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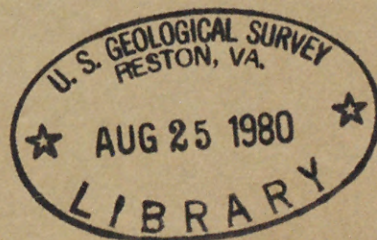
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Geotechnical Engineering

"MICROZONATION OF THE MEMPHIS, TENNESSEE AREA" (Phase 1)

Sunil Sharma
and
William D. Kovacs

A REPORT ON RESEARCH SPONSORED BY
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PURDUE UNIVERSITY

The Microzonation of the Memphis, Tennessee, Area

by Sunil Sharma
Purdue University
School of Civil Engineering
Grissom Hall
West Lafayette, Indiana 47907

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ABSTRACT

Although the probability of a major earthquake in the central U.S. is only one-tenth of the Californian events, any such event is expected to result in damage ten times greater than that anticipated in California. The city of Memphis, which is situated very close to the inferred epicenter of one of the three major 1811-1812 earthquakes, is in a potentially hazardous zone which will be susceptible to the usual seismic hazards. By recognizing the high level of seismicity in the New Madrid area, this study attempts to microzone the potential hazards in the city of Memphis.

This study examines the pertinent criteria necessary for any microzonation study. The subjects considered include:

- (i) The seismicity of the central United States,
- (ii) Design earthquakes,
- (iii) Response analysis which allows us to construct the necessary microzonation maps.

The seismicity of the region is evaluated according to the state-of-the-art literature available as there is no recorded strong motion data available for the central U.S. The maximum credible earthquakes which are likely to affect Memphis are evaluated as the one in a thousand year occurrence. However, earthquakes of a lower intensity (and lower recurrence rates) are selected as design earthquakes to permit a more realistic microzonation to be performed.

For the response analysis, these earthquakes are simulated using synthetically generated accelerograms which exhibit the relevant parameters. These accelerograms displaying the significant characteristics of amplitude, predominant frequency and duration were selected to display, as accurately as possible, the anticipated nature of the horizontal bedrock-motions at

Memphis. The horizontal motions (SH-waves) were applied at a depth of 45m below ground surface at numerous sites in the city of Memphis, where the soil stratigraphy had been conceptualized from borehole data.

The soils data made available by local sources was the only information used for this study as any laboratory testing was outside the scope of this study. The dynamic soil properties were thus established from the available Standard Penetration Resistances and soil classifications.

The results of the response analysis were transformed into microzonation maps depicting

- (i) zones showing qualitative estimates of ground response,
- (ii) zones showing the natural frequency of the soils,
- (iii) zones showing the peak spectral acceleration for
2% damping ratio,
- (iv) zones of liquefaction potential.

These maps are essentially useful for preliminary investigation and we do not expect them to be used on a quantitative basis. However, further investigation is necessary in determining the stratigraphy and soil properties to a more accurate level if an earthquake hazard is anticipated.

Microzonation of the Memphis, TN Area

Final Report

(Phase I)

Sunil Sharma and William D. Kovacs
School of Civil Engineering
Purdue University, W. Lafayette, IN 47907

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Introduction

The city of Memphis is located within one of the most active seismic zones in the central United States. Although the recurrence intervals for major earthquakes is one-tenth compared to California, the damage potential is expected to be ten times greater.

Historically, the 1811-1812 earthquakes are regarded as some of the most powerful shocks ever felt in the United States. A total of 2,000,000 sq. miles, or half of the entire United States, was so disturbed that vibrations were perceived without the aid of any instrumentation. The landscape in the vicinity of the epicenters was altered extensively with subsidence, rifting and sand-blowouts being reported. As the area was only sparsely populated at the time of these earthquakes, loss of life and property was small.

The epicenter of the December 16, 1811 earthquake was located very close to the city of Memphis and thus any repetition of that event is liable to have grave consequences. It was with this past turbulent seismic history in mind that we investigated the seismicity and its effects on the region.

Briefly, we are concerned with the preparation of numerous maps depicting the various zones which would suffer from the traditional earthquake hazards of:

- (i) Shaking from strong ground motion,
- (ii) Ground failure due to slope instability, liquefaction or subsidence.

Thus we shall attempt to microzone the city of Memphis from the available data and establish the suggested hazardous areas. We feel that there will be numerous limitations due to our approach but these are mainly due to the lack of:

- (i) Strong-motion data,
- (ii) Geotechnical data for the city of Memphis.

This study does not generate any new experimental field or laboratory data (e.g., soil testing); we have used existing pertinent data regarding the stratigraphy, geology and seismicity of the area. However, any microzonation is highly influenced by local soil conditions and thus any lack of detailed information is expected to be directly reflected in the final results of this study. We hope that local sources, upon reviewing the results of this study, will appreciate the limitations and will try to minimize the usage within the intended scope of the study.

All the procedures used for obtaining and evaluating the content of any information are documented in the remainder of this report. This will enable any future microzonation studies of the Memphis area to concentrate on aspects which cannot be fully evaluated within the scope of this study. Conversely, the procedures developed for this study are directly applicable to other regions, in the United States, requiring preliminary microzonation maps.

CHAPTER ONE

Scope and Methodology

As this study is essentially the microzonation of Memphis, Tennessee, our investigative area is determined by the city limits. However, we shall also investigate the surrounding area, where possible, as it is very difficult to impose strict boundaries on such a study.

The large seismic events recorded in 1811-12 were not entirely unexpected because there were traditional Indian legends which suggested past seismic events. It has also been estimated, that continuing modern movements along the "inferred" buried fault are the largest ever over the last 130 million years (Sloss and Speed, 1974). These movements may be episodic or oscillatory and thus must be comparable to tectonic movements and their associated seismicity. Due to the active nature of the faulting, Memphis is likely to experience earthquakes. In the eastern U.S. the recurrence interval between major shocks is about ten times longer than that of the Californian Earthquakes (Algermissen, 1972; Nuttli, 1974).

However, for any major shock, the damage potential may be ten times that of a comparable Californian event. It is expected that the fluvial deposits will magnify any bedrock motion which in combination with the lower attenuation rates east of the Rocky Mountains is liable to cause extensive ground-shaking. Hence the damage is likely to be of major proportions, depending on the magnitude and location of the event.

For any microzonation study there are three essential steps which must be followed:

- (i) Collection of geotechnical data from local sources and evaluation of the local stratigraphy. In addition, the following information must also be investigated:

- (a) Ground water table location,
 - (b) Geologic origin of sediments (i.e., consolidated/unconsolidated),
 - (c) Degree of compaction or density and classification,
 - (d) Depth to "bedrock" below ground surface.
- (ii) Determination (or estimation, if no instrumental data is available) of earthquake parameters which form an accelerogram to be used as bedrock motion input.
- (iii) Response analysis, which will evaluate the effects of the "bedrock" motion on the various soil-layers and surface features.

The above procedure is shown diagrammatically in Fig. 1-1. These concepts will be discussed further in CHAPTER TWO of this study.

The geological data will be procured from local sources who have performed soil-borings and other geotechnical investigations in the city of Memphis (see Fig. 1-2). This use of existing geological data will minimize any site-work which will thus reduce the cost of this study quite considerably. However, our end product, the microzonation map of the Memphis area, will not be perfect. Within the scope of this study we cannot investigate areas for which no information is available.

A literature search pertaining to the importance of fault zones which are likely to be the source of future earthquakes has also been performed. The geological aspects of the areas which will have a direct influence on Memphis have thus been considered in CHAPTER THREE. The data extracted from the soil-borings made available will be utilized for the response analysis and is considered further in Chapter Six.

The design earthquakes and the seismicity of the region have been obtained from a search through all pertinent literature. We are interested in

the seismicity which is expected to be of a sufficient magnitude to cause damage, this being defined as "strong motion". However, due to the inherent lack of strong motion data, we are restricted to tentative recurrence intervals and associated probabilities. These are discussed in CHAPTER FOUR.

A complex part of this study will involve the production of synthetic seismograms which are to be input as "bedrock motion" for the response analysis. There is an enormous lack of data in the field of strong motions. As no strong-motion seismograms have ever been recorded for an earthquake of $m_b > 5.0$ in the central United States, it is very difficult to predict the necessary parameters which are present in any seismogram. Hence, there is a need to generalize in the preparation of these seismograms in order to allow the inclusion of varying parameters. This variation would then allow the reader to adopt any new (or conflicting) parameters which may be more easily defined in the near future. The usual earthquake parameters (i.e., earthquake size, epicentral distance, predominant frequency and duration) will be further discussed in CHAPTER FIVE. After producing the pertinent seismograms, it is then possible to compute the Response Analysis.

The program SHAKE, developed at the University of California, Berkeley (1972) will be used to compute the effects of the "bedrock" motion upon horizontal soil layers. It is not possible to consider non-horizontal layers because the theory basically utilizes the wave-equation by considering the reflection and refraction of the SH-waves at the soil-layer interfaces. Only SH-waves will be considered in this study as they are expected to have the greatest effect on Engineering-design. Thus we shall not study the P, SV, Lg waves which also radiate from all earthquakes. The output data will allow us to evaluate the effects of the motion applied to the "bedrock". We shall then prepare the maps at a suitable scale, reflecting the amount of data available, depicting:

- (i) Zones showing qualitative estimates of ground response,
- (ii) Zones showing the natural frequency of the soils,
- (iii) Zones of liquefaction and subsidence potential,
- (iv) Zones of landslide susceptibility.

The Response Analyses will be presented in CHAPTER SIX of this Study. Conclusions will be presented along with the microzonation maps in CHAPTER SEVEN. It is hoped that they will be valuable for preliminary planning. It should be emphasized that because of the limitations imposed by the lack of any "site-work", any results should be treated in perspective. It is hoped that any area which is considered as an 'earthquake-risk' will be re-examined, with the relevant experimental site-work being performed, before the abandonment of any "potential" site. However, this report will be very useful in delineating areas which are considered to be free of any 'earthquake hazards', due to possible deamplification effects.

The conclusions and future recommendations will be presented in CHAPTER EIGHT. This chapter will essentially list the basic assumptions which have been incorporated in this study and also offer recommendations. We feel that it is essential to include them as they will be useful for future microzonation studies, especially in the central United States. The limitations will convey the true cost/benefit nature of this study so that any future microzonation of Memphis could be extended to include any omissions which occur in this study. The next step from this study would be the production of a comprehensive report which would include new geological data, especially in some of the areas for which we were unable to collect data from previously drilled boreholes due to their proprietary nature.

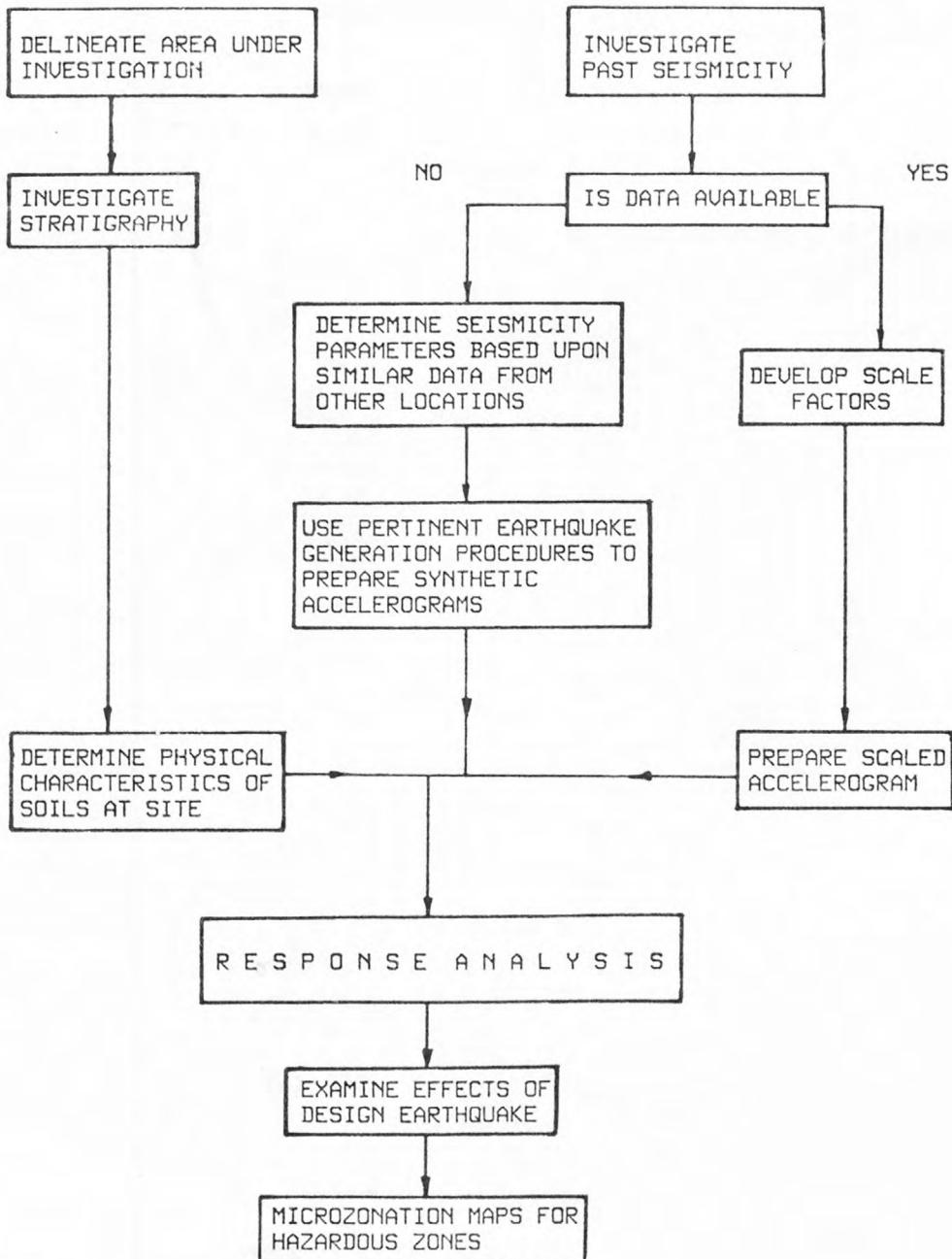


FIG. 1-1, Microzonation Procedures

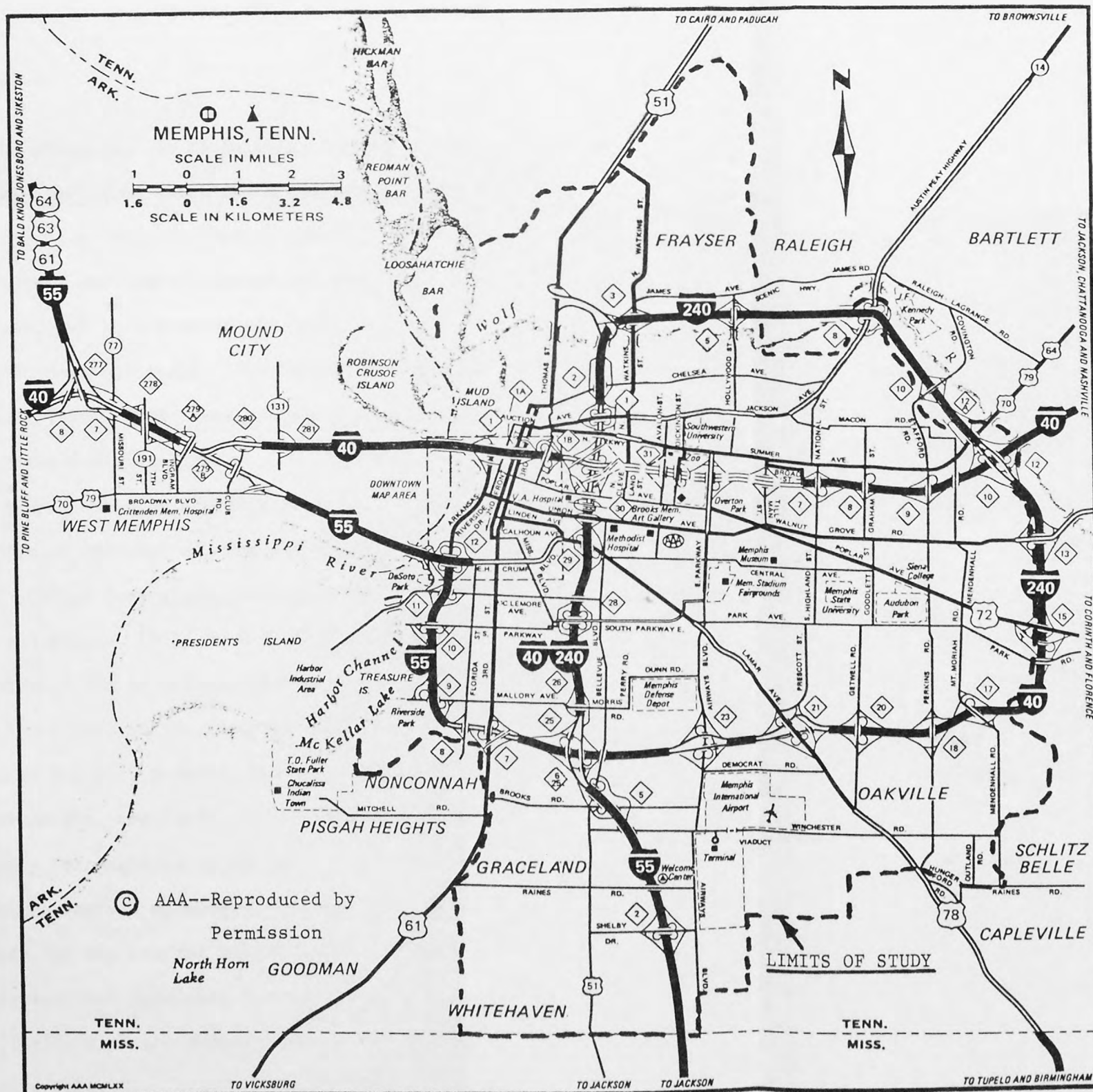


FIG. 1-2, Map of Memphis, Tennessee and vicinity

CHAPTER TWO

Literature Review

The term 'microzonation' is tentatively defined as the variation in response at a series of sites which share the same seismic "exposure". This concept of microzonation, if performed accurately, can be very valuable as it will prevent 'over' and 'under' design for the seismic load cases.

The significance of this concept has been recognized and numerous microzonations have been performed. The classic microzonation of Long Beach (Wiggins and Moran, 1970) is an ideal example as the results were incorporated in the Long Beach building codes.

Before the commencement of any microzonation, it is necessary to determine the seismicity of the region. For the seismicity of the New Madrid region, numerous studies have attempted to delineate the active faults in the region (Stearns and Wilson, 1972; M. & H. Eng. and Memphis State U., 1974). The design earthquakes can be calculated, based on microtremor ($3 < M_b < 5.5$) data (Gutenberg & Richter, 1956). Such an analysis has been performed by Nuttli and Herrmann (1978). However, for any earthquake, one has to assign the necessary parameters, amplitude, predominant frequency and duration, which are necessary for engineering design.

An extensive review was necessary, due generally to the scarcity of strong-motion data for the central United States. Numerous magnitude versus acceleration relationships developed for the western United States were considered (Idriss, 1978) with the summary from the study conducted by M. and H. Eng. and Memphis State U., 1974 being used with some modification. It was also necessary to modify the frequency content relationships which have been determined for western United States (Seed, et al, 1969). The greatest uncertainty, however, is the earthquake duration. Again numerous relationships

exist (Gutenberg and Richter, 1956; Hoffman, 1974; Housner, 1970; Lee and Chan, 1973) but these were found to be unacceptable for the present study. The reason being that the attenuation characteristics are very different east of the Rockies and the dispersion component of surface waves has to be considered. The combination of attenuation and dispersion was considered and durations were selected to define the necessary motions (O. W. Nuttli, 1979, personal communication). With the determination of the earthquake parameters, it is then possible to produce synthetic accelerograms.

These accelerograms were generated artificially (Ruiz and Penzien, 1969; Vanmarcke et al, 1976; Wong and Trifunac, 1978) rather than using available data recorded from Californian earthquakes (California Institute of Technology, Pasadena). For the purpose of this study, only one representative accelerogram will be generated for any one particular earthquake and source location. Donovan and Valera (1972) recommend that numerous synthetic accelerograms be processed to give an 'average' result which may be expected for a response spectrum for any particular site. However, we feel that as the generation process is random, which is analogous to any earthquake-accelerogram, no added accuracy can be obtained by considering a suite of accelerograms representing one particular earthquake. Once input motions have been assessed for the determination of ground response it is necessary to accurately determine variations in the local soils and geology.

The variation of the intensity of shaking according to the local geology has been recognized. MacMurdo (1824) and Wood (1908) were one of the first to infer the variation of intensity depending on whether buildings were situated on outcropping rock or unconsolidated soils. However, we have refined this qualitatively in the past thirty years and now recognize variations according to different types of unconsolidated deposits. Gutenberg

(1957) and Borchardt (1970) in the U. S. and Kanai, et al (1954, 1959) in Japan have also defined the microzonation evidenced during earthquakes. Due to the greater frequency of earthquakes (and perhaps longer historical records), this concept has been widely studied in Japan (Ohsaki, 1969).

Figure 2-1 shows the effects of the 1967 Caracas earthquake. As the city of Caracas lies in a basin containing an extensive accumulation of unconsolidated materials, a 'uniform' variation of depth can be found. From the figure it can be seen that the taller buildings suffered greater damage if they were located above deep deposits. This increasing depth causes an increase in the natural period of the soil layer which then approaches the resonant period of taller structures and results in greater shaking and heavier damage.

The location of a structure within a particular zone is very important as significant variations in design loads need to be considered. From motions recorded during the 1957 San Francisco earthquake, for a typical 10-story building, the maximum base-shear varied by 450% for buildings founded on 300 feet of clay and sand when compared to rock (Seed and Idriss, 1971).

For the city of Memphis, a reliable determination of the soil stratigraphy is imperative. We expect to achieve some reliability throughout Memphis by using borehole data from local sources. In areas where soil data is lacking, we shall infer the soil profile based on the general stratigraphy indicated by Stearns and Wilson (1972) and M. and H. Engineering and Memphis State University (1974). Some microzonation studies (e.g., M. and H. Engineering and Memphis State University, 1974) have presented the final maps based exclusively on the local geology and leave it to the reader to ascribe suitable amplification (or deamplification) factors to the input design amplitudes.

These intensity maps can be prepared directly from the known local geology. Intensity variations, depending on local ground conditions, are added (or subtracted) to a base intensity computed for a specified design earthquake. This technique has been used extensively in the Soviet Union (Barosh, 1969) and has been applied to the State of California (Evernden et al, 1972). However, we feel that this analysis will provide general qualitative guidelines and is not suitable for the determination of engineering design parameters.

With suitable refinement this study will be extended to compute the nature of the surface motions resulting from either a nearby or a distant earthquake. To perform such an analysis, the dynamic soil properties are required. As these soil properties cannot be inferred directly, we shall ascribe these properties based on typical correlations which are available in the pertinent literature (Seed and Idriss, 1970; Hardin and Drnevich, 1970; Richart et al, 1970; Ohsaki and Iwasaki, 1973; Hara et al, 1973; Arango et al, 1978). This is the only approach feasible in the absence of actual laboratory testing, which is outside the scope of this study. From a review of borehole data, it is apparent that basement rock is found at great depths (approximately 850 meters, which is easily outside the scope of most commercial borings). However, as the input earthquake motions have to be assigned at the base of the unconsolidated soils we have to assume a lower boundary to the soil profile. It is readily apparent that the introduction of a limiting boundary will reduce the accuracy of the final study. Unfortunately, this assumption is necessary and its effects are anticipated to be negligible.

Studies by Dezfulian and Seed (1969) have shown that the thickness of the deposit may or may not cause a substantial change in response. However, other studies (Barkan, 1962; Seed, et al, 1969; Duke and Leeds, 1962) show

that the uppermost 35 meters of soil-material has the greatest effect on computed ground motions. Thus we should concentrate on available data in the upper 45 meters and assume "bedrock" below this level. The term "bedrock" is defined as a soil-layer exhibiting a shear-wave velocity in excess of 600 m/s. The range of values anticipated at this depth will be obtained directly from local sources or from data already published (M. and H. Engineering and Memphis State University, 1974).

However, there are significant variations which may occur according to the 'hardness' of this bedrock. Lysmer et al (1971) have shown that spectral frequencies may be very erroneous for a combination of soft-rock above an undeformable bedrock (or bedrock which has a shear-wave velocity of approximately 1800 m/s. For the purpose of our study we shall consider a 'hard' layer of bedrock below a depth of 45 meters. We feel this would reduce any gross errors in the calculation of spectral intensities which might have resulted if increasing hardness of the layers below the soil was to be considered.

When the soil profile with its ascribed dynamic properties has been evaluated, numerous methods are available for performing a response analysis. From a review of literature, the procedures are essentially divided according to whether the soil layers are considered to be horizontal or dipping. For the horizontally layered system the problem can be solved either by the wave-equation approach (Schnabel et al, 1972) or by a lumped mass approach (Penzien et al, 1964; Idriss and Seed, 1968, 1970). However, when the soil-layers are not horizontal or there exist discontinuities (e.g., banks, cliff-faces, etc.), one must utilize the finite element method. There are numerous finite element computational procedures available for general usage (Idriss et al, 1973; Lysmer et al, 1974). The finite-element programs are usually very rigorous in their computational procedures and thus it is felt that

they should be used only in critical cases which have been previously evaluated using one of the other methods.

The computed motions using the more complex finite-element method have, however, not really been "tested" qualitatively for the critical soil-profiles. Thus, because of our 'generalized' approach, it is perhaps not feasible to use such complex methods for this study. In addition, as no correlations between computed and actual ground motions is available, one can perhaps discount these very complex approaches for the purpose of microzonation as they are really more suitable for individual sites where an accurate soil investigation has been conducted.

Thus it is felt that the microzonation zones determined in this study are dependent on the available accuracy used to predict the dynamic soil properties. Once these maps have been prepared, they may be found to be useful for general planning purposes and for specific major plans during their preliminary phases. On the other hand, the designers of proposed structures that are located in critical areas or that will have high occupancies might desire to have a dynamic analysis performed. An extended dynamic analysis would evaluate the site (and structure) on an individual basis as a means of assessing their safety and design.

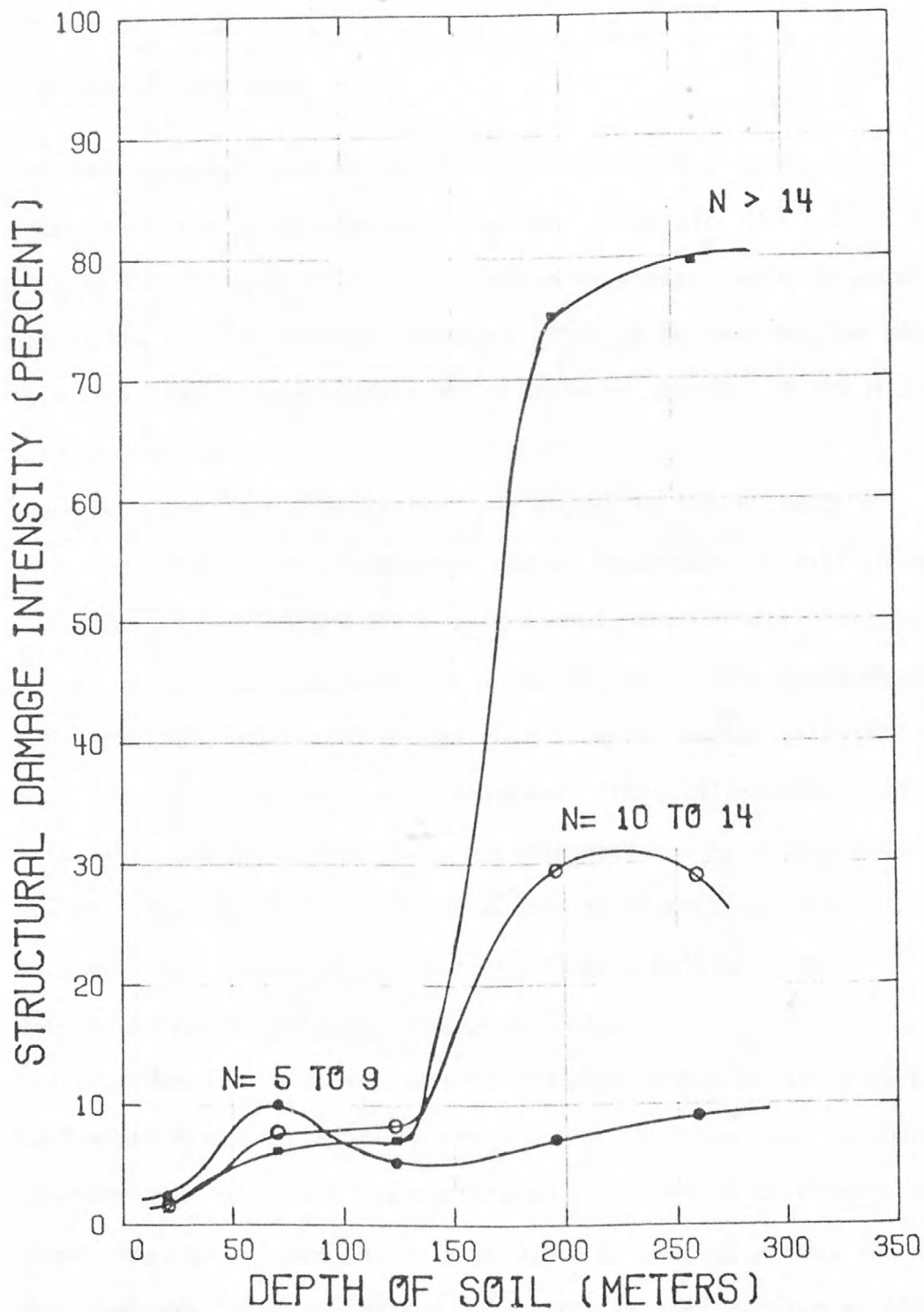


FIG. 2-1, Relationship between structural damage intensity and soil depth in the Caracas earthquake of 1967
(after Seed and Schnabel, 1972)

CHAPTER THREE

Regional Geology and Faulting

It has been accepted that it is faulting which is the direct cause of earthquakes and thus it is advisable that we investigate the extent of the faulting in the vicinity of Memphis. Hence we shall briefly examine the geology of the area as it will, perhaps, allow us to have an idea about the possible locations of earthquakes which might be expected to occur in the vicinity of Memphis.

Whenever movements occur at faults, the ground in the vicinity of the fault vibrates due to the dislocation and an earthquake is said to have occurred. Before an equilibrium state is achieved, further dislocations are likely and are usually manifested in 'aftershocks'. This phenomenon was evidenced by over 1800 individual shocks that occurred during 1811-1812 when the three well known shocks ($m_b > 7.0$) occurred. These aftershocks can cause considerable damage to structures already weakened by a preceding major shock(s). Upon examination of the active fault systems likely to have an influence upon the seismicity of Memphis, it is possible to delineate zones according to their individual characteristics.

The city of Memphis is located within the major geologic division known as the Mississippi Embayment Syncline (Fig. 3-1). This low area is underlain to considerable depth with unconsolidated sediments of Cretaceous and Tertiary ages (Fig. 3-2). Due to the vast depth of sedimentation, the (many) active faults are deeply concealed and only a few exhibit surface manifestations. In areas where surficial evidence is scarce, deep faults have been inferred from seismographic data.

Fig. 3-3 shows the location of earthquake epicenters in the central Mississippi valley between 1974 and 1978. From data presented in this form

it is possible to distinguish between the relative magnitudes of the seismicity in different regions. It is also possible to use this data to "infer" deep faults which are the source (focal) centers of these microearthquakes. Thus it is possible to delineate zones of faulting according to their levels of seismicity and concentration of active faults.

Five main regions of active faulting that can affect the Memphis area have been delineated in the central United States (Nuttli and Herrmann, 1978). These regions are within/or border the Mississippi Embayment. The activity in these regions cannot be isolated to one particular source due to the complexities of the basement structures. However, numerous tectonic models have been hypothesized to explain this phenomenon and thus we are still uncertain about the mechanisms leading to the seismicity in the central U.S.

The most recent models relate the present activity to pre-existing geological features in the earth's crust. The mechanisms causing the deformation may, however, be a complete contrast to the forces which initially produced this geologic feature and are grouped under a common term, 'resurgent tectonics' (Hinze et al, 1980). The models which have been postulated include:

- (i) Crustal Rifting
- (ii) Zones of Weakness and Crustal Boundaries
- (iii) Local Basement Inhomogeneities
- (iv) Thermal Expansion and Contraction
- (v) Isostatic Warping

Figure 3-4 is included to illustrate the postulated mechanisms listed above. Any discussion of these postulated mechanisms will not be attempted in this study due to their inherent complexities. However, the inclusion of these

theories will remind the reader about the lack of surface features (and strong-motion data) which prevents the development of a 'true' mechanistic model of the New Madrid seismicity.

Recent studies in the New Madrid area have indicated the existence of a basement structural feature (Braile et al, 1980). This feature (Fig. 3-5) was interpreted from regional gravity and magnetic maps. It is interesting to note that a major proportion of past earthquake epicenters lie within a close proximity of this feature. This feature, the New Madrid Linear Tectonic Feature (NMLTF), can be traced northeastward from the New Madrid area into Southern Indiana. It has been postulated (Woollard, 1958) that this faulted zone may extend, as a buried "rift", all the way north to the Canadian St. Lawrence fault system which is contrary to the evidence presented by Fox (1970). However, active seismicity is only displayed along part of the NMLTF with the major recent microtremors being located in the vicinity of the north-south trending portion of the basement feature (refer to Fig. 3-5).

It can be seen that the density of the epicenters is reduced away from the NMLTF. The densest zone, which includes the epicenters of the 1811-1812 New Madrid Earthquakes, is expected to be the most active region in the central U.S. Fig. 3-6 illustrates the seismic zones proposed by Nuttli and Herrmann (1978) superimposed onto Fig. 3-5 where the expected seismicity can be visualized with more clarity.

The New Madrid zones A and B along with the Wabash Valley fault zone are related to the NMLTF as their seismicities are a result of this feature. Zone A being the most active region in the central U.S., is expected to be the source of the larger shocks which are likely to prove hazardous for the city of Memphis. However, damaging earthquakes with larger recurrence intervals can also be expected to emanate from the zones surrounding the New

Madrid Zone A. This surrounding seismicity is believed to be the result of the reactivation of ancient faults due to the associated tectonic nature of the nearby NMLTF. From Fig. 3-6, it can be seen that the city of Memphis is located within the highly active New Madrid Zone.

Due to the proximity of the city of Memphis to the active zone suggested by recent microearthquakes, the city of Memphis will be susceptible to earthquake hazards. Any future earthquakes may also reactivate old fault systems such as the Big Creek fault system which may increase the possibilities of damage.

Local Surficial Geology

Because of the location of possible future earthquake epicenters near Memphis, the nature of the surficial soils (up to 50m depth) is of concern. Fig. 3-2 shows a general description of the strata anticipated in the Mississippi Embayment Syncline. The surficial soils directly reflect upon the final results of any type of response analysis and thus due to their significance require more specific information on their properties and classification.

The uppermost soils are clayey silts belonging to, or redeposited from, the loess which constitutes the upper strata over most of Memphis. These strata are usually 6m to 10m thick. They overlie the Lafayette Formation, a zone of terrace sand and gravel deposits of Pleistocene Age with a top stratum of clayey sand or sandy-clay. The grading, density and geotechnical properties of the terrace deposits vary and depths can range from 17m to 20m thick. Under the terrace deposits is the Lower Jackson Formation, a series of old shoreline deposits of Eocene age, consisting of hard clays interspersed with beach deposits of very dense fine sands. Naturally, disconformities are encountered near layer interfaces and thus this description

is very general.

It is anticipated that the redeposited, saturated loess will be susceptible to liquefaction under prolonged vibration and thus the loess must be considered as 'suspect' for any foundations. However, it does not exist in any great thickness, being limited to 3m to 10m, usually found in areas surrounding small rivers or flood-susceptible valleys.

