake. Since the State of Oregon plans to undertake a rigorous ground response effort, better information will be available.

Since Portland's last experience with Federal Disaster assistance in 1972, there have been many changes in Federal, State and local funding, approach and requirements. The section on disaster declarations and information needs draws on interviews with Oregon and Washington State emergency planning officials as well as FEMA Region X staff.

Findings

Although information on pipeline damages (distributive facilities) is not yet complete, algorithms have been developed and the approach is being utilized to demonstrate how earthquakes will affect the water and sewer pipes.

Particular findings are as follows:

- Two different GIS software systems have been used on the separate drainage basins to better understand the data needs and ease of data analysis. Each has advantages and disadvantages in the PC environment that will be discussed more fully in the final report.

- Graphic and data layers have been developed for geology, seismicity, complete water and sewer pipeline systems, and the relationships of supply, storage, and pumping that serve the test areas.

- Graphic and data layers grouping factors such as geology and seismicity to develop ground response and liquifaction potential maps have been demonstrated.

- Two scenario earthquakes have been applied producing ground response maps for each of the basins.

- Loss algorithms (relationships) have been prepared to display the effects of ground response on selected water facilities and sewer facilities.

- Loss curves have been chosen to describe ground response and damage.

- Earthquake ground response has been applied to concentrated and distributed facilities and loss estimates developed.

- The relationship of earthquake intensity and ground response has been discussed.

- The activities and functions involved in emergency and disaster response have been discussed.

Implications

The broad implications of these findings are that ways and means are available to inventory and model complete water and sewer systems and to describe the expected effects of damaging earthquakes on these facilities. These model approaches in inventorying and developing replacement costs can be used for multiple purposes. Some of those purposes include mitigation planning and programming, emergency response, and documentation of existing systems. To the author's knowledge this is the first lifeline analysis that includes detailed treatment of inventories, modeling, damage, restoration, and planning applications. The relative ease of developing these tools will be more clearly displayed in the final report prepared for the USGS NEHRP program.

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LOSS ESTIMATES AS UNKNOWN NUMBERS

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Abstract

This discussion addresses two types of loss estimates -- region-wide dollar estimates of prospective losses and vulnerability assessments -- in considering the uses of loss estimates. The limits in producing and using the region wide estimates are discussed, casting doubts on efforts to provide more refined estimates of prospective regional losses. Vulnerability assessments tied to decisions about upgrading facilities are found to be more useful. Several implications concerning production and use of loss estimates are drawn from the discussion.

Introduction

In guidebooks and commentary about formulation of earthquake and other hazards policies, there is a strong presumption of rationality in the way policies are formed and programs are implemented. Armed with sufficient knowledge about earthquake risks and estimates of prospective losses, the reasoning goes, rational decision-makers will choose appropriate actions to reduce earthquake risks and avert prospective losses. Similarly argued is the need for such information as a basis for rational decisions about the priorities for earthquake research and planning activities among different geographic areas of the country. At an even broader level of decision-making, it is argued that determining the types of hazards with priority for federal attention requires information about potential losses for each hazard type. The scientist's contribution is to produce knowledge about risks and to quantify losses as a prelude to policy action.

When one asks what use has been made of the loss estimates that have been produced, the results are underwhelming. In a very useful technical review of over thirty loss estimation studies, Robert Reitherman comments: "It is easy to cite numerous damage estimate methods, but difficult to cite even a few examples of how these estimates have been put to practical use" (1985, p. 811). A recent National Research Council panel charged with developing a compendium on earthquake loss estimation methodology found it difficult to identify users of earthquake loss estimates (NRC, 1989).

Given such a mis-match between the assumptions and the realities of decision-making, something must be wrong. But what is it? The standard responses are that existing loss estimates need to be refined; that more comparable sets of loss estimates need to be produced for different regions; that better ways of communicating loss estimates need to be developed; and that users need to be better educated in applying loss estimates to their needs. While improvements can surely be made in the quality of loss estimates, conventional wisdom is that the problems with existing large scale studies of urban area losses (i.e., those produced for Los Angeles, San Francisco, the Puget Sound area, and elsewhere in the 1970's) is not a lack of knowledge, but in putting such information to use.

My reasoning is different. I have come to the conclusion that too much emphasis in talking about earthquake policy formulation and preparedness planning is placed on the presumed need for such region-wide loss estimates. Region-wide estimates of dollar value of losses, or of potential deaths or casualties, have but limited utility. I refer to loss estimates as "unknown numbers" in part because of the uncertainties about their value, but also because of the myths that often develop around such unknown numbers. The interesting part for those of us who have an interest in studying policy formulation is how such estimates get aggrandized, or otherwise distorted, in policy debates.

Although region-wide loss estimates have limited utility, other types of loss estimates are potentially useful. In particular, the process of conducting vulnerability analyses and the resultant assessments for particular classes of structures or systems can be very useful. As such, we should be talking much more about vulnerabilities of key elements of our physical and social systems than region-wide dollar losses.

What follows is an elaboration of these conclusions drawing mainly on my research about earthquake risk reduction efforts in the Puget Sound and Portland areas (see May, 1989). This work has involved interviews with some 170 people who have responsibilities for building regulation, land use regulation, and facilities management. These includes individuals at the state and local level, from cities, counties, special districts, and utilities.

Loss Estimates Come In Many Forms

So that I not be accused of throwing the baby out with the bath water, it is useful to make some distinctions in the varieties of loss estimates. Rather than attempting my own classification scheme, let me simply cite the distinctions Reitherman (1985, p. 811) makes in his review of loss estimation methodologies, quoting directly his comments about each type of estimate:[1]

> *Life safety estimate* (predicted casualties, qualitative scale of safety ratings, statistical scale of safety statements, etc.). This is frequently the most emphasized aspect of earthquake damage estimates...
>
> *Property loss estimate* (such as predicted percentage of replacement value damaged). The insurance industry actively uses such information in very practical ways in writing earthquake insurance policies... This is the most common practical use to which earthquake hazard ratings are put at present...
>
> *Estimated post-earthquake impact on facility function* (whether a generator will supply power, or the chance that the generator will supply power ...). This information is less frequently supplied by existing rating methods...

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[1] A different set of distinctions is made in the National Research Council (1989, working paper A, pp. 85-99) report on loss estimation methodologies. A distinction is made among potential uses of loss estimates -- general, hazard reduction, emergency planning, financial risk, and economic impact.

> *Weak point identification* (pointing out the portions of the building that will probably be responsible for damage and which, if upgraded, would allow for much better performance). Perhaps least commonly included in existing methods is an identification of the weak links in a facility...

In my scheme of things, the first two types of estimates refer to *losses* -- whether quantified in terms of absolute dollar losses or percentage of building value or stock -- whereas the latter two types of estimates refer to *vulnerabilities.* Reitherman points out that most of the federal funding for loss estimates has been toward producing the first type of estimate, whereas funding the latter types of more location-specific or system-specific vulnerability assessments have been left up to the owners or trustees of the various systems.

Loss Estimates for the Puget Sound Area

Regional-loss estimates for the Puget Sound area project very large losses -- even discounting for a host of uncertainties in the estimates -- when considering potential major, credible events. Whether put in dollar terms or more human terms of potential life loss and homeless, the numbers concern billions of dollars or thousands of affected individuals.

The most comprehensive regional loss-estimate study that has been undertaken for this region was the USGS activity in the early 1970s to characterize potential losses from two different scenario earthquakes for six counties in the Puget Sound region (Hopper et al., 1975). Both scenarios involved Richter magnitude 7.5 earthquakes, but with different epicenters and assumptions about the time of day of the event. As a worst case, the USGS estimated 2,200 deaths, 8,700 people requiring hospitalization or immediate medical treatment, and as many as 23,500 people left homeless. No dollar values were put on prospective losses.

Various factors can be cited as suggesting a potential for even greater damage than projected in the 1975 study. As noted in the 1986 Washington Seismic Safety Council Report, there is greater credence to the potential for a magnitude 8.0 or greater subduction-style event. The state's population growth in the six county area (as of 1985) has increased by some 25 percent, with the growth in property values being over 240 percent over the 1975-85 decade. In addition, there is greater recognition of the interplay between earthquakes and secondary damages resulting from fires, chemical spills, and so forth. Counteracting these factors are suggestions that building practices have improved as the result of strengthened state building codes and building regulatory practices, and that emergency preparedness has improved with greater recognition of the earthquake risk.

A more recent preliminary assessment was made for a consortium of insurance companies in developing background materials for the consortium's proposal for a national earthquake reinsurance program (National Committee on Property Insurance, 1989, Appendix E). Assuming a scenario of a 7.25 Richter magnitude event occurring in different locations within the Puget Sound area the preliminary estimate of potential residential property losses are between $1.7 and $7.1 billion dollars.[2] As noted in the study, these estimates a very general and are very sensitive to assumptions about the assumed events.

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[2] No estimates are provided for non-residential property losses. The study was undertaken to estimate the affordability of residential earthquake insurance under a federal-backed insurance program.

Loss Estimates as Unknown Numbers

Guidelines for assessing loss estimates underscore the inherently uncertain aspects of estimating essentially unknown numbers. The assumptions that must be made, and the limited data available, result in cautionary remarks like those found in the National Research Council review (1989, p.3):

> Even using the best of today's methods and the most experienced expert opinion, losses caused by scenario earthquakes can only be estimated approximately. Overall property loss estimates are often uncertain by a factor of 2 to 3, and estimates of casualties and homeless can be uncertain by a factor of 10.

Earthquake loss estimates are not alone in these respects, as similar problems are evident in documenting the extent of many social problems -- the extent of homelessness, the value of crime attributable to drug use, the number of children annually abducted by strangers, the number of illegal immigrants, and so on.

It is interesting to note that even after earthquakes or other major disasters, it is very difficult to obtain valid, quick estimates of the dollar value of losses (see May, 1982). The early damage assessments tend to higher than later estimates, typically by a factor of three.

The fact that the data supporting loss estimates are so uncertain leaves room for several types of biases that can be introduced in the production and use of loss estimates. The technical biases related to use of expert judgement and incomplete data have been extensively discussed in the literature concerning "judgment under uncertainty" (for an overview see, Kahneman, Slovic, and Tversky, 1982). These include such things as an upward bias associated with more familiar or recent events, compounding of biases in aggregating estimates made of multiple components, and other problems that lead to a mix of under- and over-estimation biases that do not necessarily cancel out.

Of more interest to me are the patterns of abuse of unknown numbers as part of the policy process. Perhaps the most common pattern is that of ready acceptance of "mythical numbers." One example discussed by Max Singer (1971; also see Mosteller, 1977) is the ready acceptance of estimates of the value of property crime committed by heroin addicts. Singer shows that the value of such crime in New York City is probably less than ten percent of the figures popularized by law enforcement personnel and the then US attorney general. The problem was not inappropriate estimation methods -- the range of techniques look like those for estimating earthquake losses -- but a compounding of errors in applying different assumptions. This isn't just a case of pulling numbers out of a hat in order to satisfy media needs; the estimates were based on detailed assessments.

The point is not that the numbers were wildly wrong, but that such "mythical numbers" would be readily accepted as part of policy debates. In discussing why this is so, Peter Reuter (1984), a respected RAND Corporation authority on crime, notes three reasons. First, there is no constituency for keeping the numbers accurate, while there is a large constituency for keeping the numbers high. Second, he cites the relative lack of scholarly interest in developing techniques for better estimation of such numbers. Third, Reuter notes that because the estimates themselves have very little direct policy consequence, pressures to refine the estimates are limited.

An opposite set of circumstances resulting in willingness to accept downward estimates of the magnitude of a problem in some circles is illustrated by efforts, prior to the 1990 census, to estimate the number of homeless in this country (see Rossi, Wright, Fisher, and Willis, 1987). Here again the methodological problems are typical of those of "unknown numbers." Allegedly within the Reagan administration the pressures were to downplay the extent of the problem. In this instance, the numbers did have direct policy consequence in terms of food distribution programs run by states and there were countervailing pressures from outside groups to distrust official statistics. The result was a set of claims and counter-claims concerning the accuracy of the statistics.

My own sense is that the earthquake loss estimation situation is more like the value of heroin crime that the number of homeless. Earthquake loss estimates have a limited constituency for accurate numbers, improvements in estimates are hard to come by, and the policy consequences are limited. As such, my guess is that the guesstimates at any point in time, for a given size event, tend to be high. However, as I point out in the following sections of this paper, whether the estimates are high or low is not as important as the uses that are made of the estimates.

Reassessing Potential Uses of Loss Estimates

The preceding sets of comments lead me to argue that region-wide loss estimates of damages should be treated with caution. My point in what follows is that region-wide estimates can be made using general techniques and still be useful. However, efforts to provide more precise estimates, as are often called for, are unlikely to substantially increase the utility of loss estimates. In particular, I suggest the appropriate estimates are ones entailing the order or magnitude of potential losses -- millions, tens of millions, billions or tens of billions of potential dollar losses -- rather than misleadingly precise point estimates.

Such order of magnitude estimates can have value in several ways without needing much precision. Probably the most common use, and perhaps the only realistic use, of such region-wide estimates is simply calling attention to potential earthquake risks in providing numbers for policy debates and headlines for newspaper articles. As long as specific policy actions such as establishing funding formulae are not tied to the numbers, it is realistic to offer less refined, order of magnitude estimates.

The region-wide, order of magnitude estimates may also have some analytic utility. In justifying region-wide policy actions (e.g., land use restrictions for seismic sensitive areas, building code revisions for broad classes of buildings), potential loss reduction become the benefits of the proposed policy actions. Calculations of such benefits can become quite complicated, but in reality the policy decision will be based on fairly general estimates of appropriateness. This is illustrated by cost-benefit analyses of the city of Los Angeles ordinance concerning existing URM buildings (see Sarin, 1983). Similar ordinal rankings of potential losses within geographical areas provide all that can realistically be expected for guiding emergency planning.

The one use for which much more thought should be given to the application of region-wide losses is establishing insurance premiums for earthquake insurance. The insurance industry has perhaps the greatest stake in such assessments, while state insurance commissioners and other regulatory authorities have an obligation to review insurance industry use of such assessments. My understanding is that several insurance companies have developed extensive models for computing region-wide losses primarily for purposes of assessing potential exposure. Yet, because of the uncertainties concerning potential losses and the infrequency of past losses, conventional techniques in applying loss/exposure ratios or past history of losses cannot be used in developing earthquake insurance premiums.

