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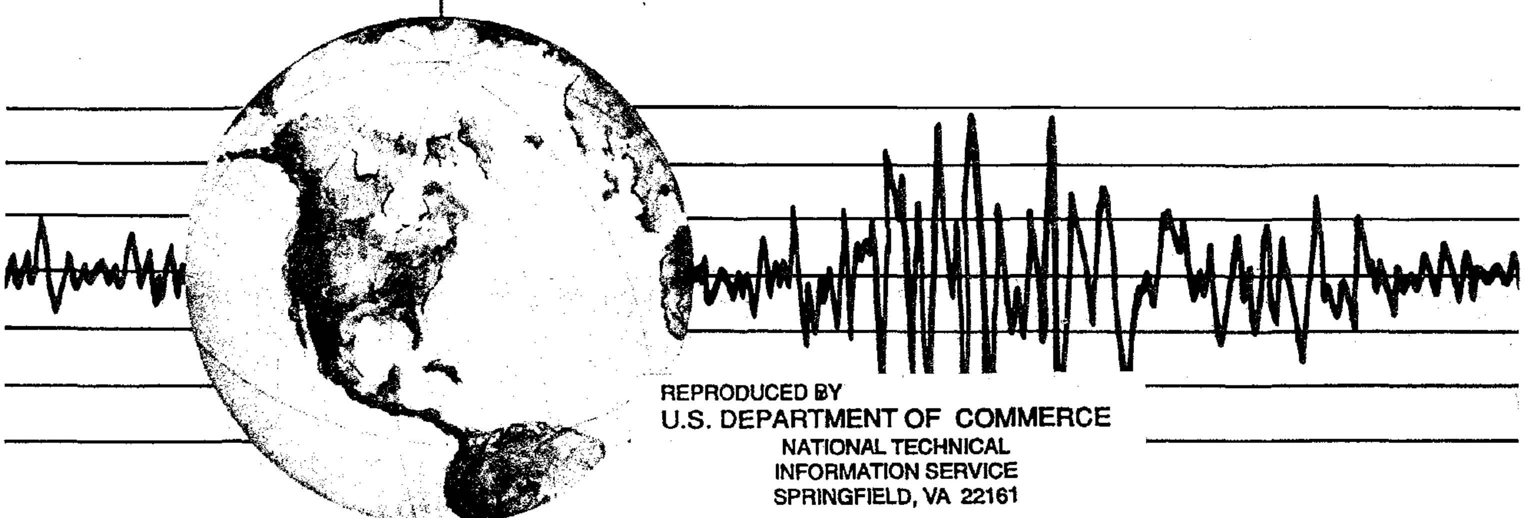
EARTHQUAKE ENGINEERING RESEARCH CENTER

IMPLICATIONS OF SITE EFFECTS IN THE MEXICO CITY EARTHQUAKE OF SEPT. 19, 1985 FOR EARTHQUAKE-RESISTANT DESIGN CRITERIA IN THE SAN FRANCISCO BAY AREA OF CALIFORNIA

by

H. BOLTON SEED
JOSEPH I. SUN

A report on research sponsored by
the National Science Foundation



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One of the most dramatic aspects of the earthquake effects in the Mexico City earthquake of September 19, 1985 was the enormous differences in intensities of shaking and associated building damage in different parts of the city. This report examines the factors which are likely to have influenced the response and degree of damage to structures in the heavy damage area of Mexico City in the earthquake of 1985, attempts to relate these factors to the intensity of damage which occurred, uses the results of the studies to examine the possible extent of damage to structures constructed on sites underlain by clay in other seismic regions, such as the San Francisco Bay area, which like Mexico City, is located near the edge of a deep deposit of clay soil, examines the implications of structural performance in Mexico City for buildings in San Francisco in the light of the seismicity of the region, and examines the effects of possible modifications in building codes which might seem desirable in the light of the Mexico City disaster in 1985.

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TABLE OF CONTENTS

IMPLICATIONS OF SITE EFFECTS IN THE MEXICO CITY EARTHQUAKE OF SEPT. 19, 1985 FOR EARTHQUAKE-RESISTANT DESIGN CRITERIA IN THE SAN FRANCISCO BAY AREA OF CALIFORNIA

by

H. Bolton Seed[1] and Joseph I. Sun[2]

I. INTRODUCTION

One of the most dramatic aspects of the earthquake effects in the Mexico City earthquake of September 19, 1985 was the enormous differences in intensities of shaking and associated building damage in different parts of the city. In the south-west part of the city ground motions were moderate and building damage was minor. However in the north-west part of the city, catastrophic damages occurred and a record of the earthquake motions near the southern end of this heavy damage area showed a very high intensity of shaking. Similar patterns of building damage intensities have been observed in previous earthquakes and the differences attributed to the differences in soil conditions in different parts of the city. In the 1985 Mexico earthquake these differences seem to be somewhat more accentuated than in other earthquakes in the past 40 years, and the availability of recordings of ground motions in different parts of Mexico City makes it possible to explore, in greater detail than heretofore, the relationships between soil conditions, intensities of shaking, and the associated extent of structural damage.

Analyses of ground response for five sites in Mexico City in relation to the soil conditions at the recording stations (Seed et al., 1987) have

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shown that if allowance is made for possible small deviations from the best average deterministic ground response analysis parameters, and ground response is considered on a probabilistic basis, ground response analyses can provide very useful data for assessing the influence of local soil conditions on the characteristics of the ground motions likely to develop at sites in the old lake-bed area of Mexico City where motions varied widely depending on the depth and stiffness of the clay deposits. Furthermore, because of the generally good results obtained in using ground response analyses to predict ground motions for the five sites at which motions and soil characteristics are known in Mexico City, the same procedures can be expected to provide a good basis for predicting motions at sites where motions were not recorded in the September 19, 1985 earthquake. Thus it has been possible to make analyses for a number of different soil depths existing in the heavy-damage area of Mexico City and to develop a representative spectrum for the average ground motions occurring in this area in the 1985 earthquake (Seed et al., 1987).

Within the heavy damage area itself, the intensity of structural damage was found to be different for structures of different heights, presumably reflecting the influence of the soil conditions, the intensity and frequency characteristics of the ground motions, the characteristics of the structures and the criteria controlling the design of the structures.

It is the purpose of this report to examine the factors which are likely to have influenced the response and degree of damage to structures in the heavy damage area of Mexico City in the earthquake of 1985, to attempt to relate these factors to the intensity of damage which occurred, to use the results of the studies to examine the possible extent of damage to structures constructed on sites underlain by clay in other seismic regions, such as the San Francisco Bay area, which like Mexico City, is located near the edge of a deep deposit of clay soil, to examine the implications of structural performance in Mexico City for buildings in San Francisco in the light of the seismicity of the region, and to examine the effects of possible modifications in building codes which might seem desirable in the light of the Mexico City disaster in 1985.

II. RELATIONSHIP BETWEEN DAMAGE INTENSITY, GROUND MOTIONS AND DESIGN LATERAL FORCE COEFFICIENTS FOR BUILDINGS IN THE HEAVY DAMAGE ZONE OF MEXICO CITY

Damage Intensity in the Heavy Damage Area of Mexico City

The location of the heavy damage area in Mexico City in the 1985 earthquake is shown in Fig. 1. Following the earthquake, a detailed survey was made of the intensity of damage to different classes of structures in different parts of the city (Borja-Navarrete, et al., 1986). The damage statistics for the heavy damage zone of the city are summarized in Table 1, which shows, for buildings with different story height ranges, the damage intensity, defined as the ratio of the number of structures in any given category which suffered major damage divided by the total number of structures in that category existing in the heavy damage zone.

It is readily apparent that it was the mid-height buildings, with about 6 to 20 stories, which suffered the highest damage intensities. This trend is also clearly evidenced by the plot of these data shown in Fig. 2.

Since most seismic design procedures for buildings are based on structural period rather than building height, it is useful to examine the natural periods of the structures in Mexico City in relation to the number of stories of the buildings. Emphasis will be placed on large-deformation periods since it is these periods which are most indicative of building behavior during major earthquakes (Bertero et al., 1988).

For North American practice, the fundamental period (in seconds) for typical buildings is typically about N/10, where N is the number of stories. However, in Mexico City, buildings are somewhat less stiff as compared to United States practice, the foundation soils are much more compressible, in-fill walls tend to crack early in an earthquake and buildings become less

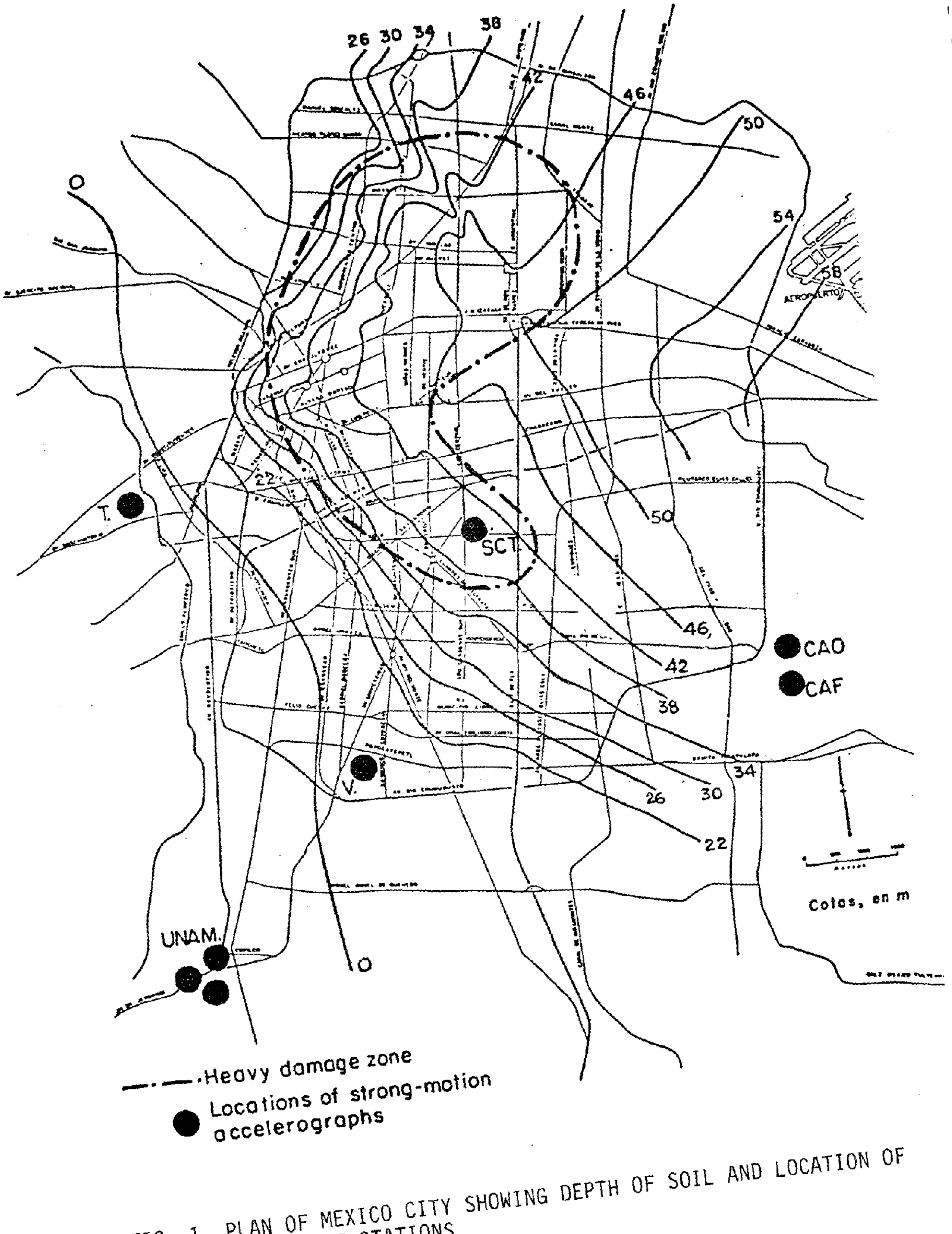


FIG. 1 PLAN OF MEXICO CITY SHOWING DEPTH OF SOIL AND LOCATION OF INSTRUMENT STATIONS

TABLE 1 DAMAGE STATISTICS FOR THE HEAVY DAMAGE AREA OF MEXICO CITY IN THE SEPT. 19, 1985 EARTHQUAKE

(modified after Borja-Navarrete et al., 1986)

Number of Stories	Number of Bldgs. with Serious Damage	Total Number of Buildings	Damage Intensity
1 - 2	≈ 297	≈ 15,000	≈ 2 %
3 - 5	≈ 154	≈ 5,400	≈ 3 %
6 - 8	≈ 117	≈ 650	≈ 18 %
9 - 12	≈ 62	≈ 215	≈ 29 %
> 12	≈ 21	≈ 92	≈ 23 %

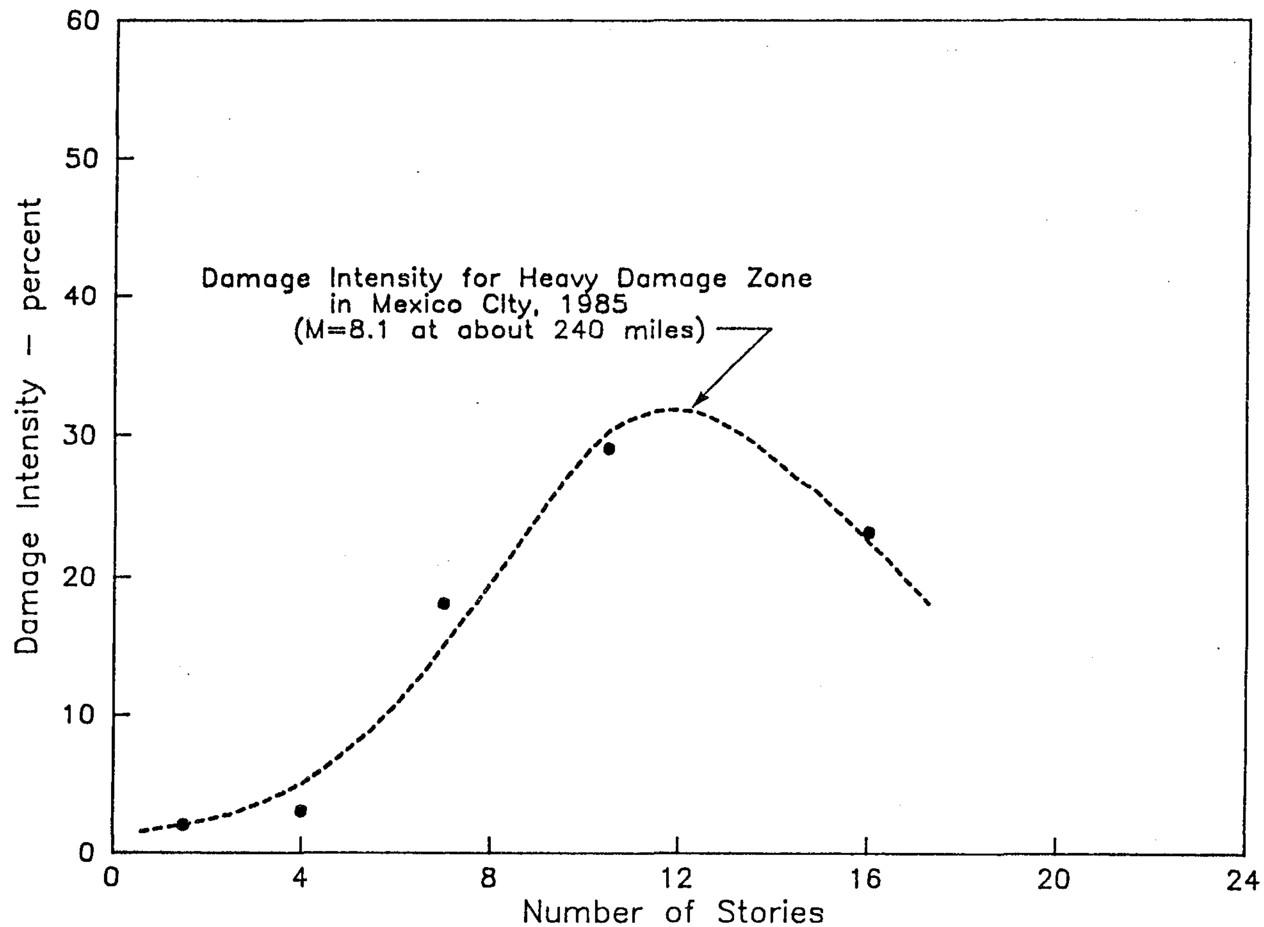


FIG. 2 DAMAGE INTENSITY FOR HEAVY DAMAGE ZONE IN MEXICO CITY DURING THE SEPT. 19, 1985 EARTHQUAKE

stiff, while periods also lengthen if the structures go beyond the elastic range. Thus the "effective" building periods for structures in Mexico City can be expected to be significantly longer than those normally estimated for U.S. structures. Quantitative estimates of these effects are shown in Table 2, and it seems reasonable to expect that the effective building periods for many structures in the heavy damage area of Mexico/City is more likely to have been of the order of N/6 (seconds) where N again represents the number of stories. Taking this into account, the data in Fig. 2 are replotted in Fig. 3 to show the damage intensity as a function of effective building periods. It is evident that structures suffering the highest damage intensities were those with fundamental periods in the range of 1.5 to 2.5 seconds.

Evaluation of Potential for Building Damage Due to Earthquake Shaking

In previous studies of earthquake damage intensity in relation to ground motions it has been suggested that a useful index of the vulnerability of a structure to damage caused by earthquake shaking can be evaluated in terms of a simple ratio establishing a Damage Potential Index (Seed et al., 1970): This index, which incorporates the idea of capacity (the forces that a building is designed to withstand) and demand (the forces induced on the building by the earthquake shaking), was found to be extremely useful in examining damage in Caracas, Venezuela (Seed et al., 1970) and it will therefore be used, with minor modifications in the present study.

In the approach proposed by Seed et al., 1970 for ductile buildings, of the type which suffered major damage in Mexico City, the Damage Potential Index (DPI) is evaluated as follows:

TABLE 2 ESTIMATION OF BUILDING PERIODS IN HEAVY DAMAGE AREA OF MEXICO CITY WITH RESPECT TO NUMBER OF STORIES IN THE SEPT. 19, 1985 EARTHQUAKE

For U.S. structures, $T \approx N / 10$, where N = no. of stories	
Conditions in Mexico City	**Effect on Building Period**
1. Soil more compressible	Increase by about 50%
2. Infill walls crack easily	Increase by about 30%
3. Building go beyond elastic range	Increase by about 30%
Thus for conditions in the heavy damage area of Mexico City: Effective building period $\approx$ N/5 to N/6 seconds	

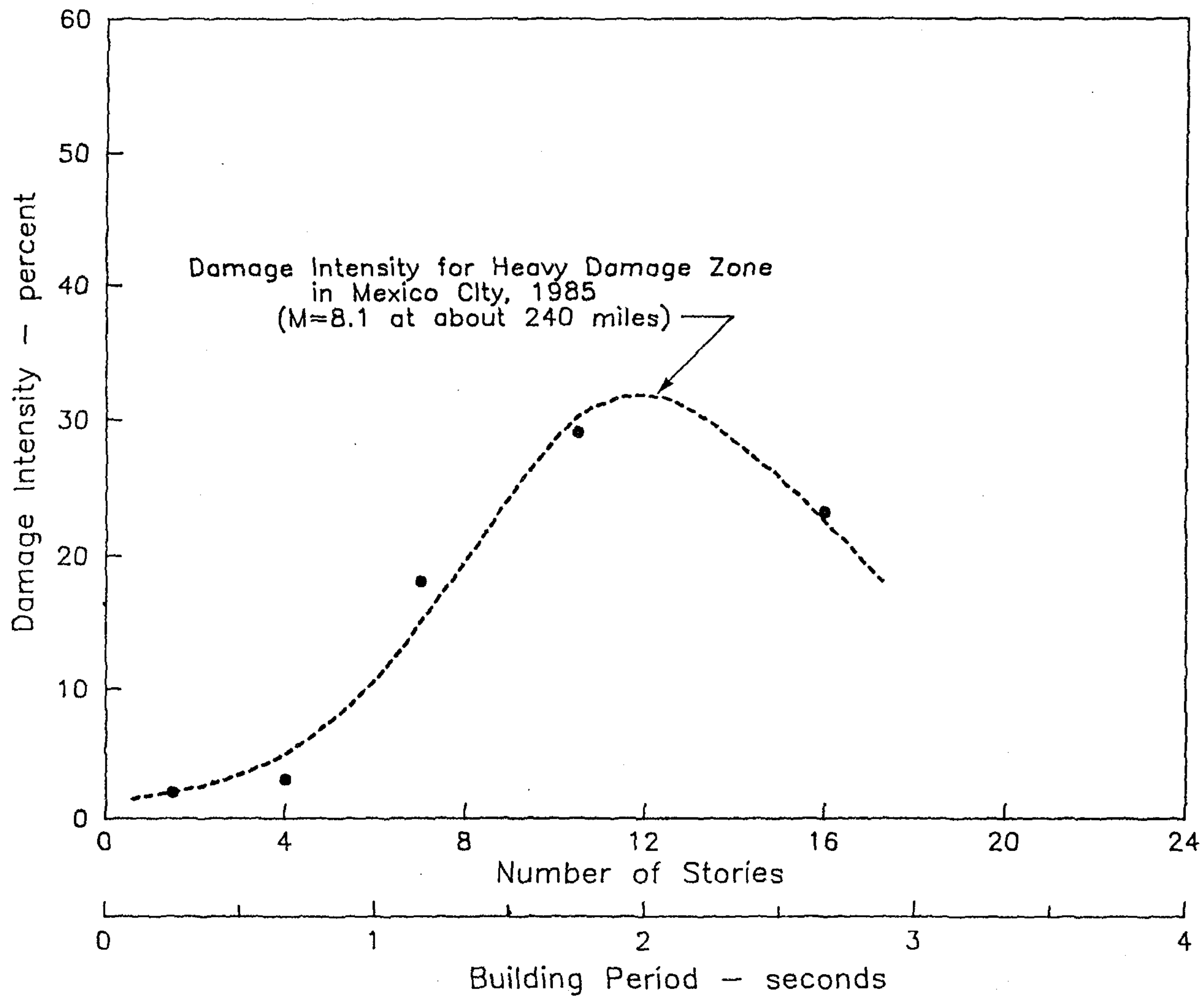


FIG. 3 DAMAGE INTENSITY FOR HEAVY DAMAGE ZONE IN MEXICO CITY DURING THE SEPT. 19, 1985 EARTHQUAKE IN RELATION TO ESTIMATED BUILDING PERIOD IN MEXICO CITY

$$\text{Force induced on building by earthquake} \quad \propto \quad W \cdot \frac{S_a}{g}$$

$$\text{Damaging Potential of ground motion} \propto \text{(Induced Force)} \times \text{(Duration of force)}$$

$$\propto \quad W \cdot \frac{S_a}{g} \times T \times \text{No. of cycles}$$

where the duration of the force is considered to be proportional to the period of the building and the number of load cycles induced by the earthquake. The number of cycles will be determined primarily by the duration of shaking during the earthquake and this in turn will depend on the Magnitude of the earthquake (Bolt, 1973). The damaging potential of the ground motion, as expressed above, can be considered an approximate expression of the Demand imposed on a structure by the earthquake shaking.

The design resistance of a structure (Capacity) is usually determined by the building code requirements and for most codes, including the Mexico City code, it is expressed as follows:

$$\text{Design Lateral Force} = k \cdot W$$

where k is the design lateral force coefficient. Generally speaking, the higher the design lateral force coefficient, the greater is the capacity of a structure to withstand the effects of earthquake shaking.

The capacity will also depend, however, on the load combinations and the allowable stresses prescribed by the Building Code. In any one city these will be the same for all structures, but in comparing structures in different cities, the relative values of these factors, which also affect design resistance, will have to be taken into account. On a comparative basis they can be expressed by a factor termed the building resistance factor, R_f, which expresses the relative design resistances as they are affected by allowable stresses and load combinations, all other factors being equal. Thus the capacity of a building to withstand earthquake damage can be expressed by:

$$\text{Design Resistance} \quad \propto \quad k \cdot W \cdot R_f$$

and the vulnerability of a structure to earthquake damage can be expressed in an approximate way by the ratio of Demand/Capacity, leading to the development of a Damage Potential Index as follows:

$$\text{Damage Potential Index} \approx \frac{\text{Induced Force x Duration of Force}}{\text{Design Resistance}}$$

$$\approx \frac{W \cdot S_a/g \text{ x } T \text{ x No. of cycles}}{k \cdot W \cdot R_f}$$

$$\approx \frac{S_v}{k\ R_f} \cdot \text{Duration Weighting Factor}$$

where the Duration Weighting Factor reflects the influence of the duration of shaking and is a function of the earthquake magnitude. Since the intent of this index is only to compare the relative vulnerabilities of different structures, the Duration Weighting Factor can be assigned relative values, based on judgment, which are determined by the Magnitude of the earthquake involved. Suggested values of the Duration Weighting Factor (DWF) are listed in Table 3. It is recognized that other engineers may make different estimates of the effects of duration of shaking on potential damage, but the values shown in Table 3 will be used in the present study as a first approximation, and the results obtained with this approach will be compared with the actual damage statistics for the heavy damage area of Mexico City in the 1985 earthquake.

TABLE 3 SUGGESTED VALUES OF DURATION WEIGHTING FACTOR (DWF) FOR STRONG GROUND MOTIONS WITH DIFFERENT DURATIONS

Duration	DWF
15 sec	0.6
40 sec	1.0
65 sec	1.35
120 sec	2.0

In addition, since the term R_f is intended to express relative values of design resistance in different cities or areas, it is most convenient to assign this factor a value of unity for Mexico City; values for other cities and areas would then be somewhat higher or lower than unity depending on their individual Code requirements. Thus, for example, based on the Code requirements for California (SEAOC, 1980), the value of R_f applicable for reinforced concrete structures in California might be estimated to be about 1.3 (Bertero, 1988). For other areas appropriate values could be assessed by knowledgeable structural engineers familiar with Code requirements in Mexico City and local design codes.

Seismic Code Provisions for Mexico City

The first seismic design provisions adopted for use in Mexico City were developed in 1942, and they have been under constant revision since that time (1957, 1966 and 1976). The 1976 code microzoned the Federal District of Mexico into three parts: (1) the hilly and hard soil or rocky zone; (2) the transition zone and (3) the lake bed zone, in recognition of past experience which indicated that different intensities of shaking developed in the city depending on the subsoil conditions. Table 4 shows the required lateral force coefficients (the ratio of design lateral force to total building weight) for buildings located in the lake-bed zone. For all multi-story buildings having periods longer than 1 second, the required design lateral force is equivalent to 6% of the building's weight. Lower design requirements were used for the hilly zone and the transition zone.

Ground Response in the Heavy Damage Area of Mexico City

In a previous report (Seed et al., 1987), ground response analyses were performed to study the ground motions developed in the lakebed areas of

Mexico City and for the heavy damage area of Mexico City during the 1985 earthquake. Fig. 4 shows, in terms of acceleration response spectra, the recorded motions at the SCT recording station, located within the heavy damage zone, together with the spectra for computed motions likely to have been developed for clay depths ranging from 25 to 45 meters. Based on these results a representative average response spectrum was determined which could be considered to represent the general characteristics of the earthquake motions in the heavy damage zone, as shown in Fig. 4. These spectral accelerations can readily be converted to spectral velocities to evaluate Damage Potential Index values for buildings in this zone.

Damage Potential Index for Heavy Damage Zone in Mexico City

The Damage Potential Index (DPI) has been defined previously as (see page 12) as:

$$\text{DPI} = \frac{S_v}{k \cdot R_f} \cdot \text{Duration Weighting Factor}$$

and DPI thus has the units of velocity.

For Mexico City, R_f has been assigned a value of 1 in this study. Thus with the aid of spectral velocities determined from the results shown in Fig. 4 and lateral force coefficients determined from Table 4, values of the Damage Potential Index for buildings with different periods can readily be determined for buildings in the heavy damage zone of Mexico City as shown in Table 5. The results shown in Table 5 are plotted in Fig. 5 to show the computed Damage Potential Index values for buildings with a wide range of periods. It can be seen that buildings which have large-deformation natural periods in the neighborhood of 2 seconds exhibit the highest damage potentials, which corresponds well with the damage observed in Mexico City. The

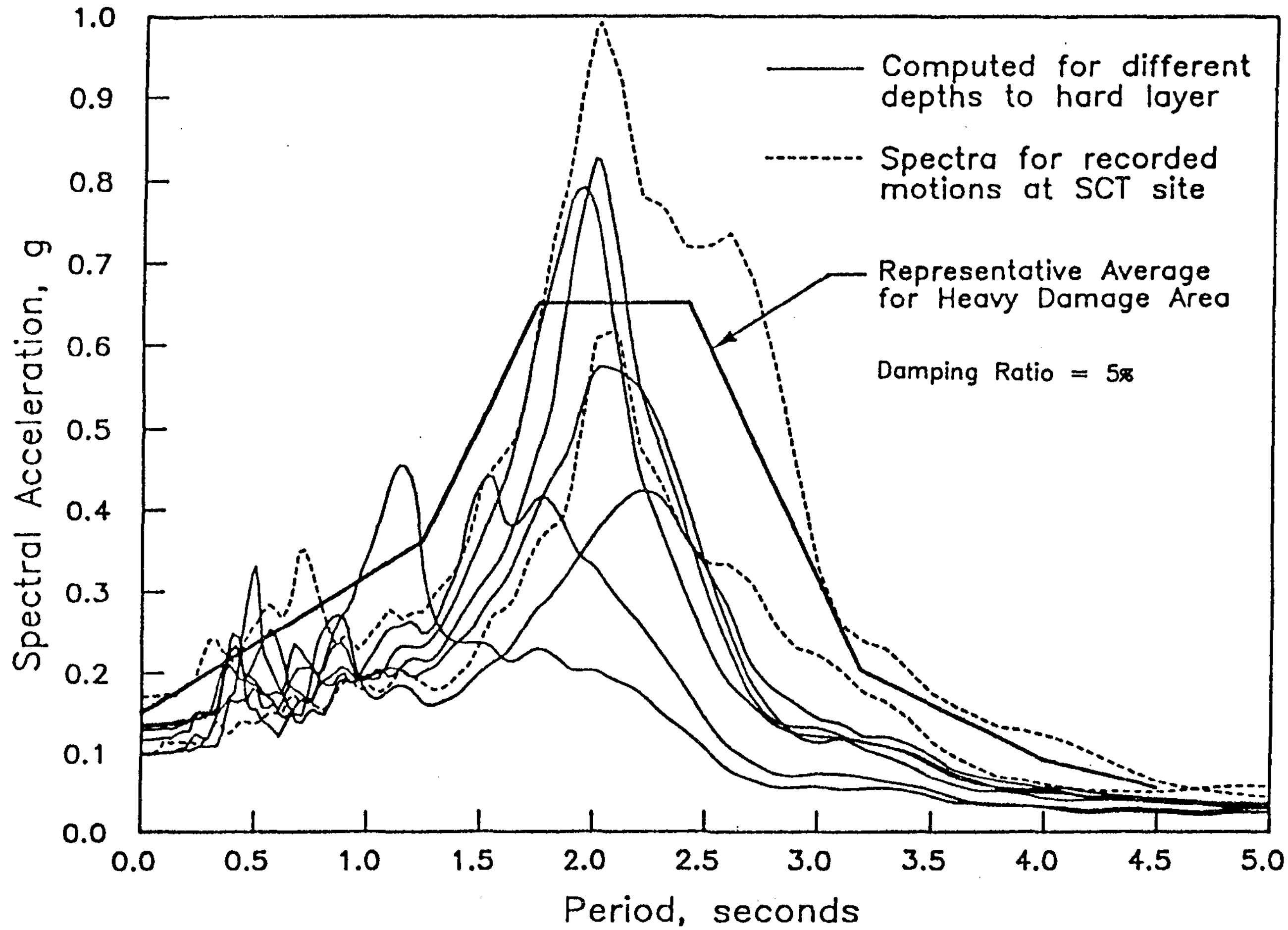


FIG. 4 COMPUTED SPECTRA FOR HEAVY DAMAGE AREA FOR SOIL DEPTHS RANGING FROM 25 TO 45 METERS AND EVALUATION OF REPRESENTATIVE AVERAGE SPECTRUM FOR HEAVY DAMAGE AREA

TABLE 4 DESIGN LATERAL FORCE COEFFICIENTS FOR MEXICO CITY LAKEBED ZONE (1976-1985)

Building Period (seconds)	Design Lateral Force Coefficient
0.0	0.030
0.5	0.045
1.0	0.060
1.5	0.060
2.0	0.060
2.5	0.060
3.0	0.060
3.5	0.060

TABLE 5 CALCULATED DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY IN THE 1985 EARTHQUAKE

MEXICO CITY - 1985

For Representative Spectrum in Heavy Damage Part of the City

Building Period	Spectral Accelera-tion	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	(Sv/k•R)•DWF (fps)
0.0	0.15	0.00	0.030	0
0.5	0.23	0.59	0.045	26
1.0	0.31	1.61	0.060	54
1.5	0.50	3.84	0.060	128
2.0	0.65	6.66	0.060	222
2.5	0.60	7.69	0.060	256
3.0	0.30	4.61	0.060	154

Notes:

1. The Duration Weighting Factor (DWF) used is 2.0 for the duration of strong ground motions in the 1985 earthquake, which lasted over 2 minutes.
2. The building resistance factor (R_f) is assigned a value of 1.0 for typical reinforced concrete structures in Mexico City.

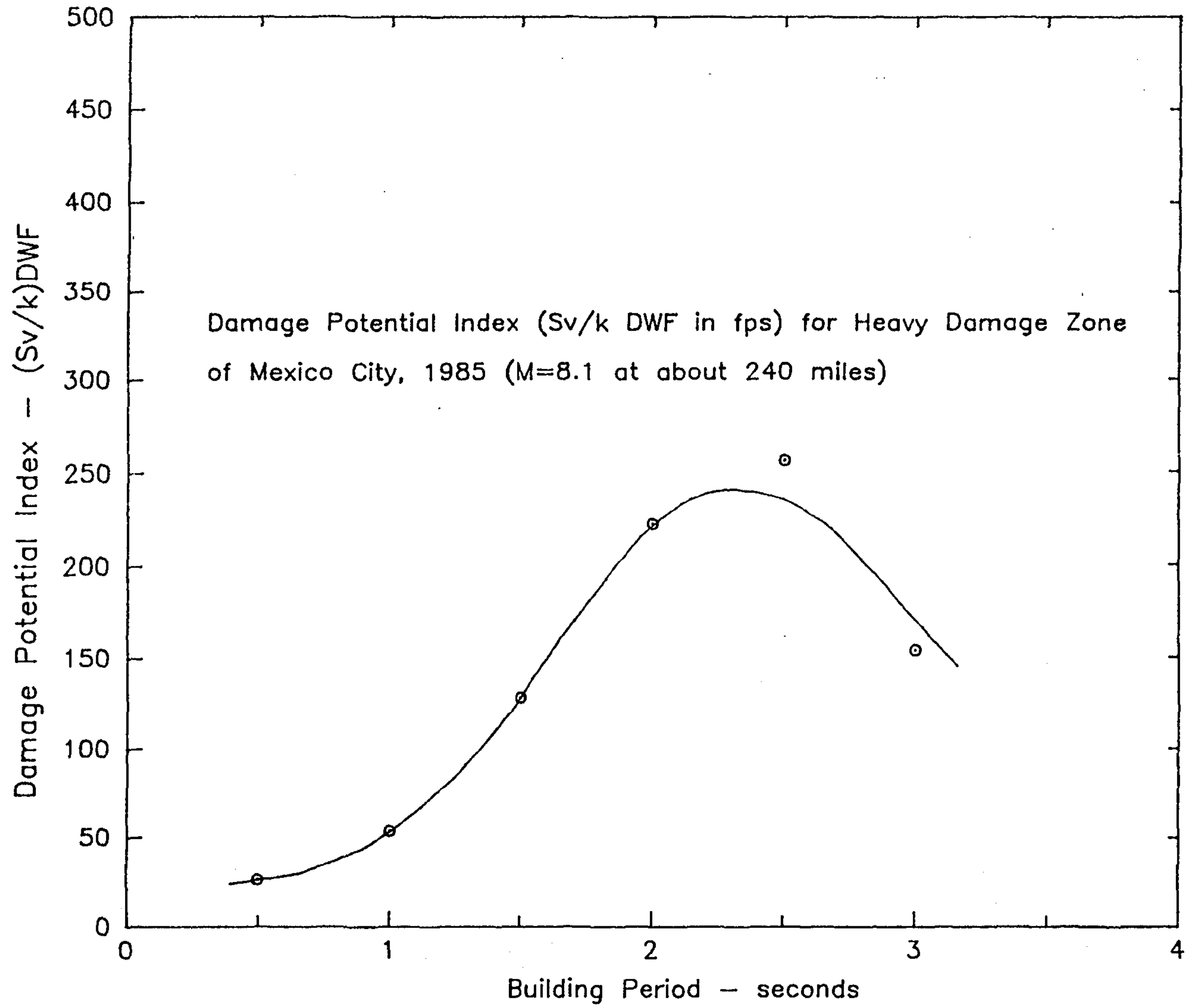


FIG. 5 CALCULATED DAMAGE POTENTIAL INDEX FOR HEAVY DAMAGE ZONE IN MEXICO CITY IN THE SEPT. 19, 1985 EARTHQUAKE

general trend of the relationship shown in Fig. 5 is also in good accord with the relationship based on observed damage intensity previously shown in Fig. 3. The relationships between observed damage intensity and computed Damage Potential Index in the heavy damage area of Mexico City can be more easily compared in Fig. 6, which superimposes the results presented in Fig. 3 and Fig. 5. Again it can be seen that buildings which had natural periods close to about 2 seconds suffered the most severe damage in Mexico City in the September, 1985 earthquake and also that a calculated Damage Potential Index of about 200 fps corresponds roughly to an observed damage intensity of about 30% for the damage developed in Mexico City in this earthquake.

