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GEOTECHNICAL FACTORS IN SEISMIC DESIGN OF FOUNDATIONS STATE-OF-THE-ART REPORT

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SUMMARY

This paper revises the factors that influence the behavior of foundations in seismic environments. It discusses aspects related with seismic load definition, dynamic soil properties, field and laboratory testing equipment, geoseismic instrumentation of prototypes, foundation seismic stability, use of artificial intelligence, among others. It also points out areas where more research is needed to better our knowledge on the physics of the problem and to improve experimental and numerical techniques, with the purpose of making more reliable and less costly foundation systems.

INTRODUCTION

Proper analysis and design of foundations under seismic loading involves a broad variety of factors related with seismology, earthquake geotechnical engineering, geology and applied mechanics. Thus, topics on seismic soil site effects, techniques to define dynamic properties, soil-structure interaction phenomena, foundation stability, code requirements and instrumentation of soil-foundation systems are reviewed. Although many of the themes touched upon herein are applicable to most geotechnical structures, the arguments are focused only on onshore foundations. For recent studies on offshore foundations the reader is referred to Clarke [1992].

This keynote paper is not intended to give detailed accounts of the processes involved in a good foundation design for seismic (an thus for static) conditions. Pender [1995] made an excellent dissertation on this subject matter. Rather, only the aspects that have decisive influence on earthquake foundation engineering are discussed herein. Particular emphasis is given to recent developments highlighting case histories, new procedures in soil testing and the use of artificial neural networks in earthquake foundation engineering.

In view of the wide spectrum of soil characteristics, foundation types and environmental conditions we may encounter in real life problems, factors that affect the analysis and design of foundations are treated rather generically and only in areas where knowledge is thought to be fragmentary or sketchy more detailed analyses are offered. Accordingly, the aim of this paper is to put in perspective the elements that impinge on the seismic design of a foundation. Thus, specific design methods or procedures are not recommended.

2 SOIL SITE INVESTIGATIONS

Field and laboratory investigations should, in general, be oriented to define soil deposit stratigraphies; hydraulic conditions of site pore water; soils index properties; static and dynamic stress-strain soil behavior; and post-earthquake shaking behavior of soils. Potentially liquefiable granular materials should be identified at this stage.

2.1 Field tests

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It has long been recognized that soil dynamic properties depend on the level of strain induced. With the advent and significant developments of field tests to evaluate soil shear modulus, it became imperious to define the strains at which moduli were obtained by the various available field techniques.

In an effort to classify in situ tests in terms of the level of strains and thus make field measurements compatible with laboratory techniques, the grouping depicted in figure 2.1 was proposed by an expert committee [Burland, 1995].

Group A. Very small level of strain, defining the elastic behavior range. In this group, the diverse procedures for field measurements of shear wave velocity are gathered, providing the *in situ* elastic stiffness at very small strain level. Among them:
(a) Geophysical exploration from ground surface (reflection and refraction methods, and surface wave technique)
(b) Geophysical borehole logging (PS-suspension, DH-downhole and CH-crosshole methods), and
(c) Seismic cone penetration test (downhole and crosshole procedures).
Group B. Small level of strain, defining moderate or pre-failure behavior. Pressuremeter test and plate load tests belong to this group.
Group C. Large level of strain, characterizing failure, and even residual conditions. Pressuremeter and plate load tests reaching failure conditions as well as other field techniques as SPT and CPT can give this information.

Procedures of Group A provide the bench-mark stiffness or initial shear modulus G_o for strain levels of 10^{-5} or less, giving the reference value to normalize the strain dependent shear modulus (G) values. However, it is well recognized that most engineering works induce strains beyond the elastic range. Accordingly, due to the soil nonlinear behavior the values of G are lower than G_o.

Procedures of Group B are characterized by a "theoretical background" through which the calculated stiffness is associated to a certain strain level. So, using the pressuremeter data in terms of applied pressure and measured cavity strain, the stiffness is usually calculated on elasticity-based theory. Precisely due to this, and because the induced strains go beyond the elastic limit, the modulus must be calculated using plasticity-based theories, and be specified for the corresponding strain level. A comprehensive review of advantages and disadvantages of the pressuremeter test is presented elsewhere [Tani, 1995]. An important limitation of this technique is that only boundary displacements are measured where, precisely, disturbance effects are significant [Burland, 1995].