All of this suggests that policy actions regarding earthquakes should not be tightly tied to earthquake loss estimates made either before or after an event. Despite the apparent rationality in doing so, it would be unwise to tie funding formula for earthquake planning funding or earthquake insurance premiums to region-wide estimates. Similarly, it is misleading to tie progress in reducing earthquake hazards to estimates made over time of potential losses. In each of these circumstances, the numbers are too uncertain and too fallible to justify such use.

Greater Potential for Vulnerability Assessments

My experience in talking with local officials about loss estimates is that few find any value in pre-event estimates of potential dollar losses. Yet, many envision potential use of assessments of the vulnerability of classes of buildings or facilities to potential earthquake damage. In terms of the classification scheme of loss estimation methods developed by Reitherman, this calls attention to vulnerability assessments as potentially useful and usable information. The obvious difficulties for local officials are having the technical capacity to carry out such studies, and marshalling the resources to finance such studies.

These types of studies differ from the region-wide loss estimates in several respects (see Reitherman, 1985). First, the vulnerability assessments do not attempt to put a dollar value on potential losses. Second, the vulnerability assessments are limited to certain classes of structures (e.g., schools, a utility system) or to single structures. Third, the emphasis is upon the cost of upgrading facilities to reduce vulnerability, and not the dollar losses associated with potential damages.

These types of assessments have been undertaken sporadically by governmental or quasi-governmental entities within this region for different classes of structures. Examples include:

- Completed or on-going assessments by several water utility districts and by METRO of key aspects of utility systems.
- Seattle schools assessment of school vulnerability undertaken in the late 1970s, contributing to the closing of some schools and the upgrading of others.
- Assessments of the vulnerability of state buildings on the Capitol campus, resulting in upgrading of several of the buildings.
- Steps following the Loma Prieta earthquake by the state Department of Transportation to improve its inventory of bridge conditions.

- A study of URM buildings in the city of Aberdeen.
- Hundreds of privately undertaken site-specific studies of vulnerabilities of older buildings prior to renovation Seattle, Tacoma, and other areas undergoing historic or other renovation.

To my knowledge, the range of vulnerability studies and their results have not been systematically cataloged for this region.

The value and potential of vulnerability assessments comes from the decision-orientation of such information. Knowing what types of facilities are vulnerable (or parts of the system) and the costs of lessening the vulnerabilities, officials can take action in establishing priorities for upgrading. The process used by the Seattle School District to include seismic vulnerability as an element in its decisionmaking about schools illustrates this potential.

One source of reticence of public officials to undertaking such vulnerability assessments is their fear that such undertakings will increase liability exposure because unaddressed risks will be better documented. In considering this issue, the Washington State Seismic Policy Council (1986, p. 13) noted:

> There are several, somewhat conflicting, legal reasons why this may be incorrect justification for not undertaking vulnerability studies... First, state and local governments (and by extension, state and local officials) may be found immune from liability claims under the discretionary function exception set forth by Washington State Supreme Court Decisions. Second, even if not immune, state and local officials are only negligent if they have not exercised prudent judgment in balancing risks and advantages when deciding not to retrofit or take other actions to reduce earthquake hazards. Third, if nothing is done now to either further document the hazard or implement hazard reduction efforts, state and local officials may be held liable for negligent actions.

Implications for Loss Estimation

The implications I draw from this discussion concern both the priorities for undertaking different types of loss estimation studies, and the expectations for the use of loss estimation studies. The conventional recommendations concerning loss estimates consist of pleas for more refined estimates, more comparability across geographic regions, and better ways of communicating loss estimates. Such recommendations strike me as being based on an unrealistic set of expectations concerning the potential uses of loss estimates.

Federal loss-estimation efforts have historically emphasized region-wide estimates of dollar losses. My conclusion is that priorities should be reversed, with much more attention given to local efforts to assess vulnerabilities of selected building types, classes of structures, and utility and lifeline systems. The emphasis should be upon the types of studies that inform decision-making and priority setting for upgrading facilities, rather than calculation of the dollar value of potential losses.

To the extent that region-wide estimates of potential losses are desired to call attention to earthquake risks, greater emphasis should be placed on order of magnitude estimates rather than fine-tuned estimates. It is not necessary to attempt to provide fine-tuned estimates for policy decisions that only require order of magnitude estimates.

Nor is it desirable to link policy actions to regional loss estimates. Funding formula for allocating earthquake planning monies and establishing geographic areas for research priorities should not be tied to highly uncertain loss estimates. Similarly, estimates of damages made immediately after a major earthquake are highly fallible means for establishing relief amounts.

Acknowledgments

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SPATIAL ANALYSIS OF THE PUBLIC'S ATTITUDES AND BEHAVIORS TOWARD THE EARTHQUAKE HAZARD IN TACOMA AND PUYALLUP, WASHINGTON

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Recent geological research suggests that the Puget Sound Region will experience a major subduction zone earthquake. The public's perception of this risk may have a profound impact on property damage, personal injury, and loss of life. In addition risk perception plays a key role in the degree of earthquake hazard planning and mitigation occurring at the state and local levels. The goal of this research is to investigate the public's earthquake hazard awareness and preparedness in the cities of Tacoma and Puyallup, Washington. A geographic perspective provides a means of analyzing spatial variations in attitudes and/or behaviors.

A random sample of residents in all 8 neighborhoods of Tacoma and 2 neighborhoods of Puyallup was drawn to represent the population. A total of 2,000 questionnaires were mailed to residents in these two cities. The percentage of housing units sampled for Tacoma and Puyallup were 2% and 5%, respectively. The number of questionnaires mailed to each neighborhood was proportionate to the number of housing units. The questionnaire addresses the public's perceived earthquake risk on a regional and local scale. Attitudes regarding personal control over the amount of damage their home or person may sustain are solicited. Respondents were also asked to reveal their opinions about financial responsibility for damage. An inventory of personal mitigation actions is provided by respondents as well as standard socio-economic characteristics.

The questionnaire, accompanied with a cover letter from the Pierce County Department of Emergency Management, was mailed October 13, 1989. The 34% return rate indicates a high public interest in the hazard. Over half (56%) of the returns were postmarked on or before October 17, 1989, the day of the California Loma Prieta Earthquake. This provides an unparalleled opportunity for evaluating the impact of a distant earthquake disaster on local attitudes and behaviors. The results of this study can be utilized to enhance the effectiveness of local hazard management by providing guidelines for public education and resource allocation efforts. Finally, this research establishes baseline data for future comparison.

LENDERS, INSURERS, AND EARTHQUAKE LOSS ESTIMATION

by

C. Taylor, C. Tillman, and W. Graf

ABSTRACT

Both mortgage lenders and insurers are to varying degrees at risk from earthquakes. Potential mortgage default losses and losses on earthquake policies written are some of the types of risks incurred. These risks are mitigated by a number of risk spreading buffers: conservatism on mortgage equity requirements, land appreciation, high deductibles, low limits of liability, reinsurance purchase, multi-line insurance and investments, control over direct risks assumed, tax deduction allowances, and disaster relief policies. To analyze and control these earthquake risks--especially evident in catastrophic earthquakes-- it is necessary both to adapt previous earthquake loss estimation models and to augment them with suitable financial models.

Introduction

Both mortgage lending institutions and property and casualty insurance companies have definite interests in potential earthquake losses. For lenders, earthquakes can cause mortgage defaults which in turn can lead to losses to lenders. For insurers, those clients that desire earthquake policies may suffer losses, as well as clients with policies in other lines such as auto and workers' compensation. Both insurers and lenders may be affected by other earthquake-related losses, including losses to investments in their real estate equity portfolios. Potential losses from damage to critical equipment and buildings can pose many problems both for lenders and insurers.

Both insurers and lenders also need to respond to various regulations pertaining to earthquake losses. For lenders that are federally guaranteed, there may be a need to respond to Executive Order 12699 of January 5, 1990. This states that each federal agency assisting in the financing or guaranteeing the financing of newly constructed buildings shall within three years develop a plan to assure that new buildings leased for federal use are constructed in accordance with appropriate seismic design and construction standards (the President, 1990). Federally backed lenders thus may be faced with regulatory concerns when financing buildings that may be leased to federal agencies. (See Brown and Gerhart, 1989, for a public policy analysis of lending institutions.) For insurers, state insurance regulation is of primary consideration. In the State of California, for instance, both insurers and reinsurers (companies that contract with insurers to limit their liabilities) are required to report their Probable Maximum Earthquake Losses (PMLs) (See Roth and Sam, 1989). After the recent passage of Proposition 103, there was initial concern that earthquake premiums would be required to be set similar to other forms of insurance, such as auto. As we shall see, earthquake hazards pose a very different problem for insurers.

The "risks" that lenders and insurers incur in earthquakes are chiefly financial, except with respect to their own employees. However, these risks can be translated into risks for consumers and others as well who bear the burden of insolvencies and poor or risky investments. Hence, regulators, lenders, insurers, stockholders, and consumers alike have interests in minimizing these risks, or more properly, in assuring that prudent tradeoffs are made to control these earthquake risks.

Earthquake loss estimation consists of four basic steps: exposure definition or inventory of assets at risk, seismic hazard identification and assessment, seismic vulnerability analysis, and seismic loss calculation (see Figure 1). Much of the attention at this workshop, for instance, is on seismic hazard identification and analysis: how much seismicity can be expected to affect facilities in the states of Washington and Oregon, how might the propagation of seismic waves and their amplification through soil columns to particular sites be modeled, and how much ground deformation may be anticipated in future earthquakes. Other portions of this workshop have focused on inventory of both buildings and lifeline facilities and assessment of the hazard to life and vulnerability of facilities to strong ground motions and anticipated permanent ground displacements.

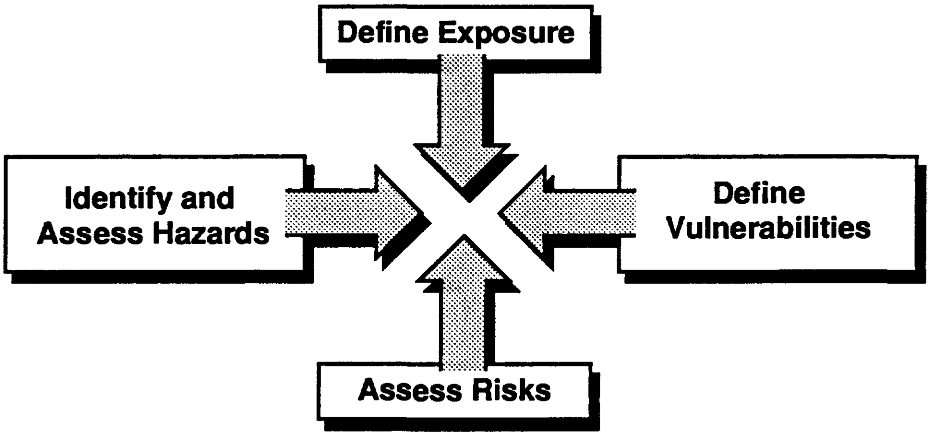

Figure 1. General Risk Analysis Approach

In addition to the greater knowledge under development in these key areas, we shall maintain that to understand and to control earthquake risks to lenders and insurers, financial models are needed. These quantify the degree to which lenders and insurers, respectively, bear the losses, as opposed to losses borne by others. As these financial models are used to augment conventional seismic loss estimation models, and because of the nature of earthquake loss estimation, seismic loss models become submerged in and part of asset management or resource allocation models. The multidisciplinary team required to develop conventional loss analyses becomes expanded to include actuaries, underwriters, insurance and banking executives and others knowledgeable in analyzing lending and insurance risks. Within these models, there remain for earthquakes extremely low level probabilities of risk that one may "sleep easy" about but that entail at least small risks incurred.

Suitable Quantitative Portrayal of Earthquake Losses

In order to begin to understand the risks incurred by lenders and insurers (or, for that matter, other key institutions, organizations, and firms), Figures 2 and 3 provide the types of quantitative outputs that can serve to explain risk positions. Figure 2 portrays the risk to a hypothetical California residential portfolio. Figure 3 portrays the earthquake direct risk to selected Seattle water system facilities.

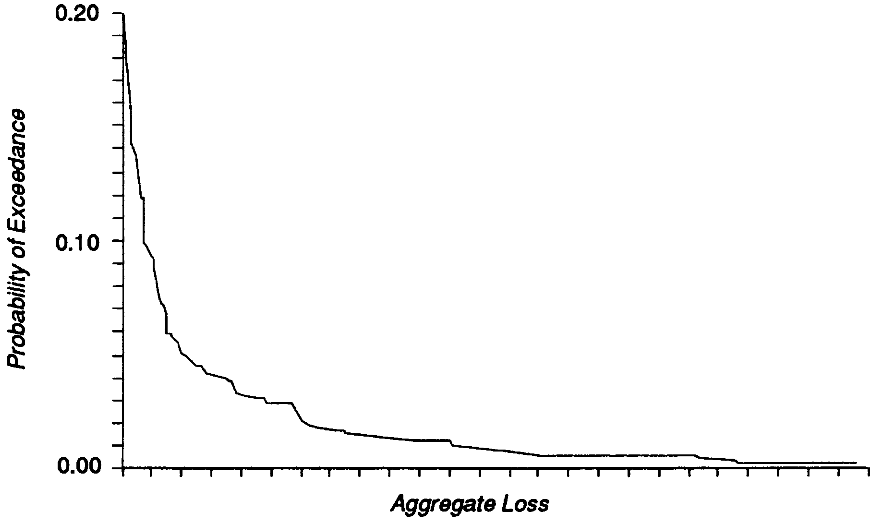

Figure 2. Annual Probability of Aggregate Earthquake Losses to a Hypothetical Residential portfolio in California

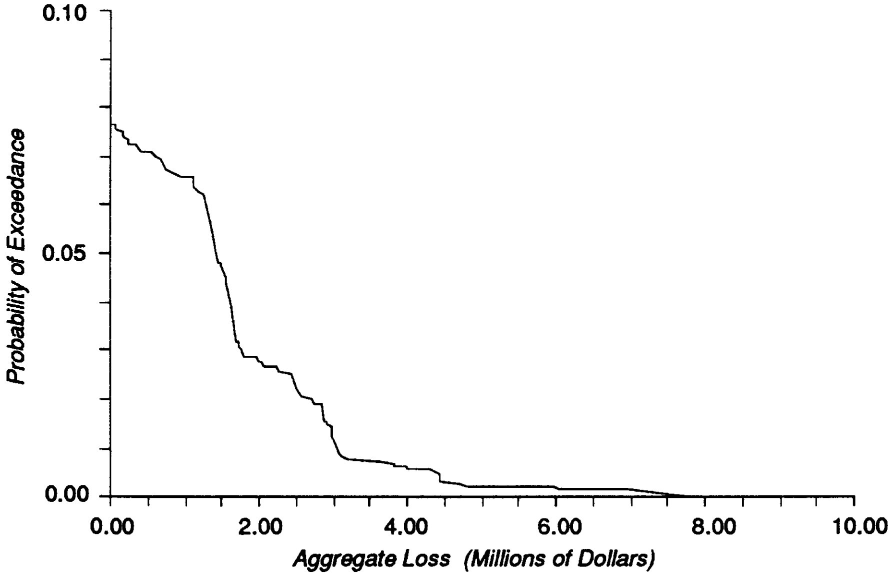

Figure 3. Aggregate Direct Loss by Probability of Exceedance for Selected Seattle Water Facilities

Developing such quantitative representations of earthquake risks requires, as in conventional seismic risk analyses, intensive development of earthquake source zone models (for known faults, for speculated faults, and for random earthquake sources), seismic attenuation models, local relative site response factors, models of potential ground deformation, and models of the expected response of facilities to strong ground motion and/or permanent ground deformation. (Some of the modeling advances are consolidated in such computer programs as SEISRISK-III, Bender and Perkins, 1987.) The chief difference between this and conventional representations of earthquake losses lies simultaneously in (a) the application to multiple sites and (b) the estimations of probabilities of exceedance.