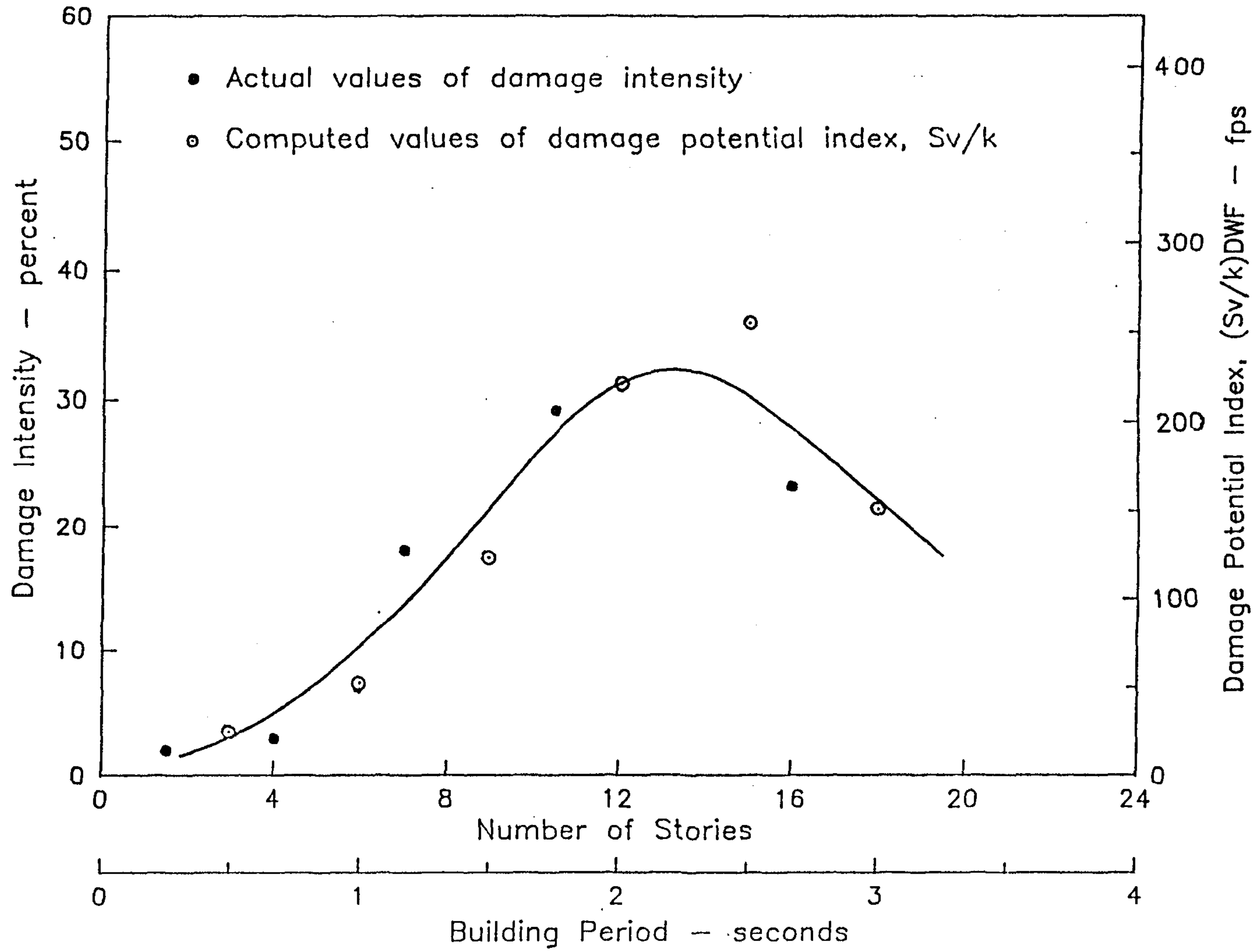


FIG. 6 ACTUAL DAMAGE INTENSITY AND CALCULATED DAMAGE POTENTIAL INDEX FOR HEAVY DAMAGE ZONE IN MEXICO CITY

III. EVALUATION OF POTENTIAL FOR BUILDING DAMAGE DUE TO EARTHQUAKE SHAKING IN THE SAN FRANCISCO BAY AREA

Seismic Environment of the San Francisco Bay Area

In view of the good relationship observed between Damage Potential Index values and actual damage intensities for the heavy-damage zone of Mexico City, it is of interest to examine the significance of these results to other areas of North America where structures are constructed on deep layers of clay. Areas of major interest in this respect would certainly include San Francisco, California, Salt Lake City, Utah, and Anchorage, Alaska. The San Francisco Bay area was selected for special study in this investigation. The Bay area has experienced 12 damaging earthquakes during the past 150 years (Goldman, 1969), including the major San Francisco earthquake of 1906 (Magnitude $\approx$ 8.2), and can be expected to be subjected to similar events in the future. In its latest evaluation the U.S. Geologic Survey predicts a 50% probability of a Magnitude 7 earthquake in the San Francisco Bay area in the next 30 years (Fig. 7). However a repetition of the 1906 Magnitude 8 earthquake, although assigned a relatively low probability of about 10% in the next 30 years, is also an important consideration in the next 100 years.

Most of the major earthquakes that have occurred in the San Francisco Bay area have been closely related to the active faults in the area, shown in Fig. 8. These include the San Andreas fault which transects the San Francisco and Marin Peninsulas, the Hayward fault which runs along the base of the Berkeley Hills in the East Bay, and the Calaveras fault located south of the Hayward fault. These faults and their branches are closely related and together comprise an important part of the major fault system which governs the seismicity of the San Francisco Bay area.

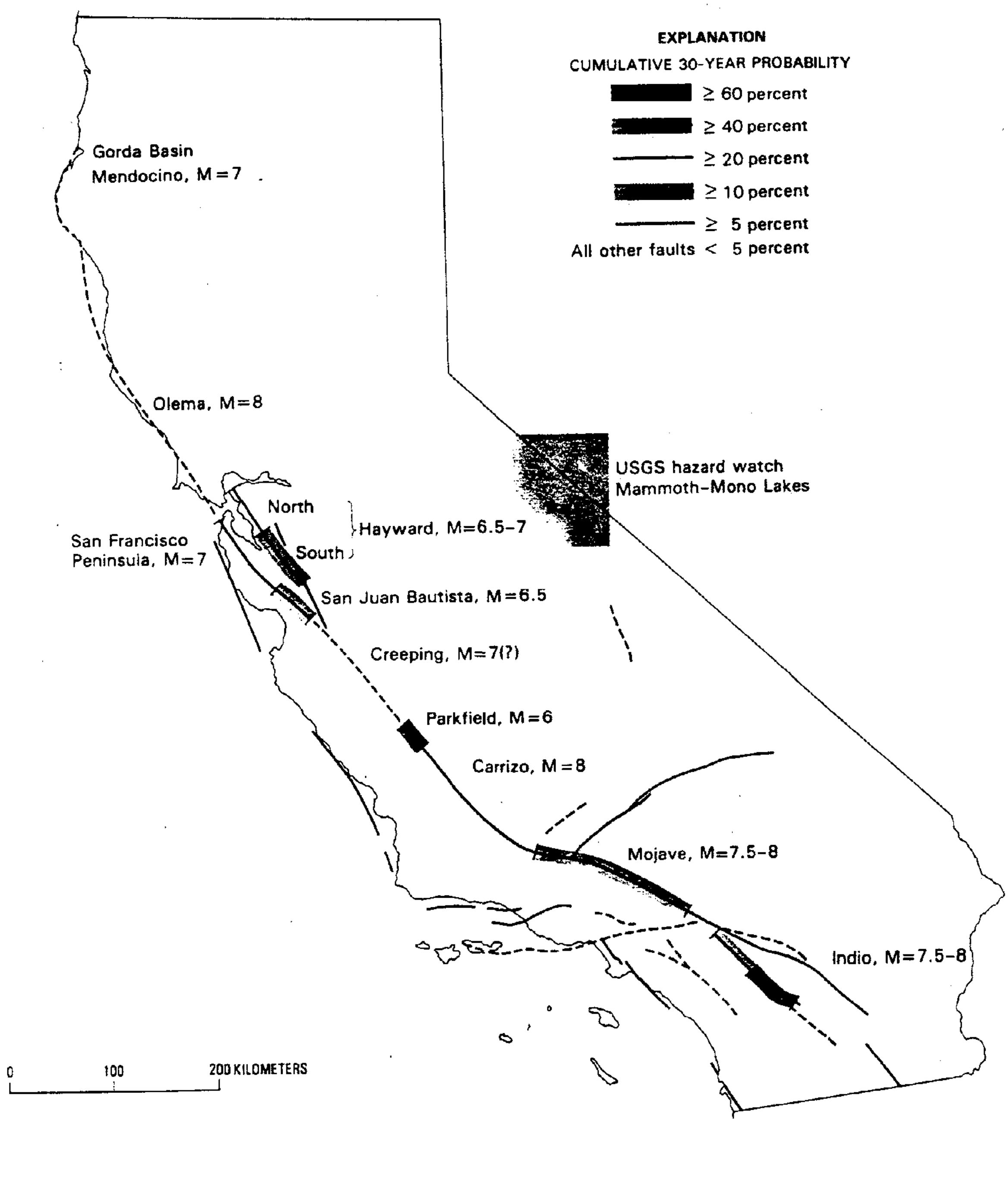


FIG. 7 30-YEAR CUMULATIVE PROBABILITIES OF OCCURRENCE OF EARTHQUAKE ALONG SELECTED FAULT SEGMENTS OF THE SAN ANDREAS FAULT SYSTEM (after Lindh, 1983)

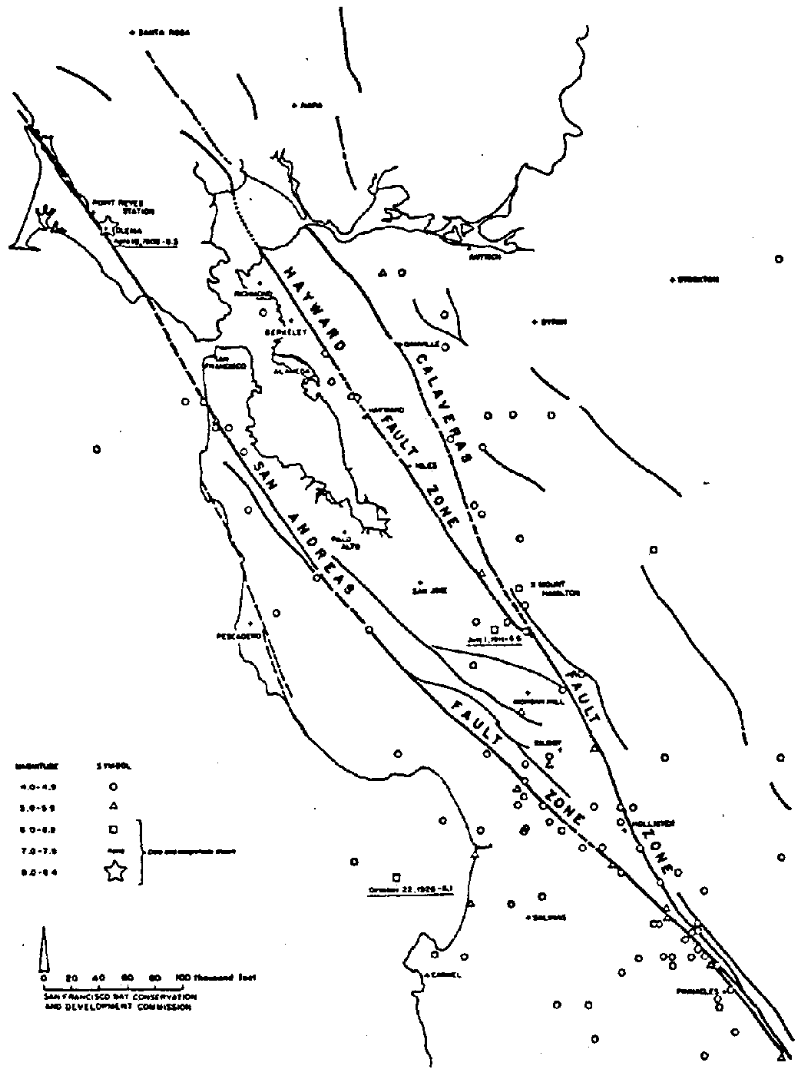


FIG. 8 ACTIVE FAULT SYSTEM NEAR SAN FRANCISCO FAULT SYSTEM (after Bolt, 1978)

Soil Conditions in the San Francisco Bay Area

San Francisco Bay is located in a northwest-trending valley. The bay is bounded mostly by marshlands, alluvial plains and beyond, by the Coast Ranges. The major geological units of the San Francisco Bay area, shown in Fig. 9 can be categorized broadly as bedrock, alluvium, and Bay mud, as follows (Borcherdt, et. al, 1975):

(1) Bed rock in the area is composed mainly of sandstone, siltstone, chert and greenstone of the Franciscan formation.

(2) Alluvium. This unit contains late Quaternary floodplain deposits of silt and clay, inter-layered with alluvial fan and stream-bed deposits of sand and gravel, derived from weathering and erosion of the uplands surrounding the San Francisco Bay. Some older alluvial deposits (early Quaternary) may be more consolidated and/or partially cemented.

(3) Bay Mud. These sediments are Holocene age, soft, water-saturated, organic-rich silts and clays,occasionally interlayered with sand deposits. They generally are derived from the suspended materials brought into San Francisco Bay by the rivers draining the Central Valley of California, as well as streams from the southern Bay area. Table 6 shows some of the physical and engineering properties of San Francisco Bay mud. It is the behavior of this soft clay under seismic loading that concerns many seismologists and engineers (after Goldman, 1969; Borcherdt, 1970; Borcherdt and Gibbs, 1976; Wilson, Warrick and Bennett, 1978).

As clearly illustrated in Mexico City, local geological conditions can substantially change the characteristics of seismic waves and the intensity

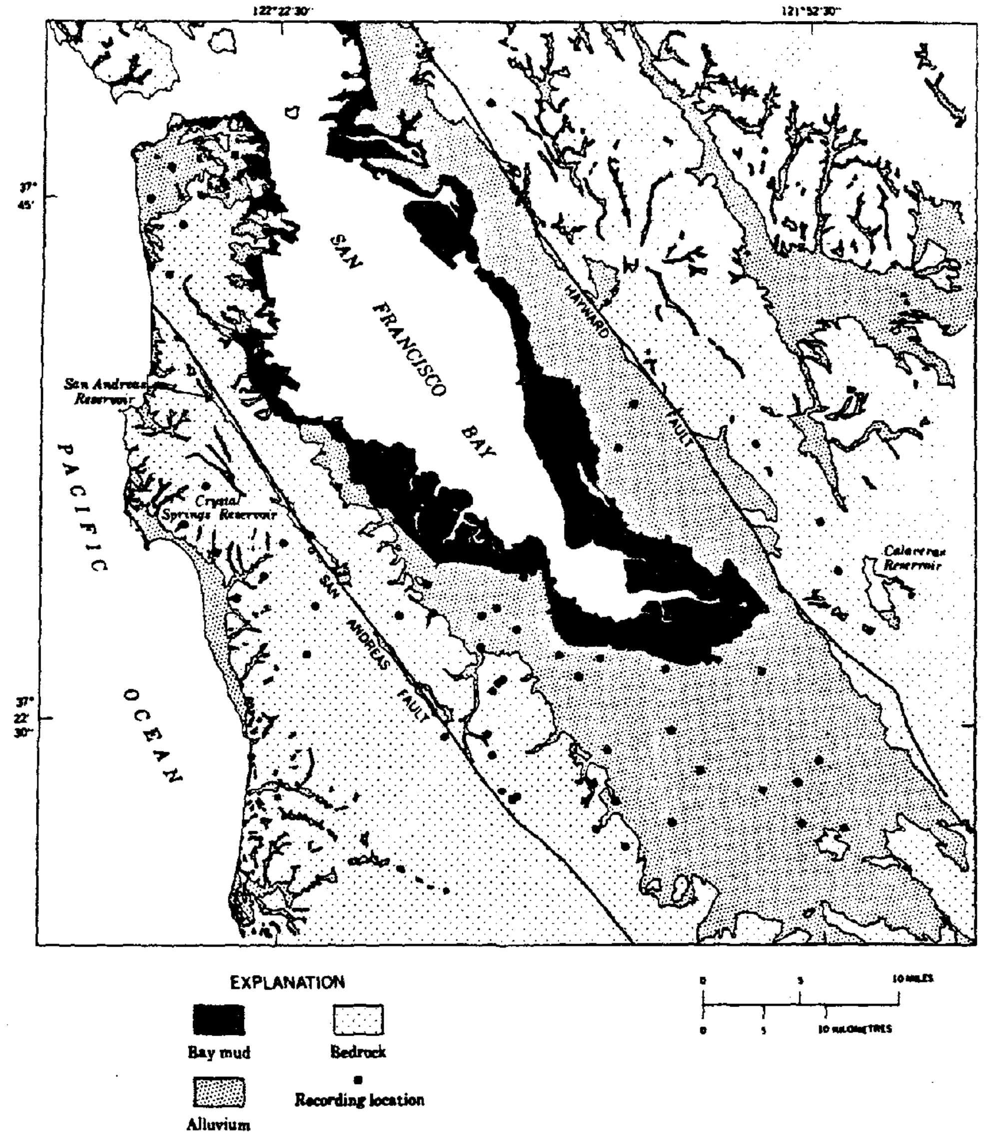


FIG. 9 GEOLOGICAL CONDITIONS NEAR SAN FRANCISCO BAY
(after Borcherdt et al., 1975)

TABLE 6 PHYSICAL AND ENGINEERING PROPERTIES OF SAN FRANCISCO BAY MUD (modified after Bonaparte and Mitchell, 1979 and ERTEC, 1981)

Location	Reference	Thickness of mud (feet)	Natural water content (percent)	Liquid limit (percent)	Plastic limit (percent)	Unit weight lb/ft^3	Mean diameter D_{50} in microns	Shear Strength lb/ft^2	Compression index C_c	$s_u/\bar{p}$	s_u/ft psf/ft
Proposed southern crossing	Trask and Rolston (1951)	0-45				94-107	1.4-10.5				
Proposed parallel crossing	Trask and Rolston (1951)	0-65	35-80	47-68	-	93-113	3.7-13.5	200-400	--		
Oakland Estuary	Trask and Rolston (1951)	0-45				96-101	3.6-5.8				
Adjacent to bridge Oakland Mole	Seed and Reese (1957)	20	35-50	41.5	23.5	112	-	100-500	0.36-0.45		
Islais Creek Basin	Lee (1953)	128	52-62	-	-	102-107	-	100-1000	--		
Candlestick Cove	Private Communication	>65	50-75	65-119	28-35	98-108	2-10	150-800	0.5-1.0		
Hamilton Air Force Base	Duncan (1965)	60	90	88	43	93-96	-	400-1200	1.1	0.32	10
Lash Terminal, Port of San Francisco	Duncan and Buchignani (1973)	100	55	50	30	100	-	200-700	--	0.27	10
Islais Creek S2	Clough and Sitar (1977)	50	55-95	75-100	30-45	105	-	400-1000	--	0.40	17
Islaid Creek S2	Clough and Sitar (1977)	120	75-95	75-100	30-45	99	-	400-500	--	0.11	4
Ferry Building	Private Communication (1965)	-	-	-	-	98	-	-	--	0.24	10
Embarcadero Center	Private Communication (1971)	-	-	-	-	110	-	400-1500	--	0.27	13
Hamilton	Denby (1978)	58	86-90	85-88	38-40	94	-	400-1300	.8-11	0.34	10.7
Menlo Park	Holzer, et al, (1973)		86-97	99	47				~1.1		
South S.F. Bay	Arulanandan, et al, (1971)		96-103	93	45	90.5					
Suisun Marsh	This study	75	58-120	90-98	52-62	80-95					

of earthquake shaking. This was also made abundantly evident in the 1906 San Francisco earthquake. Fig. 10 shows the geological conditions in San Francisco and Fig. 11 shows the intensity of shaking for San Francisco during the 1906 earthquake (Borcherdt, 1975). Except for the south-western part of the city where the San Andreas fault passes directly through the city, most of the parts of the city showing high values of "apparent shaking intensity" were those underlain by thick layers of Bay mud. This was especially true for the downtown area of San Francisco. The need to re-evaluate seismic damage potentials for the San Francisco Bay area in the light of the Mexico City earthquake experience seems therefore especially appropriate.

Dynamic Soil Properties of San Francisco Bay Mud

It is well-recognized that the appropriate forms of the modulus reduction and damping ratio relationships with shear strain play a key role in performing successful ground response analyses. A number of studies have been performed to evaluate these dynamic properties for San Francisco Bay mud. Bay mud samples tested in these investigations were taken from: Suisun marsh in the north Bay (ERTEC, 1981), Hamilton Air Force Base in the west-central part of the Bay (Isenhower, 1979; Isenhower and Stokoe, 1981) and from Ravenswood, Dumbarton West and Agnew, three sites located in the south Bay (Stokoe and Lodde, 1978; Lodde, 1982).

The dynamic properties of young Bay mud, summarized from these investigations, are shown in Fig. 12. The upper part of the figure shows how the shear modulus reduces with increasing shear strain while the lower part of the figure shows the increase in damping ratio with shear strain. The rate of reduction in modulus with increasing strain for young Bay mud is

A4 REDUCTION OF EARTHQUAKE HAZARDS, SAN FRANCISCO BAY REGION

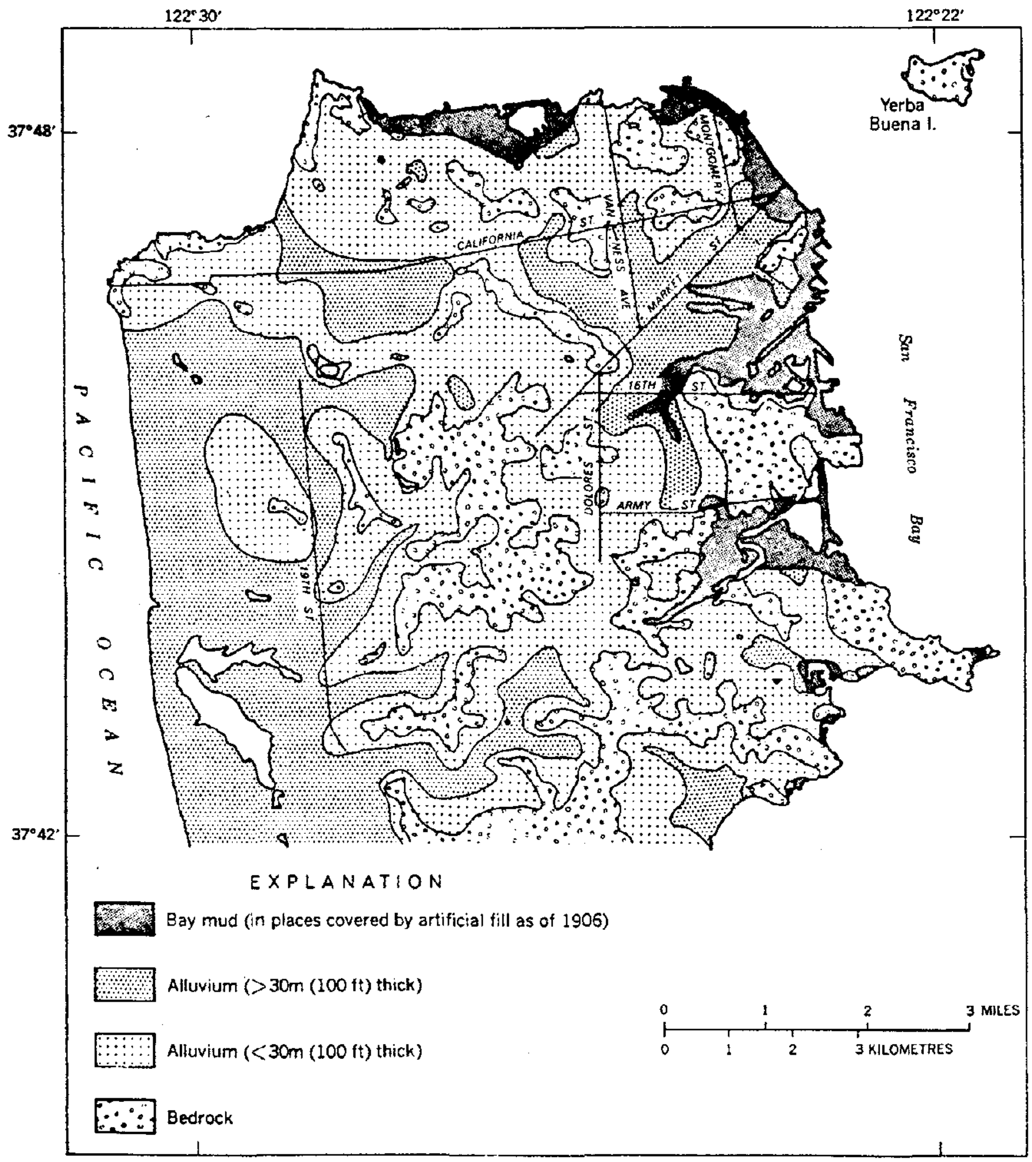


FIG. 10 GENERALIZED GEOLOGICAL MAP OF SAN FRANCISCO (compiled by K. R. Lajoie from data of Schlocker et al., 1958)

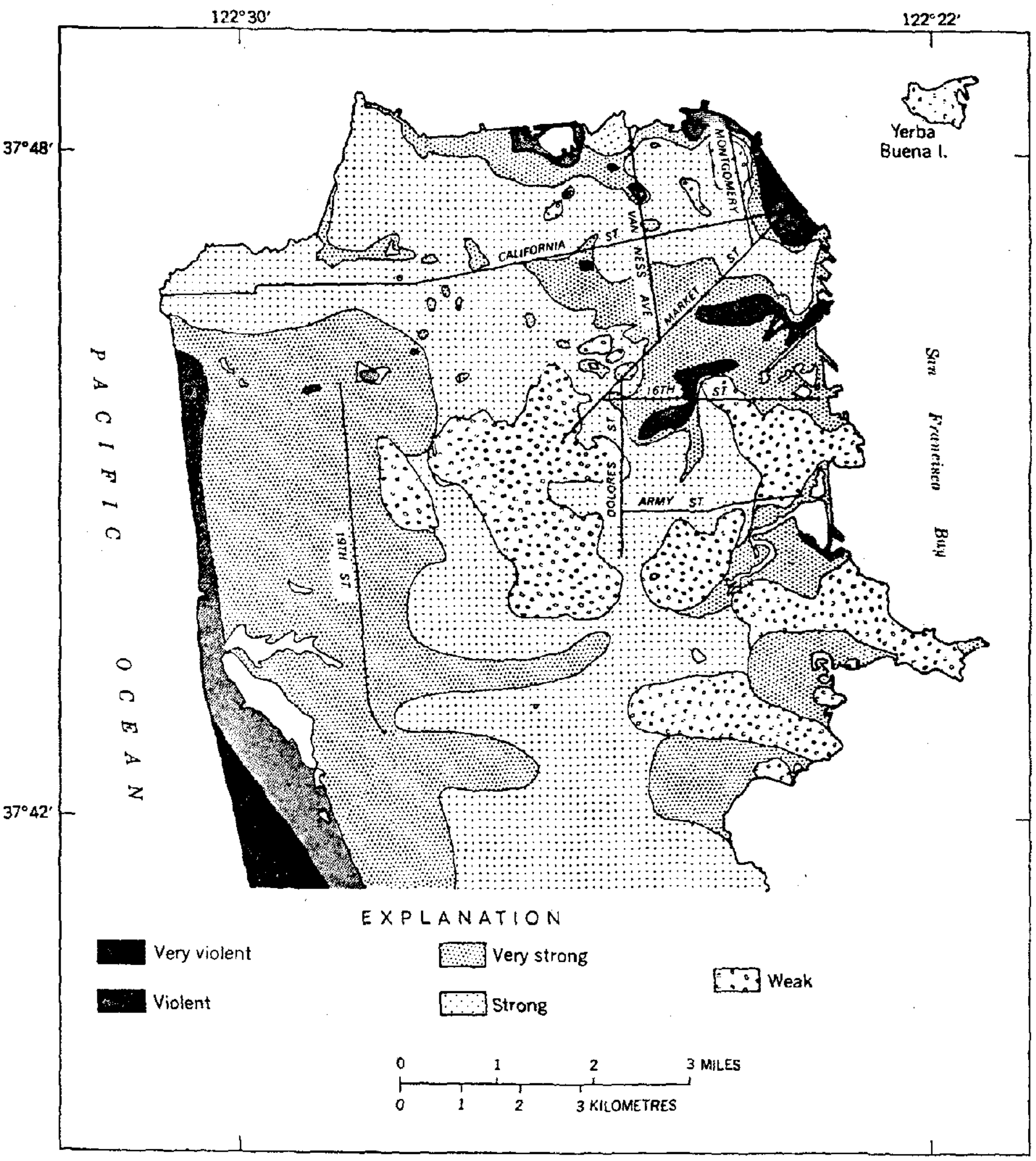
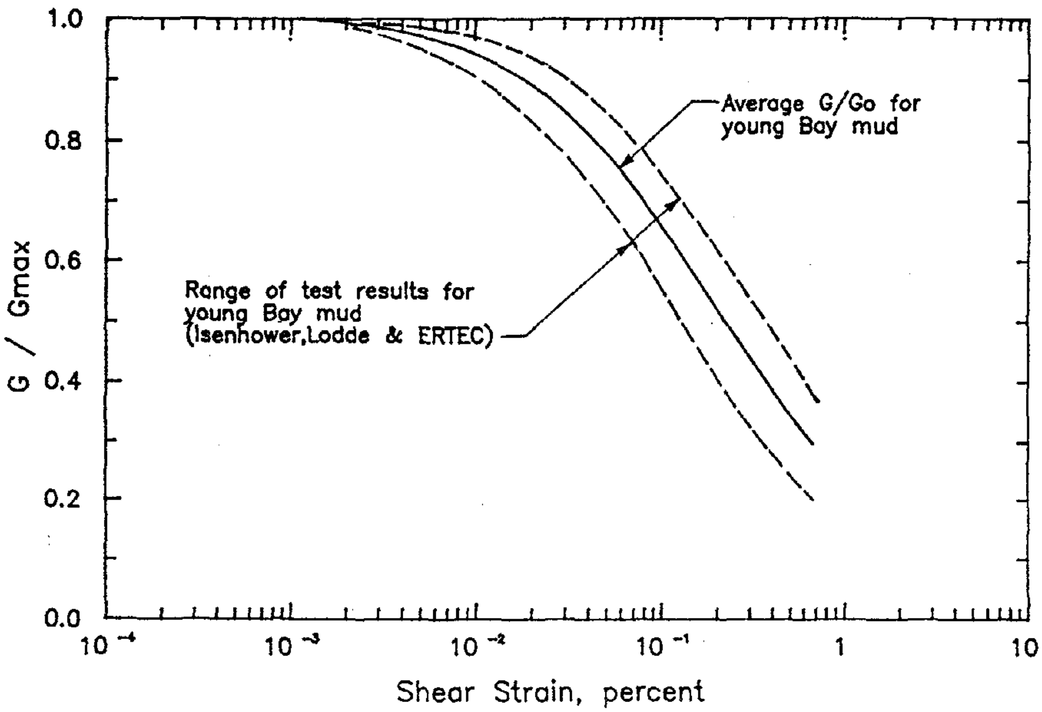


FIG. 11 DISTRIBUTION OF APPARENT SHAKING INTENSITY IN SAN FRANCISCO IN THE 1906 EARTHQUAKE (compiled by Borcherdt, 1975 after data from Wood, 1908)



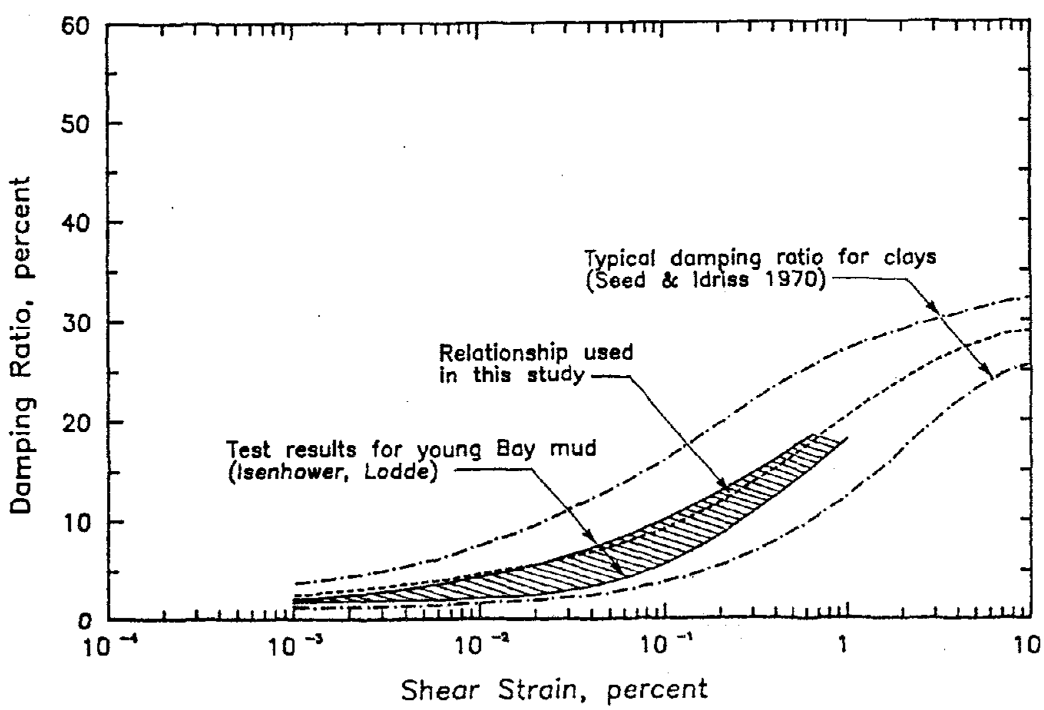


FIG. 12 VARIATION OF SHEAR MODULUS AND DAMPING RATIO WITH SHEAR STRAIN FOR YOUNG SAN FRANCISCO BAY MUD

significantly less than that for typical sands, but it is in generally good accord with values determined for other clays (see Sun et al., 1988). The damping characteristics fall well within the range proposed by Seed and Idriss (1970) for typical clays. Lodde (1982) has also tested some older Bay sediments near Dumbarton West site. The average modulus reduction relationship for these older Bay sediments, which consisted mainly of gravelly sands and silts having a void ratio of about 0.63, was more like that for sands than for Bay mud.

Shear Wave Velocity Profiles for Young San Francisco Bay Mud

Values of shear wave velocity for young San Francisco Bay mud have been measured in studies by Warrick, 1974; Gibbs et al., 1975; Gibbs et al., 1976; Gibbs et al., 1977; Wilson et al., 1978; Pyke, 1987; and Liu et al., 1988. The results provided by these investigations are summarized in Fig. 13. The figure indicates that shear wave velocities for young Bay mud are essentially constant for the top 30 feet, with a value of 250 fps, but the velocity then gradually increases to about 500 fps at a depth of 60 feet.

Site Conditions for Three Bayshore Sites

Three Bayshore sites underlain by soft Bay mud were chosen for the purpose of studying the ground motions that are likely to develop on such sites in the event of earthquakes with Magnitudes $7\frac{1}{4}$ and 8+ occurring on the San Andreas fault. The soft Bay mud at these three sites appears to have the same general characteristics as for other San Francisco Bayshore sites. Two of these sites, the Southern Pacific Building site and the Embarcadero Center Four site are in the city of San Francisco, and the Ravenswood site is in the south Bay area.

