DYNAMIC ANALYSES OF TALL BUILDINGS FOUNDED IN DEEP FILL MATERIAL

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SYNOPSIS

The purpose of this paper is to illustrate the practical application of modern structural and soil dynamics techniques in the design of tall buildings founded in deep fill material. The results of an analysis of a 484' steel frame tower are presented. This structure is located in an area of high seismicity. Grade is approximately 245' above bedrock. The response of the building and resulting individual member stresses are discussed for four input earthquake motions. Comparisons with code wind and static earthquake requirements are also presented.

INTRODUCTION

Great strides have been made over the past several years in the fields of earthquake engineering and structural dynamics. Seismological and geological investigations into the effects and characteristics of past earthquakes has made it possible to determine the influence of Magnitude and hypocentral distance on the amplitude and frequency content of seismic motions. With the advent of large capacity, high speed digital computers, it has become technically and practically feasible to perform time-history dynamic analyses of high rise buildings. Finally, the results of extensive studies into the behavior of soil masses under vibrating loads has resulted in methods of modeling soil overburden and the capability of analytically determining the filtering effect of such soil on the earthquake motions occurring at the base rock. The purpose of this paper is to illustrate with an actual analysis the practical application of these modern techniques and show their effectiveness in evaluating the earthquake resistant capabilities of large high rise buildings.

DESCRIPTION OF STRUCTURE

Building Description

The building under consideration is 34 stories above grade and 135' by 185' in plan. The lateral forces on this structure are resisted by five welded steel frames in the longitudinal direction and six in the transverse direction. These frames, which extend 484' above the surrounding grade, consist primarily of

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square box columns and heavy steel wide flange girders. Shear forces are transferred by concrete floor diaphrams so that the displacement relationship between frames remains constant. A typical frame elevation and floor framing plan are shown in Figure 1. The exterior facing for the building is made up of four inch granite panels mounted with mechanical fasteners. For large earthquake motions, the concrete elevator core walls and moveable interior partitions provide negligible lateral resistance and their effect was not included in the analysis.

The foundation mat, located 40 feet below the surrounding grade, is supported by concrete piles which extend to Elevation -60. Below grade, in addition to the steel frames, there are heavy shear walls which transfer the lateral forces to the supporting soil.

Seismology and Geology of Building Site

The building site is located in San Francisco near two of California's major earthquake faults. The San Andreas passes about ten miles to the west of the site and the Hayward fault about ten miles to the east. There have been four great earthquakes recorded in this area; two, centered on the Hayward fault, occurred in 1836 and 1868 and the others, centered on the San Andreas fault occurred in 1838 and 1906. The intensities of the 1836 and 1868 quakes were X (MM) and IX-X (MM) respectively. The 1906 event had an intensity of XI (MM) and its Magnitude is estimated to have been 8-1/4. Although records are scanty, the 1838 earthquake is considered to have been similar to that of 1906.

The building site is located on made ground which was formerly part of San Francisco Bay. The loose sandy fill extends to a depth of about 20 feet and is underlain by about 35 feet of soft to medium clay. Below this is a 25 foot layer of dense silty fine sand followed by 155 feet of stiff clay and 10 feet of decomposed shale which extends to the Franciscan bedrock at a total depth of about 245 feet. An idealized soil profile with representative unit weights and shear strengths is presented in Figure 1.

POSTULATED EARTHQUAKE MOTIONS

Because of the potential earthquake exposure it was considered appropriate to evaluate the subject site for a great earthquake caused by movement along one of the adjacent faults. An evaluation was made of the effect this earthquake would have if centered on a fault directly opposite the site, and the effect if centered at varying distances from the site. These possible earthquakes were considered representative of those that could occur over the life of the structure.

The soil and structural response are a function of many parameters including the maximum rock input accelerations and the predominant frequency content of the base rock motion. Because of these variables, three postulated earthquakes having different motion characteristics were selected in order to bracket the probable structural response. The three earthquakes were made up of various combinations of past earthquake records with modifications in frequency and amplitude. All of these records were extended for sixty seconds of earthquake motion, however, the maximum response occurred within twenty seconds.