In both cases, the Y-axis represents the probability of exceedance. The X-axis represents aggregate loss level, or losses expected all at once to the facilities surveyed. This quantitative picture reaffirms what those in earthquake studies knew all along. Most of the losses expected from earthquakes are small. But very large-scale losses can occur from a variety of sources including a very large magnitude (8.5) Cascadian subduction zone event, large magnitude (6.5-7.5) nearer field random events, and in California, earthquakes generated from the Santa Monica-Malibu, San Andreas, San Jacinto, Hayward, and Newport-Inglewood fault systems.

As Butler, Doherty, and Kunreuther (1988) have maintained, one frequently used criterion for the "insurability" of a risk is independence of risks, viz., that risks have a low correlation with each other. Risks with these low cross-correlations may be treated as more or less independent exposure units. Your death or my death may be weakly correlated or uncorrelated unless, say, an epidemic occurs. One auto accident may be uncorrelated or weakly correlated with many others. One fire may be weakly correlated with other fires.

With earthquakes we find that exposure units are both typically

- weakly correlated or uncorrelated in smaller magnitude or more distant earthquakes
- strongly correlated (within regions) for larger magnitude or more proximate earthquakes.

Hence, the magnitude 5.5 earthquake on February 28, 1990 near Upland, California, did not create very large-scale losses. Yet this sort of earthquake has a much higher probability of occurrence than many of the other past and possible high-consequence earthquakes that are discussed. If all earthquakes were of the Upland variety, they would satisfy the criterion of independence.

The other feature of Figures 2 and 3 is that they contain estimates of annual probabilities of exceedance. In initially modeling losses to selected Seattle water system facilities, we developed earthquake scenarios that have the following direct loss levels (see Ballantyne, et al., 1990):

$1.1 million	(6.5 magnitude)
$3.5 million	(7.5 magnitude)
$7.6 million	(8.5 magnitude)

In analyzing what these loss levels mean, one may attempt to attribute probabilities to each of the events developed. However, with rare exceptions it becomes extremely difficult if not impossible to state a (non-zero) probability for each of the earthquake scenarios selected. How does one model, say, the probability of one very precisely defined rupture occurrence when rupture is a spatially complex random process? Ordering aggregate losses, as in Figures 2 and 3, provides loss estimates with significance relative to the question "What does this loss level show about risks assumed?"

Figures 2 and 3 bear on recent discussions as to how PML is to be defined (and some definitions in effect state that PML is the loss estimate derived from a specific set of accounting procedures). However, Figures 2 and 3 show why such definitions are no longer needed. PML is some aggregate loss-value presumably somewhere on the tail of an aggregate loss-value curve. One can select the degree of conservatism desired and select from that the PML. The notion of PML, as argued elsewhere (Taylor, Hayne, and Tillman, 1990; Eguchi, et. al., 1989, Russ, et. al., 1989) is both systematically ambiguous and now superfluous in loss-estimation procedures. As maintained in the previous paragraphs, further confusion may be engendered if one assigns probability estimates to PML events.

The Other Side of the Model: Assets to Cover Losses Incurred

Neither lenders nor insurers assume all losses incurred to mortgages or the insured. Most mortgagees, for instance, will assume their own losses, or else will pass them on to insurers. The insured, likewise, assume losses up to deductible levels and losses exceeding limits of liability.

For lenders, the following buffers (risk diversification factors) exists to reduce expected mortgage default losses:

- appreciation over the years in the value of the property (including land) relative to the structure and its contents
- decrease over the years in the mortgage-balance
- the undesirability or costs of bad credit ratings for those who default
- catastrophic insurance purchased on potential mortgage defaults
- favorable business or homeowner locations that lead to decisions not to default even if losses exceed owner's equity

- state and federal disaster relief to small businesses and homeowners

- insurance purchases by mortgagees

- possible increase in market value of surviving properties

- geographic spread of risks

As has been pointed out, at an appreciation rate of 6 1/2 percent per year, after fifteen years on a mortgage there is no or little chance that a building loss could exceed owner's equity. (See Anderson and Weinrohe, 1984) California appreciation rates have been considerably higher, thus providing a large buffer from prospective default losses to lenders.

Other factors may serve to make losses from mortgage defaults more serious:

- the presence of low mortgage equity amounts, or of mortgagees that have used their equity positions to develop additional loans, mortgages, etc.

- low rates of earthquake insurance purchase

- administrative, legal and other costs of selling foreclosed properties (along with individuals and firms leaving earthquake damaged regions)

- possible depreciation in land values after earthquakes

- losses of rents or other business income after earthquakes that jeopardize better risks

- nonuniform seismic construction standards across the country and over time that lead to potentially correlated risks.

- geographic concentration of risks

Overall, the current buffers for lenders may greatly reduce their exposure to direct earthquake losses. However, it is apparent that financial models are needed (and available) to estimate more precisely their residual risks. Even with buffers, given the large number and value of financial interests that lenders have in seismic regions, it is clear that these residual risks exists. These risks may not be so severe as direct earthquake risks to banking operations and employees, but they are nonetheless worth examining.

Insurers also have buffers from prospective earthquake losses to the insured: These buffers include:

- high deductible levels

- low limits of liability

- reinsurance arrangements

- premiums that tend (with major exceptions) to be conservative
- small proportions (modest volumes) of earthquake insurance purchase
- policy language and legal support that denies payment for earthquake damage unless earthquake is specifically covered
- multi-line insurance sales which provide other sources of income that can be used to offset potential earthquake losses
- limited geographic spread of risks
- tax deductions from insurer losses

Extreme fluctuations in earthquake premiums have occurred (see Figure 4; Cheney and Whiteman, 1987). This suggests that market factors--perhaps especially the availability of reinsurance--bear on existing market premiums more than any results of loss-estimation.

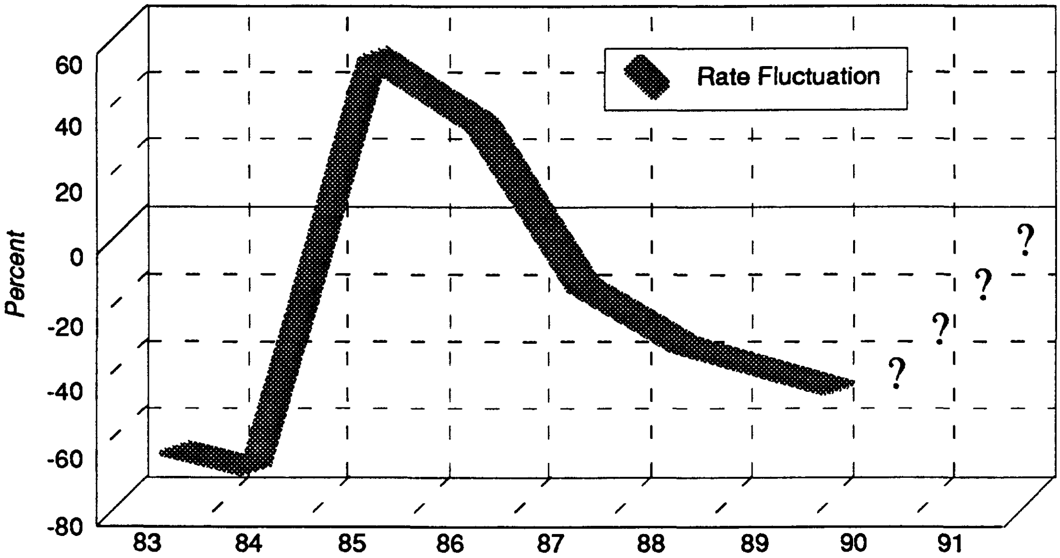

Figure 4. Approximate Fluctuations in Property Industry Rate, Earthquake Included (Ken Goodchild, 1990)

Hence, while buffers exist for insurers, these are less than for lenders with respect to mortgaged properties. But, for both lenders and insurers financial models can be constructed to evaluate residual risks.

Because the low-probability tails of earthquake aggregate loss distributions involve very large losses, earthquakes have the potential to cause serious concerns even to large insurers and lenders. For insurers, potential problems of insolvency can arise. For lenders, assets can be seriously eroded. Hence, earthquakes have the potential to cause problems that ultimately require executive-level decisions.

In order to analyze these sorts of problems, not only are financial models needed to determine how likely losses are to insurers and lenders alike, but consideration of assets ultimately enters. Considering the extreme tails of the distributions implied in Figures 2 and 3, the chief decisions to be made are how serious these losses are relative to company assets:

- Could earthquake losses cause "ruin"?

- Could they seriously erode the company's assets and market position?

- What are the strategies for reducing these extreme discomforts or potential company-wide risks and are these remedies (such as the purchase of catastrophic insurance or reinsurance) worth the price?

Quantitative techniques in earthquake loss-estimation are now available for addressing some of these major business or financial concerns. They were available in 1971 and before with respect to flood perils, and were conceptually available for earthquake perils also at that early date (see Kaplan, 1971-1972). However, these quantitative techniques require consideration of company-wide assets, the domain of actuaries, chief executive officers and others in finance, and the serious threats that large-scale earthquakes may pose to insurers and lenders.

Summary

Viewed probabilistically, earthquake losses to lenders and insurers can now be quantified through (a) new aggregate loss-estimation models as indicated in Figures 2 and 3 and (b) financial (allocative models that indicate how much of the gross losses are retained by insurers and lenders. Except at very high costs (as for reinsurance purchase), some earthquake risks are likely to be retained by both insurers and lenders, in spite of numerous buffers (risk diversification factors and strategies). Tradeoffs between risk retained and costs of reducing these risks are of interest to executives, regulators, and consumers alike.

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TECHNIQUES FOR REDUCING EARTHQUAKE HAZARDS--AN INTRODUCTION

By William J. Kockelman

ABSTRACT

Many techniques are available for reducing earthquake hazards; 36 are identified in this paper. Six are described with examples--redevelopment plans, regulatory zones, nonstructural building components, public information, unreinforced masonry buildings, and loss estimates. An overview of these techniques is useful to planners who implement hazard-reduction programs, to engineers who serve as advisors to local or state governments, and to decisionmakers who select the most appropriate technique for a given situation. Prerequisites for the successful use of these techniques are adequate and reliable scientific and engineering information, translation of such information for use by nontechnical users, and effective transfer of the translated information to those who will, or are required to, use it.

INTRODUCTION

Numerous techniques for reducing earthquake hazards are available to planners, engineers, and decisionmakers. Some of these techniques, such as public acquisition of hazardous areas, are well known to the planning profession. Others, such as design of resistant structures, are commonly used by engineers. Still others, such as warning systems and emergency preparedness, are obvious and practical, but require maintenance and persistence in their implementation.

To give the reader an overview, examples of various techniques are shown in list 1. These techniques are divided into six groups but can be grouped in other ways, for example, chronologically:

- Pre-event mitigation techniques, which may take 1 to 20 yr
- Preparedness measures, which may take 1 to 20 wk
- Response during and immediately after an event
- Recovery operations after an event, which may take 1 to 20 wk
- Post-event reconstruction activities, which may take 1 to 20 yr

These estimated time periods vary depending upon the postulated or actual size of the earthquake, its damage, and the resources available to a state, its communities, its corporations, and its citizens.

The techniques (list 1) have the following specific objectives: awareness of, avoidance of, resistance to, or response to, the effect of the earthquake phenomena on people and their land uses, structures, and activities. The general goal of these objectives is to reduce human casualties, property damage, and socioeconomic interruptions. Many of the reduction techniques are complex, interconnected, and require special

List 1

EXAMPLES OF VARIOUS TECHNIQUES FOR REDUCING EARTHQUAKE HAZARDS

Incorporating hazard information into studies and plans
  Community-facilities inventories and plans
  Economic-development analysis and plans
  Emergency and public-safety plans
  Land-use and transportation inventories and plans
  *Redevelopment plans (pre-event and post-disaster)
  Utility inventories and plans

Regulating development
  *Creating special hazard-reduction zones and regulations
  Enacting building and grading ordinances
  Enacting subdivision ordinances
  Requiring engineering, geologic, and seismologic reports
  Requiring investigations in hazardous areas
  Reviewing annexation, project, and rezoning applications

Siting, designing, and constructing safe structures
  Evaluating specific sites for hazards
  Reconstructing after a disaster
  *Securing nonstructural building components and contents
  Selecting the most resistant building system and configuration
  Siting and designing critical facilities
  Training design professionals and building inspectors

Discouraging new development in hazardous areas
  Adopting utility and public-facility service-area policies
  Clarifying the liability of developers and government officials
  Creating financial incentives and disincentives
  *Informing and educating the public
  Posting public signs that warn of potential hazards
  Requiring nonsubsidized insurance related to level of hazard

Strengthening, converting, or removing unsafe structures
  Condemning and demolishing unsafe structures
  Reducing land-use intensities or building occupancies
  Relocating community facilities and utilities
  Repairing unsafe dams or lowering their impoundments
  Retrofitting bridges and overpasses
  *Strengthening unreinforced masonry buildings

Preparing for and responding to emergencies and disasters
  Conducting emergency or disaster training exercises
  *Estimating casualties, damage and interruptions
  Initiating community and corporate education programs
  Operating monitoring, warning, and evacuation systems
  Preparing emergency response and recovery plans
  Providing for damage inspection, repair, and recovery

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* Technique described and illustrated in this paper.

skills--legal, financial, legislative, design, economic, communicative, educational, political, and engineering.