The other major material which may be encountered in the Memphis area is "fill". In general, the fills close to the river, are uncontrolled random rubble fills while the newer ones are well controlled. It is probable that a majority of fills of all types have been placed on relatively soft alluvial soils. Thus if the lower sand is not fully confined, liquefaction may be expected with moderate tremors. Loose sands which extend under 'fills' supporting structures should also be fully investigated.

Table 3-1 lists the various soils which may be anticipated during any sub-soil investigation (M. and H. Engineering and Memphis State University, 1974).

The subject of physical properties pertinent to Response Analysis will be discussed further in Chapter 6 of this report.

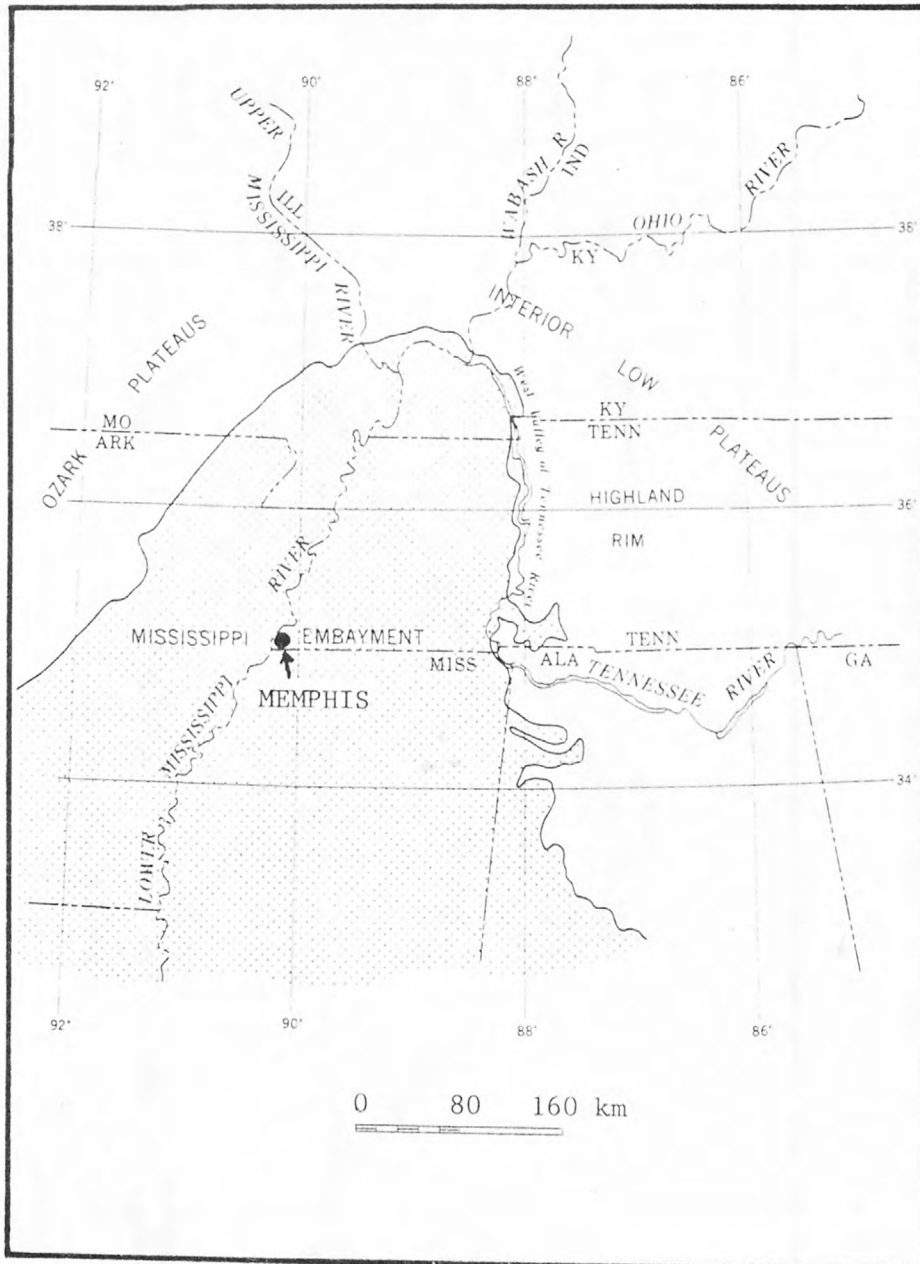


FIG. 3-1, Major geologic provinces

(after Stearns and Wilson, 1972)

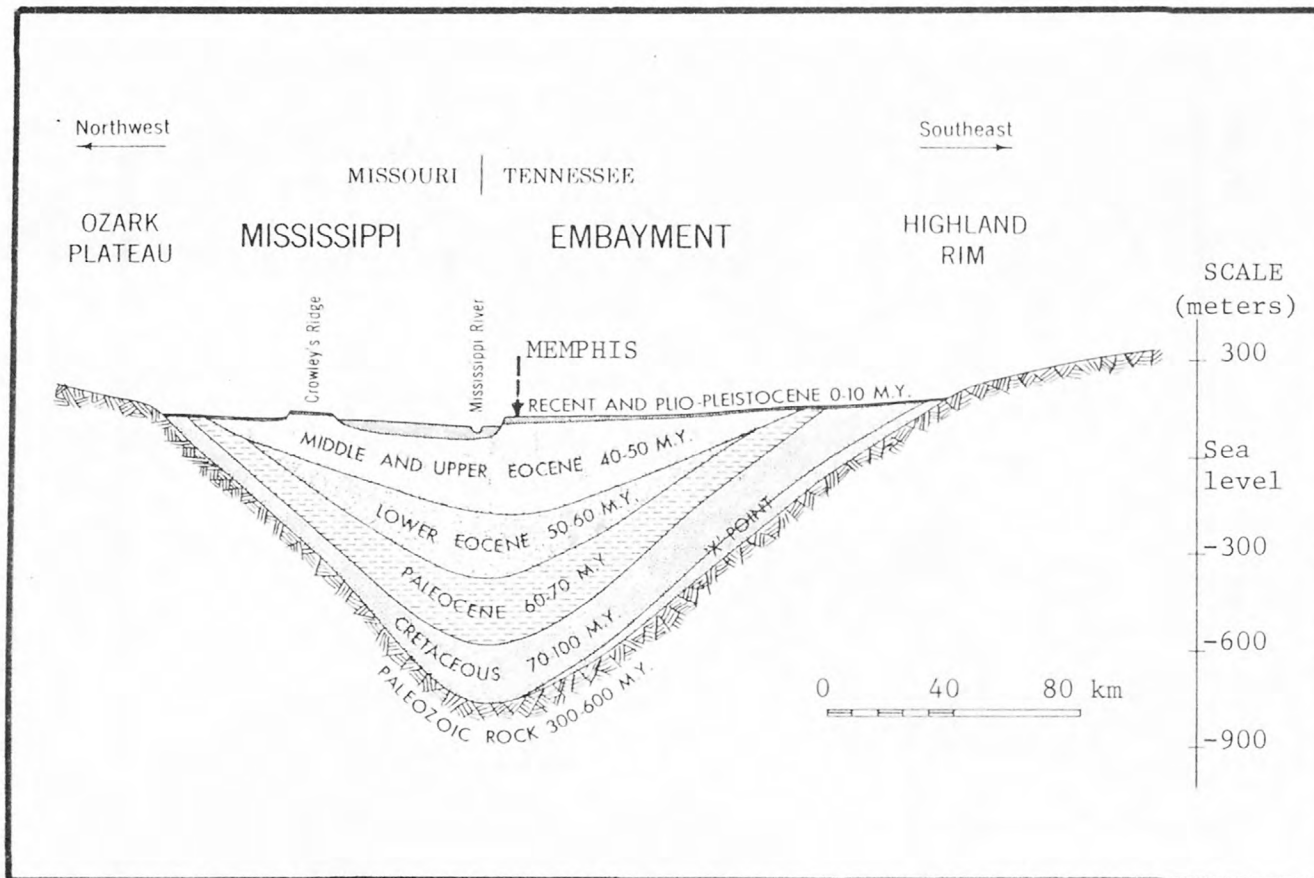


FIG. 3-2, Diagrammatic cross-section of the Mississippi Embayment
(after Stearns and Wilson, 1972)

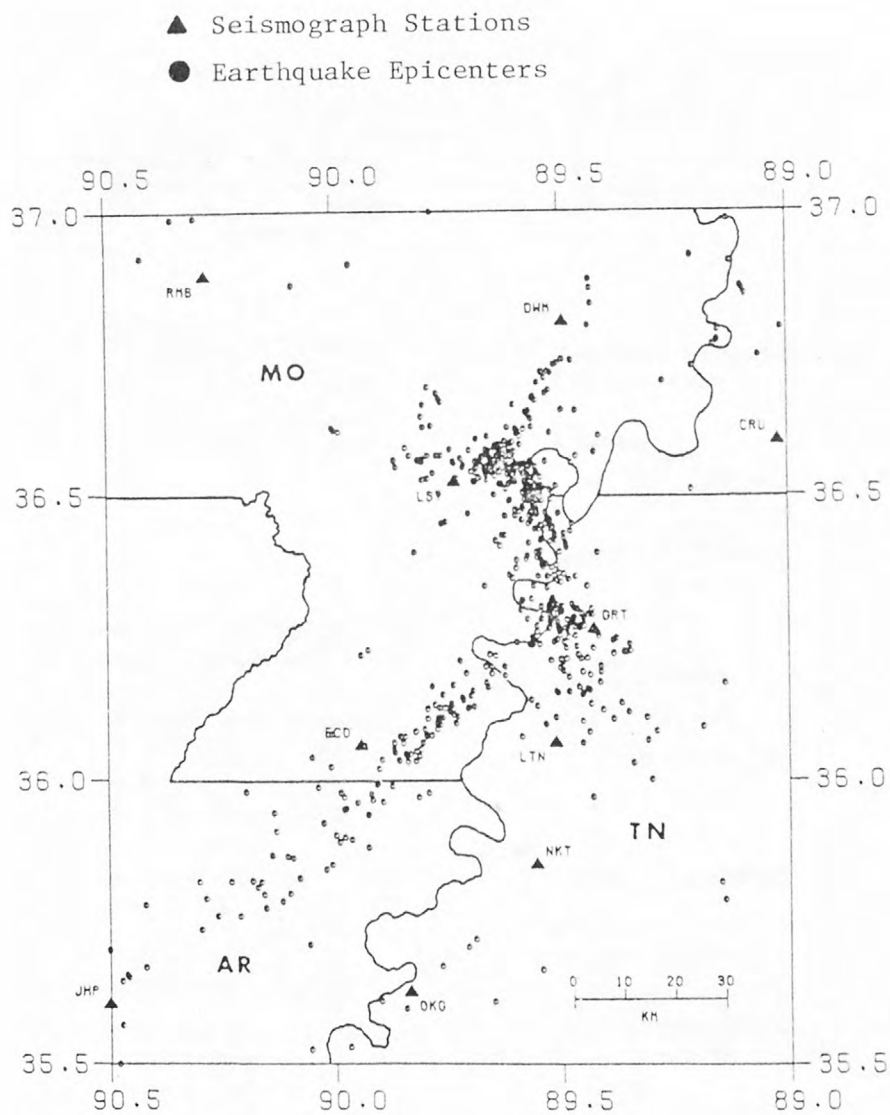
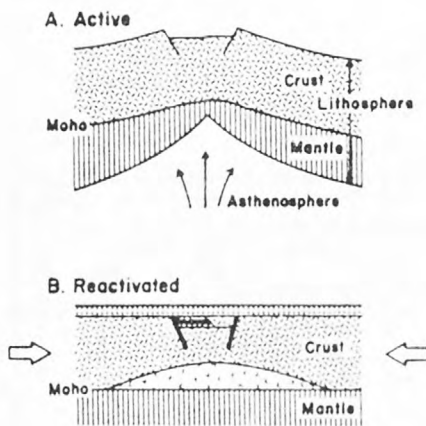


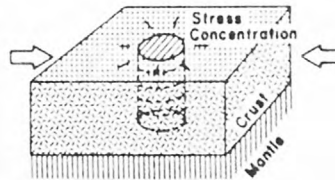
FIG. 3-3, Earthquakes located in the central Mississippi Valley between 1 July, 1974 and 31 March, 1978.

(after Nuttli and Herrmann, 1978)

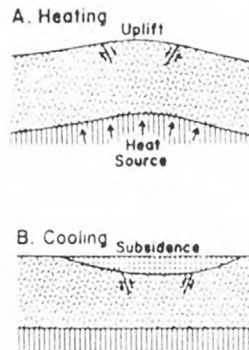
I. CRUSTAL RIFTING



III. LOCAL BASEMENT INHOMOGENITIES

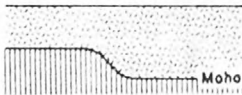


IV. THERMAL EXPANSION AND CONTRACTION

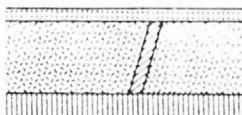


II. ZONES OF WEAKNESS AND CRUSTAL BOUNDARIES

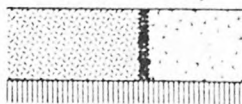
A. Crustal Thickness Variation



B. Ancient Fault Zone



C. Lithologic Boundary



V. ISOSTATIC WARPING

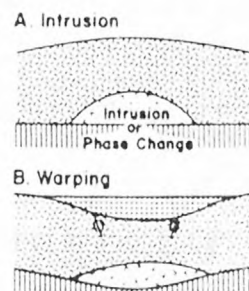


FIG. 3-4, Schematic diagrams of proposed tectonic mechanisms
(after Hinze et al, 1980)

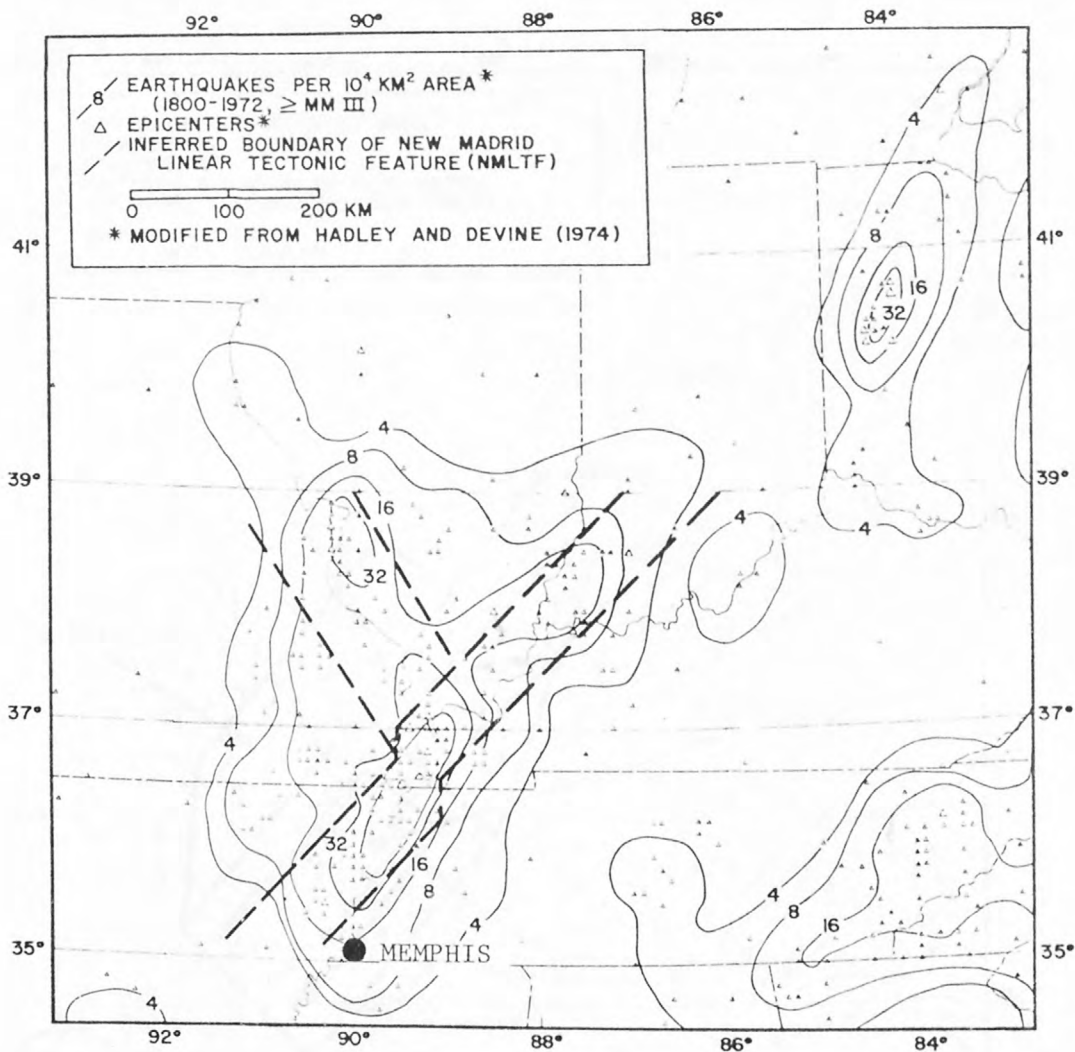


FIG. 3-5, The New Madrid Linear Tectonic Feature (NMLTF) and a gravity map of the New Madrid region

(after Braile et al, 1980)

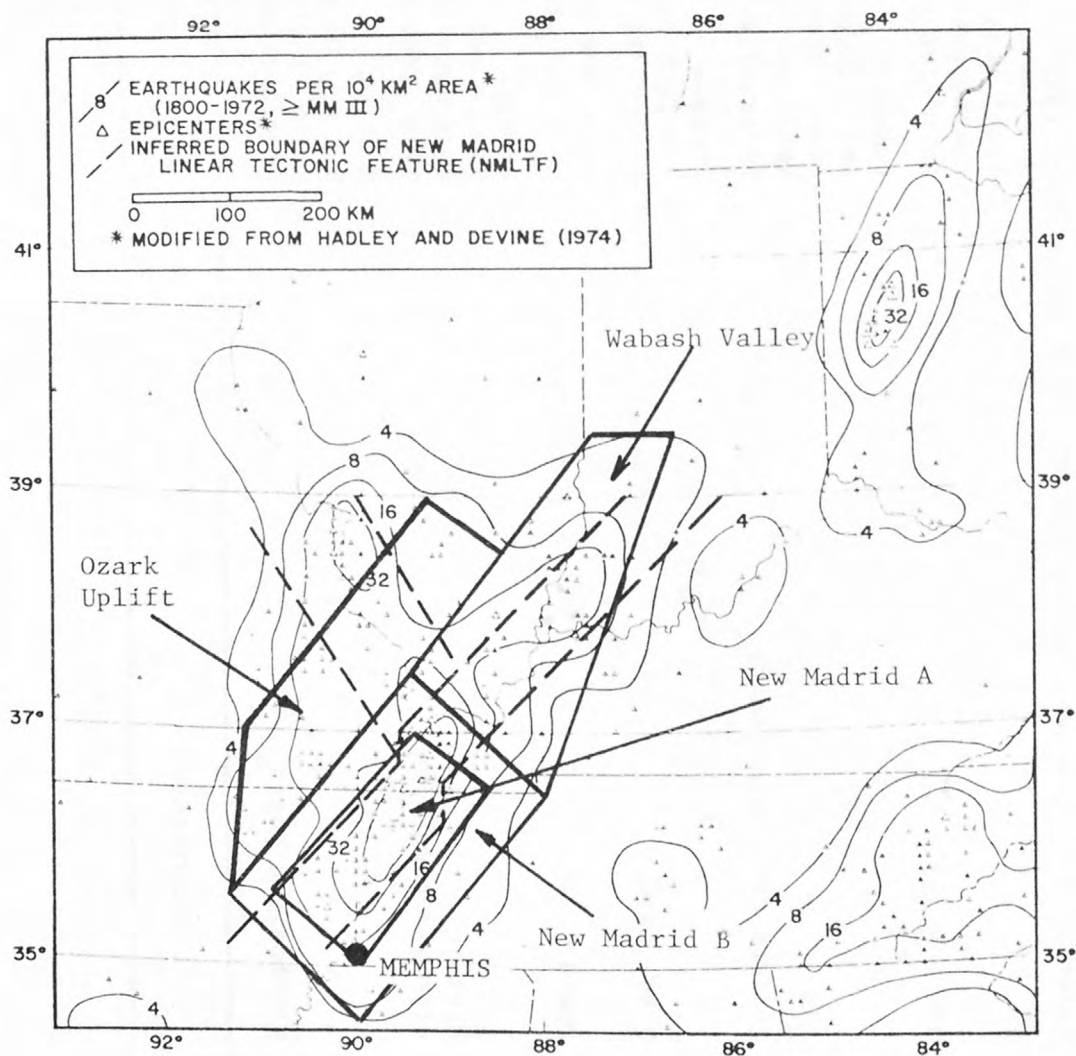


FIG. 3-6, Superposition of seismic zones proposed by Nuttli and Herrmann (1978) over the gravity map of the New Madrid region

TABLE 3-1, STRATIGRAPHIC SECTION

| Series | Subdivision | Range of Thickness - meters | Description |
|-------------|----------------------------|--------------------------------|--|
| Recent | Redeposited Loess | 0 - 10 | Generally water-logged silts or silty clays with a 1 - 2 m. crust in dry weather. |
| | Alluvial sands and gravels | 0 - 6 | Gray, fine to medium sands with occasional gravel, low to medium relative density. |
| Pleistocene | Loess | 0 - 16 | Wind-deposited clayey silts and silty clays. |
| | Sandy clay | 0 - 3 | Very stiff silty clay, possibly old erosional surface. |
| | Terrace sand and gravels | 0 - 60 | Fluviatile medium grained sands and gravels, very dense, generally brown or red, frequently iron-oxide-cemented. |
| Eocene | Jackson (?) | 0 - 150 | Hard, fat clays interbedded toward east and south with fine, very dense white sands. |

(from M. & H. Engineering and Memphis State University, 1974)

CHAPTER FOUR

Design Earthquakes

The New Madrid region has had a very turbulent seismic history ever since the large earthquakes of 1811-12. Nuttli (1973b) estimated the body-wave magnitudes of these three principal earthquakes to be 7.35 (16 December 1811), 7.2 (23 January 1812) and 7.5 (7 February 1812). The body wave magnitude scale is estimated to have a maximum of approximately 7.5. Hence, due to these large earthquakes, one has to consider the surrounding regions as perhaps being similar in destructive potential to the earthquake zones along lithospheric plate boundaries.

The epicenters of all earthquakes with intensities of IX or greater coincide in the region where the North-North East trending New Madrid Linear Tectonic Feature displays a discontinuity in its direction. The tectonic activity has reactivated old fault systems and has thus transferred activity to surrounding zones, which can now be termed as a "faulted-belt" due to the apparent width of the active zone. The "faulted-belt", however, does not display any surface fault-manifestation due to the tremendous depth of deposition in the Mississippi Embayment.

Thus although there are no visible surface features, an inference of the possible source can be made using microearthquake data (refer to Fig. 3-3). Additional studies are, presently, being conducted by the personnel of the USGS using seismic and trenching methods in an attempt to determine the location of recently activated faults. The discovery of any such active faults in the vicinity of Memphis will have a very significant effect on the choice of any design earthquake.

The main factors which have to be taken into account when attempting to adopt a design earthquake are:

- (1) Historical seismicity of area.
- (2) The maximum credible future earthquakes in the region and their location based on available geological data.
- (3) The estimated return period of these events based on local statistical and probability studies.
- (4) Attenuation.

Fig. 4-1 shows the various seismic zones as proposed by Nuttli and Herrmann (1978) in relation to Memphis. These zones have been postulated by considering the distribution of earthquake epicenters and tectonic features in the central United States. It must be emphasized that these boundaries are not to be adhered to rigidly. For locations near these boundaries, one must judge critically the applicability of any choice. Naturally, if the zone of a higher seismicity is adopted, it can only be a conservative estimate. The precise boundaries of these regions are described in Appendix A.

For the analysis of the historical data, we will use the method proposed by Gutenberg and Richter (1954), to develop a relationship between the frequency of occurrence of earthquakes and their magnitudes. This relationship when plotted as number of earthquakes of magnitude (equal to or greater than m_b) vs magnitude on a semi-log plot gives a straight line equation:

$$\text{Log } N_c = a - bm_b$$

where N_c = Number of shocks in a finite area ($100,000 \text{ km}^2$) per unit of time (1 year) having a body wave magnitude equal to or greater than m_b .

a = constant, known as the seismicity index of a region

b = constant, the seismic severity rate

Fig. 4-2 shows the cumulative magnitude-recurrence for the New Madrid regions A and B, excluding the three major shocks of 1811-1812. The ordinate represents the total number of shocks per year equaling (or exceeding) the value m_b , which is the abscissa. Only recent data was used by Nuttli and Herrmann (1978) for calculating the relationship for the small magnitudes. Most of the earlier earthquakes were only ascribed an intensity value; there exists a conversion of Intensity I_o to m_b , which will be applied in this study for the magnitudes considered. Then the straight line can be defined by the following equation:

$$\log N_c = a - 0.92 m_b$$

where $b = 0.92$, as found by Nuttli (1974) for a 140 year set of data for the Central Mississippi Valley. It should be emphasized that the data for the range $m_b < 4.0$ is incomplete because it is unlikely that shocks of that magnitude would have been perceived (and referenced) with any great accuracy during the early part of the 19th century. However, this was corrected for incompleteness by Nuttli before determining the b -value.

Five regions have been selected on the assumption that any design earthquake will be propagated from that particular seismic zone. These areas are:

- (1) New Madrid A: - which is the most active region in the eastern USA and the epicentral zone of the 1811-1812 earthquakes.
- (2) New Madrid B.
- (3) Wabash Valley: - 5 earthquakes with $m_b > 5$ having been recorded during the last 100 years.
- (4) Wichita-Ouachita: - this zone will allow the largest epicentral distance to be selected for attenuation.

- (5) Ozark uplift: - containing the Centralia, Illinois fault zone, near which there were several damaging shocks in the 19th century.

The same data as used by Nuttli and Herrmann (1978) have been used to show the basis of selection of the maximum credible earthquake. However, it is not repeated here as their analysis is beyond the scope of this report. However, Fig. 4-3 is included to portray an impression of the seismicity of the region. Figure 4-4 presents the magnitude-recurrence data for the chosen seismic regions. It should be noted that the data has been reduced proportionally for zones greater than $100,000 \text{ km}^2$ while any area less than $100,000 \text{ km}^2$ required no normalization.

Following Nuttli and Herrmann (1978), the curves are straight-line fits and have been assigned a slope of 0.92 with one-half weight assigned to the largest magnitude data point.

It is assumed that the maximum earthquake that is possible for a region occurs for the 1000 year recurrence. Hence, if the straight line is extrapolated to the point where $N = 0.001$, the value of m_b is then the maximum magnitude. This is shown by the maximum earthquake evolved from Fig. 4-2, an m_b value of 7.35, which is very close to the 1812 earthquake which was rated with an $m_b = 7.5$.

A word about recurrence intervals is important. An earthquake with a 1000 year recurrence means that on the average, it will occur once in 1000 years. The event could occur tomorrow! Likewise it is also possible for two 1000 year events to occur within a year or even a decade, but, on the average the 1000 year event will occur once in one thousand years.

Similarly, values of maximum credible earthquakes have been extrapolated for the five regions under investigation. These values are tabulated in

Table 4-1 which also lists the areas of each region. This design earthquake is of the "floating" type, i.e., the epicenter of the earthquake could be anywhere within the prescribed source region.

All these design earthquakes will have a significant effect on the city of Memphis but it is necessary to differentiate between the close earthquakes and distant earthquakes. It has been shown by Seed (1969) that as an approximation, the natural frequency of a multi-story structure is

$$f_N = 10/N \quad \text{where } N = \text{number of stories}$$

$$f_N = \text{natural frequency of structure}$$

The distant earthquakes will always have a larger percentage of lower frequency waves than an earthquake of the same magnitude nearby. Nuttli and Dwyer (1978) showed that as the coefficients of absorption for the 1HZ and 10HZ waves were 0.0006 km^{-1} and 0.006 km^{-1} , respectively, the lower frequencies will predominate at an increasing rate proportional to the distance from the epicenter. An example of propagation of these long waves was observed at the seismographic station in Berkeley, California during the 1964 Alaskan earthquake (Bolt, 1978). Ground displacements up to 1 cm were measured (at Berkeley) from that distant earthquake but were generally not noticeable as the peaks were spaced every 50 km and the periodic time was 17 seconds. (Hence by Seed's approximation, these waves would have been hazardous to multi-story buildings of approximately 170 stories! No such size structure exists.) However, because of the large epicentral distance involved, the energy in the high-frequency wave was expended by geometrical attenuation and thus no damage occurred in Berkeley.

We will have to examine these distant earthquakes by calculating their intensity on arrival at a location in Memphis from a distant source.

Gupta and Nuttli (1976) investigated the attenuation characteristics of the area and concluded that the dissipation of the intensity can be described by an equation of the form:

$$I_{MM} = -0.4 + 2 m_b - 2.46 \log_{10} R$$

where I_{MM} = Modified Mercalli intensity at a distance R from source

m_b = body-wave magnitude of earthquake at source

R = distance to source in km

Table 4-2 shows the effect of attenuation on the previously chosen design earthquakes. Hence, it is quite feasible that a maximum credible earthquake 130 km away in the Ozark Uplift region, for example, would be more damaging than a small earthquake occurring very close to Memphis in the New Madrid region B.

Thus for design purposes it can be concluded that a range of both earthquake intensity and epicentral distances must be adopted. Hence, the range varies from $I_{MM} = \text{XI} - \text{XII}$ at a distance not greater than 20 km from Memphis to $I_{MM} = \text{V}$ from an earthquake 1200 km away in the Wichita-Ouachita zone.

The ranges perceived are listed in Table 4-3 for any epicentral distance up to 1200 km as it is anticipated that damage is unlikely to result from an $I_{MM} < \text{V}$. This intensity has been chosen arbitrarily because a landslide was reported in Lauderdale County, Tennessee on November 17, 1970 due to a $m_b = 4.6$ ($I_o = \text{V}$) earthquake near Blytheville, Arkansas (Stearns and Wilson, 1972). However, it is possible that the vibrations may only have acted as a triggering agent in causing the failure.

A few recent earthquakes centered in Illinois have had their focal depths at approximately 20 km. However, the focal depths anticipated for the New Madrid region are expected to be approximately 15 km as the presumed

mechanisms are somewhat different (as cited by Hinze et al, 1977). This shallower focal depth also agrees with the results of a prediction-model, which inferred a "deep fault", suggested by Herrmann et al, 1978.

The final criteria necessary for specifying design earthquakes is their "duration". Nuttli (Nuttli, 1973a, 1973b) and Krinitzsky (Krinitzsky, 1972) have reasoned that due to the low-attenuation characteristics of this area, it is feasible that at large epicentral distances the duration could be as much as 1 or 2 minutes. However, due to the lack of strong-motion data for this area, it is not possible to predict any strong motion duration values with confidence. Krinitzsky (1972) had speculated that a duration in the central United States is likely to be four times that experienced in a West Coast earthquake of a similar magnitude.

Fig. 4-5 shows some of the intensity duration relationships which have been developed by Gutenberg and Richter (1956, revised by R. B. Hoffman in 1974), Housner (1970) and Lee and Chan (1973). These relationships have been developed for Californian earthquakes and as the areas east of the Rockies exhibited different attenuation characteristics, these values are perhaps significant only in the vicinity of the epicentral region.

It is very difficult to determine the probable durations for ground motions which have been attenuated from the epicenter to Memphis. Another ambiguity which exists with the intensity-duration relationships is the lack of clear definition of duration.

For this study, duration is defined as the time between the first and last excursion of the absolute value of acceleration above 0.05g, (Bolt, 1973). This procedure is illustrated for a clearer understanding in Chapter Five in conjunction with the selection of significant earthquake parameters for the generation of suitable synthetic accelerograms. Fig. 5-5 shows

tentative relationships between epicentral intensity (I_0) and bracketed duration which has been prepared from western U.S. data. It is felt that this will provide sufficient accuracy for the prediction of the necessary bracketed durations owing to the scarcity of data available.

Thus we shall utilize the Fig. 5-5 relationship for zones in the vicinity of the epicenter. An allowance for the smaller absorption, in the central United States, is also necessary. For increasing epicentral distance we shall find that the durations will increase for the strong motions which are of most interest for this study. Hence, we can now visualize these durations to be of the same magnitude as suggested by Krinitzsky (1972) for epicentral distances of about 200 km. We shall thus find that the combined effect of surface wave dispersion and attenuation will result in a more critical condition in the vicinity of Memphis than that experienced in California.



FIG. 4-1, Seismic source zones of the central United States

(from Nuttli and Herrmann, 1978)

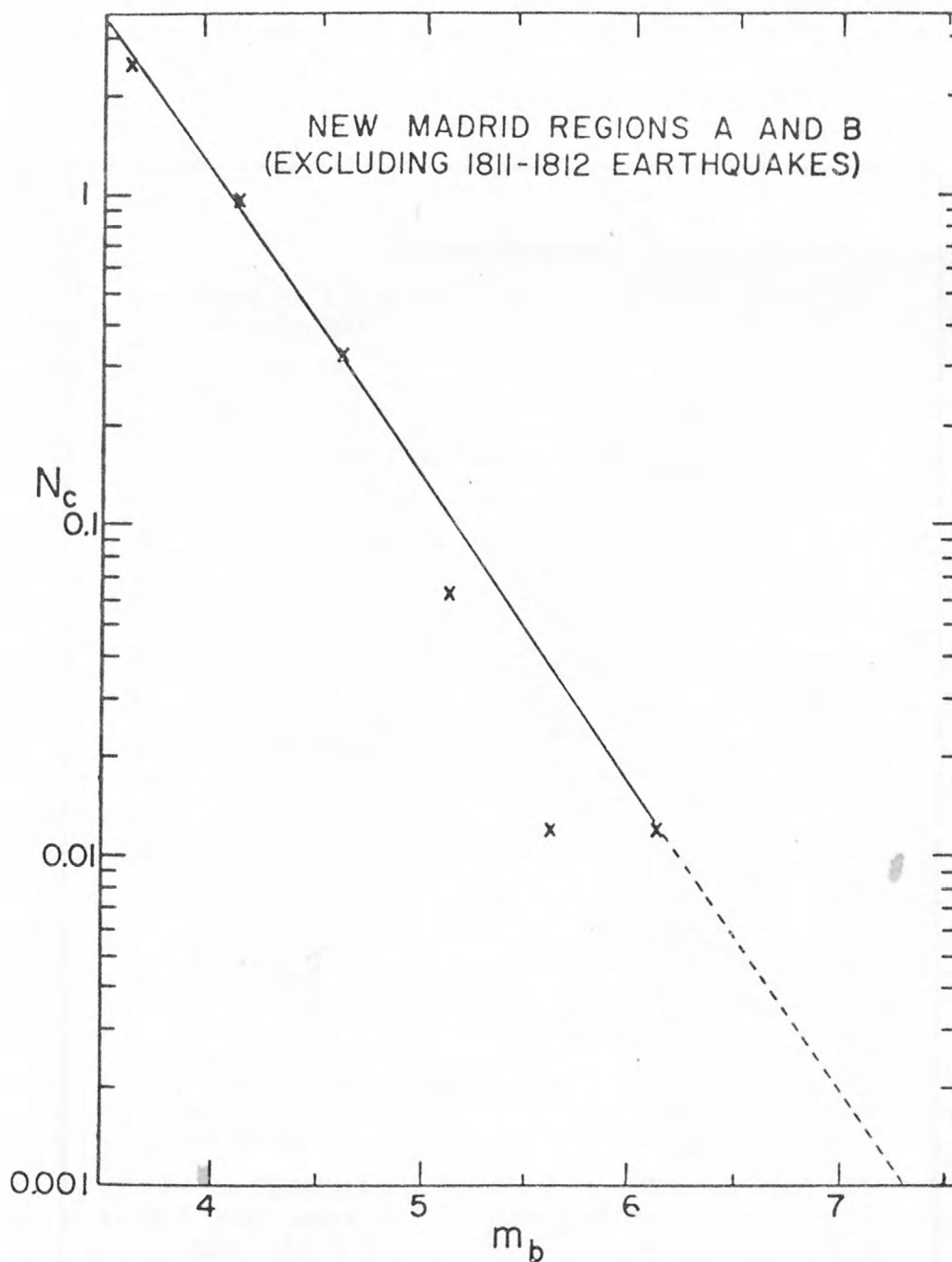


FIG. 4-2, Normalised cumulative magnitude-recurrence curve for New Madrid regions A and B, excluding the 1811-1812 earthquakes.