Procedures of Group C are those field tests for which their stiffness values are not related to specific strain levels. Tests like SPT, CPT, dilatometer, among others, do not have a theoretical background oriented to identify the corresponding strain. Then, test results must be empirically or semiempirically correlated with certain reference stiffness given by Group A or B procedures. In this sense efforts to correlate SPT and CPT values with shear wave velocities (i.e. [Ohta and Goto, 1976; Ovando and Romo, 1990] among many others) have been directed throughout the years for different soil types under various conditions.

2.2 Laboratory tests

When laboratory tests are planned to measure the dynamic stiffness, damping and strength of in-situ soils and rocks, the following aspects should be considered:

- the representativeness of samples of a mass in the field, reviewing if this one is stratified, erratic, or pseudohomogeneous,
- the level and effects of sample disturbance,
- the field conditions and the strain range of interest for the particular problem, in order to define the appropriate apparatus and the required accuracy and resolution of strain and stress measurements,
- the driving system compliance and possible bedding effects in a laboratory equipment, that could mask the true deformation characteristics of the geomaterial specimen, and
- the testing conditions oriented to reproduce, in a practical way, the field conditions, including the reconsolidation procedure and the shear stage with the proper strain level.

The analyses of a large amount of geotechnical case histories [Tatsuoka and Kohata, 1995] have disclosed lower observed deformations or movements than those predicted. Most of them have been explained in terms of smaller-than-actual stiffness values used in the analysis. In conventional laboratory and even field testing for routine engineering practice, some of the above mentioned aspects are ignored. In such conditions, stiffness values measured by different procedures are often compared neglecting the crucial role of the strain level. Sample disturbance is responsible that laboratory stiffness be, sometimes, lower than the back-calculated from field full-scale displacements. However, in many cases this influence is not enough to explain the discrepancies.

Since the seminal paper written by Burland [1989], a huge amount of laboratory experimental research has been done around the world, oriented to learn about the stiffness of soils at very small strain levels. The authors consider that laboratory testing techniques have achieved a more rapid progress than field tests. Ingenious solutions and recent technological developments have been put in practice to measure bedding-error free local strains. Measurement of local strains was the key to enhance the knowledge on stress-deformation behavior at small amplitude strain levels.

It has been well established that for all practical purposes, soil stiffness at strains smaller than the elastic threshold strain remains constant both during monotonic and cyclic loadings; strains are essentially recoverable and strain-rate independent. These observations have provided basis to some researchers [i.e. Lo Presti et al, 1995] to argue that static tests seem preferable to dynamic tests to obtain the stress-small strain response of a soil sample under monotonic or cyclic loading. This statement may be generally accepted for granular materials, where strain-rate effects have been shown to be negligible for strain levels lower than about 0.001% [i.e. Hardin and Drnevich, 1972; Teachavorasinskun et al, 1991; Shibuya et al, 1992]. However, many laboratory studies have clearly indicated the strain-rate influence in cohesive soils and its influence within the small strain range requires further research [i.e. Isenhower and Stokoe, 1981; Kramer et al, 1992; Shibuya et al, 1995; Vucetic et al, 1998].

Since the early developments of the resonant column by Drnevich and the cyclic triaxial by Seed and Lee, there have been many improvements and advances in dynamic testing equipment. This boost is mainly due to the significant advances in monitoring quality and data acquisition techniques. Of the many developments in soil testing equipment perhaps the most relevant in recent years (for dynamic-property determinations) are the combination of resonant column and cyclic torsional shear (RCTS) testing of the same sample, and the use of bender elements to measure wave propagation velocities in the soil specimen. A detailed description of the RCTS equipment can be found in Stokoe et al [1994]. This device eliminates the variability of results produced by testing "twin samples" in different equipments. Bender elements convert mechanical deformation into electrical energy and *vice versa*. To avoid electrical shorting, bender elements must be water proofed. Based on wave velocities and soil unit weight, maximum Young's and shear modulus, and Poisson's ratio can be determined for a wide range of confining pressures and strain levels. Advantages and shortcomings of this device are put forth in Bray et al [1999].

On a larger scale, important developments on centrifuges and shaking tables have added to the arsenal of laboratory testing equipment that makes possible investigations on soil-structure interaction, soil contaminants migration, liquefaction phenomena, and so on.