Many of the hazard reduction techniques have been discussed and illustrated by Blair and Spangle (1979), Kockelman and Brabb (1979), Brown and Kockelman (1983), Kockelman (1985, 1986), Jochim and others (1988), Mader and Blair-Tyler (1988), Blair-Tyler and Gregory (1988), and the United Nations Office of the Disaster Relief Coordinator (Lohman and others, 1988).

Prerequisite to the use of these reduction techniques are scientific and engineering studies. Such studies are vital, because in the words of a former U.S. Geological Survey (USGS) director, Walter C. Mendenhall: "There can be no applied science unless there is science to apply." It has been my experience that it is not prudent for urban planners to develop land-use regulations, civil engineers to design structures, and lenders and public works directors to adopt policies reducing earthquake hazards without reliable scientific and engineering assessments.

Six earthquake-hazard reduction techniques were selected for this paper:

- Preparing redevelopment plans
- Creating regulatory zones
- Securing nonstructural building components
- Informing the public
- Strengthening unreinforced masonry buildings
- Estimating casualties, damage, and interruptions

These six techniques are briefly discussed and generally illustrated for nontechnical readers. The references for each technique discussed will provide scholars and practitioners with more details and examples.

PREPARING REDEVELOPMENT PLANS

Incorporating earthquake-hazard information into plans for the development or redevelopment of a community's land use, housing, transportation, and other public facilities is a common natural-hazard reduction technique. One of these plans is the redevelopment plan. State laws authorizing the creation of public redevelopment agencies usually provide for: the preparation and adoption of redevelopment plans; the acquisition, clearance, disposal, reconstruction, and rehabilitation of blighted (including damaged) areas; and the relocation of those persons displaced by the project. Redevelopment agencies usually are empowered to issue bonds, receive part of the taxes levied on property in the project, and use grants or loans available under various state and federal programs. Such plans may be divided into three categories; namely, those which incorporate:

- Damaged areas into a redevelopment plan created prior to a damaging earthquake.

- Vulnerable structures (identified prior to an earthquake) into reduction and redevelopment plans.

o Damaged areas into a redevelopment plan created after an earthquake.

Santa Rosa illustrates the first category. It is a city of about 50,000 people which was hit within two hours by two earthquakes in 1969. Almost all the resulting property damage was caused by intense ground shaking. Many buildings, including numerous old unreinforced masonry buildings were damaged. Mader and others (1980, p. C1 to 15) report that:

> In 1961, Santa Rosa embarked on a redevelopment project covering part of the downtown area. Just prior to the earthquake, the city had adopted a central business district plan which covered an area adjacent to the redevelopment area. After the earthquake, this area, with a high percentage of damaged buildings, was added to the original redevelopment area. With a federal contribution of about $5 million, properties were acquired and cleared for development of a major regional shopping center integrated with the rest of downtown. Construction of the shopping center began in late 1978 ....

The time and effort to get the redevelopment project funded and underway was significantly less because of the existence of an adopted up-to-date plan (fig. 1).

Spangle and others (1987, app. A) describe the second category, a new technique called "pre-earthquake planning for post-earthquake rebuilding." They present four preevent activities: evaluate vulnerability to damage; organize for preparedness and response; mitigate hazards; and plan for post-earthquake response. They comment that it is possible to develop damage estimates sufficiently accurate for pre-earthquake programming for post-earthquake recovery activities and to define the nature of the post-earthquake recovery organization needed.

The City of Whittier Redevelopment Agency (1987) adopted a plan that represents the third category. The plan provides for redevelopment powers to be used for projects to maintain, repair, restore, demolish, or replace property or facilities damaged or destroyed as a result of an earthquake. The earthquake damage in their city exceeded 70 million dollars. The project is within the disaster area determined by the agency to be in need of redevelopment as a result of the earthquake damage.

Preparing and implementing redevelopment plans that recognize and reduce earthquake hazards is unusually important because reconstruction commonly takes place in the same hazardous areas after an earthquake. Youd and others (1978, p. 111), for example, observed that, after the San Fernando earthquake, "... buildings had been repaired, new buildings have been built, and a freeway interchange has been constructed across the trace of the 1971 fault rupture."

CREATING REGULATORY ZONES

Various types of land-use and land-development regulations are available to state and local governments. Controlling use and development

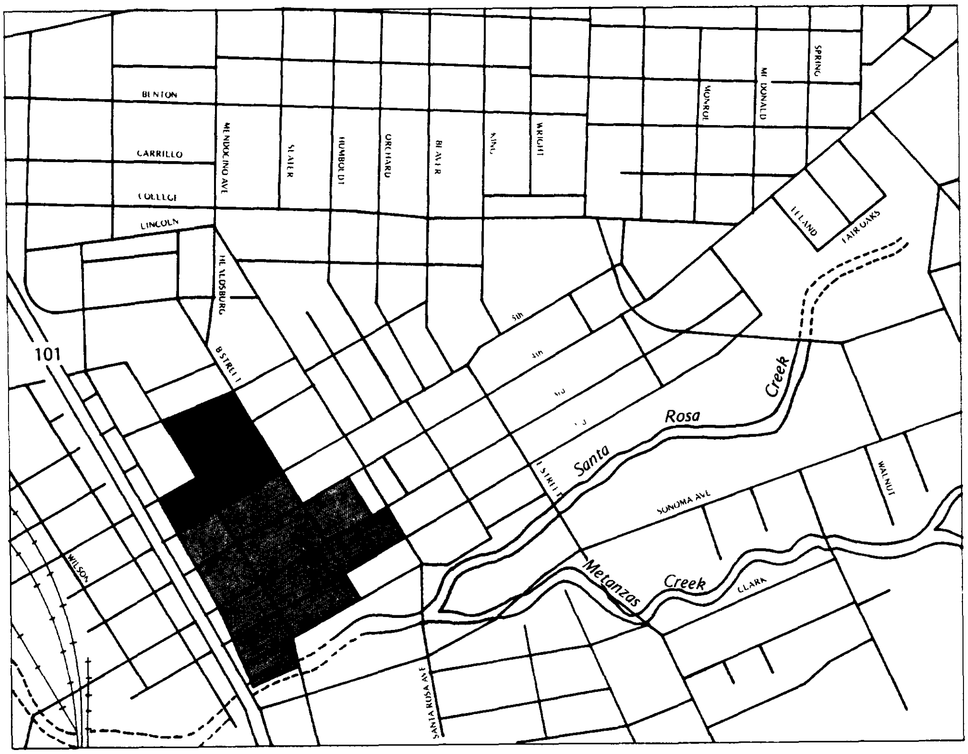


FIGURE 1.--Part of a city urban renewal project area from Mader and others (1980, fig. 7, p. C-8). Phase I--original project area is bounded by Sonoma and Santa Rosa avenues and 4th and E streets. Medium screen indicates Phase II--area added following 1969 earthquakes; and dark screen, Phase III--survey area of additional land required for regional shopping center.

by zones can be one of the most economical and effective means available to government regulatory agencies. The regulations can be used to reduce earthquake hazards--surface-fault rupture, ground shaking, liquefaction, landslides, and tsunamis. Such regulations may be divided into four categories:

- Requiring site investigations and building setbacks
- Reducing the density of development or the number of occupants
- Permitting only less vulnerable land uses and land developments
- Designing and constructing structures to withstand anticipated forces

The first category can be illustrated by the Alquist-Priolo Special Studies Zones Act enacted by the California Legislature (1972). The Act provides for public safety by restricting development near or over the surface traces of active faults (fig. 2). In addition, the act provides for geologic reports, approval of projects by cities and counties, and the charging of reasonable fees for administrative costs. The State Geologist delineates the zones which include all "potentially active" traces of faults that he deems sufficiently active and well defined" to constitute a potential hazard from surface faulting or fault creep (Hart, 1988, app. A).

Cities and counties must require, before approval of a project in the zone, "a geologic report defining and delineating any hazard of surface fault rupture." The legislature defines "project" to include structures for human occupancy and any subdivision which contemplates the eventual construction of structures for human occupancy but exempts single-family wood frame buildings (including mobile homes) not exceeding two stories when not part of a development of four or more dwellings. The approval of a project must be in accord with the policies and criteria established by the California Mining and Geology Board. The board (Hart, 1988, app. B) prohibits a project across the trace of an active fault; requires a geologic report if a project lies within 15 m (50 ft) of an active fault; and requires a registered geologist retained by the city or county to evaluate such reports. The act allows cities and counties to establish more restrictive policies and criteria. Some cities and counties, like the Portola Valley Town Council (1973), require greater setbacks in certain instances.

The San Mateo County Board of Supervisors (1973) is using the second category. It is a resource-management zoning district that also carries out the objectives and policies of their open-space and resource-conservation plans. The district regulations limit the number of dwellings in zones with a surface-fault rupture hazard, flood hazard, or unstable slopes to one unit per 16 hectares (40 acres) and require geologic site investigations to ensure that the reduced development is located in safe areas. The lower net number of dwellings permitted may then be clustered at a higher density in the nonhazardous areas (fig. 3).

An example of the third category may be seen in Colorado, where geologic hazards have been declared by the state legislature to be matters of state interest. To assist communities in designing land-use regulations, the Colorado Geological Survey prepared model geologic hazard area control regulations for adoption by local governments. The model regulations permit

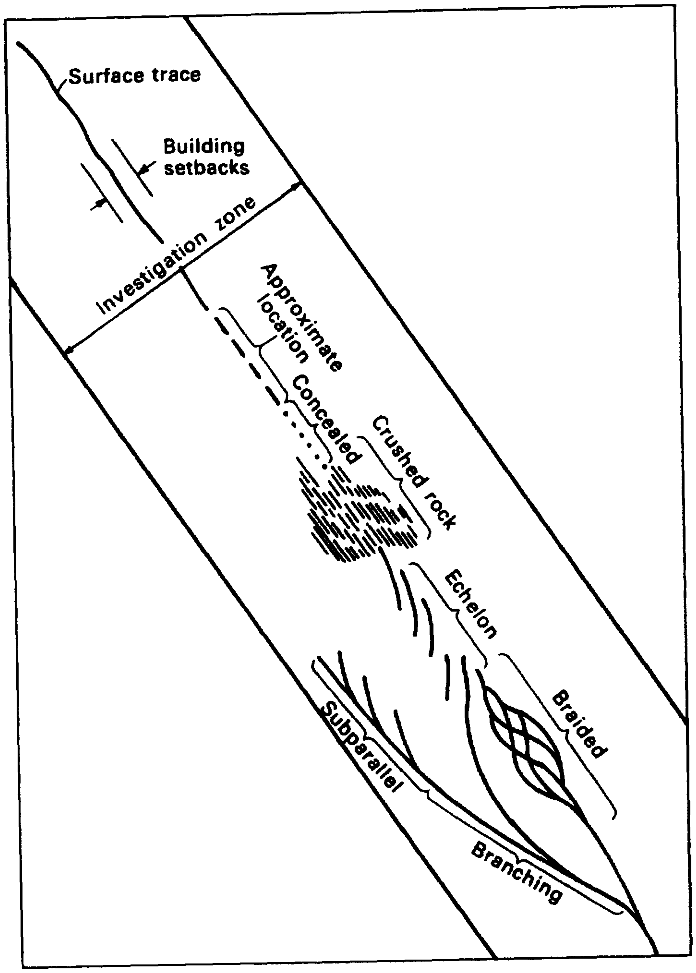


FIGURE 2.--Hypothetical surface-fault rupture regulatory zone from Brown and Kockelman (1983, fig. 30, p. 8) illustrating the complexities of faulting, the necessity for an investigative zone, and the location of building setbacks.

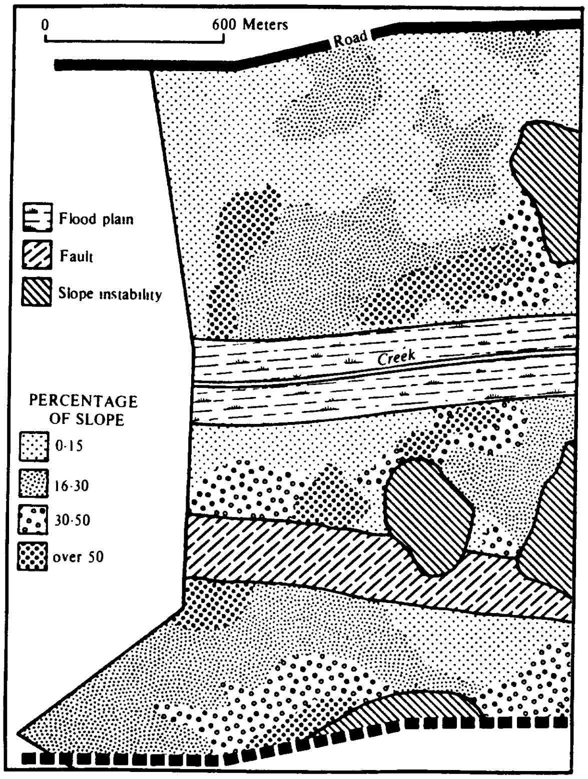


FIGURE 3.--Hypothetical property from Kockelman and Brabb (1979, figure 6, p. 82) showing seismic and other geologic constraints. Dwelling units in the flood, surface-fault-rupture, and slope-instability zones are limited to one per 16 hectares (40 acres) by the San Mateo County Board of Supervisors (1973).

only the following "open" uses in designated geologically hazardous areas: (1) Agricultural uses such as general farming, grazing, truck farming, forestry, sod farming, and wild-crop harvesting; (2) Industrial-commercial uses such as loading areas, parking areas not requiring extensive grading or impervious paving, and storage yards for equipment or machinery easily moved or not subject to geologic-hazard damage; and (3) Public and private recreational uses not requiring permanent structures designed for human habitation such as parks, natural swimming areas, golf courses, driving ranges, picnic grounds, wildlife and nature preserves, game farms, shooting preserves, target ranges, trap and skeet ranges, and hunting, fishing, skiing, and hiking areas, if such uses do not cause concentrations of people.

The fourth category is well illustrated by the Redwood City Council (1974, 1977) ordinance that provides for special seismic requirements relating to design and construction standards. These standards supplement those recommended by the International Conference of Building Officials for structures in seismic zone 4 under the Uniform Building Code--the code adopted by the city as its own building code.