(from Nuttli and Herrmann, 1978)

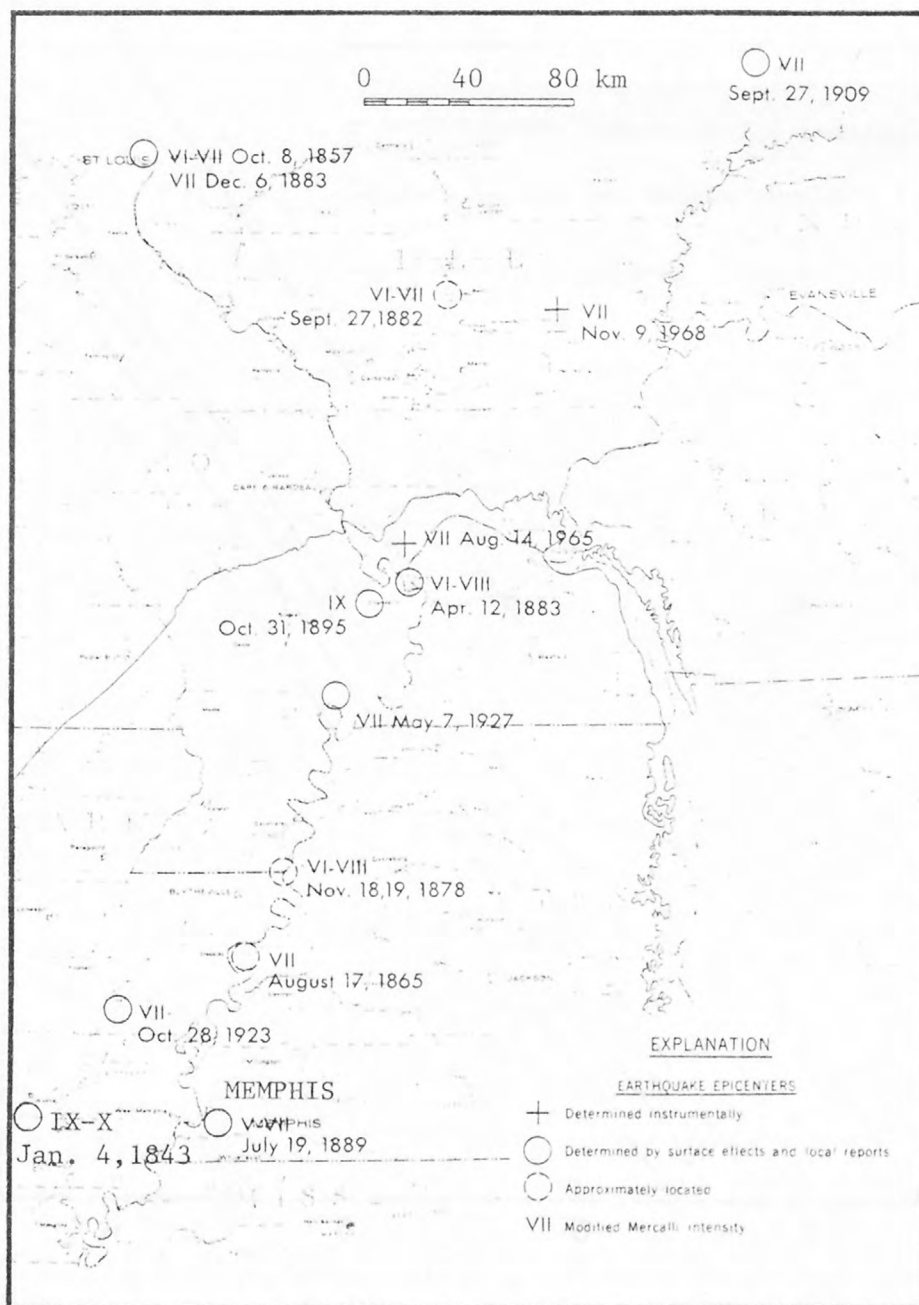


FIG. 4-3, Epicenters of damaging earthquakes (VII or greater) since 1813 (after Stearns and Wilson, 1972)

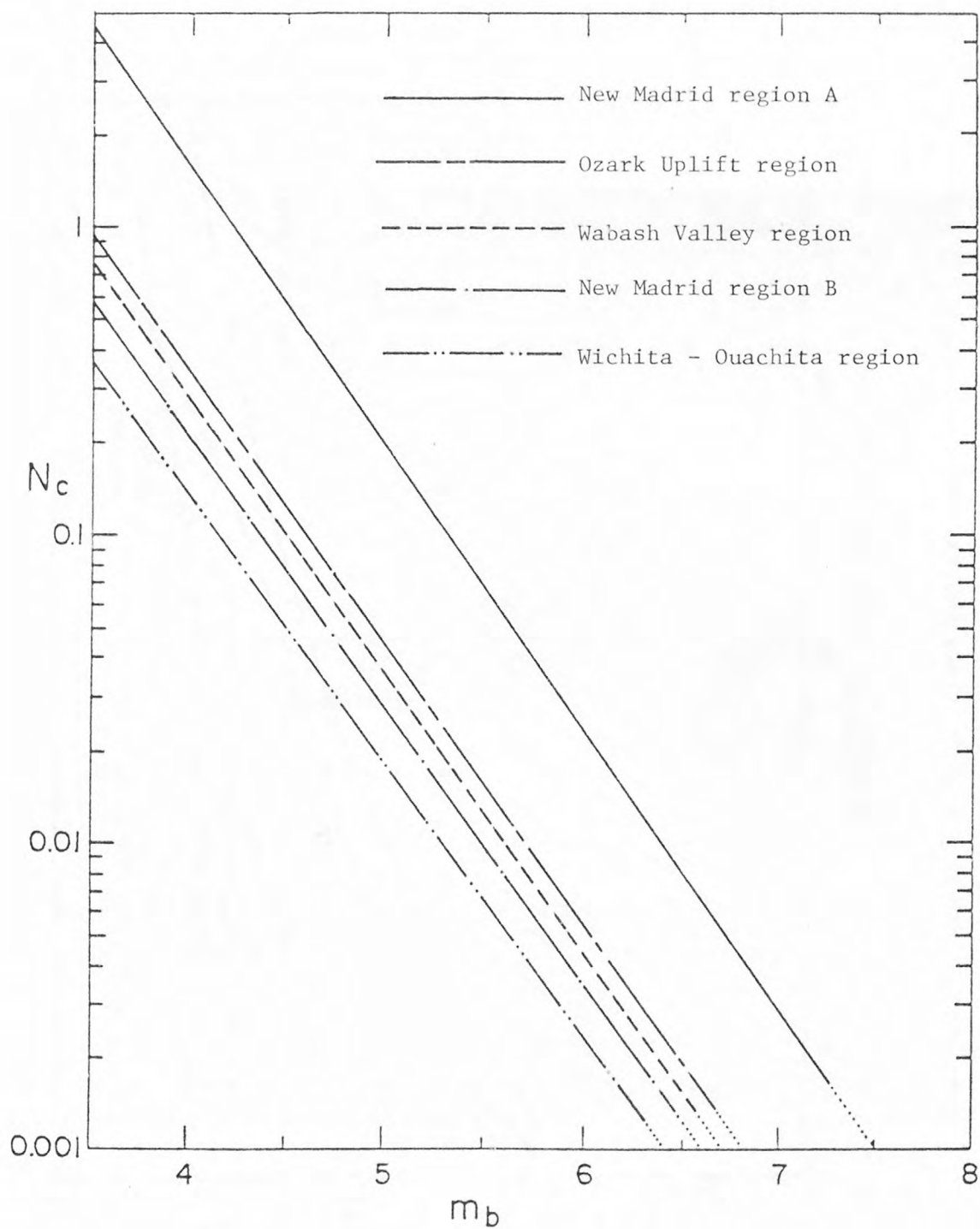


FIG. 4-4, Normalised cumulative magnitude-recurrence curves for regions affecting Memphis (after Nuttli & Herrmann, 1978)

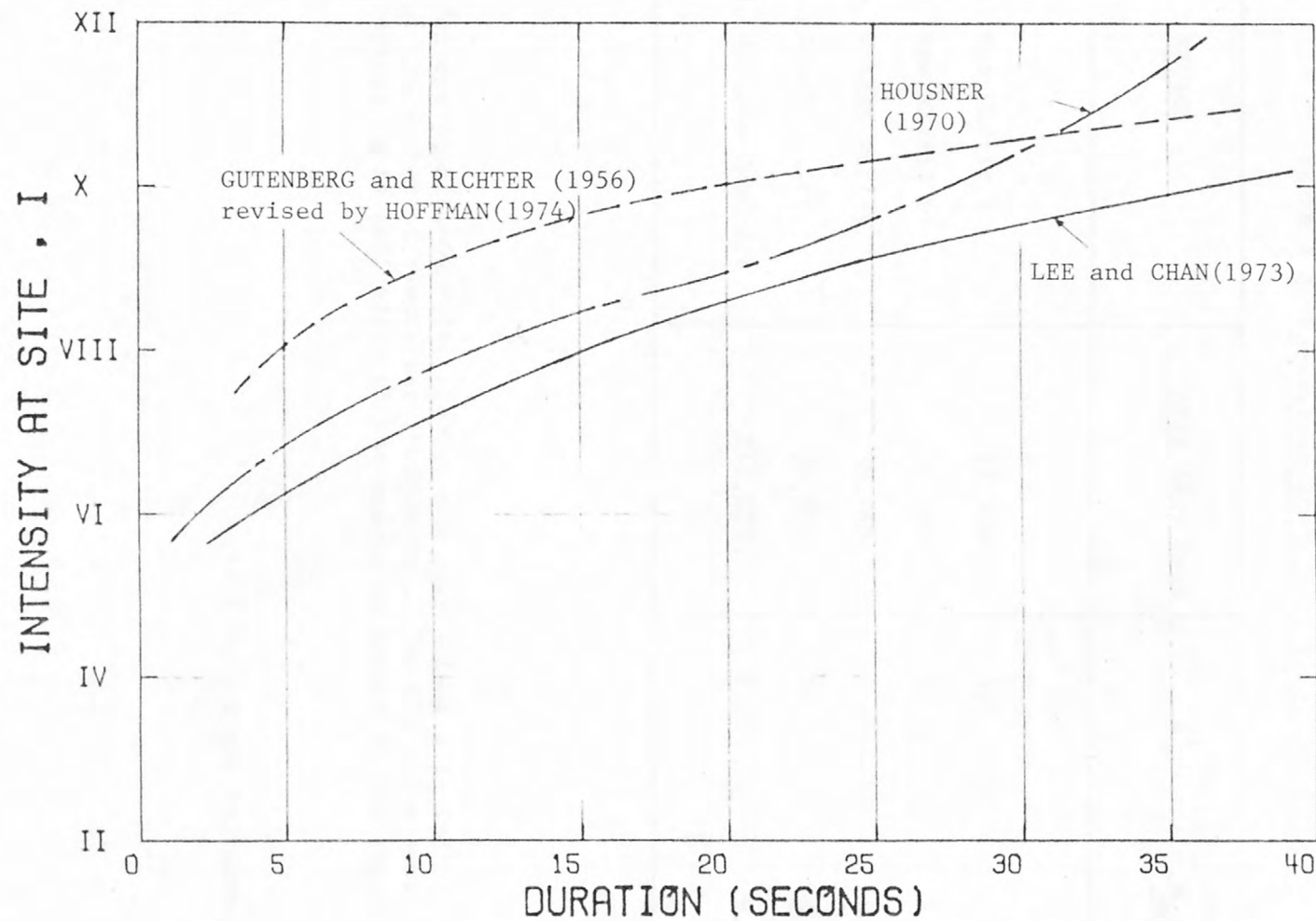


FIG. 4-5, Intensity-Duration relationships

TABLE 4-1, Parameters of Seismic Source Regions

| <u>REGION</u> | <u>AREA</u> (sq. km.) | <u>$\frac{1}{a}$</u> | <u>m_b, max</u> |
|---------------------|-----------------------|---------------------------------|-------------------------------------|
| 1. New Madrid A | 22,506 | 3.9 \pm 0.53 | 7.5 |
| 2. New Madrid B | 27,506 | 2.99 \pm 0.30 | 6.5 |
| 3. Wabash Valley | 39,780 | 3.10 \pm 0.16 | 6.6 |
| 4. Ozark Uplift | 36,557 | 3.19 \pm 0.12 | 6.7 |
| 5. Wichita-Ouachita | 261,829 | 2.79 \pm 0.47 | 6.3 |

¹For the Wichita-Ouachita events, the coefficient a is indicative of the number of events per 100,000 km². For the other source regions, a is indicative of the number of events in the region.

(from Nuttli and Herrmann, 1978)

TABLE 4-2, Attenuation Effects

| REGION | Max. m_b AT EPICENTER | EPICENTRAL DISTANCE (km.) | | ATTENUATED I_{MM} AT MEMPHIS | |
|------------------------|----------------------------|------------------------------|-----|--------------------------------|--------|
| | | Max | Min | Max | Min |
| 1. NEW MADRID REGION A | 7.5 | 230 | 0 | XI-XII | IX |
| 2. NEW MADRID REGION B | 6.5 | 280 | 0 | IX-X | VII |
| 3. WABASH VALLEY | 6.6 | 600 | 240 | VII | VI |
| 4. OZARK UPLIFT | 6.7 | 440 | 130 | VIII | VI-VII |
| 5. OUACHITA-WICHITA | 6.3 | 1200 | 0 | IX | V |

TABLE 4-3 Intensities and Durations

| REGION | BRACKETED DURATION in vicinity of epicenter (seconds) | MAX. INTENSITY I_o |
|------------------------|--|-------------------------|
| 1. New Madrid Region A | 30 | XI-XII |
| 2. New Madrid Region B | 23 | IX-X |
| 3. Wabash Valley | 25 | IX-X |
| 4. Ozark Uplift | 27 | IX-X |
| 5. Ouachita-Wichita | 17 | IX |

CHAPTER FIVE

Artificial Accelerograms

In the last chapter, design earthquakes from various seismic zones were selected. However, one cannot use these magnitudes and durations directly for detailed engineering analyses. An earthquake shock usually produces an almost infinite number of directional forces consisting of oscillations made up of numerous frequencies. Although it is possible to 'judge' an overall effect directly from the magnitude, the method has numerous limitations and is thus very approximate for the purpose of microzonation.

We shall thus have to evaluate the content of an earthquake shock and then use the larger components of motion. This motion can then be used directly for structural analysis, once its limits have been clearly defined. An accelerogram is generally used for engineering purposes as the accelerations can be directly transformed into forces which directly influence the structure under investigation.

The best method for obtaining an accelerogram for a particular location is by using a previously recorded accelerogram for a very similar site and region of the United States. Once a true recording of an event is available, it is possible to scale the amplitude to match any design amplitudes. It is not advisable to lengthen the duration of an accelerogram by stretching it in time as the frequency content will be altered significantly. However, one cannot always rely on actual recordings due to their scarcity and thus must resort to producing synthetic accelerograms which attempt to simulate the earthquake source mechanism and the propagation characteristics.

For the purpose of this report we shall be using synthetic accelerograms exclusively due to the lack of strong motion data for the central

United States. We shall use a computer program, "Artificial Generation of Earthquake Accelerograms" (Ruiz and Penzien, 1969) developed at the University of California, Berkeley. A short explanation of the program's theory which was a part of a larger study, is listed in Appendix B.

There are numerous variations in the shapes and sizes of these accelerograms and thus we shall attempt to create only the significant characteristics which are necessary for analysis. It should be noted that the accelerograms are for the peak component of horizontal motion at bedrock-level. The amplification factor due to surficial soils is discussed in Chapter Seven of this report.

The synthetic accelerograms produced will be manipulated so as to display the following features:

- (i) Peak acceleration which corresponds to the intensity at site, after having been attenuated from the source,
- (ii) Strong motion duration,
- (iii) Predominant frequency, reflecting the effects of greater attenuation of the higher frequencies contained in the seismogram plus effects of magnitude.

All the above parameters are very significant properties of any seismogram and thus it is necessary to further elaborate on their exact "roles" in the preparation of an acceleration time-history.

Peak Acceleration

The 1811-1812 earthquakes were perceived over a much larger area (nearly 2,000,000 sq. mile) than for comparable Californian earthquakes. From Fig. 5-1, it is evident that any future earthquake of this size would also have the same area of perception. The normal perception threshold for human

response is approximately 0.1% g which is not of a sufficient magnitude to cause any damage. Thus areas susceptible to damage are likely to be much smaller than the vast area where motion is merely "perceived". It is also evident that the attenuation of acceleration is not as rapid as that which has been observed in other regions of the world (see Fig. 5-2). Although the attenuation is smaller, the accelerations near the epicentral region are expected to be similar to western U.S. However, due to the lack of strong motion data, it is very difficult to predict accelerations away from the epicenter as no relationships have ever been determined.

Thus we shall use Fig. 5-3 (M. and H. Engineering and Memphis State University, 1974) to a certain extent, especially the limiting minimum and maximum ranges suggested. The new line introduced into the existing envelope shows a trend of 'minimum' values at the higher intensities. It is the general opinion, which has evolved from the review of the pertinent literature, that anticipated accelerations will be equal to or slightly smaller than those of comparable earthquakes occurring in California. Thus we shall endeavor to abstract values from Fig. 5-3 based almost exclusively on judgment and expectation as it is almost impossible to base it on any available data.

Strong Motion Duration

Literature regarding this critical parameter provides conflicting views. (Krinitzsky, 1972 and Nuttli, 1973a, 1973b). It has been suggested that durations may be as great as four times those of equivalent earthquakes in California (Krinitzsky, 1972). However, due to the lack of a universal definition of 'strong motion duration' it is evident that one cannot use these "empirical" factors. During the 1811-1812 earthquakes, it was reported that ground-shaking was perceived for two minutes (Penick, 1976; Fuller, 1912). Hence,

if the accelerations over a long period were only at the level of the perception threshold, it is anticipated that no damage would be caused by the "tail-end" of the accelerogram. As most of the Californian earthquakes are usually defined in terms of "Bracketed Duration" (time for which accelerations are greater than 5% g), we should, perhaps, try to evolve similar periods of duration.

The Taft recording for the Southern California earthquake of July 21st, 1952 is shown in Fig. 5-4 as an example of this definition. It can be seen that this major earthquake ($M_L = 7.7$; $m_b = 6.8$) has a bracketed duration of approximately nineteen seconds. As the Taft earthquake represents one of the largest earthquakes, it is unlikely that the New Madrid earthquakes of a smaller magnitude would produce larger durations close to the epicenter.

Data collected from past western U.S. events for this bracketed duration are shown in Fig. 5-5 (see Table 5-1). A tentative line has been drawn, by visual judgment, to suggest a possible relationship between this duration and magnitude of earthquake. We shall use this figure to obtain relevant durations for our design earthquakes established earlier.

Although this duration only really defines the strong motion, it is necessary to develop a proportional rate of 'build-up' and also a decay period. We shall try to have a larger rise and decay time with epicentral distance. This would then result in seismograms which indicate a slightly longer "perceived-duration".

Predominant Frequency

This parameter is the one that will be most difficult to simulate in our synthetic seismograms. For Eastern earthquakes, higher frequencies are encountered at similar epicentral distances than for comparable Californian events. It has been shown that the attenuation of high frequency waves is

smaller in the central region of the U.S. when compared with Californian events (Nuttli and Dwyer, 1978). Thus our synthetic seismograms should contain a representative sample of all frequencies between 1 and 10 Hz.

The predominant frequencies causing damage usually have the following values (Page et al, 1975): -

| | | |
|-------|--------------|----------------|
| (i) | Acceleration | 2 to 10 Hz |
| (ii) | Velocity | 0.5 to 2.0 Hz |
| (iii) | Displacement | 0.06 to 0.5 Hz |

These tentative values are based upon California data and would be expected to occur within 40 km of the epicenter. As the expected attenuation of intensity is lower (see Fig. 5-2) one would expect higher frequencies to persist over a longer distance. Fig. 5-6 shows a relationship between predominant period and epicentral distance for various Richter Magnitudes. As this is based on data from Californian events (Seed, et al, 1969), it can be used only as an approximate guide to predict periods for the area under investigation.

Fig. 5-7 has been prepared for the Central United States and is essentially an improvement of Fig. 5-6. Due to the lower absorption values for the higher frequencies, curves from Fig. 5-6 have been displaced by 20 km to include an area of 60 km diameter. This is the area that would experience the highest frequencies (or lowest period). It is felt that this is most indicative of the nature of the 'shocks' expected in the vicinity of Memphis.

Hence we shall use Figs. 5-3, 5-5 and 5-7 for ascribing the significant parameters for our Synthetic Accelerograms. It is expected that these will reflect the anticipated characteristics of most strong motion accelerograms in the central United States.

Generation of Synthetic Accelerograms

It is necessary to generate an ensemble of time-histories which include the necessary parameters previously discussed to enable a response analysis to be performed. These time-histories are essentially the product of a stochastic model which utilizes random numbers to generate the accelerogram. Hence it is conceivable there are an infinite number of combinations which could provide the necessary significant earthquake parameters which are required. Thus we will generate sufficient accelerograms to enable us to proceed with the response analysis and obtain results which would be anticipated from earthquakes with epicenters within 250 km of Memphis.

The earthquake, $I_0 = IX$, which represents a 100 year recurrence frequency for the New Madrid zone A has been adopted. It is hoped that this would provide the best indication of the significant ground motions expected in Memphis. At the same time different locations of the same event can be considered to give a 'spectrum' of the response values due to a variation in the earthquake design parameters with epicentral distance. The adopted Design Earthquake was located at 50, 100 and 200 km from Memphis and its effects attenuated to the city, using the relationship shown in Fig. 5-8. These characteristics are listed in Table 5-2. The data necessary for the preparation of these time-histories is discussed (and also listed) in Appendix B.

As there are an infinite number of locations and intensities which could be considered, it is left for future studies to consider any new significant characteristics which may have been omitted in this study.

Figures 5-9, 5-10 and 5-11 show the time-histories generated for events at three epicentral distances: 50, 100, 200 km. The acceleration time-history was generated initially with the velocity and displacement time-histories being produced by integration. However, a baseline correction was made to ensure that the velocity at the end of the record attained a zero-value.

Figures 5-12 through 5-14 are the Response Spectra for these representative motions for 2% structural damping. These have been included to show the frequency content of the accelerograms generated and also the frequencies at which Peak Response is encountered. It will be noticed that these peak frequencies do not entirely correspond to the parameters listed in Table 5-2. As stated earlier, this parameter is one of the most difficult to simulate. Due to the complex computational procedures involved one cannot produce ideal accelerograms with predicted predominant frequencies.

These accelerograms will be utilized in the next part of the study which will involve the response analysis in the latter part of this report.

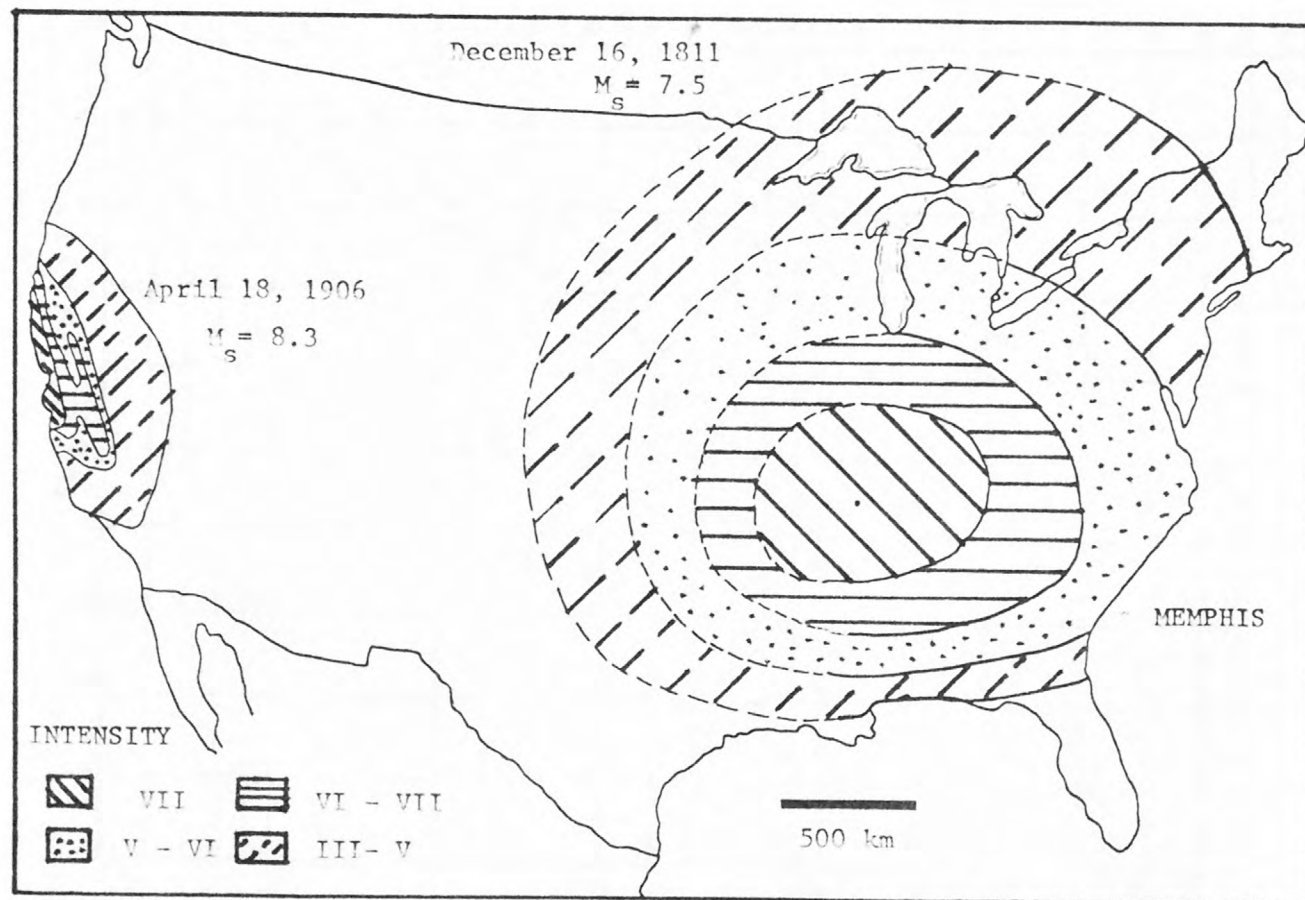


FIG. 5-1, Variations of Modified Mercalli intensity in affected areas for central and western U.S. (after Nuttli, 1972)

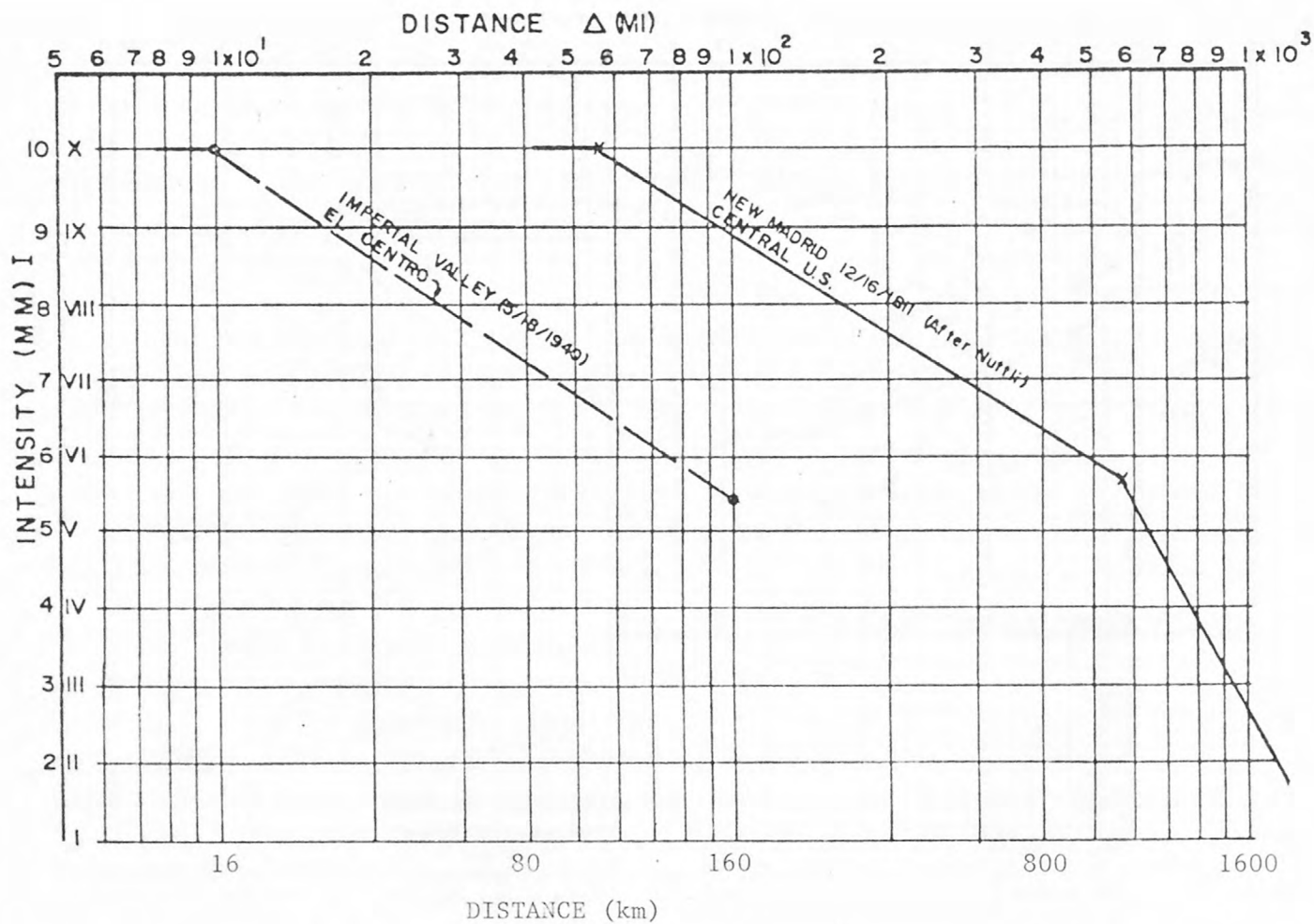


FIG. 5-2, Comparison of attenuation for California and central United States.
(from M. & H. Engineering and Memphis State U., 1974)

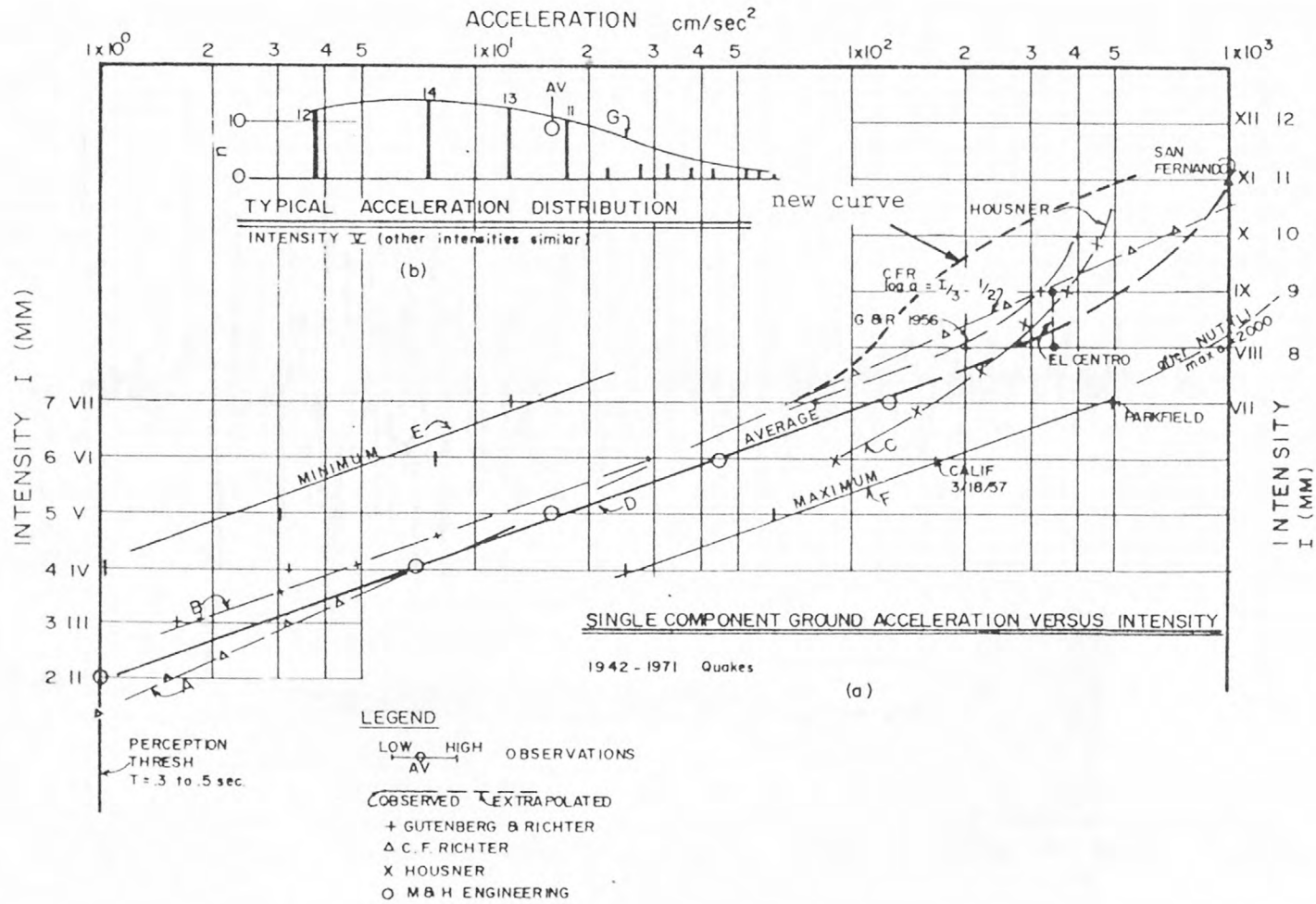
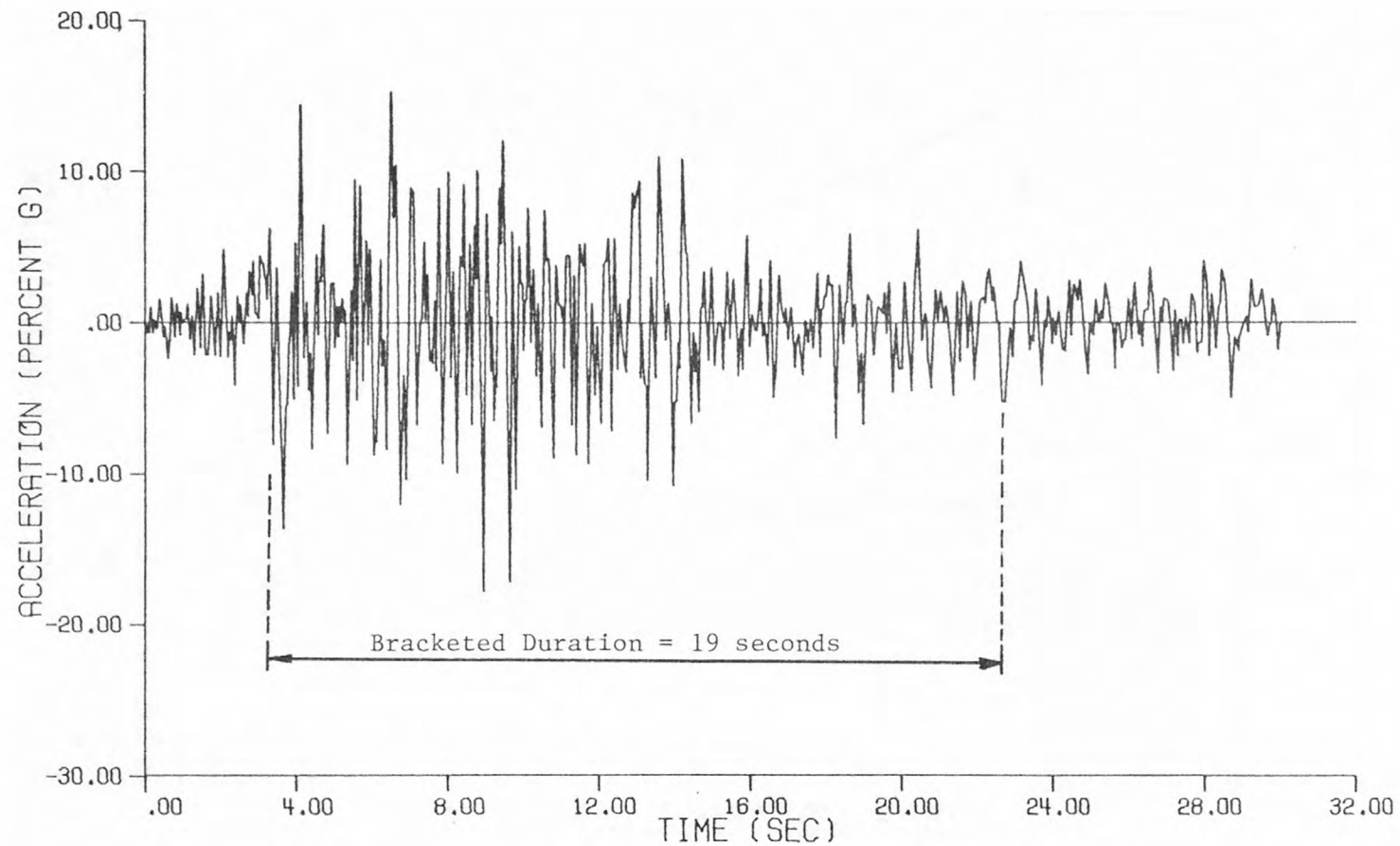