A number of investigators have studied the effects of Magnitude and distance on the frequency content and maximum accelerations of earthquake motions. The work of Gutenberg & Richter (1) and Esteva & Rosenblueth (2) are of particular significance in this area. The results of their analyses suggest that the period of peak response of the acceleration spectra tends to increase with increasing Magnitude and epicentral distance. Based on extensive study of past earthquake records, Wiggins (3) has shown the influence of these two parameters on the attenuation of ground accelerations. Using this information as a basis the characteristics of the three postulated input motions were determined.

Earthquake-1 is considered to be representative of a Magnitude 8.3 earthquake with an epicentral distance of 40 to 50 miles. This motion produced a maximum base rock acceleration of 0.15 gravity at the site. The acceleration response spectra corresponding to this motion is characterized by a peak response at a period of about 0.50 seconds. Earthquake-2 is considered to be an event of similar Magnitude but with an epicentral distance of about ten miles. The resulting base rock acceleration at the site and the period of peak response are 0.50 gravity and 0.25 seconds respectively. Earthquake-3, reflecting an earthquake of Magnitude 8.3 at a distance of 15 to 20 miles, produces values of 0.40 gravity and 0.35 seconds.

ANALYTICAL PROCEDURES

Analysis of Soil Response

The response of the soil at the site was determined for each of the three postulated base rock earthquake motions. For this analysis the soil layers were idealized as a lumped mass system separated by springs with non-linear hysteretic characteristics. The response of this system was evaluated using equivalent linear springs and damping factors taking into account the fact that these values are strain dependent. The soil response was calculated using iterative procedures. First, an average cyclic strain distribution was assumed and moduli and damping factors were determined. Then a response analysis was performed using the input earthquake to determine the average strains in the soil layers. These revised strains were used as the basis for the next iteration and the procedure was repeated until the actual soil strain conformed to the soil characteristics assumed in the analysis. A separate calculation was made for each input earthquake and the final iteration was used to produce the time-history of accelerations at the surface. A complete description of these procedures is contained in Seed & Idriss (4) and Idriss & Seed (5).

Analysis of Building Response

Using the results of the soil analysis described above, the building was analyzed for the three postulated earthquake motions at the foundation level. For comparison purposes, the time-history of the N-S component of the 1940 El Centro earthquake was also applied at the building foundation. The computer program used for these analyses employed the "exact" method described by Clough (6) and generates the total building response as well as the maximum member forces during the time-history excursion. The program assumes that all members respond elastically and includes the effects of axial and shear deformations in the columns. Because of the relatively soft soil below the supporting sand layer, the effect of foundation rocking was also included in the analyses. Results of the analyses were combined with previously determined static dead and live loads to obtain the maximum member stresses.

DISCUSSION OF RESULTS

Results of Soil Response Analysis

Using the analytical procedures described earlier, the soil model was subjected to the three postulated base rock motions. The near surface time-history of accelerations determined from these runs are reflected in the five per cent damped response spectra shown in Figure 2. For comparison purposes, a similarily damped response spectrum for the N-S component of the 1940 El Centro earthquake is also presented. For all three postulated motions, the ratio between maximum rock and surface accelerations, or the amplification factor, was somewhat less than 1.0. The maximum surface accelerations were 0.15, 0.39 and 0.34 gravity for Earthquakes-1, 2 and 3 respectively. In addition, the peak response periods and the form of the surface spectra are significantly different from those of the rock motions. These differences reflect the properties and characteristics of the soil overburden and its response to dissimilar earthquake motions.

Results of Building Analysis

The building was subjected to the surface motions of the three postulated earthquakes and, for comparison only, the N-S component of the 1940 El Centro event. The total building response in the form of displacements, moments, shears and acceleration are presented in Figures 3 through 6. These results are for 10 per cent damping for Earthquakes-2 and 3 and 5 per cent damping for Earthquake-1 and El Centro, all without including the effect of rocking. The effects of damping and rocking were evaluated separately and are discussed below.