This ordinance is consistent with the city's initial Seismic Safety Element (Redwood City Planning Department, 1974), which had placed the bay mud in a moderately high risk zone and recommended that the Uniform Building Code be reviewed and amended as "frequently as may be prudent." The supplemental structural-design and construction standards called for in the ordinance relate to special foundation-design criteria, design provisions for greater lateral force, foundation systems to resist settlement, wood-frame sheathing, moment-resisting frames, response spectrum, reinforced-masonry construction, elements of structural redundancy, and reinforcement of structural members. These standards apply only to those lands within the city that are underlain by bay mud, as shown on a map adopted by reference in the ordinance (fig. 4).

SECURING NONSTRUCTURAL BUILDING COMPONENTS

Proper siting, design, and construction of structures are well-known techniques to reduce earthquake casualties and damage but often the contents and other nonstructural components of buildings are overlooked. People have been injured by falling light fixtures, flying glass, overturning shelves, and spilled chemicals. The Federal Emergency Management Agency (1981, table 2) estimates that one-third of the property lost in future earthquakes will be attributed to building contents. Such contents are only one part of the nonstructural components of buildings.

Nonstructural damage is caused by object inertia or building distortion. For example, if an office computer or file cabinet is shaken, only friction will restrain it from overturning or falling on its user. As the structure bends or distorts, its windows, partitions, and other items set in the structure are stressed, causing them to shatter, crack, or spring out of place. Numerous protective measures are available, including:

- Bolting down sharp or heavy office equipment and fixtures
- Tying artwork to the walls

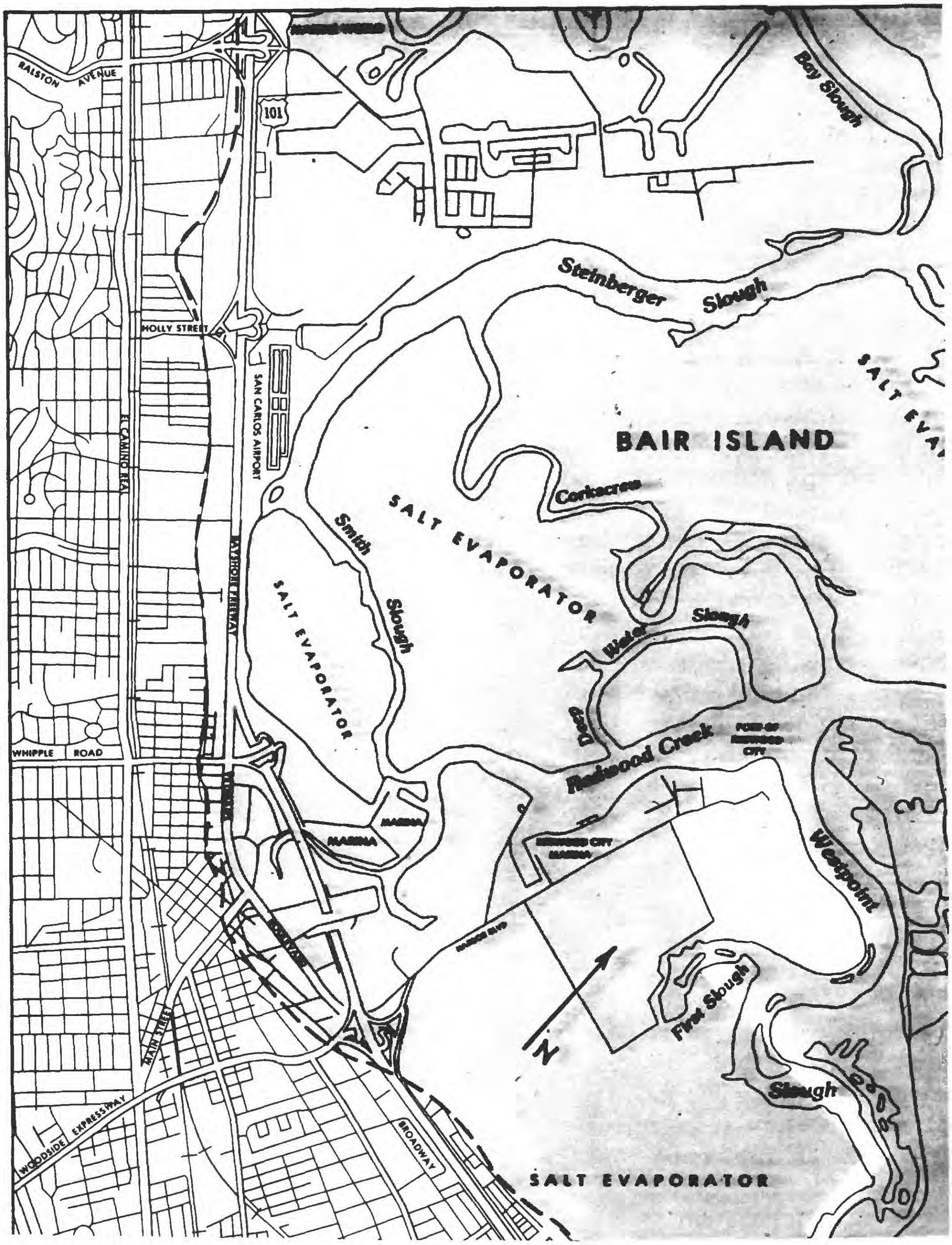


FIGURE 4.--Part of a map showing an area of a city underlain by bay mud. The map is attached to the building code (Redwood City Council, 1977), which requires supplemental structural-design and construction standards for all new development. Bay mud is indicated by shading, and its southwesterly boundary by a dashed line. The unshaded area along the Bayshore Freeway (U.S. Highway 101) lies outside the city's jurisdiction.

- Connecting filing cabinets together at their tops and to a wall
- Zigzagging free-standing, movable partitions
- Installing locks on cupboards
- Boxing large containers that contain hazardous chemicals
- Strapping hot-water heaters to wall studs

An excellent guidebook on reducing the risk of nonstructural earthquake damage was prepared by Reitherman (1983). He describes typical conditions found in office, retail, and government buildings. Measures are suggested for restraining over 20 nonstructural building components, such as office machines, electrical equipment, file cabinets, built-in partitions, suspended ceilings, exterior ornamentation, elevators, piping, stairways, and parapets. Each component is rated for existing and upgraded vulnerability for life-safety hazards, percent of replacement-value damaged, and post-earthquake outages for three levels of shaking intensity (fig. 5).

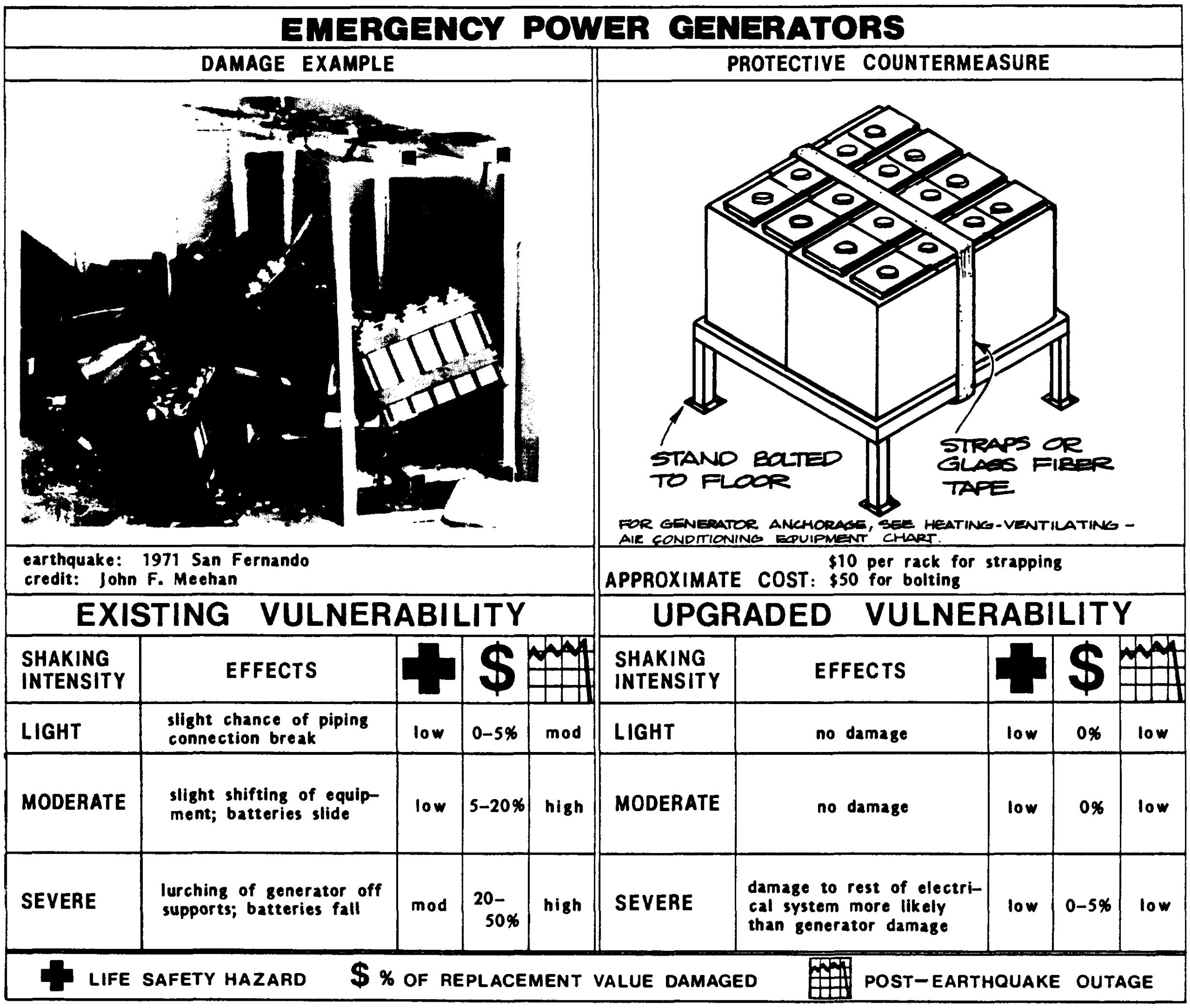
A second guidebook focuses on procedures for reducing nonstructural hazards in schools. This guidebook was issued by the Washington State Superintendent of Public Instruction (Noson, 1989) and contains precise clear drawings of methods for securing hazardous objects commonly found in schools. The objects include ceiling panels, chemicals, doors, exterior chimneys, exterior masonry, parapets, furniture, file cabinets, windows, mirrors, skylights, heaters, light fixtures, partitions, and water heaters. A general estimate of the risk of each object and the cost to secure each are provided. In addition, checklists for school administrators and custodians are included for both interior--ceilings, floors, walls, boiler rooms, cafeterias, halls, stairways, laboratories--and exterior hazards--chimneys, ornamentations, and parapets.

The application of such a guidebook may be seen in the City of Mountain View. Blair-Tyler and Gregory (1988, p. 19) observed that the city had consultants prepare a room-by-room inventory of nonstructural hazards in the Emergency Operations Center--an alternate City Hall which must function after an earthquake. They report that:

> Communications equipment was braced and interior glass is being replaced with safety glass or covered with a safety film. The City's maintenance staff is providing the estimated 320 man-hours to complete the nonstructural work during the next year. Any structural strengthening will be done by an outside contractor. Information gained from this experience will be used to reduce nonstructural hazards in the design of Mountain View's new Library and City Hall.

INFORMING THE PUBLIC

Public information programs are essential for bringing earthquake-hazard information to the attention of the public. Responsible developers and prudent citizens, when told of earthquake hazards, may not wish to risk property losses or expose their clients or families to the danger and trauma. All hazard-reduction programs depend on the understanding and support of an informed public. Preparing, announcing, and disseminating information on earthquake damage, risk, and hazard-reduction techniques can



EMERGENCY POWER GENERATORS

DAMAGE EXAMPLE	PROTECTIVE COUNTERMEASURE
earthquake: 1971 San Fernando credit: John F. Meehan	APPROXIMATE COST: $10 per rack for strapping $50 for bolting

FOR GENERATOR ANCHORAGE, SEE HEATING-VENTILATING-AIR CONDITIONING EQUIPMENT CHART.

EXISTING VULNERABILITY

SHAKING INTENSITY	EFFECTS	✚	$	▦
LIGHT	slight chance of piping connection break	low	0-5%	mod
MODERATE	slight shifting of equipment; batteries slide	low	5-20%	high
SEVERE	lurching of generator off supports; batteries fall	mod	20-50%	high

UPGRADED VULNERABILITY

SHAKING INTENSITY	EFFECTS	✚	$	▦
LIGHT	no damage	low	0%	low
MODERATE	no damage	low	0%	low
SEVERE	damage to rest of electrical system more likely than generator damage	low	0-5%	low

✚ LIFE SAFETY HAZARD $ % OF REPLACEMENT VALUE DAMAGED ▦ POST-EARTHQUAKE OUTAGE

FIGURE 5.--Excerpt from Reitherman (1983, p. 39) showing how to reduce risk from earthquake damage for one type of nonstructural building component.

be accomplished through numerous methods. Examples of the following are cited in lists 2 and 3:

- General, introductory, and index materials
- Serial publications
- Guidebooks and guidelines
- Conferences and workshops
- Outreach programs
- Examples and discussions of reduction techniques

STRENGTHENING UNREINFORCED MASONRY BUILDINGS

Numerous techniques for strengthening, converting, or removing unsafe structures are available to state and local governments. One of these--strengthening unreinforced masonry buildings--has been used by several communities. Its first phase--identification of unsafe buildings by cities and counties--has begun for an entire state.

These unsafe structures include unreinforced masonry bearing-wall buildings and steel- and concrete-frame buildings with infill walls that are of unreinforced masonry. According to a state seismic safety commission, these structures typically have four areas of weakness:

- Masonry walls, lacking reinforcing, do not have resistance to earthquake shaking without degrading, sometimes leading to collapse.

- The practice of not structurally tying the walls to the roof and floors can allow excessive movements in the walls, which may lead to collapse.

- Ground floors with open fronts and little crosswise bracing may allow excessive movement and twisting motions, damaging the building.

- Unbraced parapets may fall into the street.