FIG. 5-3, Acceleration versus Intensity (after M. & H. Engineering and Memphis State U., 1974)



KERN COUNTY EARTHQUAKE, 1952 N21E

FIG. 5-4, Bracketed Duration - an illustrative example

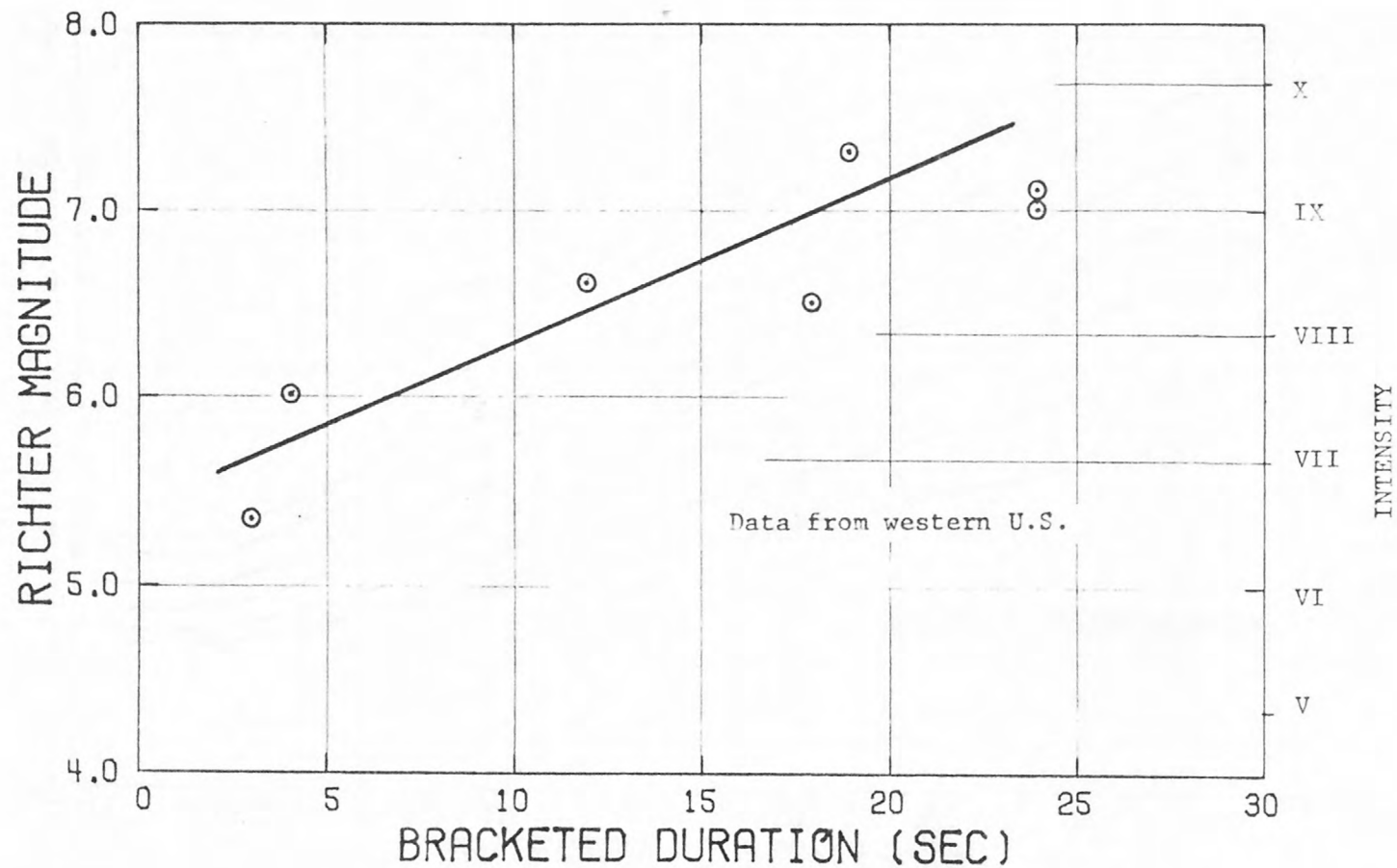


FIG. 5-5, Magnitude - Duration relationship in the vicinity of the epicenter.

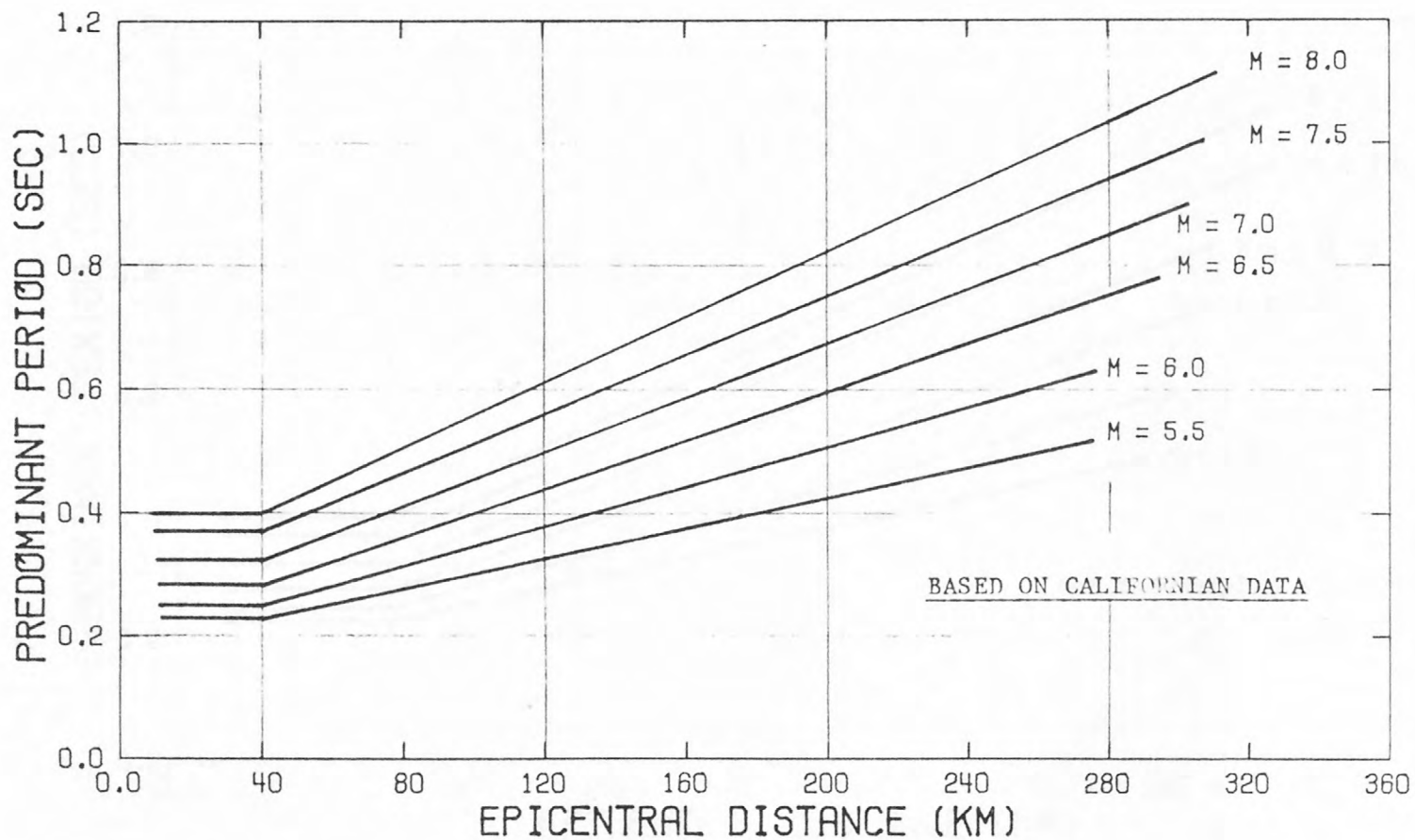


FIG. 5-6, Predominant periods for maximum accelerations for bedrock. (after Seed et al, 1969)

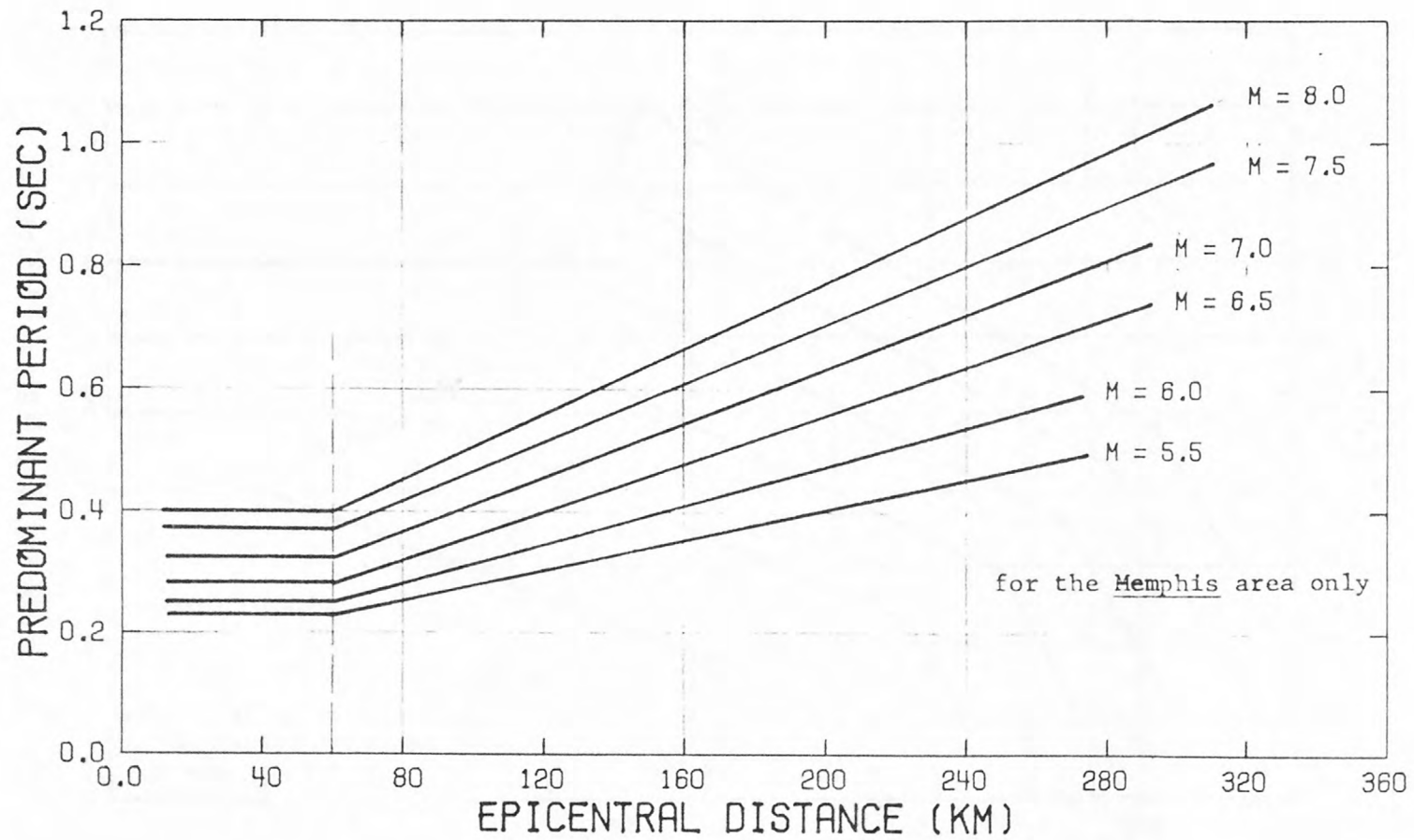


FIG. 5-7, Proposed predominant periods for maximum accelerations for bedrock

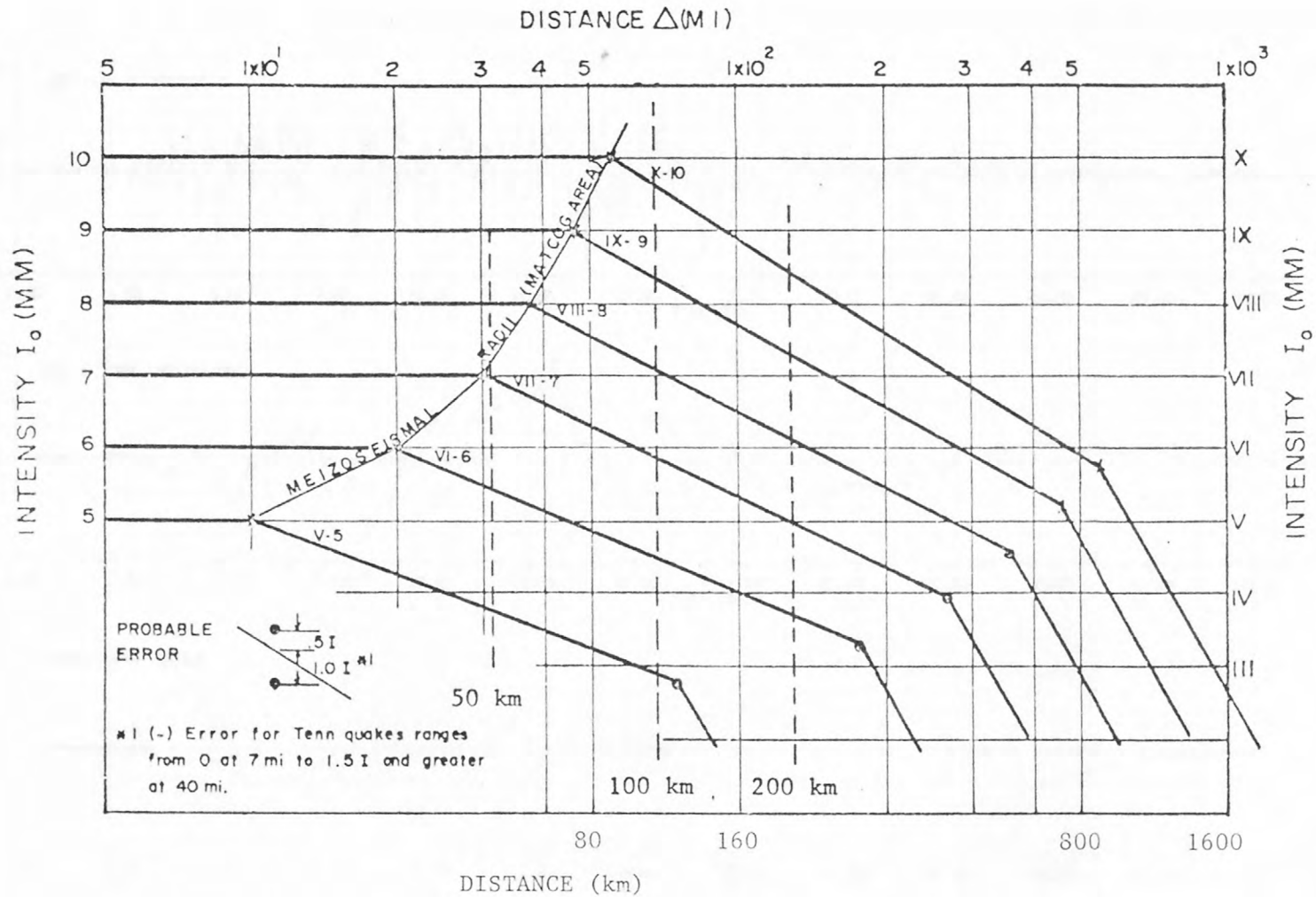


FIG. 5-8, Intensity versus Distance relationships

(from M. & H. Engineering and Memphis State U., 1974)

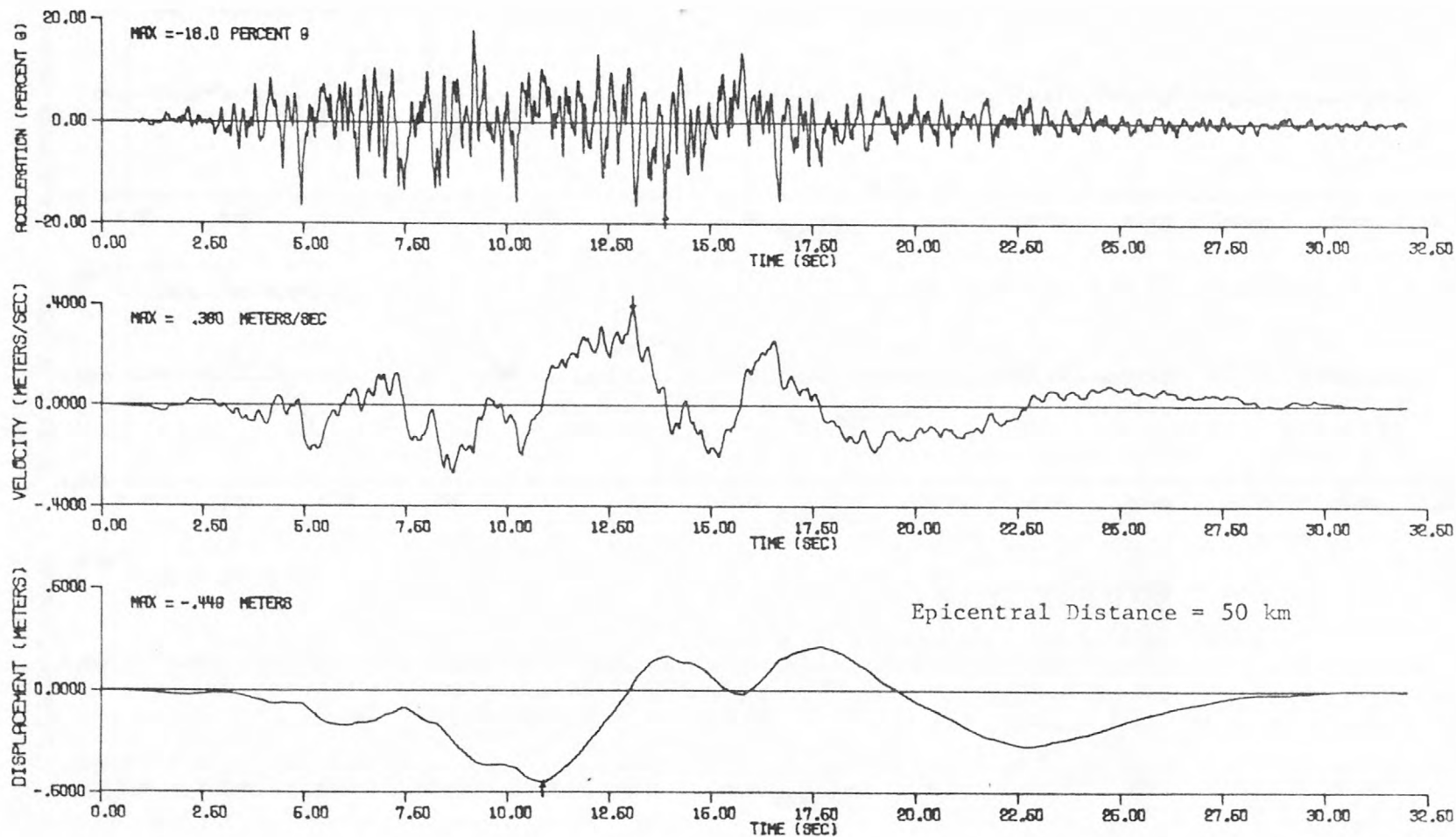


FIG. 5-2, Synthetic design Earthquake A1, for $I_0 = IX$

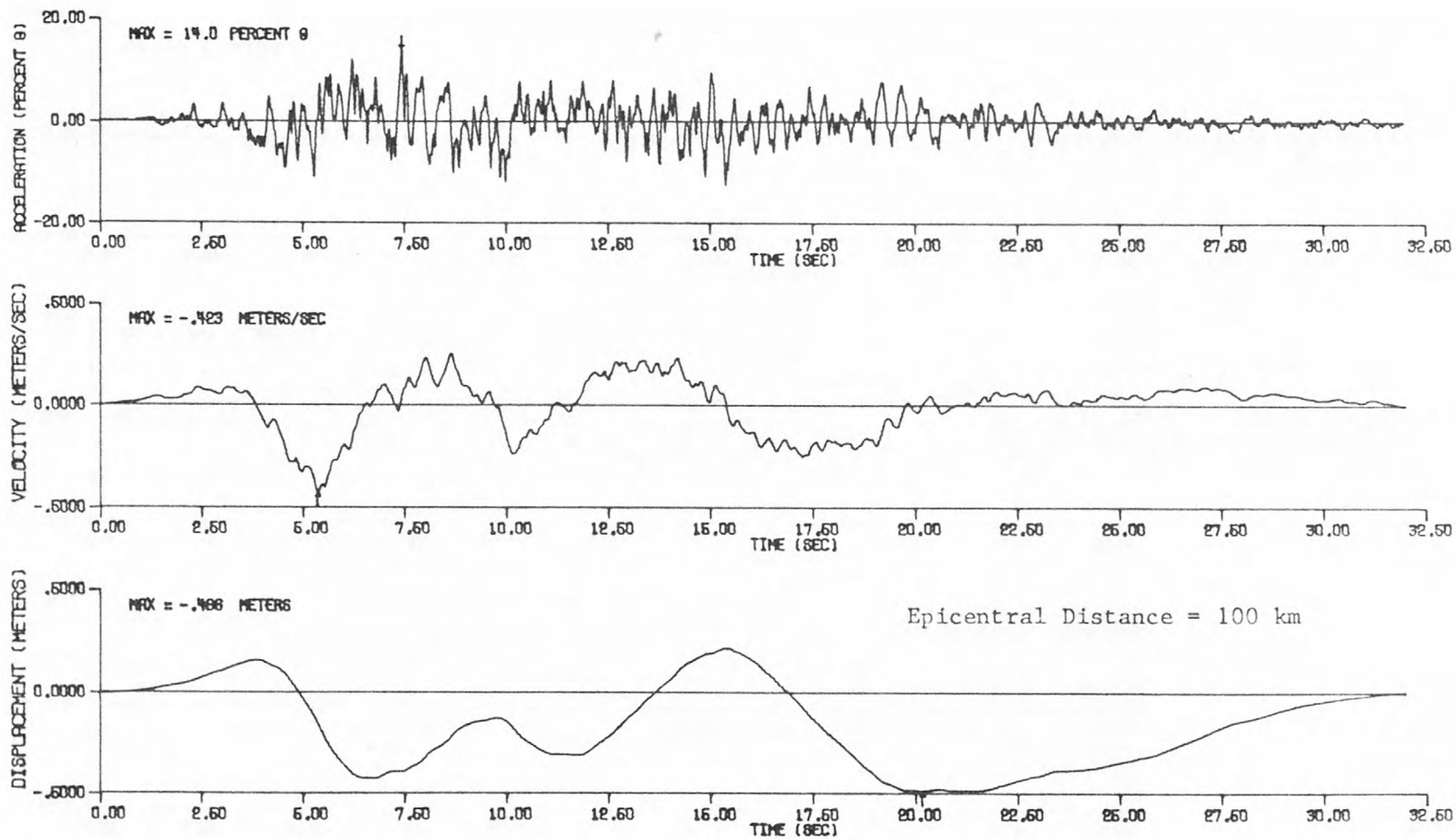


FIG. 5-10, Synthetic design Earthquake A2, for $I_o = \text{VIII-IX}$

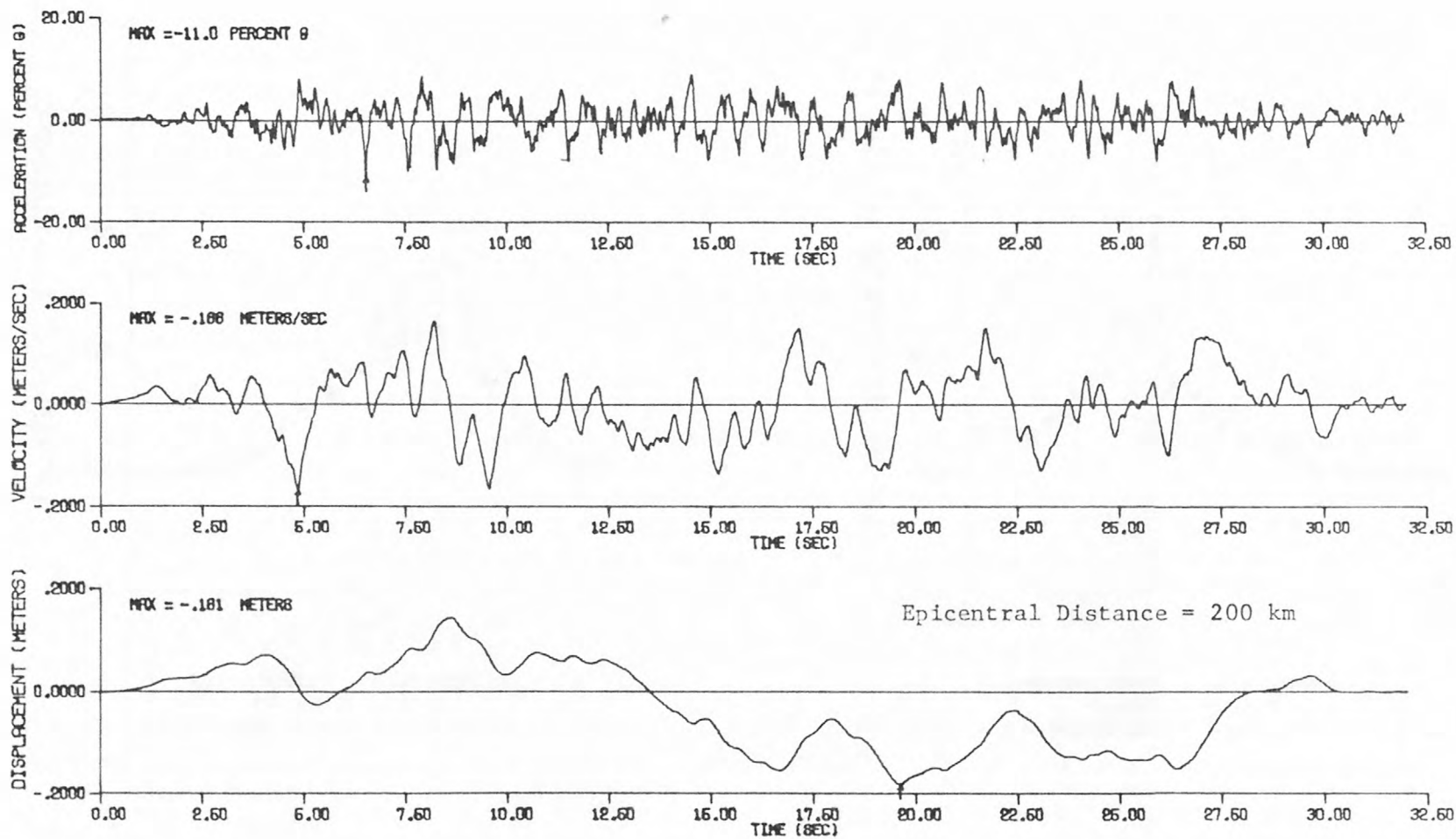
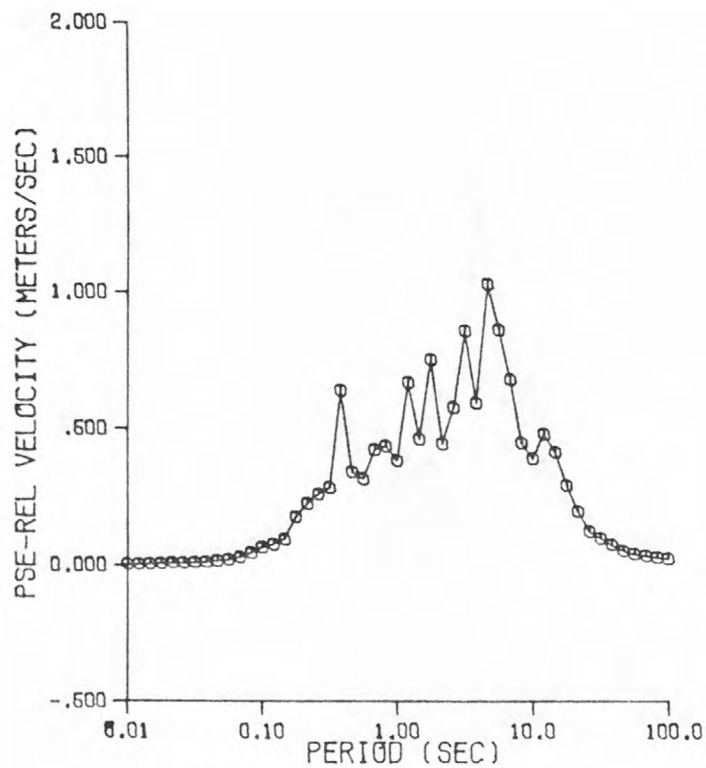
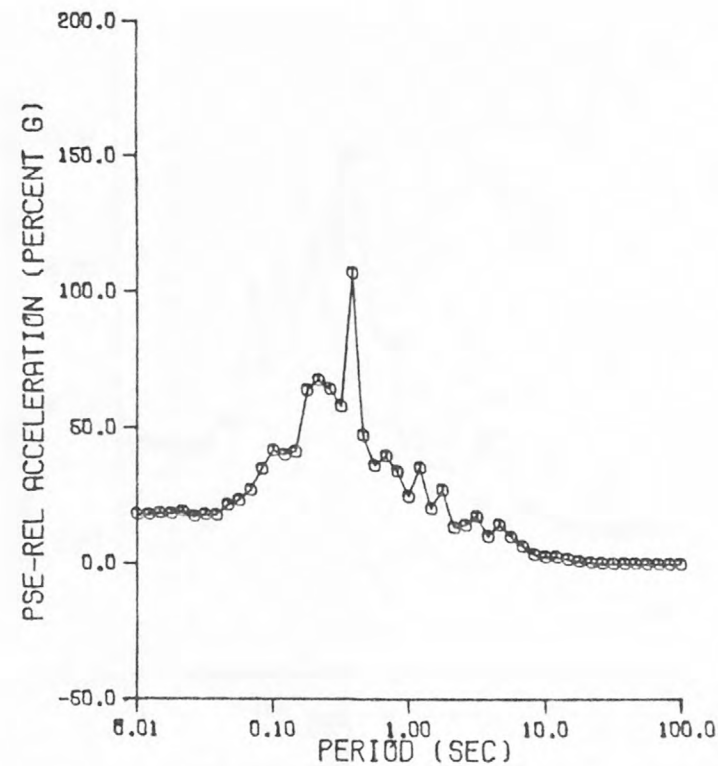


FIG. 5-11, Synthetic design Earthquake A3, for $I_o = VII-VIII$



MAXIMUM VELOCITY = 1.033 METERS/SEC
 FREQUENCY = .215 HZ
 DAMPING RATIO = .020



MAX ACCELERATION = 106.9 PERCENT G
 FREQUENCY = 2.810 HZ
 DAMPING RATIO = .020

FIG. 5-12, Response spectra for Earthquake A1

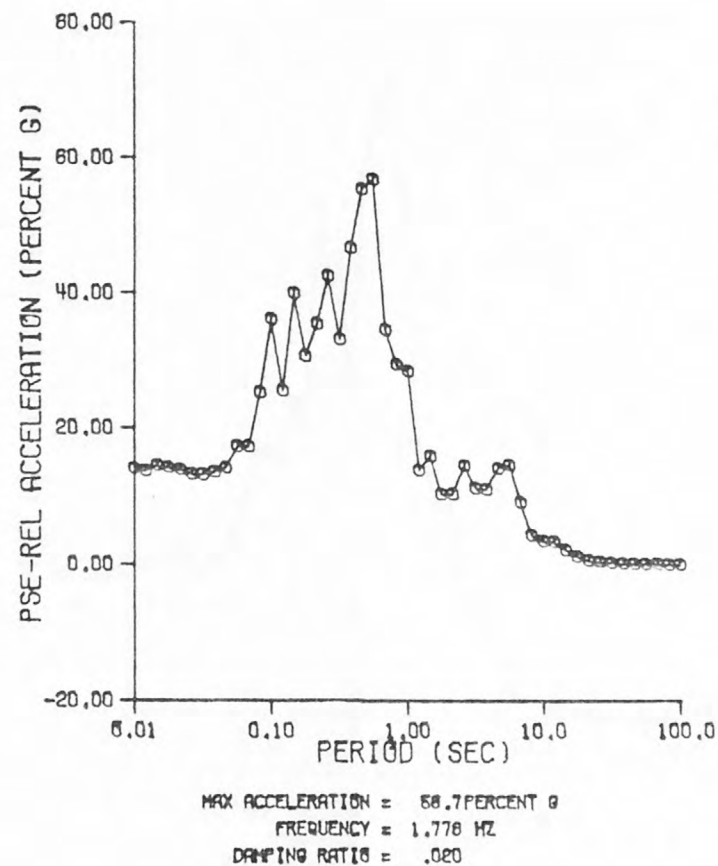
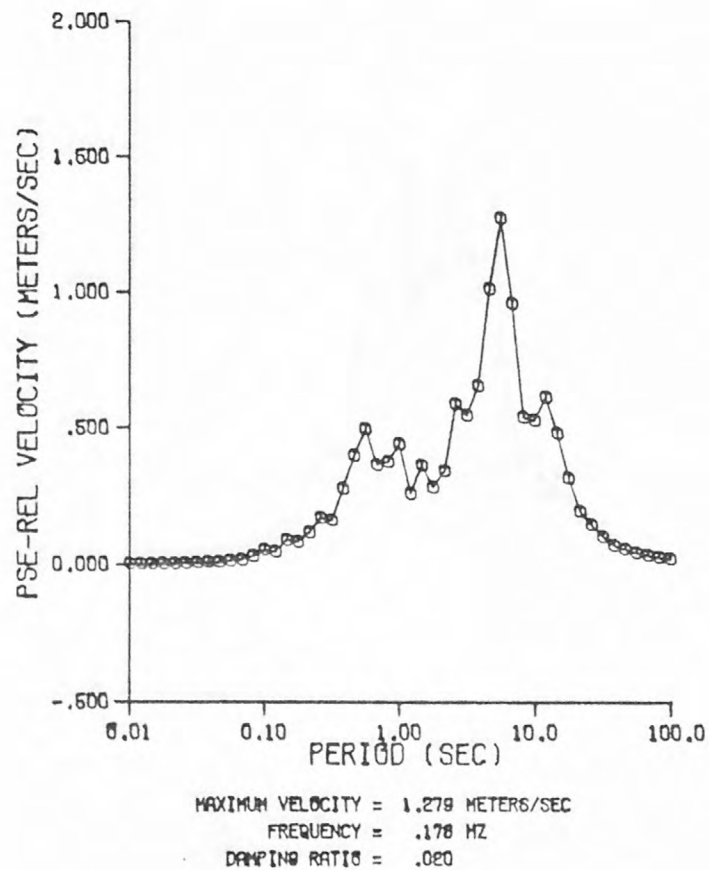