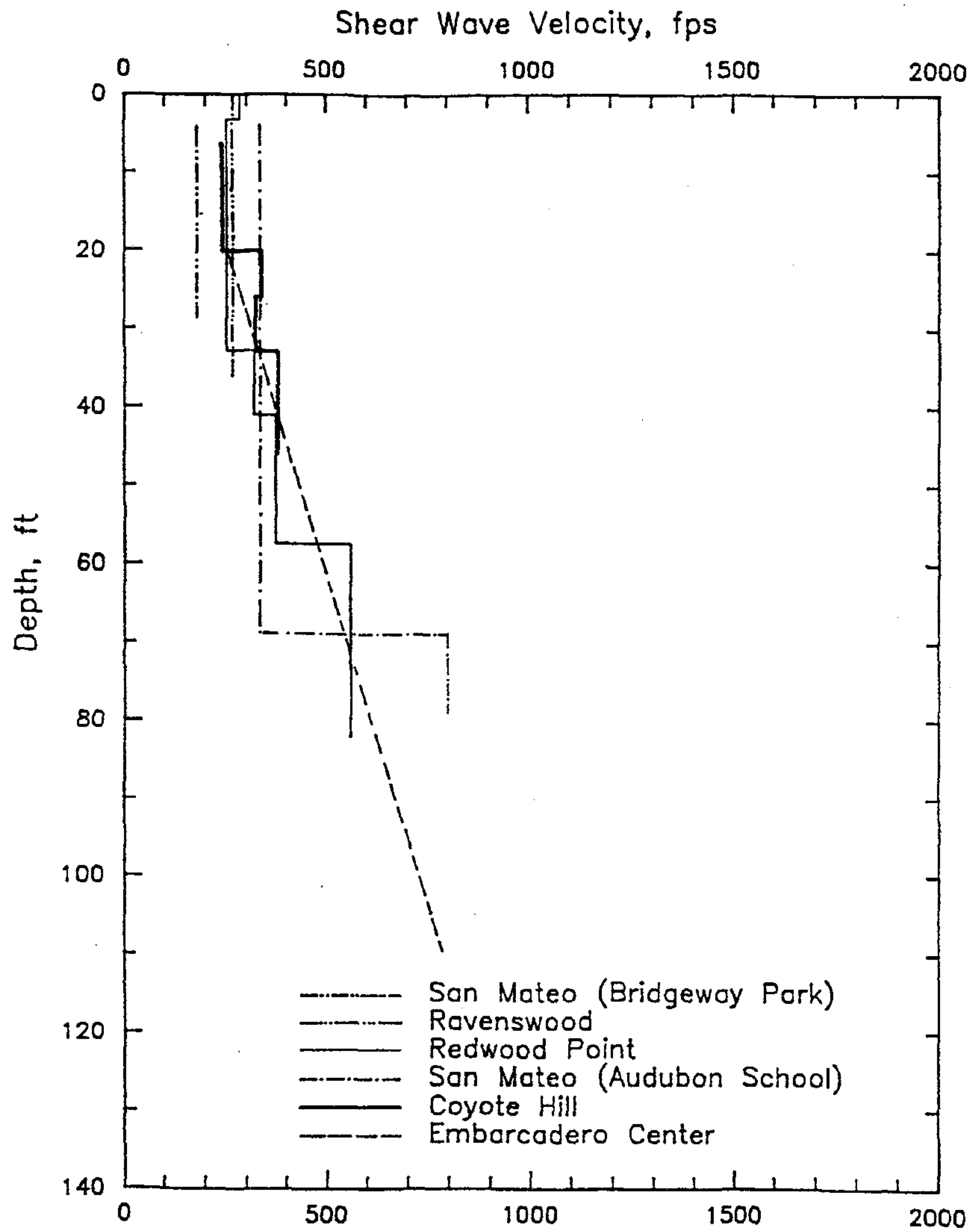


FIG. 13 MEASURED SHEAR WAVE VELOCITY PROFILE FOR YOUNG BAY MUD FOR SIX SAN FRANCISCO BAY SHORE SITES

Southern Pacific Building Site

The Southern Pacific Building Site has been the subject of several studies in the past several decades (Idriss and Seed, 1968; Singh et al., 1981). The soil conditions at the Southern Pacific Building site are shown in Fig. 14. A sandy fill extends to a depth of about 20 ft and is underlain by a 35 ft layer of soft clay followed by a 50 ft layer of medium stiff clay. Below this is a 120 ft layer of stiff clay interbedded with a 10 ft layer of dense sand at a depth of 125 ft. The stiff clay is underlain by a layer of very dense sand and gravel which extends to bed rock at a depth of about 285 ft.

Initial estimates of the shear wave velocity for each soil layer in the soil profile were based on the shear strength and the stiffness of the materials, as reported by Idriss and Seed (1968) and Rinne and Stobbe (1979), together with the representative data for Bay mud shown in Fig. 13. The shear wave velocity profile was then further calibrated by computing its response to the 1957 Daly City, San Francisco earthquake as explained below.

An earthquake of Magnitude 5.3 located along the San Andreas Fault was recorded in the basement of the 11-story high Southern Pacific Building on March 22, 1957. The earthquake was simultaneously recorded on rock in Golden Gate Park, located roughly 7 miles from the Southern Pacific Building site as shown in Fig. 15. The Golden Gate Park record was used as a rock outcrop motion in dynamic response analyses of the Southern Pacific Building site, but the acceleration values were scaled down by a factor of 0.65 to account for the different distances of the two sites from the source of energy release, as recommended by Idriss and Seed (1968) and Singh et al.

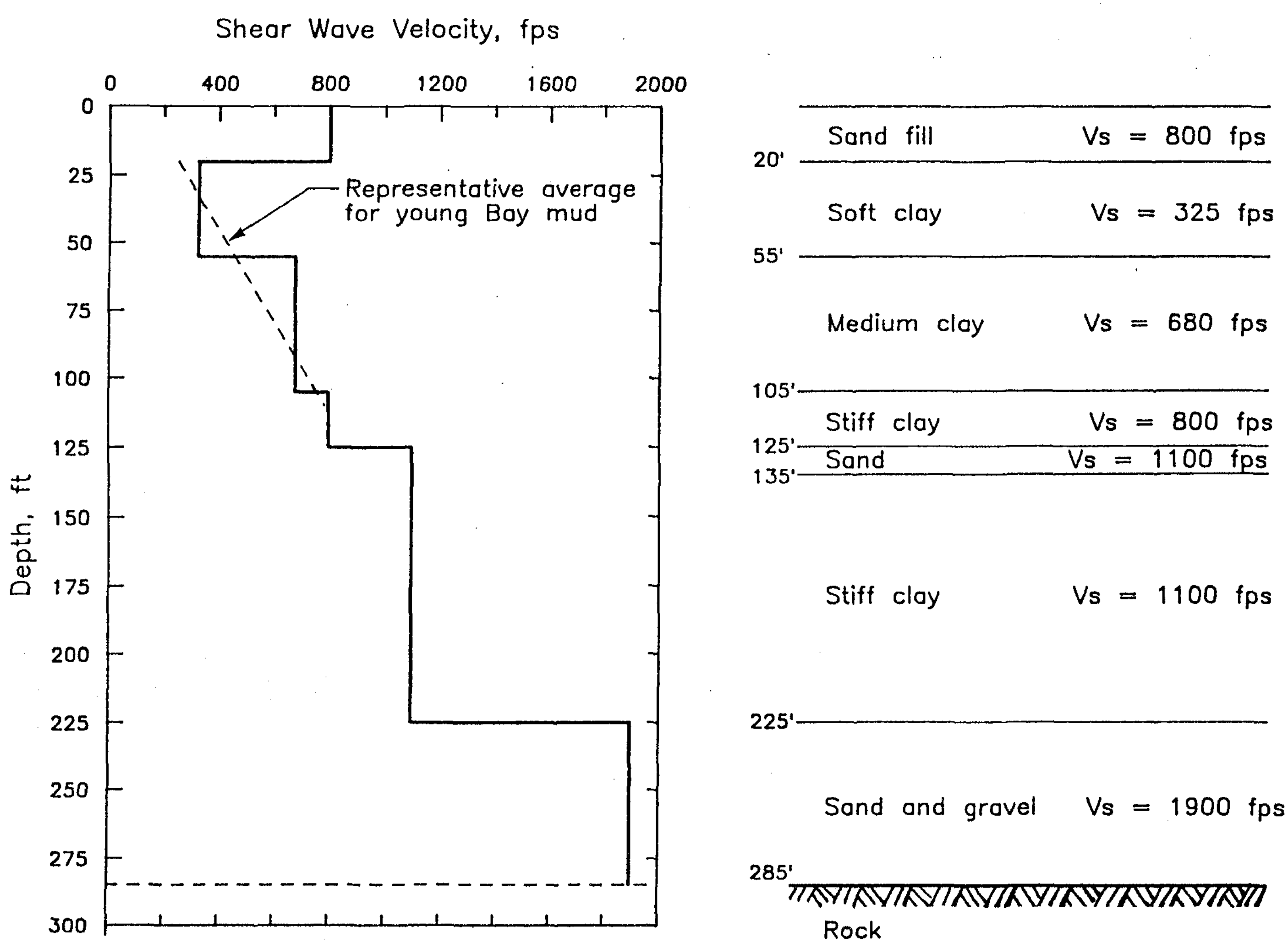


FIG. 14 SOIL PROFILE AND ESTIMATED SHEAR WAVE VELOCITIES AT SOUTHERN PACIFIC BUILDING SITE

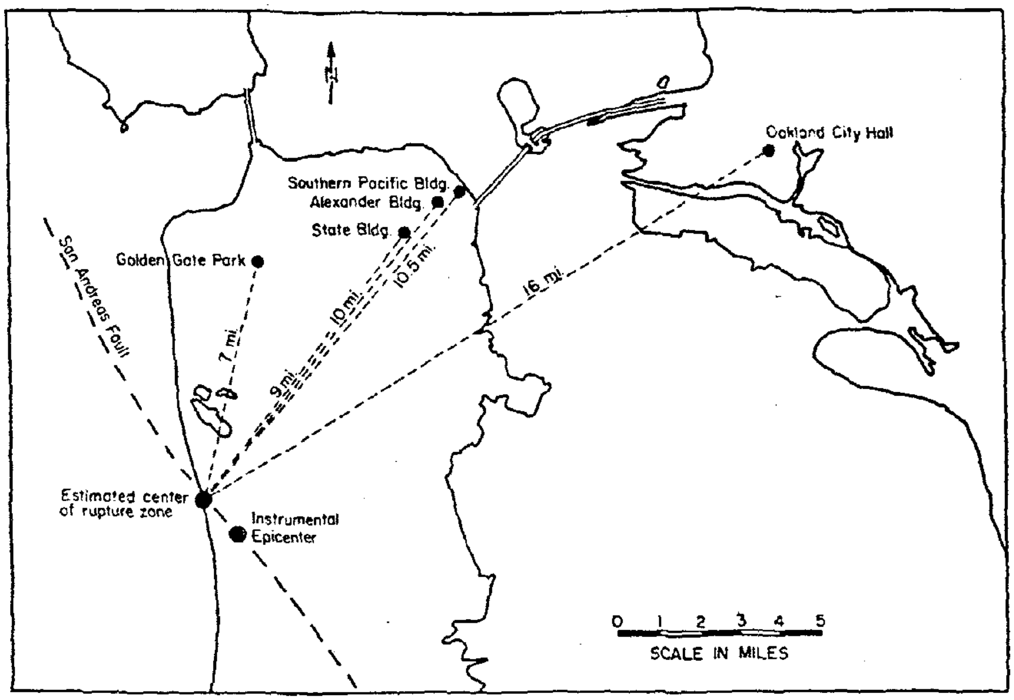


FIG. 15 LOCATIONS OF U.S. GEOLOGICAL SURVEY STRONG MOTION ACCELEROMETER STATIONS RELATIVE TO SAN ANDREAS FAULT - SAN FRANCISCO EARTHQUAKE OF MARCH 22, 1957 (after, Idriss and Seed, 1968)

(1981). The peak rock acceleration used in the analyses was thus reduced to 0.06 g. Values of shear wave velocity for the different layers were first selected based on those used in previous studies, and on the available data for soft Bay mud, and they were then modified slightly to give a velocity profile for which the computed response spectrum, for motions at the basement level, was in best agreement with the spectrum for the recorded motion. The shear wave velocity profile determined in this way is shown in Fig. 14 and the results of the ground response analysis in Fig. 16. Fig. 17 shows a comparison of the response spectra for the computed and recorded motions. It can be seen from the results presented in Fig. 14 that the shear wave velocity profile for soft Bay mud used for this site agrees well with the average shear wave velocity data for soft Bay Mud discussed previously (see Fig. 13) and that the computed motions are in excellent accord with those recorded in the 1957 earthquake.

Embarcadero Center Site

The Embarcadero Center Four site is located east of Drum Street, south of Clay Street and north of Sacramento Street near the waterfront of downtown San Francisco. The subsoil conditions, shown in Fig. 18, consist of 5 layers: about 20 ft of fill overlying roughly 90 ft of Bay mud, followed by 20 ft of sand, 30 ft of silty clay and 50 ft of silty sand. The bedrock, mainly shale and sandstone, is located at a depth of 210 ft below the ground surface. Down hole seismic surveys were performed (Harding-Lawson, 1977) to measure the shear wave velocity profile for the site. A representative shear wave velocity profile interpreted from these data, is plotted in Fig. 18.

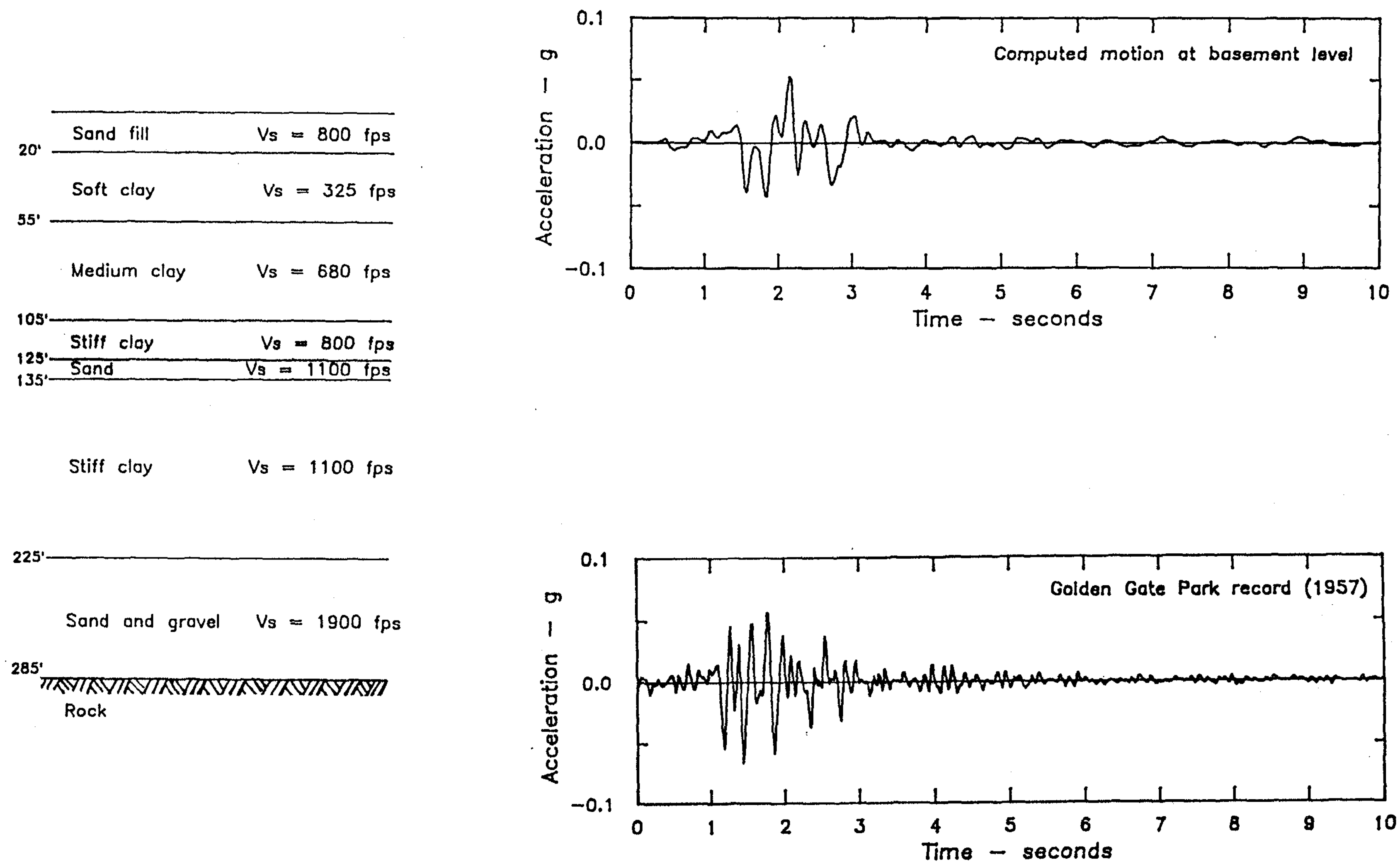


FIG. 16 GROUND RESPONSE ANALYSIS OF SOUTHERN PACIFIC BUILDING SITE IN SAN FRANCISCO IN THE 1957 EARTHQUAKE

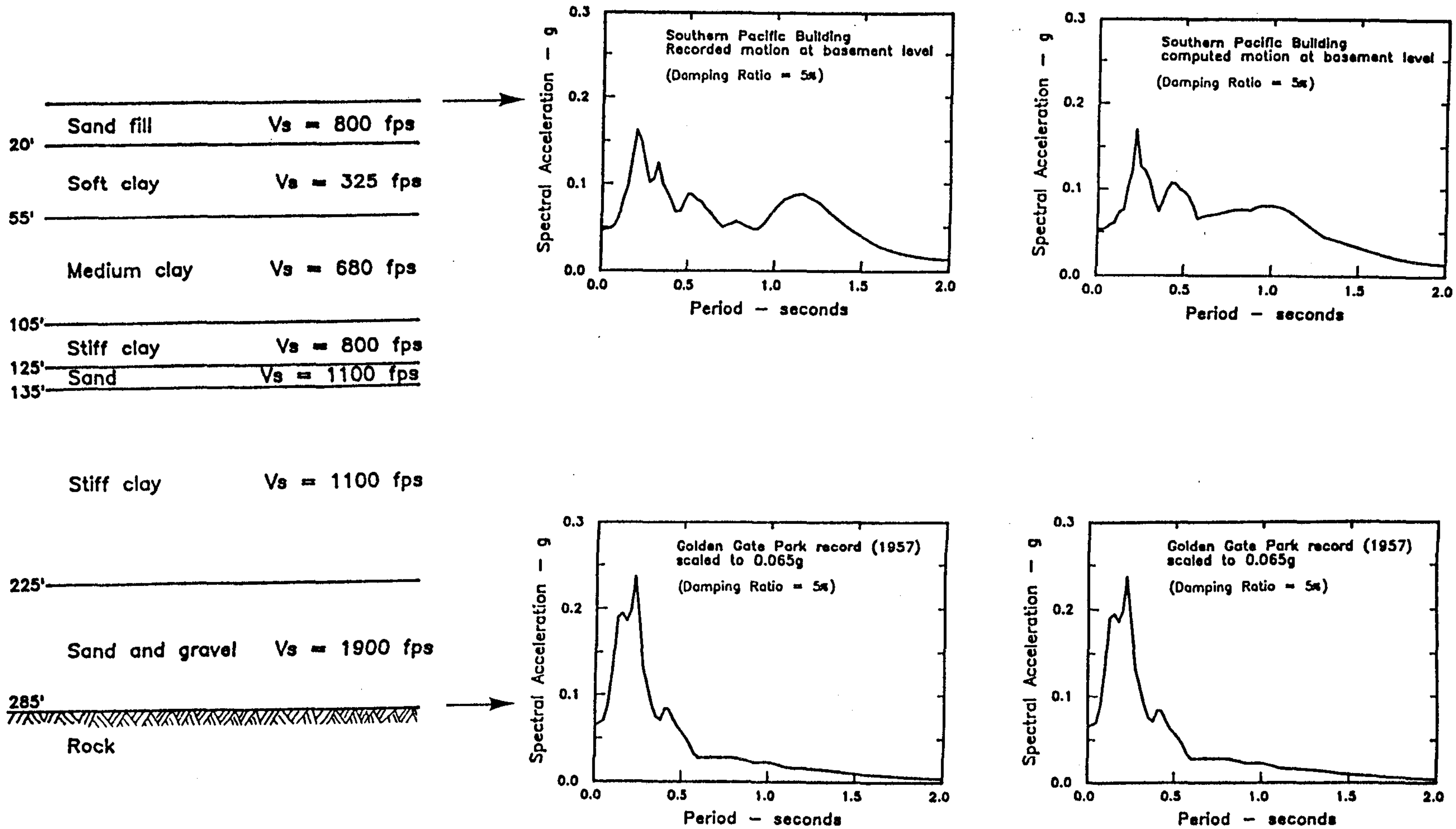


FIG. 17 ANALYSIS OF GROUND RESPONSE AT SOUTHERN PACIFIC BUILDING SITE IN SAN FRANCISCO EARTHQUAKE, 1957

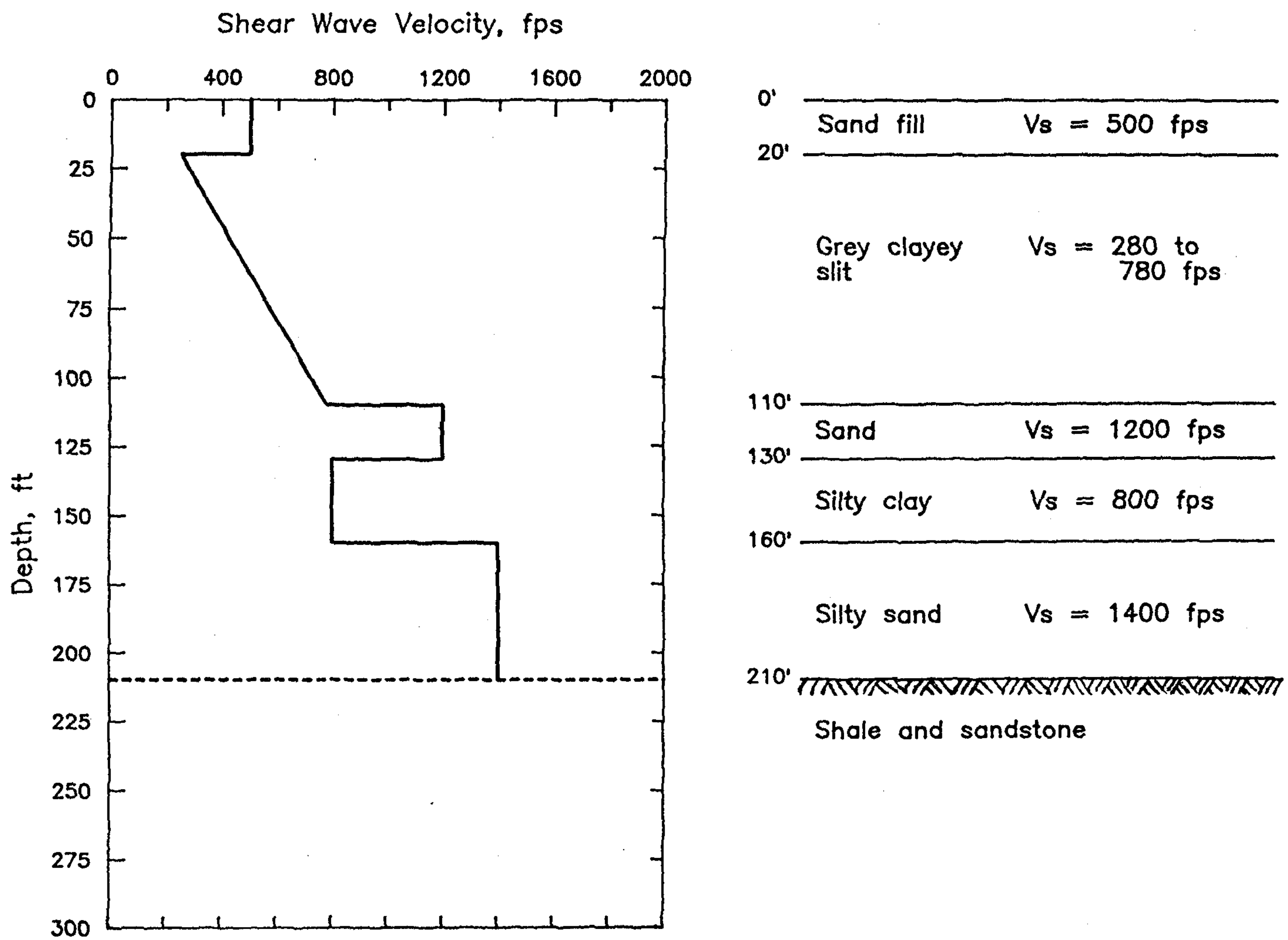


FIG. 18 SOIL PROFILE AND SHEAR WAVE VELOCITIES AT EMBARCADERO CENTER SITE

Ravenswood Site

This site was chosen as typical of large areas near the edge of the southern end of San Francisco Bay. The generalized geological section at the site consists of a surface layer of young Bay mud covering the older Bay sediments which extend to bedrock at a depth of about 600 ft, as shown in Fig. 19.

The younger Bay mud, about 33 ft thick at this site, is a soft, low shear-strength clay. Some sand and silt layers are interspersed within the Bay mud where stream channels crossed the mud flats. The older bay sediments, beneath the younger Bay mud, are of late Pliocene to Holocene age and were formed by alluvial processes. The older Bay sediments vary greatly in composition but in general, they are much stiffer than the younger Bay mud. The depth to bedrock, mainly sandstone and greywacke, is about 600 ft. The shear wave velocity profile adopted for this site was based on a downhole seismic survey (Warrick, 1974) together with some minor modifications to the profile recommended by Joyner et al., (1976).

Characteristics of Rock Outcrop Motions in San Francisco

Two levels of ground motion were used to evaluate the possible effects of earthquakes that may be generated on the San Andreas Fault. The lower level was a Magnitude $7\frac{1}{4}$ earthquake with an estimated duration of shaking of about 30 to 40 seconds and the higher level earthquake was a Magnitude 8+ earthquake with shaking lasting for about 70 to 80 seconds. The latter is compatible with the duration of shaking reported for the 1906 earthquake. Since no reliable near-field rock records are available for Magnitude $7\frac{1}{4}$ and 8+ earthquakes at the present time, artificial records were used to provide rock outcrop motions for use in the analyses. The sites were considered to

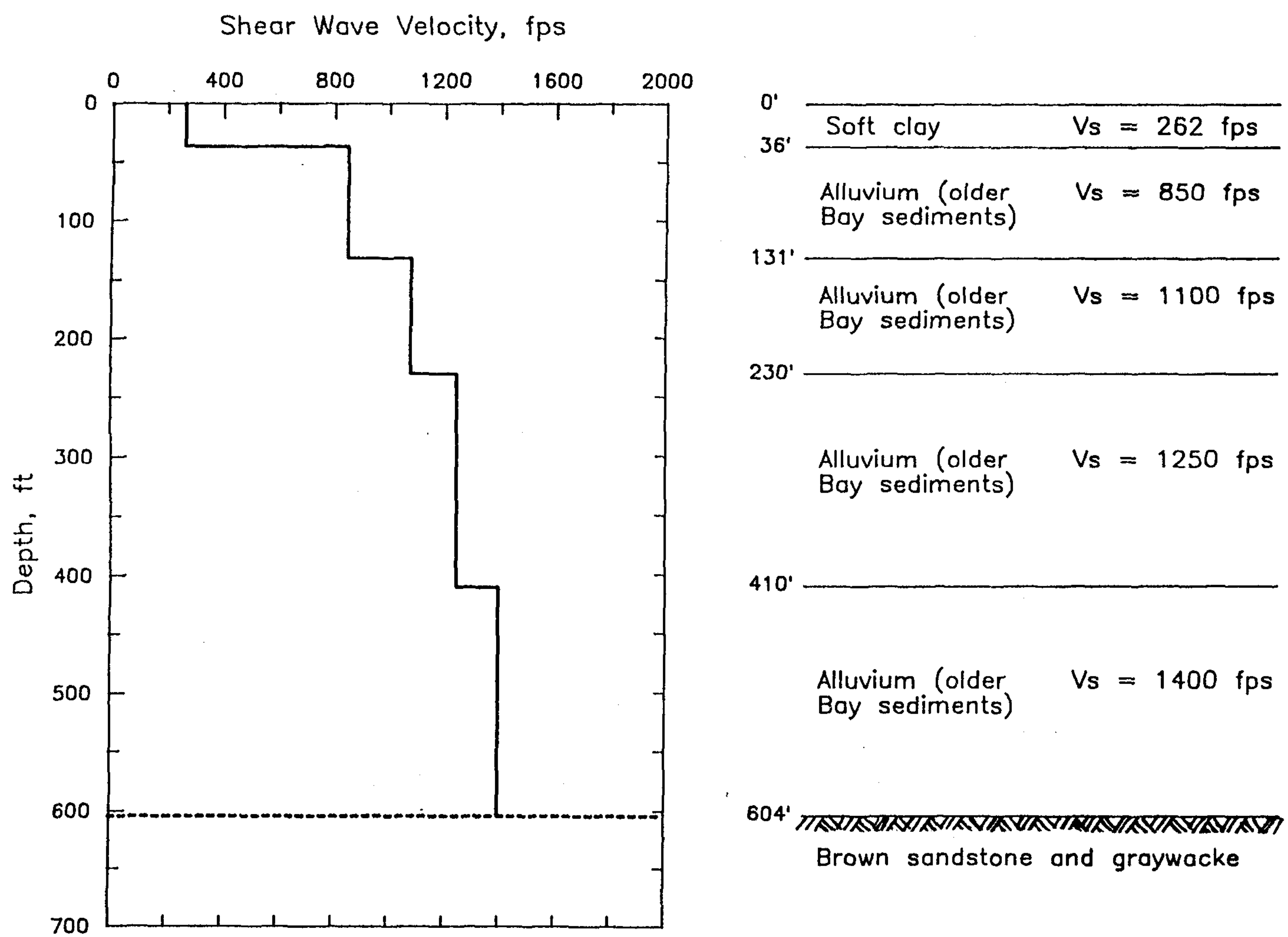


FIG. 19 SOIL PROFILE AND SHEAR WAVE VELOCITIES AT RAVENSWOOD USGS SITE

be located about 6 miles from the major fault systems, and an appropriate attenuation relationship for rock motions in Western U.S. earthquakes was adopted, as shown in Fig. 20 (Seed and Idriss, 1983), to estimate the peak ground accelerations on rock outcrops. It has been shown (Bolt, 1973) that the duration of strong ground motions generally depends on the earthquake magnitude. Table 7, which lists the duration of strong shaking for earthquakes with different magnitudes, was also used as a guideline in generating the rock records considered representative for these two earthquake magnitudes.

Table 8 lists the main characteristics of the rock motions used in the analyses and Figs. 21 and 22 show the acceleration time histories for Magnitudes $7\frac{1}{4}$ and 8+ earthquakes respectively. In addition to assigning appropriate values of acceleration amplitudes and durations, it has been shown that different magnitudes of western U.S. earthquakes also produce typical frequency characteristics (McGuire, 1974; Joyner and Boore, 1982, 1988; Sadigh, 1983; Sadigh et al., 1986; Idriss, 1985). Fig. 23 and Fig. 24 compare the response spectra for the two selected rock motions with the normalized magnitude-dependent spectra proposed by Sadigh et al., (1986). It can be seen that the acceleration response spectra for the two motions used are in good agreement with the spectra based on empirical experience. Thus it was considered that the synthetic motions employed in this study have the appropriate characteristics of natural earthquakes (of comparable magnitudes and distances to faults), both in amplitude and frequency characteristics.

<u>Ground Response Analyses for San Francisco Bayshore Sites</u>

Ground response analyses were performed for the three sites described previously to study the site responses for the two selected levels of ground

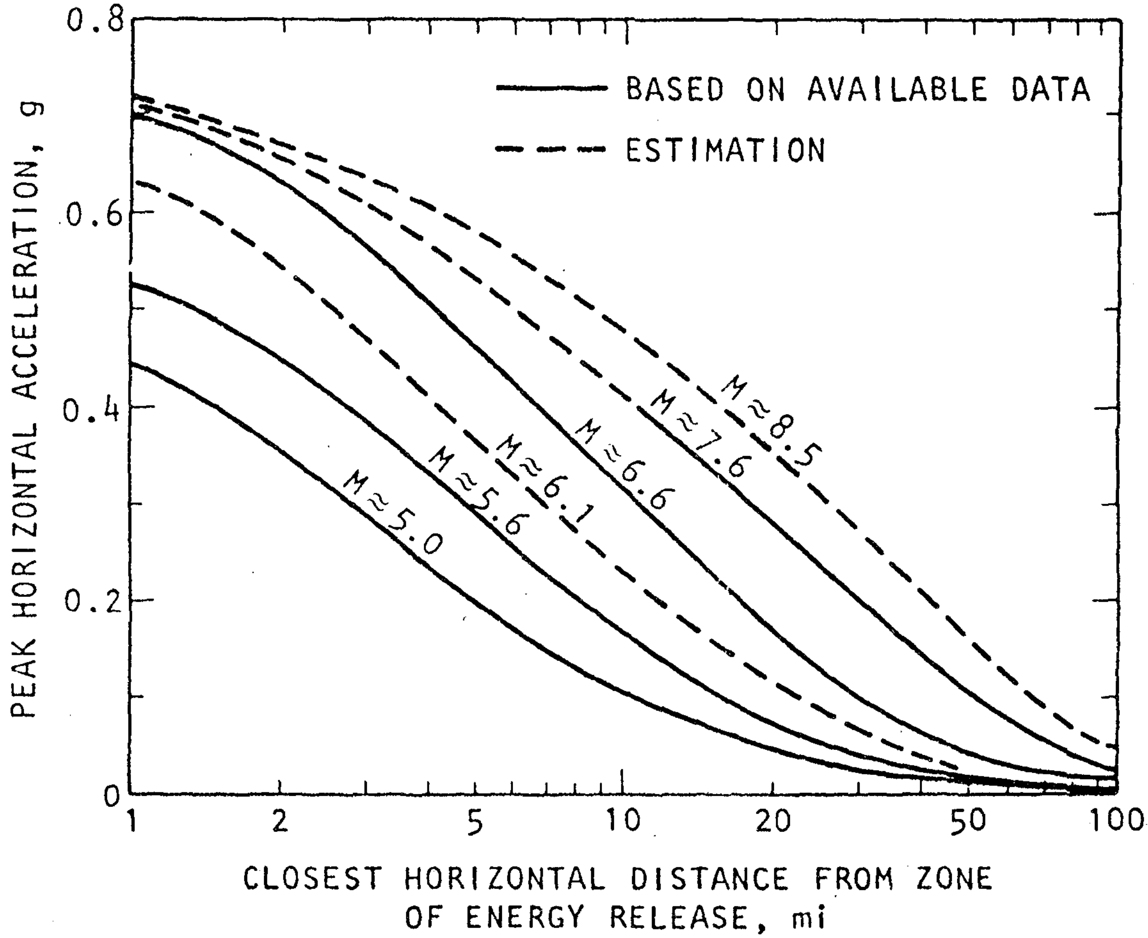


FIG. 20 ATTENUATION OF GROUND ACCELERATION IN ROCK WITH DISTANCE FOR EARTHQUAKE WITH VARIOUS MAGNITUDES (after Seed and Idriss, 1983)

TABLE 7 TYPICAL DURATIONS FOR EARTHQUAKES WITH DIFFERENT MAGNITUDES (from Gere and Shah, 1984)

Richter Magnitude	Average Number of Earthquakes per Year for the World	Average Number of Earthquakes per 100 Years for California	Duration of Strong Ground Shaking (seconds)	Radius of Region Subjected to Strong Ground Shaking (kilometers)
8.0 to 8.9	1	1	30 to 90	80 to 160
7.0 to 7.9	15	12	20 to 50	50 to 120
6.0 to 6.9	140	80	10 to 30	20 to 80
5.0 to 5.9	900	400	2 to 15	5 to 30
4.0 to 4.9	8,000	2,000	0 to 5	0 to 15

TABLE 8 CHARACTERISTICS OF ROCK OUTCROP MOTIONS USED IN THIS STUDY

Magnitude	Distance to Fault	Peak Ground Acceleration	Duration
$7\frac{1}{4}$	6 miles	0.45 g	32 seconds
8+	6 miles	0.55 g	75 seconds