An ordinance adopted by the Los Angeles City Council (1981) provides procedures and standards for identifying and classifying buildings having unreinforced masonry bearing walls; these procedures and standards are based on a building's present use and occupancy (fig. 6). Priorities, time periods, and standards are also established under which buildings are required to be structurally analyzed and anchored. Where analysis determines deficiencies, the ordinance requires that a building be strengthened or demolished. The ordinance applies to all buildings having bearing walls of unreinforced masonry that were constructed or under construction before 1933, or for which a building permit was issued prior to 1933, the effective date of the city's first seismic building code. The ordinance does not apply to detached one- or two-story single-family dwellings and detached apartment houses containing less than five dwelling units and used solely for residential purposes.

Affected buildings are classified according to type of function and occupancy as essential, high-risk, medium-risk, and low-risk buildings. The strengthening standards and time schedules for notification and compliance vary with the risk category. A structural analysis of each individual

List 2

EXAMPLES OF TRANSFER TECHNIQUES FOR INFORMING THE PUBLIC

General, Introductory, and Index Materials

Washington State Earthquake Hazards by Noson, Qamar, and Thorsen (1988).
Facing Geologic and Hydrologic Hazards by Hays (1981).
Home Guide Section on How a House Withstands an Earthquake by Kerch (1988).
Getting Ready for a Big Quake by Sunset Magazine (1982).
Bibliography and Index to Seismic Hazards of Western Washington compiled by Manson (1988).

Serial Publications

Oregon Geology by Oregon State Department of Geology and Mineral Industries (bimonthly).
Earthquake Hazard Reduction Series by the Federal Emergency Management Agency (see list 3).
Earthquakes and Volcanoes (formerly Earthquake Information Bulletin) by Spall (1971 to present).
Washington Geologic Newsletter by Washington State Division of Geology and Earth Resources (quarterly).
Wasatch Front Forum by Hassibe (1984-1986) and Jarva (1987-present).

Guidebooks and Guidelines

Geologic Principles for Prudent Land Use by Brown and Kockelman (1983).
Earthquake Advisor's Handbook for Wood-frame Houses by the University of California Center for Planning and Development Research (1982).
Reducing Earthquake Risks for Planners by Jaffe and others (1981).
Preparing a Safety Element of the City and County General Plan by Mintier (1987, p. 146-153).
Steps to Earthquake Safety for Local Governments by Mader and Blair-Tyler (1988).
Landslide Loss Reduction Guide for State and Local Government Planning by Wold and Jochim (1989).

Conferences and Workshops

Governor's Conference on Geologic Hazards by the Utah Geological and Mineral Survey (1983).
3rd Annual Workshop on "Earthquake Hazards in the Puget Sound, Portland Area" by Hays (1989).
Workshop on "Evaluation of Earthquake Hazards and Risk in the Puget Sound and Portland Areas" by Hays (1988).
Workshop on Future Directions in Evaluating Earthquake Hazards of Southern California by Brown, Kockelman, and Ziony (1986).
Third International Earthquake Microzonation Conference by Sherif (1982, particularly sessions 3, 6, and 10).

List 2 (continued)

EXAMPLES OF TRANSFER TECHNIQUES FOR INFORMING THE PUBLIC

Outreach Programs

Circuit-rider Geologist in the State of Washington by Thorsen (1981).
Planning, Reviewing, and Enforcing by City and County Geologists by McCalpin (1985) and Christenson (1988).
Advisory Services Unit of the California Division of Mines and Geology by Amimoto (1980).
Educational, Advisory and Review Services by the Southeastern Wisconsin Regional Planning Commission (1968, 1987).
Earth Science Information Dissemination Activities of the U.S Geological Survey by Information Systems Council's Task Force on Long-Range Goals of USGS Information Dissemination (1987).

Examples and Discussions of Reduction Techniques

Anticipating Earthquakes--Risk Reduction Policies and Practices in the Puget Sound and Portland Areas by May (1989).
School Earthquake Emergency Planning by Noson and Martens (1987).
Case Studies on Strengthening Hazardous Buildings by the Bay Area Regional Earthquake Preparedness Project (1988).
Using Earth-science Information for Earthquake Hazard Reduction in the Los Angeles Region by Kockelman (1985).
Putting Seismic Safety Policies to Work by Blair-Tyler and Gregory (1988).
Examples of Seismic Zonation in the San Francisco Bay Region by Kockelman and Brabb (1979).

List 3

EARTHQUAKE HAZARDS REDUCTION SERIES [3]

FEMA 67	Earthquake Public Information Materials: An Annotated Bibliography	EHRS 8
FEMA 68	Earthquake Insurance: A Public Policy Dilemma	EHRS 7
FEMA 69	Pilot Project for Earthquake Hazard Assessment	EHRS 6
FEMA 70	Earthquake Preparedness Information for People with Disabilities	EHRS 5
FEMA 71	Comprehensive Earthquake Preparedness Planning Guidelines: Corporate	EHRS 4
FEMA 72	Comprehensive Earthquake Preparedness Planning Guidelines: County	EHRS 3
FEMA 73	Comprehensive Earthquake Preparedness Planning Guidelines: City	EHRS 2
FEMA 74	Reducing the Risks of Nonstructural Earthquake Damage: A Practical Guide	EHRS 1
FEMA 83	Societal Implications: A Community Handbook	EHRS 13
FEMA 84	Societal Implications: Selected Readings	EHRS 14
FEMA 87	Guidelines for Local Small Businesses	EHRS 12
FEMA 90	An Action Plan for Reducing Earthquake Hazards of Existing Buildings	EHRS 16
FEMA 91	Proceedings: Workshop on Reducing Seismic Hazards of Existing Buildings	EHRS 15
FEMA 95	NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings Part 1: Provisions and Maps (1985 Edition)	EHRS 17
FEMA 96	NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings Part II: Commentary (1985 Edition)	EHRS 18
FEMA 98	Guidelines for Preparing Code Changes Based on the NEHRP Recommended Provisions	EHRS 21
FEMA 99	Improving Seismic Safety of New Buildings: A Nontechnical Explanation of NEHRP Provisions	EHRS 20
L-143	Preparedness in Apartments and Mobile Homes	EHRS 22
FEMA 111	A guide to Marketing Earthquake Preparedness: Community Campaigns that Get Results	EHRS 23
FEMA 112	Marketing Earthquake Preparedness: Community Campaigns that Get Results	EHRS 24
FEMA 135	Abatement of Seismic Hazards to Lifelines: Water and Sewer	EHRS 26
FEMA 136	Abatement of Seismic Hazards to Lifelines: Transportation	EHRS 27
FEMA 137	Abatement of Seismic Hazards to Lifelines: Communications	EHRS 28
FEMA 138	Abatement of Seismic Hazards to Lifelines: Power	EHRS 29
FEMA 139	Abatement of Seismic Hazards to Lifelines: Gas and Liquid Fuels	EHRS 30
FEMA 140	Guide to Application of the NEHRP Recommended Provisions in Earthquake-Resistant Building Design	EHRS 25

FEMA 142	Abatement of Seismic Hazards to Lifelines: An Action Plan	EHRS 32
FEMA 143	Abatement of Seismic Hazards to Lifelines: Papers on Political, Economic, Social, Legal, and Regulatory Issues	EHRS 31
FEMA 146	Comprehensive Earthquake Preparedness Planning Guidelines: Large City	EHRS 33
FEMA 149	Seismic Considerations: Elementary and Secondary Schools	EHRS 34
FEMA 150	Seismic Considerations: Health Care Facilities	EHRS 35
FEMA 151	Seismic Considerations: Hotels and Motels	EHRS 36
FEMA 152	Seismic Considerations: Apartment Buildings	EHRS 37
FEMA 153	Seismic Considerations: Office Buildings	EHRS 38
FEMA 154	Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook	EHRS 41
FEMA 155	Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation	EHRS 42
FEMA 156	Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume I--Summary	EHRS 39
FEMA 157	Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume II--Supporting Documentation	EHRS 40
FEMA 158	Earthquake Damaged Buildings: An Overview of Heavy Debris and Victim Extrication	EHRS 43
FEMA 162	Differences between the 1985 and 1988 Editions of the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings	EHRS 44
FEMA 172	Techniques for Seismically Rehabilitating Existing Buildings (Preliminary)	EHRS 49
FEMA 173	Establishing Programs and Priorities for the Seismic Rehabilitation of Buildings: Supporting Report	EHRS 46
FEMA 174	Establishing Programs and Priorities for the Seismic Rehabilitation of Buildings: A Handbook	EHRS 45
FEMA 175	Seismic Evaluation of Existing Buildings: Supporting Documentation	EHRS 48
FEMA 176	Estimating Losses from Future Earthquakes--Panel Report (A Non-Technical Summary)	EHRS 50
FEMA 177	Estimating Losses from Future Earthquakes (Panel Report and Technical Background)	EHRS 51
FEMA 178	A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)	EHRS 47

The publications are free of charge; copies may be requested by writing to:

Federal Emergency Management Agency
P.O. Box 70274
Washington, D.C. 20024

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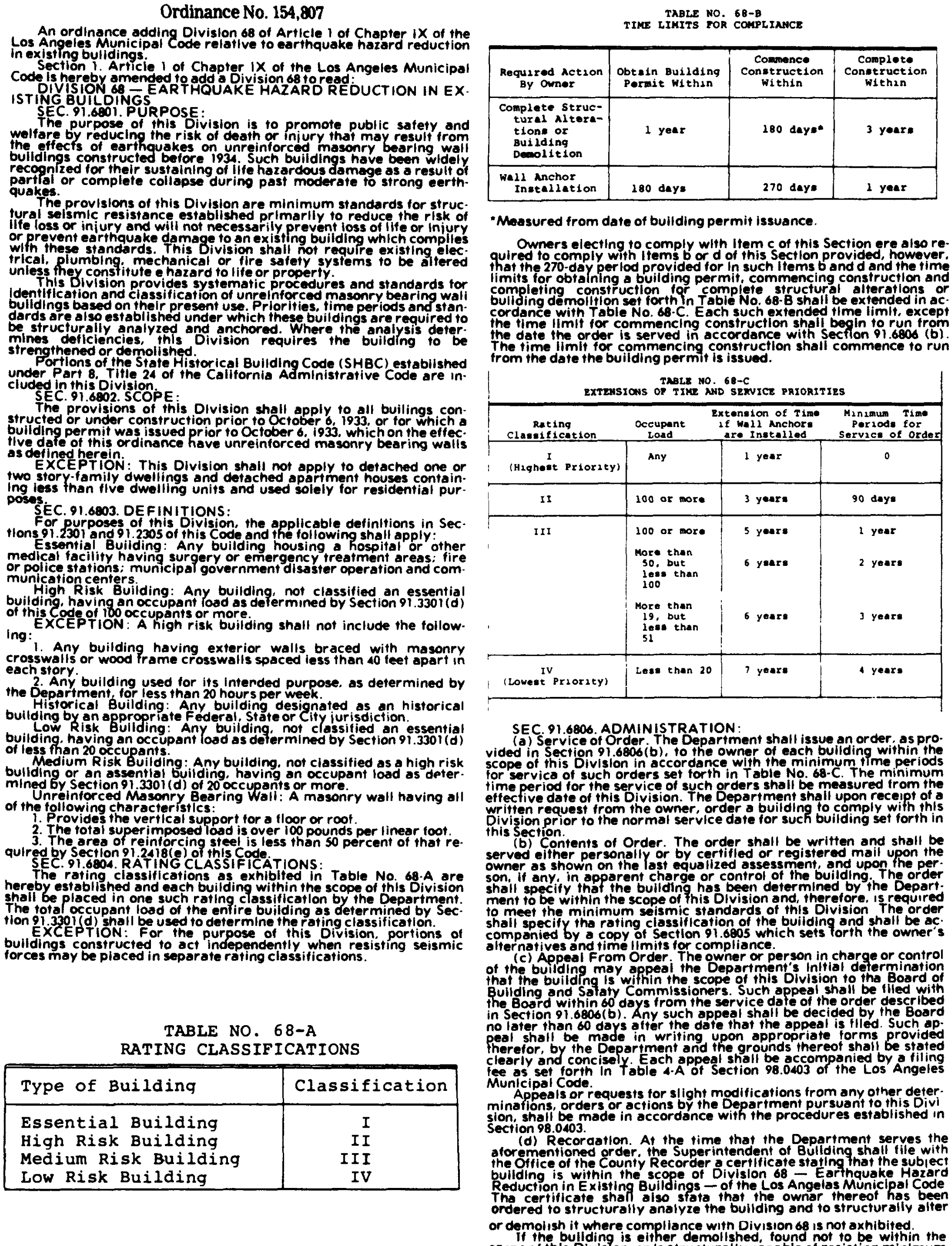
[3] Modified from an Earthquake Hazards Reduction Series list prepared by the Federal Emergency Management Agency (July 1989).



Ordinance No. 154,807

An ordinance adding Division 68 of Article 1 of Chapter IX of the Los Angeles Municipal Code relative to earthquake hazard reduction in existing buildings.

Section 1. Article 1 of Chapter IX of the Los Angeles Municipal Code is hereby amended to add a Division 68 to read:

DIVISION 68 — EARTHQUAKE HAZARD REDUCTION IN EXISTING BUILDINGS

SEC. 91.6801. PURPOSE:

The purpose of this Division is to promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes on unreinforced masonry bearing wall buildings constructed before 1934. Such buildings have been widely recognized for their sustaining of life hazardous damage as a result of partial or complete collapse during past moderate to strong eerthquakes.

The provisions of this Division are minimum standards for structural seismic resistance established primarily to reduce the risk of life loss or injury and will not necessarily prevent loss of life or injury or prevent earthquake damage to an existing building which complies with these standards. This Division shall not require existing electrical, plumbing, mechanical or fire safety systems to be altered unless they constitute e hazard to life or property.

This Division provides systematic procedures and standards for identification and classification of unreinforced masonry bearing wall buildings based on their present use. Priorities, time periods and standards are also established under which these buildings are required to be structurally analyzed and anchored. Where the analysis determines deficiencies, this Division requires the building to be strengthened or demolished.

Portions of the State Historical Building Code (SHBC) established under Part 8, Title 24 of the California Administrative Code are included in this Division.

SEC. 91.6802. SCOPE:

The provisions of this Division shall apply to all builings constructed or under construction prior to October 6, 1933, or for which a building permit was issued prior to October 6, 1933, which on the effective date of this ordinance have unreinforced masonry bearing walls as defined herein.

EXCEPTION: This Division shall not apply to detached one or two story-family dwellings and detached apartment houses containing less than five dwelling units and used solely for residential purposes.

SEC. 91.6803. DEFINITIONS:

For purposes of this Division, the applicable definitions in Sections 91.2301 and 91.2305 of this Code and the following shall apply:

Essential Building: Any building housing a hospital or other medical facility having surgery or emergency treatment areas; fire or police stations; municipal government disaster operation and communication centers.