2.3 Advantages and disadvantages of field and laboratory tests

It is recognized that despite the fact that laboratory and field testing techniques are not ideal, the different procedures intrinsically provide solid bases for the assessment of dynamic properties. However, it is usual that a comparison of their results for the same soil indicates considerable differences. Common errors or lack of precision in experimental determinations do not explain by themselves the discrepancies [Gajo y Mongiovi, 1994]. Justification must be sought for in the behavior of geomaterials, and not in the techniques, at least not directly. So, their nonlinear behavior plays an important role. Soil conditions can be better controlled in the laboratory, while field conditions do not permit adequate control of the influencing factors. Thus, interpretation and comparison of field test results commonly require laboratory tests where sensibility analysis for various parameters can be achieved, under repeatable conditions. Additionally, determination of damping ratio in the laboratory is a relatively easy task, both under forced and free vibrations.

It is clear that laboratory tests are all affected to some degree by specimen disturbance, induced by sampling, handling and preparation method, reducing in general the dynamic stiffness. In such a case, the laboratory (G_{0l}) to field (G_{0f}) ratio reaches values lower than unity. It seems that medium to hard brittle, overconsolidated soils are the most sensitive to disturbance; small size fissures could have a decisive influence. The opposite tendency has been observed for soft clays and loose sand as shown in figure 2.2, where results from tests using resonant column tests apparatus and PS suspension logging are compared [Yasuda et al, 1994]. Using the same techniques, this trend has been corroborated for the extremely compressible clay from Mexico City [Mendoza et al., 1997]. This tendency may be explained, at least partially, on the grounds that some soil remolding is induced by boring operations and, perhaps more importantly, due to yielding of bore-hole walls when stabilization is provided by a slurry. Disturbance decreases soil stiffness and yielding induces shear deformations in the soil causing that field wave velocities be measured at not-so-small strains and thus the stiffness obtained are for strains larger than those developed, for example, in resonant column tests (10^{-4}%). It should be mentioned that other conclusions might be reached when comparing field data obtained from other tests (i.e. down hole or cross hole) where shear stiffness measurements are not localized as in the PS logging technique. Finally, it is relevant to acknowledge that due to the complex processes involved in soil profile formation, it is advisable to resort to geostatistical techniques to interpret and generalize, for example, soil deposit stratigraphies and CPT profiles [Auvinet, 1999].

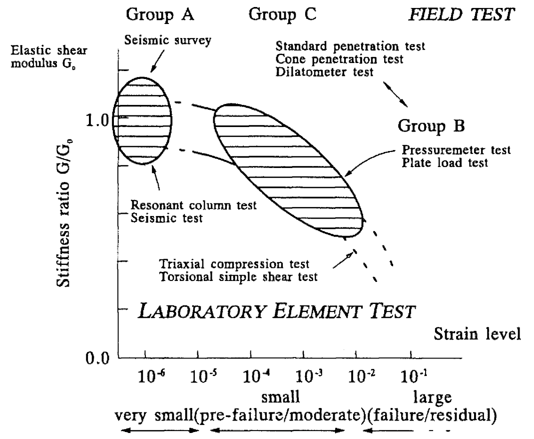

Figure 2.1 Comparative studies of nonlinear shear deformation, field tests vs. laboratory tests (after Tani, 1995)

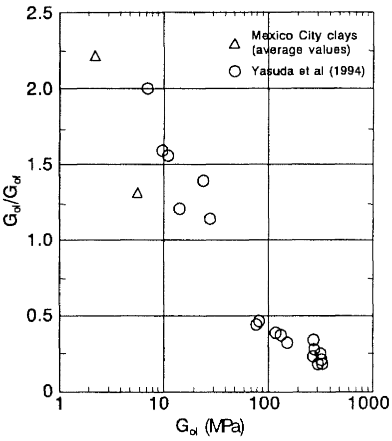

Figure 2.2 Soil-stiffness influence on lab/field (G_{ol}/G_{of}) shear modulus ratio (modified from Yasuda et al, 1994)

3. DYNAMIC BEHAVIOR OF SOILS

A key factor in seismic foundation design is the behavior of soils under static and dynamic loading conditions. Many of the "peculiar" aspects of foundation response are usually better understood when a static-behavior-reference-frame is well established. Therefore, laboratory testing programs should also include static assays.

From the geotechnical view point, seismic foundation design requires information about plastic deformability, strength, damping and stiffness of soil materials. The last two are needed to study the response characteristics of soil deposits (and soil-foundation systems), soil strength is required for foundation stability analyses and plastic deformability for the evaluation of earthquake-induced permanent displacements. Additionally, the duration of seismic excitation influences all four parameters and must be included in the designs.

