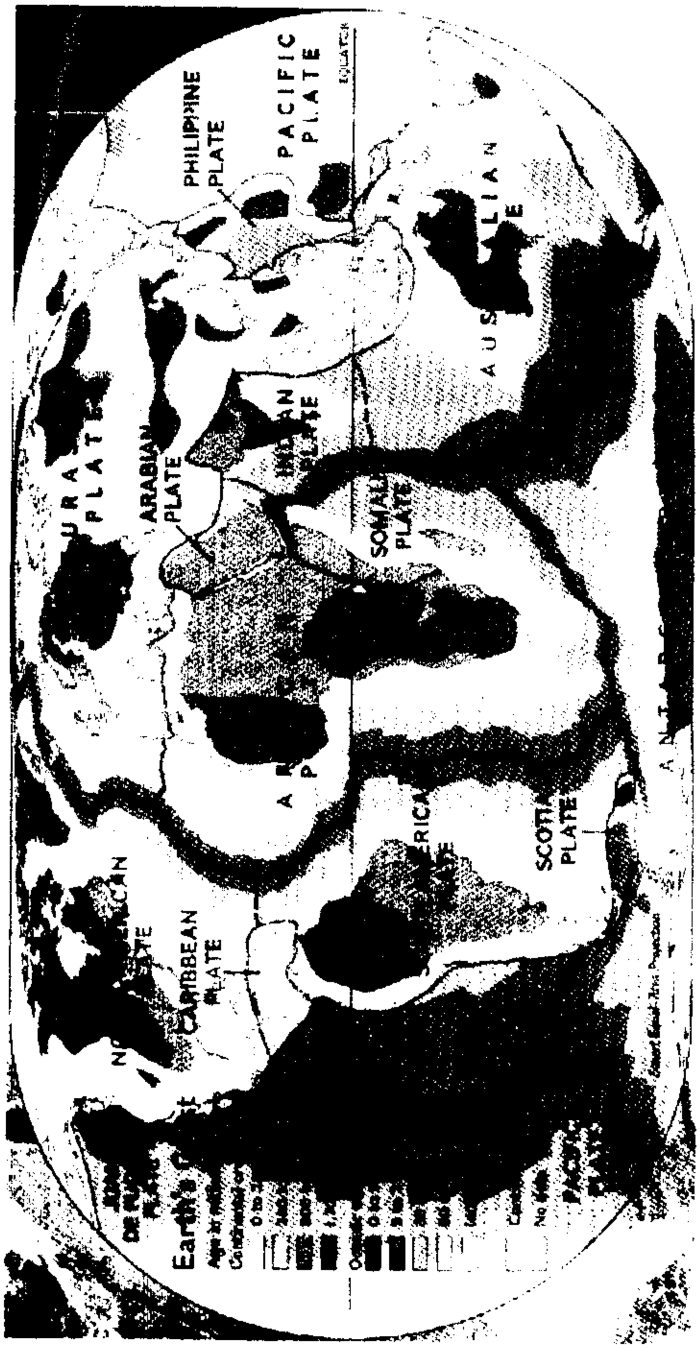
Continued Development of Earthquake Load and Resistance Factor Design Guidelines

Report 2

Seismic Hazard Characterizations



Report to

STATOIL and UNOCAL

Stavanger, Norway
and
Houston, Texas

by

Professor Robert Bea

Marine Technology & Management Group

Department of Civil & Environmental Engineering
University of California at Berkeley

May 1998

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Table of Contents

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Chapter 1

Introduction

1.0 Objective

The objective of this study is to continue development of earthquake load and resistance factor design (LRFD) guidelines as follows:

Part 1) concrete gravity based structure (GBS) LRFD guidelines (sponsored by Health and Safety Executive),

Part 2) seismic hazard characterizations (sponsored by STATOIL and UNOCAL) and

Part 3) verification of guidelines (sponsored by U. S. Minerals Management Service).

This report summarizes the results of the second part of this study; seismic hazard characterizations. Two primary topics are addressed:

a) seismic zonation and uncertainties of the UK and Norwegian Sectors of the North Sea, offshore Japan and Indonesia, and the Bay of Campeche, Mexico; and

b) Strength Level Earthquake (SLE) elastic response spectra.

1.1 Background

A first-generation LRFD ISO guideline for design of steel, pile supported, template-type platforms to resist earthquake induced loading has been developed (Bea 1991; 1997a). This guideline was based on the API RP 2A LRFD guideline and on the collective experience and judgment of the ISO P5 committee members. During the first generation developments, limited verification studies of the entire process were performed, there was limited documentation of the background for some of the key developments, and there were additional topics that needed to be addressed. These topics have been discussed and the P5 committee recommended that additional studies be conducted on the two topics cited above.

The report titled *Continued Development of Earthquake Load and Resistance Factor Design Guidelines, Report 1 - Concrete Gravity Base Structures LRFD Guidelines* documented the results of the first phase of this study (Bea, 1998).

This study was organized as follows:

- obtain recent background on seismic hazard characterizations.
- detail seismic exposure uncertainties and local site response uncertainties.
- reconcile differences between current ISO seismic hazard guidelines and those developed during this study.
- document the revised seismic hazard and response spectra characterizations in a project technical report (Report #2).

1.2 Earthquake LRFD Approach

The LRFD approach used to develop the proposed ISO earthquake design guidelines can be expressed analytically as follows:

$$\phi_E R_E \geq \gamma_{D1} D_1 + \gamma_{D2} D_2 + \gamma_{L1} L_1 + \gamma_E E$$

where ϕ_E is the resistance factor for earthquake loadings, R_E is the design capacity of the platform element (e.g. brace, joint, pile) for earthquake loadings as defined by the API RP 2A - LRFD guidelines, γ_{D1} is the self-weight of the structure (dead) loading factor, D_1 is the design dead loading, γ_{D2} is the imposed equipment and other objects loading factor, D_2 is the design equipment loading, γ_{L1} is the consumables, supplies, and vessel fluids (live) loading factor, L_1 is the live loading, γ_E is the earthquake loading factor, and E is the earthquake loading effect developed in the structure or foundation element. This development addresses the definition of the resistance factors, ϕ_E , and the loading factors, γ_E , for loadings induced by earthquakes (Bea, 1997a). The dead, equipment and live loading factors are set as $\gamma_{D1} = \gamma_{D2} = \gamma_{L1} = 1.1$.

The earthquake loading factor is determined using a Lognormal format (Bea 1991):

$$\gamma_E = \mathbf{Fe}\, \mathbf{B}_E \exp (0.8\, \beta_E\, \sigma_E - 2.57\, \sigma_E)$$

where **Fe** is the median effective loading factor, $\mathbf{B}_E$ is the median bias (actual / nominal) in the computed earthquake loadings, β_E is the annual Safety Index designated for the platform SSL, σ_E is the total uncertainty associated with the earthquake loadings (standard deviation of the annual maximum earthquake loading), 0.8 is the splitting factor used to separate the uncertainties in earthquake loadings and platform capacities, and the 2.57 is a consequence of defining the elastic design earthquake (Strength Level Earthquake) at an average return period of 200-years (2.57 Standard Normal deviations from the mean).

The effective loading factor for the platform system is expressed as:

$$\mathbf{Fe} = [\, \mu\, \alpha\,]^{-1}$$

where μ is the platform system ductility and α is the platform residual strength ratio:

$$\mu = \Delta p / \Delta e$$

$$\alpha = A / Aep$$

Δp is the maximum plastic displacement that can be developed by the platform system at 'failure', Δe is the displacement at which first significant nonlinear behavior is indicated by the platform system, A is the area under the platform loading - displacement to failure diagram, and Aep is the area under an equivalent elasto-plastic platform loading - displacement to failure diagram.

The total uncertainty associated with the earthquake loadings (σ_E) is expressed as:

$$\sigma_E^2 = \sigma_{SE}^2 + \sigma_{GS}^2 + \sigma_{RS}^2$$

where σ_{SE} is the uncertainty in the earthquake horizontal peak ground accelerations, σ_{GS} is the uncertainty related to the local geology and soil conditions and their effects on the ordinates of the earthquake response spectra, and σ_{RS} is the uncertainty associated with the response spectrum method used to determine forces in the elements that comprise the platform.

In development of the proposed ISO earthquake guidelines, Type I (natural, inherent, aleatory) uncertainties are included in σ. The effects of Type II (model, parameter, state, epistemic) uncertainties are introduced with the Biases (actual value / nominal value) in loading effects and capacities. This is done to develop as close as possible a correlation with the way in which the target Safety Indices and probabilities of failure were determined.

The platform element resistance factor is determined from:

$$\phi_E = \mathbf{B}_{RE} \exp(-0.8\ \beta_E\ \sigma_{RE})$$

where $\mathbf{B}_{RE}$ is the median bias (actual value / nominal or code value) in the element earthquake loading capacity, and is the σ_{RE} uncertainty in the earthquake loading capacity of the element .

The proposed ISO earthquake LRFD approach proceeds through the following nine steps (Bea 1997a):

1. Define the structure safety and serviceability level (SSL)
2. Define the earthquake hazard zone (EHZ) and seismotectonic conditions (Fig. 1.2.1)
3. Determine if site specific seismic exposure study is required (Table 1.2.1)
4. Define the shape of the normalized mean elastic principal horizontal acceleration response spectrum for a specified damping ratio, soil profile, and seismotectonic condition (Fig. 1.2.2, Table 1.2.2)
5. Determine the uncertainties associated with the seismic exposure, response spectrum, and the methods used to analyze earthquake forces induced in the structure (Table 1.2.3, Table 1.2.4)
6. Determine the earthquake loading factor (γ_E) for the elastic response spectra forces
7. Evaluate the biases and uncertainties in the platform element design capacities
8. Determine if a ductility analysis is required and the performance characteristics for the analyses
9. Determine the platform element resistance factors (ϕ_E) for the design code based capacities

The remainder of this report will summarize the study of seismic hazard characterizations (Steps 2 and 5) based on results from recent studies and local site response spectra characterizations (Steps 4 and 5). In the context of the loading and resistance factor formulation summarized in Eqn. (1) through (7), the key issues included in this study were:

- G_{200} - the peak horizontal ground acceleration at a return period of 200 years (Fig. 1.2.1)

σ_{SE} - the uncertainty associated with the seismic environment

- σ_{GS} - the uncertainty related to the local geology and soil conditions and their effects on the ordinates of the earthquake response spectra, and
- Sa / G vs. T - the shape of the normalized SLE horizontal acceleration response spectrum for a specified damping ratio of D = 5 % (Fig. 1.2.2).

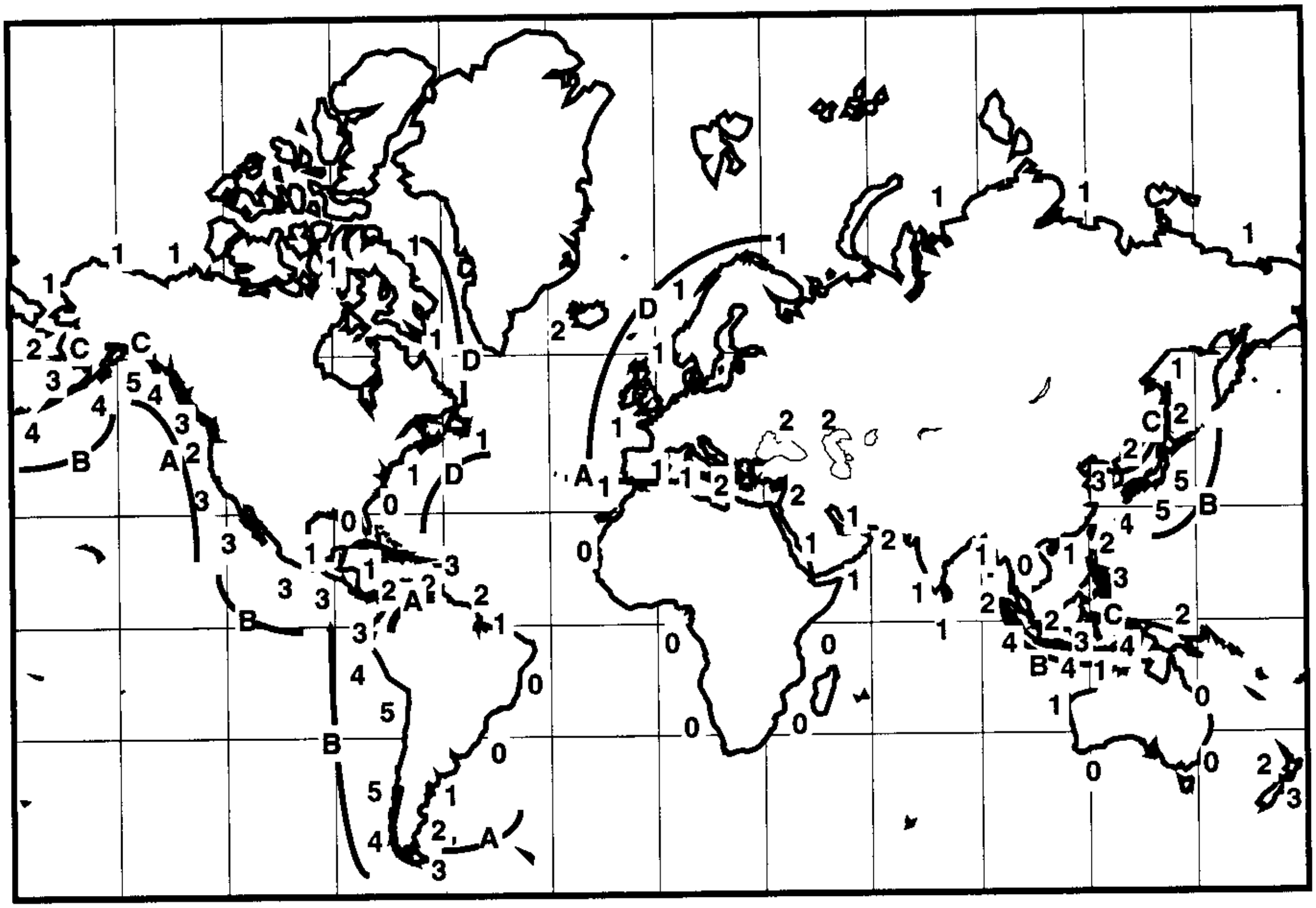


Fig. 1.2.1 - Proposed ISO Earthquake Hazard Zones (EHZ)

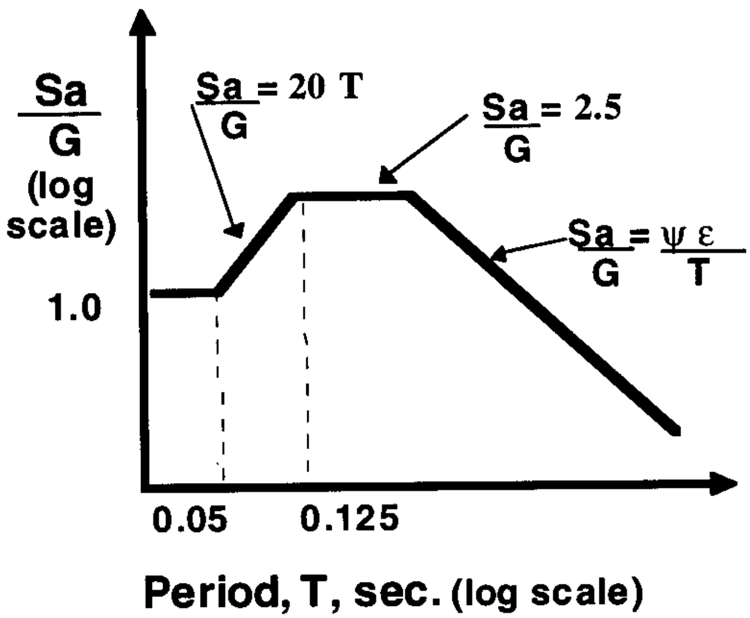


Fig. 1.2.2 - Proposed ISO Strength Level Earthquake Normalized Elastic Horizontal Acceleration Response Spectrum

Table 1.2.1 - Proposed ISO Earthquake Hazard Zones, SLE Ground Accelerations (G) and Structural Serviceability & Safety Levels (SSL)

Zone (1)	SLE G % g (2)	SSL #1 (3)	SSL #2 (4)	SSL #3 (5)	SSL #4 (6)
0	0 - 5	Allowed		Should do site specific study	MUST
1	5 -15	to use			DO
2	15 -25	these			SITE
3	25 -35	guidelines			SPECIFIC
4	35 -45				STUDY
5	45 -55				

Table 1.2.2 - Proposed Local Geology and Soil Conditions Parameter, ψ, and Seismotectonic Characteristics Parameter, ε

Soil Conditions (1)	ψ	Seismotectonic Conditions (2)	ε
SC - A Rock (Vs ≥ 2,500 fps)	1.0	**Type A** Shallow crustal faulting zones	1.0
SC - B Stiff to very stiff soils, gravels (Vs = 1200 to 2500 fps)	1.2	**Type B** Deep subduction zones	0.8
SC - C Sands, silts, and very stiff clays (Vs = 600 to 1200 fps)	1.4	**Type C** Mixed shallow crustal & deep subduction zones	0.9
SC - D Soft to medium stiff clays (H = 10 to 200 ft.; Vs ≤ 600 fps)	2.0	**Type D** Intraplate zones	0.8
SC - E Site specific studies required	-	**Default value**	1.0

Table 1.2.3
Seismic sources and attenuation uncertainties

Seismotectonic Characteristics	σ_{SE}
Shallow crustal fault zones	1.0
Deep subduction zones	1.4
Mixed shallow crustal fault and deep subduction zones	1.2
Intraplate zones	2.0

Table 1.2.4
Uncertainties associated with local geology and soil conditions

Geology / Soil Conditions	σ_{GS}
SC - A	0.30
SC - B	0.40
SC - C	0.40
SC - D	0.50
SC - E	0.50

1.3 RAM Based ISO Earthquake Guidelines

The approach used to develop the ISO earthquake guidelines for design and requalification are based on a Risk Assessment and Management (RAM) approach in which the seismic conditions are integrated with the performance characteristics of alternative platform configurations to define the reliabilities and risks associated with these configurations. ***The earthquake design and requalification criteria are the basic product of a desired reliability of the platforms.***

This is a very different approach than has been used traditionally to specify earthquake criteria for onshore facilities. Traditionally, earthquake criteria has been the result of two generally uncoordinated developments. The first development came from geologists, seismologists, and earthquake engineers. They were responsible for the characterization of the earthquake excitation that would be used for design of structures. The basis for their characterization was some earthquake return period. Often this was based on specification of a life of the structure and an acceptable probability of exceedance. For example, buildings were often designed for a 50 year life with a 10 percent probability of exceedance. This is equivalent to design of the structure for an earthquake that has a return period of about 475 years. The seismologists would then define design earthquakes that would be representative of the different types of earthquakes that could be experienced. Representative earthquake time histories and elastic response spectra were often used for this definition.