High Risk Building: Any building, not classified an essential building, having an occupant load as determined by Section 91.3301(d) of this Code of 100 occupants or more.

EXCEPTION: A high risk building shall not include the following:

1. Any building having exterior walls braced with masonry crosswalls or wood frame crosswalls spaced less than 40 feet apart in each story.
2. Any building used for its intended purpose, as determined by the Department, for less than 20 hours per week.

Historical Building: Any building designated as an historical building by an appropriate Federal, State or City jurisdiction.

Low Risk Building: Any building, not classified an essential building, having an occupant load as determined by Section 91.3301(d) of less than 20 occupants.

Medium Risk Building: Any building, not classified as a high risk building or an assential building, having an occupant load as determined by Section 91.3301(d) of 20 occupants or more.

Unreinforced Masonry Bearing Wall: A masonry wall having all of the following characteristics:

1. Provides the vertical support for a floor or roof.
2. The total superimposed load is over 100 pounds per linear foot.
3. The area of reinforcing steel is less than 50 percent of that required by Section 91.2418(e) of this Code.

SEC. 91.6804. RATING CLASSIFICATIONS:

The rating classifications as exhibited in Table No. 68-A are hereby established and each building within the scope of this Division shall be placed in one such rating classification by the Department. The total occupant load of the entire building as determined by Section 91.3301(d) shall be used to determine the rating classification.

EXCEPTION: For the purpose of this Division, portions of buildings constructed to act independently when resisting seismic forces may be placed in separate rating classifications.

TABLE NO. 68-A
RATING CLASSIFICATIONS

Type of Building	Classification
Essential Building	I
High Risk Building	II
Medium Risk Building	III
Low Risk Building	IV

SEC. 91.6805. GENERAL REQUIREMENTS:

The owner of each building within the scope of this Division shall cause a structural analysis to be made of the building by a civil or structural angineer or arthitect licensed by the State of California; and, if the building does not meet the minimum earthquake standards specified in this Division, the owner shall cause it to be structurally altered to conform to such standards; or cause the building to be demolished.

The owner of a building within the scope of this Division shall comply with the requiremants set forth above by submitting to the Department for review within the stated time limits:

a. Within 270 days after tha service of the order, a structural analysis. Such analysis which is subject to approval by tha Department, shall demonstrate that the building meets the minimum requirements of this Division; or

b. Within 270 days after the service of the order, the structural analysis and plans for the proposed structural alterations of the building necessary to comply to the minimum requiraments of this Division; or

c. Within 120 days after service of the order, plans for the installation of wall anchors in accordance with the requirements specified in Section 91.6808(c); or

d. Within 270 days after the servica of tha order, plans for the demolition of the building.

After plans are submitted and approved by the Department, the owner shall obtain a building permit, commence and complete the required construction or demolition within the time limits set forth in No. Table 68-B. These time limits shall begin to run from the date the order is served in accordance with Section 91.6806(a) and (b).

TABLE NO. 68-B
TIME LIMITS FOR COMPLIANCE

Required Action By Owner	Obtain Building Permit Within	Commence Construction Within	Complete Construction Within
Complete Structural Alterations or Building Demolition	1 year	180 days*	3 years
Wall Anchor Installation	180 days	270 days	1 year

*Measured from date of building permit issuance.

Owners electing to comply with item c of this Section ere also required to comply with items b or d of this Section provided, however, that the 270-day period provided for in such items b and d and the time limits for obtaining a building permit, commencing construction and completing construction for complete structural alterations or building demolition set forth in Table No. 68-B shall be extended in accordance with Table No. 68-C. Each such extended time limit, except the time limit for commencing construction shall begin to run from the date the order is served in accordance with Section 91.6806 (b). The time limit for commencing construction shall commence to run from the date the building permit is issued.

TABLE NO. 68-C
EXTENSIONS OF TIME AND SERVICE PRIORITIES

Rating Classification	Occupant Load	Extension of Time if Wall Anchors are Installed	Minimum Time Periods for Service of Order
I (Highest Priority)	Any	1 year	0
II	100 or more	3 years	90 days
III	100 or more	5 years	1 year
	More than 50, but less than 100	6 years	2 years
	More than 19, but less than 51	6 years	3 years
IV (Lowest Priority)	Less than 20	7 years	4 years

SEC. 91.6806. ADMINISTRATION:

(a) Service of Order. The Department shall issue an order, as provided in Section 91.6806(b), to the owner of each building within the scope of this Division in accordance with the minimum time periods for servica of such orders set forth in Table No. 68-C. The minimum time period for the service of such orders shall be measured from the effective date of this Division. The Department shall upon receipt of a written request from the owner, order a building to comply with this Division prior to the normal service date for such building set forth in this Section.

(b) Contents of Order. The order shall be written and shall be served either personally or by certified or registered mail upon the owner as shown on the last equalized assessment, and upon the person, if any, in apparent charge or control of the building. The order shall specify that the building has been determined by the Department to be within the scope of this Division and, therefore, is required to meet the minimum seismic standards of this Division The order shall specify tha rating classification of the building and shall be accompanied by a copy of Section 91.6805 which sets forth the owner's alternatives and time limits for compliance.

(c) Appeal From Order. The owner or person in charge or control of the building may appeal the Department's initial determination that the building is within the scope of this Division to tha Board of Building and Satety Commissioners. Such appeal shall be filed with the Board within 60 days from the service date of the order described in Section 91.6806(b). Any such appeal shall be decided by the Board no later than 60 days after the date that the appeal is filed. Such appeal shall be made in writing upon appropriate forms provided therefor, by the Department and the grounds thereof shall be stated clearly and concisely. Each appeal shall be accompanied by a filing fee as set forth in Table 4-A of Section 98.0403 of the Los Angeles Municipal Code.

Appeals or requests for slight modifications from any other determinations, orders or actions by the Department pursuant to this Division, shall be made in accordance with the procedures established in Section 98.0403.

(d) Recoraation. At the time that the Department serves the aforementioned order, the Superintendent of Building shall file with the Office of the County Recorder a certificate stating that the subject building is within the scope of Division 68 — Earthquake Hazard Reduction in Existing Buildings — of the Los Angelas Municipal Code Tha certificate shall also stata that the ownar thereof has been ordered to structurally analyze the building and to structurally alter or demolish it where compliance with Division 68 is not axhibited.

If the building is either demolished, found not to be within the scope of this Division, or is structurally capable of resisting minimum seismic forces required by this Division as a result of structural alterations or an analysis, the Superintendent of Building shall file with the Office of tha County Recorder a certificate terminating the status of the subject building as being classified within the scope of Division 68 — Earthquake Hazard Reduction in Existing Buildings — of the Los Angeles Municipal Coda.

(e) Enforcement. If the owner or other person in charge or control of the subject building fails to comply with any order issued by the Department pursuant to this Division within any of the time limits set forth in Section 91.6805, the Superintendent of Building shall ordar that the entire building be vacated and that the building remain vacated until such order has been complied with. If compliance with such order has not been accomplished within 90 days after the date the building has been ordered vacated or such additional time as may have been granted by the Board and the Superintendent may order its demolition in accordance with the provisions of Section 91.0103(o) of this Code.

SEC. 91.6807. HISTORICAL BUILDINGS:

(a) General. The standards and procedures established by this Division shall apply in all respects to an historical building except that as a means to preserve original architectural elemants and facilitate restoration, an historical building may, in addition, comply with the special provisions set forth in this Section.

(b) Unburned Clay Masonry or Abode. Existing or re-erected walls of abode construction shall conform to the following:

1. Unreinforced abode masonry wall shall not exceed a height or length to thickness ratio of 5, for exterior bearing walls and must be provided with a rainforced bond beam at the top, intarconnecting all walls. Minimum beam depth shall be 6 inches and a minimum width

FIGURE 6.--Part of the Los Angeles City Council (1981) earthquake-hazard reduction ordinance requiring owners of buildings having unreinforced masonry bearing walls constructed before 1933 to obtain a structural analysis. If the building does not meet the minimum standards, the owner is required to strengthen or remove it according to a specific time schedule.

building is also required in order to determine the remedial measures necessary to meet the appropriate standards. The city provides a specific time schedule.

An alternative compliance schedule, intended to lessen the financial and social impacts of the ordinance, gives a building owner the option of performing a portion of the remedial work within 1 yr of notification in exchange for a longer time in which to reach full compliance. The work to be performed within a year involves the anchoring of unreinforced masonry walls to the roof and to each floor of the building with bolts and washers. According to the Los Angeles City Planning Department (1979, p. 5), this procedure yields an immediate and substantial improvement in safety for perhaps one-fifth the cost of full compliance.

Using the experience of the City of Los Angeles, the California Legislature (1986) requires all cities and counties in seismic zone 4 to identify hazardous unreinforced masonry buildings, establish a mitigation program, and notify the building owners. Local building departments are authorized to establish fees to recover the costs of identification. The mitigation program may include:

> the adoption by ordinance of a hazardous buildings program, measures to strengthen buildings, measures to change the use to acceptable occupancy levels or to demolish the building, tax incentives available for seismic rehabilitation, low-cost seismic rehabilitation loans ..., application of structural standards necessary to provide for life safety above current code requirements, and other incentives to repair the buildings which are available from federal, state, and local programs.

Compliance with an adopted hazardous buildings ordinance or mitigation program is the responsibility of building owners. Nothing in the law makes any local government responsible for paying the cost of strengthening a privately owned structure, reducing the occupancy, demolishing a structure, preparing engineering or architectural analysis, conducting investigations, or other costs associated with compliance of locally adopted mitigation programs.

A model ordinance and guidebook has been developed by the California Seismic Safety Commission (1987). The guidebook contains a series of steps for both identifying potentially hazardous buildings and developing and implementing a hazard-mitigation program. Other discussions on costs to local government, costs to building owners, incentives, and where to go for information are included.

Some of the advantages of such ordinances are that deaths and injuries will be substantially reduced; economically-obsolete buildings will eventually be removed and the land reused; and repair or demolition will provide work for the construction industry. Some of the disadvantages of such ordinances are that some low-income housing will be lost; tenants probably will have to be relocated; and businesses will be interrupted.

ESTIMATING CASUALTIES, DAMAGE, AND INTERRUPTIONS

Several techniques to assist state and local governments in preparing for, responding to, and recovering from earthquake emergencies and disasters are available. One of the techniques is commonly called "loss estimates." A National Research Council (1989) panel defines an earthquake loss estimate as "a forecast of the effects of a hypothetical earthquake, Depending on its purpose, a loss study may include estimates of deaths and injuries; property losses; loss of function in industries, lifelines, and emergency facilities; homelessness; and economic impacts." These loss estimates are also effective techniques to create public awareness of hazards and support for the preparedness measures, response, and recovery operations. Four examples of loss estimates follow.

The Federal Emergency Management Agency (FEMA) (1981) estimated dead, hospitalized, injured but not hospitalized, loss to buildings, and loss to building contents for four postulated earthquakes in California (fig. 7). In addition, damage to or impact on selected facilities or needs were discussed. These included temporary housing, key communication facilities, military command circuits, all transportation modes, businesses, and industries. FEMA and the California Office of Emergency Services then conducted an analysis of readiness and discussed Federal, State, and local responses and response planning.

Davis and others (1982) prepared a planning scenario for a postulated earthquake in the Los Angeles region: A scenario is usually thought of as a synopsis or outline of a play or a movie; thus, a scenario for an earthquake can be considered a synopsis or outline of a large seismic event and its severe impacts on an urban region.' Their scenario is used to assess the effects of a future earthquake on principal lifelines for emergency planning purposes. An analysis of readiness can then be used to provide planning insights, recommend further work, and serve as a basis for making or improving emergency preparedness, response, recovery, and reconstruction plans.

They include individual scenarios which show damage to critical facilities[4/], specifically lifelines such as highways, airports, railroads, marine facilities, communication lines, water-supply and waste-disposal facilities, and electrical power, natural gas, and petroleum lines. The scenarios for lifelines are based on evaluation of earthquake-engineering literature, comments by numerous engineers and officials of public agencies, and judgments by the authors. This assessment of the effects of the

---

[4/] The term "critical facilities" is used here to include (1) lifelines such as major communication, utility, and transportation facilities and their connections to emergency facilities; (2) unique or large structures whose failure might be catastrophic, such as dams or buildings where explosive, toxic, or radioactive materials are stored or handled; (3) high-occupancy buildings such as schools, churches, hotels, offices, auditoriums, and stadiums; and (4) emergency facilities such as police and fire stations, hospitals, communications centers, and disaster-response centers.

Fault	Time	Dead	Hospitalized[2]
Northern San Andreas	2:30 a.m.	3,000	12,000
	2:00 p.m.	10,000	37,000
	4:30 p.m.	11,000	44,000
Hayward	2:30 a.m.	3,000	13,000
	2:00 p.m.	8,000	30,000
	4:30 p.m.	7,000	27,000
Southern San Andreas	2:30 a.m.	3,000	12,000
	2:00 p.m.	12,000	50,000
	4:30 p.m.	14,000	55,000
Newport-Inglewood	2:30 a.m.	4,000	18,000
	2:00 p.m.	21,000	83,000
	4:30 p.m.	23,000	91,000

---

[1] Uncertain by a possible factor of two to three.

[2] Injuries not requiring hospitalization are estimated to be from 15 to 30 times the number of deaths.

FIGURE 7.--Estimated consequences of a catastrophic earthquake occurring on each of four faults for three different times from the Federal Emergency Management Agency (1981, table 3, p. 23).

earthquake on lifelines was made to evaluate the resulting performance of lifeline segments throughout the region. The communications map, for example, assesses telephone-systems performance following the postulated earthquake (fig. 8). Other maps (those for water-supply and waste-disposal facilities, for example) show the location of and estimates of damage to specific facilities. Most of the planning maps for the scenario contain notations that are explained in the text; for example, one notation reads, "Water deliveries through the MWD Upper Feeder will be temporarily interrupted by pipe rupture where this major transmission line crosses the Santa Ana River." Most of the lifelines will sustain significant damage that could require a major emergency-response effort. Each scenario map is accompanied by a discussion of the general patterns of effects of the earthquake, for example:

> Interstate 5 from the San Joaquin Valley and Interstate 15 through Cajon Pass will be closed, leaving U.S. 101 along the coast as the only major viable route open from the north. Highway connections with San Diego will remain open.
>
> Not all of the (telephone) systems in the greater Los Angeles region are set up to process emergency calls automatically on previously established priority bases. Thus overloading of equipment still in service could be very significant.