Eight modes were considered in the analysis although a study of the time-history response showed that for all of the earthquake motions, the building responded primarily in the first and second modes and the effect of the higher modes was small. The periods of vibration for the eight modes as well as the corresponding spectral accelerations for each of the four earthquake motions are presented in Table 1.

The results presented indicate that the building response and resulting member forces are controlled by different earthquakes at various levels in the structure. This result is due to the diversity of amplitudes and frequency characteristics of the input motions. For example, Figure 3 shows that Earthquake-3 causes the maximum displacement above the fifteenth floor while Earthquake-2, having a very high peak in its response spectrum near the second mode period of the building, controls the displacements below the fifteenth floor. Therefore, assuming any or all of the postulated earthquakes could occur during the life of the structure, the envelope of the maximum response due to all three motions was considered appropriate criteria for the evaluation of the structure

Effects of Damping

A review of the response spectra for the earthquake motions indicated that Earthquake-2 and 3 would produce stress levels substantially above yield and, therefore, a value of ten per cent damping was considered representative of the energy absorption under these motions. The stress levels anticipated for Earthquake-1 and El Centro were at or slightly above yield and five per cent damping was considered appropriate for these earthquakes.

Effects of Rocking

The sand layer into which the piles extend is underlain by some 155 feet of medium and stiff clays. Because these soils are subject to compression under overturning loads, the effect of including the coupled rocking springs below the foundation was also studied. The stiffness of these springs is a function of the soil modulus which in turn is strain dependant. The value selected was based on the strain caused by the maximum dynamic overturning moment. The inclusion of these springs lengthened the first mode period of vibration by six per cent to 4.62 seconds. The periods of the other modes were not affected. This change in period reduced the first mode spectral accelerations for all three earthquakes. The first mode shape was also changed because of the rotation about the foundation. When compared to the fixed foundation case, the deflections remained substantially unchanged while the shears and moments were reduced by about ten per cent.

Code Requirements and Member Stresses

The results of the dynamic analyses presented in Figures 3 through 6 are between 3 and 3.75 times greater than would be obtained by applying Zone 3 code lateral forces. This relationship between code static and actual dynamic results is rather common for high rise buildings but does not, in itself, provide a basis for evaluating the adequacy of the structure. The semi empirical seismic criteria set forth in the building code are primarily based on observations of the performance of a limited number of stiff, traditional structures during past earthquakes. Such structures are characterized by a relatively short first mode period and the participation of many non structural elements in resisting the lateral forces. Modern building concepts, on the other hand, produce flexible structures having very few participating non structural elements that could assist the structural frame in resisting lateral forces. While this trend in modern buildings does not necessarily invalidate the code seismic criteria, it does suggest that the code be supplemented with dynamic analyses. Such supplemental analyses will provide the designer with valuable insight into the behavior of the building during earthquakes and assist him in locating and strengthening potentially vulnerable components of the structure.

One of the main factors which provide the designer with a means of evaluating the building under dynamic loadings is the level of member stresses. The total member stresses were obtained by combining the maximum seismic stresses with those determined for the dead plus live load condition. These total stresses for Earthquake-1 and El Centro were generally less than yield for the columns and at or slightly above yield for the girders. For Earthquake-2 and 3, the girders had actual/yield moment ratios of from 2 to 2-1/2. The relationship between girder stresses and column stresses suggests that when plastic hinges form in the building, they will tend to form in the girders rather than the columns. In view of this fact, and considering the welded steel frames are capable of withstanding significant excursions into the inelastic range and that the duration of these excursions would be relatively short, the resulting total stress levels are not considered excessive.

The columns under Earthquake-2 and 3 were stressed substantially above yield. Criteria for their evaluation was that the ultimate capacity of the section should not be exceeded. Based on this criteria, increases in certain third floor and exterior corner columns were recommended in order to bring the capacity of these members up to the level of the remainder of the structure.