The second development came from structure design engineers. They were responsible for the determination of how the structures would be configured and designed. They defined the factors of safety that would be used in design of the structures. Often, the design procedures and factors of safety were the result of past of experiences with the performance of buildings that had experienced intense earthquake excitations. If the performance of components in buildings proved to be insufficient, the configurations, procedures, and factors of safety were changed to improve the performance.

The RAM procedure used in development of the ISO earthquake guidelines is based on an integration of the two developments and is focused on the reliability of the resulting structure. In this instance, the earthquake design spectra and design procedures are intimately related. The design spectra characteristics are a direct consequence of the defined reliability and the procedures and processes that are used to configure and proportion the structure. This point can be illustrated by the following RAM based equation to define the design elastic horizontal response spectra ordinate (Sa_D) for a structure that has a given natural period (Tn) and damping ratio (D) (Fig. 1.2.2):

$$Sa_D = (Ru_{50} / \mathbf{Fe}\ M)\ (\mathbf{B}_{\mathbf{Ru}} / \mathbf{B}_{\mathbf{FQ}})\ \exp(\beta_Q\, \sigma_Q - Z_D\, \sigma_{FQ})$$

Ru_{50} is the median static ultimate lateral loading capacity of the structure. **Fe** is the factor that recognizes the combination of the transient earthquake induced loadings and the nonlinear hysteretic performance characteristics of the structure. M is the total effective mass of the structure including the structure, live and dead masses, and hydrodynamic masses (often the total mass of an offshore structure is 50 % to 70 % hydrodynamic mass). β_Q is the design Safety Index for the structure (directly related to the structure reliability or its compliment the probability of failure). σ_Q is the total uncertainty in the structure capacity and the earthquake induced loadings (square root of the sum of the squares of the standard deviations of the logarithms of the structure capacity and earthquake loadings).

Z_D is a factor that depends on the return period that is used to define the design earthquake; often, this is taken to be a 100-year or 200-year condition for elastic working stress or load and resistance factor design formats; for 100-year and 200-year return periods $Z_D = 2.33$ and $Z_D = 2.57$, respectively (this is the number of standard deviations from the mean to the 99-th and 99.5-th percentiles, respectively). σ_{FQ} is the uncertainty in the earthquake loadings induced in the structure. $\mathbf{B}_{\mathbf{Ru}}$ is the median bias in the structure ultimate capacity (best estimate or true value divided by nominal or predicted value). $\mathbf{B}_{\mathbf{FQ}}$ is the median bias in the earthquake induced loadings (demands). If unbiased estimates are used to determine the earthquake loadings and structure capacities, the biases are unity.

The factors Tn, D, Ru_{50}, Fe, and M are fundamentally structural design factors that depend on the structure materials, configuration and proportioning. β_Q is a derivative of the desired or accepted reliability of the structure. σ_{FQ} is a derivative of how the structure responds to the earthquake (induced loadings or forces) and the long-term uncertainty in the earthquake ground accelerations. The total uncertainty σ_Q reflects the uncertainties in both the structure capacity and the earthquake induced loadings. It should be apparent that the spectral acceleration is a direct product of structure and reliability considerations and can not or should not be developed or defined in isolation of these considerations.

In the long period range ($Tn \geq \approx 1$ sec), the spectral acceleration can be expressed as:

$$Sa_D = (K\, \varepsilon\, \psi / Tn)\ G_D$$

K expresses the number of standard deviations of the response spectra ordinate from the mean value. If other than the mean value is used (K = 1), then the bias in the earthquake loadings must be recognized. ε is a factor that depends on the seismotectonic (earthquake sources) environment (for strike slip sources e $\approx$ 1.0 and for subduction zone sources $\varepsilon \approx 0.8$). ψ is a factor that depends on the local geology and soil conditions ($\psi \approx 1.8$ for deep soft alluvium and $\psi = 0.8$ for rock or very strong sedimentary formations). G_D is the design ground acceleration that is defined at the specified return period. It is apparent that the majority of these factors are those that should be determined by the earthquake engineer or geo-seismologist.

Based on the RAM approach, the 'design' or 'requalification' spectrum (Sa vs Tn) is dependent on an explicit and integrated evaluation of the following categories of factors:

1) characteristics of and uncertainties in the seismic - tectonic - geologic earthquake conditions (expressed with G, ψ, ε, $\mathbf{B}_{FQ}$, σ_{FQ}),

2) characteristics of and uncertainties in the seismic performance characteristics of the structure and the associated process used to design or requalify the structure (expressed with Ru, Tn, D, **Fe**, M, , Z_D , K, σ_{Ru} , $\mathbf{B}_{ru}$), and

3) the target reliability for the structure (expressed with β).

The foregoing RAM based development results in an intimate integration of:

1) management's direction regarding acceptable or desirable reliability of a structure,

2) input from the earthquake engineer or geo-seismologist, and

3) input from the structure design engineer.

In this context, the design conditions or design spectra can not and should not be developed in isolation by the earthquake engineer or geo-seismologist. Many of the earthquake and structure parameters involved in the RAM are highly coupled; they can not and should not be separated; for example,the loads induced in the structure by earthquake ground motions are a direct function of the characteristics of the structure.

It is important to recognize that the RAM based earthquake spectrum that is used to design or requalify a structure explicitly incorporates the uncertainties in the seismic environment. For example, even though a mean spectrum is used in the specification given to the engineer, the potential for ground motions that are much larger or more intense are directly recognized in the uncertainties attributed to the mean spectral ordinates. One should not be unduly concerned when peaks in some response spectra exceed the design or requalification spectrum. The uncertainties in the ground motions have a direct impact on the factors of safety and Reserve Strength Ratios that are specified for a particular structure. More uncertainty leads to larger factors of safety and Reserve Strength Ratios.

There are four major parts to the seismic exposure models used to characterize earthquake ground motions (Fig. 1.3.1):

1) characterization of the seismic sources,

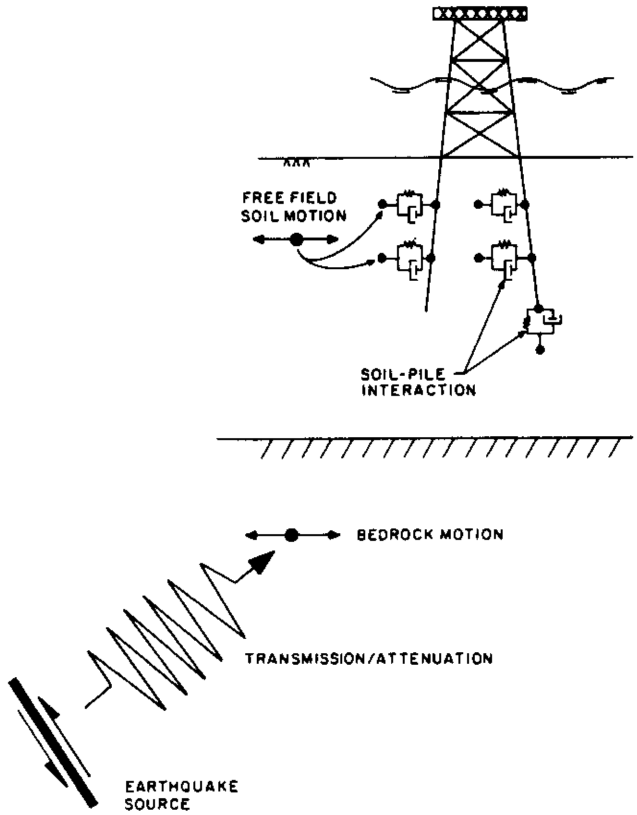
2) characterization of the transmission of earthquake energy from the sources to the site,

3) characterization of the local geology and geotechnical effects on the 'free-field' motions of the soils, and

4) description of the soil-structure interactions that develop during the earthquakes ('near-field motion characterizations).

For pile supported platforms, the pile foundations supporting the platforms receive their input energy from earthquakes from two distinctly different points in the soil column. The input of lateral energy occurs below the sea floor where the maximum lateral earth pressures can be generated on the piles (Fig. 1.3.2). Generally, this point is located at five (stiff soils) to ten (soft soils) pile diameters below the sea floor. Frequently, the seismic exposure characterizations are developed for the level of the sea floor. This is not appropriate for pile supported offshore structures. This can be appropriate for mat supported structures that receive the majority of their support from the soils in the immediate vicinity of the sea floor. The error introduced by the sea floor reference for the earthquake characteristics will be dependent on the soil profile and soil characteristics. If there is a substantial layer of soft clays or soils overlying the stronger soils, then the bias introduced by the sea floor reference can be very large (1.5 to 2.0).

The input of vertical energy for pile supported offshore platforms occurs at the point along the pile where the soil - pile shaft



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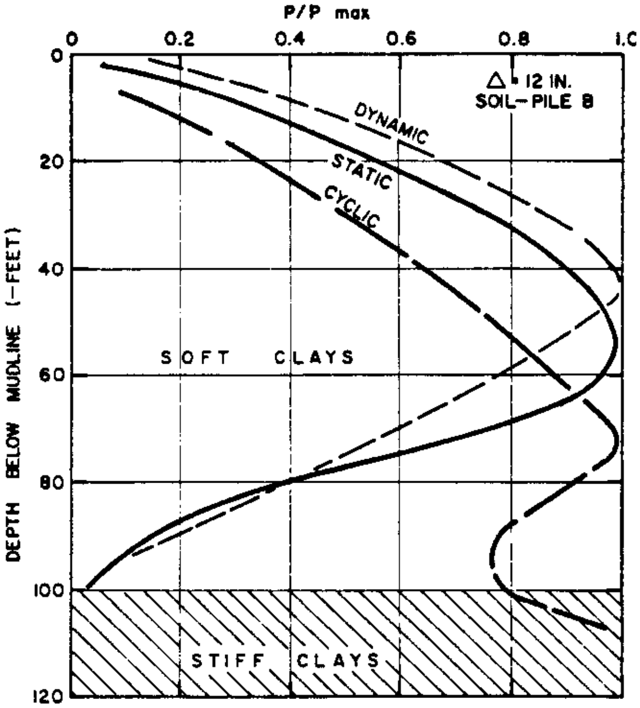


Fig. 1.3.2 - Earthquake Effects on Pile Foundation Lateral Soil Pressures

shears are maximized. This is generally along the bottom one-third of the pile shaft. This can be a depth below the sea floor of 100 m or more. Because most attenuation relations are developed for onshore conditions and recorded earthquakes where there is a dramatic change in the impedance ratio at the earth surface, and hence large reflections of the vertically propagating compression earth waves, these attenuation relations incorporate these reflections and shallow effects. Offshore, at the sea floor, there is no dramatic change in impedance for vertically propagating compression waves at the sea floor (saturated sediments interface with the water column). Thus, attenuation relations for onshore conditions generally tend to dramatically overestimate the vertical motions at the sea floor. This can be important for mat-supported offshore structures. It is also important for pile supported platforms whose vertical motion characteristics are based on measured data from onshore surface locations.

Chapter 2

Earthquake Hazard Zones & Uncertainties

2.0 Introduction

During this study, access was obtained to recent earthquake hazard exposure studies from the following regions:

- UK Sector North Sea
- Norwegian Sector North Sea
- Japan
- Indonesia
- Mexico

This Chapter summarizes the results from these studies as they pertained to the characterization of the seismic zones, the design SLE horizontal ground accelerations, and the uncertainty associated with the seismic environments.

The author contacted three firms known internationally for their expertise in seismic exposure assessments: EQE International, Dames & Moore Consultants, and Geomatrix Consultants. The author was not able to obtain any significant information that could be used to revise the current ISO seismic zonation maps. Uniformly, the firms felt that this information was confidential since it had been developed for clients.

2.1 UK Sector North Sea

Based on the study performed by EQE International (1998), Fig. 2.1.1 shows the Magnitudes and epicenters of earthquakes that have affected the North Sea area during the period from 1904 through 1990. There have been 7 earthquakes having magnitudes greater than M = 5 during this period.

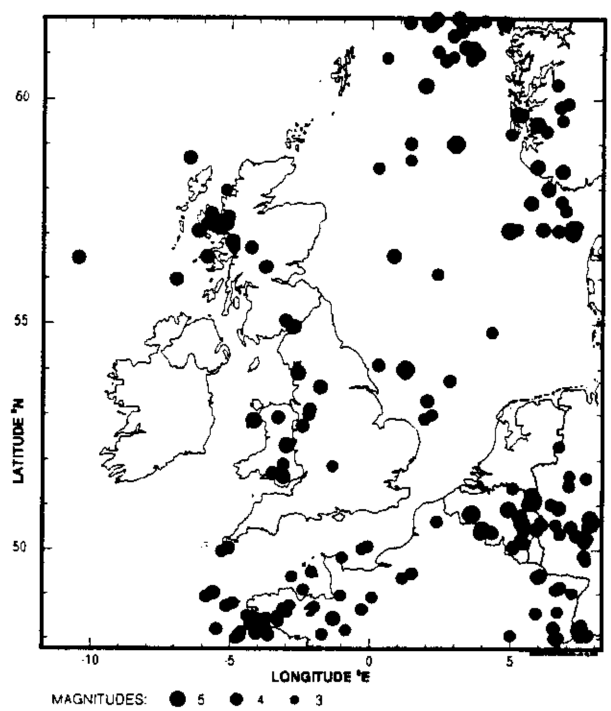


Fig. 2.1.1 - UK Sector Historic Earthquake Epicenters and Magnitudes

Based on the seismotectonics of this region, EQE developed two zonation models, one with 38 area zones and the second with 26 area zones. The historic seismicity was associated with these zonation models, an attenuation characterization based on an synthesis of attenuation models appropriate for the seismotectonic characteristics of this area, and peak ground accelerations (PGA)

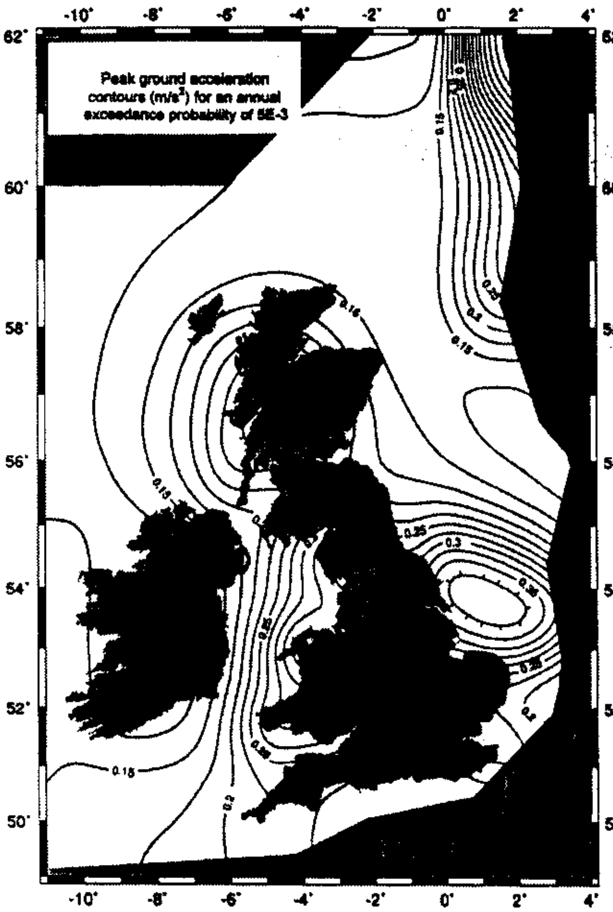


Fig. 2.1.2 - 200-Year Peak Ground Acceleration Contours

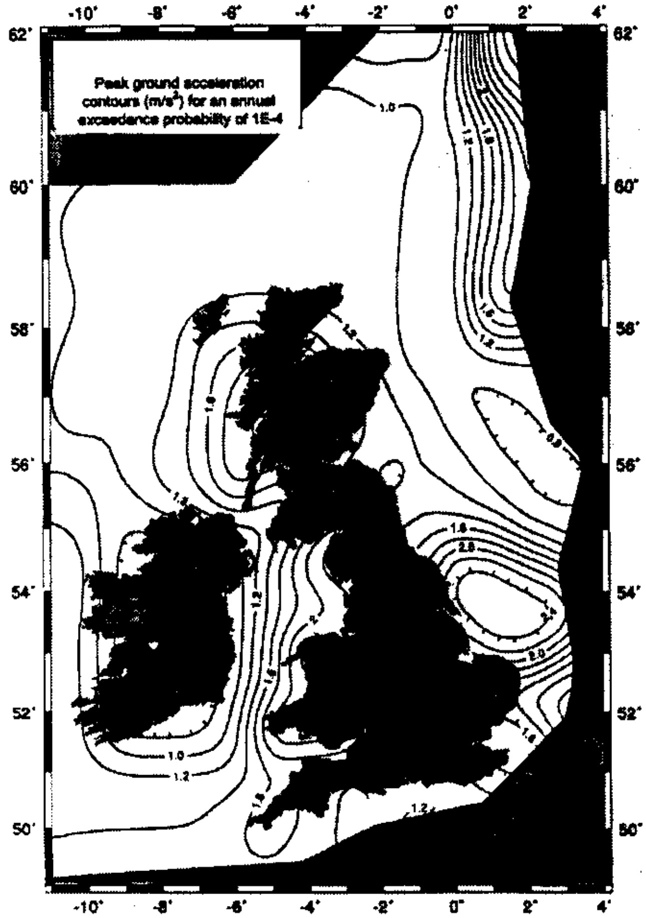


Fig. 2.1.3 - 10,000 Year Peak Ground Acceleration Contours

on rock or very hard soil determined. The results are summarized in Fig. 2.1.2 and Fig. 2.1.3 for return periods of 200 years and 10,000 years, respectively. Although not explicitly stated in this report, it is presumed that these return periods incorporate the uncertainties associated with the attenuation relationships. To be consistent with the definition of earthquake uncertainties used in the proposed ISO earthquake guidelines, the uncertainty associated with the attenuation relationships need to be incorporated into the seismic hazard characterizations.