FIG. 5-13, Response spectra for Earthquake A2

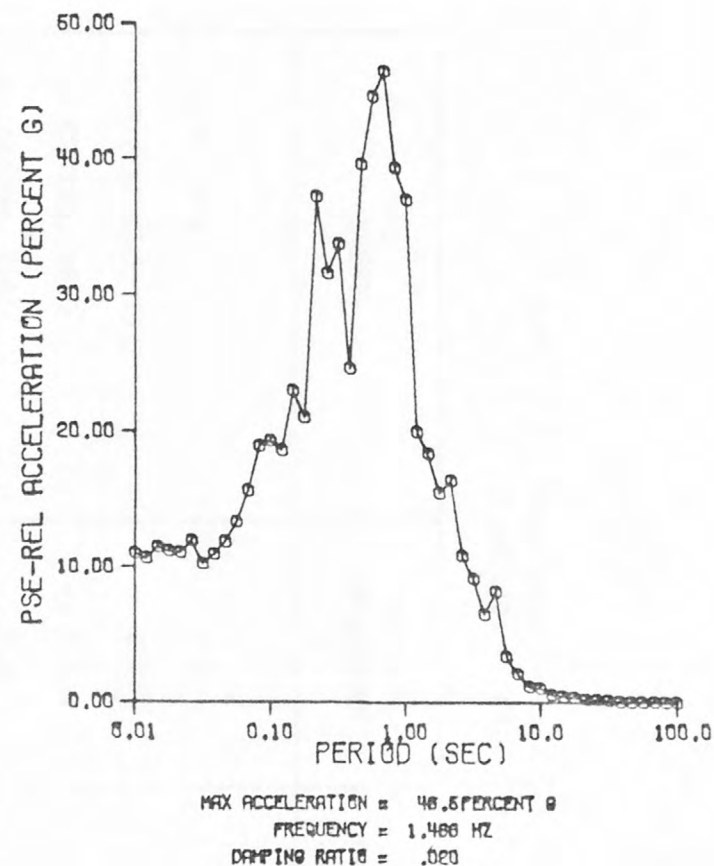
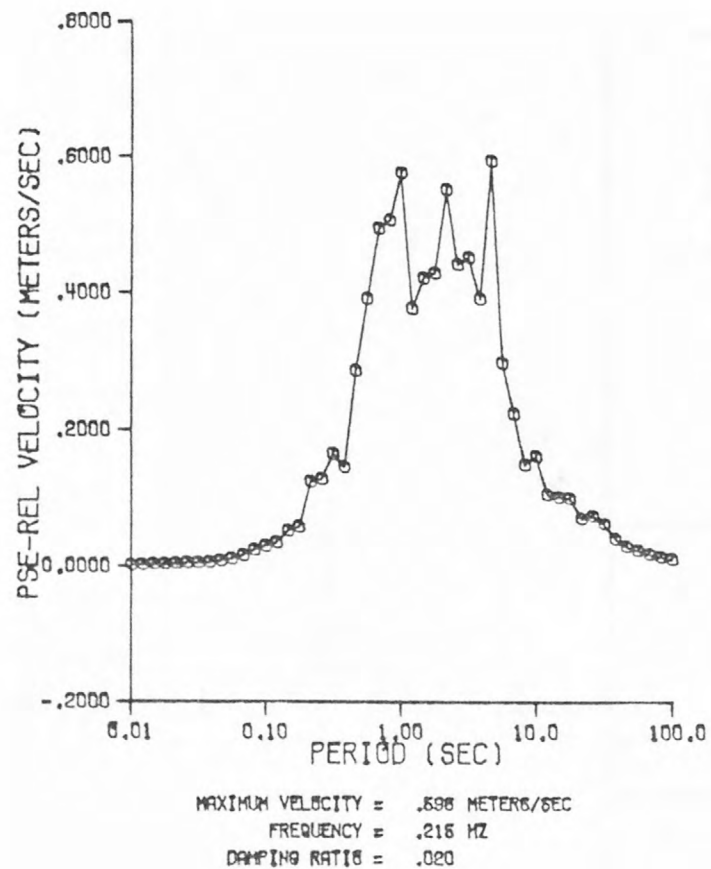


FIG. 5-14, Response spectra for Earthquake A3

TABLE 5-1, Magnitude and Bracketed
 Durations of Past Earthquakes

| EVENT | RICHTER MAGNITUDE | BRACKETED DURATION (Sec) |
|---|----------------------|-----------------------------|
| TAFT, 1952 | 7.7 | 19 |
| SAN FERNANDO, 1971 | 6.6 | 12 |
| EL CENTRO, 1934 | 6.5 | 18 |
| EL CENTRO, 1940 | 7.0 | 24 |
| GOLDEN GATE PARK SAN FRANCISCO, 1957 | 5.33 | 3 |
| HELENA (MONTANA), 1935 | 6.0 | 4 |
| OLYMPIA, 1949 | 7.1 | 24 |

TABLE 5-2, Bedrock Parameters for Intensity IX Earthquake in Region A

| EPICENTRAL DISTANCE | MAGNITUDE (RICHTER) | INTENSITY | BEDROCK ACCELERATION (Percent g) | PREDOMINANT PERIOD (Sec.) | BRACKETED DURATION (Sec.) |
|------------------------|------------------------|-----------|--|---------------------------------|---------------------------------|
| 50 km. | 7.0 | IX | 18 | 0.35 | 19 |
| 100 km. | | VIII-IX | 14 | 0.40 | 16 |
| 200 km. | | VII-VIII | 11 | 0.67 | 25 |

CHAPTER SIX

Response Analysis

Following in the aftermath of the famous San Francisco earthquake of 1906, a study by Wood (1908) showed the extensive variation of intensity according to site conditions. Hence, for any particular earthquake, although motions on rock-outcrops will be similar, there will be great variation at sites underlain by unconsolidated deposits (Seed, 1969; Ohsaka, 1969).

The city of Memphis, situated in the Mississippi Embayment, is expected to display response variations according to the engineering properties of the underlying soils. It has been shown that the uppermost 30m influence the extent of ground response (Seed et al, 1969; Duke and Leeds, 1962), which fortunately is the region for which we have the bulk of the borehole data for this study. At depths of over 60m, it is anticipated that the shear wave velocities are of a similar magnitude to those associated with rock, i.e., 1500-1800 m/sec (M. and H. Engineering and Memphis State University, 1974). From Figure 6-1, it can be seen that the general profile anticipated under Memphis is very "variable".

Thus, for the continuation of this study, it has been necessary to make approximations and assumptions based upon the present state-of-the-art. Hence, it is imperative that the importance of some of these assumptions be further elaborated.

The response analysis can be performed for any area where the nature of the underlying stratigraphy has been determined and the engineering properties evaluated.

The most important parameter has been found to be the water table. This influence was first recognized by the Russian investigator, S. V. Medvedev

(as cited by Gaus and Sherif, 1972). Based on a relationship developed by Medvedev, it is possible that where the water table is at the surface, the intensity may be one unit greater than where the water table is 10m deep. The general water-level within the city of Memphis does not exhibit any great seasonal variation. However, between the months of December and June, a perched water level is known to exist in the area under investigation (T. S. Fry, personal communication). As our analysis cannot include such discontinuities, we are unable to determine the true effects of such a condition. We shall thus use the more conservative approach and assume the water level to be the free-water-level. This level, when omitted from borehole-data, thus was assumed to be at an elevation of 66m (~ 220 feet) above sea level, approximately 6m below the surface (M. and H. Engineering and Memphis State University, 1974). The response due to bedrock motions is also highly dependent on engineering properties of the underlying soils.

The dynamic soil properties for the study were obtained from borehole data made available. However, these boreholes did not always extend to the depths which are necessary to determine the ground motion parameters. The response analysis used in this study makes use of the computer program SHAKE developed at Berkeley, California (Schnabel et al, 1972). The computational procedures used in this program have been developed for horizontally layered soils which is not always the case in nature. However, as other methods (e.g., Finite Element) involved very rigorous methods for only individual sites, we have used SHAKE, due to its general availability and applicability, for this study. Initially, the bedrock motions are input at an "outcropping" rock and then subsequently transferred to the base of the profile under investigation. The motion is introduced on an 'outcrop' as most of the available data has been recorded on such outcrops. These motions are usually applied at

depths of over 30m or at bedrock level, the bedrock being defined as the first consolidated layer of material exhibiting a shear wave velocity equal to or greater than 600 m/sec. Thus, we expect that the soils encountered at depths of over 45m will meet this criterion. For boreholes which do not extend to such depths it is necessary to assume the deeper stratigraphy on a tentative basis. The extra layers are necessary to provide reasonable parameters for ground motions. A study was performed on a borehole which only extended to a depth of 21.5m. The ground accelerations and spectral accelerations are shown in Figures 6-2 and 6-3 for different thicknesses of sand (at 100% relative density) at the base of this 'short' borehole with a bedrock acceleration of 18%g. It can be seen that variations do occur and thus it would be more accurate if a 45m depth was to be adopted. Thus it can be readily inferred that gross inaccuracies will occur if the earthquake motion is applied at a depth of only 21.5m.

Thus, we shall include 'extra' layers in some of the boreholes based on Fig. 6-1 and some of the deeper boreholes which are also available for this study. Between elevations 27m and 36m (Fig. 6-1), it is expected that dense sands exhibiting 100% relative density (D_R) will be encountered. However, the greatest variability exists between elevations 36m and 57m. In this variable zone, a multilayered system of clays and sands is encountered. In this zone we have included three distinct layers: sand (90% D_R), clay (s_u , the undrained shear strength = 150 kN/m^2) and sand (100% D_R). The relative densities and shear strength (for clay), have been selected from a review of the available borehole-data. Although it is almost impossible to be certain about these assigned values, we feel that the final ground motions will not be greatly affected. Fig. 6-4 shows the profile developed to provide a more realistic simulation of the characteristics anticipated in Memphis.

A computational study was also conducted to see if there was any major effect on surface motion for different stiffnesses for the clay layers. Figs. 6-5 and 6-6 show the results of the study for one of the boreholes with a bedrock acceleration of 18%g. It can be seen that there is very little variation in response and thus we shall use the undrained shear strength as 150 kN/m^2 which is more indicative of the borehole data. With these results available we feel that we can tentatively proceed to evaluate the ground response for areas where the boreholes extend to depths less than 45m.

As no data for the unit-weight of the soils was available, an approximation based on judgment and pertinent literature (M. and H. Engineering and Memphis State University, 1974) was necessary. The values used generally varied between $16\text{--}19 \text{ kN/m}^3$ for the soils and 23 kN/m^3 for "bedrock" below 45m.

The largest error is likely to result in determining the necessary dynamic soil properties, especially in the shear modulus. These dynamic soil properties (shear modulus and damping ratio) vary according to the shear-strain. However, this variability is only minimized at shear strains below 10^{-4} percent which is outside (below) the range of shear strains which would be expected for moderate to strong ground motion. The greatest variability exists at the high shear strain values expected for potentially damaging earthquakes.

The computational procedures for calculation of the response allow for shear strain compatibility with the shear modulus and damping ratio, and the calculations need only be performed to a degree of accuracy which is dependent on the 'true' reliability of the output. Hence it can be seen that to perform an accurate response analysis we must have sufficient dynamic soil data for each soil encountered at various sites. This approach has been

utilized previously for the determination of dynamic soil properties at different strain levels for numerous accelerograph stations in California. (Shannon & Wilson, Inc. and Agbabian Associates, 1978).

A similar approach cannot be applied to this study. Hence we will have to essentially infer data which will provide sufficient reliability for microzonation purposes. Fortunately, studies by Hardin and Drnevich (1970) and Seed and Idriss (1970), have shown that the range of expected soil property values can be very small. Their studies have indicated that there is an apparent relationship for clays and sands (and some gravels) for which average 'curves' may be assumed. For the data available for sands, they showed that the shear modulus was a function of the relative density (or void ratio) and the effective confining pressure. For clays, the average curve was found to be dependent on the undrained shear strength of the soil.

To use these relationships, we need to determine those parameters which are necessary for the determination of the shear moduli. The published damping ratios exhibited some scatter at different shear-strain levels but their importance in response analysis is only marginal so one can confidently use the average curve for sands and for clays.

The Standard Penetration Test blow count values (N_{SPT}), for which numerous empirical correlations are available, was the common denominator in all the boreholes in this study. Some of the correlations have been found to be accurate over the years, but the systems generally used in determining the N_{SPT} values are very susceptible to errors (Kovacs, et al, 1977 and Schmertmann, 1977). These different systems have resulted in correlations which, perhaps, may 'only' be suitable for the procedures used in obtaining the N_{SPT} values both for the initial correlations and the 'implied' parameters.

There are numerous N_{SPT} vs Shear Moduli relationships available in Japanese data. As the N_{SPT} systems used in Japan remain unclear (Kovacs,

1979), we cannot use these relationships with confidence since we cannot instigate any comparisons within the scope of this study. However, a note is included at the end of this Chapter describing a 'possible' approach if Japanese correlations are considered for any future studies.

For the determination of the necessary relationships we have to rely on correlations available for the Relative Density (Fig. 6-7) of sands and undrained shear strength of clays (Fig. 6-8). Once we have these initial parameters available we can then select the necessary parameters versus shear strains relationships. Fig. 6-9 illustrates the nature of the values which will be expected for sands at different relative densities (D_R). However, we intend to use the 75% D_R curve (Fig. 6-11) for the basic relationship with modifications for different D_R values.

The curves in Fig. 6-9 are not parallel and thus if we are to modify the 75% D_R curve, the modification factor is also shear-strain dependent. However, we cannot really attempt to iterate this modification factor and thus we shall only use a mean-value for a particular range of shear-strain anticipated. The variation of this modifying factor is illustrated in Fig. 6-10. Generally, it was found that the effective strains which were encountered varied from 0.01 to 0.1 percent and so for a conservative outcome factors for the 0.1 percent shear-strain were adopted. These factors can be applied to the curve shown in Figure 6-11 to simulate various other relative densities between presumed shear-strain levels.

Fig. 6-12 shows the average curve assigned for the determination of the shear moduli at different shear strains for a known undrained shear strength. No modifications are necessary for the clays as they are essentially dependent on the undrained shear strengths.

The dynamic soil properties of gravels were, also, investigated by Seed and Idriss (1970) and these were also used for this study. Fig. 6-13 shows

some of these relationships and the one adopted for this study. Some of the gravels encountered in this study exhibited very high N_{SPT} values which indicated that the layer could not be treated as sand. It is felt that the curve adopted is a conservative estimate of the shear modulus and will result in higher stresses. However, for the gravels, the damping ratio was limited to 90 percent of that which would be expected for sands at similar shear-strains. This assumption, although tentative, is expected to portray the expected additional stiffness (or rigidity) anticipated from gravelly soils.

As the Washington gravel adopted for this study, has a K_2 value of approximately 65 for the range 0.01 to 0.1 percent shear strain, it is necessary to modify the relationship presumed earlier for the 75% D_R sand. This sand had an average K_2 value of approximately 26 over the same range of shear-strains. Hence a factor of 2.5 was assigned to the sand relationship to simulate a suitable relationship for gravels.

The damping ratio-shear strain relationships for sands and clays are shown in Figures 6-14 and 6-15, respectively. These values were, also, used in the same manner as that for the shear moduli, without any modifications. However, it was not possible to locate any such relationships for silts. It is expected that these soils can represent either the sand and/or clay relationships already suggested depending on the silt plasticity. For this study, we have used the sand relationships for clean silts and sandy-silts. However, clay characteristics have been attributed to clayey silts. Although there are correction factors which may be used to correct the field N_{SPT} values for silts (Terzaghi and Peck, 1967), we have excluded these due to the uncertain nature of the soils. We feel that we cannot differentiate between dilatant and non-dilatant fine grained soils with sufficient accuracy

and thus have proceeded to omit the use of any correction factors. Table 6-1 tabulates all of the above assumptions for easier comparison.

Fig. 6-16 shows the various paths which are followed in reducing a typical borehole into data which is suitable for use in the response analysis. Thus it is possible to use these previously discussed approximate relationships for the calculation of strain-compatible properties based entirely on the N_{SPT} values recorded.

The shear-wave velocity is expected to be greater than 600 m/s below a depth of 50m. For the response analysis we will assume a bedrock depth of 45m and assign our accelerations to this level.

With the above assumptions and the methods for evaluating seismic properties suggested we feel that the response analysis can be performed. The analysis is performed with the three synthetic earthquakes developed in Chapter Five. The results of this analysis are discussed in the next chapter.

Note

It was noticed that although these Japanese relationships do not indicate the shear-strain level for which they are applicable (a very important omission), independent means may be utilized to arrive at a value of reliable shear-strain level. During this study, some of the boreholes were investigated and their shear moduli (at unknown shear-strain level) were calculated on the basis of the available direct relationships (Hara et al, 1973; Arango et al, 1978, Ohsaki and Iwasaki, 1973). An independent approach was also utilized by using the Holtz and Gibbs (1969) relationship for inferring the relative density of sands. In conjunction with data presented by Seed and Idriss (1970) and the relative density at a certain confining pressure, we were then in a position to compare the two shear modulus values. Upon comparison, one could then determine the pertinent shear-strain level. Then, this approach would enable us to use the Japanese correlations and would allow a reasonable determination of the shear strain dependent variables.

As this approach seemed to initially depend on a correlation, which may be susceptible to the procedures used, we decided to exclude this approach. We feel that it would be perhaps more reliable to use correlations developed in the United States as these will probably offer greater reliability.

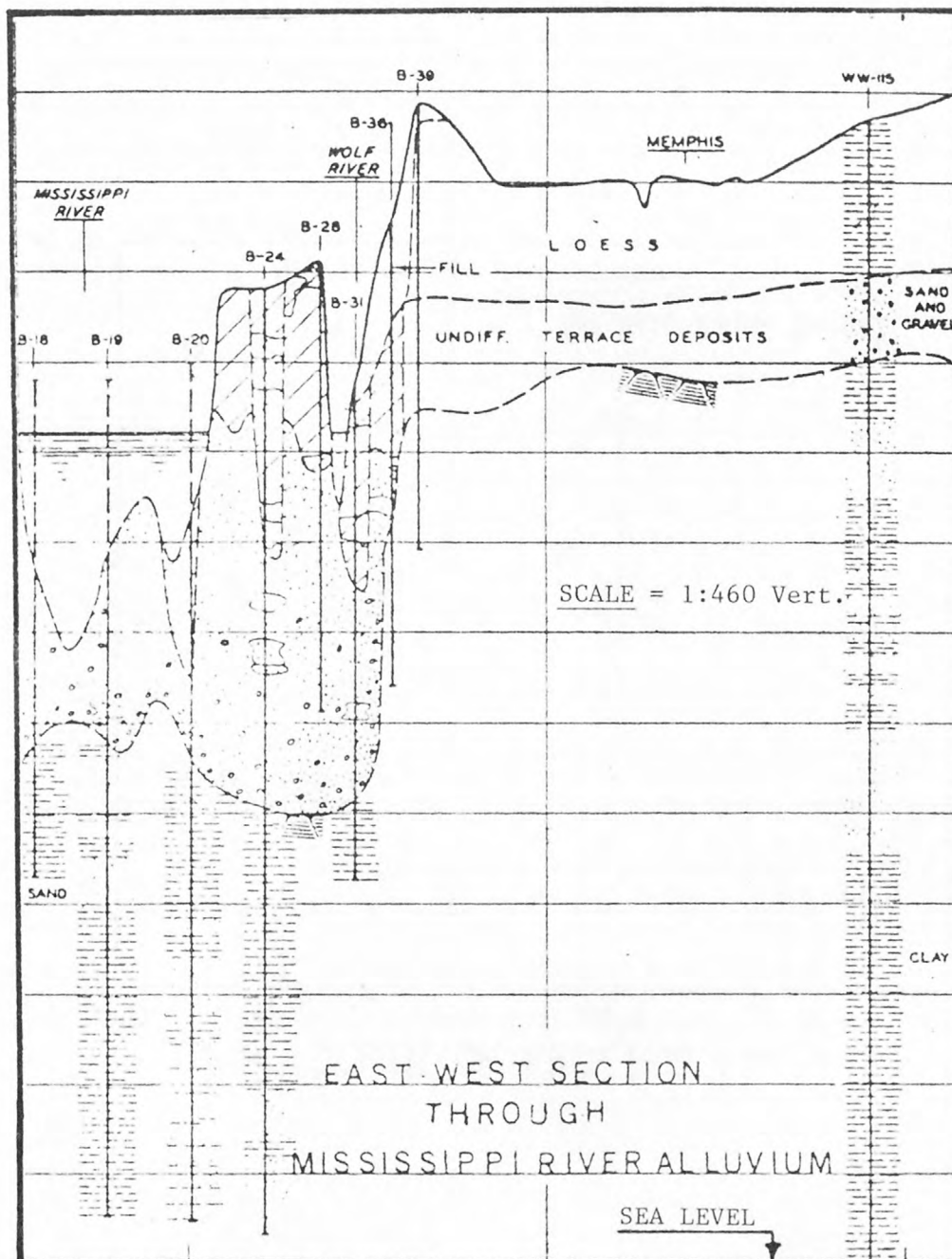


FIG. 6-1, Geological stratification beneath Memphis

(after M. & H. Eng. and Memphis State U., 1974)

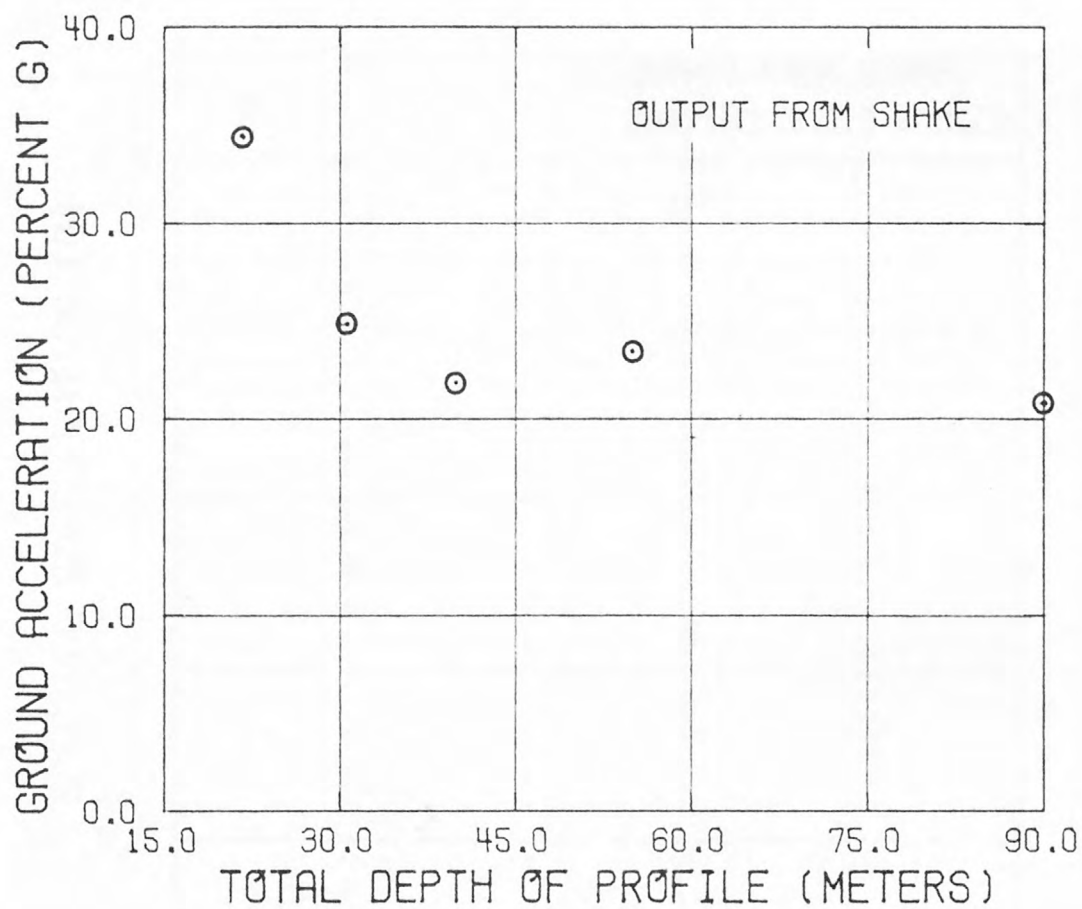


FIG. 6-2, Ground accelerations for additional sand layers

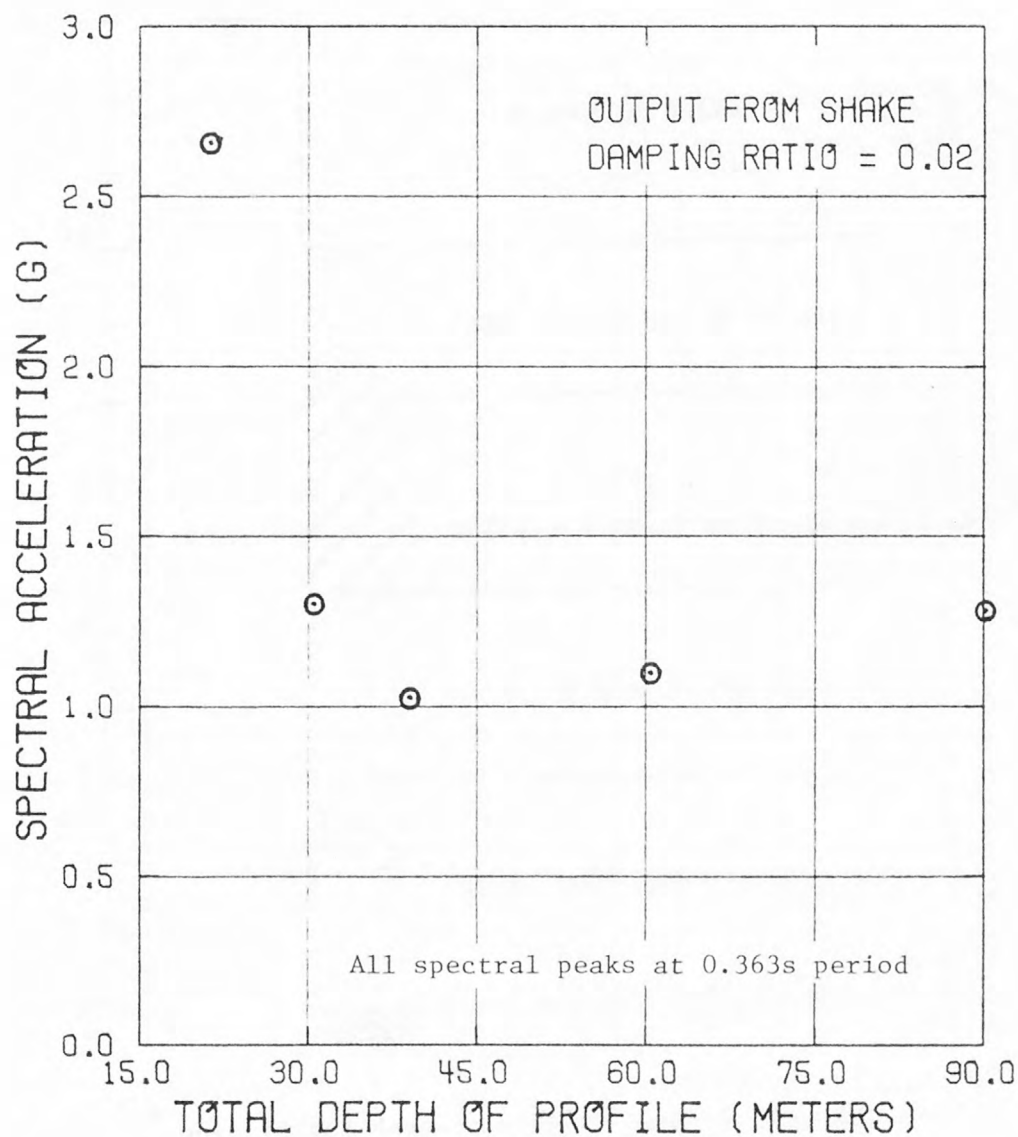
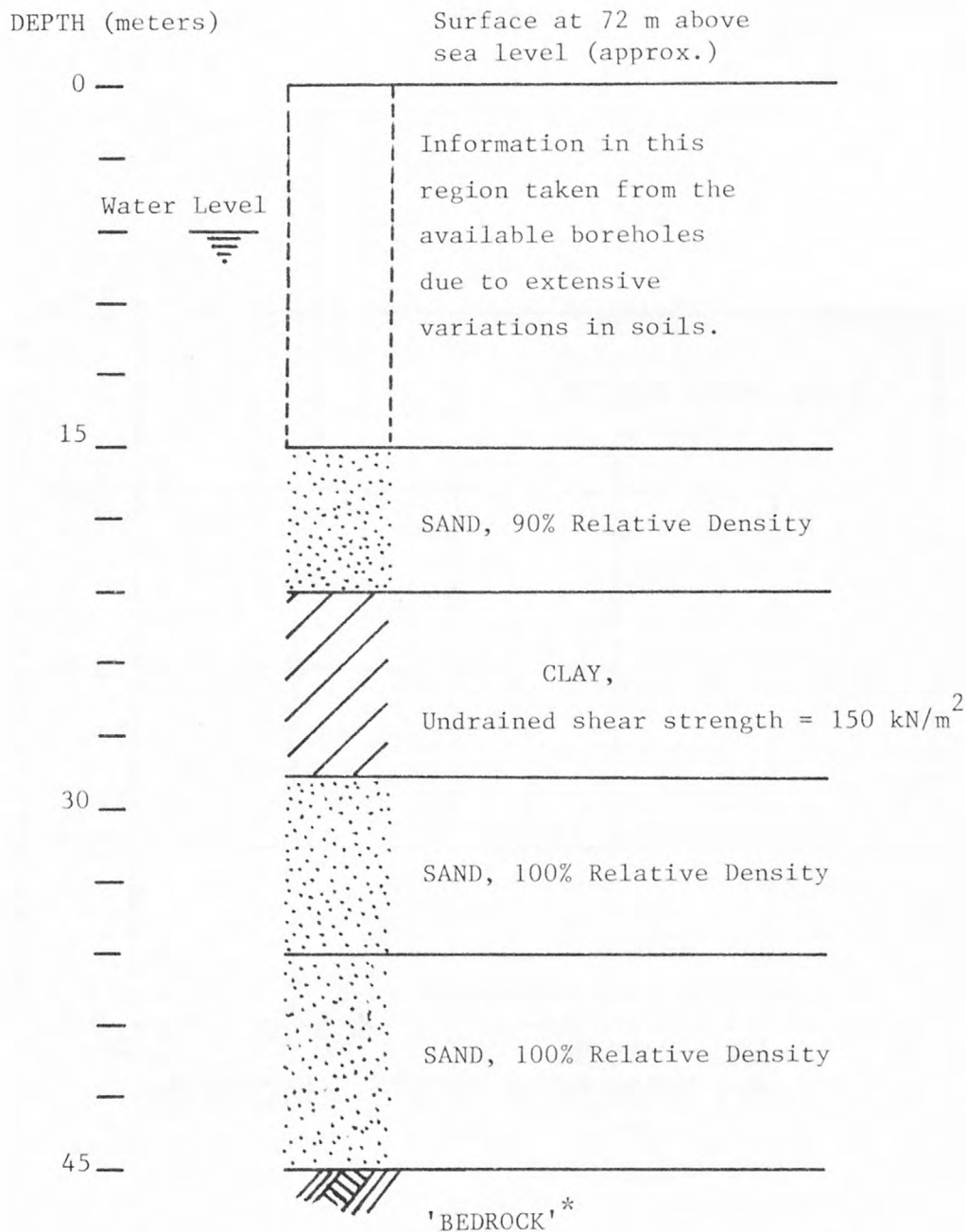


FIG. 6-3, Spectral accelerations for additional sand layers



* Consolidated deposit with V_s greater than 600 m/s

FIG. 6-4, Anticipated profile beneath Memphis

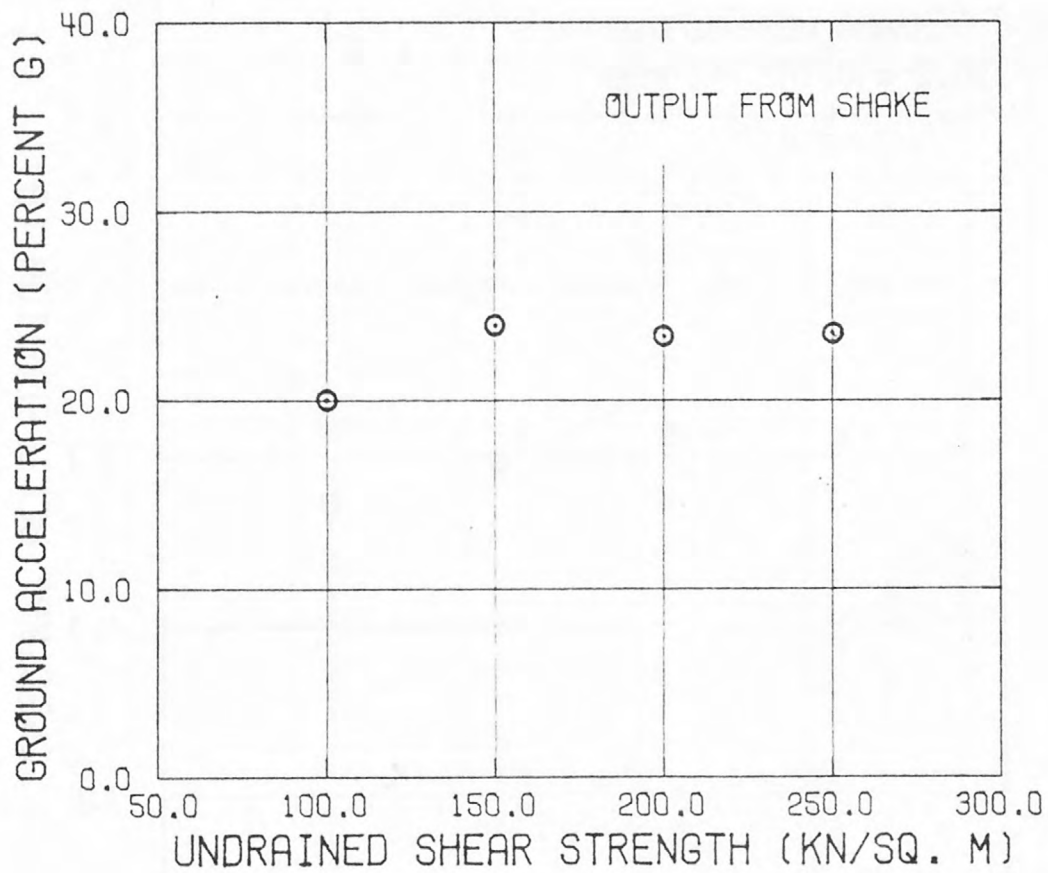


FIG. 6-5, Ground accelerations for different stiffnesses of additional clay layers