CONCLUSIONS

This paper has demonstrated, by example, the feasibility of applying sophisticated analytical techniques to determine the earthquake resistant capabilities of a large high rise building. The class of structure represented by the sample building is believed to be typical of many now being constructed in areas of high seismic activity, and the procedures described emphasize the importance of combining the efforts of several engineering disciplines to achieve a satisfactory seismic design.

Based on the results presented, the significant conclusions can be summarized as follows:

(1) The filtering effect of the soil overburden at the site cannot be neglected. For instance, the use of the rock motion in the analysis of this building would have produced significantly smaller response than produced by the near surface motion.

(2) Since building response is controlled by different earthquakes at different levels in the structure, the importance of considering several earthquakes having different frequency and amplitude characteristics rather than just one earthquake is obvious.

(3) The results of a dynamic analysis of this type can be employed to produce a structure with more nearly uniform seismic resistant capabilities.

(4) The results suggest that building codes should not be the sole basis for evaluating the earthquake resistant capabilities of large high rise buildings if such buildings have unusual foundation conditions or do not conform to the type of buildings used by the code as the basis for seismic criteria.

(5) For this structure it appears that the assumption of a fixed base would be sufficient.

(6) The building's structural integrity would not be impaired by the postulated great earthquakes, although architectural damage is inevitable.

ACKNOWLEDGEMENTS

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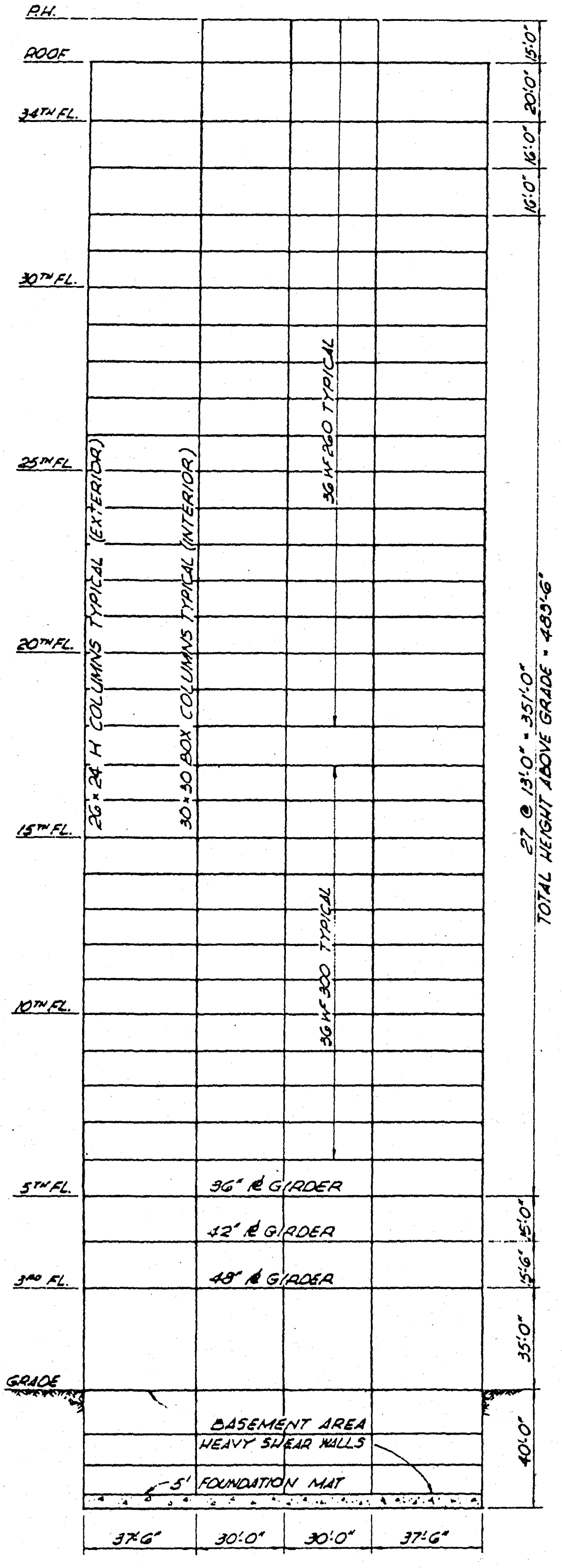
PG&E BUILDING
MODE PERIODS AND SPECTRAL ACCELERATIONS