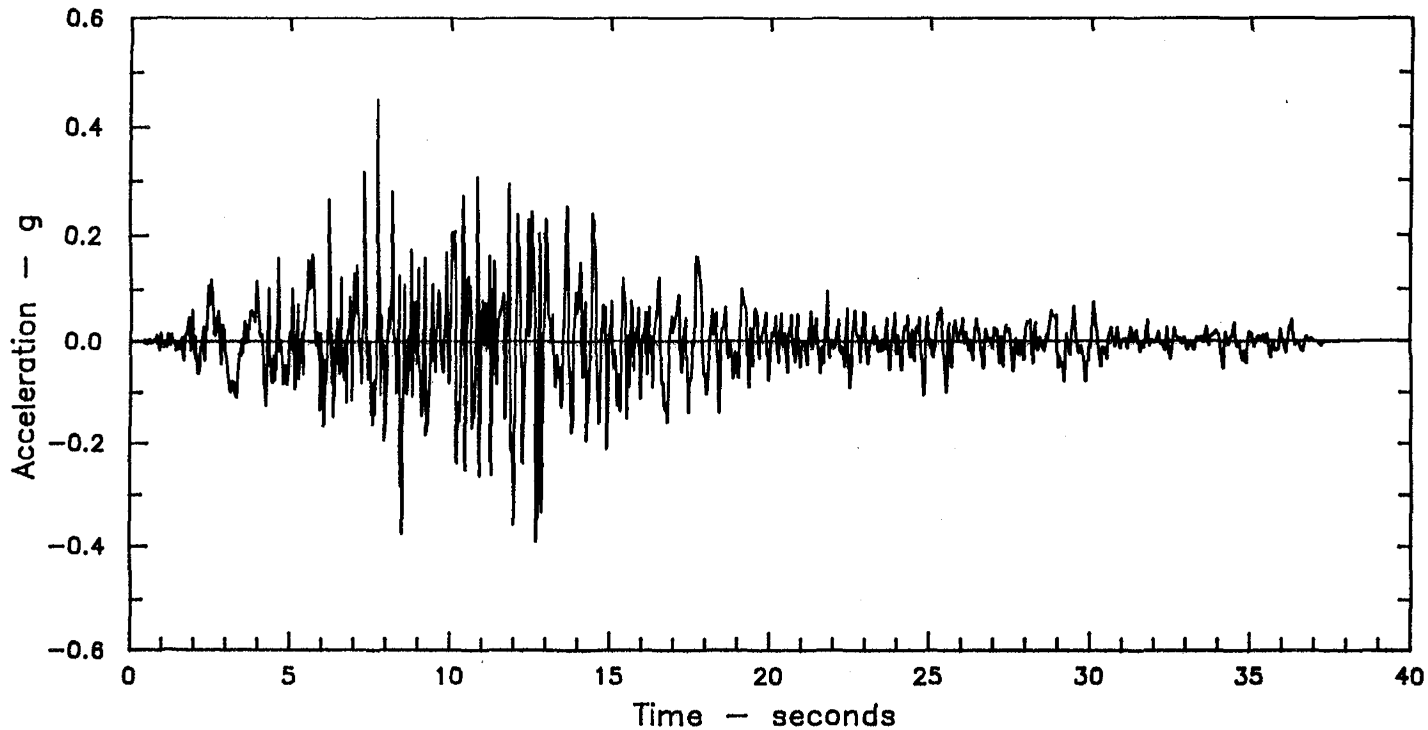


FIG. 21 SYNTHETIC RECORD FOR MAGNITUDE 7-1/4 EARTHQUAKE MOTION ON ROCK AT A DISTANCE OF 6 MILES

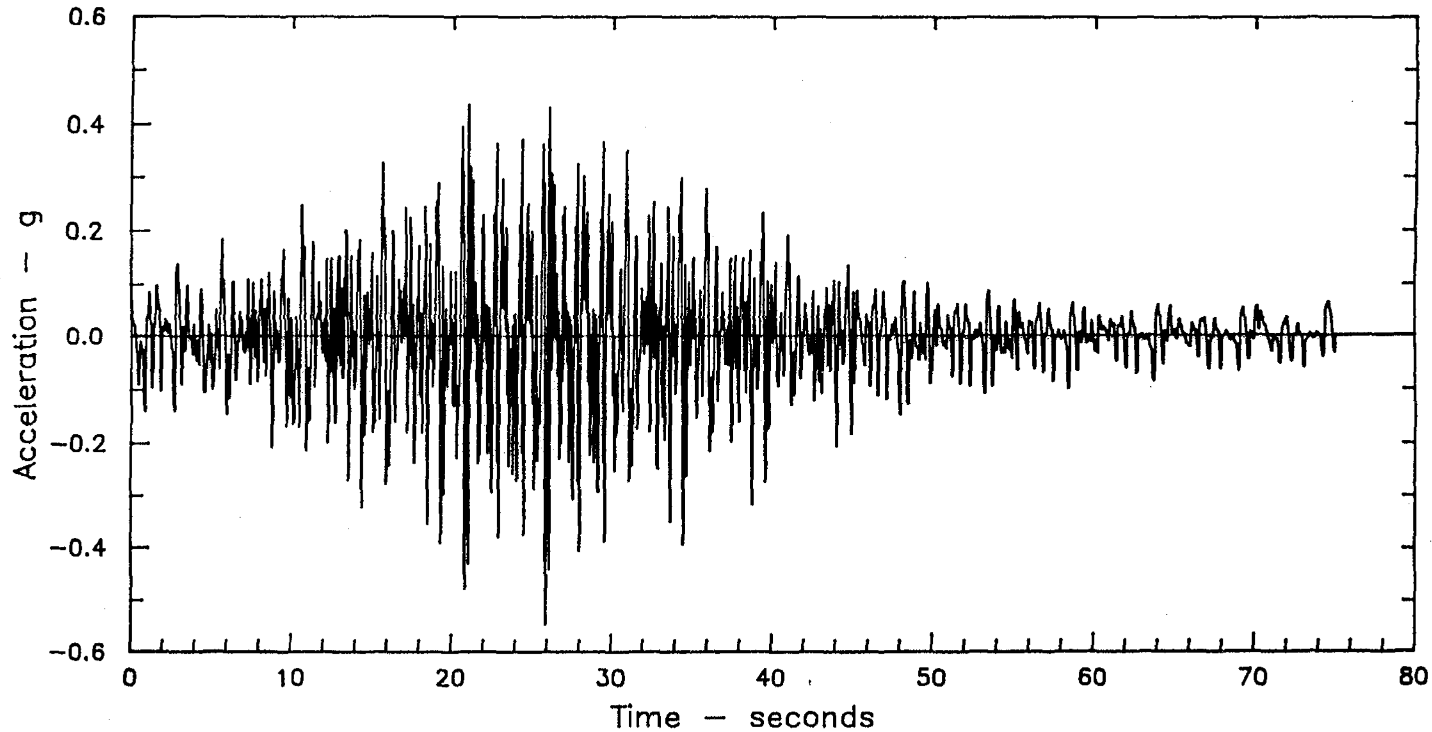


FIG. 22 SYNTHETIC RECORD FOR MAGNITUDE 8+ EARTHQUAKE MOTION ON ROCK AT A DISTANCE OF 6 MILES (modified after Seed and Idriss, 1969)

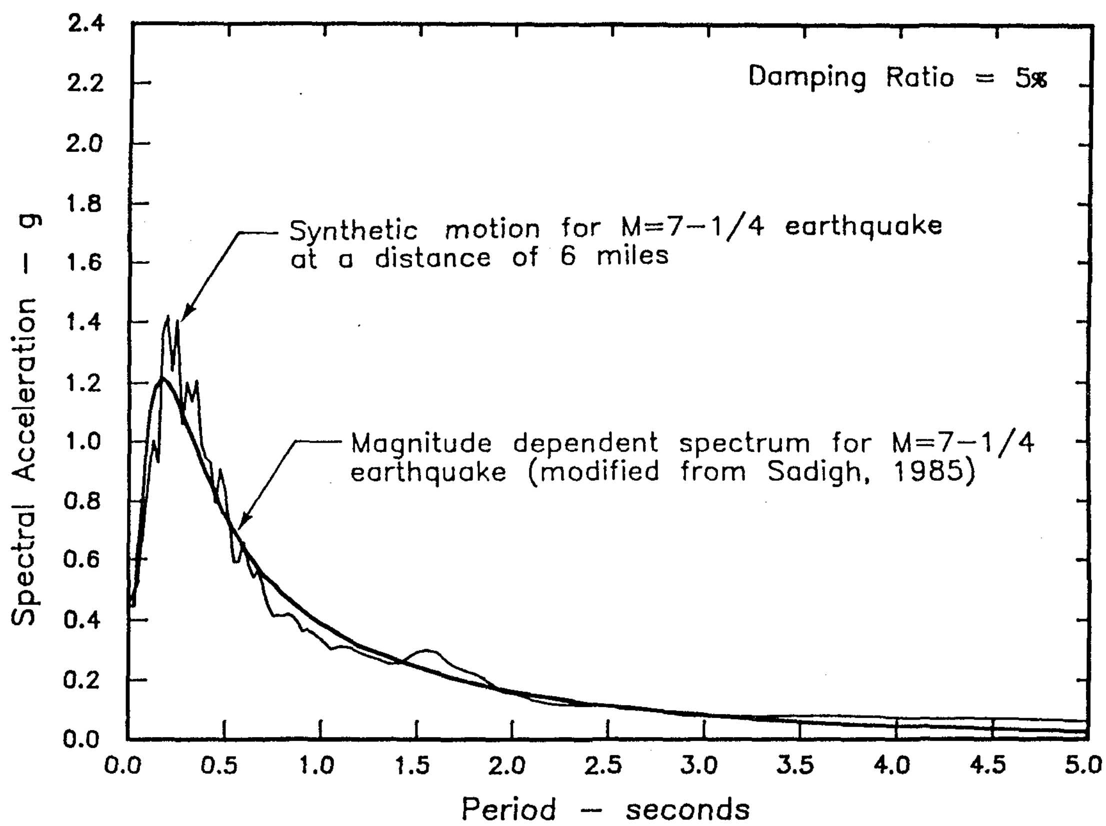


FIG. 23 SPECTRAL SHAPES FOR MAGNITUDE 7-1/4 EARTHQUAKE MOTION ON ROCK

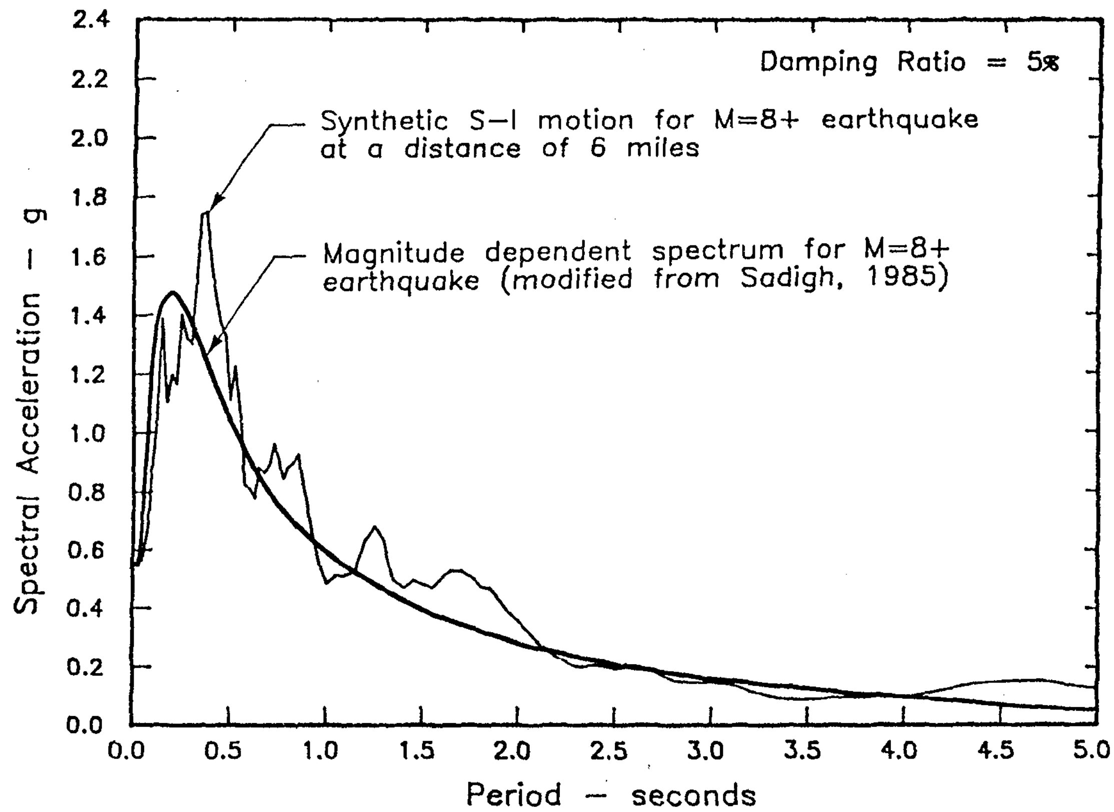


FIG. 24 SPECTRAL SHAPES FOR MAGNITUDE 8+ EARTHQUAKE MOTION ON ROCK

shaking. Computed surface motions were expressed in terms of the corresponding acceleration response spectra. Uncertainties in the measured shear wave velocity profiles, which played an important role in the ground response studies of the Mexico City sites (Seed, et al., 1987) were also taken into account in the analyses.

Ground Response Analyses for Magnitude $7\frac{1}{4}$ Earthquake Near San Francisco Bay Area

(1) Southern Pacific Building Site

Fig. 25 shows the soil profile for the Southern Pacific Building site together with the response spectra for both the rock outcrop motion and the computed surface motion for a hypothetical Magnitude $7\frac{1}{4}$ earthquake. The ground response analysis was made using the computer program SHAKE-86 (after Schnabel et al., 1971). For a peak acceleration of 0.45 g at a rock outcrop, the peak ground acceleration was computed to be about 0.26 g. The maximum strain induced in the soil profile was about 0.62% at a depth of 52 feet, while the site period lengthened from 1.08 seconds to 1.66 seconds due to the non-linear behavior of the soils.

As a result of the study for the Mexico City earthquake (Seed et al., 1987), which indicated that a 10 per cent uncertainty in the in-situ measured shear wave velocities can have significant effects on the computed ground surface motions, the effects of a similar variation in shear wave velocity on the computed surface motions were also evaluated for this site. The results of this study are shown in Fig. 26. It may be seen that the effect of a 10 per cent variation in shear wave velocities does not produce as significant an effect for the Southern Pacific Building site (shown in Fig. 26) as for the Mexico City sites.

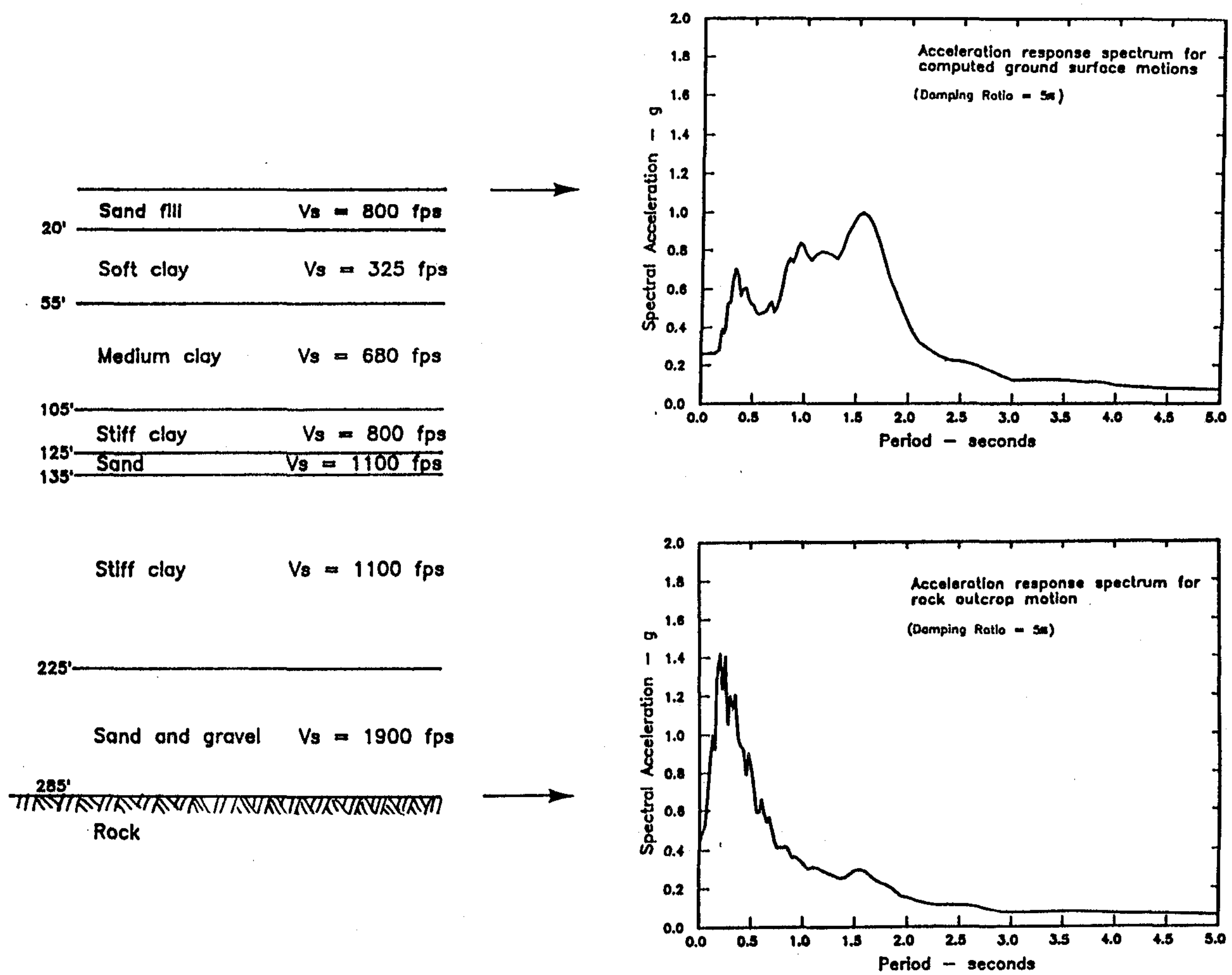


FIG. 25 RESULTS OF GROUND RESPONSE ANALYSES FOR SOUTHERN PACIFIC BUILDING SITE - MAGNITUDE 7-1/4 EARTHQUAKE AT A DISTANCE OF ABOUT 6 MILES

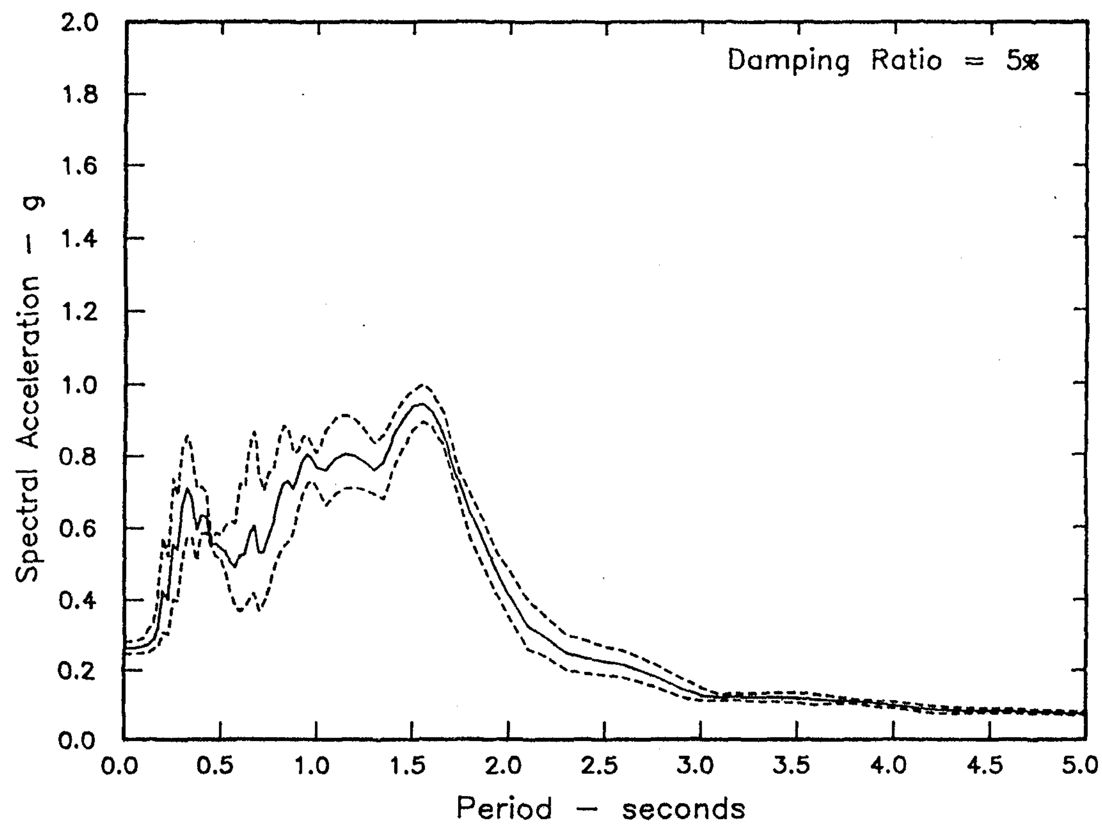


FIG. 26 INFLUENCE OF ±10 PERCENT VARIATION IN SHEAR WAVE VELOCITY ON RESPONSE SPECTRUM FOR COMPUTED GROUND SURFACE MOTION AT SOUTHERN PACIFIC BUILDING SITE - EARTHQUAKE MAGNITUDE = 7-1/4

(2) Embarcadero Center Site

Figure 27 shows the response spectrum for the computed surface motions at the Embarcadero Center site together with that for the rock outcrop motions used in the analyses. The computed peak ground acceleration was 0.3 g, the maximum induced strain was about 0.53% at a depth of 25 feet and the site period lengthened from 1.01 seconds to 1.36 seconds due to the non-linear behavior of soils resulting from seismic straining. A 10 per cent deviation from the measured shear wave velocities showed only a small effect on the response spectrum for the computed surface motions as shown in Fig. 28.

(3) Ravenswood Site

The results of the ground response analyses for the Ravenswood site in the south Bay are presented in Fig. 29 in the same format as before. The computed peak ground acceleration was 0.30 g, the maximum strain along the profile was about 0.48% at a depth of 30 feet and the site period was 2.54 seconds as computed from the strain-compatible soil properties. The initial small-strain site period was 1.81 seconds before seismic straining. The effect of a 10 per cent variation in measured shear wave velocities on the computed surface motions is relatively small, as shown in Fig. 30. It is interesting to observe, however, that the effects of this small variation in the shear wave velocities used in the soil profiles has a more noticeable influence on spectral accelerations in the lower period range, say below one second, than for the higher period range for all three sites included in this study (see Fig. 4-20, Fig. 4-22 and Fig. 4-24).

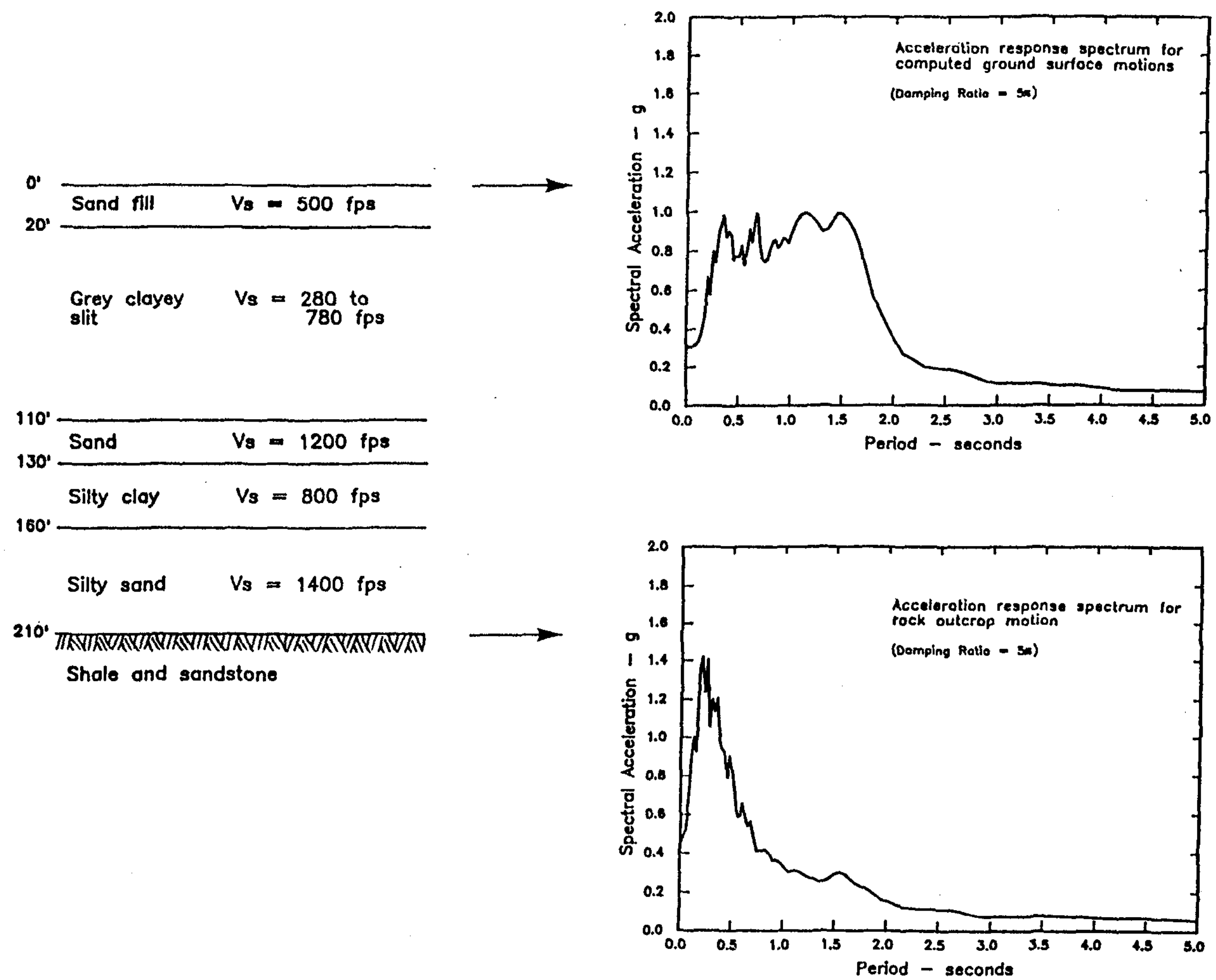


FIG. 27 RESULTS OF GROUND RESPONSE ANALYSES FOR EMBARCADERO CENTER SITE - MAGNITUDE 7-1/4 EARTHQUAKE AT A DISTANCE OF ABOUT 6 MILES

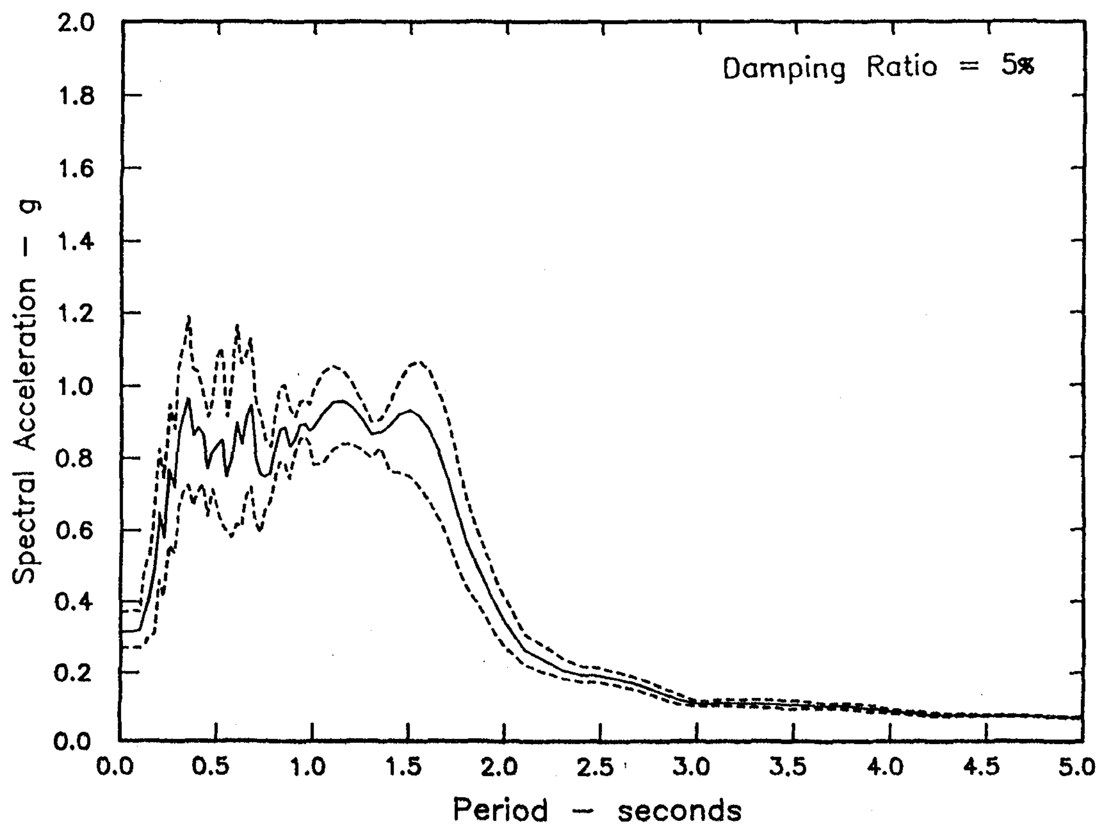


FIG. 28 INFLUENCE OF ±10 PERCENT VARIATION IN SHEAR WAVE VELOCITY ON RESPONSE SPECTRUM FOR COMPUTED GROUND SURFACE MOTION AT EMBARCADERO CENTER SITE - EARTHQUAKE MAGNITUDE = 7-1/4

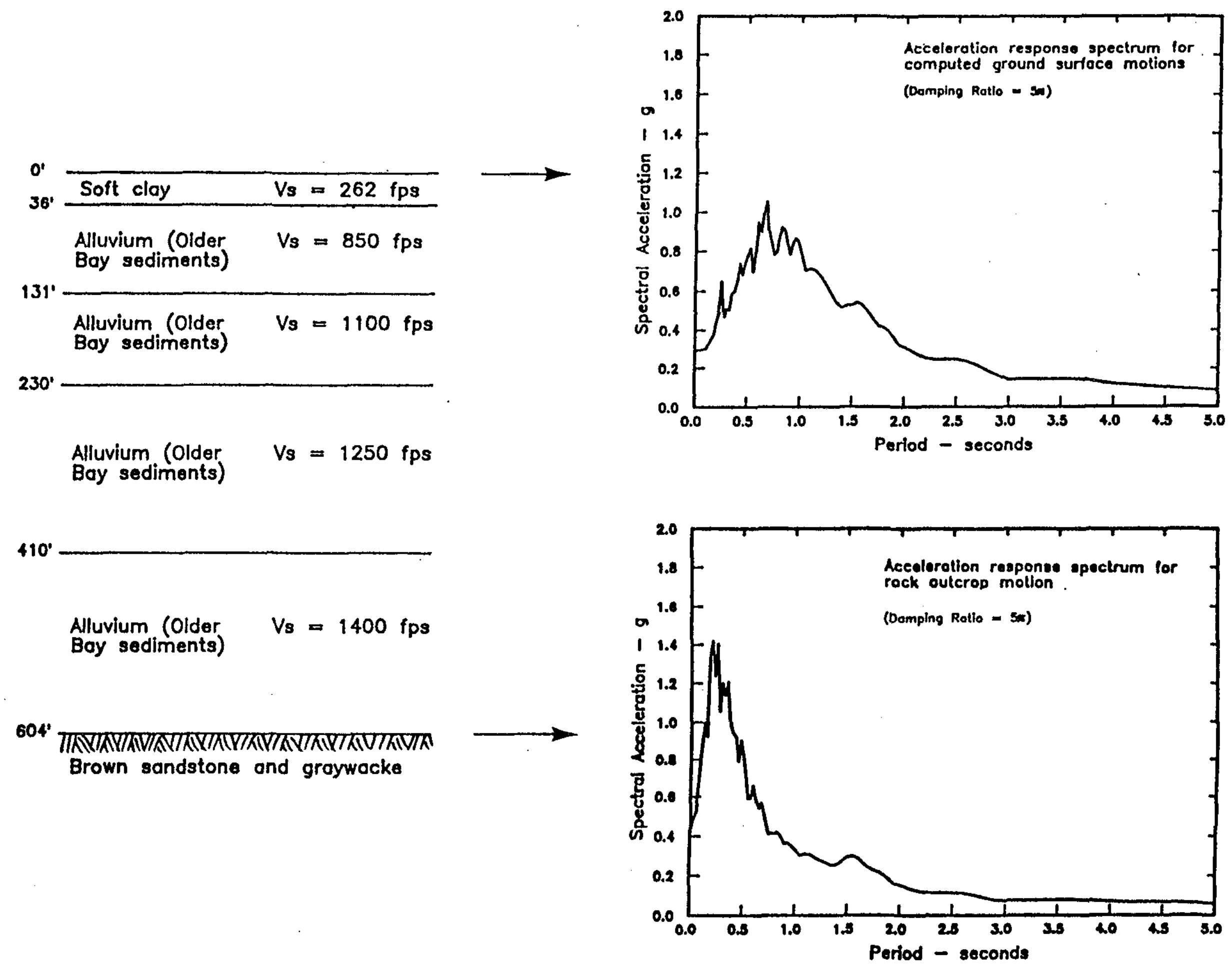


FIG. 29 RESULTS OF GROUND RESPONSE ANALYSES FOR RAVENSWOOD SITE - MAGNITUDE 7-1/4 EARTHQUAKE AT A DISTANCE OF ABOUT 6 MILES

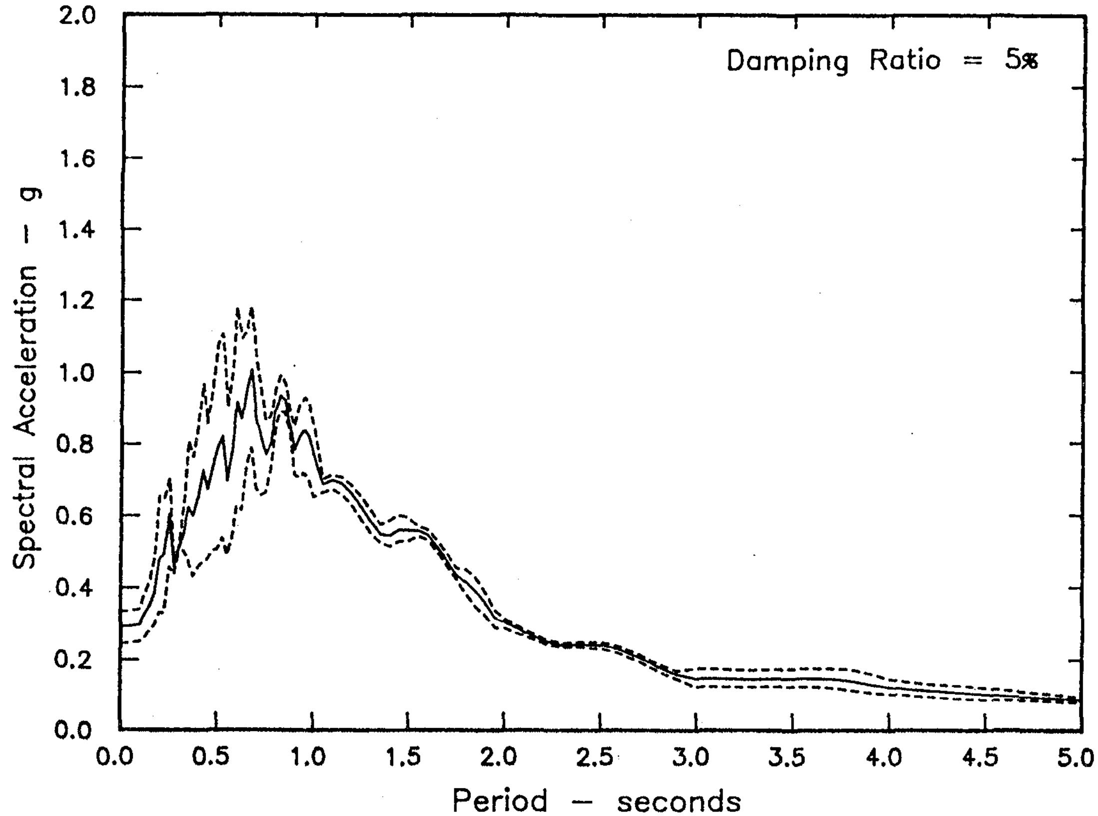


FIG. 30 INFLUENCE OF ±10 PERCENT VARIATION IN SHEAR WAVE VELOCITY ON RESPONSE SPECTRUM FOR COMPUTED GROUND SURFACE MOTION AT RAVENSWOOD SITE - EARTHQUAKE MAGNITUDE = 7-1/4

Summary of Ground Response Analyses for Magnitude $7\frac{1}{4}$ Earthquake

The acceleration response spectra for the computed surface motions at the three sites are summarized in Fig. 31. The peaks of the response spectra for all three sites reached values of about 1.0 g. A representative average spectrum is also shown in Fig. 31.

In the Applied Technology Conference study (ATC-3, 1978) three sets of site conditions were established and recommended for use in seismic building codes. The site conditions, which were later adopted by the Seismology Committee of the Structural Engineers Association of California (SEAOC) in their "Tentative Lateral Force Requirements" blue book, ranged from rock-like material and shallow stiff sites (S1), deep stiff and dense soil conditions (S2), to clay and loose sand site conditions (S3). Following a study of the damage in Mexico City during the Mexico earthquake of 1985, an additional site condition S4 was added to these categories,as shown in Table 9, to represent the anticipated response on deeper deposits of soft clays. Soft clays are described as clays with shear wave velocities generally in the range of 200 to 500 fps and a tentative spectrum for S4 soils was indicated. It is interesting to note that this response spectrum for a soil profile containing more than 40 feet of clay (the S4 site condition) is in excellent general agreement with the representative spectrum for Bayshore sites computed in this study for a Magnitude $7\frac{1}{4}$ earthquake, as indicated in Fig. 32.

The representative spectrum for bay-shore sites underlain by clay for a Magnitude $7\frac{1}{4}$ earthquake near the San Francisco Bay area shows significantly higher spectral acceleration values at periods less than about 2 seconds than the representative spectrum for the heavy damage area of Mexico City in the 1985 Mexico earthquake, as shown in Fig. 33.