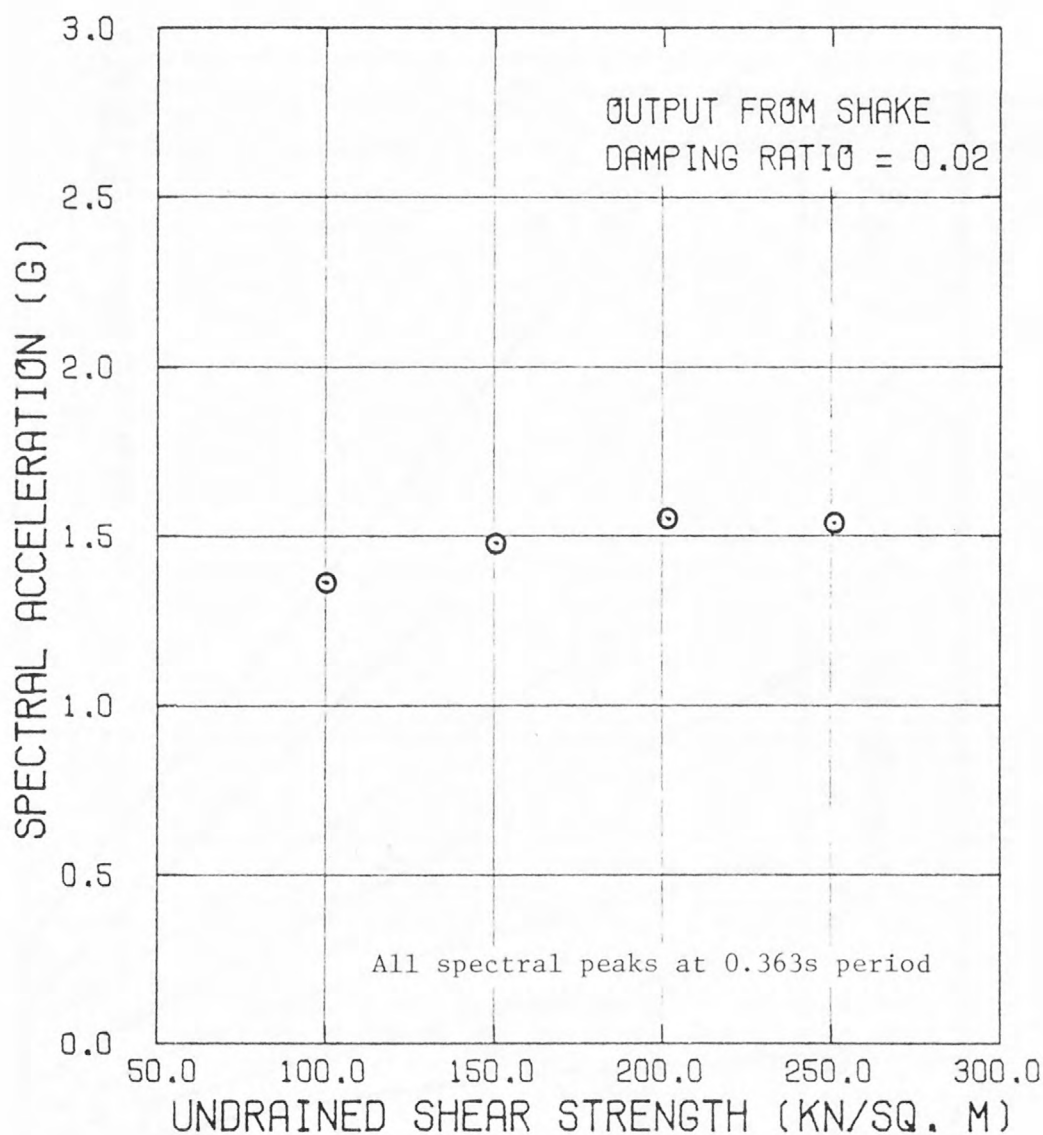


FIG. 6-6, Spectral accelerations for different stiffnesses of additional clay layers

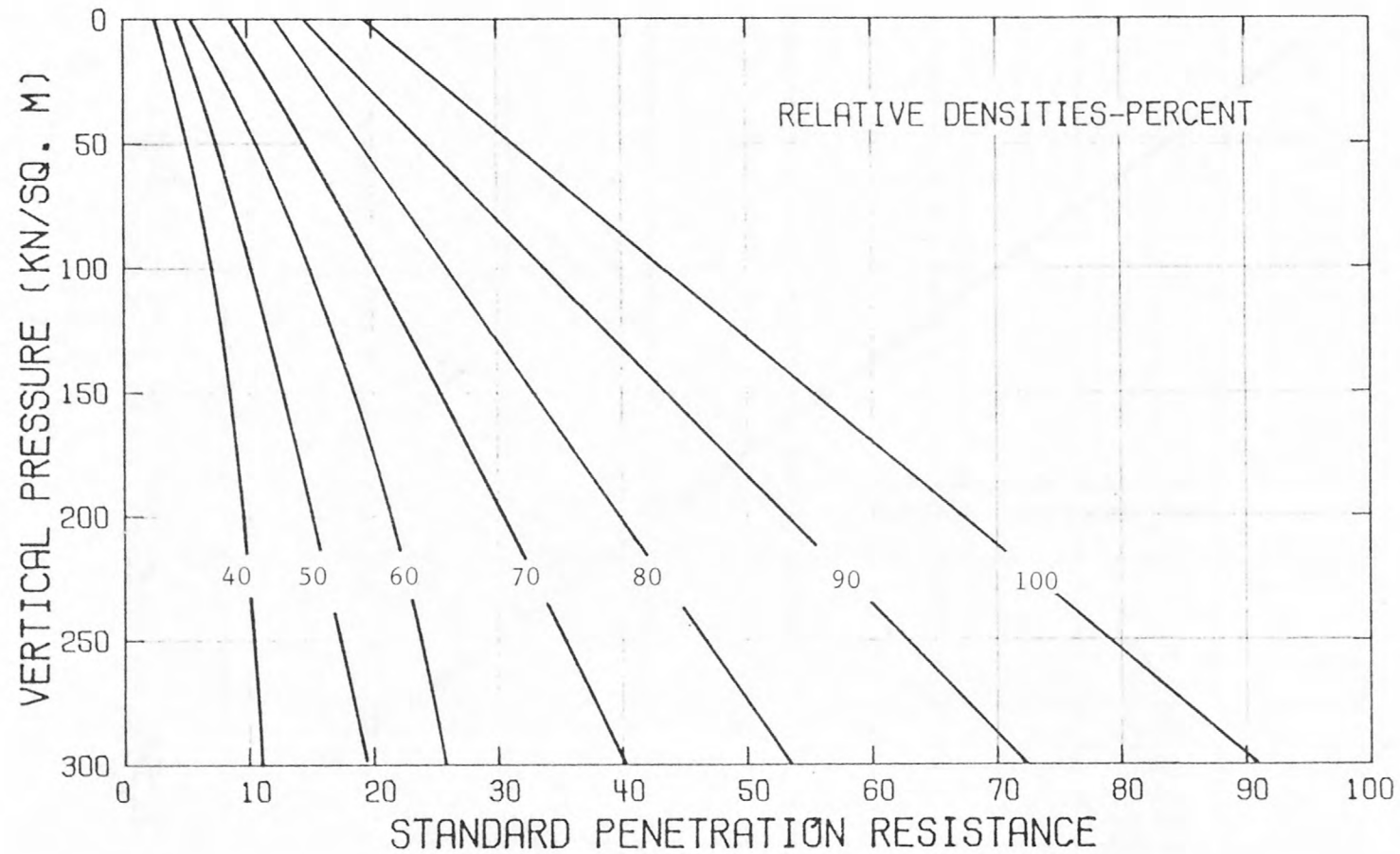


FIG. 6-7, Relative Density calculation from Standard Penetration Resistances
(after Holtz and Gibbs, 1969)

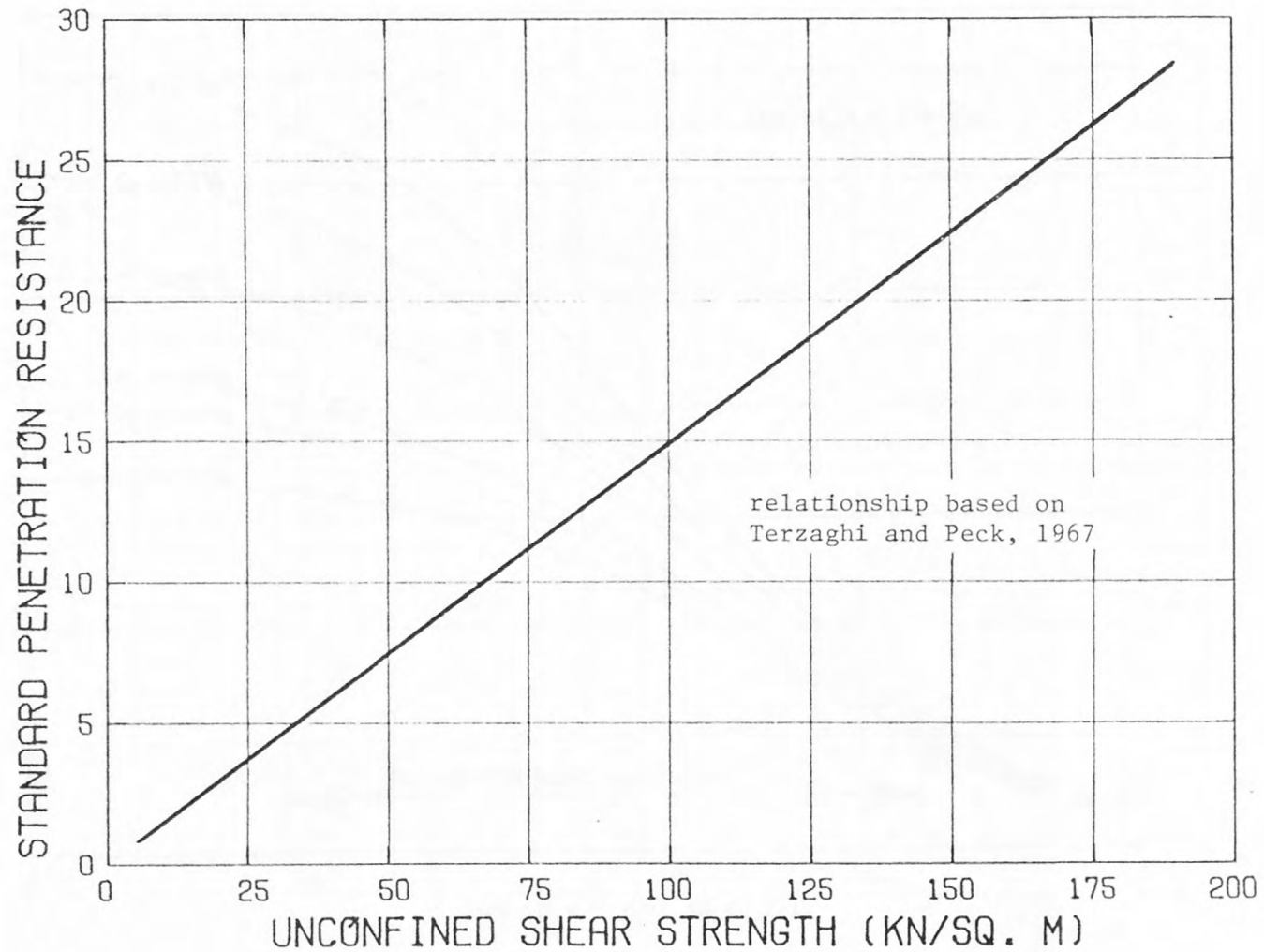


FIG. 6-8, Shear strength calculation from Standard Penetration Resistances

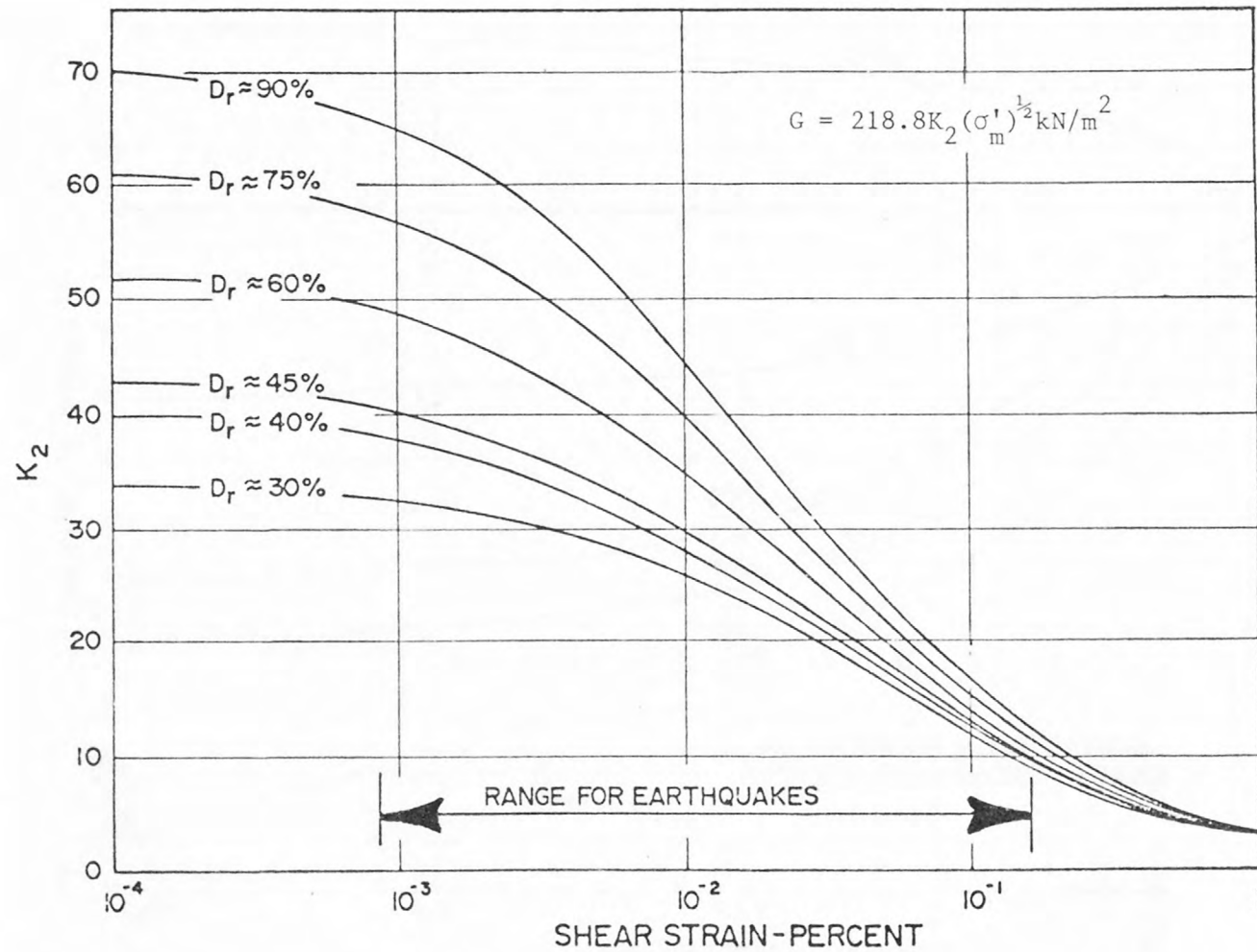


FIG. 6-9, Average shear moduli for sands at different relative densities

(after Seed and Idriss, 1970)

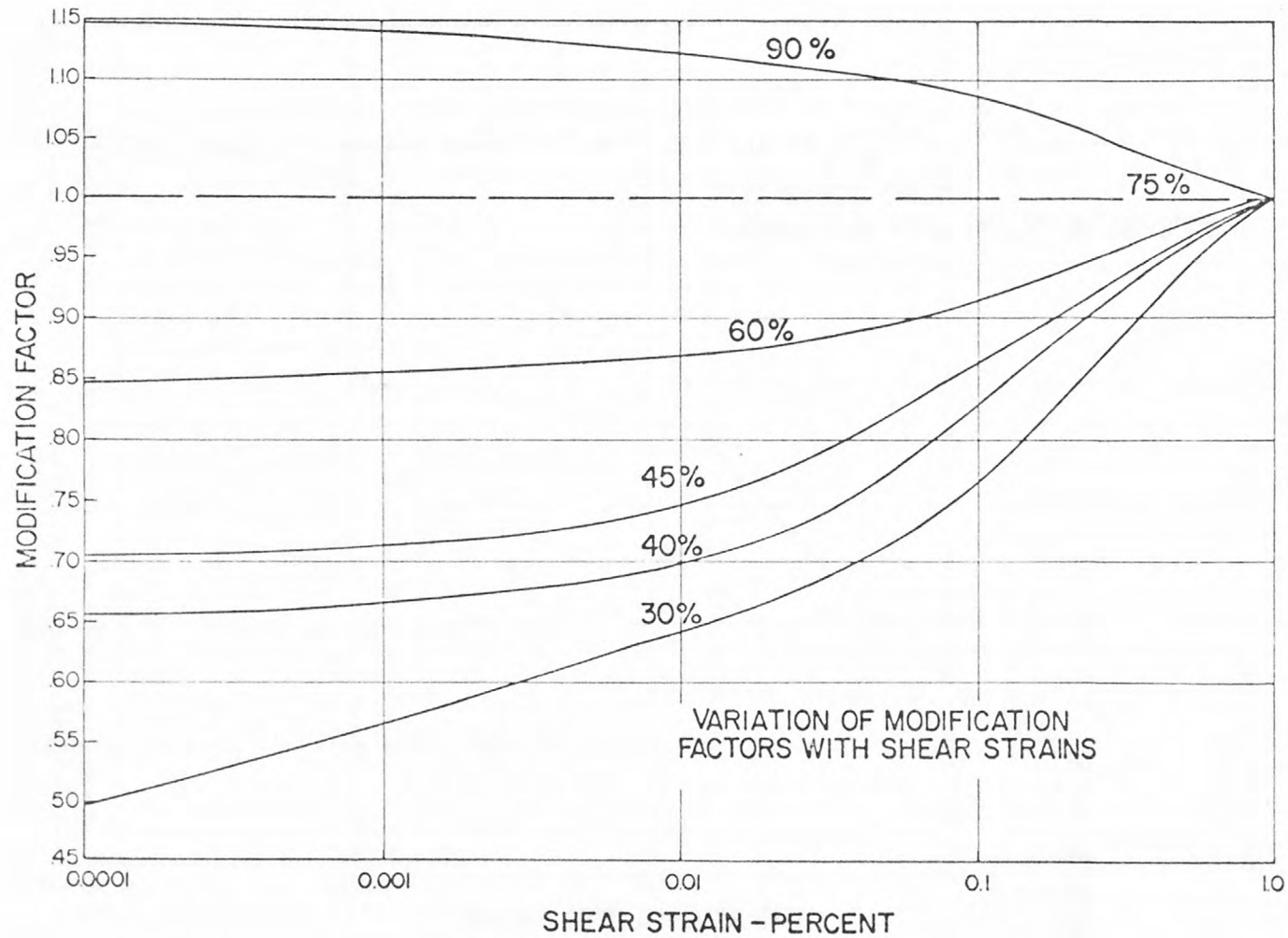


FIG. 6-10, Factor for modifying the 75% relative density relationship

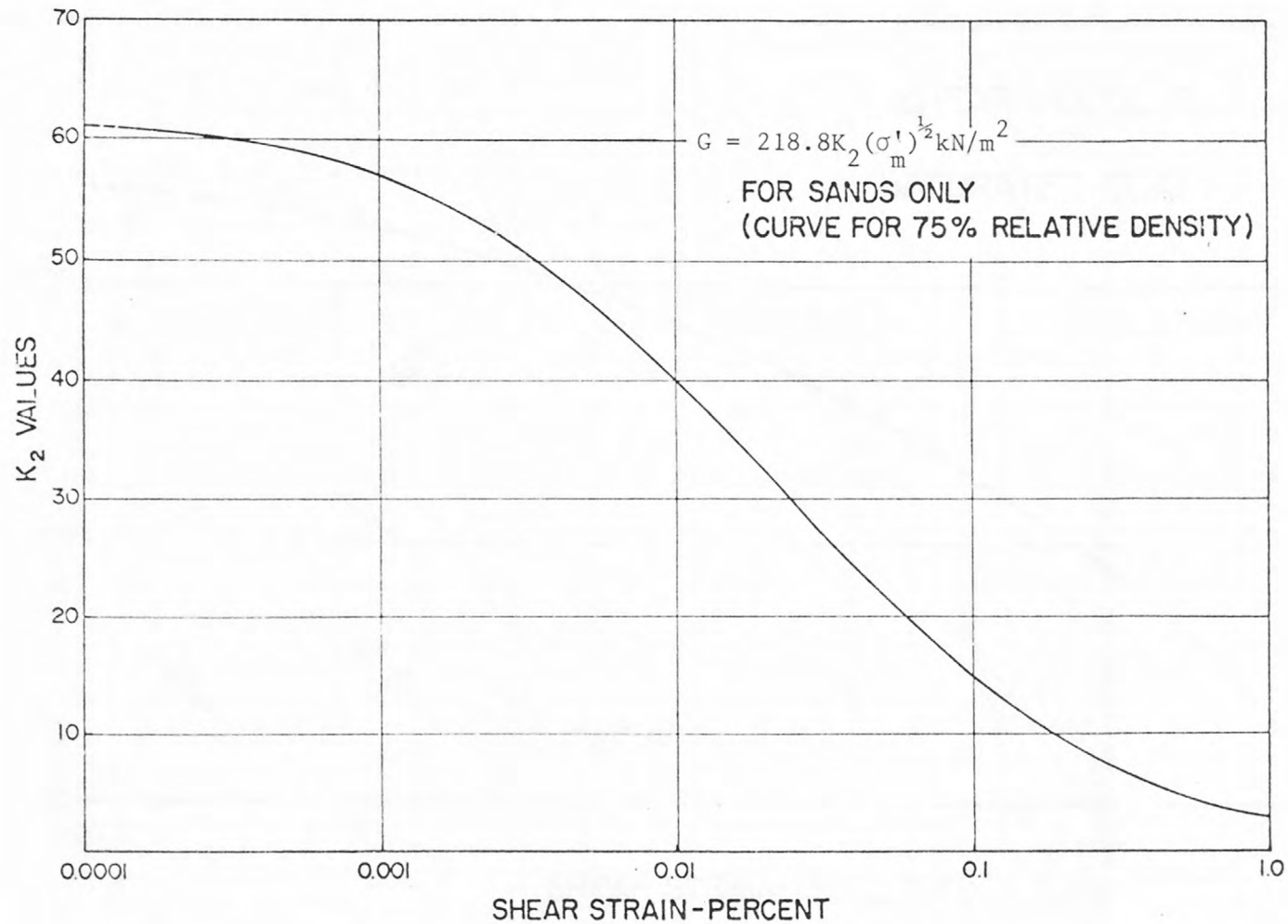


FIG. 6-11, Average shear moduli for sands at 75% relative density (after Seed and Idriss, 1970)

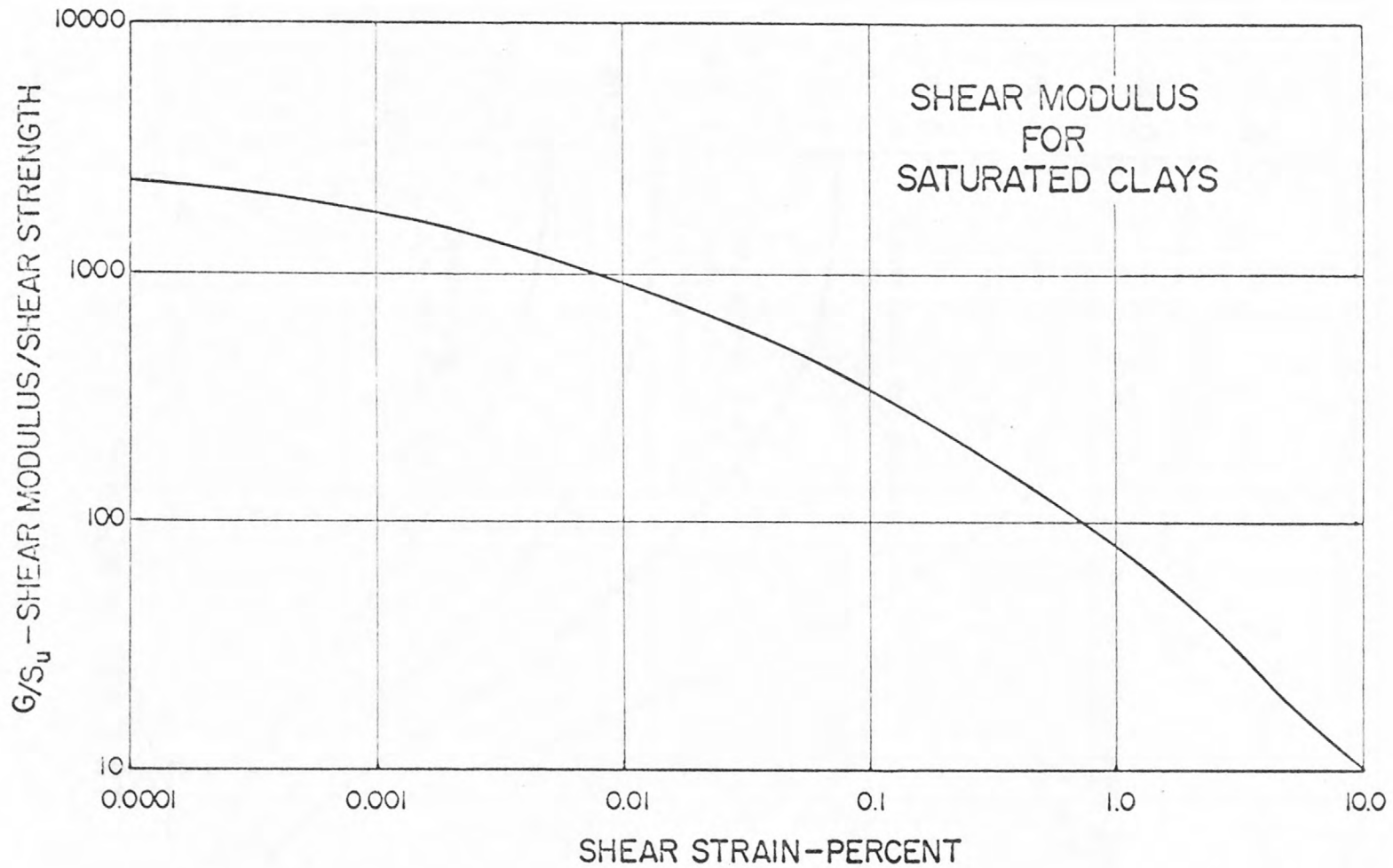


FIG. 6-12, Average shear moduli for clays for known shear strengths (after Seed and Idriss, 1970)

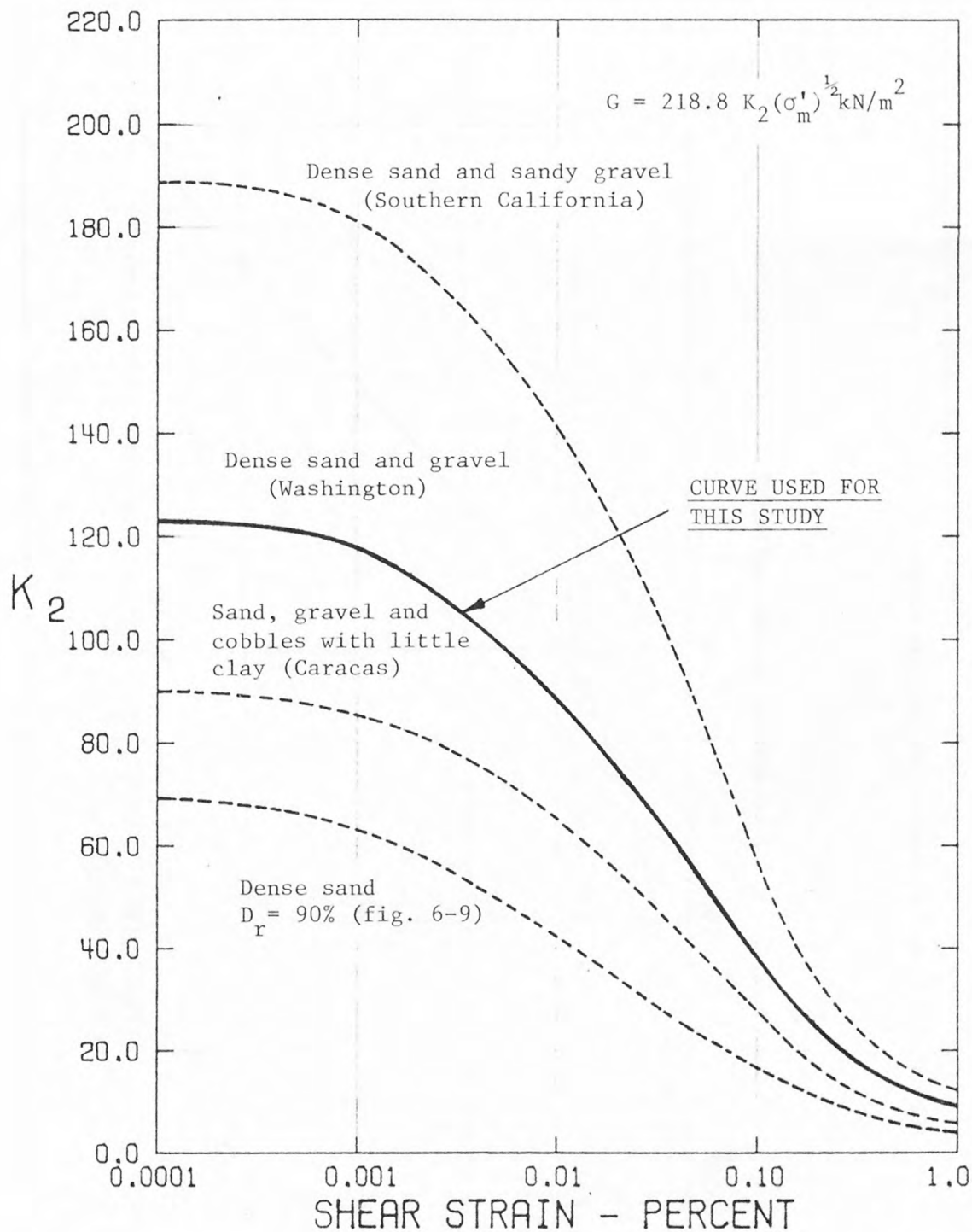


FIG. 6-13, Moduli determination for Gravelly soils

(after Seed and Idriss, 1970)

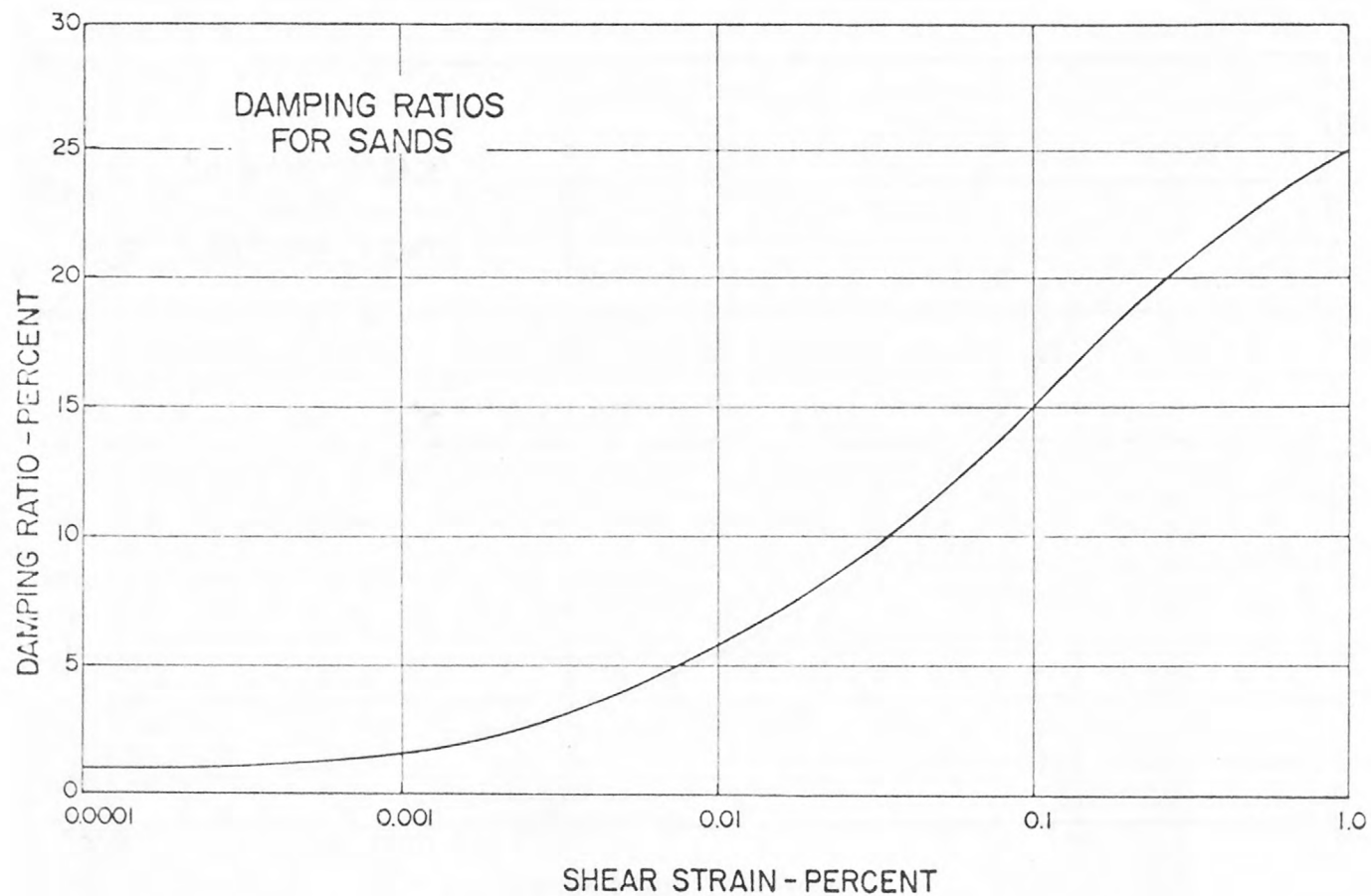


FIG. 6-14, Average damping ratios for sands

(after Seed and Idriss, 1970)