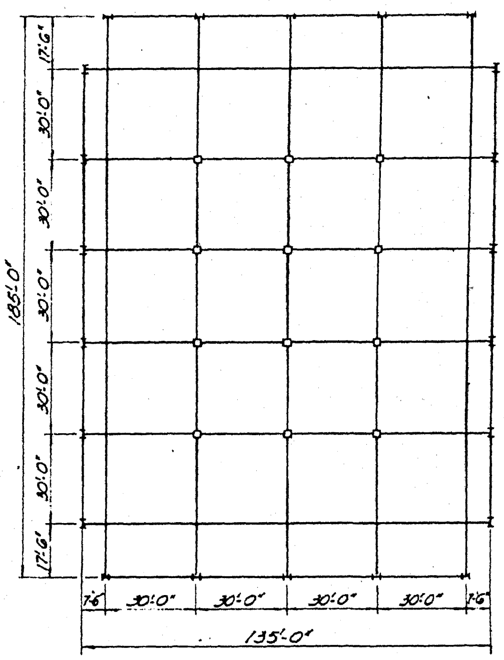
(rocking not included)

MODE	PERIOD (SECONDS)	SPECTRAL ACCELERATIONS			
		E.Q. - 1 DAMPING 5%	E.Q. - 2 DAMPING 10%	E.Q. - 3 DAMPING 10%	EL CENTRO DAMPING 5%
1	4.37	0.06	0.05	0.06	0.07
2	1.48	0.29	0.76	0.58	0.21
3	0.83	0.50	0.46	0.70	0.47
4	0.59	0.33	0.38	0.47	0.83
5	0.45	0.29	0.50	0.43	0.98
6	0.37	0.24	0.47	0.40	0.86
7	0.31	0.18	0.42	0.39	0.81
8	0.26	0.16	0.37	0.38	0.87