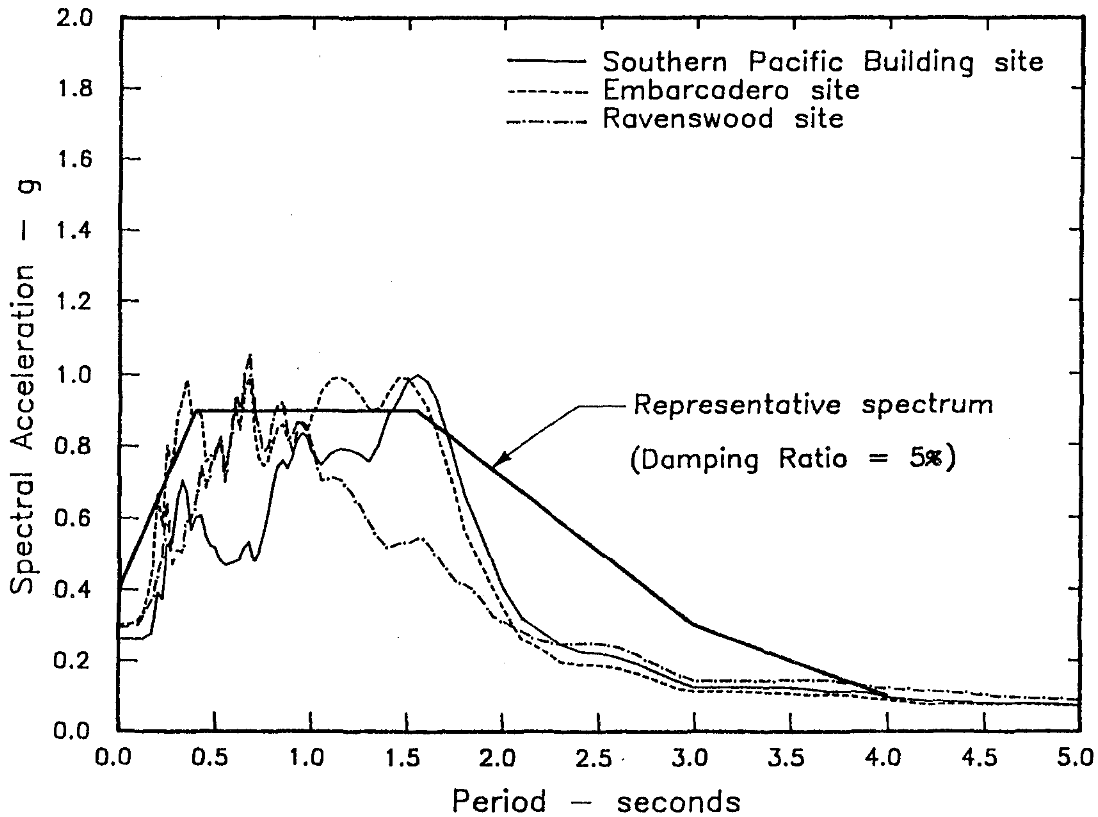


FIG. 31 REPRESENTATIVE SPECTRA FOR BAY-SHORE SITES IN M = 7-1/4 EARTHQUAKE OCCURRING AT DISTANCE OF ABOUT 6 MILES

TABLE 9 SITE COEFFICIENTS RECOMMENDED BY SEAOC (1988)

Type	Description	S Factor
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil thickness exceeds 200 feet, or a profile consisting of a thin layer of soft clay up to 20 feet thick overlying rocklike material.	1.2
S_3	A soil profile 40 feet or more in depth containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.	1.5
S_4	A soil profile containing more than 40 feet of soft clay. Alternatively soils falling into this category may have a design spectrum determined by special geotechnical study reflecting the soil specific conditions.	2.0

The site factor shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used unless the building official determines that S_4 type soil may exist at the site in which case S_4 shall be used.

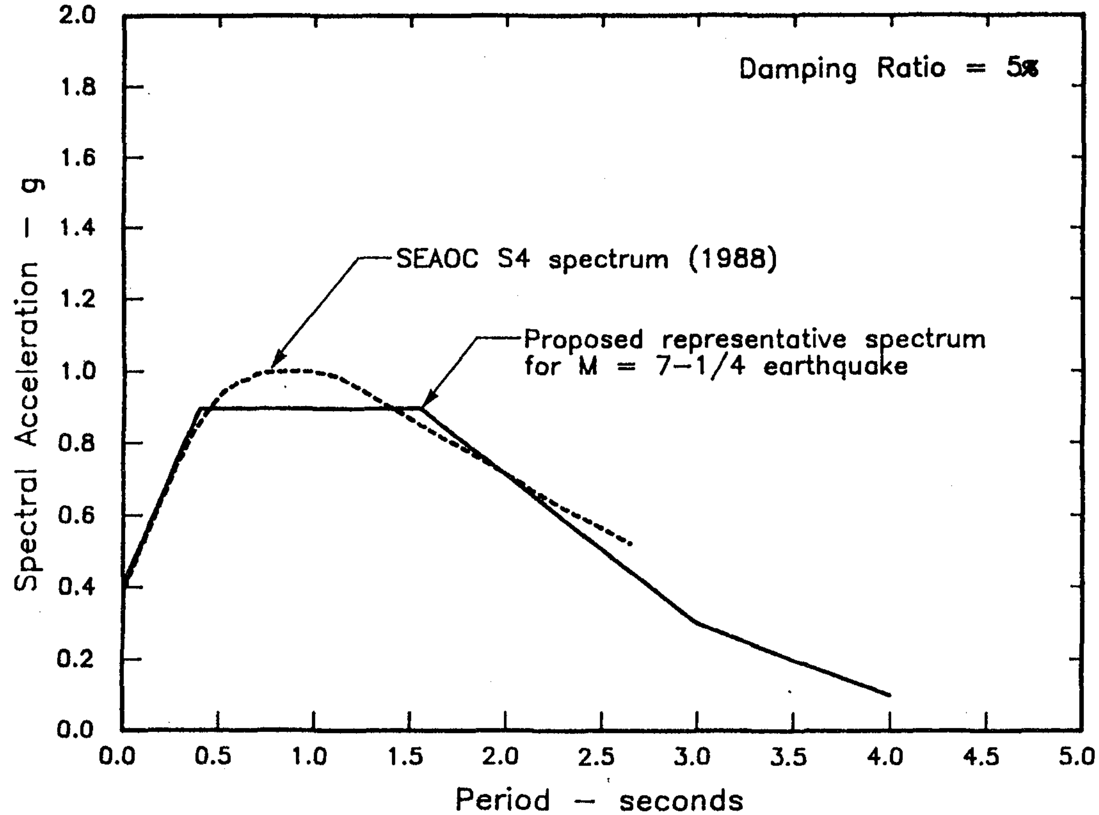


FIG. 32 REPRESENTATIVE SPECTRA FOR BAYSHORE SITES IN M = 7-1/4 EARTHQUAKE OCCURRING AT A DISTANCE OF ABOUT 6 MILES

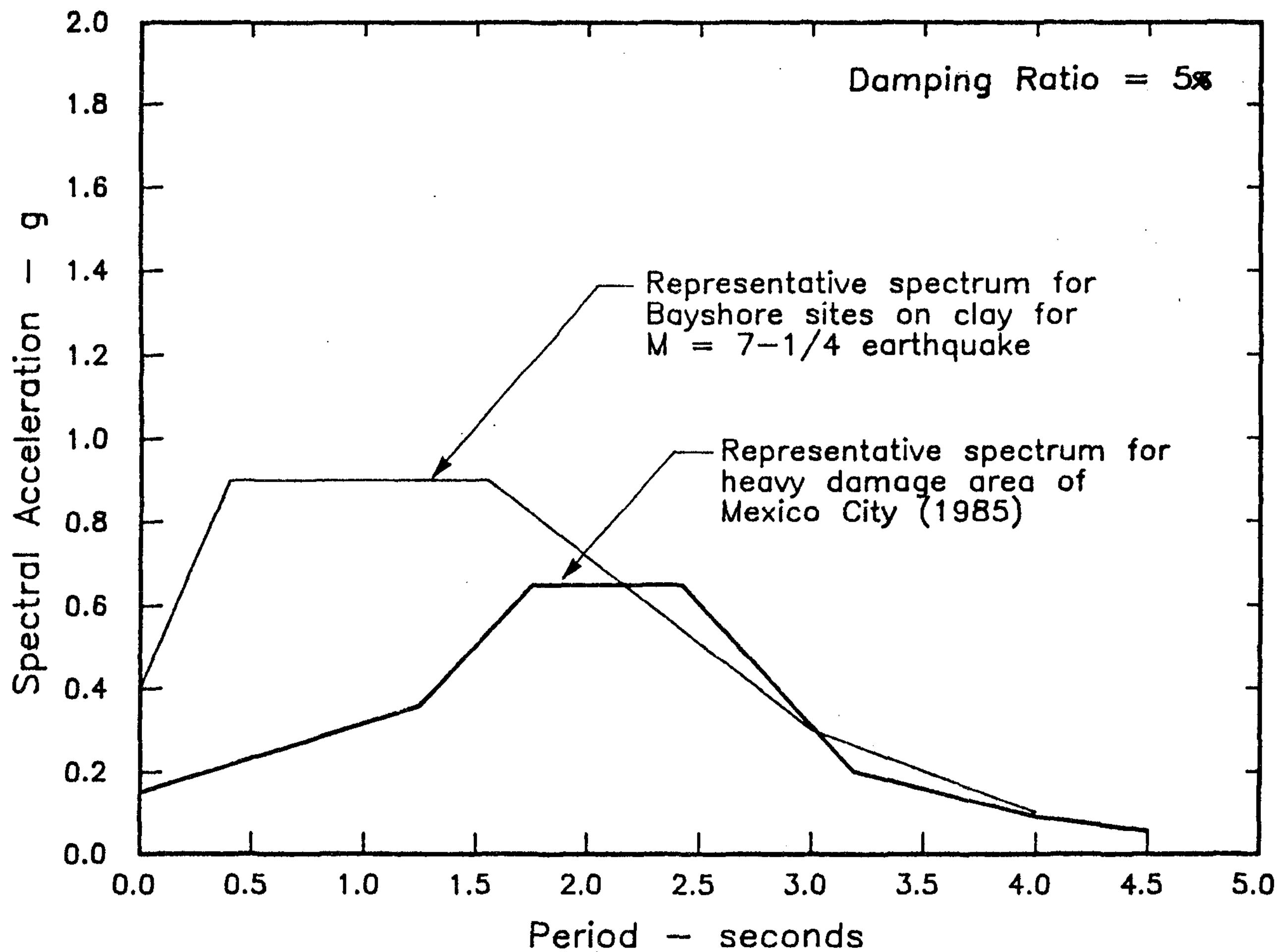


FIG. 33 COMPARISON OF HEAVY DAMAGE AREA OF MEXICO CITY (1985) AND MAGNITUDE 7-1/4 EARTHQUAKE ACTING NEAR SAN FRANCISCO BAY AREA

Ground Response Analyses for Magnitude 8+ Earthquake Near San Francisco Bay Area

Analyses were also made to compute the response of the same three sites to the rock motions corresponding to a Magnitude 8+ earthquake on the San Andreas fault located at a distance of about 6 miles, and producing a peak acceleration in rock of about 0.55 g. The rock motion record used in these analyses was that shown in Fig. 22. Analyses were made using the SHAKE-86 computer program.

(1) Southern Pacific Building Site

Fig. 34 shows the response spectra for the computed surface motions and the rock outcrop motions used in the analysis for the magnitude 8+ earthquake. The computed peak ground acceleration was about 0.34 g, the maximum shear strain developed in the soil profile was about 1% at a depth of 52 feet and the strain-compatible site period was 1.88 seconds as compared to the low strain-level site period of 1.08 seconds. The surface acceleration response spectrum for the Magnitude 8+ earthquake (Fig. 34) is significantly higher than that for the Magnitude $7\frac{1}{4}$ earthquake (Fig. 25). This is especially evident at longer periods. The higher acceleration level, the longer duration of the shaking, the more abundant long period motions in the 8+ rock outcrop motion used in the analyses, and the larger strain-softening effects of the soil, which reduce the ability of the soil to transmit high frequency motions effectively, are some of the factors that contribute to this higher response for the magnitude 8+ earthquake. The effects of small (±10%) variations in the shear wave velocities of the soils on the computed surface motions for the Southern Pacific Building site (Fig. 35), although more evident than for the Magnitude

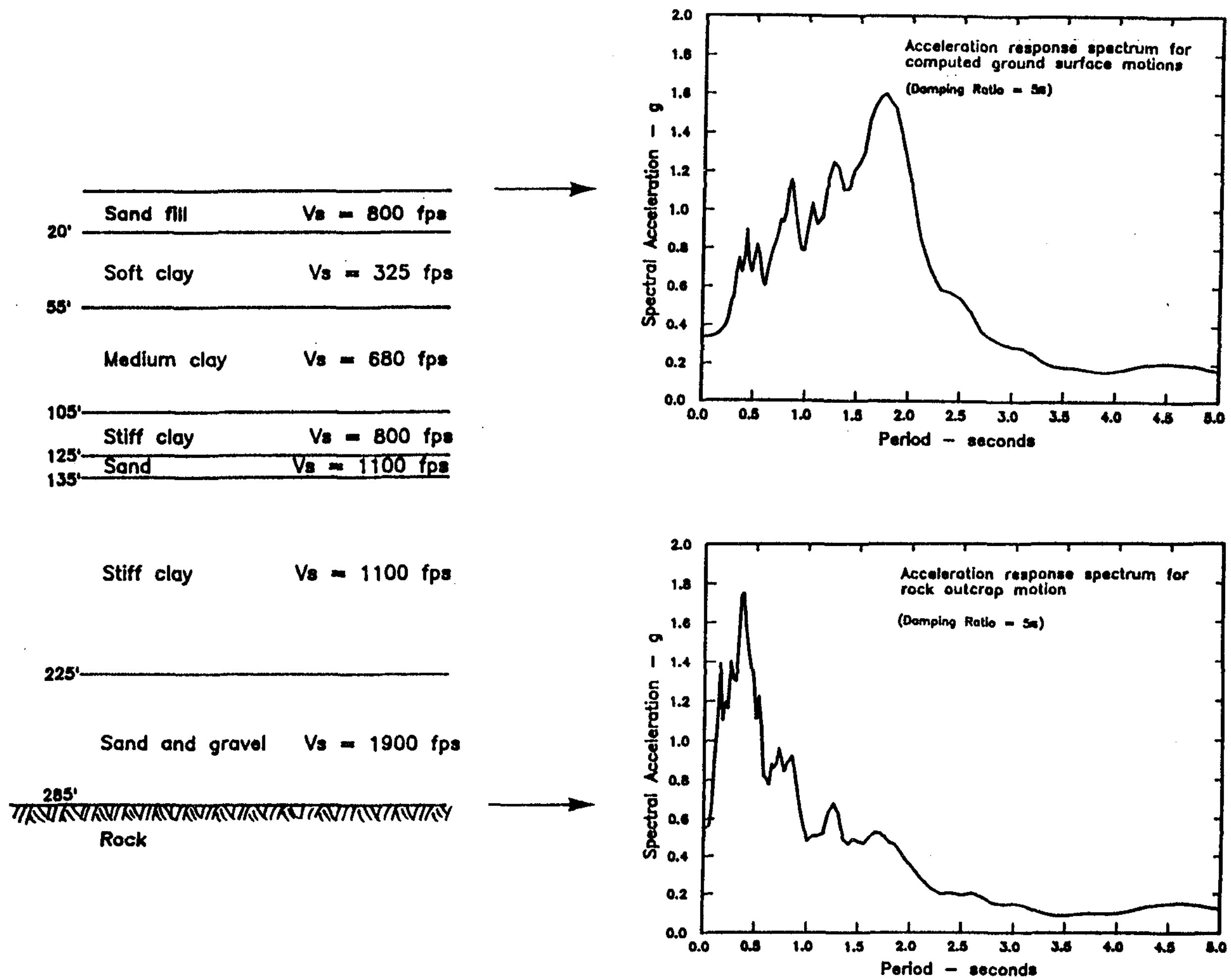


FIG. 34 RESULTS OF GROUND RESPONSE ANALYSES FOR SOUTHERN PACIFIC BUILDING SITE - MAGNITUDE 8+ EARTHQUAKE AT A DISTANCE OF ABOUT 6 MILES

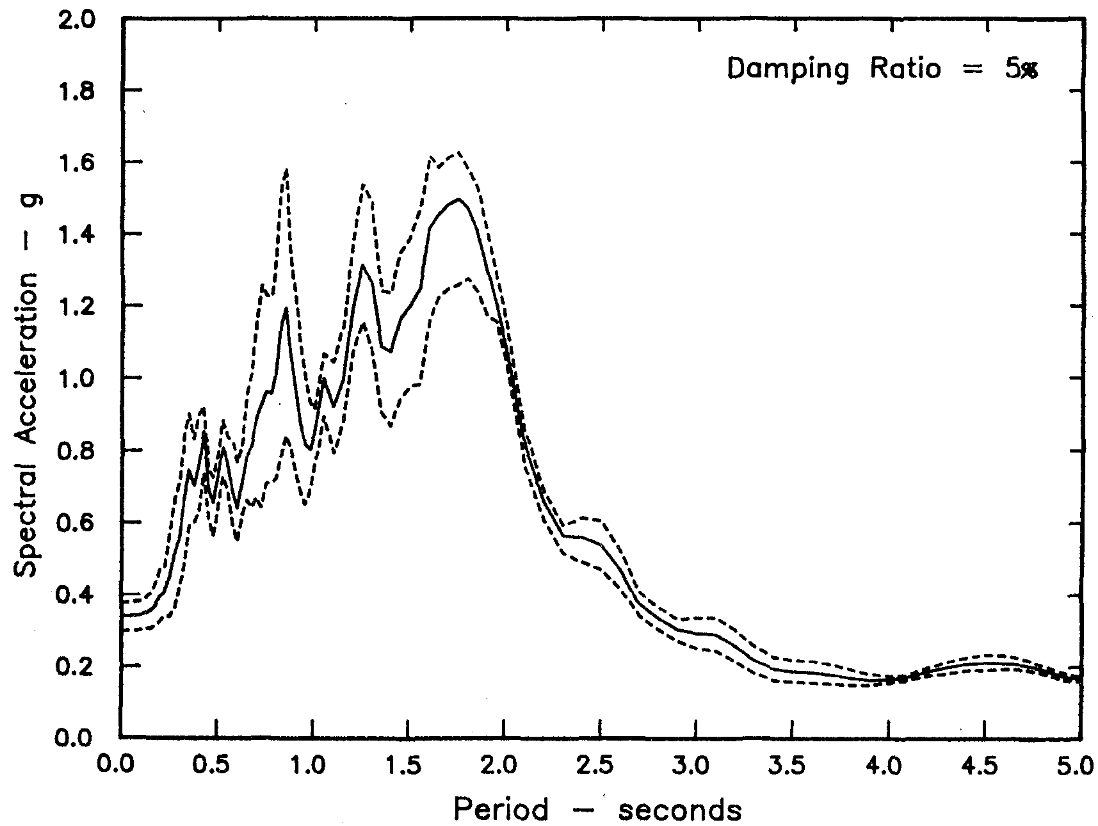


FIG. 35 INFLUENCE OF ±10 PERCENT VARIATION IN SHEAR WAVE VELOCITY ON RESPONSE SPECTRUM FOR COMPUTED GROUND SURFACE MOTION AT SOUTHERN PACIFIC BUILDING SITE - EARTHQUAKE MAGNITUDE = 8+

$7\frac{1}{4}$ earthquake (Fig. 26) for the same site, are still less significant than for Mexico City sites underlain by clay.

(2) Embarcadero Center Site

The results of the analyses for the Embarcadero Center site are shown in Figs. 36 and 37. The computed peak ground acceleration for this site was 0.45 g, the maximum shear strain developed in the soil profile was close to 1% at a depth of 25 feet, and the strain-compatible site period was computed to be 1.57 seconds compared with the initial value of 1.01 seconds determined from the in-situ measured shear wave velocities. Fig. 36 shows the response spectra for the computed surface motions and the rock outcrop motions used in the analyses. By comparing these two spectra, it may be seen that the high frequency motions are generally attenuated and the long period motions are amplified, with the boundary lying at a period of about 0.7 seconds. This phenomenon was also observed for the other two sites analyzed. It can be seen in Fig. 37 that the effects of small variations in shear wave velocities have only a limited influence on the computed ground motions for the Embarcadero Center Site.

(3) Ravenswood Site

Figure 38 shows the analytical results for the Ravenswood site, together with the soil profile for the site. The computed peak ground acceleration was 0.36 g, the maximum shear strain developed along the soil profile was about 1% at a depth of 30 feet, and the period computed from the strain compatible soil properties was 3.24 seconds as compared with the low strain-level site period of 1.81 seconds. The limited effects on the computed surface response of a ±10% variation in shear wave velocities for the analytical model are shown in Fig. 39.

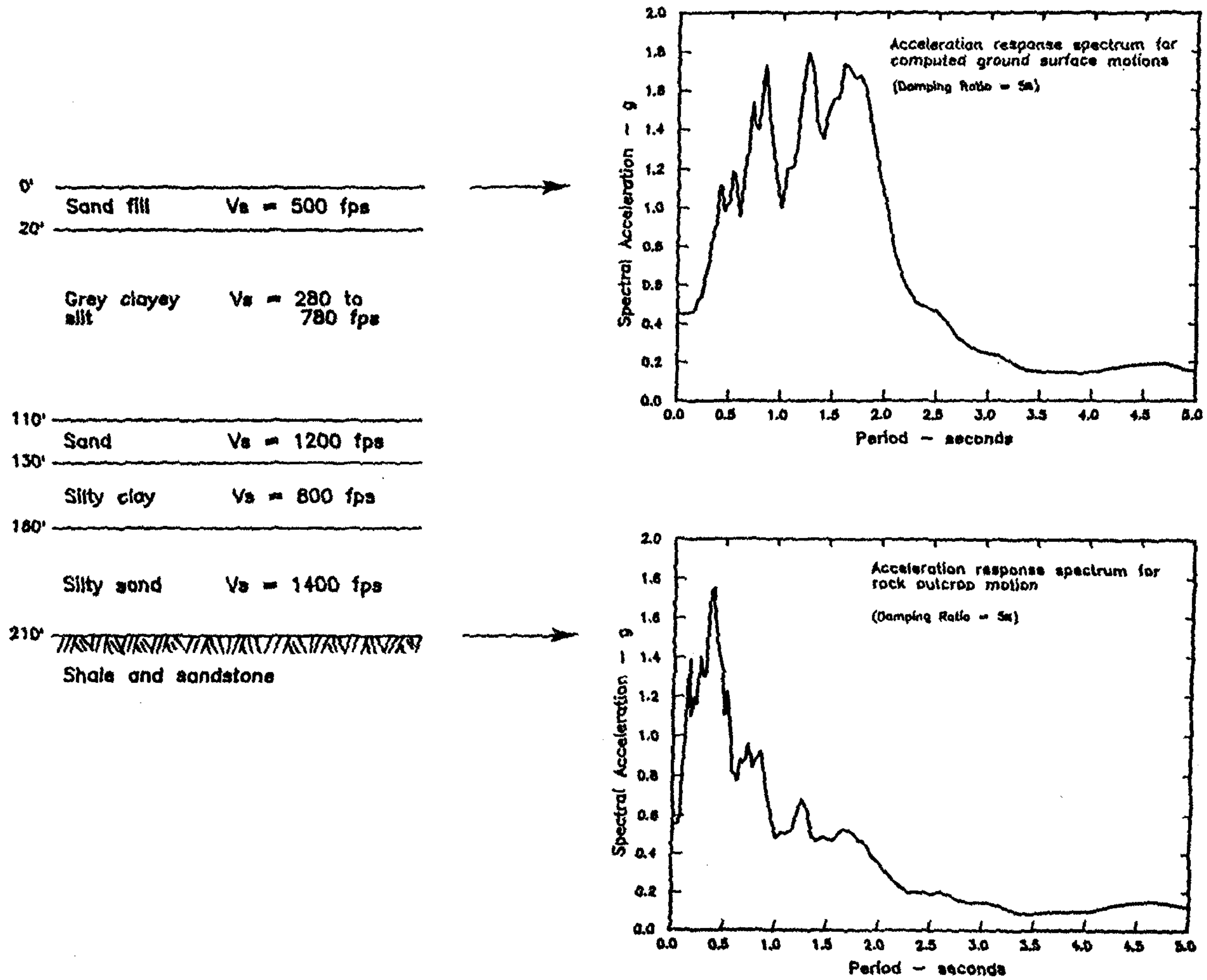


FIG. 36 RESULTS OF GROUND RESPONSE ANALYSES FOR EMBARCADERO CENTER SITE - MAGNITUDE 8+ EARTHQUAKE AT A DISTANCE OF ABOUT 6 MILES

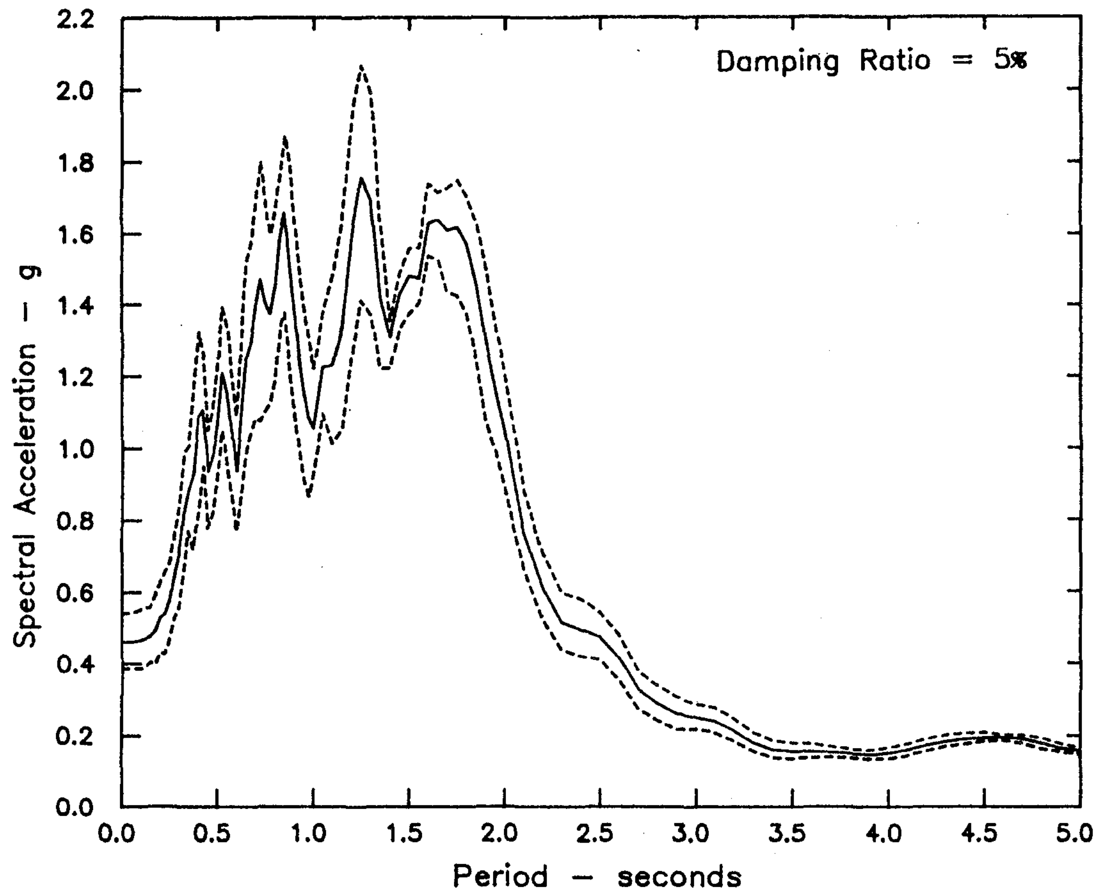


FIG. 37 INFLUENCE OF ±10 PERCENT VARIATION IN SHEAR WAVE VELOCITY ON RESPONSE SPECTRUM FOR COMPUTED GROUND SURFACE MOTION AT EMBARCADERO CENTER SITE - EARTHQUAKE MAGNITUDE = 8+

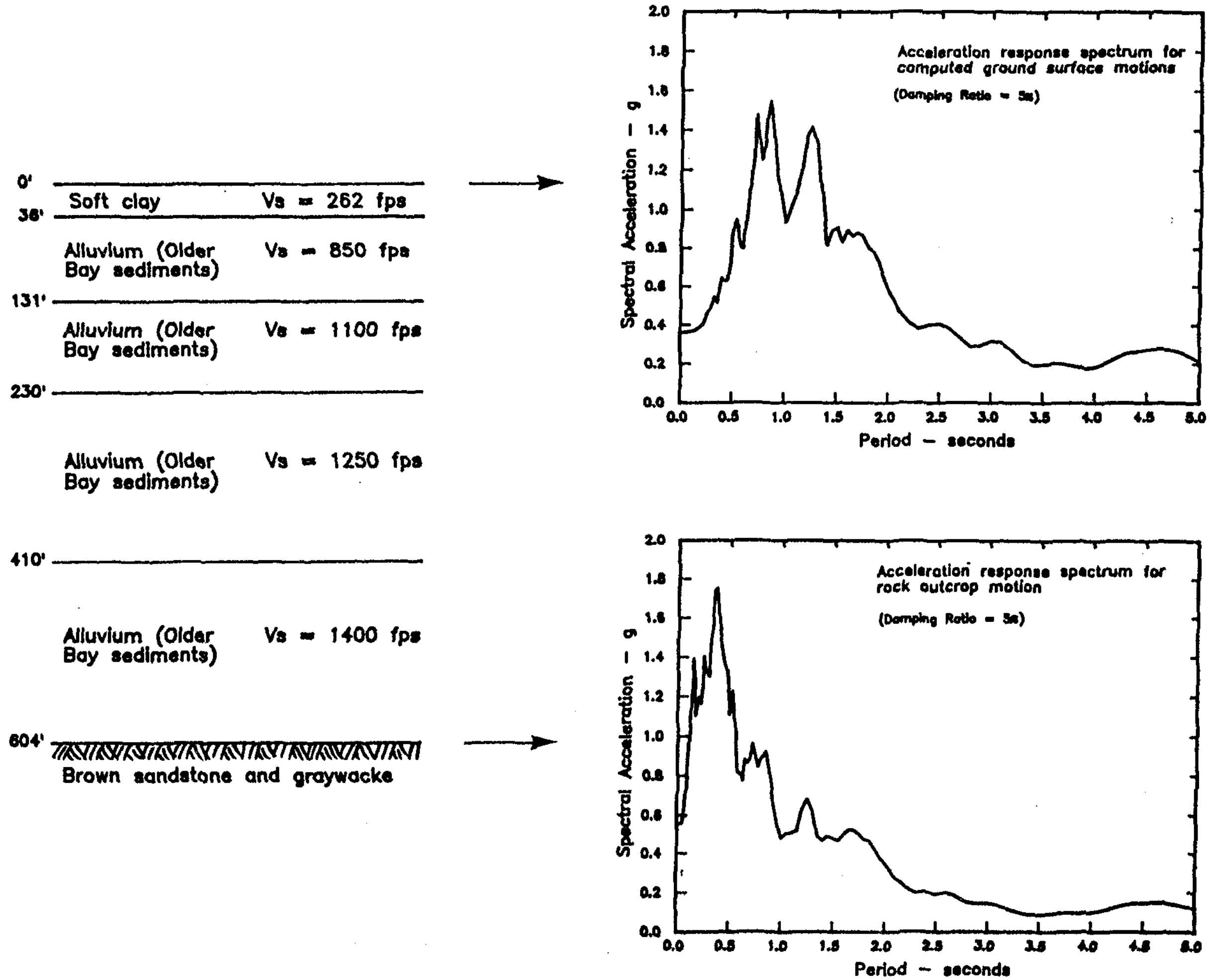


FIG. 38 RESULTS OF GROUND RESPONSE ANALYSES FOR RAVENSWOOD USGS SITE - MAGNITUDE 8+ EARTHQUAKE AT A DISTANCE OF ABOUT 6 MILES

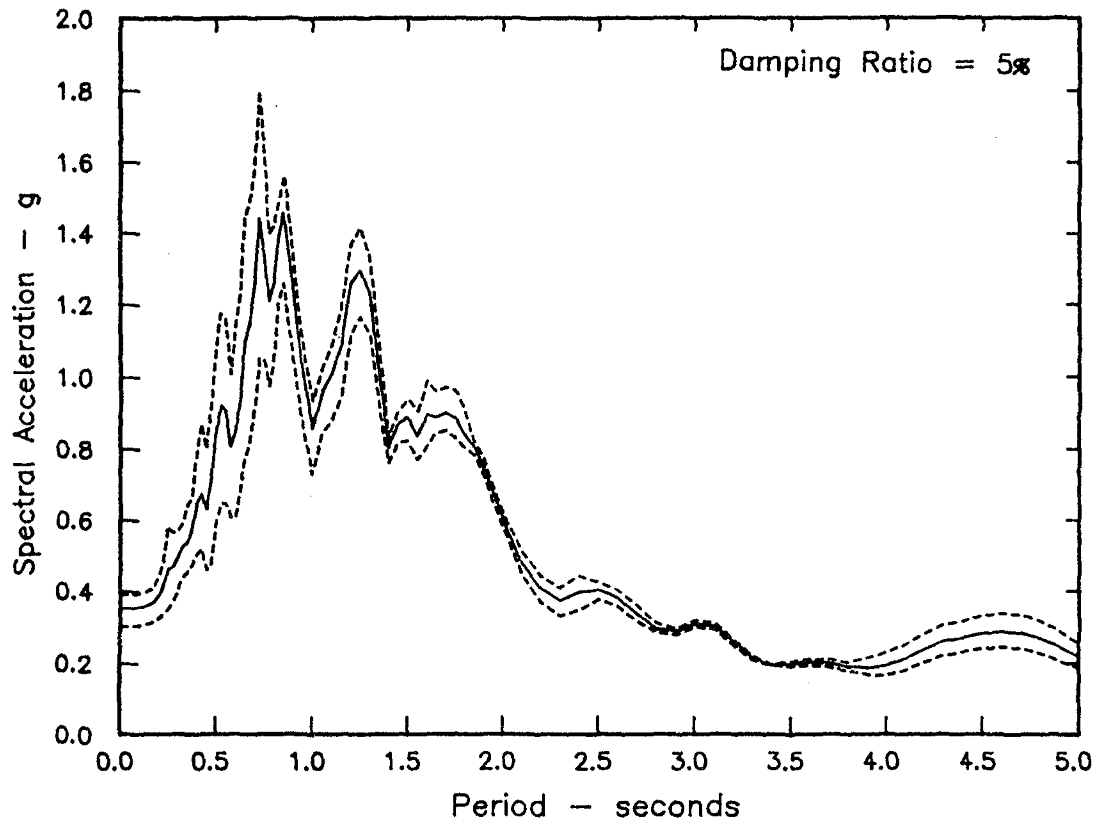


FIG. 39 INFLUENCE OF ±10 PERCENT VARIATION IN SHEAR WAVE VELOCITY ON RESPONSE SPECTRUM FOR COMPUTED GROUND SURFACE MOTION AT RAVENSWOOD SITE - EARTHQUAKE MAGNITUDE = 8+

Summary of Ground Response Analyses for Magnitude 8+ Earthquake

The surface response spectra for the three sites subjected to a Magnitude 8+ earthquake are summarized in Fig. 40 together with a representative spectrum drawn for these three sites. Fig. 41 compares this spectrum, which represents the expected response for San Francisco Bayshore sites underlain by clay and subjected to a Magnitude 8+ earthquake on San Andreas fault, with the representative spectrum for the heavy-damage area in Mexico City in the 1985 Mexico earthquake. Based on this study, it can be seen that the San Francisco Bayshore sites will respond much more strongly than did the heavy-damage areas in Mexico City in the 1985 earthquake. The ordinates on the acceleration response spectrum for the Bayshore sites for a Magnitude 8+ earthquake on the nearby San Andreas fault are on the average about 100% greater than those for the heavy damage area of Mexico City, as shown in Fig. 41. This result does not seem unreasonable when it is considered that the Magnitude 8.3 earthquake, which caused so much damage in Mexico City in 1985, had its source about 350 kms from the city, whereas the potential source of the strong shaking for San Francisco sites, is located at a distance of only about 10 kms from the Bayshore sites underlain by soft clay.