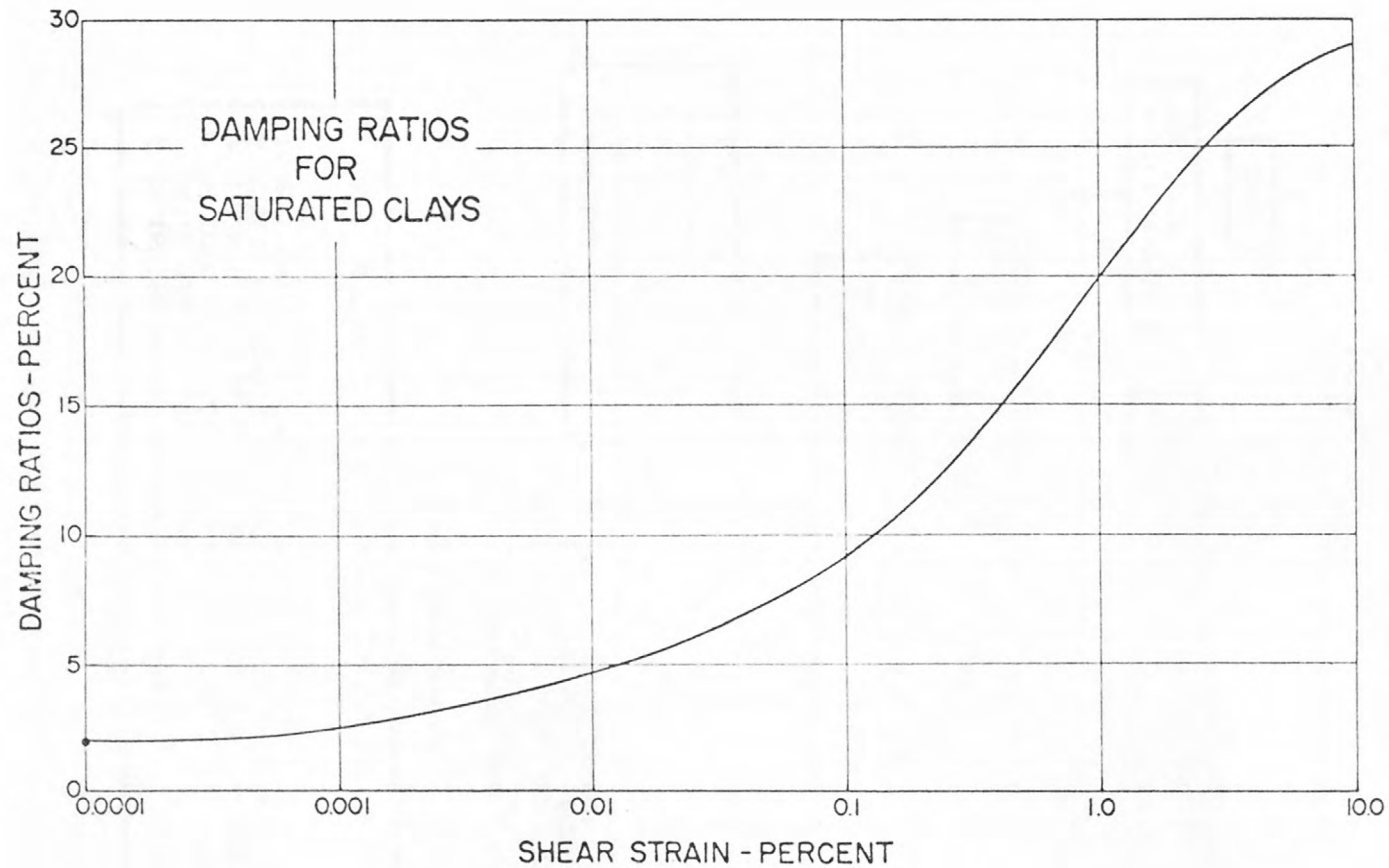


FIG. 6-15, Average damping ratios for clays

(after Seed and Idriss, 1970)

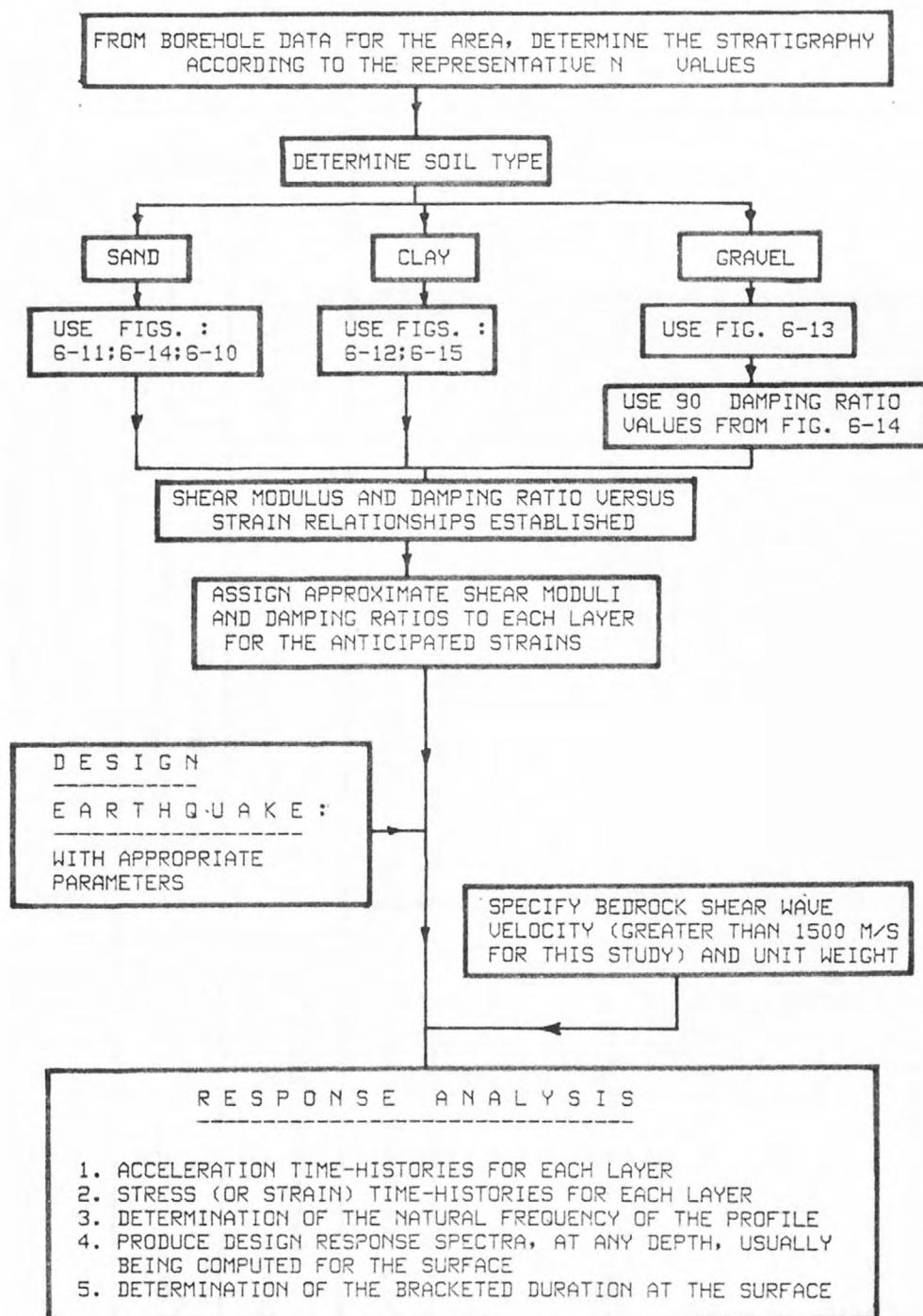


FIG. 6-16, Flow diagram summarizing procedures used
for the response analysis

TABLE 6-1, Summary of Design Relationships

| SOIL TYPE | DESIGN RELATIONSHIP UTILIZED | |
|---------------------------------|--|--|
| | Shear Modulus | Damping Ratio |
| Sands, Silts and sandy silts | figure 6-11 | figure 6-14 |
| Clays and clayey silts | figure 6-12 | figure 6-15 |
| Gravels | use the curve for the Washington dense sand and gravel in figure 6-13 | use 90% of the values pertinent for sands as shown in figure 6-14 |

CHAPTER SEVEN

Microzonation Results

The identification of potential effects due to any postulated earthquake are usually entirely dependent on the amount of data available. These effects cannot be considered in a deterministic approach due to the extensive variability of the characteristics which describe such an event.

The Romanian city of Bucharest, for which microzonation maps had been prepared, is a typical example of a case where the hazard-predictions were very inaccurate for the March 4, 1977 earthquake. The maps had been prepared with the postulated earthquake being an almost exact contradiction of the actual event. However, if the analysis had adopted the 1941 event as the postulated earthquake, there would have been a very good correlation. Thus we would like to impress on the reader that as the earthquake-prediction model is very unreliable, microzonation maps should not be adopted for critical design without any additional investigations.

This study has considered the effects of the surficial geology and evaluated the selective amplification which may be anticipated in various parts of the city. The amplifying effects were computed for most of the borehole data which had been collected from numerous local sources.

Fig. 7-1 indicates the areas for which sufficient bore-hole data was available to enable a tentative determination of the stratification. This figure indicates the relative amount of data which was collected for each zone. Although all soils-data was treated individually, we have precluded the exact locations to protect the confidentiality of the sources.

The limitations of each map are thus related directly to the amount of information which was available for this study. We have proceeded with these

data to evaluate the necessary hazards which represent our initial micro-zonation proposals.

Figs. 7-2, 7-3 and 7-4 illustrate the anticipated ground motion amplification due to the three design accelerograms adopted for this study.

Fig. 7-2 defines the amplification experienced within the city due to the nearby earthquake with an epicentral distance of 50 km and a peak "bed-rock" acceleration of 18%g. It can be seen that there appears to be considerable variation in the zones where the bulk of the soils-data was available. The amplification thus tends to vary according to the general nature of the stratigraphy presumed from the boreholes. Some of the higher amplifications were determined for the zones close to the Mississippi and Wolf rivers. These regions probably would include the 'softer' profiles consisting of depositional materials which are of a loose nature. However, areas where the large pockets of stiff clays exist show very small amplifications. This perhaps could be attributed to the relative rigidity of these clays when compared to the sand deposits. The amplification appears to diminish towards the S.E. direction which is as anticipated due to a lower water-table and also the denser nature of the soils away from the rivers.

Similar results were also obtained for the other design earthquakes which are illustrated in Figs. 7-3 and 7-4. For the earthquake located at an epicentral distance of 100 km and a "bedrock" acceleration of 14%g, we find that the variations are reduced as the general amplification away from the river is only 1.2 - 1.3 times the "bedrock" motions. This range appears to be very similar for these two earthquakes and possibly can be attributed to the very similar frequency contents of the two design accelerograms.

There are higher amplifications experienced towards the south-east in response to the third design earthquake. Fig. 7-4 illustrates the variation

of the amplification. Although zones with locally high amplifications are not evident, the general magnification is appreciably greater with the general amplification being about 1.5. This higher value is the result of the low frequency content of the design earthquake at an epicentral distance of 200 km. We feel that these incoming 'shock-waves' have a relatively higher energy content due to the duration of the acceleration time-history, and this results in larger deformations. Also, this may be directly attributable to the natural period of the soils being in the range 0.7 - 1.0 seconds such that a resonant condition develops.

We also microzoned the city of Memphis according to the natural periods of the numerous soil-profiles which were developed for the analysis. These are shown in Figs. 7-5 to 7-7. These figures also illustrate the variations which are indicative of the relevant strain levels which were developed for the three different design accelerograms. These values can be used in any preliminary investigation which may require an analysis of earth structures. We also feel that a greater magnification would result for any distant earthquake that had a predominant period of 1 second at a site within the city of Memphis. This would be very close to the resonant period of the soil profile and would naturally be amplified by a significant amount. However, a distant earthquake might result in an attenuation of the amplitude to a negligible amount, which then would produce surface accelerations comparable to the ones already computed. These natural period-dependent amplifications are also evident for the spectral accelerations.

Figs. 7-8 to 7-10 show the amplification factor for the spectral accelerations for the three design earthquake, all assuming 2% damping. The response expected for the nearby earthquake can be seen to vary from about 5 to 9 times the specified "bedrock" motion. Most of the sites had a maximum response at a period of 0.363 second which was very similar to the predominant

frequency of the input accelerogram. Slightly higher spectral accelerations were also computed for the earthquake located at an epicentral distance of 100 km. For this earthquake the peak response occurred at a period of 0.912 seconds for most areas analyzed. The amplification was slightly higher than that experienced for the earthquake A1 and the general distribution was found to be very similar. It was noted that this greater magnification tended to occur at a higher period than the predominant period of the input accelerogram. The magnification at these periods was of a sufficient magnitude to result in greater peak spectral acceleration than those occurring at a period corresponding to the input accelerogram.

There is, however, an appreciable variation of the spectral acceleration due to earthquake A3 (of an epicentral distance of 200 km). We feel that this response shown in Figure 7-10 illustrates a very good correlation with the natural period of the soils shown in Fig. 7-7. As the input accelerogram displayed a predominant period of approximately 0.7 seconds, large magnifications are evident in the S.E. section of the city due to very similar natural periods of the soils. This spectral acceleration approach is used regularly for the dynamic analysis of structures.

It should be realized that the nearby earthquake is likely to be more damaging to structures of 3-4 stories. Whereas the two more distant earthquakes will be more hazardous to structures of 9-10 stories. This was previously illustrated in Fig. 2-1 where taller buildings were found to have sustained greater damage with increasing depth of soils (analogous to increasing period). Additional structural damage may occur due to liquefaction causing a loss of bearing capacity.

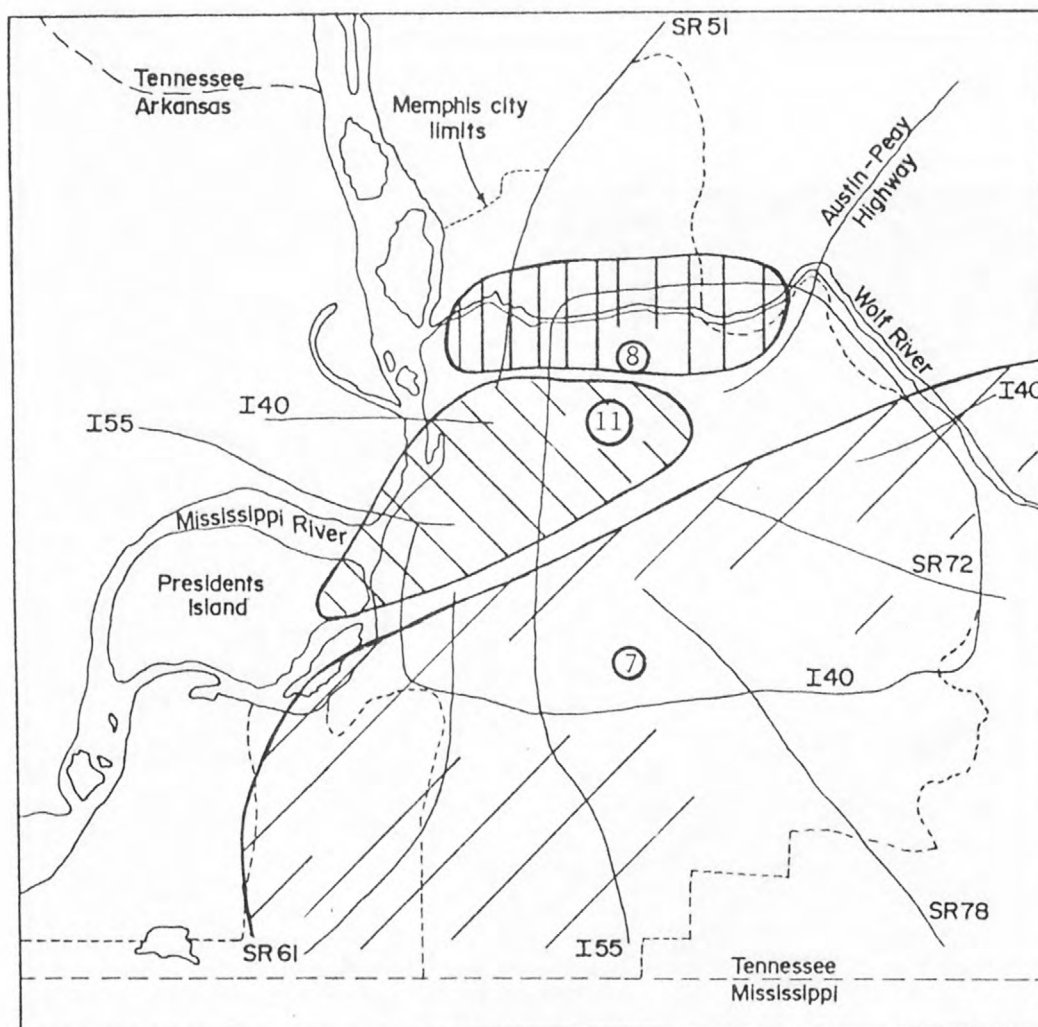
The liquefaction potential of some of the layers was investigated from available data. The procedural analysis for this evaluation can be found

in Appendix C. As only a simplified approach can be utilized within the scope of the study, only empirical relationships based on previous laboratory studies and field observations may be used. Only layers which exhibited a relative density (evaluated from the N_{SPT} value) less than 75% were investigated. It was considered that any sands with a relative density greater than 75% would not exhibit liquefaction for a sufficient time period to initiate loss of bearing capacity and were thus excluded from the liquefaction study. During part of the year, a perched water table is encountered beneath Memphis. This anomaly is not included when the liquefaction potential is assessed and thus a conservative result is assumed. Fig. 7-11 shows areas which may be susceptible to liquefaction. It should be realized that areas susceptible to liquefaction have been determined only from available data. Most areas in the vicinity of the Mississippi river or on the flood-plain (e.g., Presidents Island) will also be very susceptible to liquefaction and the problem would be recognized by most competent Engineers. The location of some of these zones is in the areas of river deposits which tend to be relatively loose. Although the liquefaction potential was investigated for all three design earthquakes, we have not attempted to isolate the possibility of liquefaction for individual design earthquakes. We feel that if liquefaction is suspected, laboratory tests are necessary to evaluate the failure criteria before limiting parameters can be applied.

In our original proposals we were also interested in the investigation of landslide susceptibility. Fuller (1912) had expressed fears about the stability of the bluffs on the eastern bank of the Mississippi river. However, he mainly studied the Chickasaw Bluff area where some landslide activity has been reported. We feel that because Memphis is situated 50-100 miles south of the area studied by Fuller, we cannot compare the pertinent characteristics. Fuller's investigations of these landslides was initiated

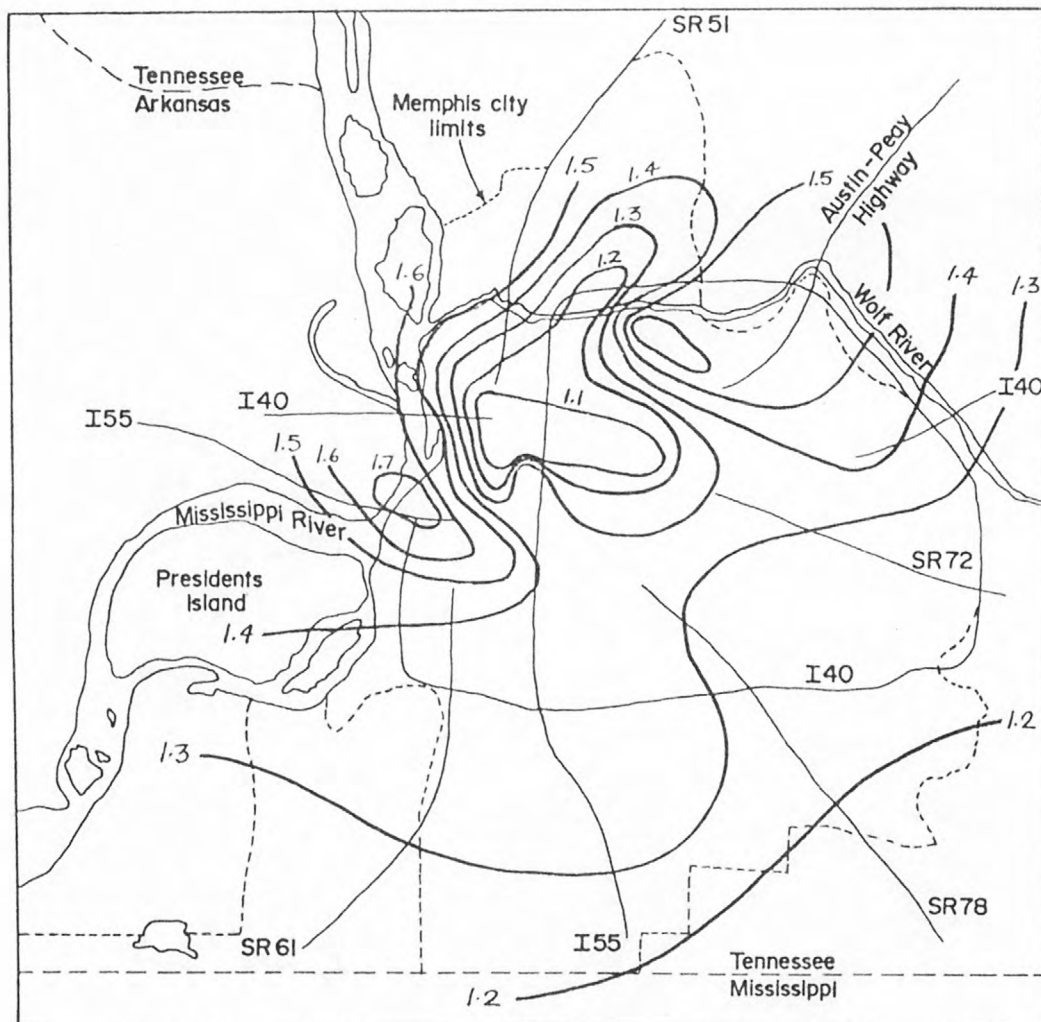
almost one hundred years after the major shocks of 1811-1812 and the cause of the landslides cannot be attributed entirely to the earthquakes. Additional information is thus required to analyze the possible failure of the bluffs in Memphis due to future damaging earthquakes. At present, a bigger risk is the annual 'flooding' of the loess which leads to slope-failure due to the hydrostatic pressure in the soils after recession of floodwaters. We feel that it would involve a very conservative design if flooding and earthquake loads were to be considered for design purposes. We have omitted an analysis of this aspect for microzonation because there is a lack of information as to which areas are susceptible to slope-failure.

To conclude this chapter, again we would like to impress upon the reader that the maps developed in this study are only of a qualitative nature. Any attempt to use this information on a quantitative basis is entirely misleading (or very erroneous in our opinion). The limitations and assumptions implied in this study are extensive and are further discussed in the next chapter.



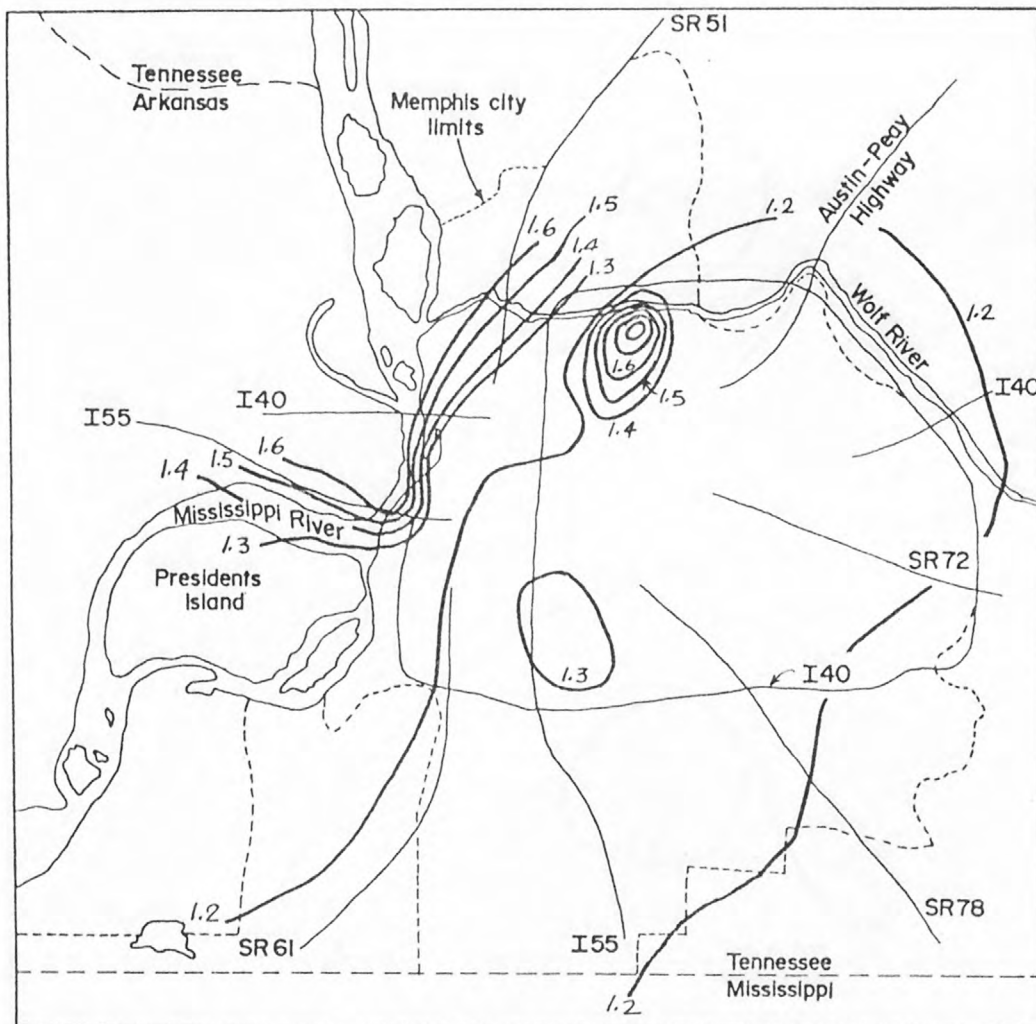
figures within shaded areas indicate the number of sites investigated for this study.

FIG. 7-1, Borehole data locations



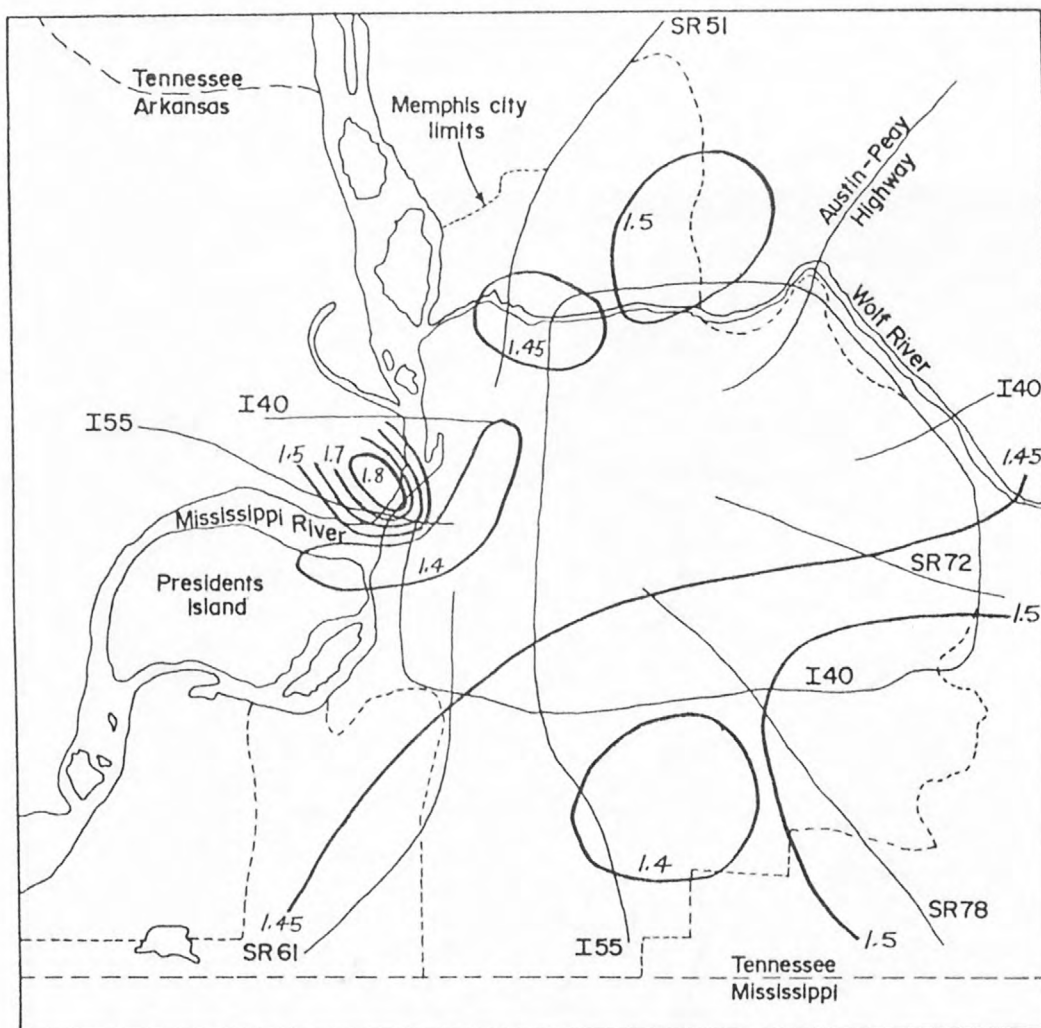
Contours indicate amplification factors for the assigned "bedrock" motion of 18% g.

FIG. 7-2, Ground acceleration amplifications due to Earthquake A1



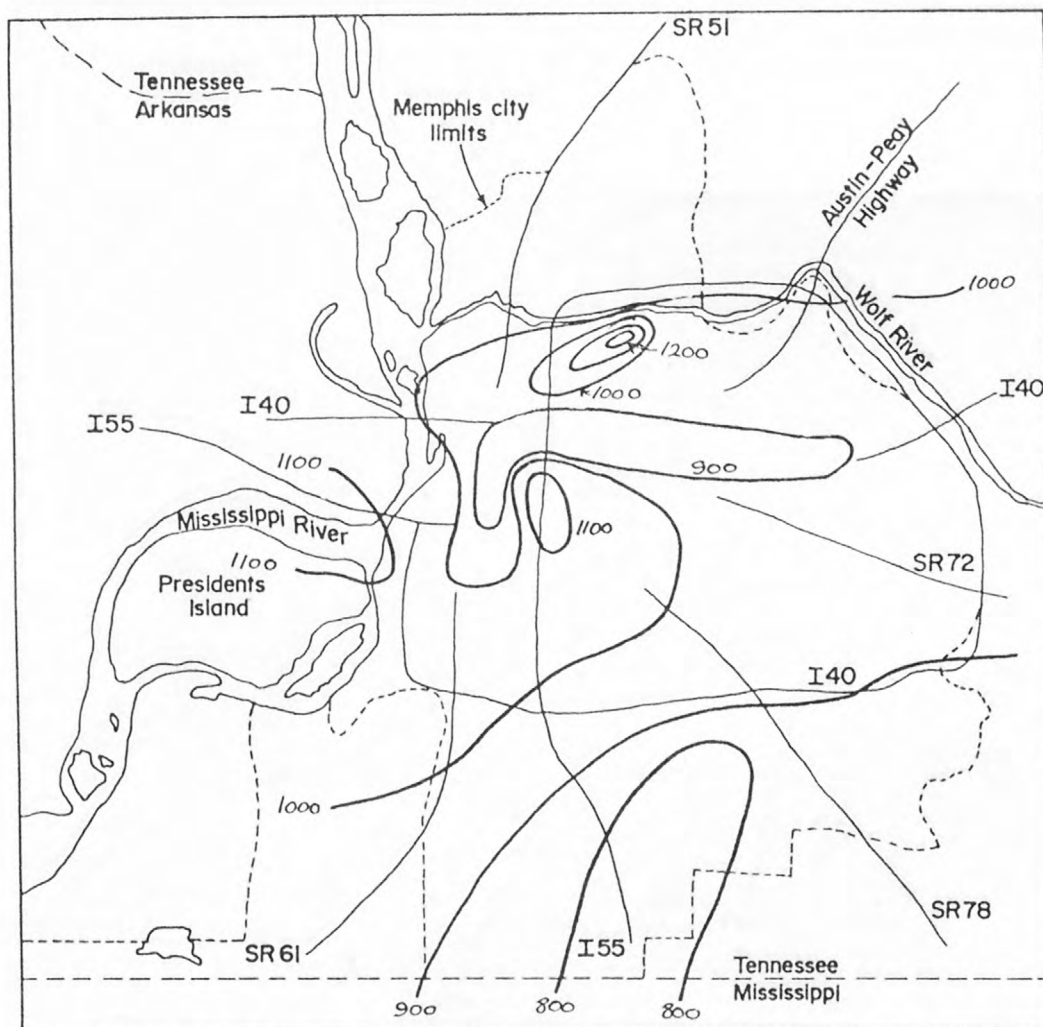
Contours indicate amplification factors for the assigned "bedrock" motion of 14% g.

FIG. 7-3, Ground acceleration amplifications due to Earthquake A2



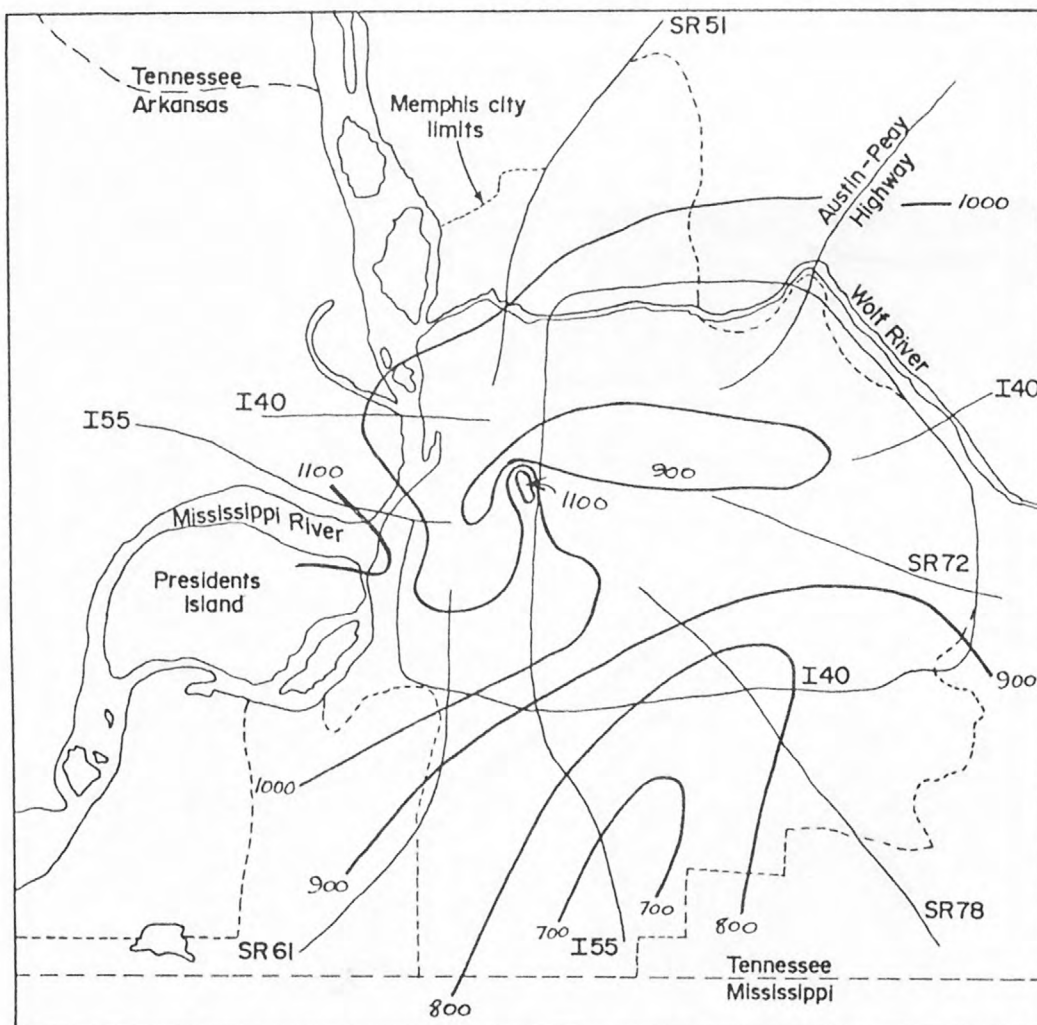
Contours indicate amplification factors for the assigned "bedrock" motion of 11% g.