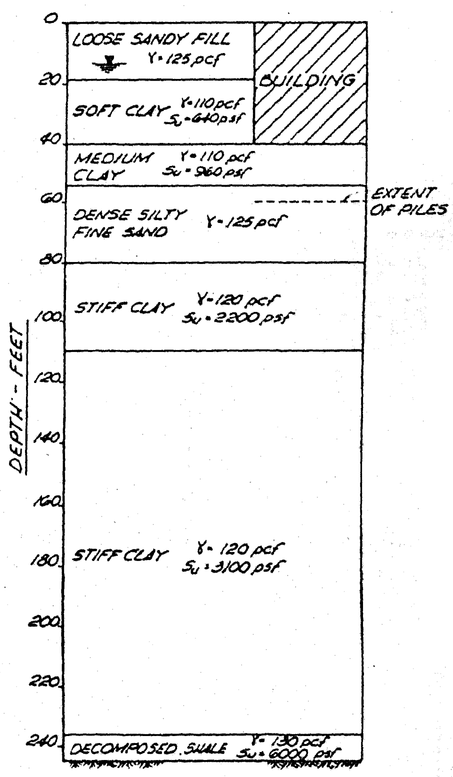
TABLE 1



TYPICAL INTERIOR FRAME



TYPICAL FLOOR FRAMING PLAN



SOIL PROFILE AT SITE

FIGURE 1

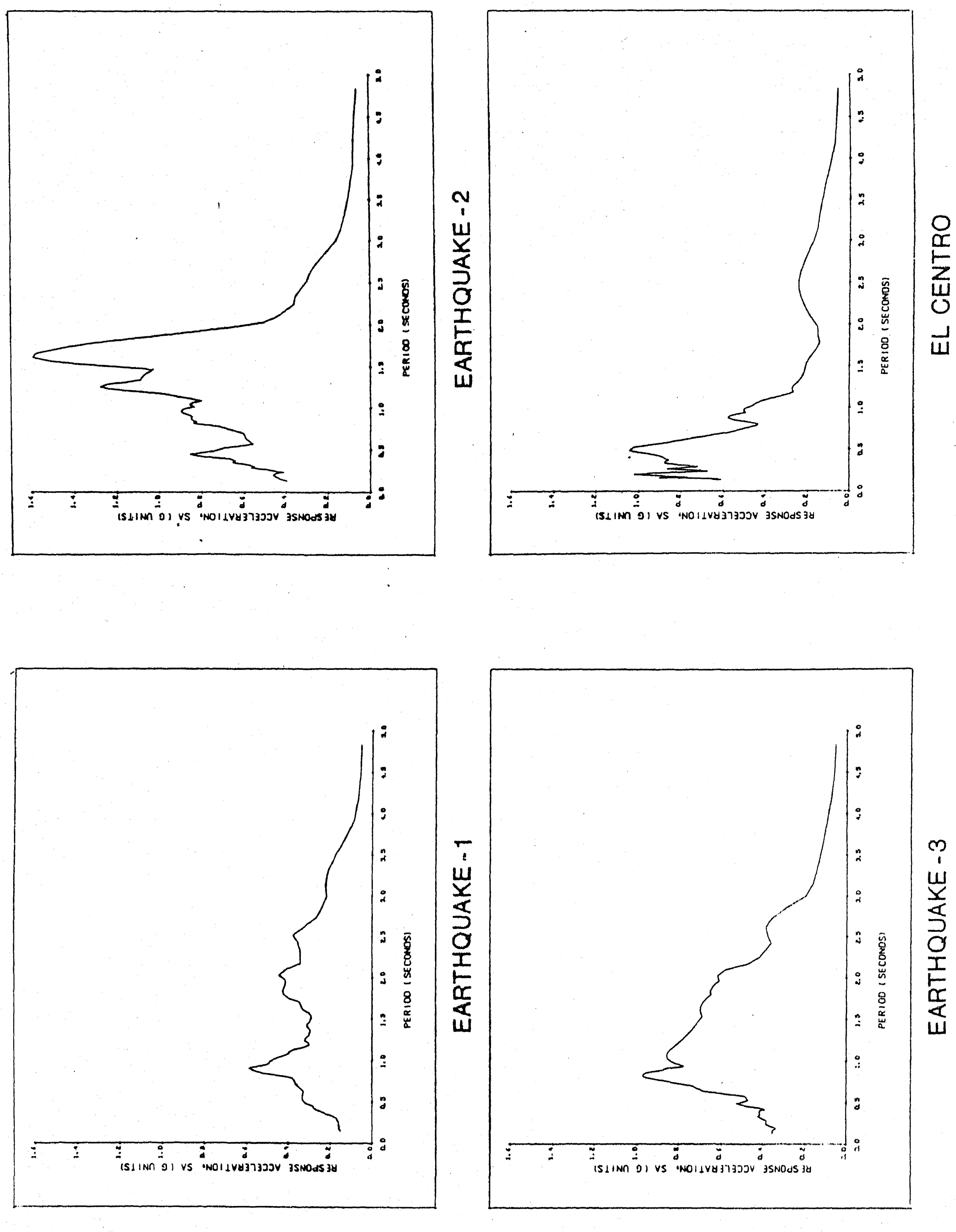

EARTHQUAKE - 2
EL CENTRO
EARTHQUAKE - 1
EARTHQUAKE - 3
RESPONSE ACCELERATION, SA (G UNITS)
PERIOD (SECONDS)