Ground Motions in Central Parts of San Francisco

As can be seen from Fig. 11, by far the greater part of the city of San Francisco is underlain either directly by bedrock or by shallow alluvial deposits of 150 feet or less in thickness. A significant amount of earthquake data has been gathered for sites having similar geological conditions in recent years, and the availability of this information makes it possible to make reasonable assessments of probable ground motion spectra

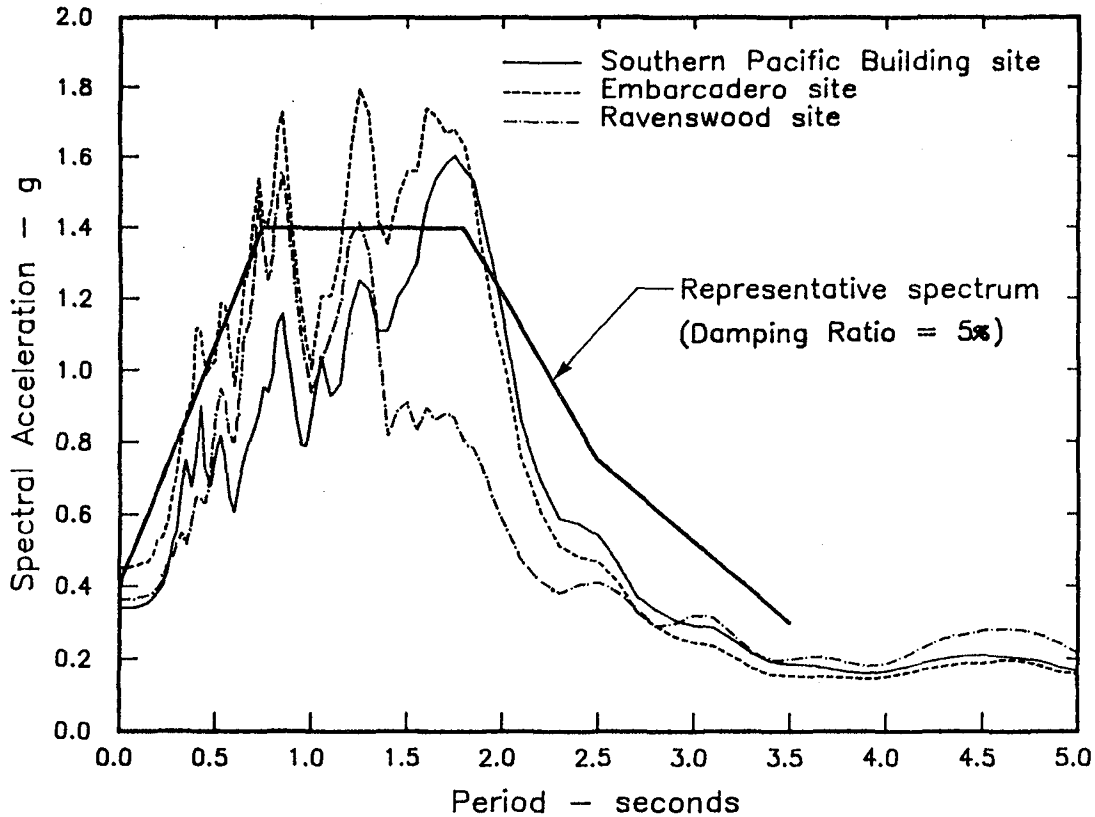


FIG. 40 REPRESENTATIVE SPECTRUM FOR THREE BAY-SHORE SITES WITH M = 8+ EARTHQUAKE OCCURRING AT A DISTANCE OF ABOUT 6 MILES

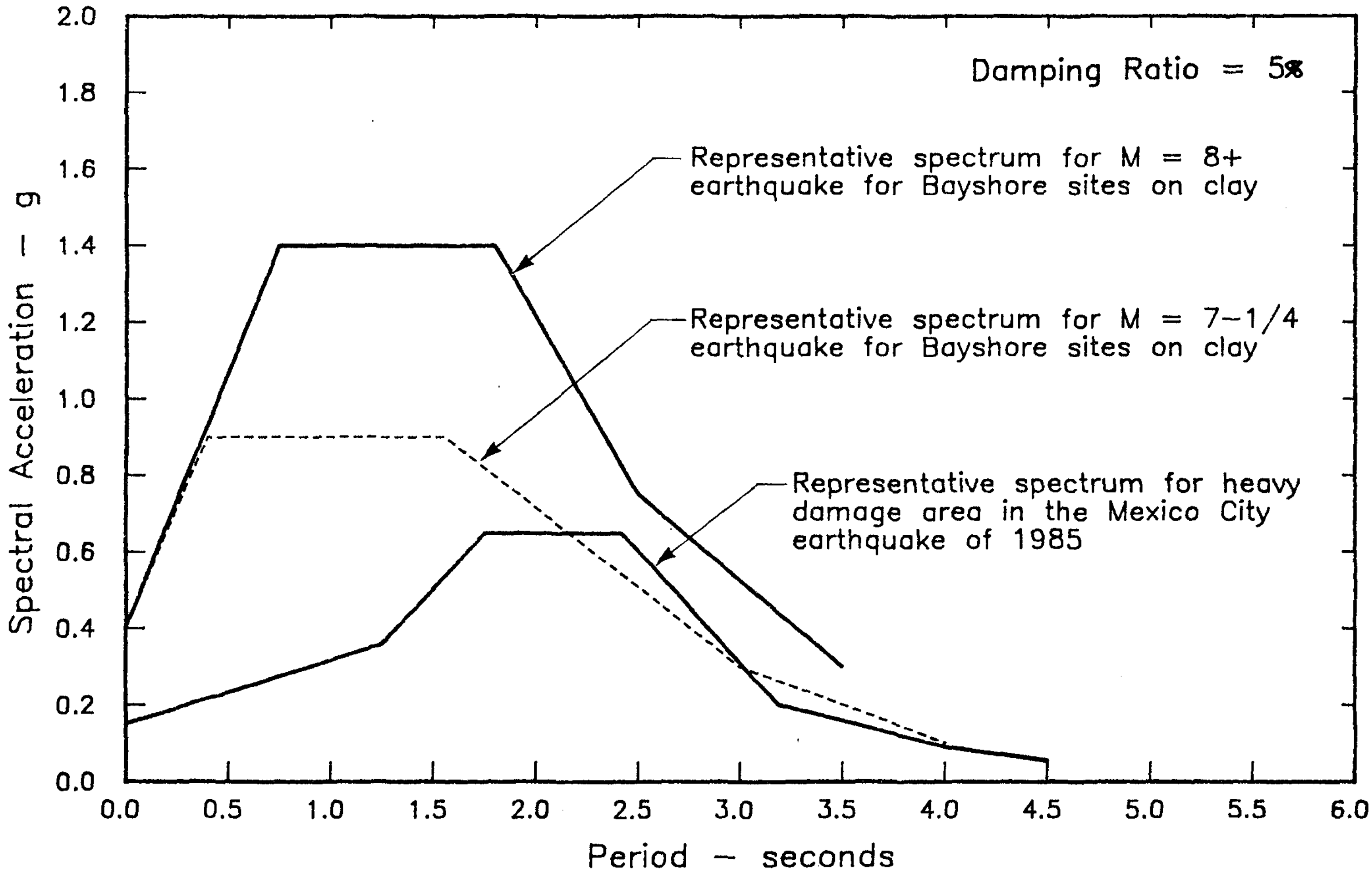


FIG. 41 COMPARISON OF REPRESENTATIVE SPECTRUM FOR HEAVY DAMAGE AREA OF MEXICO CITY (1985) AND BAYSHORE SITES IN SAN FRANCISCO FOR MAGNITUDE = 7-1/4 AND 8+ EARTHQUAKES

for such conditions on the basis of available empirical data. Thus, for example, the shape of the normalized average response spectral curves for such site conditions can be expected to be generally similar to those shown in Fig. 42 (after ATC-3 report). Other data provides a basis for estimating probable levels of peak accelerations for such sites for different combinations of earthquake magnitude and distance of earthquake energy sources. On this basis, estimates of mean spectral shapes for the stiff soil conditions in the main parts of San Francisco were determined for earthquakes with Magnitudes of $7\frac{1}{4}$ and 8+, occurring on the San Andreas fault system. The maximum surface accelerations at stiff soil sites for these two earthquake magnitudes were determined to be about 0.45 g and 0.55 g respectively. These values are very similar to the peak accelerations likely to be developed in rock outcrops, as used in the analyses for San Francisco Bayshore sites.

Representative surface acceleration response spectra for stiff soil sites in San Francisco (rock and stiff soil conditions) are shown in Fig. 43 for earthquakes with Magnitudes of $7\frac{1}{4}$ and 8+. For comparison purposes, the representative ground motion spectrum for the heavy damage zone of Mexico City (1985) is also shown. It can be seen that the spectral accelerations in San Francisco for buildings with periods up to about 1.25 seconds are significantly higher than those for the heavy-damage area of Mexico City in the 1985 earthquake.

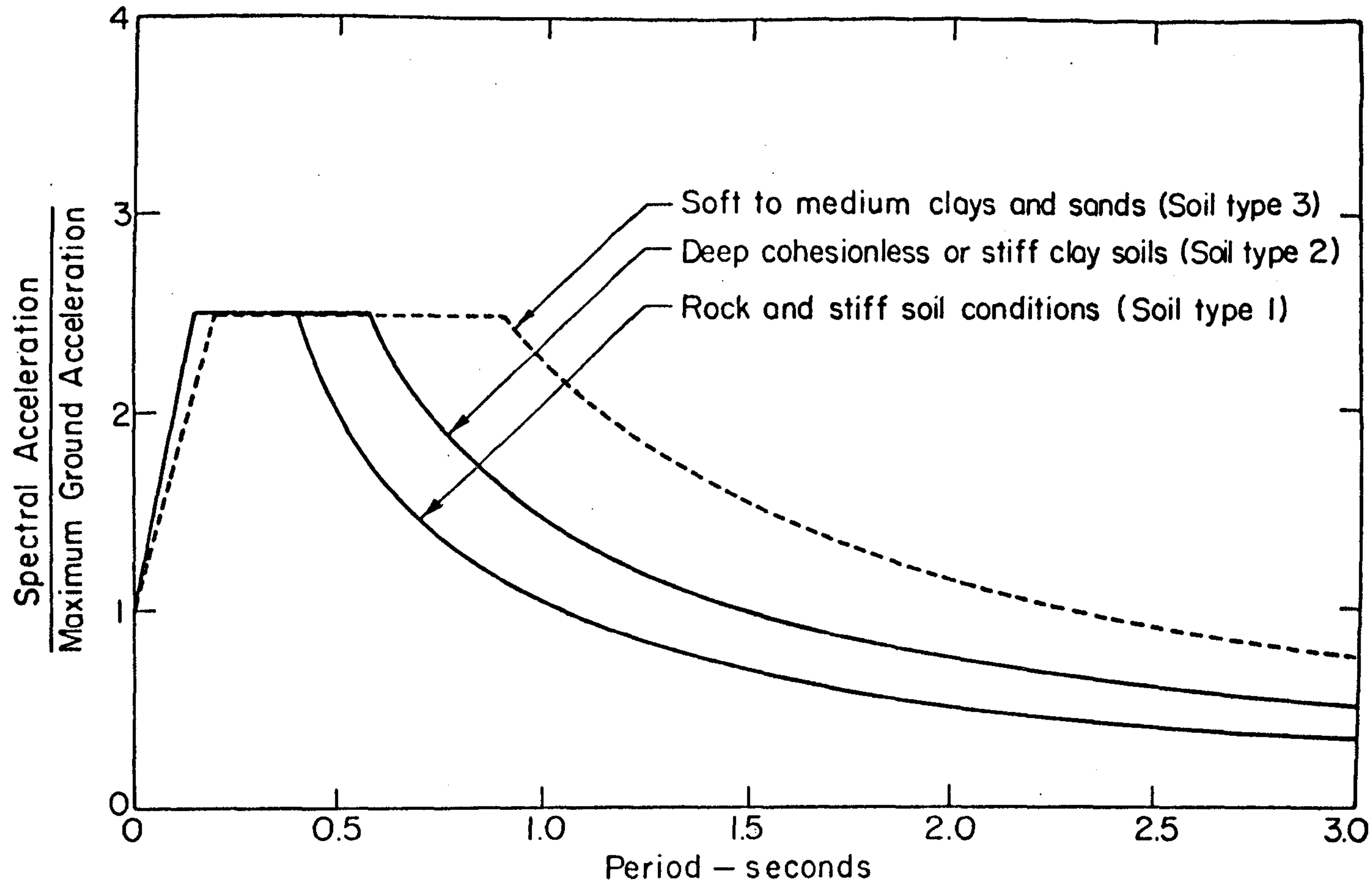


FIG. 42 AVERAGE SPECTRA FOR DIFFERENT SITE CONDITIONS (after ATC-3, 1978)

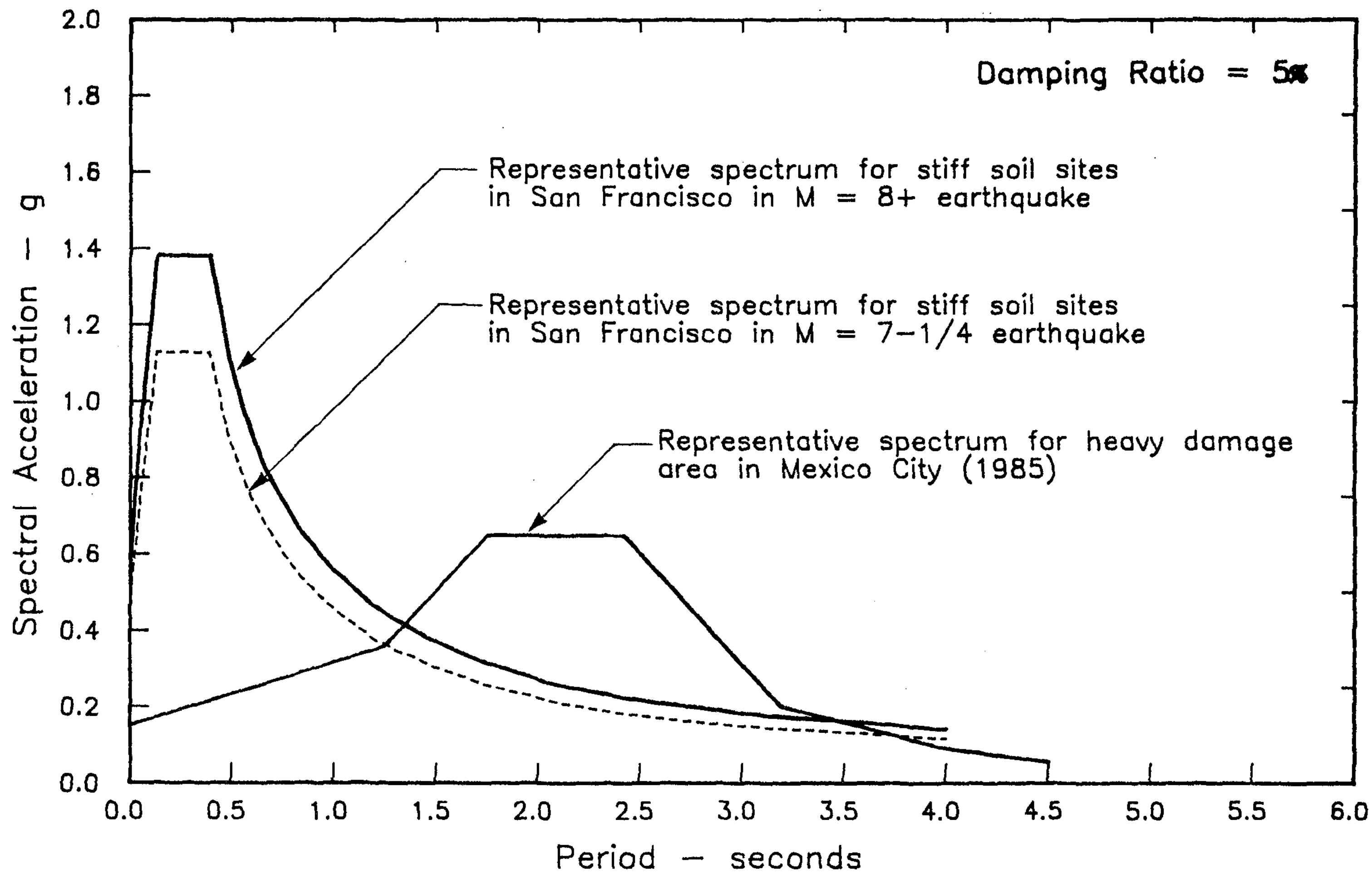


FIG. 43 COMPARISON OF REPRESENTATIVE SPECTRUM FOR HEAVY DAMAGE AREA OF MEXICO CITY (1985) AND STIFF SOIL SITES IN SAN FRANCISCO FOR MAGNITUDE = 7-1/4 AND 8+ EARTHQUAKES

IV. EARTHQUAKE-RESISTANT DESIGN PROVISIONS FOR CALIFORNIA

Earthquake-resistant design provisions first came into practice in California in the early 1900's. Shortly after the 1906 earthquake, the City of San Francisco was rebuilt under provisions requiring the use of a uniform 30 psf lateral pressure to represent the potential effects of wind and earthquake loadings.

The concept of using lateral earthquake forces which are proportional to structural masses seems to have been first introduced in the 1927 Uniform Building Code, with the proportionality constant ranging from about 7.5% to 10% and being dependent on the bearing capacity of the foundation soil.

Shortly after the invention of the strong motion seismograph, and as a consequence of the 1933 Long Beach earthquake, the Building Code for the City of Los Angeles was revised and in this revision the importance of different structural systems was recognized; masonry buildings without frames were assigned the highest lateral force coefficient of 10%, while for other structural systems the coefficient was assigned values of 2% to 5%.

In 1943, the importance of building flexibility on design lateral forces was recognized and the lateral force coefficient was expressed as a function of the number of stories in the structure, with higher values for low-rise stiffer structures than for higher more flexible buildings.

In 1952, a joint committee on Lateral Forces of the San Francisco Section, ASCE and the Structural Engineers Association of Northern California recommended a code in which design lateral force coefficients were related to the natural periods of structures through a coefficient $C = f(T)$. Thus the lateral force coefficients were determined in terms of the type of structure (K) and the natural period of the structure (T). After a further revision of the factors K and C in 1957, values of the lateral force coefficient ranged from about 3.5% to 7.5%. In 1959, the first SEAOC Recommendation on Lateral Forces was published. Although the basic concepts for determining the lateral force coefficient remained the same, the influence of building periods was changed so that more conservatism was incorporated in values required for the design of taller buildings.

The SEAOC Code has been constantly under revision since 1959; however, it was not until a re-evaluation of the Code in the light of damage caused by the 1971 San Fernando earthquake that further major modifications were introduced. The new recommendations at that time introduced several new ideas into the design provisions for California, including the concept of soil/structure response interaction and a categorization of the importance of structures. A detailed description of this code, which has been applied for the past twelve years, will be given in the following section.

In 1978, a comprehensive document on earthquake-resistant design, entitled "Tentative Provisions for Development of Seismic Regulations for Buildings," was published by the Applied Technology Council. This document, commonly referred to as ATC-3, was intended to serve as the basis of a nationally-recognized model code for earthquake-resistant design. The 1988 SEAOC report on "Recommended Lateral Force Requirements and Tentative Commentary" has adopted some of the concepts of the ATC-3 approach. One of the major changes in the 1988 'Recommendations' is that the Soil/Structure Resonance Factor, S, approach is replaced by site-dependent ground motion spectra and lateral force coefficients. A detailed description of the new tentative code will be presented in a later section of this report.

Current Seismic Design Provisions for Structures in California

The current seismic design provisions for California (UBC Code, 1976) were originally recommended by the Seismology Committee of the Structural Engineers Association of California (SEAOC) in 1974 in recognition of the extensive damage in the 1971 San Fernando earthquake. Under these provisions, which represent current practice, the design lateral force for a structure is determined by the expression:

$$V = Z\ I\ K\ C\ S\ W \tag{1}$$

where Z = a seismic zoning factor whose value depends on the seismic zone in which the structure is located, ranging from 0.25 to 1. For the San Francisco Bay area, and many other areas that are close to (15 to 25 miles) from major fault systems in California, Z is assigned the highest value of its four categories, i.e., Z = 1.

I = an occupancy importance coefficient to provide for the assignment of higher force levels to structures housing certain critical facilities.

K = a factor determined by the type of structural system used. This factor is intended to account for difference in the available ductilities or energy-dissipation capacities of various structural systems. The values of K assigned to each structural system have been influenced to a large extent by observations of the performance of these systems in actual earthquakes.

C = a factor related to the building period and equal to $1/15\sqrt{T}$.

T = building period in seconds

S = a soil/structure interaction factor which is a function of the ratio of the building period T to the site period T_s. This factor was introduced as a result of several major earthquakes that occurred in the 1960's, e.g., the 1963 Skopje earthquake, the 1967 Caracas earthquake, the 1970 Gediz earthquake and the 1971 San Fernando earthquake, which indicated that structural damage intensity is related to, among other factors, the natural period of the structure and the fundamental period of the underlying soil deposits.

and W = weight of the building.

Table 10 lists typical ranges of values for the factors used in the expression for the determination of the lateral force coefficient. For typical buildings of normal importance constructed in the San Francisco Bay area, the coefficients Z, I, K used for calculating the design lateral force coefficient are 1.0, 1.0 and 0.8 to 1.0 respectively. Site periods are determined from ground response analyses or by means of a recommended procedure specified in Appendix B of "Recommended Lateral Force Requirements and Commentary" (1980) by the Seismology Committee of SEAOC. The site-structure resonance factor, S, shown in Fig. 44, is a direct function of the ratio of building period to site period (T/T_s). The curve defining the S factor reaches a peak value of 1.5 when the building period coincides with the site period, i.e., $T/T_s = 1.0$.

With all parameters involved in the determination of the design lateral force thus established, the lateral force coefficient (expressed as a fraction of the building weight) can readily be obtained as the product of the parameters Z, I, K, C and S, while at the same time satisfying certain imposed constraints (e.g. that C should be less than 0.12 and that the product of C and S should not exceed 0.14). This calculation can be repeated for buildings with different periods to obtain a spectrum of

TABLE 10 PARAMETERS USED IN THE DETERMINATION OF LATERAL FORCE COEFFICIENT AND THEIR VALUES

Coefficients	Typical Range of Values
Z	1/8, 3/16, 3/8, 1
I	1.0 - 1.5
K	0.65 - 1.33 (for buildings)
C	$1 / (15 \cdot \sqrt{T}) < 0.12$
S	1.0 - 1.5 (see Fig. 44)

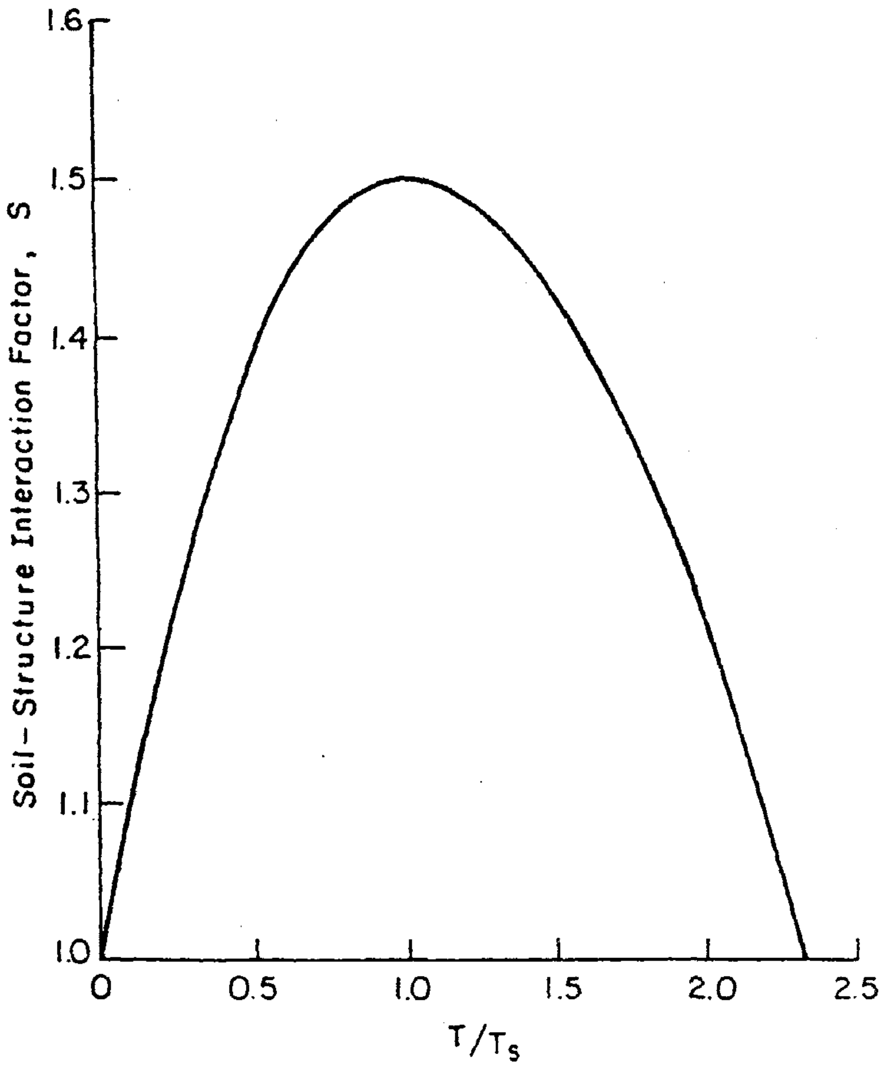


FIG. 44 VARIATION OF SOIL-STRUCTURE INTERACTION FACTOR IN U.S. UNIFORM BUILDING CODE, 1976

lateral force coefficients for a wide range of building periods. Figure 45 shows values of design lateral force coefficients determined in this way for San Francisco Bayshore sites underlain by soft clay and for stiff soil sites in the main parts of San Francisco. Values of the required design lateral force coefficient vary with the natural periods of structures (and correspondingly with the building heights and numbers of stories). It can be seen that the design requirements for Bayshore sites are generally more stringent than those for stiff soil or rock sites in the main part of San Francisco, especially for taller buildings with natural periods longer than about 1 second. The design lateral force requirements for Mexico City, enforced prior to the 1985 earthquake, are also shown on the same plot for comparison purposes. For buildings with periods between 1.5 and 2 seconds the lateral force coefficients required by the 1976 Mexico City Building code and the building code for San Francisco Bayshore sites are not significantly different. However, for longer period structures, the pre-1985 Mexico City code requirements for sites underlain by clay are higher than those for San Francisco Bayshore sites.

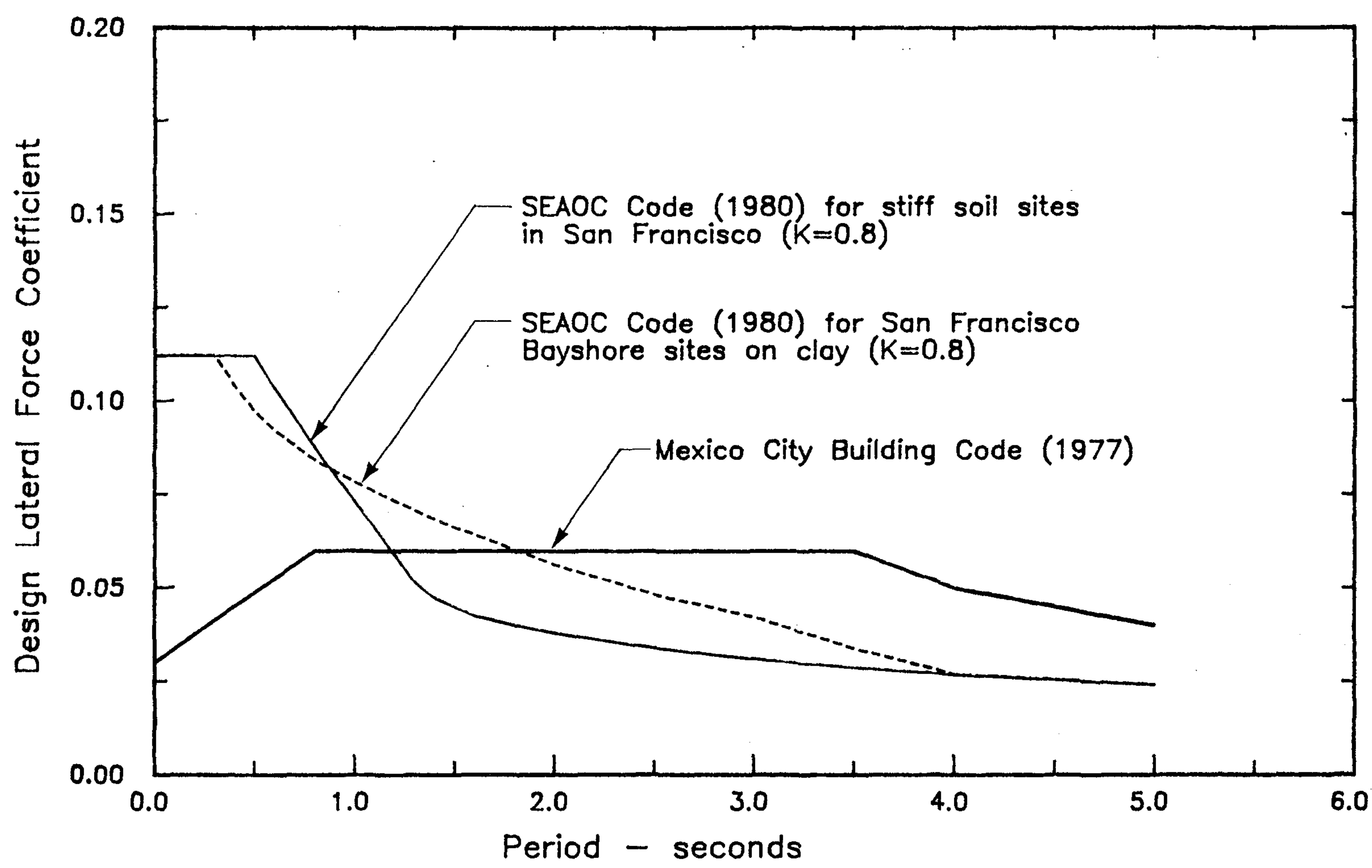


FIG. 45 COMPARISON OF DESIGN LATERAL FORCE COEFFICIENTS REQUIRED IN MEXICO CITY CODE (1976) AND SEAOC CODE FOR SAN FRANCISCO (1980)

V. EVALUATION OF DAMAGE POTENTIAL INDEX VALUES FOR STRUCTURES IN SAN FRANCISCO

Some indication of the potential vulnerability of structures in the San Francisco area to damage resulting from major earthquakes on the San Andreas fault can be obtained by comparing the ground motion spectra and design lateral force requirements for different site conditions in Mexico City in 1985 and in San Francisco for earthquakes which may affect the area. Such comparisons are shown in Fig. 46 and Fig. 47.

Figure 46(a) shows a comparison of the expected ground response spectra for sites underlain by clay deposits in the San Francisco Bay area for Magnitude $7\frac{1}{4}$ and 8+ earthquakes and a representative ground response spectrum for the heavy damage area of Mexico City in 1985; Fig. 46(b) shows the design lateral force coefficients required by the Mexico and SEAOC codes for these site conditions. Figures 47(a) and (b) show similar comparisons for clay sites in the heavy damage area of Mexico City and stiff soil sites in the San Francisco Bay area. These comparisons are extremely enlightening in view of the heavy damage suffered in Mexico City and in themselves suggest the desirability of a careful review of U.S. Code requirements.

Computation of Damage Potential Index Values for San Francisco Bay Area

While comparisons such as those shown in Figs. 46 and 47 suggest the possible need for re-evaluation of Code requirements for lateral force coefficients in the San Francisco Bay area, the combined effects of differences in earthquake shaking intensity and code requirements for lateral force coefficients and allowable stresses is best illustrated by comparisons of Damage Potential Index values for the different regions. Accordingly calculations of Damage Potential Index values, as defined

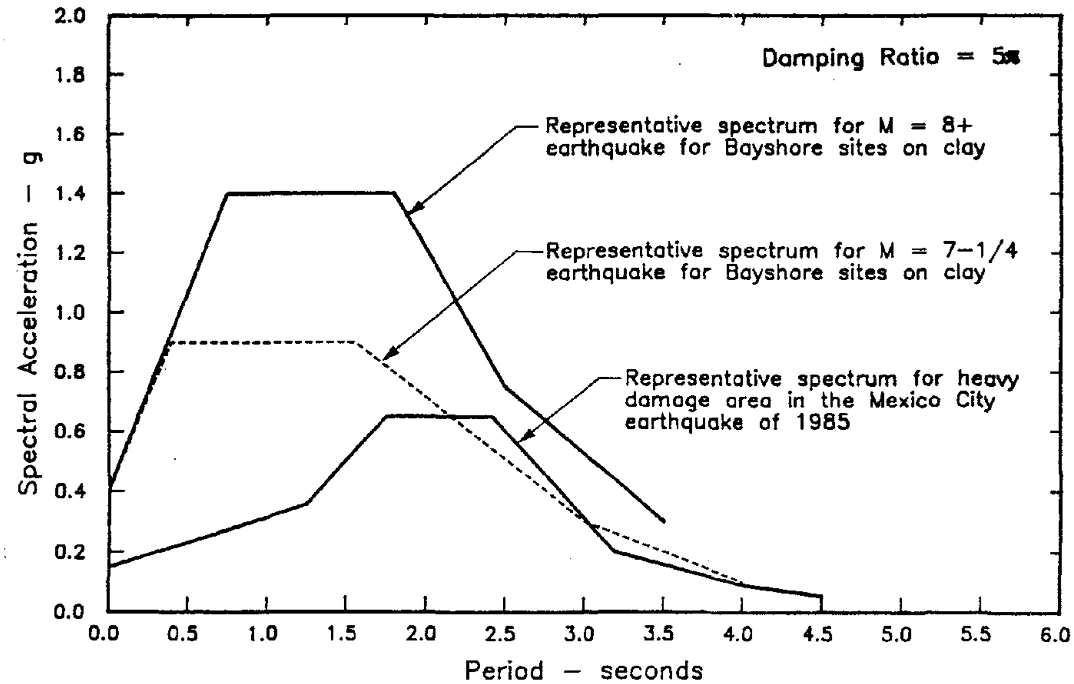


FIG. 46(a) REPRESENTATIVE SPECTRA FOR BAYSHORE SITES IN SAN FRANCISCO AND HEAVY DAMAGE AREA OF MEXICO CITY (1985)

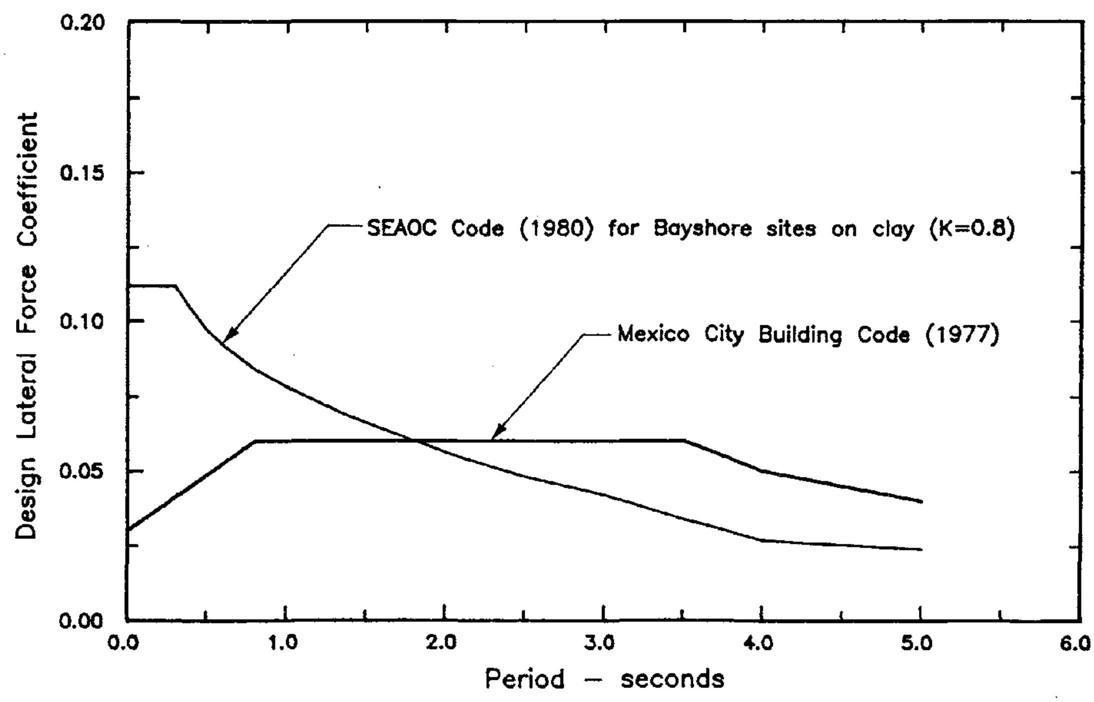


FIG. 46(b) DESIGN LATERAL FORCE COEFFICIENTS FOR BAYSHORE SITES IN SAN FRANCISCO AND HEAVY DAMAGE AREA OF MEXICO CITY (1985)

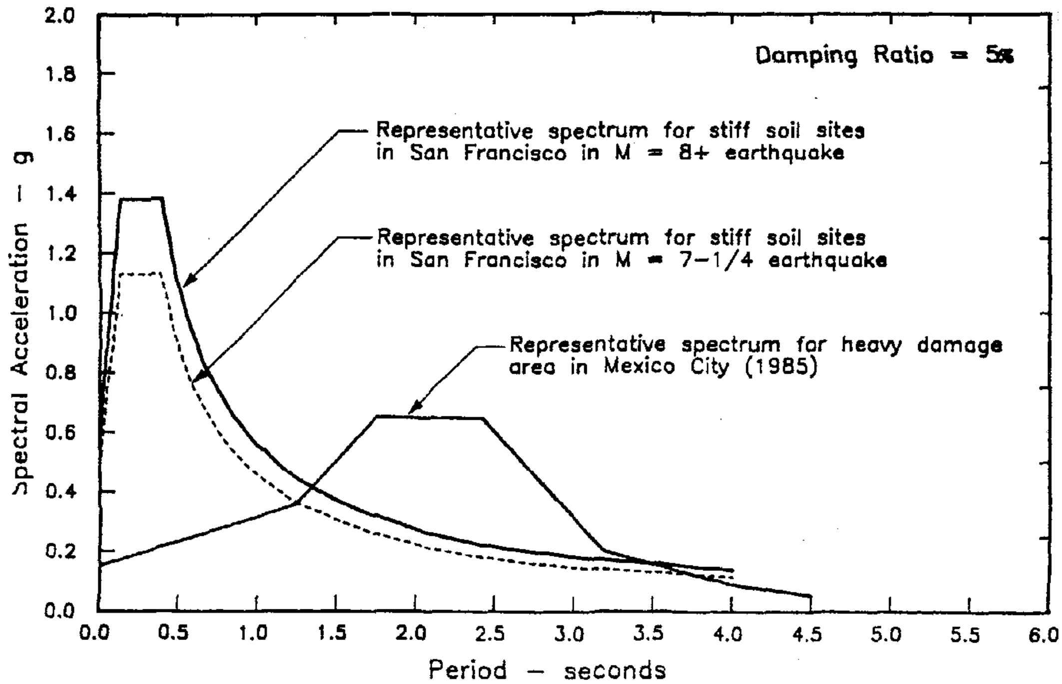


FIG. 47(a) REPRESENTATIVE SPECTRA FOR STIFF SOIL SITES IN SAN FRANCISCO AND HEAVY DAMAGE AREA OF MEXICO CITY (1985)

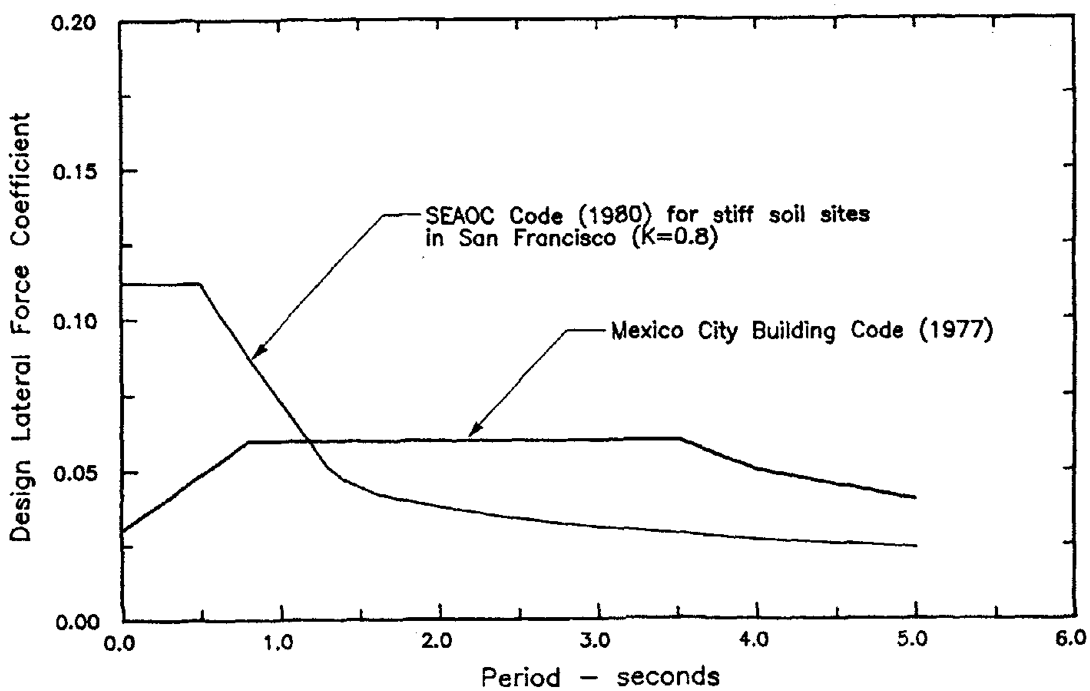


FIG. 47(b) DESIGN LATERAL FORCE COEFFICIENTS FOR STIFF SOIL SITES IN SAN FRANCISCO AND HEAVY DAMAGE AREA OF MEXICO CITY previously, for sites underlain by substantial thicknesses of San Francisco Bay mud and for earthquakes with Magnitudes of $7\frac{1}{4}$ and 8+ earthquakes are shown in Tables 11 and 12 respectively. The duration weighting factors (defined in Table 3) used in the calculation of these Damage Potential Index values were 1.0 and 1.35 for $7\frac{1}{4}$ and 8+ earthquakes respectively to reflect the potential significance of the different durations of shaking on damage intensities for the two earthquakes, and the term R_f was assigned a values of 1.3 (Bertero, 1988), based on a judgmental assessment of the different Code requirements. Similarly Damage Potential Index values for stiff soil sites in San Francisco for the same two earthquake magnitudes are presented in Tables 13 and 14 respectively.