Fig. 2.1.2 could be used as a basis for an ISO micro-zonation map of the UK Sector of the North Sea. The PGA generally range from 0.015 g to 0.030 g. The current ISO guidelines for this region indicate a seismic zone of 1 with PGA = 0.05 g to 0.15 g. The lower end of this range provides a conservative estimate of the Strength Level Earthquake (SLE) conditions for this region.

Fig. 2.1.4 summarizes the results from the EQE study in the form of the horizontal Peak Ground Accelerations (PGA) versus the average return periods for two locations; one in the southern portion of the UK Sector (54 degrees north) and one in the northern portion (58 degrees north). These results can be used to determine the uncertainties associated with the earthquake ground motions. In the proposed ISO guidelines, the earthquake uncertainties in annual expected PGA are based on a Lognormal distribution that is fitted to the extreme values and expressed with the standard deviation of the logarithms of the expected annual maximum PGA's, σ_E. If it were desirable to base the fitted

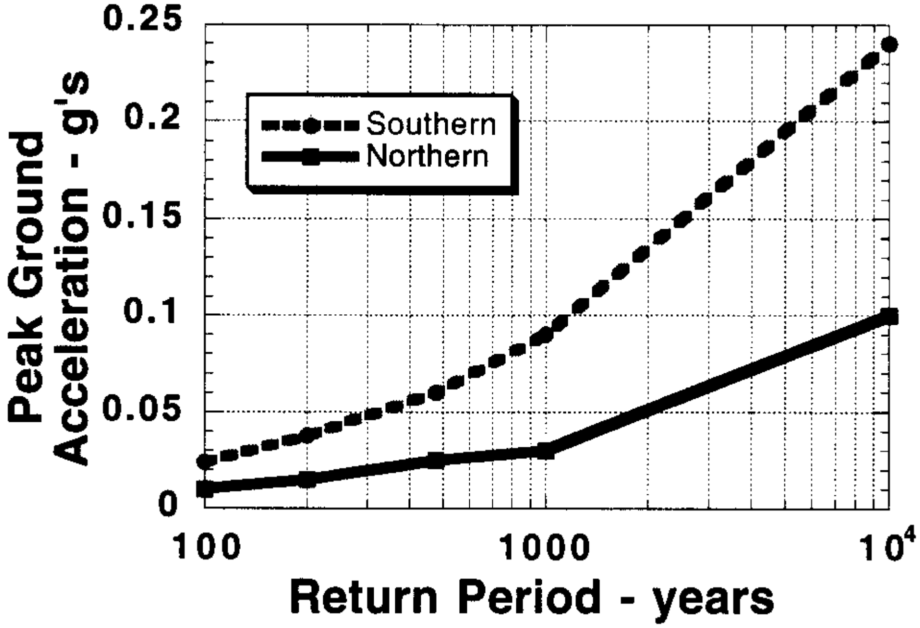


Fig. 2.1.4 - Peak Ground Accelerations for Southern and Northern Locations

Lognormal distribution on the 10,000 year and 100 year return period PGA's, the uncertainty could be found from:

$$\sigma_{SE'} = 0.72 \ln (G_{10{,}000\ yr} / G_{100\ yr})$$

For the southern location,

$$\sigma_{SE'} = 0.72 \ln (0.24\ g / 0.024\ g) = 1.66$$

For the northern location

$$\sigma_{SE'} = 0.72 \ln (0.10\ g / 0.010\ g) = 1.66$$

The proposed ISO guidelines suggest a value of $\sigma_{E'} = 2.0$ for the intraplate seismotectonic characteristics of this region. Again, the proposed ISO guidelines provide a conservative estimate of the seismic exposure uncertainties.

If it were desirable to base the characterization of the seismic exposure uncertainty on the 10,000 year and 1,000 year PGA's, then:

$$\sigma_{SE'} = 1.61 \ln (G_{10{,}000\ yr} / G_{1{,}000\ yr})$$

Based on the results developed by EQE for the southern location:

$$\sigma_{SE'} = 1.61 \ln (0.24 / 0.085) = 1.67$$

and for the northern location

$$\sigma_{SE'} = 1.61 \ln (0.10 / 0.035) = 1.69$$

The seismic uncertainties are not sensitive to the extreme earthquake return periods used to fit the equivalent Lognormal distributions.

2.2 Norwegian Sector North Sea

Based on the study performed by NORSAR and the Norwegian Geotechnical Institute (1998), Fig. 2.2.1 summarizes the historic earthquake epicenters and magnitudes. The zonations used in the seismic exposure analyses are also shown. The NORSAR study shows that these earthquake epicenters are closely associated with faulting in the region. Study of recent earthquake records shows that the majority of the earthquakes originate on reverse thrust faults characteristic of intraplate seismotectonic regions.

The historic seismicity developed by NORSAR was associated with the zonations indicated in Fig. 2.2.1, an attenuation characterization based on an synthesis of attenuation models appropriate for the seismotectonic characteristics of this area (calibrated with recordings from recent earthquakes), and peak ground accelerations (PGA) on rock determined. The results are summarized in Fig. 2.2.2 and Fig. 2.2.3 for return periods of 100 years and 10,000 years, respectively.

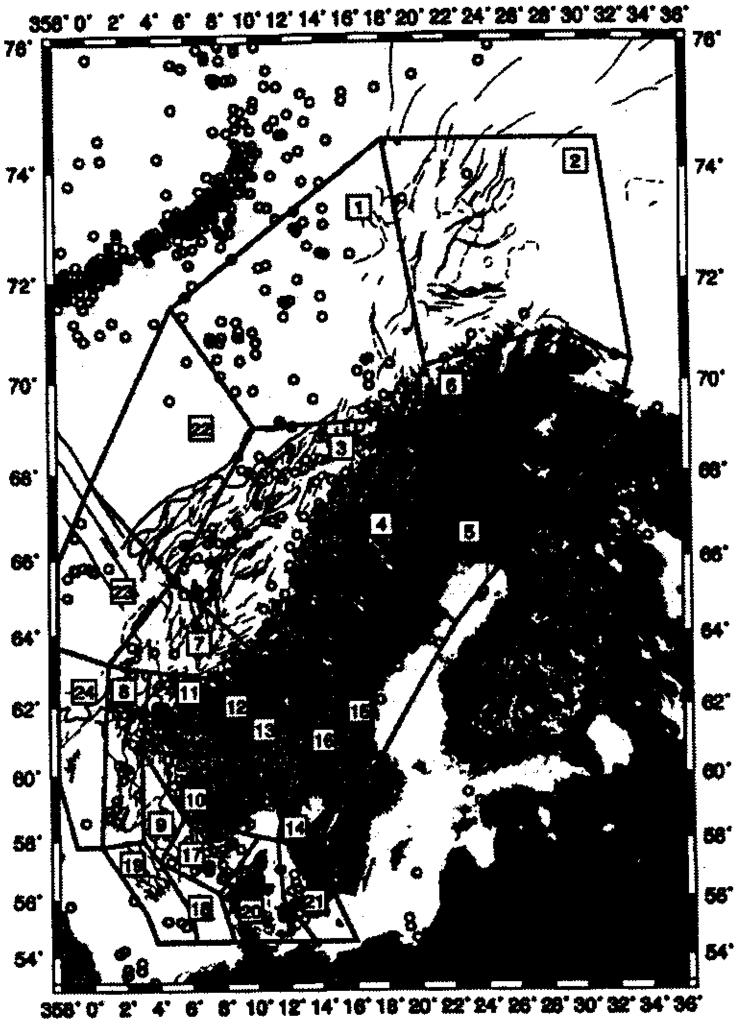


Fig. 2.2.1 - Norwegian Sector Earthquake Epicenters, Magnitudes, and Seismic Zonation

Site specific studies performed to determine the 100-year return period earthquake peak ground acceleration can be converted to the 200-year PGA (= G) with following relationship:

$$G_{200} = G_{100} \exp (0.24\ \sigma_{SE})$$

Fig. 2.2.2 could be used as a basis for an ISO micro-zonation map of the UK Sector of the North Sea. The 200-year PGA generally range from 0.015 g to 0.08 g. The current ISO guidelines for this region indicate a seismic zone of 1 with PGA = 0.05 g to 0.15 g. The lower end of this range provides a conservative estimate of the Strength Level Earthquake (SLE) conditions for this region.

Fig. 2.2.4 summarizes the results from the NORSAR study in the form of the horizontal Peak Ground Accelerations (PGA) versus the average return periods for two locations; one in the southern portion of the Norwegian Sector (62 degrees north) and one in the northern portion (66 degrees north). Based on the fitted Lognormal distribution to the 10,000 year and 100 year return period PGA's, the uncertainty for the southern location is:

$$\sigma_{SE'} = 0.72 \ln (0.33 \text{ g} / 0.042 \text{ g}) = 1.48$$

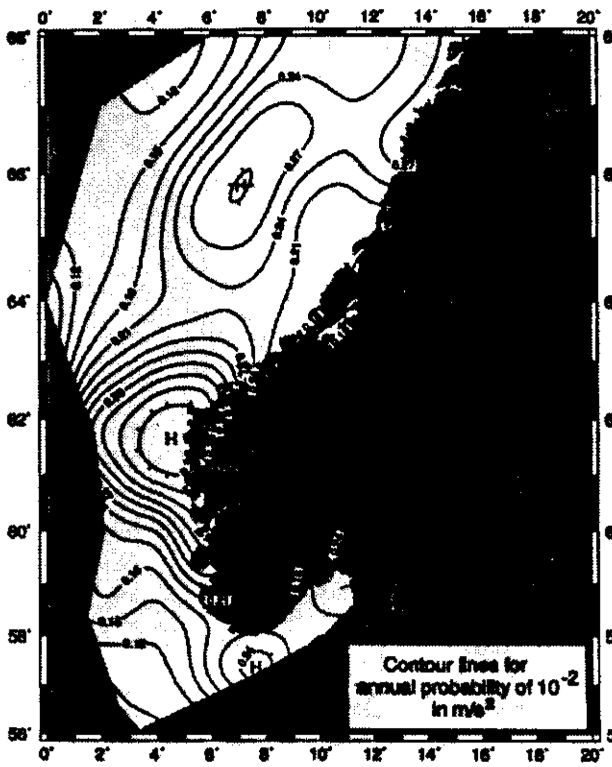


Fig. 2.2.2 - 100-Year Peak Ground Acceleration Contours

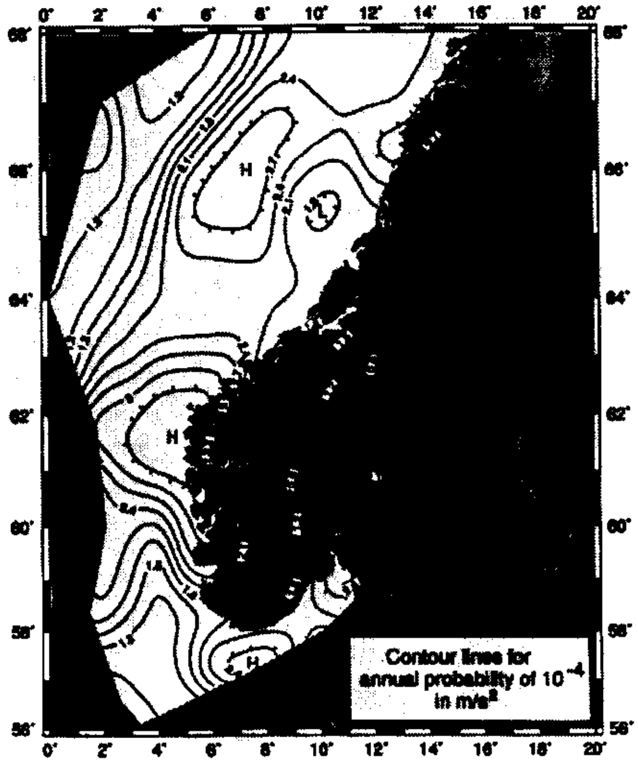


Fig. 2.2.3 - 10,000 Year Peak Ground Acceleration Contours

For the northern location

$$\sigma_{SE'} = 0.72 \ln (0.27 \text{ g} / 0.027 \text{ g}) = 1.66$$

The proposed ISO guidelines suggest a value of $\sigma_{E'} = 2.0$ for the intraplate seismotectonic characteristics of this region. This value provides a conservative estimate for this region.

Basing the characterization of the seismic exposure uncertainty on the 10,000 year and 1,000 year PGA's, then for the southern location:

$$\sigma_{SE'} = 1.61 \ln (0.33 / 0.13) = 1.49$$

and for the northern location

$$\sigma_{SE'} = 1.61 \ln (0.27 / 0.10) = 1.60$$

These values again are in excellent agreement with those based on the 100-year and 10,000-year PGA.

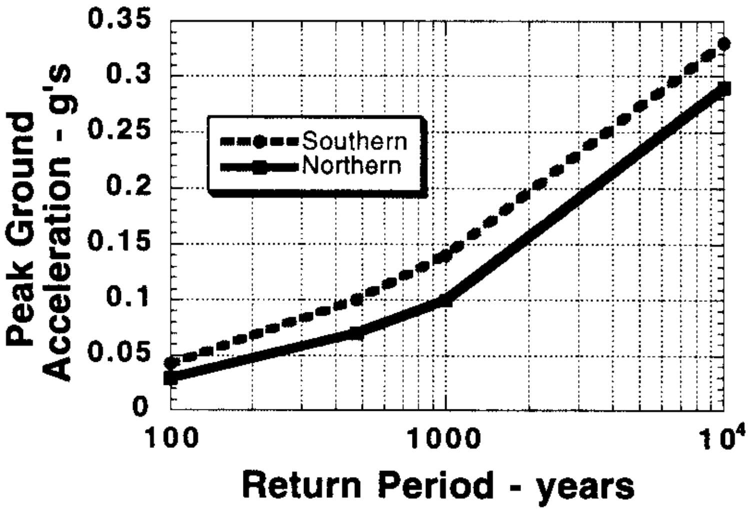


Fig. 2.2.4 - Peak Ground Accelerations for Southern and Northern Locations

2.3 Japan

Fig. 2.3.1 summarizes the earthquake epicenters and magnitudes in the region surrounding Japan during the period 1885 through 1995 (Utsu, 1982). The data set includes more than 500 seismic events having magnitudes greater than M -= 6.0. Attenuation relations for peak bedrock accelerations were based on results from a recent study of recorded earthquake motions in Japan (Nozu et al. 1997). The standard deviation of the attenuation relationship was found to be $\sigma_{\ln PGA}$ = 0.62. This value characterizes the uncertainty associated with the attenuation relationship.