Similar scenarios have been prepared for other earthquakes, for example, on the Hayward fault in the San Francisco Bay region by Steinbrugge and others (1987).

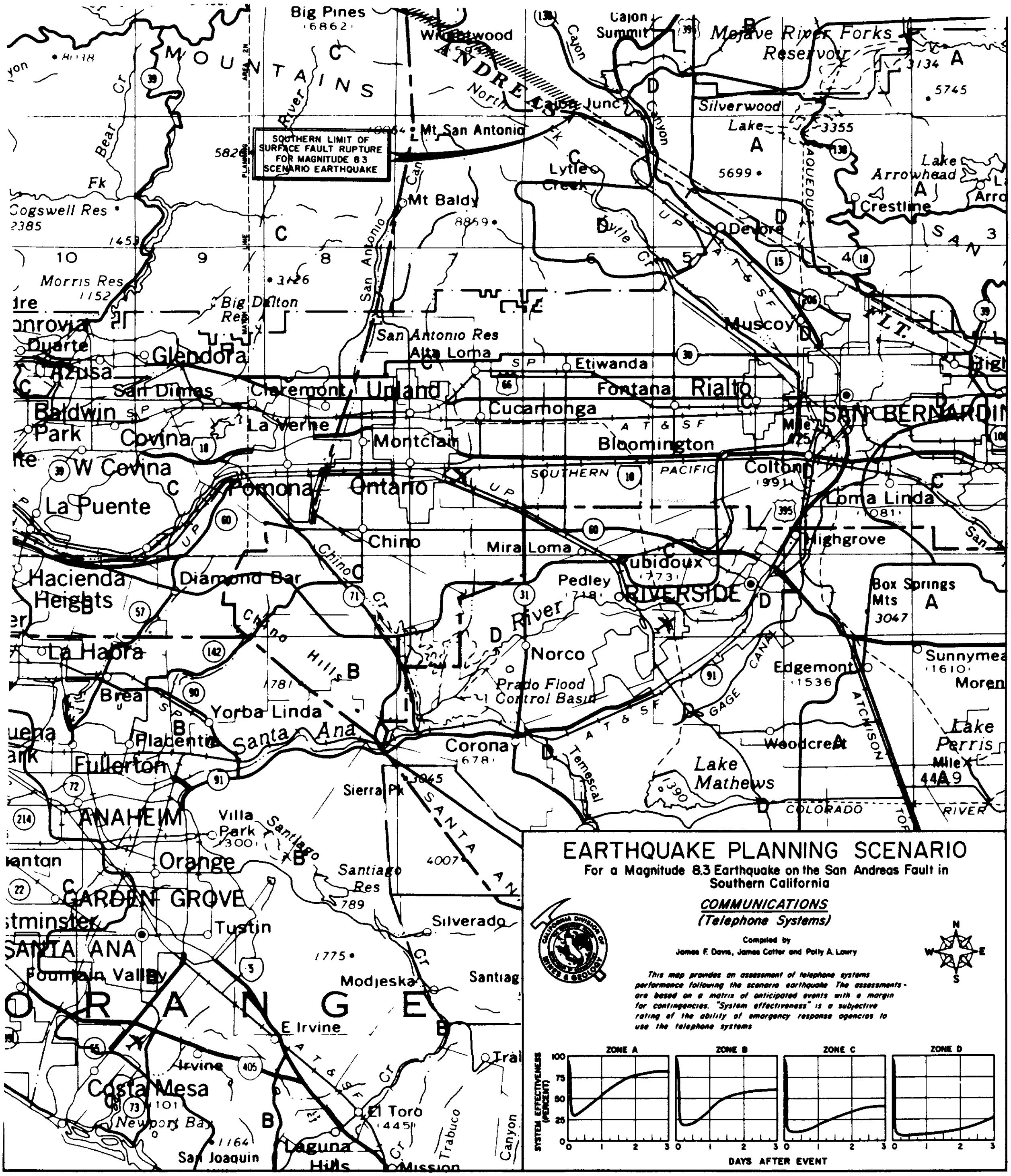
The U.S. Geological Survey (1975) postulated an earthquake in two locations in the Puget Sound area and concluded that under the worst conditions of exposure, as many as 2,200 deaths, 8,700 injuries, and 23,500 homeless were possible. In addition, anticipated damage patterns for five counties--Snohomish, King, Pierce, Thurston, Mason, and Kitsap--were estimated for both earthquakes. A degree of impairment was assigned for selected critical facilities, equipment, or supplies (fig. 9). A detailed presentation of each of the impairments is included, for example:

- Damage to general hospitals having capacities of 50 or more beds
- Deaths to physicians and nurses at nonhospital locations
- Stock losses at retail drugstores and pharmacies
- Damage to railroad bridges and tunnels
- Probability of fatalities based upon siting of schools in areas of high damage intensities

It should be noted that loss estimates, damage scenarios, and degrees of impairment are for planning purposes only, and some may consider them overly pessimistic. However, in emergency planning, it is important to consider severe levels of casualties and socioeconomic disruption to be better able to prepare, response, and recover.

CLOSING COMMENTS

Prerequisites to the selection and implementation of an appropriate earthquake hazard-reduction technique from list 1 are:



EARTHQUAKE PLANNING SCENARIO

For a Magnitude 8.3 Earthquake on the San Andreas Fault in Southern California

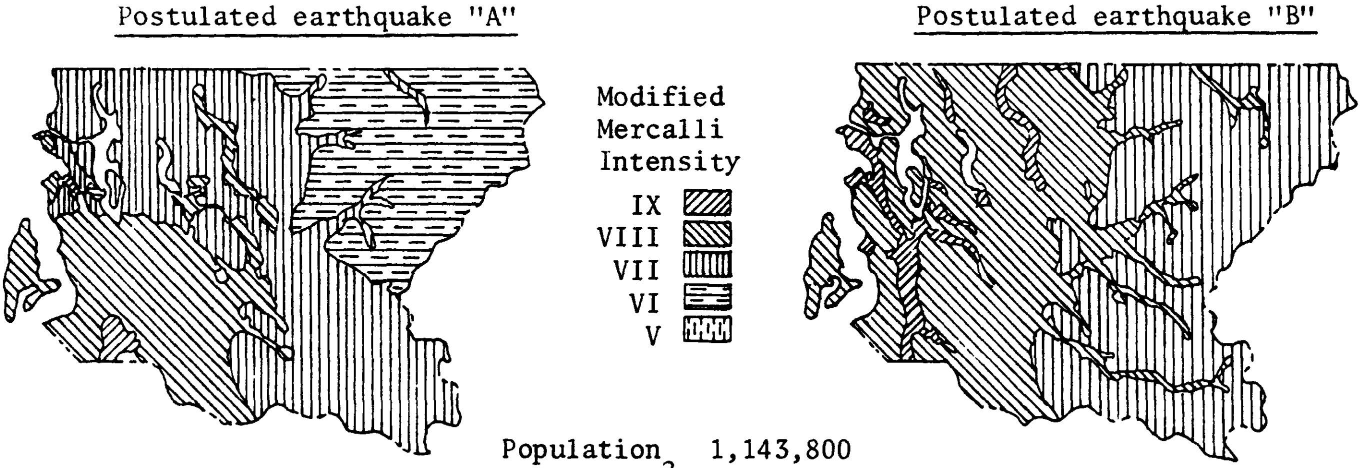
COMMUNICATIONS (Telephone Systems)

Compiled by James F. Davis, James Cotter and Polly A. Lowry

This map provides an assessment of telephone systems performance following the scenario earthquake. The assessments are based on a matrix of anticipated events with a margin for contingencies. "System effectiveness" is a subjective rating of the ability of emergency response agencies to use the telephone systems.

SOUTHERN LIMIT OF SURFACE FAULT RUPTURE FOR MAGNITUDE 8.3 SCENARIO EARTHQUAKE

FIGURE 8.--Planning scenario impact of an earthquake on the telephone systems for part of a metropolitan region. Compilation by Davis and others (1982) shows the percentage of telephone-system effectiveness in four zones designated A, B, C, and D up to 3 days after the postulated earthquake.



Population 1,143,800
Area in mi^2 2,128

Vital needs	Degree of impairment					
	Earthquake "A"			Earthquake "B"		
	Minimal	Minor	Major	Minimal	Minor	Major
Communications		●				●
Fire		●				●
Police		●				●
Electric power			●			●
Water		●				●
Access roadways			●			●
Medical:						
Manpower			●			●
Hospitals			●			●
Ambulances		●				●
Blood bank		●				●
Supplies		●			●	
Food supplies	●				●	
Schools (as shelters)		●				●

	Estimated losses	
	Earthquake "A"	Earthquake "B"
Deaths	1,500	1,650
Serious injuries	6,000	6,600
Homeless	7,130	18,630

FIGURE 9.--Anticipated damage, impairment, and casualties from two postulated earthquakes in King County from a study on earthquake losses by the U.S. Geological Survey (1975, table 2, p. 5).

- Conducting scientific and engineering studies of the physical processes of earthquake phenomena--source, location, size, likelihood of occurrence, triggering mechanism, path, ground response, structure response, and equipment response.

- Translating the results of such studies into reports and onto maps at an appropriate scale so that the nature and extent of the hazards and their effects are understood by nontechnical users.

- Transferring this translated information to those who will or are required to use it, and assisting and encouraging them in its use.

Scientific and engineering studies

Numerous geologic, geophysical, seismologic, and engineering studies are necessary to assess potential earthquake hazards. To give the nontechnical reader an overview, some of the studies are shown in the Proceedings of the 3rd Annual Workshop on "Earthquake hazards in the Puget Sound, Portland Area" (Hays, 1989, list 1, p. 193, 194). Most of these studies are complex, interconnected, have limitations because of lack of data, and require special technical skills.

It is not prudent for planners to develop land-use regulations, engineers to design structures, and lenders and public works directors to adopt policies reducing earthquake hazards without adequate and reliable scientific and engineering assessments. Many of these studies were envisioned and are described in the "Regional Earthquake Hazards Assessments" draft work plan for the Pacific Northwest. This plan is reproduced in a workshop proceedings by Hays (1988, p. 12-33).

Translation for nontechnical users

The objective of translating scientific and engineering information for nontechnical users is to: make them aware that a hazard exists which may affect them or their interests; provide them with information that they can easily present to their superiors, clients, or constituents; and provide them with materials that can be directly used in a reduction technique.

My experience with reducing potential natural hazards indicates that natural-hazard information successfully used by nontechnical users has the following three elements in one form or another:

- Likelihood of the occurrence of an event that will cause casualties, damage, or disruption.

- Location of the effects of the event on the ground.

- Estimated severity of the effects on the ground, structure, or equipment.

These elements are needed because usually engineers, planners, and decisionmakers will not be concerned with a potential hazard if its likelihood is rare, its location is unknown, or its severity is slight; neither will lenders, politicians, or citizens.

Transfer to nontechnical users

The objective of transferring hazard information to nontechnical users is to assist in and encourage its use to reduce losses from future earthquakes. Translated hazard information is a prerequisite for transfer to nontechnical users.

Various terms are used to convey "transfer" of information to users, namely, disseminate, communicate, circulate, promulgate, or distribute. Often these terms are interpreted conservatively, for example, merely issuing a press release on hazards or distributing research information to potential users. This level of activity usually fails to result in effective hazard reduction techniques and may even fail to make users aware of the hazard. Therefore, I suggest that we use "transfer" to mean the delivery of a translated product in a usable format at a scale appropriate to its use by a specific person or group "interested" in, or responsible for, hazard reduction. To delivery of a product, we must add assistance and encouragement in its use for hazard reduction.

Evaluation and revision

The effectiveness of each hazard reduction technique varies with the time, place, and persons involved. Therefore, it is prudent to include a continuing systematic evaluation as part of any comprehensive earthquake-hazard reduction program. An inventory of uses made of the information, reports of interviews with the users, and an analysis of the results and responses will also result in identifying new users, innovative uses, as well as any problems concerning the scientific and engineering studies, their translation, transfer, and use. The evaluation will be helpful, even necessary, to those involved in producing, translating, transferring, and using the research information as well as to those funding and managing the program.

Performing the studies and then translating and transferring the research information is expensive and difficult because of the limited number of scientists and geotechnicians--national, state, local, corporate, and consulting--particularly when aligned with the needs of communities throughout the United States. The adoption and enforcement of an appropriate hazard reduction technique is time-consuming, and requires many skills--planning, engineering, legal, and political--as well as strong and consistent public support.

Scarce financial and staff resources must be committed; necessarily persistent and difficult actions must be taken to enact a law, adopt a policy, or administer a reduction program over a long period of time. To discover later that the hazard reduction technique selected is ineffective, unenforced, or its cost is greatly disproportionate to its benefits is not only disheartenting but may subject those involved to criticism and withdrawal of financial support!

CONCLUSION

The examples of earthquake-hazard reduction techniques presented in this paper include: preparing redevelopment plans, creating regulatory zones, securing nonstructural building components, informing the public, strengthening unreinforced masonry buildings, and estimating casualties, damage, and interruptions.

The effect of these techniques is to provide greater public safety, health, and welfare for individuals and their communities. The decision to adopt each technique was influenced by many factors--the nature of the earthquake hazard, public concern, strong community interest, state enabling legislation, the availability of scientific and engineering information, and the ability of geologists, engineers, planners, and lawyers to incorporate the information into a hazard reduction technique.

Some of the geologic and seismologic information needed for prudent land use and general planning in the Pacific Northwest region is available, but generally not at the level of detail and scale needed for engineering and decisionmaking. Even greater detail at larger scales ranging from 1:1,200 to 1:12,000 (1 in = 100 to 1,000 ft) is needed for other purposes, including development planning, site investigation, ordinance administration, project review, and permit issuance.

Earthquake-hazard research is continuing, the information base is improving, the methods for evaluating hazards are being developed, and new reduction techniques may be tested. Planners, engineers, and decisionmakers (both public and private) need to recognize these facts and use the latest information, methods, and techniques. However, they cannot be expected to have the training or experience necessary to understand and use untranslated scientific information. Therefore, if nontechnical users are to benefit from this information, it must be translated for and transferred to nontechnical users.

Within the Pacific Northwest region, planners, engineers, and decisionmakers (public and private) live and work in a complex environments. Moreover, the geologic environment is just one aspect of their life and work. Other aspects include social, economic, political, and esthetic considerations; some of these aspects are more apparent or more important to individual planners, engineers, or decisionmakers and their constituents.

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