The computed values of Damage Potential Index for San Francisco Bayshore sites and stiff site conditions for ground motions likely to be produced by a nearby Magnitude $7\frac{1}{4}$ earthquake are summarized and plotted in Fig. 48, together with the values determined for the heavy damage area of Mexico City in the earthquake of 1985. The cross-hatched band illustrate the variations in the calculated damage potentials due to the use of different structural systems corresponding to K = 0.8 and K = 1.0. It is apparent that for the anticipated shaking produced by a Magnitude $7\frac{1}{4}$ earthquake, probably lasting about 40 seconds, the calculated Damage Potential Index values for Bayshore sites underlain by soft clay and for stiff soil sites in San Francisco are both significantly lower than those developed in the heavy damage zone of Mexico City in 1985. These results are extremely encouraging and are clearly indicative of a much lower intensity of damage for buildings in San Francisco than that which occurred in Mexico City in 1985. It may also be noted, however, that for structures with more than about 6 stories, the calculated Damage Potential Index for

TABLE 11 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO BAYSHORE SITES FOR MAGNITUDE 7¼ EARTHQUAKE BASED ON 1980 SEAOC RECOMMENDATION

SAN FRANCISCO BAYSHORE SITES **Magnitude 7¼ earthquake occurring at a distance of 6 miles**				
Building Period	Spectral Accelera-tion	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.40	0.00	0.112	0
0.5	0.90	2.31	0.097	18
1.0	0.90	4.61	0.078	45
1.5	0.90	6.92	0.066	81
2.0	0.71	7.27	0.056	100
2.5	0.51	6.47	0.048	103
3.0	0.30	4.61	0.042	84
3.5	0.19	3.41	0.034	77

Notes:

1. The Duration Weighting Factor (DWF) used is 1.0 for the expected duration of strong ground motion for a M = 7¼ earthquake.

2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

TABLE 12 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO BAYSHORE SITES FOR MAGNITUDE 8+ EARTHQUAKE BASED ON 1980 SEAOC RECOMMENDATION

SAN FRANCISCO BAYSHORE SITES

Magnitude 8+ earthquake occurring at a distance of 6 miles

Building Period	Spectral Accelera-tion	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.40	0.00	0.112	0
0.5	1.08	2.77	0.097	30
1.0	1.40	7.17	0.078	95
1.5	1.40	10.76	0.066	169
2.0	1.20	12.30	0.056	228
2.5	0.75	9.61	0.048	208
3.0	0.52	7.99	0.042	198
3.5	0.30	5.38	0.034	164

Notes:

1. The Duration Weighting Factor (DWF) used is 1.35 for the expected duration of strong ground motion for a M = 8+ earthquake.

2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

TABLE 13 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO STIFF SOIL SITES FOR MAGNITUDE 7¼ EARTHQUAKE BASED ON 1980 SEAOC RECOMMENDATION

STIFF SOIL SITES IN SAN FRANCISCO Magnitude 7¼ earthquake occurring at a distance of 6 miles				
Building Period	Spectral Accelera-tion	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.45	0.00	0.112	0
0.5	0.90	2.31	0.112	16
1.0	0.46	2.38	0.073	25
1.5	0.32	2.41	0.044	42
2.0	0.23	2.31	0.038	47
2.5	0.18	2.27	0.034	51
3.0	0.15	2.35	0.031	58
3.5	0.13	2.36	0.029	63

Notes:

1. The Duration Weighting Factor (DWF) used is 1.0 for the expected duration of strong ground motion for a M = 7¼ earthquake.

2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

TABLE 14 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO STIFF SOIL SITES FOR MAGNITUDE 8+ EARTHQUAKE BASED ON 1980 SEAOC RECOMMENDATION

STIFF SOIL SITES IN SAN FRANCISCO

Magnitude 8+ earthquake occurring at a distance of 6 miles

Building Period	Spectral Acceleration	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.55	0.00	0.112	0
0.5	1.10	2.82	0.112	26
1.0	0.57	2.90	0.073	41
1.5	0.38	2.92	0.044	69
2.0	0.28	2.82	0.038	77
2.5	0.22	2.82	0.034	86
3.0	0.19	2.88	0.031	96
3.5	0.16	2.88	0.029	103

Notes:

1. The Duration Weighting Factor (DWF) used is 1.35 for the expected duration of strong ground motion for a M = 8+ earthquake.
2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

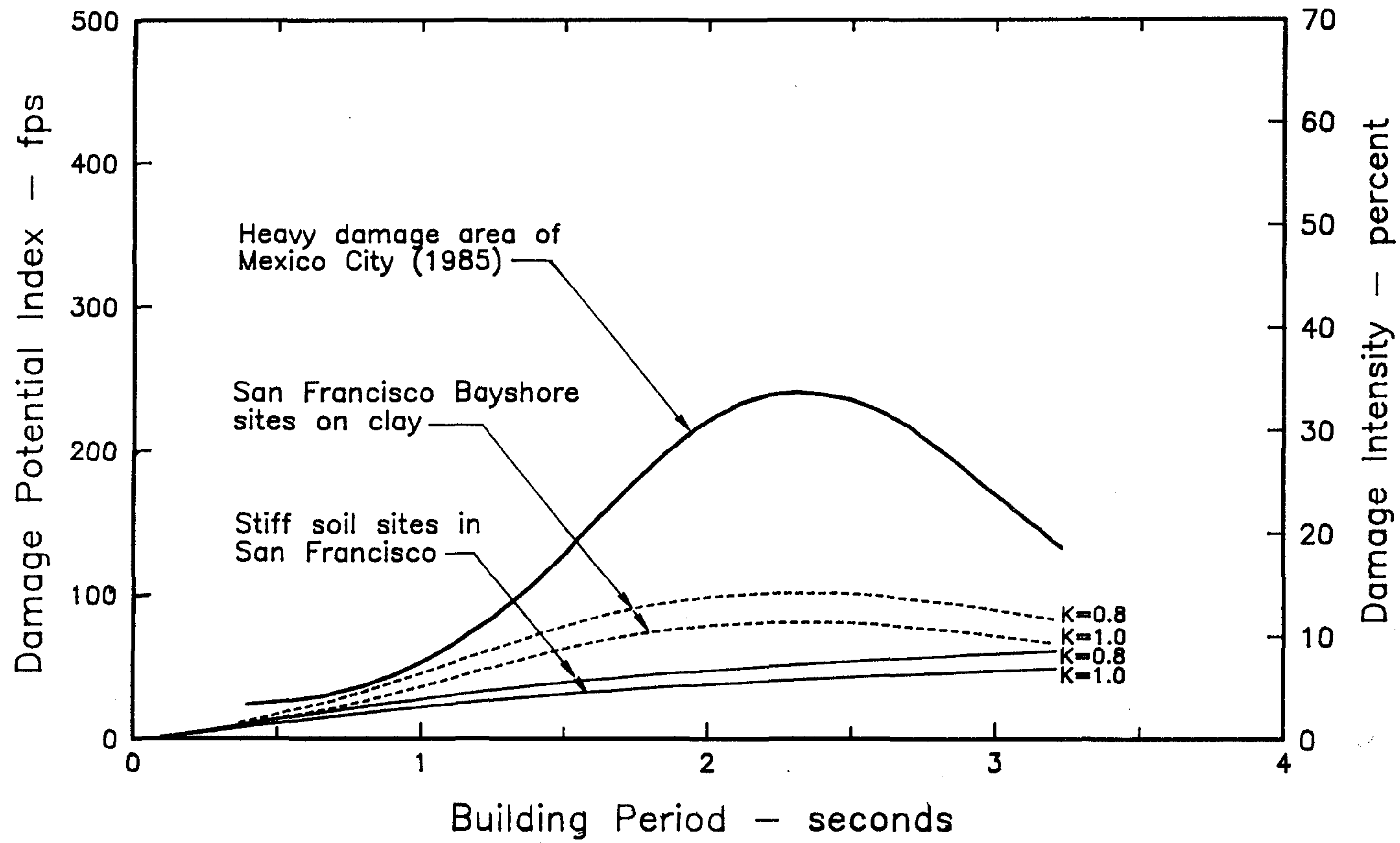


FIG. 48 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY (1985) AND IN SAN FRANCISCO FOR M = $7\frac{1}{4}$ EARTHQUAKE AND 1980 SEAOC CODE

Bayshore sites underlain by soft Bay mud typically 35 to 40 ft in thickness, is significantly higher than that for similar structures supported on the stiff soil sites in the main part of San Francisco, indicating the greater vulnerability of multi-story buildings constructed on Bayshore clay deposits in comparison with those on the stiffer alluvium underlying the greater part of the city.

Figure 49 shows the computed Damage Potential Index values corresponding to a Magnitude 8+ earthquake, with a shaking duration of about 70 seconds, in the San Francisco Bay area. Also shown for comparison are the Damage Potential Index values for the heavy damage area of Mexico City in 1985. The significant increase in calculated values of Damage Potential Index for San Francisco Bayshore sites on clay for this Magnitude 8+ earthquake over those for a Magnitude $7\frac{1}{4}$ event results from the combined effects of the increase in acceleration level of the input motion, the increase in duration of significant shaking and the increase in low-frequency content of the rock outcrop motions. It is readily apparent that the computed values of Damage Potential Index for Bayshore sites underlain by clay in such an earthquake are comparable to those developed in the heavy damage area of Mexico City in 1985.

For the three Bayshore sites that have been analyzed, buildings which have natural periods in the range of 1.5 to 2.5 seconds exhibit the highest damage potentials. The peak Damage Potential Index within this range for Bayshore sites underlain by clay is about 250 fps for buildings designed with structural systems corresponding to K = 0.8 or K = 1.0. These values are somewhat higher than those corresponding to the heavy damage area of Mexico City in 1985. In the case of Mexico City, Damage Potential Index values reached levels of about 250 fps and nearly 30% of the mid-rise 9- to

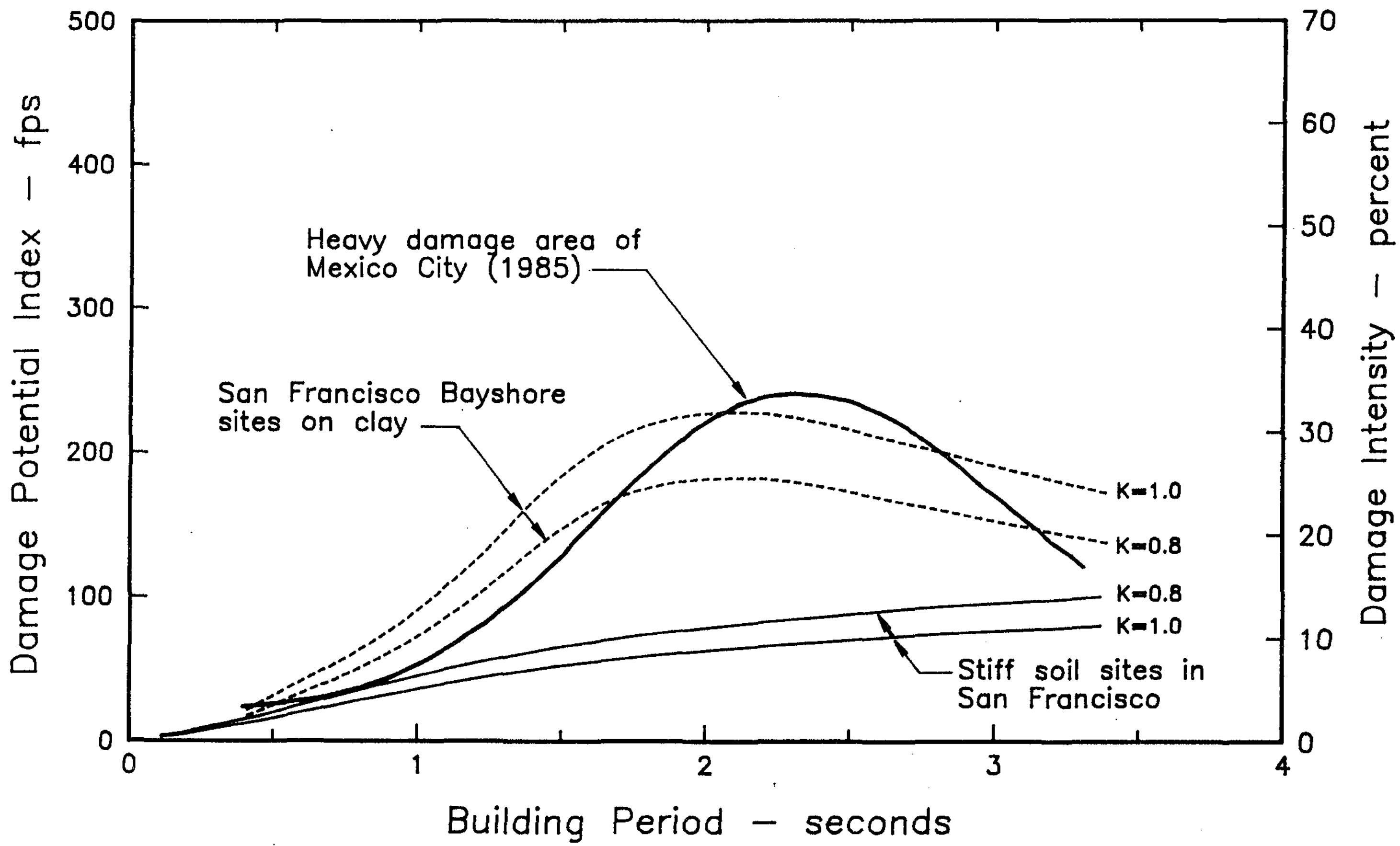


FIG. 49 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY (1985) AND IN SAN FRANCISCO FOR M = 8+ EARTHQUAKE AND 1980 SEAOC CODE

12-story buildings (typically with periods between about 1.5 to 2.5 seconds) located inside the heavy-damage zone either collapsed or suffered major damage. Fortunately only a relatively small section of San Francisco is underlain by soil conditions of this type. For buildings constructed in San Francisco on stiff soil sites, the damage potentials are significantly lower than for Bayshore sites underlain by soft clay and they are also significantly lower than the Damage Potential Index values calculated for the heavy damage area of Mexico City, indicating a significantly lower degree of vulnerability than that exhibited by buildings in Mexico City.

Lateral Force Requirements Recommended by SEAOC (1988)

The new "Recommended Lateral Force Requirements and Tentative Commentary" proposed by the Seismology Committee of SEAOC in 1988, recommends that the soil factor S in the present code be replaced by a series of site-specific spectra and corresponding lateral force coefficients. The recommended spectral shapes for three site conditions, designated S1, S2, and S3 are shown in Fig. 50.

At the same time, it was suggested (Donovan et al., 1978) that the site specific spectra shown in Fig. 50 could be effectively converted into design lateral force coefficients by means of the equation:

$$V = 1.25 \frac{Z\ I\ C}{R_w\ T^{2/3}} W$$

where Z = a seismic zone factor ranging from 0 (non-seismic) to 0.4, (see Seismic Zoning map for California shown in Fig. 51,

I = an importance coefficient having values 1.0 for standard occupancy and 1.25 for hazardous or essential facilities,

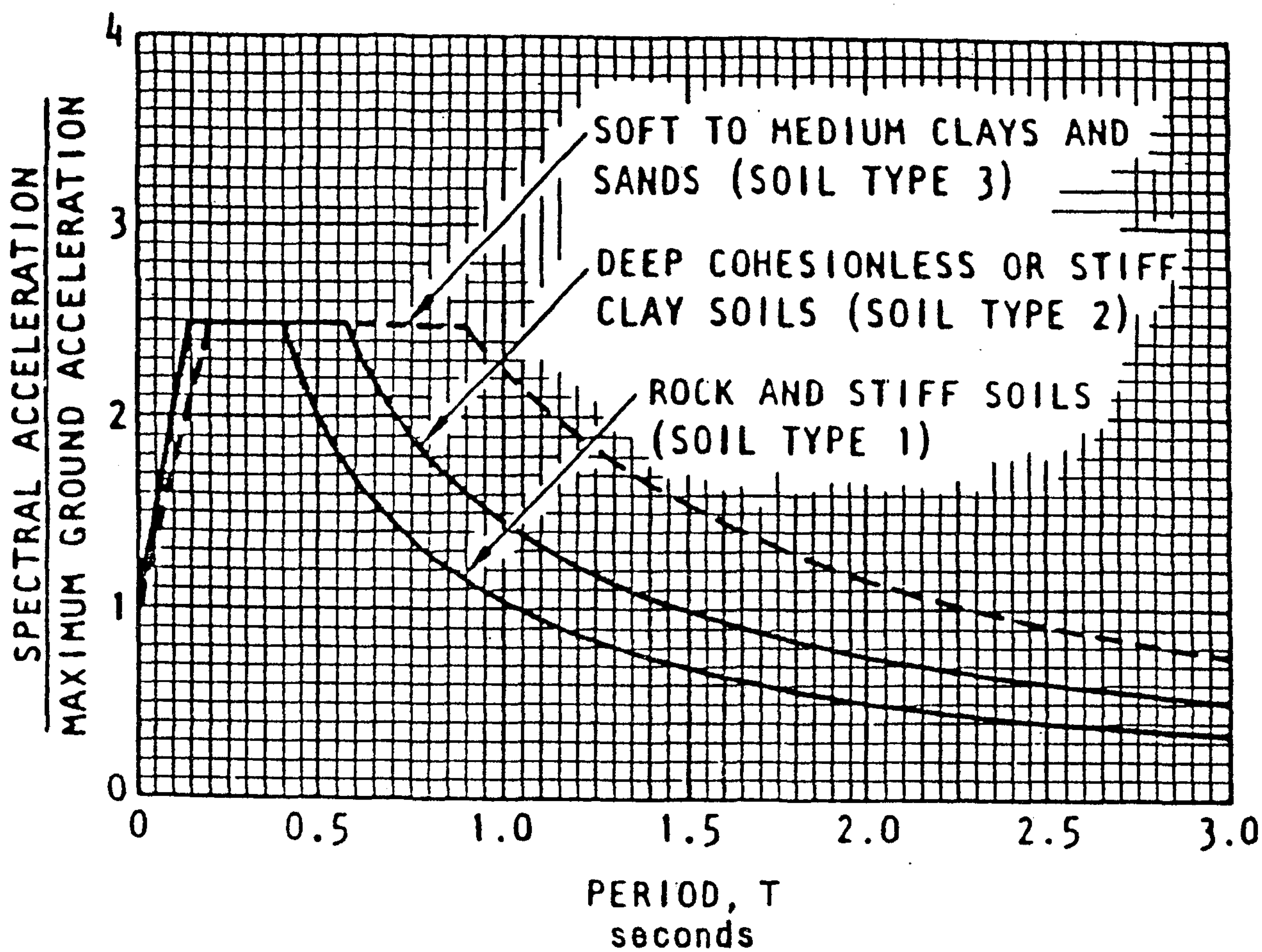
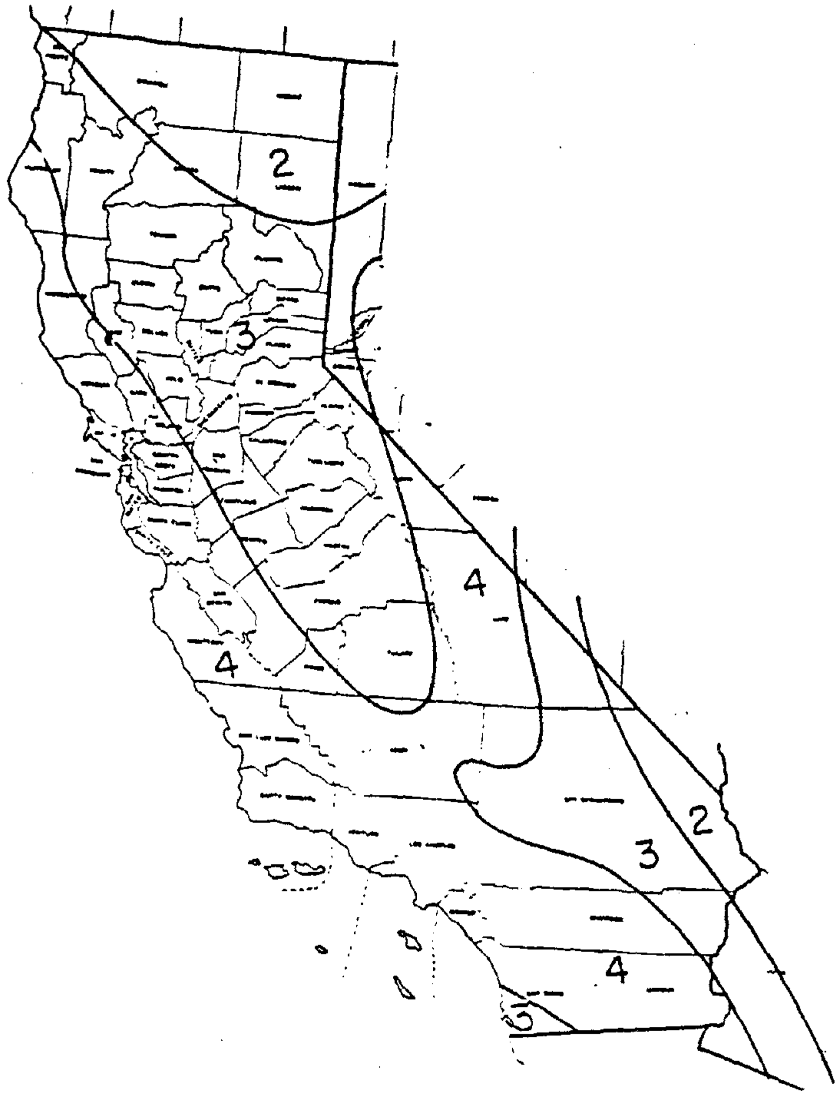


FIG. 50 NORMALIZED SPECTRAL CURVES RECOMMENDED FOR USE IN BUILDING CODE (after Applied Technology Council, 1978)



Zone*	1**	2	3	4
Z	0.1	0.2	0.3	0.4

* The Zone shall be determined from the Seismic Zone Map in Figure 1-A.

** Not used in California.

FIG. 51 SEISMIC ZONING OF CALIFORNIA AND THE SEISMIC ZONE FACTOR

R_w = A numerical coefficient, ranging from 4 to 12, depending on the structural system as shown in Table 15,

T = Fundamental period of the structure in seconds,

W = The total dead load and the applicable portions of other loads,

and S = A site coefficient, determined by the soil characteristics at the site, as shown in Table 16, and ranging in value from 1.0 for rock or stiff sites (S1) to 2.0 for sites containing thick layers of soft clay (S4). The last soil category, S4, was added in recognition of the effects observed in Mexico City in the earthquake of 1985 where the soft Mexico City clay greatly amplified the rock motions in some areas and caused severe damage to the city.

Thus with these new provisions there will be four soil conditions recognized for each of the four different seismic zones, and the design spectra for the highest intensity Zone 4 will have the general forms shown in Fig. 52.

For typical structures of normal importance constructed in San Francisco, under the new SEAOC code provisions, appropriate parameters for use in pseudo-static analyses would be as follows: Z = 1.0, I = 1.0, and R_w = 10 (Bertero, 1988). Values of S would vary with the soil conditions at the proposed building site.

Figure 53 shows a comparison of the design lateral force coefficient requirements for the present code (1974 to 1988) and the new SEAOC recommendations for stiff soil conditions. Although the expression for determining values of the lateral force coefficient has been modified significantly in the new Code, values of the coefficient itself are fairly consistent with those required by the present code, especially in the long

TABLE 15 SEAOC RECOMMENDATION (1988) FOR STRUCTURAL SYSTEM PARAMETER R_W

Basic Structural System(1)	Lateral Load Resisting System – Description	R_w(5)	H(2)
A. Bearing Wall System	1. Light Framed Walls With Shear Panels		
	a. Plywood Walls for Structures 3-stories or Less	8	65
	b. All Other Light Framed Walls	6	65
	2. Shear Walls		
	a. Concrete	6	160
	b. Masonry	6	120
	3. Light Steel Framed Bearing Walls With Tension-Only Bracing		
	4. Braced Frames Where Bracing Carries Gravity Loads		
	a. Steel	6	160
	b. Concrete(3)	4	–
	c. Heavy Timber	4	65
B. Building Frame System	1. Steel Eccentric Braced Frame (EBF)	10	240
	2. Light Framed Walls With Shear Panels		
	a. Plywood Walls for Structures 3-stories or Less	9	65
	b. All Other Light Framed Walls	7	65
	3. Shear Walls		
	a. Concrete	8	240
	b. Masonry	8	160
	4. Concentric Braced Frames		
	a. Steel	8	160
	b. Concrete(3)	8	–
	c. Heavy Timber	8	65
C. Moment Resisting Frame System	1. Special Moment Resisting Space Frames (SMRSF)		
	a. Steel	12	N.L.(4)
	b. Concrete	12	N.L.
	2. Concrete Intermediate Moment Resisting Space Frames (IMRSF)(6)	7	–
	3. Ordinary Moment Resisting Space Frames		
	a. Steel	12	160
	b. Concrete(3)	5	–
D. Dual System	1. Shear Walls		
	a. Concrete With SMRSF	12	N.L.
	b. Concrete With Concrete IMRSF	9	160
	c. Masonry With SMRSF	8	160
	d. Masonry With Concrete IMRSF(3)	7	–
	2. Steel EBF With Steel SMRSF	12	N.L.
	3. Concentric Braced Frames		
	a. Steel With Steel SMRSF	10	N.L.
	b. Concrete With Concrete SMRSF(3)	9	–
	c. Concrete With Concrete IMRSF(3)	6	–
E. Undefined Systems	See Section 1D9b	–	–

TABLE 16 SITE COEFFICIENTS RECOMMENDED BY SEAOC (1988)

Type	Description	S Factor
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 200 feet or more.	1.2
S_3	A soil profile 40 feet or more in depth and containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.	1.5
S_4	A soil profile containing more than 40 feet of soft clay.	2.0

(1) The site factor shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 will be used unless the Building Official determines that soil profile S_4 may be present at the site, in which case soil profile S_4 will be used.

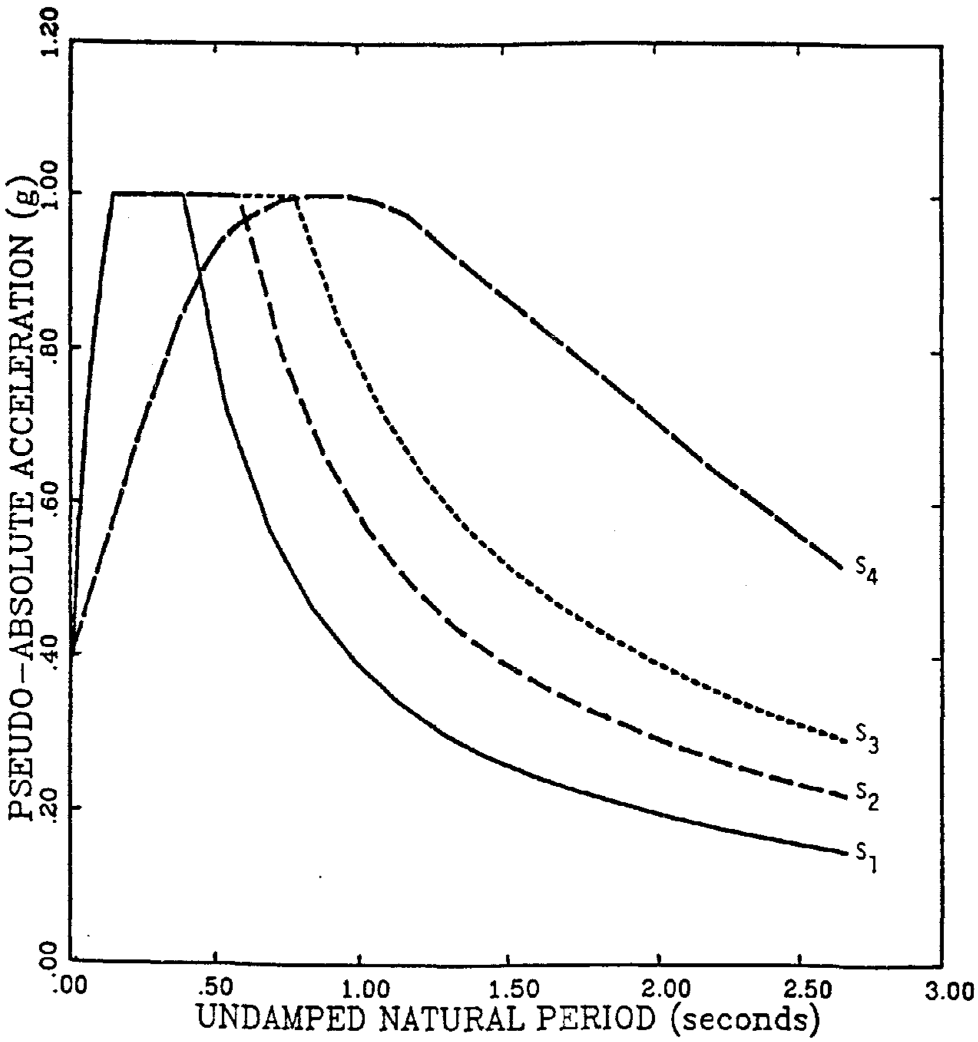


FIG. 52 DESIGN SPECTRA FOR STRUCTURES LOCATED IN ZONE 4 (after Seed, 1986)

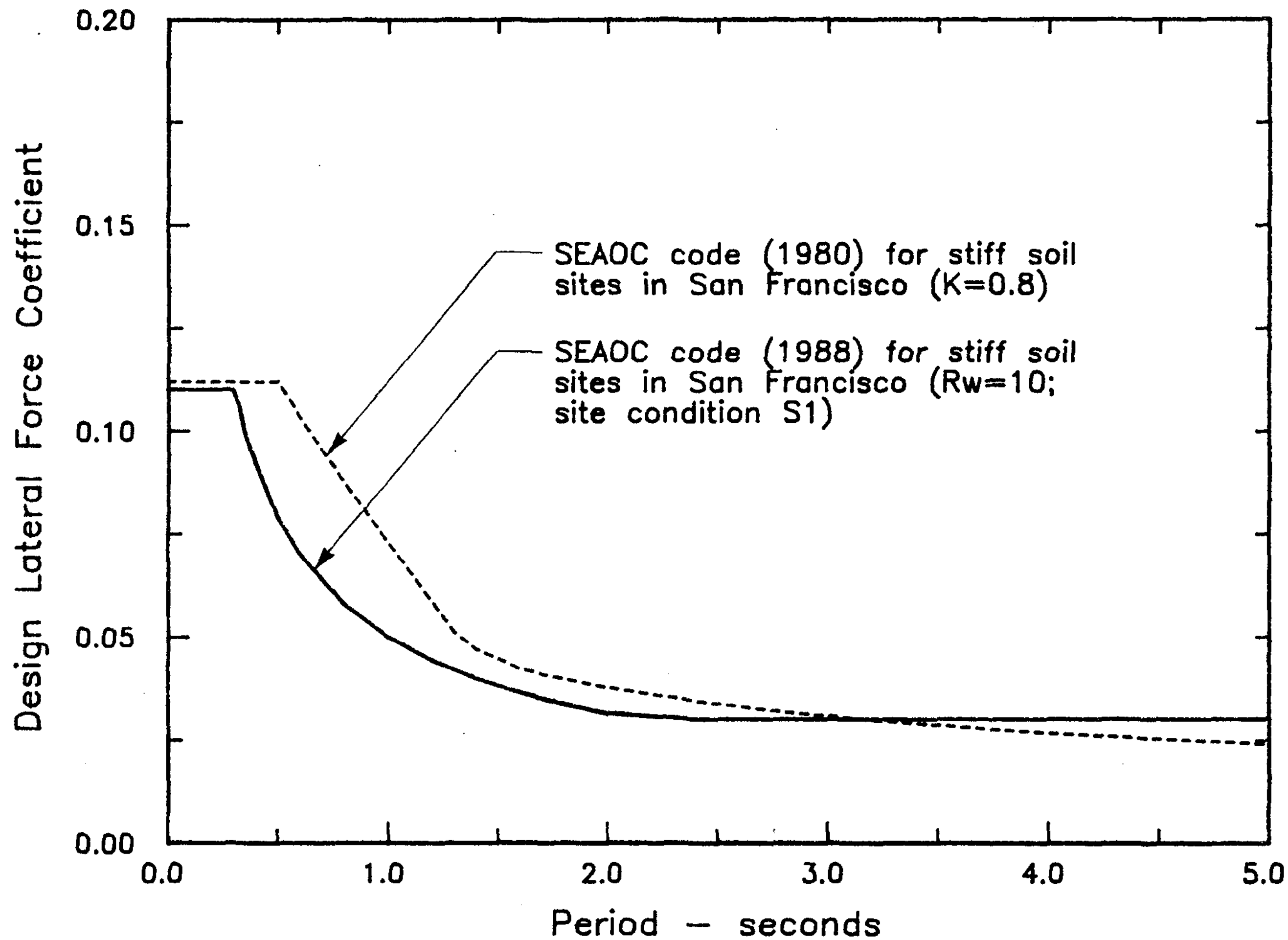


FIG. 53 COMPARISON OF DESIGN LATERAL FORCE COEFFICIENTS FOR STIFF SOIL SITES - SEAOC CODE 1980 AND 1988 RECOMMENDATIONS

period range. It does appear, however, that the new design criteria impose less stringent requirements for low- to mid-rise buildings with natural periods between about 0.5 to 1.2 seconds on stiff soil sites. The requirement for extremely stiff buildings on stiff soil sites, however, is almost unchanged.