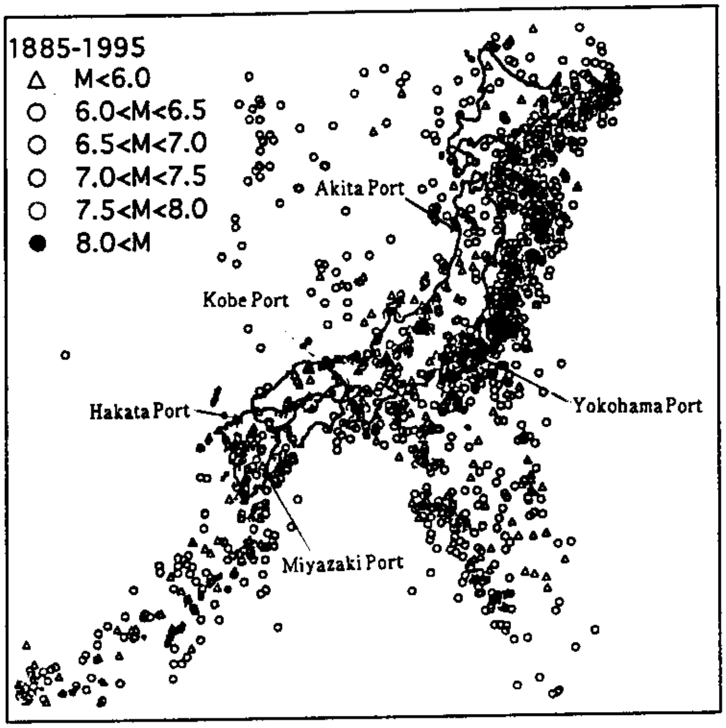


Fig. 2.3.1 - Seismic Event Data Used in Japan Earthquake Hazard Study

Fig. 2.3.2 summarizes the results of the seismic exposure study for the Yokohama Port (Nozu, Iai 1997). The 20 largest peak bedrock accelerations are shown as a function of the return period. The data are fitted with a Weibull distribution and with a Lognormal distribution that is fitted to the tail of the Weibull distribution. Similar fits were developed for a large number of coastal - offshore sites and similar results developed.

Based on these results, a seismic hazard map for the peak bedrock acceleration having a 200 year return period was developed (Fig. 2.3.3). The PGA range from 0.05 g to 0.50 g. The proposed ISO seismic zonation for this area ranges from 2 to 5 and thus indicate PGA = 0.15 to 0.25 g to PGA = 0.45 g to 0.55 g. The proposed ISO guidelines provide a conservative estimate of the 200-year PGA. An ISO Japan annex could be developed that would utilize the information shown in Fig. 2.3.3 as the micro zonation map for coastal - offshore Japan.

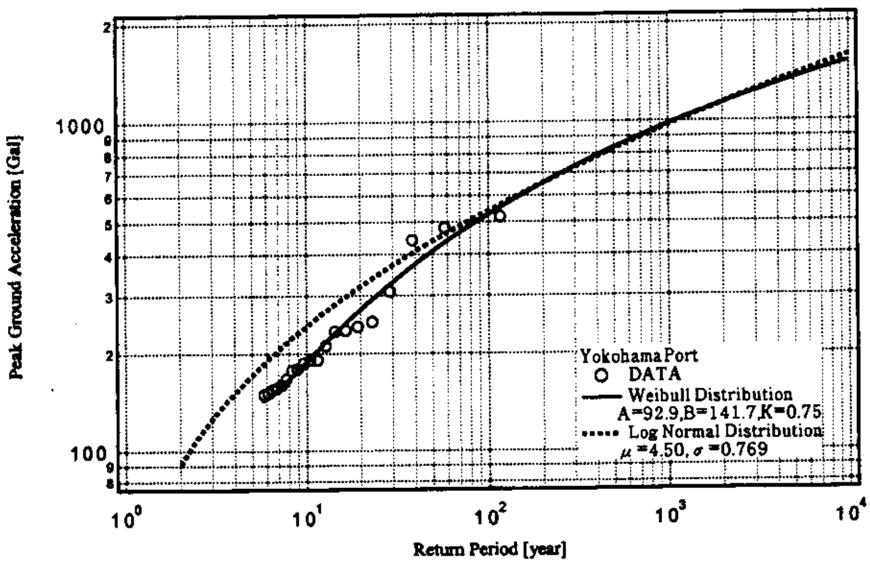


Fig. 2.3.2 - Peak Bedrock Acceleration and Return Period at Yokohama Port

This study also developed a mapping of the standard deviation of the Lognormal peak bedrock acceleration distribution that considered both the variability in the Lognormal distribution for the expected PGA and the uncertainty in the attenuation relationship (Fig. 2.3.4). The results indicated standard

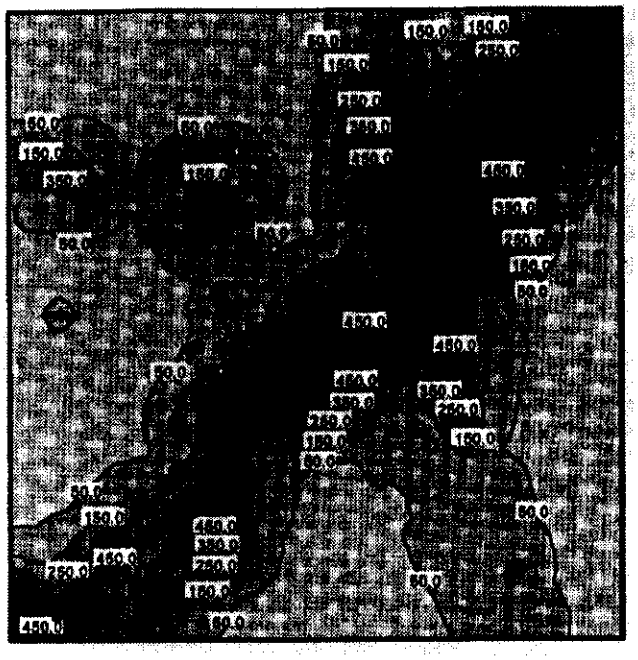


Fig. 2.3.3 - 200 Year Bedrock PGA

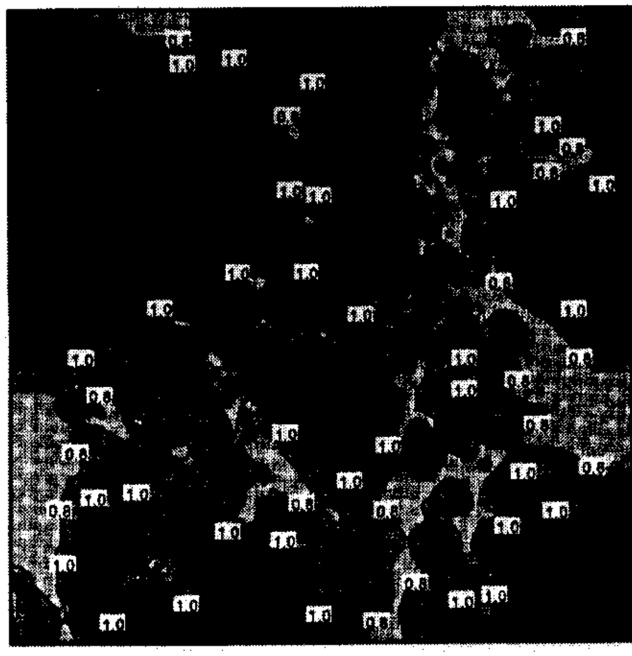


Fig. 2.3.4 - Standard Deviation of Lognormal Expected Annual Maximum PGA

deviations of the Lognormal peak bedrock accelerations distributions that ranged from $\sigma_{SE} = 0.8$ to $\sigma_{SE} = 1.0$.

The value suggested in the proposed ISO guidelines for the seismotectonic conditions in this region is $\sigma_{SE} = 1.0$. Thus, the proposed ISO guidelines provide a conservative estimate of the uncertainties associated with the seismic exposure of this region.

2.4 Indonesia

The extreme condition earthquakes offshore Indonesia are dominated by two principal seismic sources (Beca Carter Hollings & Ferner Ltd. 1979; Dames & Moore 1992; Untung et al 1985). The major tectonic feature in the region is the Sunda Arc which extends approximately 5,600 km between the Andaman Islands in the northwest and the Banda Arc in the east. This feature results from convergence and subduction of the Indo-Australian plate beneath the Southeast Asia plate.

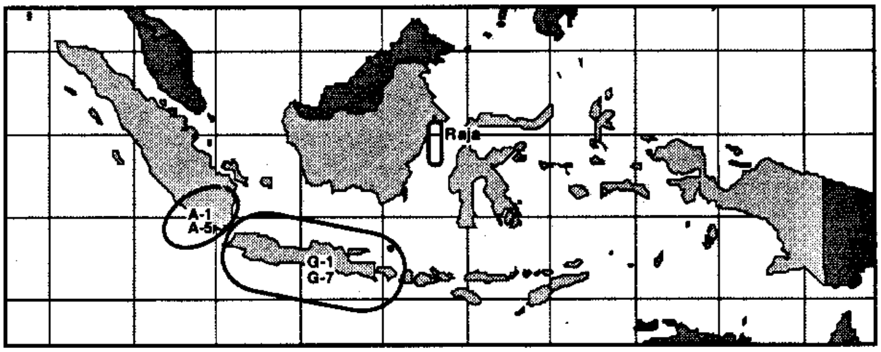


Figure 2.4.1 - Locations Offshore Indonesia

The Sunda Arc consists of three primary segments: the Sumatra segment (A-1 - A-5, Figure 2.4.1), the Sunda Strait segment (between A-1 - A-5 and G-1 - G-7, Figure 2.4.1), and the Java segment (G-1 - G-7, Figure 2.4.1) (Dames & Moore 1992). The locations A-1 to G-1 lie along the northern edge of the Sunda Arc. The locations A-5 to G-7 lie along the axis of the Sunda Arc. These locations represent the area of greatest seismic exposure.

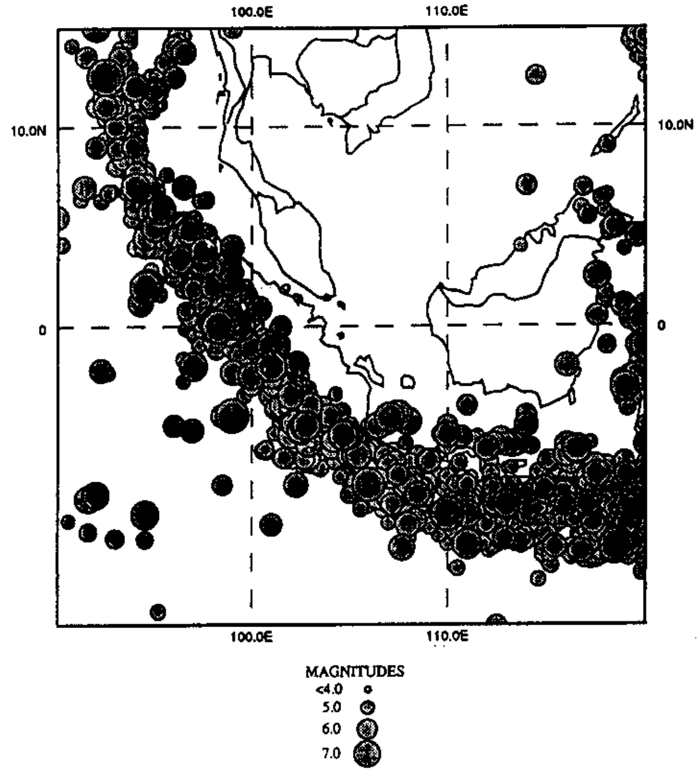


Figure 2.4.2 - Epicenters of Indonesian Earthquakes 1900 - 1996

There have been several great (Magnitude in excess of 7.75) earthquakes recorded along the Sunda Arc during this century (Figure 2.4.2). More than 150 surface Magnitude 7.0 or greater earthquakes have occurred along this Arc during this century (Beca Carter Hollings & Ferner Ltd. 1979). The majority of these earthquakes have had focal depths in excess of 70 km (Figure 2.4.3).

The second primary source of earthquakes are shallow crustal faults that generally parallel the Sunda Arc (east - west) with some transform surface faulting that marks the north - south trending boundaries of the land masses (Figure 2.4.3). The majority of recorded shallow source (less than 70 km) earthquakes that have occurred during this century have occurred along the Sunda Arc zone. Several hundred shallow focus earthquakes in the Magnitude range of 4.0 to 7.0 have occurred in this zone during this century. In the vicinity of the area indicated as 'Raja' (Dames and Moore 1990; Lamport 1992; Risk Engineering Inc 1992) in Fig. 2.4.1, there have been in excess of 12 events with magnitudes between 5.0 and 7.0 during the past century (Figure 2.4.4).

Figure 2.4.5 summarizes a proposed seismic zonation for offshore Indonesia (Bea, 1997b). The zone designations are associated with the 200-year peak ground accelerations at the sites as follows:

Zone 1 G = 0.05g to 0.15 g

Zone 2 G = 0.15g to 0.25g

Zone 3 G = 0.25g to 0.35 g

These seismic zonations are consistent with those contained in the proposed ISO seismic guidelines.

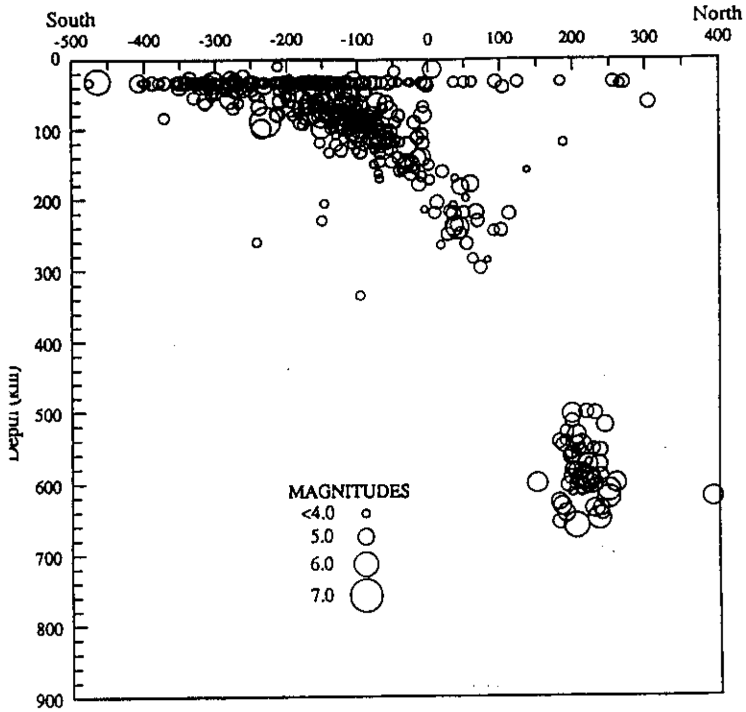


Figure 2.4.3 - Hypocentral Profile of Java Central Segment Normal to Sunda Arc - Earthquakes 1900 - 1996

In some areas, site specific studies have been performed to determine the 100-year return period earthquake peak ground acceleration. To convert the 100-year G to the 200-year G, the following relationship can be used:

$$G_{200} = G_{100} \exp(0.24\ \sigma_{SE})$$

where σ_{SE} is the uncertainty in the long-term distribution of G (assumed to be Lognormally distributed).

Based on the results from the recent studies of extreme earthquake conditions that have been performed in this area, Figure 2.4.6 summarizes the expected annual maximum peak ground accelerations (Gm) for the locations shown in Figure 2.4.1. ***The expected peak ground accelerations having return periods of 100 years at the locations ranges from 0.10 g to 0.12g (G1, A1) to 0.17 g to 0.22 g (G7 - A5).***

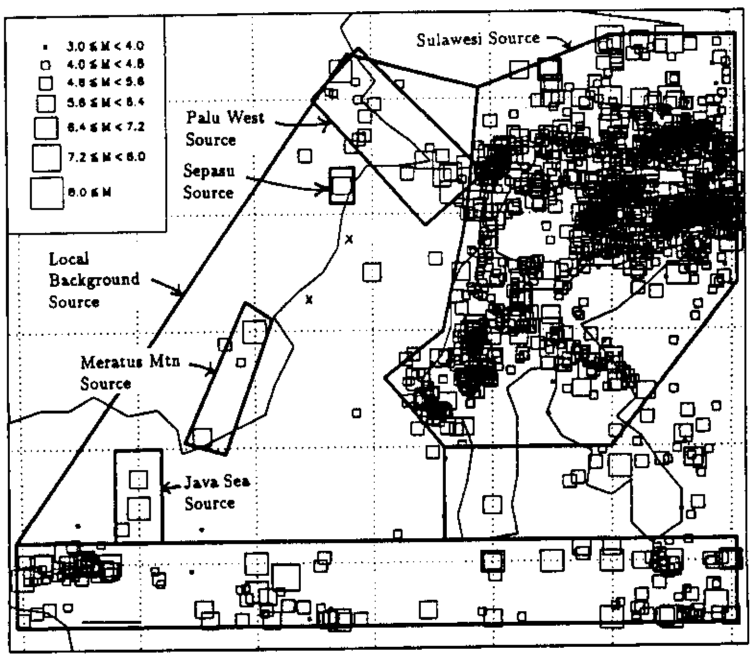


Figure 2.4.4 - Earthquake Sources and Magnitudes in Kalimantan Region

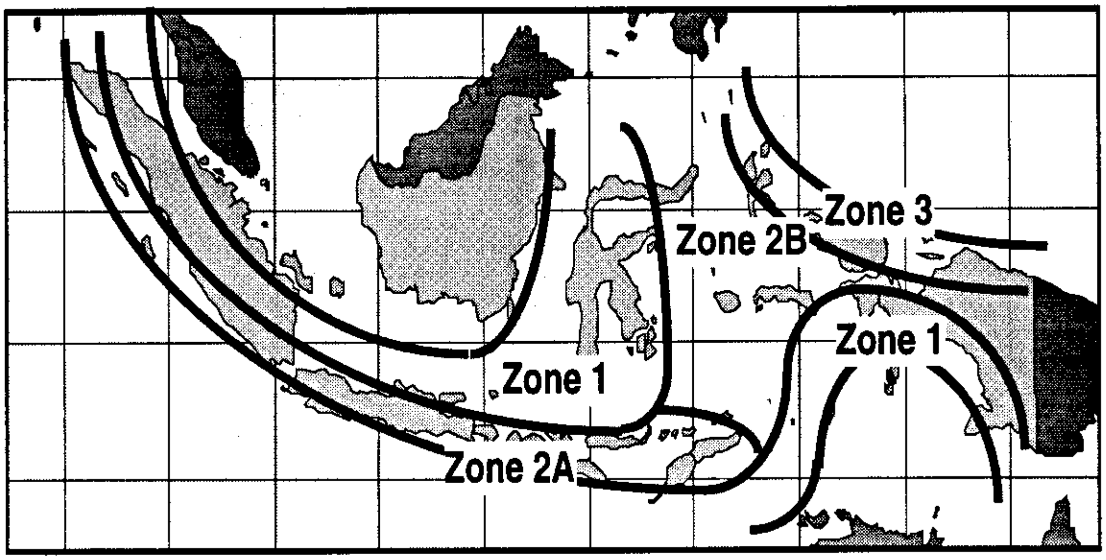


Figure 2.4.5 - Seismic Zonation for Offshore Indonesia

The uncertainties (natural variabilities) in the expected annual earthquake peak ground accelerations (Gm) range from σ_{SE} = 0.60 to 0.70 (A-1, A-5, G-1, G-5) to σ_{SE} = 1.8 to 2.0 (Raja). These values are consistent with the proposed ISO guidelines for this region and seismotectonic conditions.