FIG. 7-4, Ground acceleration amplifications due to Earthquake A3



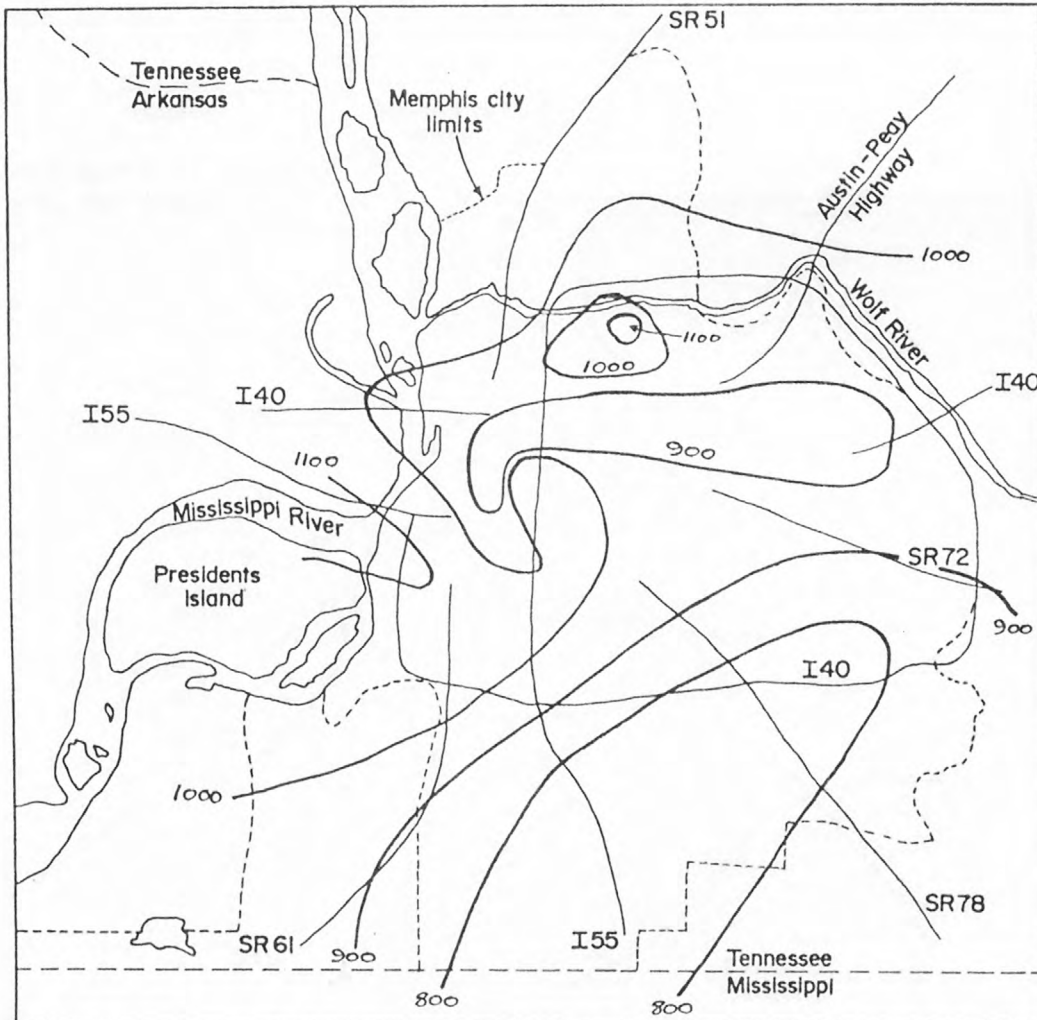
Contours indicate the natural periods of the soil-profiles in 1/1000 of a second.

FIG. 7-5, Natural periods of soils due to Earthquake A1



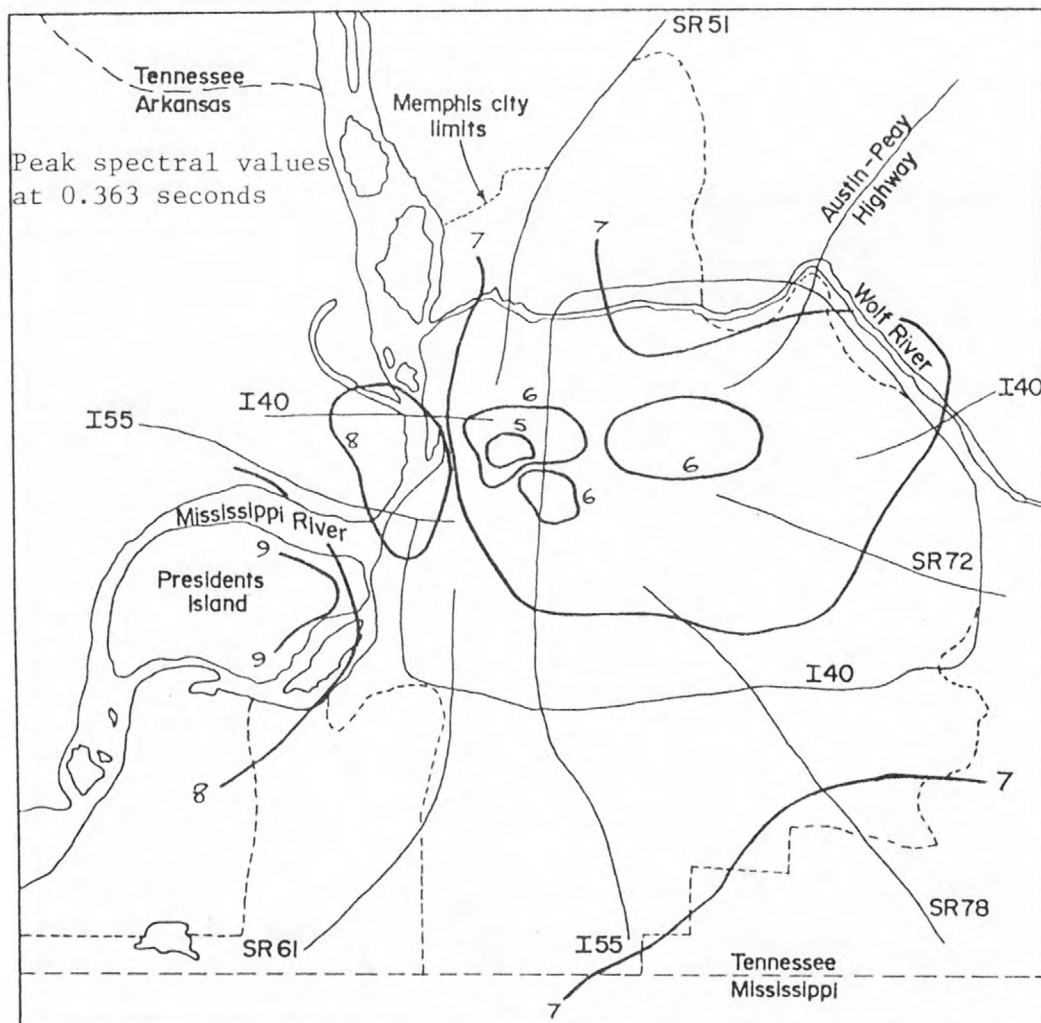
Contours indicate natural periods of the soil-profiles
in 1/1000 of a second.

FIG. 7-6, Natural periods of soils due to Earthquake A2



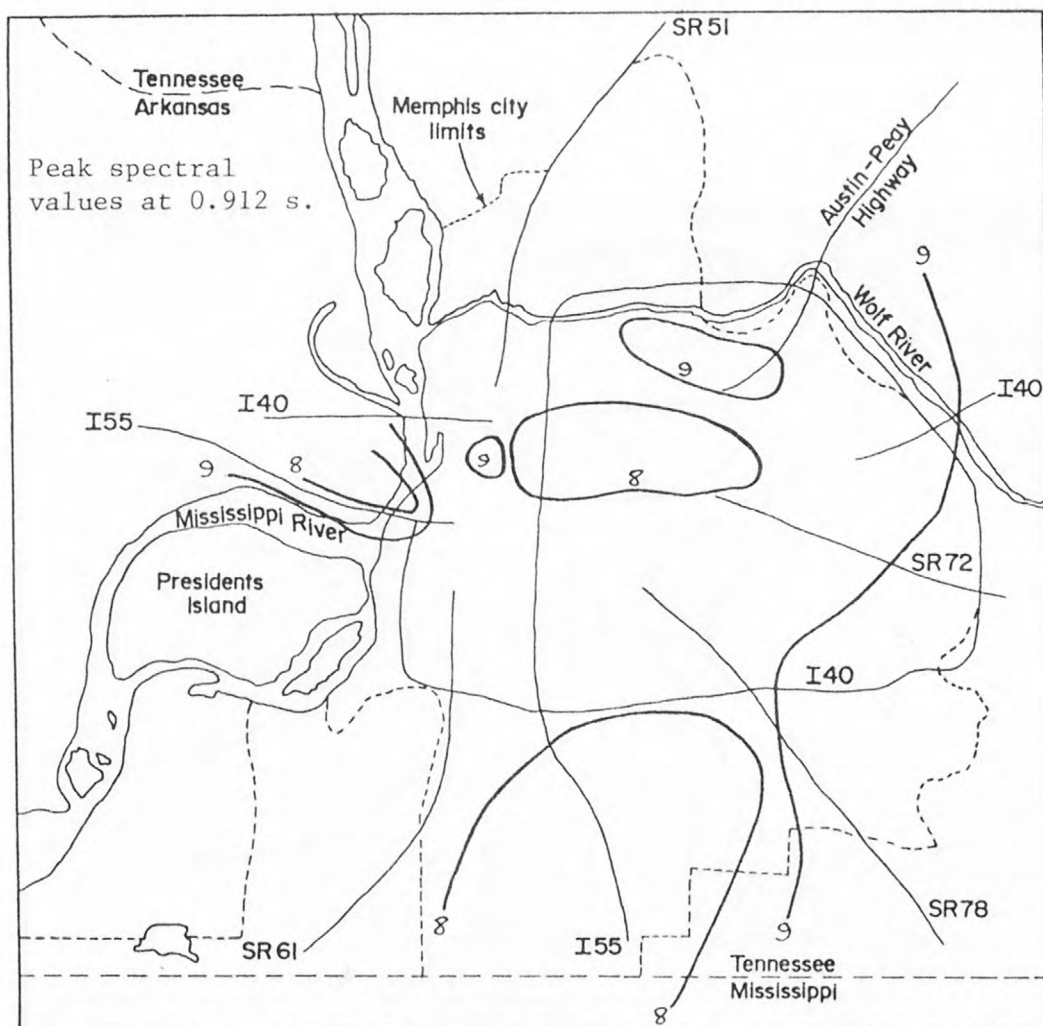
Contours indicate natural periods of the soil-profiles
in $1/1000$ of a second.

FIG. 7-7, Natural periods of soils due to Earthquake A3



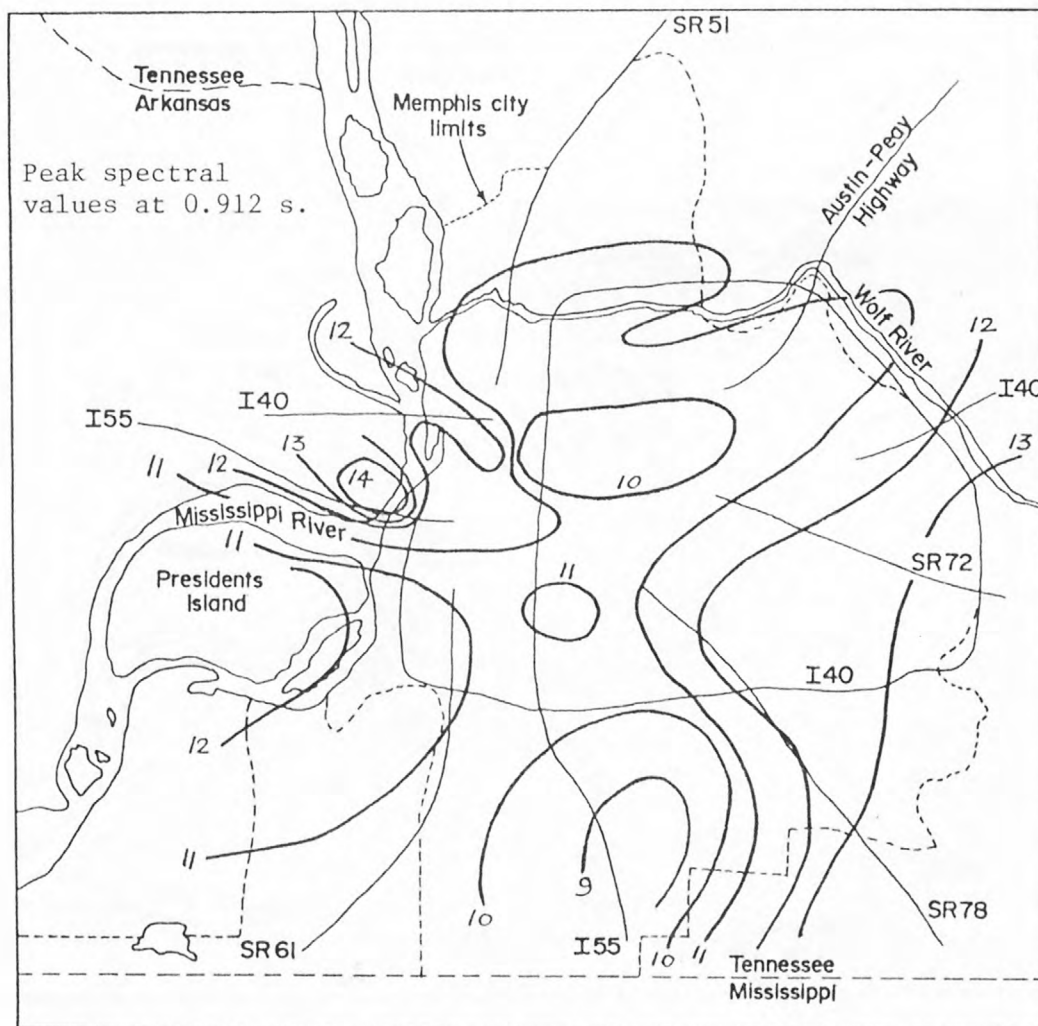
Contours indicate amplification factors for peak-spectral acceleration for a "bedrock" motion of 18% g, assuming 2% damping.

FIG. 7-8, Spectral acceleration amplifications due to Earthquake A1



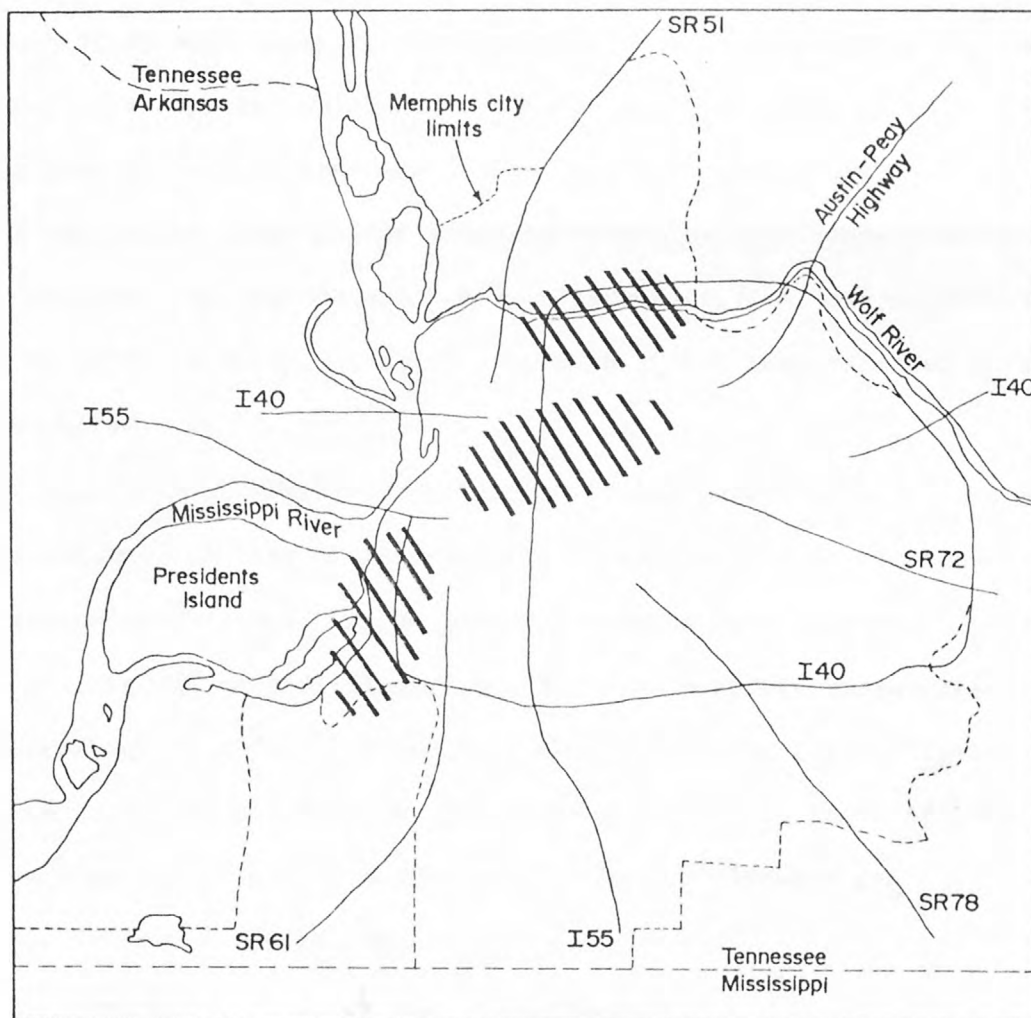
Contours indicate amplification factors for peak-spectral acceleration for a "bedrock" motion of 14% g, assuming 2% damping.

FIG. 7-9, Spectral acceleration amplifications due to Earthquake A2



Contours indicate amplification factors for peak-spectral accelerations for a "bedrock" motion of 11% g, assuming 2% damping.

FIG. 7-10, Spectral acceleration amplifications due to Earthquake A3



Shaded areas indicate zones where soils may be susceptible to liquefaction for earthquakes with Modified Mercalli Intensity greater than VII.
(see discussion in text)

FIG. 7-11, Liquefaction Potential microzonation map

CHAPTER EIGHT

Summary, Conclusions and Recommendations

This study has attempted to microzone the city of Memphis using a relatively new approach which did not include actual field testing. It is evident that if such an approach is found to be satisfactory, it may be readily applied to other cities situated within seismic zones. However, as was evidenced by the Bucharest microzonation studies, any microzonation should be based, ideally, on seismic data which has been recorded at the proposed site.

For the city of Memphis, there is no strong motion seismic data available because of the lack of instrumentation and the high recurrence intervals of hazardous earthquakes. As no strong-motion data has ever been recorded in Memphis, it was very difficult to select the possible representative characteristics of a design earthquake necessary to evaluate site-response.

This study was conducted by considering three main areas of interest, with the time shared evenly between each subject. These are:

- (1) Seismicity of the central U.S.,
- (2) Generation of artificial accelerograms,
- (3) Geology and site-response, which included the collection of borehole data from local sources.

Extensive data pertaining to earthquakes has been collected, but unfortunately, the bulk of this data is only applicable to the western United States. Thus we had to select representative characteristics of a design earthquake almost entirely by inference. Numerous studies, presently in progress, are attempting to clarify and document some of the design

earthquake data. However, the bulk of these studies rely on microtremors. Microtremors produce small strains. Since soil behavior is nonlinear, response cannot readily be extrapolated for the larger design earthquakes. After a very careful study of the 'inferred' seismicity of the area, we were able to construct a model of a possible earthquake to which we have attempted to assign pertinent seismic parameters. Suitable characteristics were selected and their quality was assessed by O. W. Nuttli (personal communication, 1979). As any earthquake appears to be a very random model, it is almost impossible to assign deterministic values to each characteristic. From numerous possible combinations, we have selected design earthquakes to represent what we believe to be a very probabilistic model.

Thus the reader should not rely on the output of this study as a predictor for the hazardous effects of future earthquakes which are likely to affect the Memphis area. For the purpose of this study we are only attempting to illustrate the extent of the variations in ground-shaking which may occur in the city of Memphis. However, we feel that any earthquake which exhibits similar characteristics to those assigned to our design earthquakes will have a very close correlation to the results presented in this report. Another area in our study for which our inference may again be very inaccurate is the use of artificially generated accelerograms.

A large portion of the time allocated for this study was spent on the development of these artificial accelerograms. We evaluated three computer programs extensively in an attempt to successfully generate accelerograms which displayed all the characteristics normally pertinent in recorded accelerograms. After careful evaluation, we excluded two of these programs due to their inconsistent results and the difficulties encountered when possible changes in seismic parameters were contemplated. It is most desirable if procedures within the artificial earthquake computer program

permit an operator to control the durations, amplitudes and the frequency content of the output accelerogram. However, we feel that such control over the generation procedures is very important but not yet available. So we proceeded to amend one of these programs (Ruiz and Penzien, 1969) to allow us more control over the final output. Once this program had been amended, extensive studies were performed in an attempt to generate accelerograms suited to our design earthquakes. Some of the algorithmic concepts of this program are discussed in Appendix B.

In the determination of the necessary dynamic soil properties we have relied exclusively on the Standard Penetration Test. The relationships used to obtain the necessary properties are known to be very deficient in their accuracy. So far as we know, this is the first study which attempts to use these available relationships for a dynamic analysis. It would have been very useful if we could have evaluated the limitations of these assumptions by performing the relevant laboratory tests. However, as laboratory testing was not within the scope of this study, it is left for future studies to investigate these relationships.

Numerous assumptions (previously discussed in Chapter Six) regarding the simplification of the borehole data are also likely to reduce the reliability of the results produced in this study. This simplification was necessary to utilize the data available and also to successfully portray the presumed stratification. Naturally, conservative values were assumed for most of these assumptions and are thus unlikely to result in the exclusion of possible hazardous zones. Then the presumed stratigraphy was subjected to the design earthquake motions.

We have also omitted the effects related to different types of faulting and the associated changes in the radiation patterns which could not be

included within the scope of this study. It is also possible that the largest amplitude waves to Memphis from the New Madrid area might well be the Lg phase. The nature of the propagation of these waveforms are unclear at present and any associated studies can only be by a state-of-the-art approach.

As we are considering horizontal accelerations being propagated from the bedrock to the surface for this study, a simplified shear-model was adopted for the evaluation of site response. The program SHAKE (Schnabel et al, 1972) was used for this analysis.

The test results produced from the response analysis show extensive variations in the response which would be expected in Memphis due to damaging earthquakes. It is imperative that one is certain about our assumptions before any attempt is made to utilize the microzonation maps. The accuracy of the contours is very dependent on the amount of data available and we cannot really imagine the likely location of these contours if substantially less or greater data was made available. Thus it is important to consider these maps in the context that they show the possible variations expected rather than the actual amplification factors suggested. To introduce typical microzonation maps is very difficult, as was realized for the Bucharest study. Hence it is perhaps important to discuss how some of the assumptions and limitations may be reduced for future studies.

The main recommendation which we propose is to permit studies of a similar nature to be performed for other cities for which the seismicity is similar to that experienced in Memphis. This would hopefully preclude almost 50% of the time expended on these subjects during the course of this particular report. Also, it would be interesting to perform an extensive microzonation study similar to the one performed for the Long Beach area.

Such a study would allow laboratory and field testing and thus produce a much more accurate model of the stratigraphy. This would permit an evaluation to be performed on the procedures used to conduct this present study and also act as a guideline for future microzonation studies of areas throughout the world.

Much time and effort was expended in obtaining borehole data from local sources. Their response to the success of this study was found to vary and there was a lot of opposition to the use of previous data. Some of the sources, mostly those who showed considerable interest in the results of such a study, were very helpful in providing the necessary data. However, many of the information sources preferred to maintain their anonymity. Perhaps the city of Memphis could set up a 'data bank' where all future bore-hole data would be stored for future evaluation. The setting up of such an establishment could perhaps be either voluntary or by ordinance and if successful, it will result in a much better model of the complex stratigraphy beneath Memphis.

In some cities, ASCE Geotechnical Engineering Division Sections have been very active and have produced publications summarizing existing bore hole information for the benefit of all. Soil information is readily available for the cities of Chicago, Indianapolis and San Francisco, to name a few.

This study has shown that local soil conditions have a very large influence on the perceived ground motions. Seismic design codes of several countries (including Argentina, Canada, Chile, Cuba, France, Greece, India, Japan, Mexico and Turkey) have provisions which include such amplifications in the assessment of a seismic coefficient. This would then reduce both under and over design in areas where the variation in soil stratigraphy

precludes the use of universal seismic design coefficients. Only the city of Long Beach in the United States includes such measures.

This study is essentially a first-attempt to evaluate the seismic-microzonation of Memphis, Tennessee, and thus should be viewed only as a preliminary analysis. As microzonation is a very complex analysis, a future multi-disciplinary approach is recommended to enable a more design oriented outlook. Nevertheless, we hope that this study will contribute to a greater understanding of how to reduce earthquake hazards to the environment and also increasing safety.

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Appendix A: Source Region Coordinates

1. The polygonal source regions are described by specifying their corners in terms of latitude and longitude pairs. For consistency, these corners are ordered such that each successive pair is the next corner, when one progresses clockwise about the source region.

1. NEW MADRID A: (35.5, 91.0), (37.0, 89.5), (36.5, 88.5),
(35.0, 90.0).
2. NEW MADRID B: (35.5, 91.5), (37.5, 89.5), (36.5, 88.0),
(34.5, 90.0); less region covered by New Madrid A.
3. WABASH VALLEY: (39.0, 88.0), (39.6, 87.5), (39.6, 86.5),
(38.5, 87.0), (36.5, 88.0), (37.5, 89.5).
4. OZARK UPLIFT: (37.0, 91.5), (39.0, 89.5), (38.5, 88.5),
(35.5, 91.5).
5. OUACHITA-WICHITA: (35.0, 103.0), (37.0, 103.0), (35.0, 90.0),
(33.0, 90.0).

(from Nuttli and Herrmann, 1978)

Appendix B

Computation of Artificial Accelerograms

The artificial accelerograms were prepared using the computer program developed by Ruiz and Penzien, 1969. The program allows the reader to select three components which are expected to simulate the characteristics normally exhibited by those accelerograms which are recorded from natural earthquakes.

The computation attempts to create a linear stochastic model, of a stationary mode, to generate records of a filtered shot noise. After the initial generation of a Gaussian shot-noise, the 'noise' is filtered and shaped to produce a set of values which illustrate those anticipated during a seismic event. These values are then shaped to display qualities which are normally inherent in recorded accelerograms.

The shaping process produces an initial build-up during which the accelerations are permitted to increase linearly to the desired maximum value. After the initial build-up follows a stationary period during which the intensity of the proposed maximum acceleration is maintained. At the end of this period, the values are allowed to decay exponentially to essentially a zero acceleration. However, further corrections are also necessary for ensuring that the initial and final velocities (based on the integration of the acceleration-time history) are always zero.

The program corrects this inaccuracy by applying a parabolic base line correction where the coefficients of the correction are chosen to minimize the mean square value of the velocity. However, for the accelerograms generated for this study, this correction was found to be inadequate as the velocity did not attain a zero-value at the end of the time-history. Thus another correction was performed by essentially applying a linear correction based on a least squares fit. This was an iterative process, correcting the

acceleration and velocity time-histories in conjunction until the final velocity became equal to zero. This correction will probably alter the frequency content of the generated wave but as each accelerogram was treated individually an allowance for this uncertain error was included.

The time-histories generated for this study are designed to exhibit the maximum accelerations proposed for the different earthquakes. Table B-1 lists the shaping values which were selected for this study. However, it is important to realize that the natural frequency relates to the filter and cannot be directly used to vary the predominant frequency of the output time-history. The times indicated in Table B-1 are illustrated in Fig. B-1 where the reader can readily visualize the exact configuration of the method pertaining to the generation of these accelerograms.

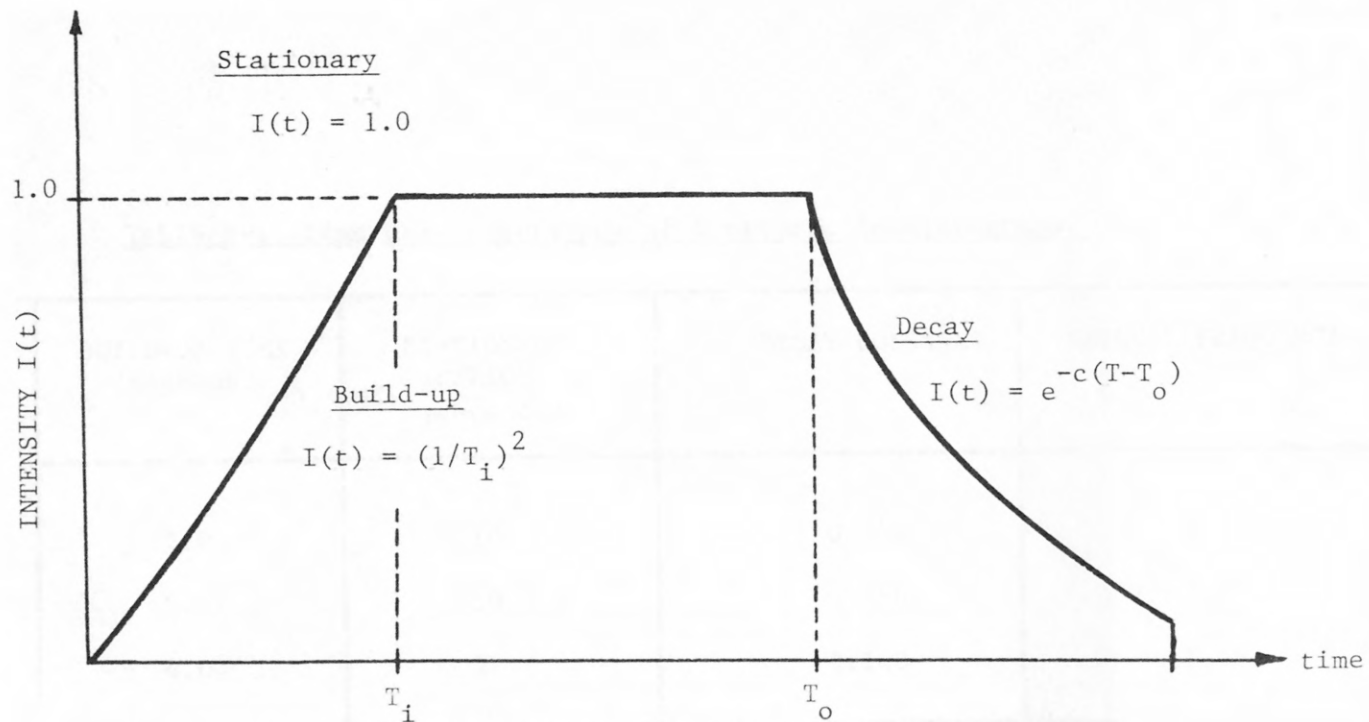


FIG. B-1, Definition of time characteristics used for generation of synthetic accelerograms (Ruiz and Penzien, 1969)

Table B-1, Time Characteristics of Synthetic Accelerograms.

| EPICENTRAL DISTANCE | BUILD-UP TIME (seconds) | STATIONARY PERIOD (seconds) | DECAY CONSTANT | NATURAL FREQUENCY |
|------------------------|----------------------------|-----------------------------------|----------------|-------------------|
| 50 km. | 5.0 | 10 | 0.155 | 2.5 |
| 100 km. | 5.0 | 10 | 0.150 | 1.75 |
| 200 km. | 4.0 | 21 | 0.140 | 1.40 |

Appendix C

Evaluation of Liquefaction Potential

A simplified procedure (Seed, et al, 1976) was used to evaluate the risk of liquefaction potential at various zones in Memphis. This method involves the comparison of the field conditions at the site under investigation with data concerning soil conditions at sites of known field performance in past earthquakes supplemented with the data from numerous large-scale tests.

This evaluation is performed with the use of Fig. C-1 which is a summary of sites at which liquefaction was reported, in areas throughout the world, as a result of earthquakes. Once detailed evaluations of the stress conditions and liquefaction characteristics (i.e., loose saturated sands) are available, this figure provides a basis for an overall evaluation.

We used Fig. C-1 to evaluate the risk of liquefaction by correcting the Standard Penetration resistance (N_{SPT}) to an effective overburden pressure of 100 kN/m^2 by means of the following expression:

$$N_C = (1 - 1.25(\log \sigma'_o - 2.0))N_{SPT}$$

where N_C = corrected penetration resistance.

N_{SPT} = Standard Penetration Resistance at depth under consideration.

σ'_o = vertical effective overburden pressure in kilonewtons per square meter at the location where the penetration resistance has been measured.

For a representative sand layer where liquefaction is suspected, it is also possible to determine the pertinent cyclic stress ratio for the relevant ' N_{SPT} ' value. During a response analysis computation using the program SHAKE, the maximum cyclic stress, τ_{max} , (or strain) may be computed

for the suspected layer. The stress ratio specified in Fig. C-1 is the average cyclic stress ratio which is given by:

$$\frac{\tau_{\text{average}}}{\sigma'_o} = 0.65 \cdot \frac{\tau_{\text{max}}}{\sigma'_o} \cdot r_d$$

where r_d = a stress reduction factor varying from a value one at the ground surface to a value of 0.9 at depth of 9 meters and a value of 0.75 at 15 meters.

Thus for any given value of maximum cyclic stress ratio and the pertinent " N_{SPT} " value it is relatively easy to evaluate the possibility of liquefaction.

To calculate these results by an analytical procedure is much more complex and not within the scope of this study. Hence we have evaluated the liquefaction possibilities by exclusive use of the procedure outlined above.

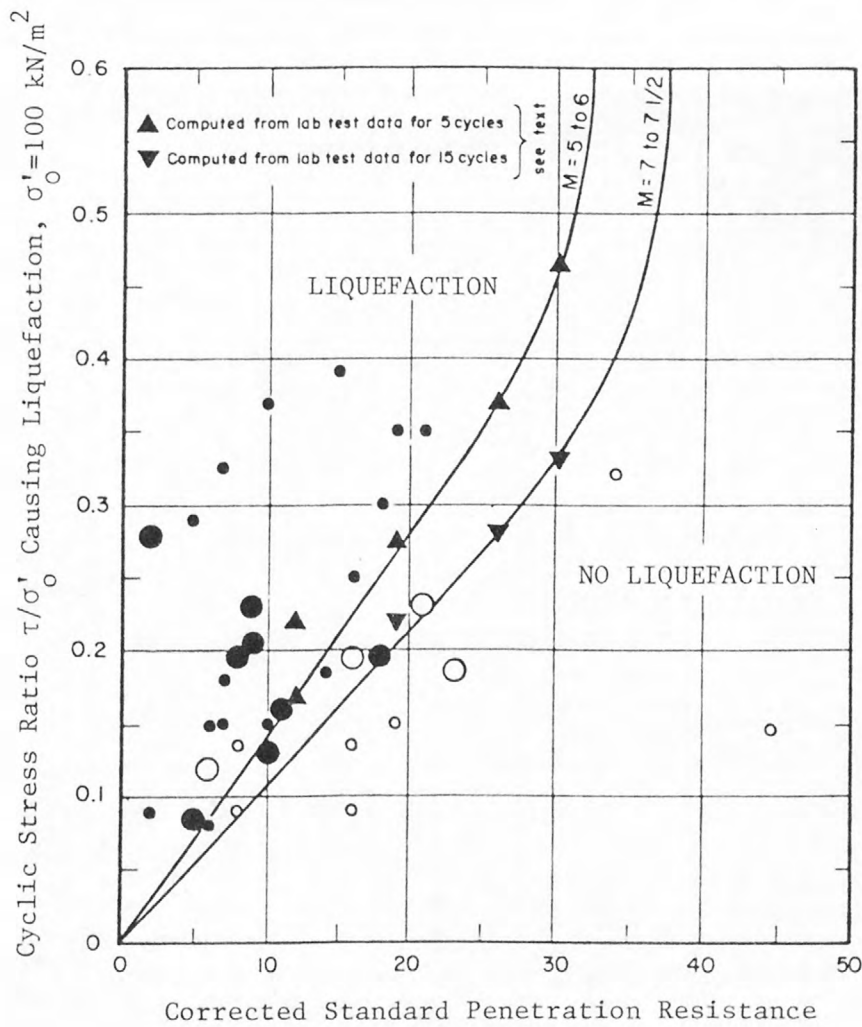


FIG. C-1, Correlation between cyclic stress ratio and penetration resistance of sand causing liquefaction

(after Seed et al, 1976)

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