The new SEAOC recommendations for lateral force coefficients for soft clay sites (type S4), on the other hand, are now somewhat more conservative than those required by the present code. This is illustrated by the comparative values presented on Fig. 54, which show an increase in design requirements of roughly 10% to 20%, over nearly the entire period range.

Calculated Damage Potential Index Values for San Francisco Stiff Soil and San Francisco Bayshore Sites Based on 1988 SEAOC Recommendations

Damage Potential Indices for San Francisco and the San Francisco Bay area have been recalculated based on the expected ground motions as derived previously (Figs. 41 and 43) together with the new Code provisions (Fig. 53 and 54).

For a Magnitude $7\frac{1}{4}$ earthquake, Table 17 and Table 18 show the calculation of Damage Potential Index values for stiff soil sites in San Francisco and San Francisco Bayshore sites underlain by clay respectively; the results of these computations are plotted in Fig. 55. Compared with the damage potentials for San Francisco under the present code requirements (Fig. 48), the damage potentials are slightly lower for Bayshore sites and about the same for stiff soil sites. The damage potentials for both site conditions are significantly lower than those for Mexico City in the 1985 earthquake.

Table 19 and Table 20 show the calculation of Damage Potential Indices for San Francisco stiff soil sites and Bayshore sites underlain by clay in

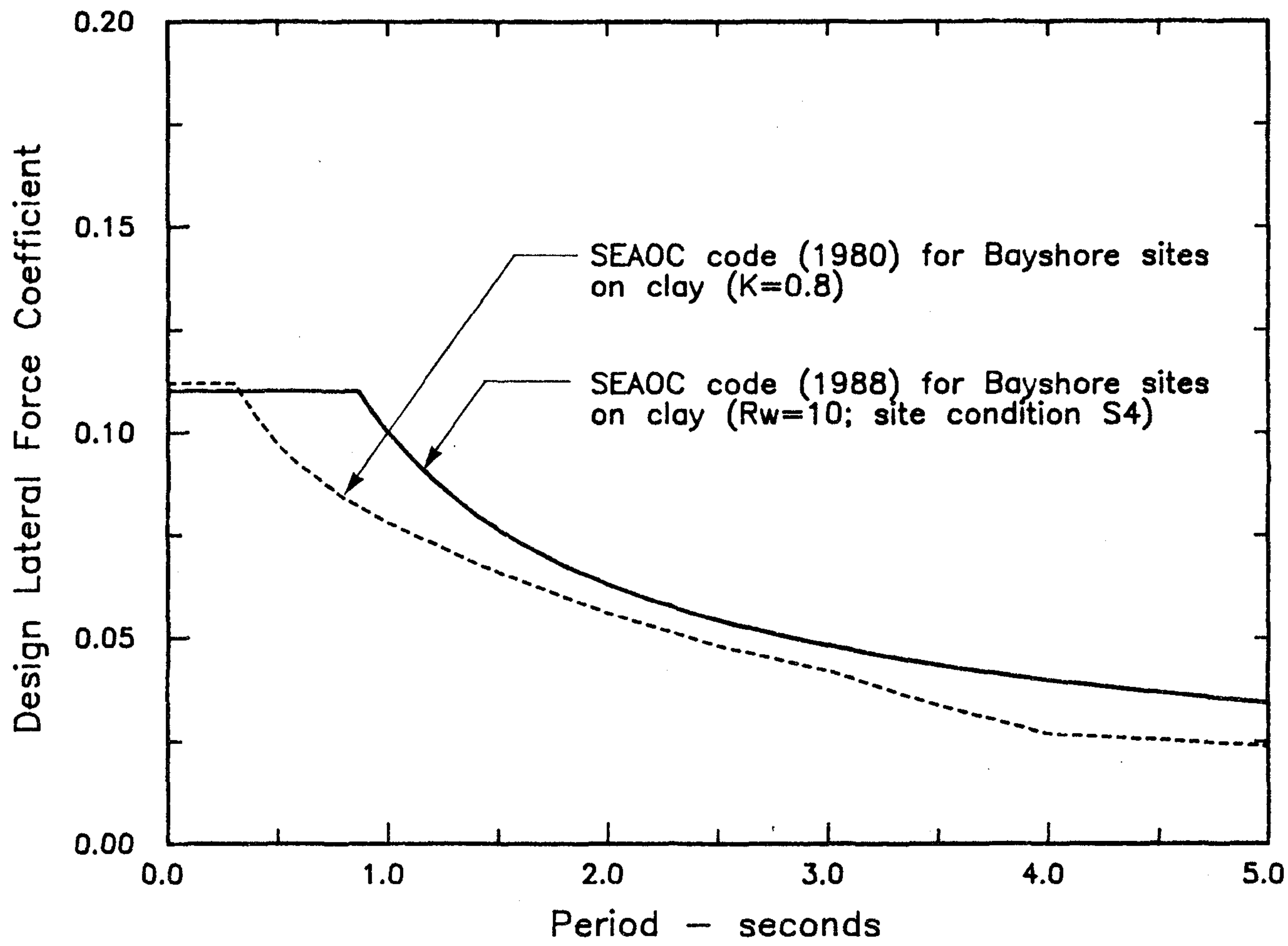


FIG. 54 COMPARISON OF DESIGN LATERAL FORCE COEFFICIENTS FOR BAYSHORE SITES ON CLAY - SEAOC CODE 1980 AND 1988 RECOMMENDATIONS

TABLE 17 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO STIFF SOIL SITES FOR MAGNITUDE 7¼ EARTHQUAKE BASED ON 1988 SEAOC RECOMMENDATION

STIFF SOIL SITES IN SAN FRANCISCO

Magnitude 7¼ earthquake occurring at a distance of 6 miles

Building Period	Spectral Acceleration	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.45	0.00	0.110	0
0.5	0.90	2.31	0.079	22
1.0	0.46	2.38	0.050	37
1.5	0.32	2.41	0.038	49
2.0	0.23	2.31	0.032	56
2.5	0.18	2.27	0.030	58
3.0	0.15	2.35	0.030	60
3.5	0.13	2.36	0.030	60

Notes:

1. The Duration Weighting Factor (DWF) used is 1.0 for the expected duration of strong ground motion for a M = 7¼ earthquake.

2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

TABLE 18 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO BAYSHORE SITES FOR MAGNITUDE 7¼ EARTHQUAKE BASED ON 1988 SEAOC RECOMMENDATION

SAN FRANCISCO BAYSHORE SITES Magnitude 7¼ earthquake occurring at a distance of 6 miles				
Building Period	Spectral Acceleration	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.40	0.00	0.110	0
0.5	0.90	2.31	0.110	16
1.0	0.90	4.61	0.100	35
1.5	0.90	6.92	0.077	69
2.0	0.71	7.27	0.063	89
2.5	0.51	6.47	0.054	92
3.0	0.30	4.61	0.048	74
3.5	0.19	3.41	0.043	61

Notes:

1. The Duration Weighting Factor (DWF) used is 1.0 for the expected duration of strong ground motion for a M = 7¼ earthquake.
2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

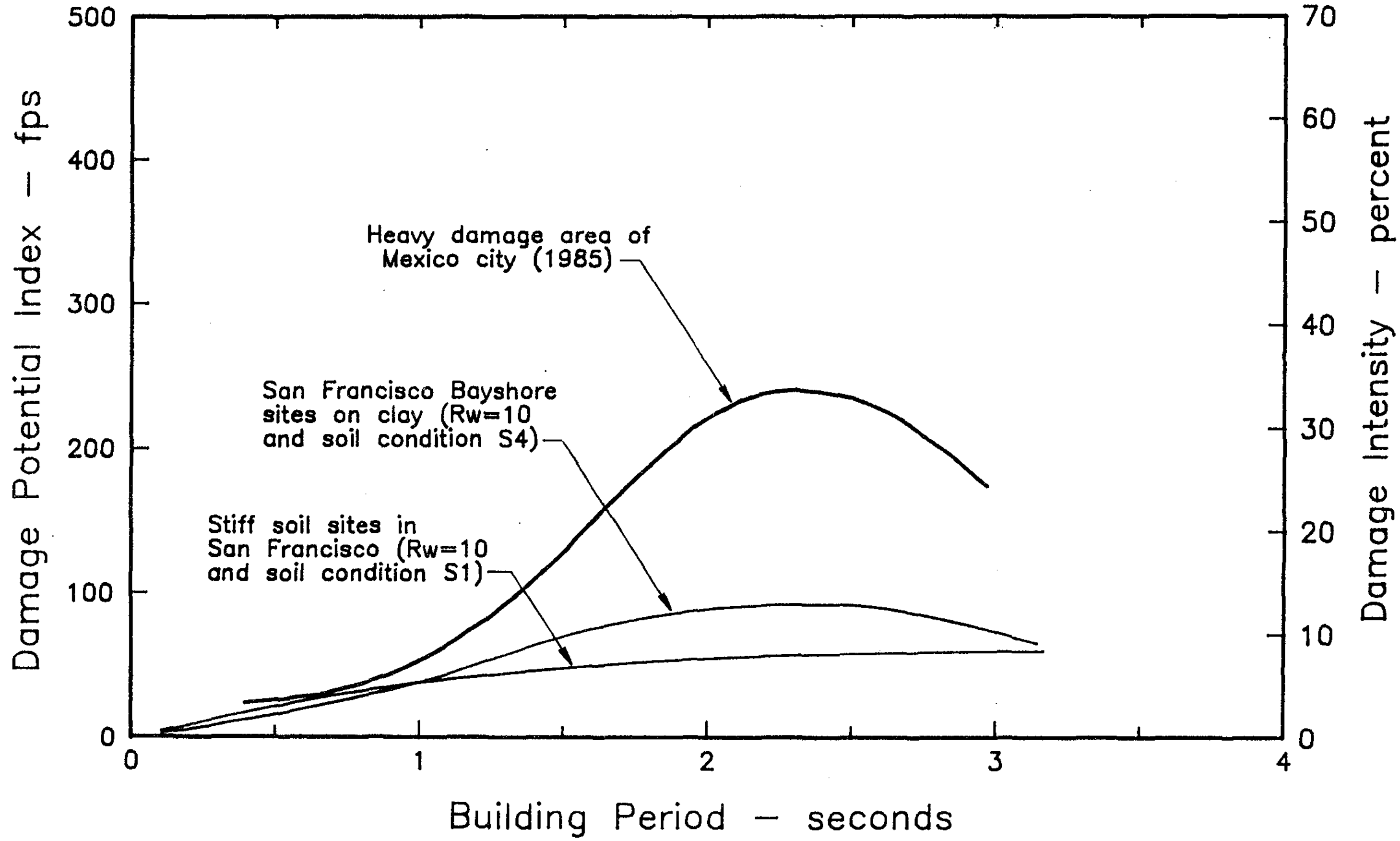


FIG. 55 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY FOR PRE-1985 CODE AND FOR SOFT SOIL SITES AND STIFF SOIL SITES IN SAN FRANCISCO FOR M = $7\frac{1}{4}$ EARTHQUAKE AND 1988 CODE RECOMMENDATIONS

TABLE 19 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO STIFF SOIL SITES FOR MAGNITUDE 8+ EARTHQUAKE BASED ON 1988 SEAOC RECOMMENDATION

STIFF SOIL SITES IN SAN FRANCISCO

Magnitude 8+ earthquake occurring at a distance of 6 miles

Building Period	Spectral Acceleration	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.55	0.00	0.110	0
0.5	1.10	2.82	0.079	37
1.0	0.57	2.90	0.050	60
1.5	0.38	2.92	0.038	80
2.0	0.28	2.82	0.032	92
2.5	0.22	2.82	0.030	98
3.0	0.19	2.88	0.030	100
3.5	0.16	2.88	0.030	100

Notes:

1. The Duration Weighting Factor (DWF) used is 1.0 for the expected duration of strong ground motion for a M = 7½ earthquake.

2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

TABLE 20 CALCULATED DAMAGE POTENTIAL INDICES FOR SAN FRANCISCO BAYSHORE SITES FOR MAGNITUDE 8+ EARTHQUAKE BASED ON 1988 SEAOC RECOMMENDATION

SAN FRANCISCO BAYSHORE SITES

Magnitude 8+ earthquake occurring at a distance of 6 miles

Building Period	Spectral Acceleration	Spectral Velocity	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	Sa (g)	Sv (fps)	k	$(Sv/k \cdot R_f) \cdot DWF$ (fps)
0.0	0.40	0.00	0.110	0
0.5	1.08	2.77	0.110	26
1.0	1.40	7.17	0.010	74
1.5	1.40	10.76	0.077	145
2.0	1.20	12.30	0.063	203
2.5	0.75	9.61	0.054	185
3.0	0.52	7.99	0.048	173
3.5	0.30	5.38	0.043	130

Notes:

1. The Duration Weighting Factor (DWF) used is 1.0 for the expected duration of strong ground motion for a M = 7¼ earthquake.

2. The building resistance factor (R_f) is assigned a value of 1.3 for typical reinforced concrete structures in California.

the event of a Magnitude 8+ earthquake. Fig. 56 shows a plot of these results together with data for Mexico City in 1985. It is clear that a Magnitude 8+ earthquake (say about 60 to 90 seconds in duration) on the San Andreas fault at a close distance from San Francisco, will generate computed values of Damage Potential Index for Bayshore sites underlain by thick layers of soft Bay mud comparable to those computed for the heavy damage area of Mexico City in the 1985 earthquake, even with the new regulations. Compared with Fig. 49, the new provisions have lowered values of the Damage Potential Index values for buildings with periods in the range of 1.5 to 2.5 seconds by about 12%. However the increase in code requirements is apparently not sufficient to reduce significantly the computed damage potentials for mid-rise (10 to 20 stories) buildings. This is especially significant in view of the fact that the emergency building code for Mexico City, enforced shortly after the 1985 earthquake, increased the code requirements for the elastic design spectrum for buildings on clay sites by about 67% over the entire period range (see Fig. 57) and imposed a more stringent requirement on the Ductility Factor (Q) with which to bring the elastic spectrum down to values of design lateral coefficient. Thus for a typical semi-ductile building constructed on clay, the new Mexico City code requires lateral force coefficients almost 2 times (actual 20/9 times) higher than the pre-1985 code.

Comparison of computed values of Damage Potential Index for

1. The heavy damage area of Mexico City as conditions existed at the time of the 1985 earthquake,
2. The heavy damage area of Mexico City under the conditions resulting from the code revisions in Mexico City following the 1985 earthquake,

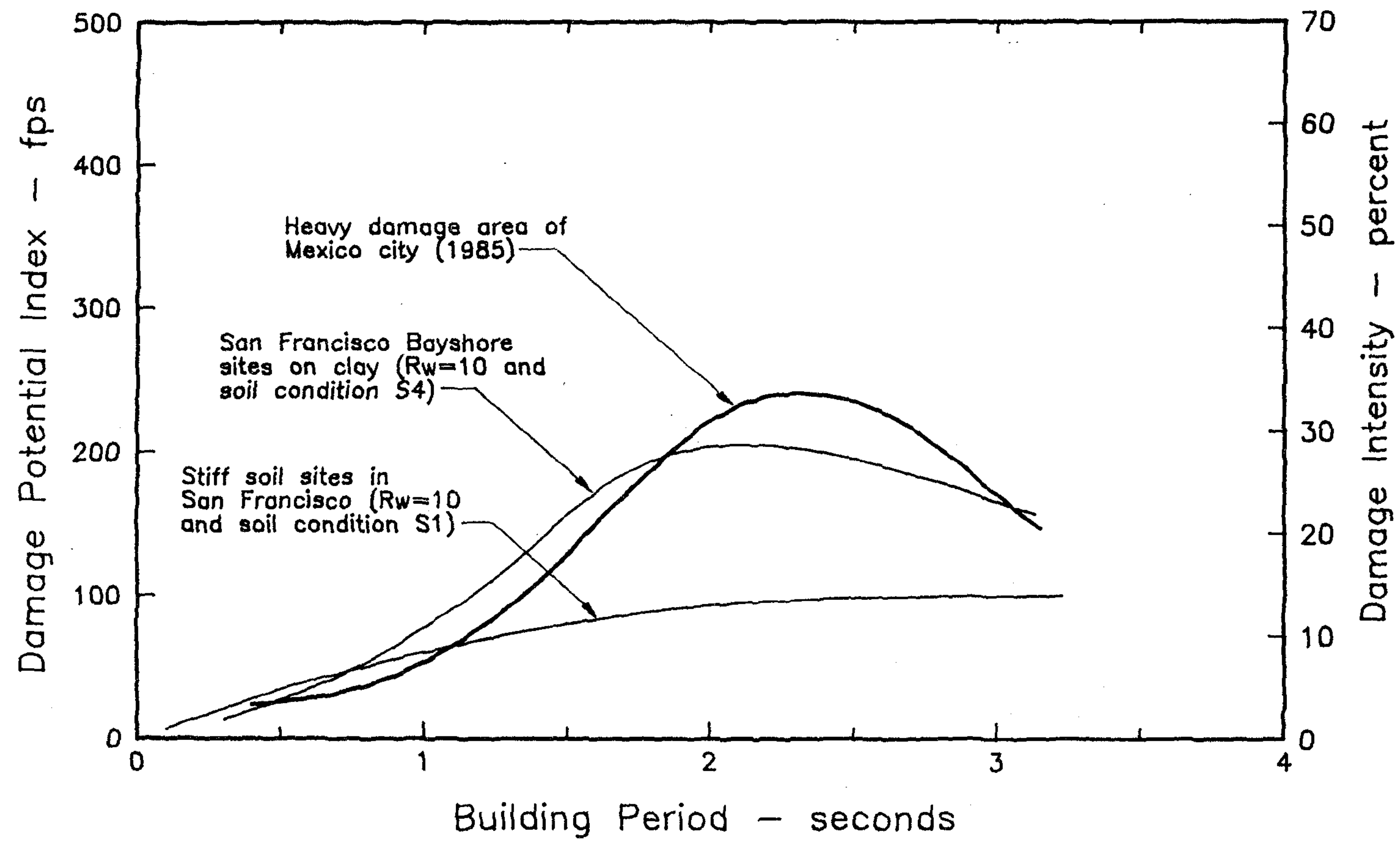


FIG. 56 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY FOR PRE-1985 CODE AND FOR SOFT SOIL SITES AND STIFF SOIL SITES IN SAN FRANCISCO FOR M = 8+ EARTHQUAKE AND 1988 CODE RECOMMENDATIONS

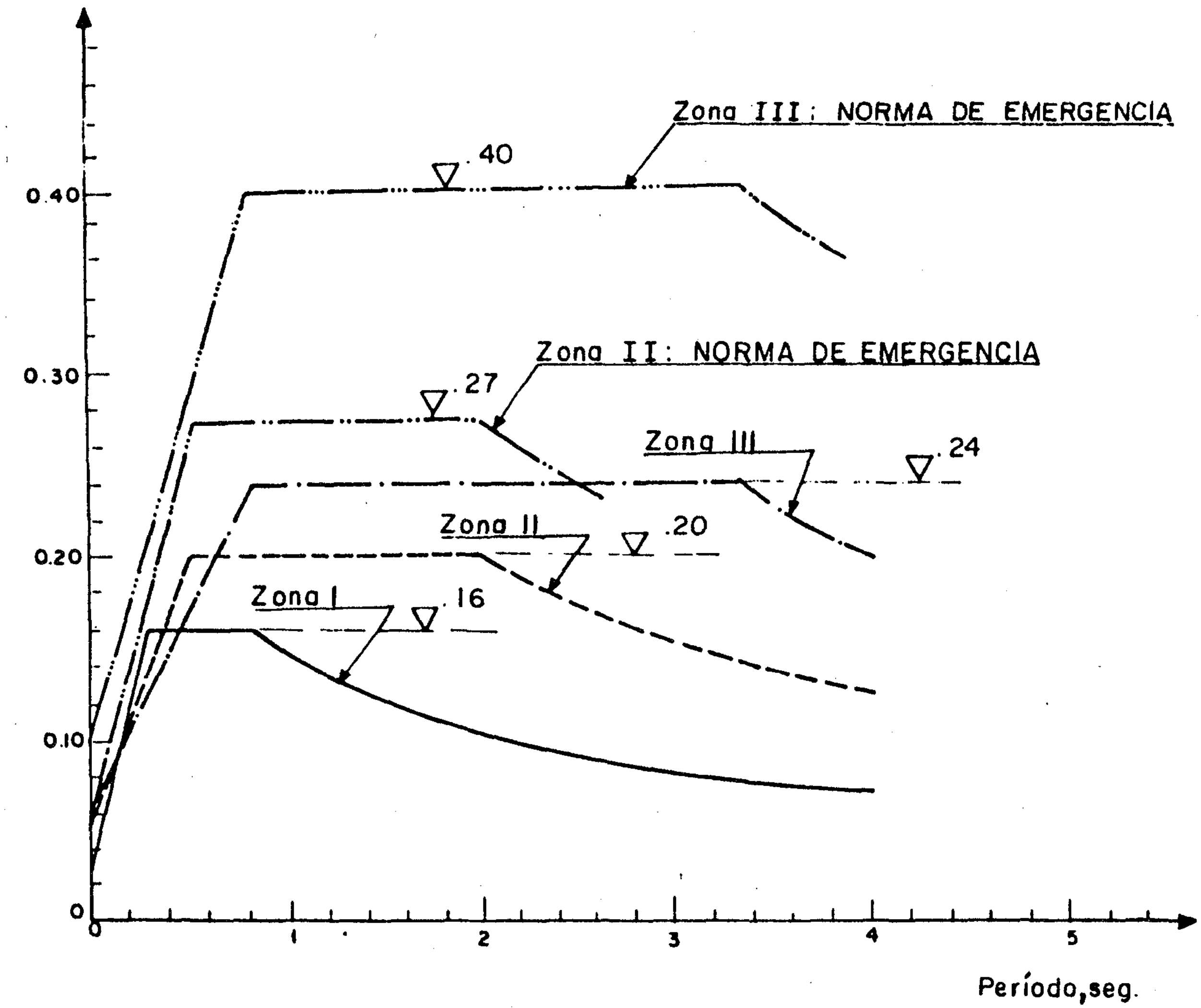


FIG. 57 COMPARISON OF THE 1976 MEXICO CITY BUILDING CODE AND THE 1986 EMERGENCY BUILDING CODE

and 3. San Francisco sites underlain by stiff alluvium and by soft clay under the provisions of the newly-revised 1988 SEAOC Code for ground motions representative of those produced by Magnitude $7\frac{1}{4}$ and 8+ earthquakes on the San Andreas fault,

are shown in Figs. 58, 59 and 60. It may be seen from these results that:

1. For a Magnitude $7\frac{1}{4}$ earthquake on the San Andreas fault in close proximity to San Francisco, the values of Damage Potentials Index provided for structures on stiff alluvial sites by the new (1988) code are significantly lower than those provided by the interim Mexico City code for buildings in the heavy damage area of Mexico City; and for structures constructed on sites underlain by soft San Francisco Bay mud, Damage Potential Index values are quite comparable to those provided by the interim Mexico City code for buildings in the heavy damage area of Mexico City (see Fig. 58).
2. For a Magnitude 8+ earthquake on the San Andreas fault in close proximity to San Francisco, similar to the 1906 earthquake, the values of Damage Potential Index provided by the new (1988) code for sites underlain by stiff alluvium are significantly lower than those for the heavy damage area of Mexico City in 1985 and they are generally about the same as those provided by the new Mexico City code for the heavy damage area in Mexico City.
3. For a Magnitude 8+ earthquake on the San Andreas fault in close proximity to San Francisco, similar to the 1906 earthquake, the values of Damage Potential Index provided by the new (1988) code for sites underlain by deposits of soft clay are generally similar to those existing in the heavy damage zone of Mexico City in 1985 and very much higher (by about 100%) than those now provided by

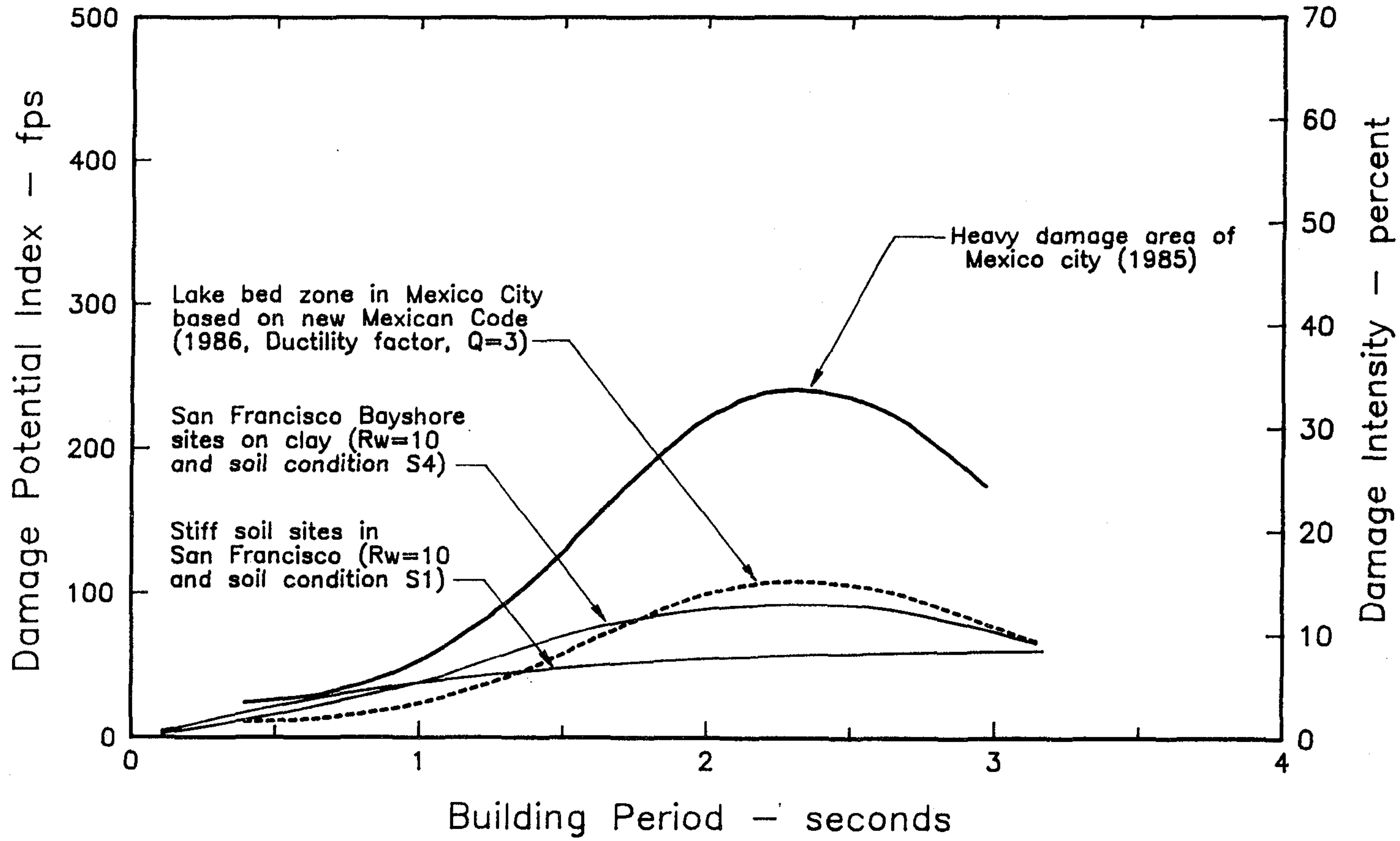


FIG. 58 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY FOR PRE-1985 AND 1986 CODES AND FOR SOFT CLAY SITES IN SAN FRANCISCO FOR M = 7¼ EARTHQUAKE AND 1988 CODE REQUIREMENTS

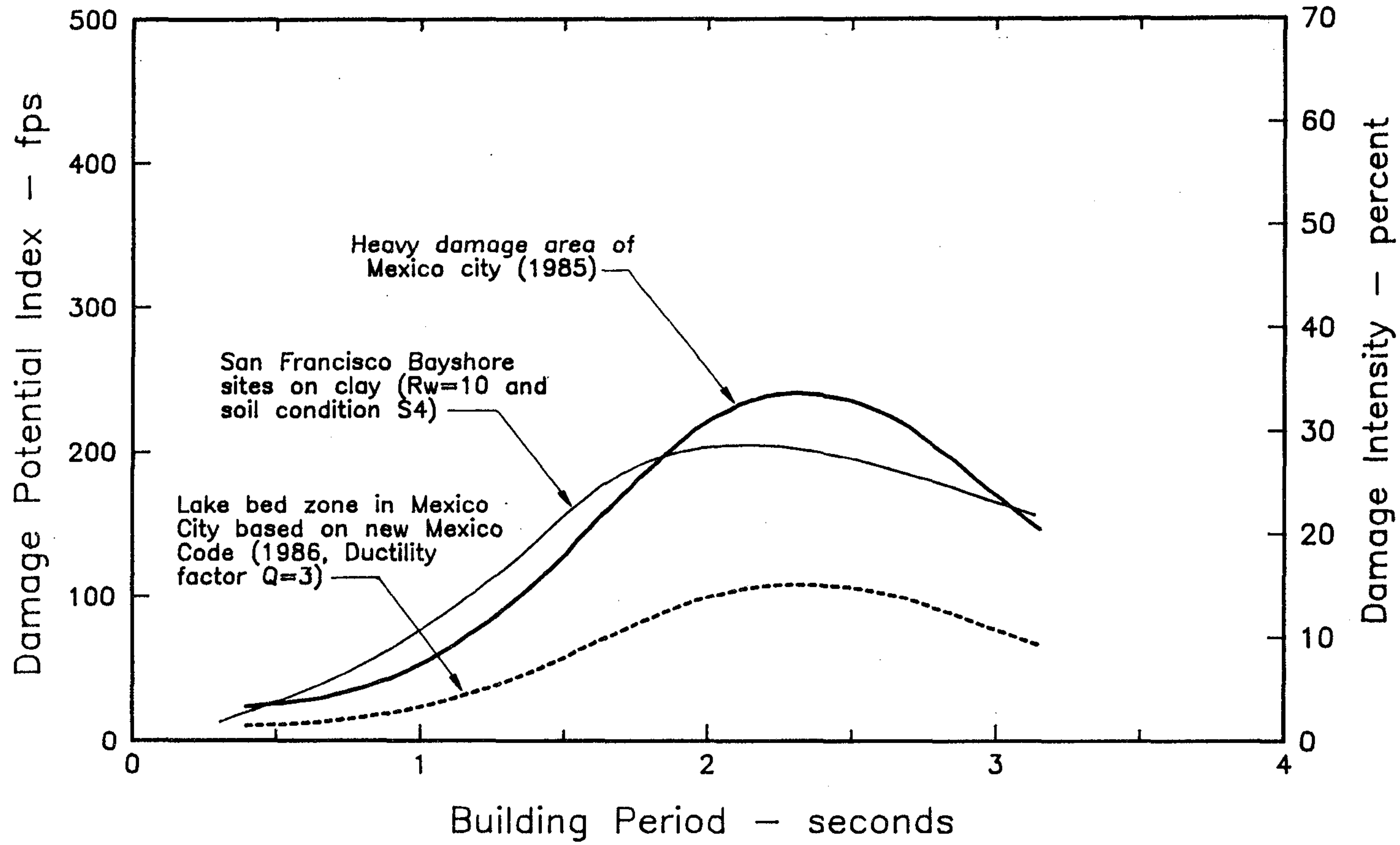


FIG. 59 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY FOR PRE-1985 AND 1986 CODES AND FOR SOFT CLAY SITES IN SAN FRANCISCO FOR M = 8+ EARTHQUAKE AND 1988 CODE REQUIREMENTS

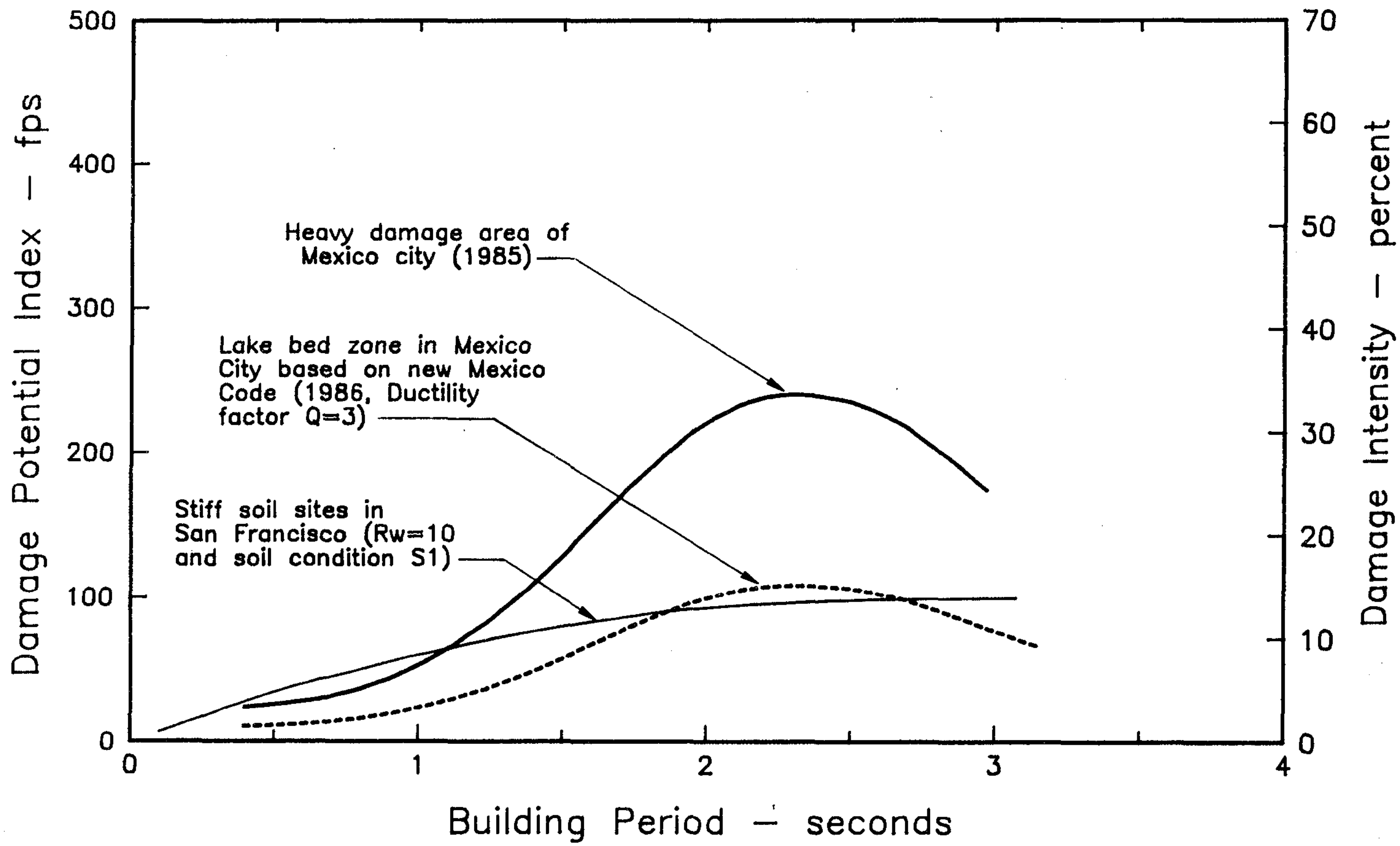


FIG. 60 COMPARISON OF DAMAGE POTENTIAL INDEX VALUES FOR HEAVY DAMAGE AREA OF MEXICO CITY FOR PRE-1985 AND 1986 CODES AND FOR STIFF SOIL SITES IN SAN FRANCISCO FOR M = 8+ EARTHQUAKE AND 1988 CODE REQUIREMENTS

the new interim code for clay sites in the heavy damage area of Mexico City.

This apparently high vulnerability of San Francisco Bayshore sites underlain by significant depths of clay would seem to indicate the need for a careful re-evaluation of the new SEAOC Code recommendations if it is desired to provide acceptable levels of safety for such sites in a possible repetition of the 1906 earthquake. Fortunately the area affected in San Francisco itself is relatively small and the probability of a Magnitude 8+ earthquake in the San Francisco Bay area in the next 30 years is rated relatively low (less than about 10%) in a recent study by the U.S. Geological Survey. However over a longer time period the probability of such an earthquake increases significantly and the implications of this study would seem to merit careful consideration in the light of this fact, not only for sites in San Francisco itself but also for areas around the Bayshore which are currently under development.

It is not the purpose of this report to suggest necessary actions for implementation in building codes; this can only be done by professional organizations like the Structural Engineers Association of California. It would seem desirable however that code requirements for sites underlain by San Francisco Bay mud in the San Francisco area be re-evaluated to ensure that they are compatible with their intended goals in the light of all of the factors involved, some of which (e.g. quality of construction) are not taken into account by such simple parameters as Damage Potential Index. At the same time, it is believed that comparisons based on Damage Potential Index values or similar parameters provide a reasonable basis for assessing in a quantitative way the combined effects of a number of considerations including ground motion characteristics, soil conditions, design lateral force coefficients and observed damage intensities in Mexico City in comparing design criteria in different countries and for different regions. It is hoped that the studies described in this report may serve as a guide in the continuing studies of site effects and design criteria for earthquake-resistant design undertaken by SEAOC and other agencies, and that similar evaluations may also be considered appropriate and useful for other cities underlain by clay deposits.

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