The available regional and site specific information on earthquake conditions indicates that there are two general categories of Type I uncertainties associated with the primary seismic sources. The first category is associated with those locations that are adjacent to the Sunda Arc. These locations very frequently experience earthquakes from both shallow and deep sources (Fig. 2.4.3). Those locations have Type I uncertainties in the expected annual earthquake peak ground accelerations that range from σ_{SE} = 0.60 to 0.70.

The second category of locations is associated with those locations that are distant from the Sunda Arc. These locations much more infrequently experience earthquakes primarily from shallow sources. These locations have Type I uncertainties in the expected annual earthquake peak ground accelerations that range from σ_{SE} = 1.80 to 2.00 (Raja).

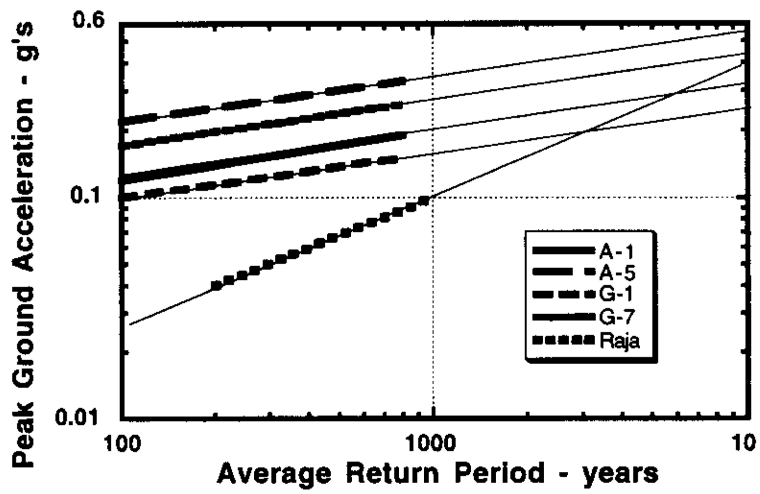


Figure 2.4.6 - Expected Annual Maximum Earthquake Peak Ground Accelerations at Study Locations

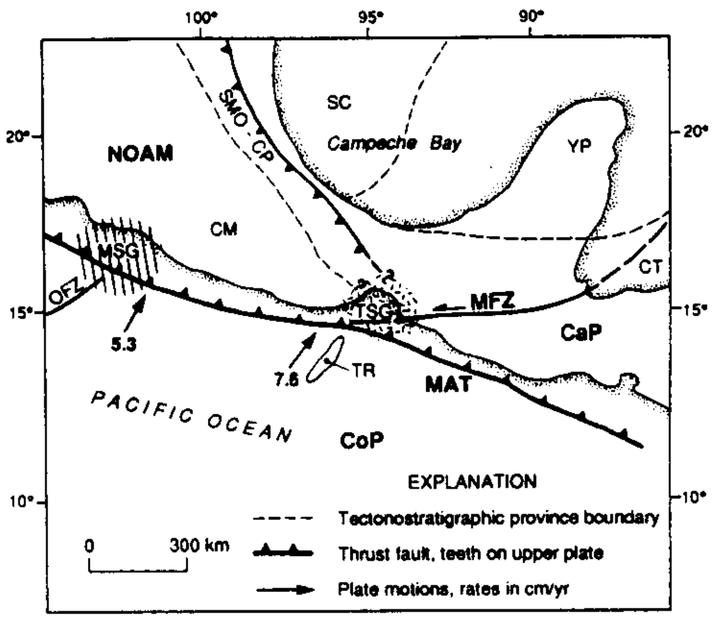
2.5 Bay of Campeche, Mexico

The information on earthquake seismic hazard characteristics for the Bay of Campeche, Mexico, was provided by Chavez (1987, 1997a, 1997b, 1998). Additional information on the earthquake characterization was provided by Guerra and Esteva (1978), and Guzman (1982).

The seismic environment is influenced by three primary types of earthquake sources (Figure 2.5.1). The Type 1 source is associated with the subduction zone on the western Pacific coast of Mexico. The earthquakes in this zone occur at depths of 15 km to 20 Km and have magnitudes up to M = 8. 2. The Type 2 source is associated with the lithospheric slab within the central portion of Mexico. Earthquakes occur in this zone at depths of 60 km to 250 km and have magnitudes up to M = 7.5. The Type 3 source occur in the Trans-Mexican volcanic belt located primarily along the east coast of Mexico, have depths up to 20 km, and magnitudes up to M = 6.7 (Chavez 1987; 1998).

The historic catalog of earthquake occurrences during the period 1970 - 1996 were used by Chavez to determine the activity rates in each of the three sources and the occurrences of varying earthquake intensities and magnitudes in each of the three sources. Figure 2.5.2 shows the seismic events that have occurred in this region during the period 1800 to 1996 for all earthquakes that have surface magnitudes (Ms) greater than M = 5.0. The majority of records contained in this catalog date from the 1930's when reasonably comprehensive teleseismic instrumentation was in place.

The largest earthquakes that have occurred in the vicinity of the Bay of Campeche are in the range of M = 3.0 to M = 4.0. The largest earthquake that has



CM = Chiapas Massif, CaP = Caribbean Plate, CoP = Cocos Plate, CT = Cayman Trough, MAT = Middle Atlantic Trench, MFZ = Motagua Fault Zone, MSG = Michoacan Seismic Gap, NOAM = North American Plate, OFZ = Orozco Fracture Zone and ridges, SC = Sigsbee-Campeche province, SMO-CP = Sierra Madre Oriental-Chipas Petén province, TR = Tehuantepec Ridge, TSG = Tehuantepec Seismic Gap

Figure 2.5.1 - Bay of Campeche Earthquake Plate Tectonics and Sources

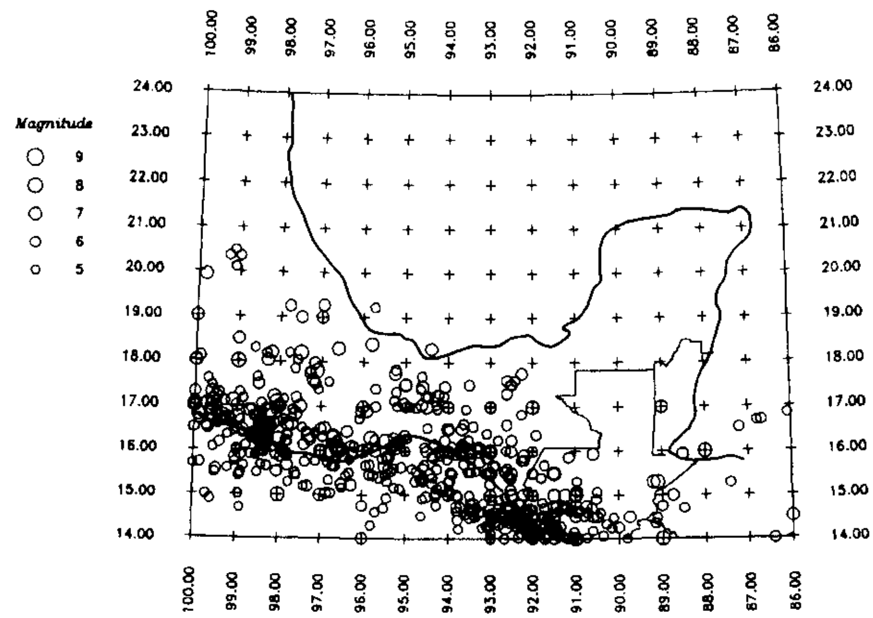


Figure 2.5.2 - Seismic Events Affecting the Bay of Campeche Region 1800-1996, Events M > 5.0

occurred during the past 100 years within a 100 km radius of the Bay of Campeche is M = 6.0. The vast majority of the seismic activity is concentrated along the Pacific coast of Mexico. The source of nearby large earthquakes are associated with the Motagua Fault Zone and the fault system (Sierra Madre Oriental-Chipas Peten province) that generally parallels the east coast of Mexico. There are no major seismic sources that have been identified within the Sigsbee-Campeche province that underlies the Bay of Campeche.

Based on previous analyses of pile supported platforms subjected to earthquake excitations in soil columns of the Bay of Campeche, a depth below the sea floor of -12 m was chosen to reference the lateral accelerations and a depth of -115 m was chosen to reference the vertical accelerations from the seismic exposure analyses (Bea, 1997c).

Figure 2.5.3 summarizes the results for the Bay of Campeche for each of the three seismic sources. A mean or average value for the four representative soil columns studied by Chavez (1997a) are shown. The Type 2 and Type 3 sources develops ground motions that dominate the seismic exposure in the Bay of Campeche. At an average return period of 10,000 years, the indicated peak horizontal ground acceleration (Am) is indicated to be Am ≈ 0.25 g. Table 2.5.1 summarizes the ground motions at return periods of 200 and 4000 years and the uncertainties (natural or Type 1, aleatory) associated with the ground motions (expressed as the standard deviation of the logarithms of expected annual G, σ_{ISE}).

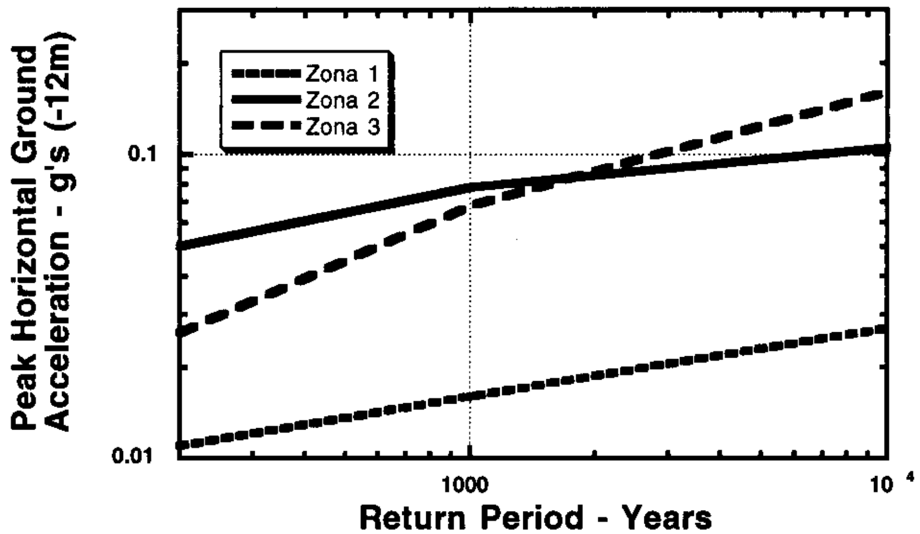


Figure 2.5.3 - Peak horizontal ground accelerations

Table 2.5.1 - Mean Horizontal PGA at Bay of Campeche sites (-12m) and uncertainties of peak horizontal ground accelerations

Type	200 years - %g	1,000 years - %g	Uncertainty σ_{SE}
1	1.2	1.6	0.58
2	5.0	7.8	0.84
3	2.7	6.8	1.75

These results can be compared with those developed by Guerra and Esteva (1978) for the Bay of Campeche and firm alluvium sites. At a return period of 100 years, the peak horizontal ground acceleration obtained by Guerra and Esteva is G = 6.6 % g. This compares with a maximum value of G = 5.0 % g obtained by Chavez (1998) for a 100 year return period for the Zone 2 sources. The results developed by Guerra and Esteva indicate an uncertainty in the annual expected peak horizontal ground acceleration of $\sigma_{SE} = 1.26$.

The extensive Bay of Campeche seismic exposure study performed by Guzman (1982) provides some additional information on the expected ground motions. At a return period of 100 years, the peak horizontal ground acceleration obtained by Guzman is G = 10 % g. This ground motion is more intense than developed by either Chavez or Guerra and Esteva. The more intense ground motion appears to be coupled with the attenuation relationships that were developed and used by Guzman (1982). The results developed by Guzman indicate an uncertainty in the annual expected peak horizontal ground acceleration of $\sigma_{SE} = 0.97$. This uncertainty is slightly greater than that of the Chavez results and somewhat less than indicated by the Guerra and Esteva results.

The proposed ISO guidelines for this region indicate a 200-year PGA in the range of 0.05g to 0.15g. The lower end of this range provides a good estimate of the seismic characteristics in the Bay of Campeche. The uncertainties in the annual maximum PGA are consistent with those given in the proposed ISO guidelines.

The Type 1 and Type 2 sources have uncertainties in the expected annual maximum Am that are comparable with that of offshore California/ The Type 3 (coastal - local) sources have an uncertainty that is comparable with 'intra-plate' seismic zones. Based on the results provided by Chavez (1997a), Table 2.5.2 summarizes the mean vertical peak ground accelerations at a depth below the sea floor of -115 m together with the associated uncertainties.

Table 2.5.2 - Mean Vertical PGA at Bay of Campeche sites (-115m) and uncertainties of peak horizontal ground accelerations

Type	200 years - %g	1,000 years - %g	Uncertainty σ_{SE}
1	0.2	0.3	0.75
2	0.9	1.2	0.55
3	0.2	0.3	0.75

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Chapter 3

Strength Level Earthquake Response Spectra

3.0 Introduction

Strength Level Earthquake (SLE) elastic response spectra were assessed in the several of the studies of regional seismicity discussed in Chapter 2. The SLE response spectra developed for each of these regions will be summarized in this Chapter. In addition, results from recent general studies of response spectra will be summarized, including consideration of water column effects on vertical motions.

3.1 Response Spectra Ordinates

Recent studies of response spectra ordinates in the low period range (0.1 sec to 0.3 sec) have indicated that one should expect that these ordinates will vary as a function of both the local soil conditions (soil column classification)and intensity of earthquake motions (Martin, Dobry 1994; Crouse, McGuire 1997). Figure 3.1.1 summarizes the results from the study by Crouse and McGuire of the median amplification factors (Fa = peak horizontal ground acceleration at surface / peak horizontal ground acceleration on rock = PGAs / PGAr) for periods (T) in the range of T = 0.1 sec to 0.3 sec.

The site soil conditions indicated as SC-A, B, C, and D correspond to those in the proposed ISO guidelines. The amplification factor in this period range presently incorporated into the proposed ISO guidelines is Fa = 2.5 for all soil conditions. These results indicate substantially larger amplifications in the short period range. While these findings would relatively unimportant for the lateral motions of most offshore platforms (T = 2 to 3 sec), the effects would be potentially important for the vertical motions (T = 0.1 to 0.3 sec) of these structures (particularly for heavily loaded deck structures with large cantilevers). The National Center for Earthquake Engineering Research (NCEER) workshop defined a comparable range of amplification factors for this period range (Martin, Dobry 1994).