FIGURE 2

MAXIMUM DISPLACEMENT DIAGRAM

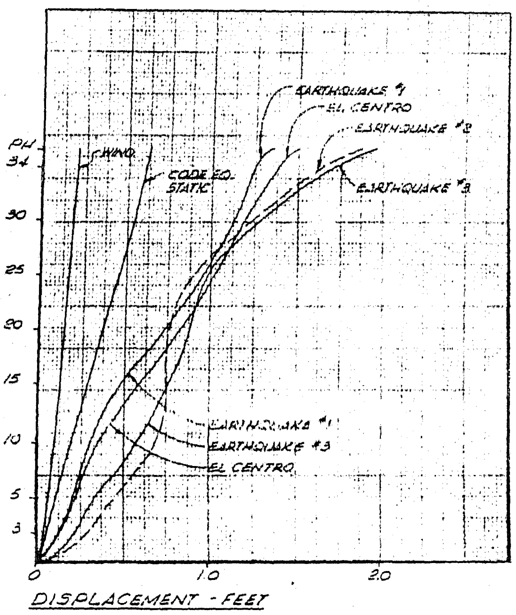


FIGURE 3

MAXIMUM MOMENT DIAGRAM

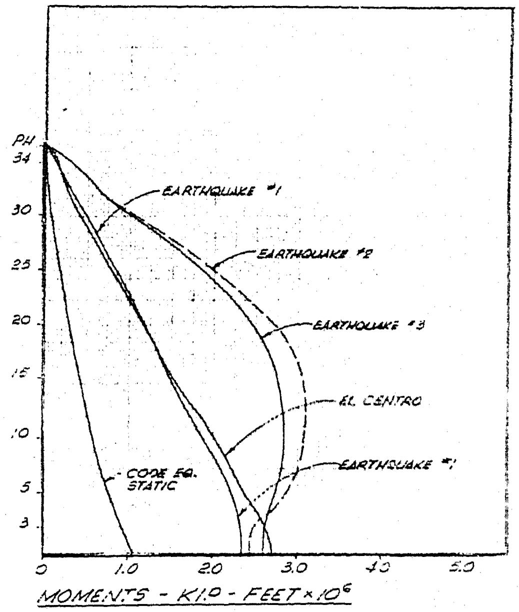


FIGURE 4

MAXIMUM SHEAR DIAGRAM

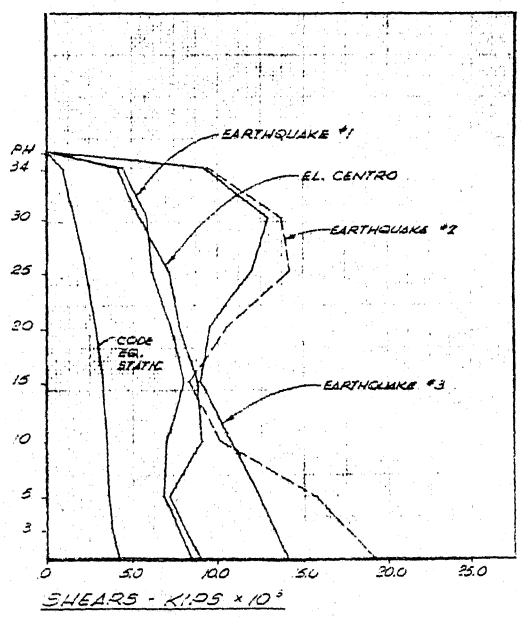


FIGURE 5

MAXIMUM ACCELERATION DIAGRAM

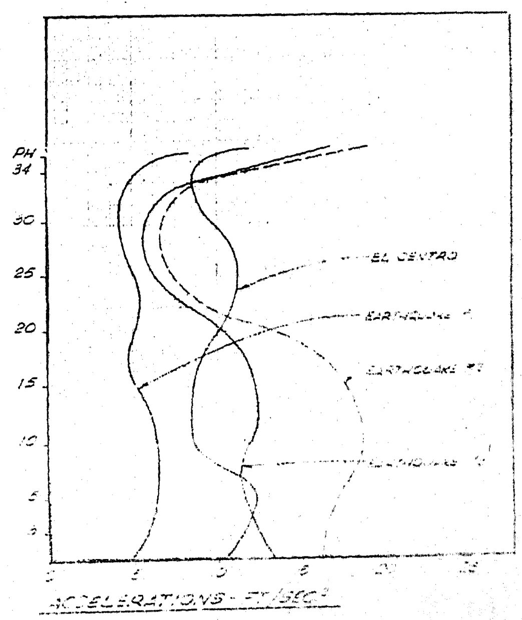


FIGURE 6