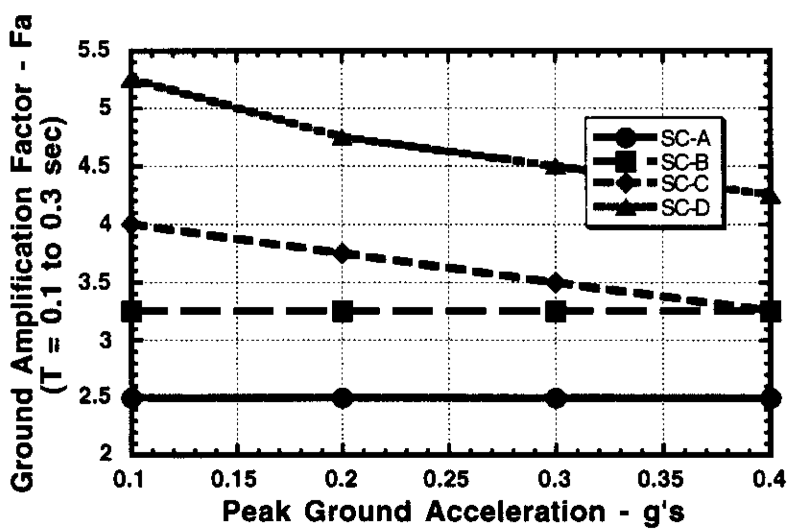

Fig. 3.1.1 - Amplification factors as function of acceleration level and soil conditions

As a result of these findings, the following modifications to the proposed ISO earthquake guidelines are proposed. The elastic design response spectra (Figure 3.1.2) would be a function of the following variables:

- T = period of system
- D = damping of system
- ν = site classification factor for short period range (new factor)
- ψ = site classification factor for long period range
- ε = seismotectonic conditions

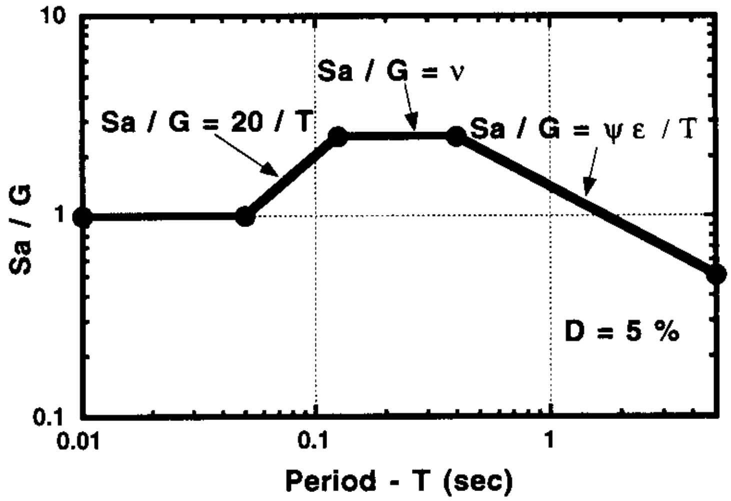

Fig. 3.1.2 - Proposed elastic design response spectra

Table 3.1.1 summarizes the elastic design response spectra characteristics based on the results of the work by Crouse and McGuire (1996) and the previous work incorporated into the proposed ISO design response spectra.

Table 3.1.1 - Local geology and soil conditions parameters for SLE response spectra

Soil Conditions			Seismotectonic Conditions	
Site Class	ν	ψ		ε
SC-A	2.5	1.0	Type A	1.0
SC-B	3.3	1.2	Type B	0.8
SC-C	3.5	1.4	Type C	0.9
SC-D	4.5	2.0	Type D	0.8
SC-E	Site specific studies required		Default value	1.0

3.2 Vertical Response Spectra Ordinates

Several recent large magnitude earthquakes have indicated that vertical ground motions can be equal to or greater than the horizontal ground motions. This observation has been particularly true for ground motions close to the epicenter of large magnitude earthquakes. The present ISO guidelines prescribe an elastic design response spectrum for vertical motions that is one-half of the horizontal motions.

Figure 3.2.1 summarizes results from a recent study of the ratios of vertical to horizontal spectral ordinates as a function of the response frequency (Bozorgnia, Niazi, Campbell 1995). Results for the Loma Prieta and Taiwan earthquakes are shown. The ratios indicated are mean values. The horizontal line indicated at the ratio of 2/3 references the present Uniform Building Code (UBC) specifications. For near-by earthquakes (R = 10 km), the ratio of the vertical to horizontal spectra ordinate for periods in the range of $T = 0.1$ sec are in the range of 1.4 to 1.7. The ratio is close to unity at distances of $R = 20$ km to 30 km. Comparable results have been found for other large magnitude earthquakes in other semismotectonic zones.

For periods in the range of $T = 2$ to 3 sec, the ratios are in the range of 0.2 to 0.3 for all distances. Current studies of recordings from different earthquakes generated by different types of seismotectonic environments indicates that these trends are "very likely to be universal." (Bozorgnia, Niazi, Campbell 1995). These results show the problems that are associated with specifying a constant ratio of vertical to horizontal spectra ordinates for all periods and distances. The results indicate that the horizontal and vertical spectra have decidedly different shapes.

The author's application of these results to the 'unmodified'[1] ISO response spectra is illustrated in Figure 3.2.2. The present proposed ISO horizontal and vertical response spectra for Site Classification C (firm alluvium, $\nu = 3.5$, $\psi = 1.4$) and Type A seismotectonic conditions (shallow crustal faulting, $\varepsilon = 1$). For periods greater than about 0.3 sec, the unmodified vertical response spectra have ordinates that are less than the present proposed ISO guideline vertical spectra. For periods less than 0.3 sec, the unmodified ordinates are substantially larger.

The foregoing results have been developed based on earthquake ground motion measurements that have been made on land. Measurements of ground motions offshore have indicated that the presence of the water column can have a marked influence on the sea floor motions (Smith 1997; Sleefe 1990). The measurements indicate that the vertical motions at the sea floor are much less than would expected based on on-land measurements. For mat supported structures that receive the majority of their input motions from soils that are located near the sea floor, this can be an important consideration.

For vertically incident earthquake compression waves, the ratio of the reflected to the incident wave amplitude at the ground

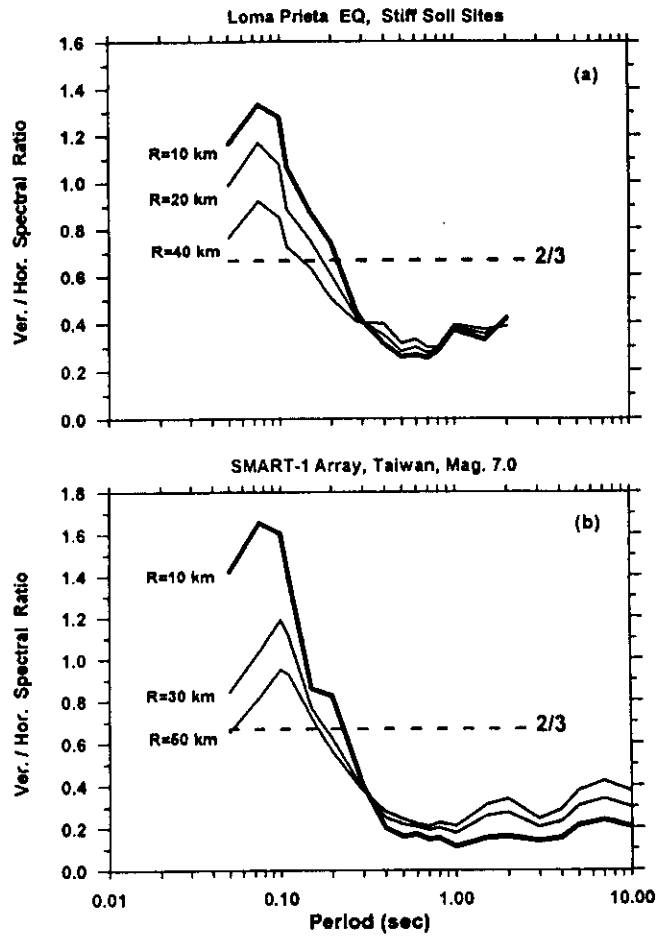

Fig. 3.2.1 - Ratio of Vertical to Horizontal Elastic Response Spectra Ordinates

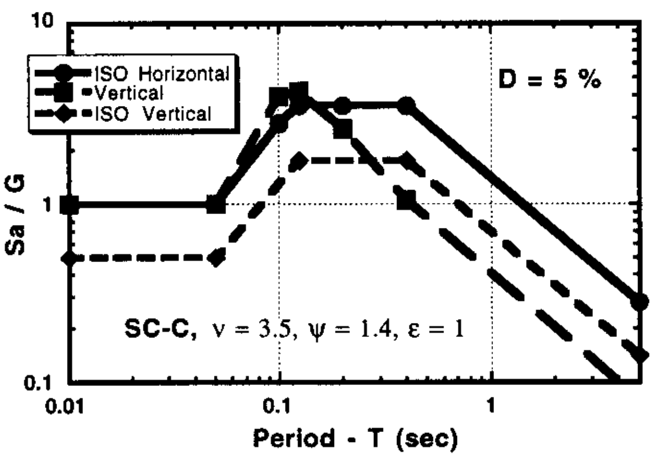

Fig. 3.2.2 - Horizontal and Vertical Elastic Response Spectra for SC-C, $\varepsilon = 1.0$

[1] unmodified for water column effects

surface is a function of the impedance ratio, IR, at the ground surface:

$$IR = \rho s \, Vs / \rho a \, Va$$

where ρ is the mass density of the soil (s) and air (a), and V is the compression wave velocity in the two media. On land, this impedance ratio is very large, and one would expect almost total reflection of the energy at the ground surface. This would indicate a doubling of the amplitude of the incident compression waves at the ground - air interface.

In water, the compression wave velocity in the water and in the water saturated sediments would be almost the same. The density difference would be a factor of about 2. This would indicate that about 50 % of the compression wave would be transmitted through the interface and the remaining 50 % would be reflected. The total wave amplitude at the interface would be 1.5 times the incident amplitude or 75 % of the amplitude expected at an air - soil interface.

This analysis assumes that the water is infinitely deep and ignores the reflection of the incident compression waves at the water - air interface. Crouse and Quilter (1991) developed an analysis that includes the finite water column effects. Depending on the water depth, some frequencies can be reinforced and amplified by the finite water depth, while others can be canceled. The analyses performed by Crouse indicates that the peak vertical accelerations at the sea floor can be reduced by as much as 50 % (Crouse, Quilter 1991; Dolan, Crouse, Quilter 1992; Smith 1997; Sleefe 1990). These observations are in agreement with the analyses of recorded earthquakes at the sea floor reported by Smith (1997) and Sleefe (1990)

The earthquake ground motions in the vicinity of the sea floor are the result of a very complex variety of wave forms, frequencies, and incidence angles. Thus, the foregoing simplified analyses can be relied upon only to provide some insights into the potential effects of the water column on the vertical movements at the sea floor. Measurements of earthquake movements at the sea floor provide the only reliable source of information to quantify these effects. The measurements that are available confirm the inferences that have been drawn based on simplified analyses.

Figure 3.2.3 summarizes the results for the water column modified vertical response spectra compared with the present ISO vertical response spectra. The present ISO vertical response spectra provides a conservative envelope to the modified vertical spectra for periods greater than about 0.2 sec. However, the present ISO vertical response spectrum is somewhat unconservative for periods less than about 0.2 sec.

Until better information becomes available, it is the author's conclusion that the present proposed ISO guidelines for vertical response spectra provide a reasonable, and likely somewhat conservative approximation to the vertical movements that can be expected in the vicinity of the sea floor.

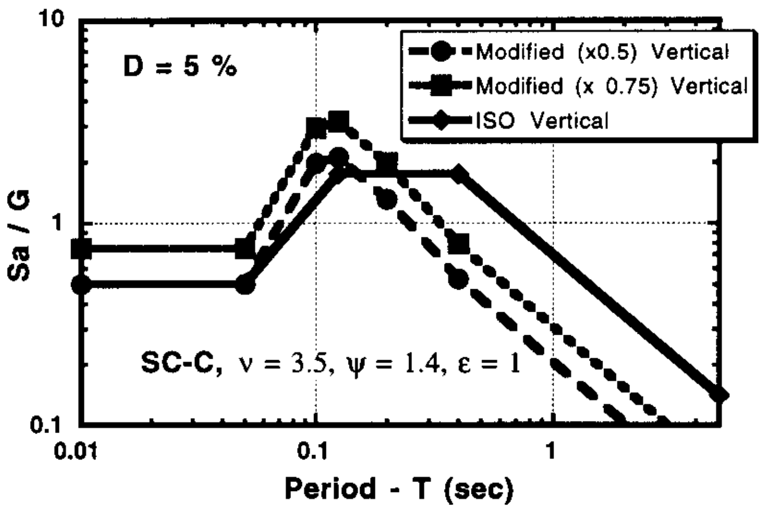

Fig. 3.2.3 - Water Column Modified Vertical Response Spectra

3.3 Computed Response Spectra

Most current seismic exposure analyses and evaluations characterize the earthquake ground motions on a rock or very firm alluvium surface. Then often one-dimensional 'shear beam' type analyses are performed to characterize the earthquake ground motions within the soil column that overlies the rock or very firm alluvium surface. A variety of soil constitutive models can be used in these analyses, but most often, 'equivalent' linear models are used with modifications introduced to model the soil non-linear stress-strain behavior and the changes in this behavior with cyclic strains. Damping is generally introduced as an equivalent viscous damping (Dobry 1991). The bottom boundary of the soil column at the rock interface is generally characterized with an absorbing boundary element to prevent unrealistic trapping of the earthquake energy in the soil column due to reflections of the energy at the rock interface.

Relative to the ISO earthquake guidelines, this practice raises issues regarding the uncertainties and biases that can be introduced by different analyses of soil column effects. The ISO guidelines do not include any definitive information on how such analyses should be performed. There is a very wide range in how such analyses are performed, and how the soil column analyses are performed can have extremely important effects on the earthquake ground motions that are utilized in analyses of the soil - structure interactions.

Fig. 3.3.1 summarizes earthquake acceleration response spectra from four locations in Mexico City based on recordings obtained during the 1985 earthquake (Seed, 1987). The soft clay soil column has discernible effects on the shapes of the response spectra and the magnitudes of the spectra ordinates. The computed natural periods of the soil columns (Ts) at each of the locations is noted based on the relationship:

$$Ts = 4\ d\ /\ Vs$$

where d is the depth of the soil column and Vs is the soil shear wave velocity (appropriate for the soil strains induced by the earthquake). For a given soil column thickness, the natural period of the soil column is controlled solely by the shear wave velocity. The shear wave velocity is a function of the shear modulus which is a function of the cyclic shear strains induced in the soils by the earthquake. Higher strains result in lower shear modulii and shear wave velocities.

The response spectra exhibit definite peaks at the natural periods of the soil columns. The noticeable increase in the response spectra energy associated with the 37 m depth site is likely due to reflected energy associated with the edge of the Mexico City basin. This basin edge influence would not be incorporated in most shear beam soil column analyses because the soil column responses would be based on the rock outcrop motions (in this case at the UNAM site).

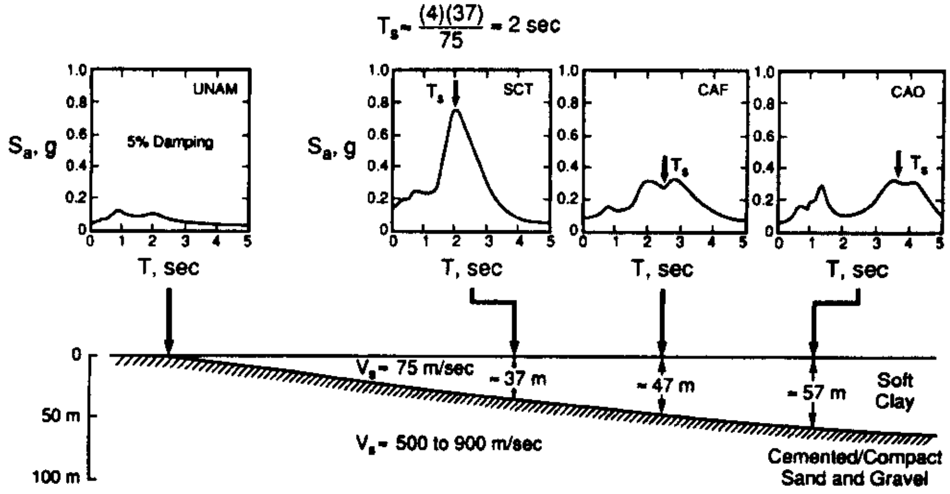

Fig. 3.3.1 - Soil Conditions and Recorded Acceleration Spectra During 1985 Mexico City Earthquake

Fig. 3.3.2 summarizes results from SHAKE analyses of motions at the soil surface identified as Oakland Outer Harbor

(Bray, et al. 1996) during the 1989 Loma Prieta earthquake (epicenter about 70 km away). The soil profile consists of 156 m of alluvium (clays, sands) overlying bedrock. The shear wave velocity in this soil column ranged from 800 m/sec (alluvium at base of column) to 63 m/s (Bay Mud). The soil column had an average shear wave velocity in the range of approximately 400 m/sec. The response spectra indicates discernible peaks at 1.5 sec and about 0.6 sec. Based on the results from testing of soil samples, a soil damping ratio of 10 % to 20 % was used.

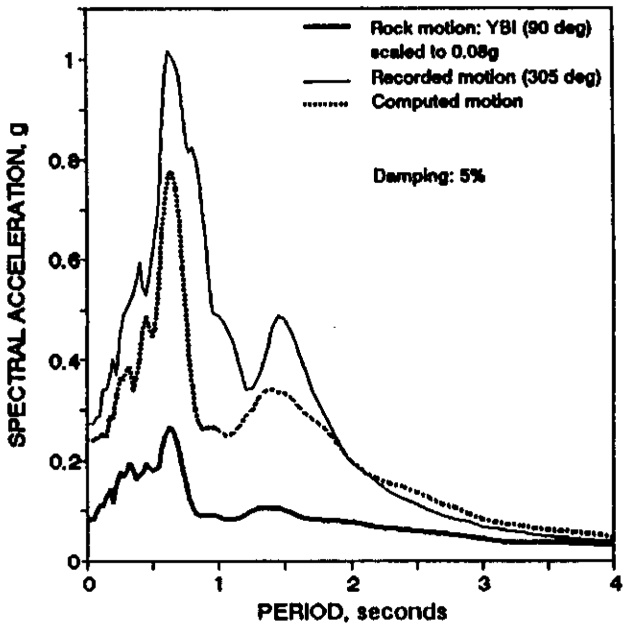

Fig. 3.3.2 - Recorded and SHAKE Computed Response Spectra (Oakland Outer Harbor, Loma Prieta Earthquake)

Response spectra from available strong motions recorded at nearby 'rock' sites are shown in Fig. 3.3.2. The response peak at 0.6 sec is obviously due to the energy in the input rock motions at this period. The response peak at 1.5 sec is obviously due to the natural period of the soil column. The spectral acceleration of the rock is amplified by a factor of about 4 at the 0.6 sec and 1.6 sec peaks of the soil column. Given a peak spectral acceleration of 0.25 g in the input rock motion, at the natural period of the soil column, the peak spectral acceleration is about 0.5 g, representing a doubling of the peak in the spectral input motion. Note that the computed motion of the soil column generally is somewhat less than the recorded motion.

Fig. 3.3.3 and Fig. 3.3.4 summarize similar results for two other nearby locations. The Oakland location consisted of 152 m of alluvium having shear wave velocities in the range of 800 m/sec to 120 m/sec. The Emryville location consisted of 137 m of alluvium having a similar range in shear wave velocities. In these cases the amplifications of the rock motions at a period of 0.6 sec were about 3 and at a period of about 1.5 sec were about 4. The peak input spectral acceleration of 0.20 g to 0.25 g was magnified at the natural period of the soil column by a factor of about 2. Again, the SHAKE analyses tend to under predict the ground motions at the natural period of the soil column but predict the amplifications reasonably well at the earthquake input peak energy period of about 0.6 sec.

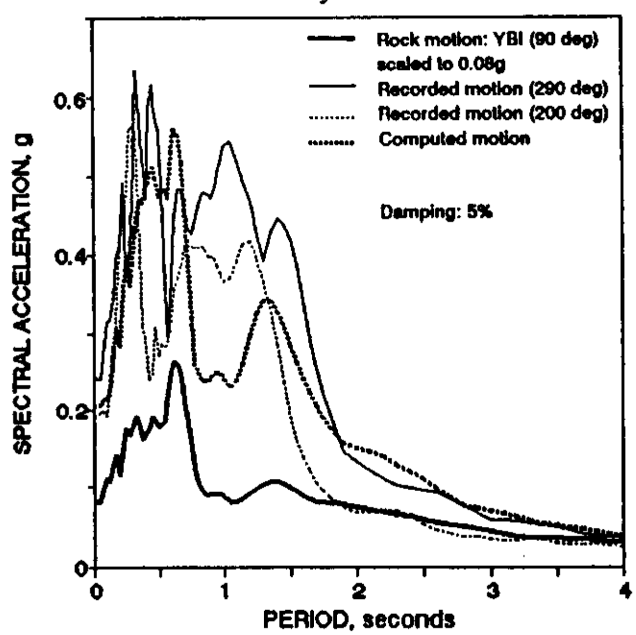

Fig. 3.3.3 - Recorded and SHAKE Computed Response Spectra (Oakland, Loma Prieta Earthquake)

These results indicate that the SHAKE analytical model and the associated characterizations of the soil stiffness and damping has a bias in the range of B = 1.5 to 2.0 for the natural period of the soil columns. Other similar results need to be developed before definitive statements can be made concerning the biases associated with the soil column analyses.

The amplification of the earthquake input acceleration on rock at the surface of the soil column can be evaluated approximately from (Dobry 1991):

$$A = [as / ar]_{max} = I / 1 + (\pi / 2)\, D_S\, I$$

where A is the ratio of the maximum acceleration at the soil surface, as, to the input acceleration at the rock interface, ar; I is the impedance ratio of the rock - soil interface, D_S is the soil damping ratio. The soil - rock impedance ratio is:

$$I = (\rho r / \rho s)\, (Vr / Vs)$$

where ρr is the density of the rock, ρs is the density of the soil, Vr is the shear wave velocity of the rock, and Vs is the shear wave velocity of the soil. The velocity ratio would generally be in the range of 5 to 30, and the density ratio in the range of 1.1 to 1.5. Thus, the impedance would generally be in the range of 5 to 50. The soil damping ratio could be expected to be in the range of 0.05 (low strains) to more than 0.3 (high strains). Fig. 3.3.5 summarizes the results of this development. Except for very low impedance, impedance is not an extremely important element. However, damping is very important. The use of low strain damping ratios in the range of 5 % to 10 % result in amplifications that are a factor of 4 to 5 times those for damping ratios in the range of 25 % to 30 %.

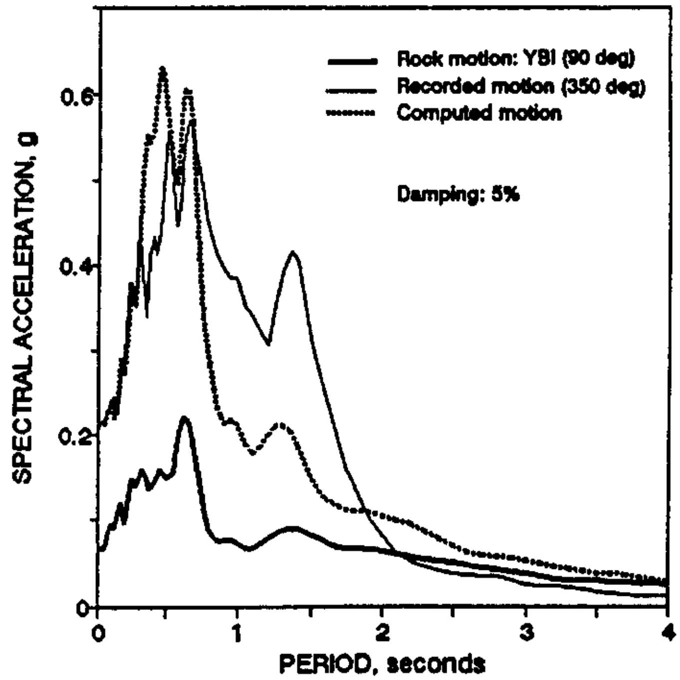

Fig. 3.3.4 - Recorded and SHAKE Computed Response Spectra (Emryville, Loma Prieta Earthquake)

The Loma Prieta site results summarized earlier had a velocity ratio in the range of 7 to 9, a density ratio in the range of 1.1 to 1.2. This would indicate an impedance in the range of 8 to 11. Given an amplification of the input rock acceleration of about 2 at these sites, a damping ratio in the range of 20 % to 30 % is indicated. This damping is much higher than indicated from the laboratory triaxial and resonant column tests. Often this has been found to be true and it has been postulated that the under-estimated damping is due to the laboratory testing boundary conditions.

Studies are currently being conducted to better characterize the bias and uncertainties associated with analyses of soil column response characteristics during intense earthquakes. These studies are attempting to account for the different methods that can be used to characterize the soil properties and the input earthquake excitations. Results from these studies will be monitored by the author and their implications integrated into future revisions of the proposed ISO guidelines.

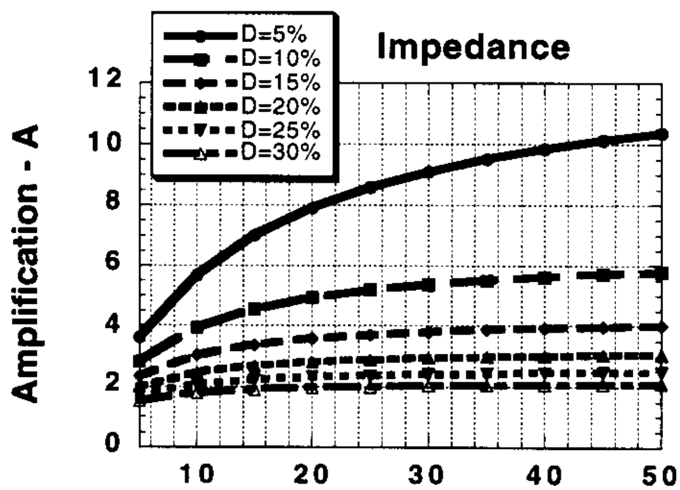

Fig. 3.3.5 - Soil Surface Amplification of Bed Rock Acceleration

3.4 North Sea Response Spectra

Elastic response spectra for North Sea seismotectonic and hard ground - rock conditions were developed as part of the EQE International study (1998) and the NORSAR study (1998). Fig. 3.4.1 summarizes the normalized response spectra results for a 5 % damping ratio from the EQE study for return periods of 100 years, 1,000 years, and 10,000 years. The response spectra do not vary significantly as a function of the intensity of the earthquakes in this range. This is to be expected since the hard ground - rock materials respond essentially in their low-strain, reasonably elastic, range. The peak amplification of the spectra is approximately 2.5.

In the EQE study, it was noted that the spectra could be modified for other values of damping by the factor K (Woo et al. 1988):

$K = 1.48 - 0.30 \ln D$

This response spectrum modification factor was compared with that contained in the proposed ISO guidelines. It was found that the modification factors are essentially identical for damping ratios less than about 20 %, and above 20 % only differ by about 10 %

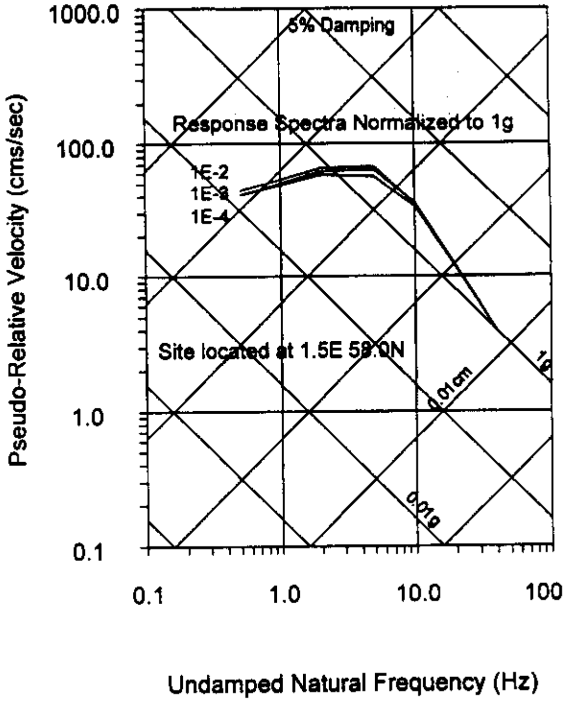

Fig. 3.4.1 - Earthquake Response Spectra for UK Sector Conditions

Fig. 3.4.2 summarizes the response spectra from the NORSAR study (1998) for a return period of 10,000 years at low seismic activity (Alta) and high seismic activity (Midgard) sites for rock or hard ground conditions. The shapes of the spectra are essentially identical.

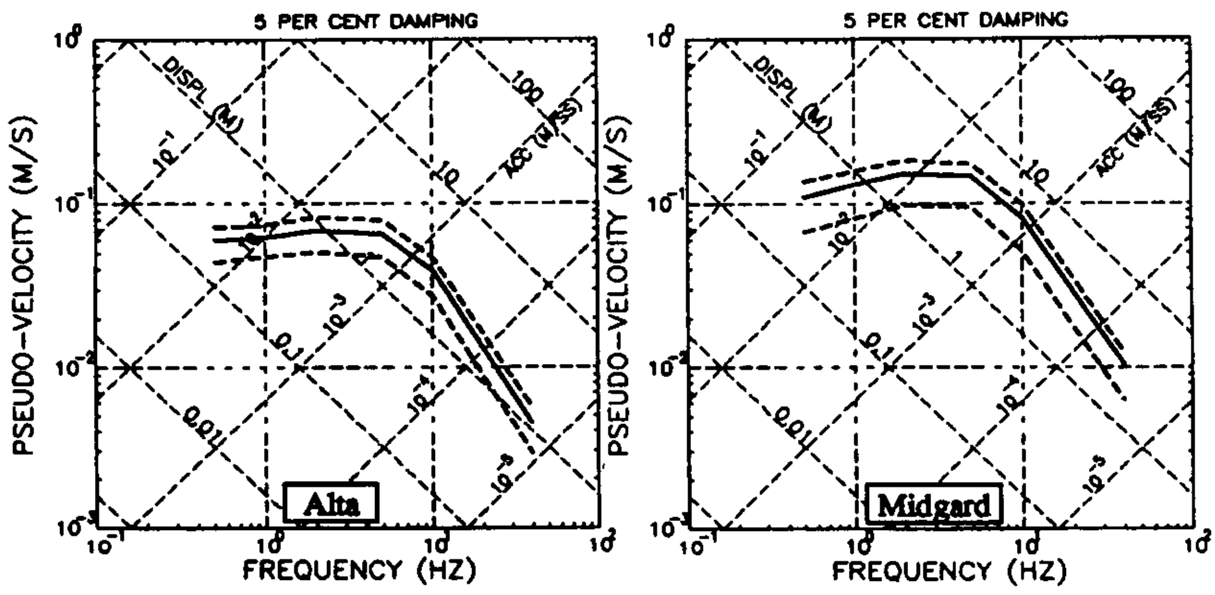

Fig. 3.4.2 - Earthquake Response Spectra for Norwegian Sector Conditions For Earthquake Return Period of 10,000 years and Rock - Hard Ground

Fig. 3.4.3 and Fig. 3.4.4 summarize results from the study of soil column effects on the rock spectra performed by the Norwegian Geotechnical Institute (1988) as part of the NORSAR study (1988). Soil column characteristics were based on results from soil exploration - soil boring - laboratory testing performed at North Sea platform locations and onshore Norway locations. The earthquake response analyses were performed using the SHAKE computer program discussed earlier. The soil columns were organized into two different categories: Soil Type B - shallow firm alluvium, and Soil Type C - deep firm alluvium. Recorded earthquake time histories characteristic of those expected for the Norwegian seismotectonic conditions were used as input to the soil column analyses.

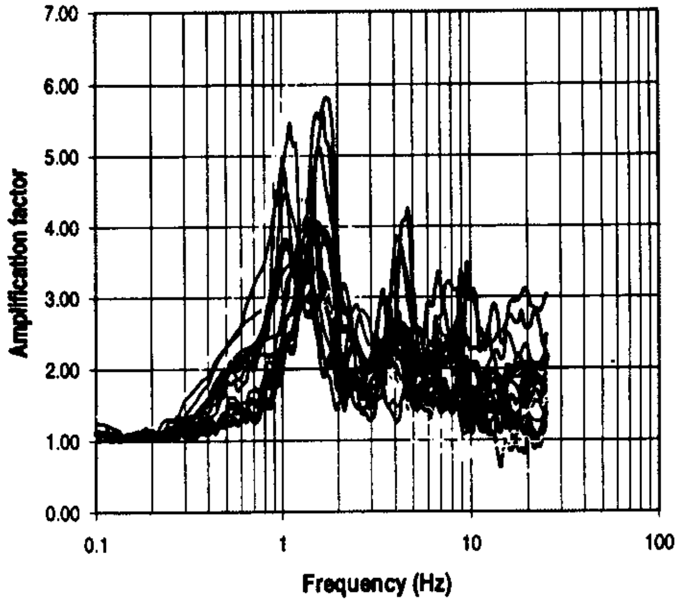

Fig. 3.4.3 - Computed Earthquake Response Spectra Amplification Factors for Deep Alluvium Soil Conditions (Soil Type C)

The amplification factors for the Soil Type B profiles has a definite peak in the period range of 0.05 sec to 0.1 sec. The peak amplifications range from about 2.5 to as high. The Coefficient of Variation (COV) of the peak amplification factors is approximately 20 % ($\approx \sigma_{GS}$). The COV suggested in the proposed ISO guidelines is 30 %. The range of results shown in Fig. 3.4.3 represents the range of results between different sites, not the range of results that reflect variations in the soil characteristics for a given site.

The amplification factors for the Soil Type C profiles has a definite peak in the period range of 0.5 sec to 1 sec. The peak amplifications range from about 2.5 to almost 6. The Coefficient of Variation (COV) of the peak amplification factors is approximately 30 %. The COV suggested in the proposed ISO guidelines is 40 %.

Fig. 3.4.5 summarizes the mean soil response spectra amplification factors for the two different soil column conditions. The mean peak soil amplification factor for Soil Type B is 3 and the same factor for Soil Type C is 3.5. These results are in excellent agreement with those summarized in Table 3.1.1. These results confirm the need to have a factor that modifies the magnitude of the peak in the response spectra as a function of the soil conditions.

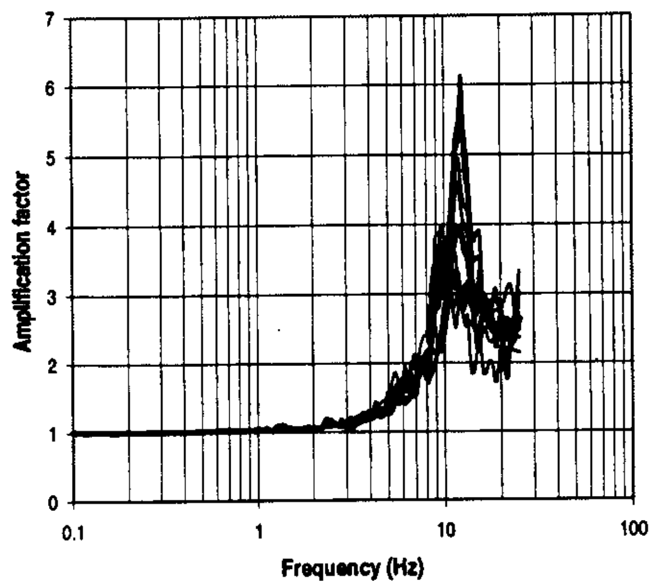

Fig. 3.4.4 - Computed Earthquake Response Spectra Amplification Factors for Shallow Alluvium Soil Conditions (Soil Type B)

The NORSAR study also addressed the vertical motion spectra and concluded that the ratio of the spectral ordinates (V/H) was frequency (f) dependent. In the vicinity of the peak of the horizontal response spectrum (f = 1 to 3 Hz), the recommendation was that the ratio should be determined as follows:

V/H = 0.9 - 0.42 log (f)

For f = 1 Hz, V/H = 0.9. These results are in reasonably good agreement with those summarized in Fig. 3.2.2. Based on the information provided in the NORSAR study, it is understood that these results are for the surface of the soil column and do not incorporate the effects of the overlying water column. Future studies of the vertical response spectra at the sea floor should take account of these effects.

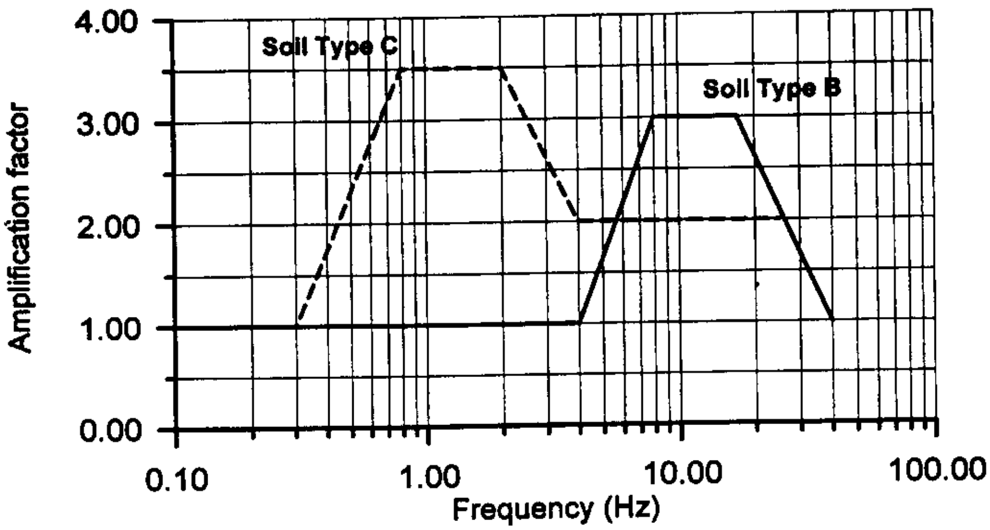

Fig. 3.4.5 - Recommended Earthquake Response Spectra Mean Amplification Factors for Deep Alluvium Soil Conditions (API Soil Type C)

3.5 Bay of Campeche Spectra

Chavez (1997a; 1997b) performed a number of site response studies for the Bay of Campeche using the computer program SHAKE. Results from recently performed high quality soil borings and laboratory static and dynamic testing programs were used to characterize the strength, stiffness, damping, and hysteretic performance characteristics of the soils. The soils consist of deep (100 m to 200 m) firm alluvium (layered calcareous sands, silts, and clays). Chavez introduced the deconvolved input motions from the three types of earthquakes at the base of the soil columns and determined the output motions close to the sea floor (at -12 m, appropriate for lateral motions characterizations for piles) and at significant penetrations below the sea floor (-100 m, appropriate for vertical motions characterizations.

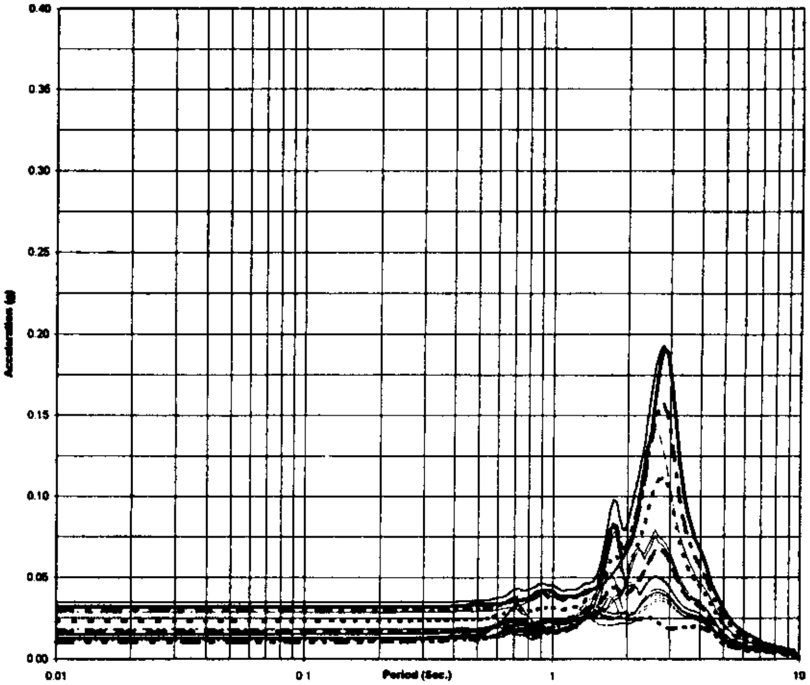

Fig. 3.5.1 - Type I 200-year Earthquake Computed Response Spectra

Results for the 200-year horizontal Strength Level Earthquake (SLE) conditions (-12 m) for a variety of sites in the Bay of Campeche are summarized in Figures 3.5.1, 3.5.2, and 3.5.3 for Type I (very distant subduction zone), Type II (distant mixed zone), and Type III (nearby surface faulting zone) earthquakes, respectively. The Type I earthquake produces a peak in the ground motions at a period of about 3 sec that represents the long-period energy from these distant subduction zone earthquakes that coincides with the natural period of the soil column. The Type II earthquake produces a set of response spectra with multiple peaks in the range of 0.3 sec to 3 sec. The Type II earthquake produces a much more conventional spectrum with peaks in the range of 0.2 to 0.8 sec.

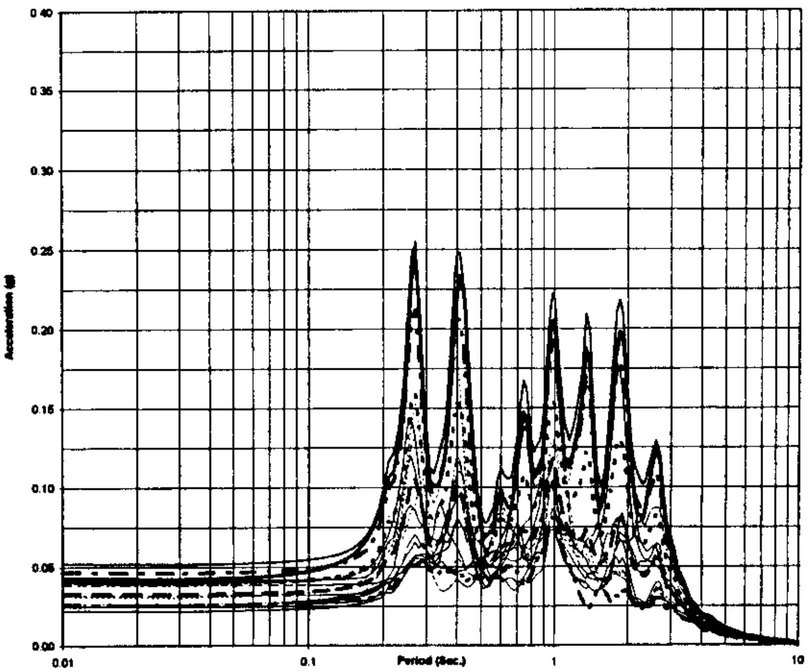

Fig. 3.5.2 - Type II 200-year Earthquake Computed Response Spectra

The resulting mean response spectra for these three types of earthquake conditions is summarized in Fig. 3.5.4, together with the mean plus one standard deviation and envelope spectra. Chavez included the range in response spectra results that developed from plus and minus one standard deviation changes in the soil shear modulus and damping characteristics. The COV of the peak response spectra values is about 100 %. This high uncertainty reflects the influences of the three different types of seismic sources, different soil characteristics at different sites, and the effects of plus and minus one standard deviation ranges in the soil properties at the different sites. Given a location that is affected by multiple types of earthquake sources, it would be preferable to define different response spectra associated with these sources rather than combine them into a single response spectrum.

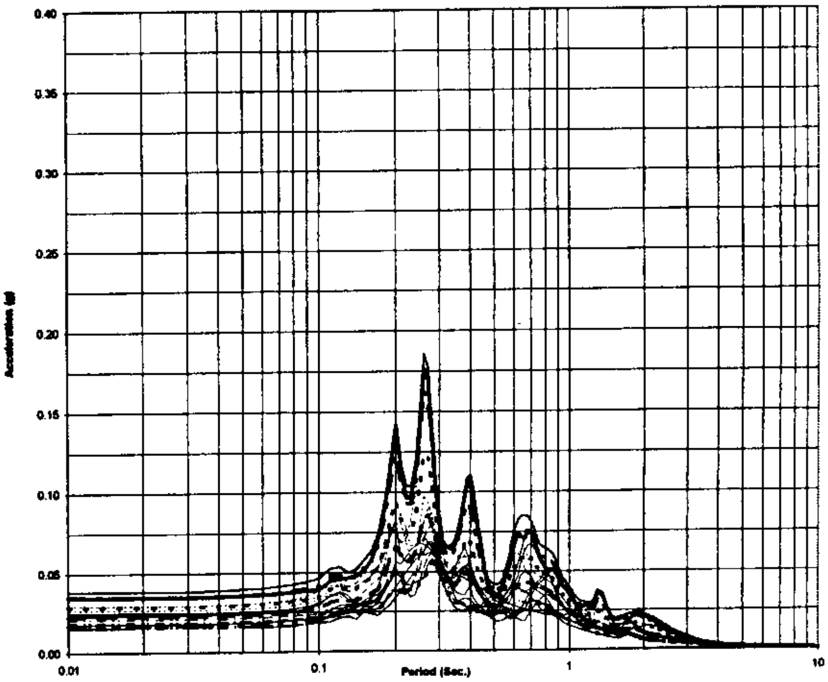

Fig. 3.5.3 - Type III 200-year Earthquake Computed Response Spectra

These results clearly indicate that the dominant horizontal and vertical motions important to the pile foundations of the platforms in the Bay of Campeche are associated with the Type II earthquake source. At a period of 1 sec (predominant natural period of platforms in Bay of Campeche), the Type II earthquake source produces horizontal motions at a penetration of -12 m that are a factor of about 10 greater than those for the Type I and Type III sources.

In general, these results are in good agreement with the proposed modifications to the ISO SLE response spectra.

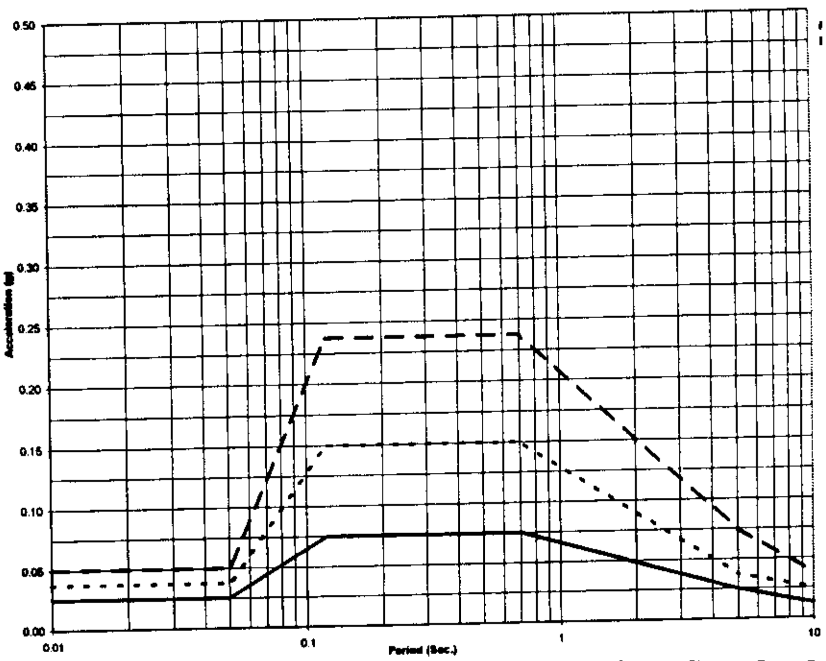

Fig. 3.5.4 - 200-year Mean, Mean Plus One Standard Deviation, and Envelope Response Spectra

Chapter 4

Summary, Conclusions, & Acknowledgments

4.0 Summary

The objective of this study was to continue development of earthquake load and resistance factor design (LRFD) guidelines with a study of results from recent seismic hazard characterizations. This study addressed two primary topics:

a) seismic zonation of the UK and Norwegian Sectors of the North Sea, offshore Japan and Indonesia, and the Bay of Campeche, Mexico; and

b) Strength Level Earthquake (SLE) response spectra.

This study was organized as follows:

- obtain recent background on seismic hazard characterizations.
- detail seismic exposure uncertainties and local site response uncertainties.
- reconcile differences between current ISO seismic hazard guidelines and those developed during this study.
- document the revised seismic hazard and response spectra characterizations in a project technical report.

Results from current seismic exposure studies of the UK and Norwegian Sectors of the North Sea indicate SLE rock PGA in the range of 0.015 g to 0.08 g and seismic uncertainties in the range of σ_E = 1.5 to 1.7.

Results from current seismic exposure studies for the Japan coastal and offshore areas indicate SLE rock PGA in the range of 0.05 g to 0.5 g and seismic uncertainties in the range of σ_E = 0.8 to 1.0.

Results from current seismic exposure studies for the Indonesia coastal and offshore areas indicate SLE rock PGA in the range of 0.05 g to 0.35 g and seismic uncertainties in the range of σ_E = 0.6 to 2.0 (lower end of range associated with subduction zone dominated sites).

Results from current seismic exposure studies for the Bay of Campeche, Mexico, indicates SLE rock PGA in the range of 0.01 g to 0.05 g and seismic uncertainties in the range of σ_E = 0.6 to 1.8.

Results from current studies of horizontal response spectra associated with SLE intensities indicates that the ISO SLE horizontal response spectra for periods in the range of about 0.1 sec to 0.5 sec should be modified (increased) to account for different types of soils. Such modifications have been developed as a result of this study. Results from region and site specific SLE horizontal response spectra generally are in good agreement with the proposed modified ISO response spectra.

Study of information relating to the vertical ground motion spectra for on-land conditions indicate that the ratios of vertical motions to horizontal motions in the low period range (0.1 sec to 0.3 sec) for near-by earthquakes can be much greater than currently specified in the proposed ISO guidelines.

However, when the effects of the water column are taken into account combined with the proposed modifications to the horizontal response spectra in the period range of 0.1 sec to 0.5 sec, the proposed modified ISO guidelines generally provide a conservative estimate of the vertical motion characteristics.

4.1 Conclusions

The results from this study indicate that the proposed ISO seismic exposure guidelines generally provide a conservative estimate of the SLE PGA for the regions studied. The current regional seismic exposure results provide the basis for updated North Sea, Japan, and Indonesia regional ISO seismic exposure maps (Fig. 2.1.2, Fig. 2.2.2, Fig. 2.3.2, Fig 2.4.6).

The results from this study indicate that the proposed ISO guidelines for seismic exposure uncertainties associated with the different seismotectonic zones generally provide a conservative estimate of the uncertainties. The current regional seismic exposure results provide the basis for updated North Sea, Japan, Indonesia, and Mexico seismic exposure uncertainties.

The results from this study indicate the need for modifications to the proposed ISO SLE horizontal response spectra in the period range of about 0.1 sec to about 0.5 sec. Based on results from current response spectra studies, proposed modifications that are a function of the type of soil column have been developed. In general, the proposed modified ISO SLE horizontal response spectra provide a conservative estimate of the expected SLE horizontal response spectra developed from detailed site specific studies. The uncertainties associated with the response spectra ordinates as suggested in the proposed ISO guidelines provide reasonable characterizations for the different types of soil columns.

The results from this study indicate that it is important to clearly identify the position in the soil column that are associated with the response spectra. The positions should be those that characterize the positions that result in maximum input of motions or energy from the earthquake ground motions. For pile supported structures, these motions would be below the sea floor for lateral motions (e.g. 5 to 10 pile diameters) and along the lower one-third to bottom of the pile shaft for axial motions. For mat supported structures, these motions would generally be much closer to the sea floor, but are still not generally at the sea floor itself. These considerations are particularly important for specification of the vertical response spectra for both pile and mat supported platforms.

The presence of the water column has important effects on the vertical motions at or very close to the sea floor. These effects need to be accounted for when evaluations are developed for the vertical response spectra for these elevations. Analysis of available information on vertical motions and the effects of the water column generally indicates that the proposed modified ISO guidelines provide a conservative characterization of the vertical motion effects.

4.2 Acknowledgments

The author would like to acknowledge the technical guidance and information provided by Dr. Ove Gudmestad of Statoil and Mr. Mike Craig of Unocal. Financial support for the second phase of this study was provided by Statoil and Unocal.

The third phase of this study will be sponsored by the U. S. Minerals Management Service. The third phase will address verifications of the proposed ISO guidelines based on site and platform specific results provided by the project sponsors.

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