3.1 Stiffness and Damping

The seminal works of Hardin and Black [1969), Seed and Idriss [1970] and Hardin and Drnevich [1972] clearly indicate that these dynamic parameters are affected by a number of variables amongst which the most significant are the confining effective stresses, the void ratio, the degree of saturation and the dynamic loading time duration. The influence of other factors like the over consolidation ratio, effective stress-strength parameters and time-sustained loading, currently being revised, especially over the small-strain range.

3.1.1 Undrained properties of Saturated Cohesive Soils

In addition to the above mentioned parameters, the plasticity index, PI, has been found to influence significantly shear moduli and to a lesser degree damping ratios, λ, of clayey materials [i.e. Dobry and Vucetic, 1987; Sun et al, 1988; Romo et al, 1989]. Experimental evidence clearly suggest that the range of quasi-elastic behavior of these materials increases with PI. Simple hyperbolic models that comply with the Masing rules may be used in practice to compute both G/G_0-γ and λ-γ curves once the PI is known [i.e. Romo, 1995]. Here, G_0 is the low strain ($\cong 10^{-4}$%) shear modulus and γ is the shear strain.

Another property that also bears an important influence on G_0 and the shape of G/G_0-γ and λ-γ curves is the liquidity index. Together with PI has been incorporated into hyperbolic stress-strain relationships to model the dynamic behavior of clays [Romo and Ovando, 1995]. This model has been used to derive p-y curves for the analysis of piles under seismic lateral loads [Romo and Ovando, 1999].

These results imply that PI values may have a tremendous impact on soil-site amplification effects. In fact the extremely high ground motion amplifications observed in Mexico City during the 1985 Michoacán earthquake were explained on account of the quasi-elastic behavior of the clayey deposits [Romo, 1987]. Soil stiffness degradation is another factor that should be considered in the seismic design of foundations. The degradation parameter proposed by Idriss et al [1978] has been shown to depend on the magnitude of the cyclic strain, the stress path followed in sample consolidation, over consolidation ratio and PI [Dobry and Vucetic, 1987].

It is also important to recognize that since dynamic loading is applied at higher rates than monotonic charges, the undrained shear strength of clays is increased when dynamically loaded. This effect is disregarded in dynamic bearing capacity computations. It is not clear why this is done, but it may very well be on account of the undrained strength drop due to fatigue effects. However, there exists experimental evidence indicating that, aside from sensitive clays and low-plastic clays that may accumulate large amounts of dynamically-induced pore water pressures, the static undrained strength of many clays remains practically unchanged after dynamic loading. For example, Mexico City clays having PI values greater than about 150% do not experience any strength loss when static plus dynamic stresses remain below the static undrained strength [Romo, 1990]. Dynamic pore water pressures are negligible for shear stresses under the undrained strength. Thus, it may be argued that for highly plastic clays rate loading effects may work in favor of foundation stability. It would seem that this aspect of clay behavior deserves further research.

3.2 Undrained Properties of Saturated Cohessionless Soils

Although it is recognized that the behavior of granular soils is very complex, and as such very difficult to take into account the many particular aspects of it. On the basis of observed field and laboratory soil responses, it may be argued that for foundation analyses under dynamic loading, consideration of their tendency for dilation or compression would suffice for most practical cases. Thus special attention should be paid to soil relative density. When granular soils have a dilative behavior then negative pore water pressures develop for undrained conditions, leading to effective stress increases that improve their bearing capacity capabilities. On the other hand, if the granular materials have a contractive behavior then positive pore water pressures ensue when loaded under constant volume conditions. Accordingly, there is a loss in effective stresses and the bearing capacity drops. When a low relative density combines with high dynamic shear stresses induced by the coupled action of seismic wave passage and stress waves radiating away from the building-foundation system, the effective stresses may decrease to values near zero, causing a sudden loss of bearing capacity to the foundation soil that may lead to soil liquefaction, as has been observed in the recent past. All aspects referred to in section 3.1.1 regarding nonlinear soil behavior and soil fatigue effects are common to granular soils too.

Therefore, foundation engineers should be more concerned with granular soils that have a tendency to decrease their volume during earthquake shaking. For this condition, it is recommended that instead of designing for such unfavorable situation, it would be better to define the most adequate method for soil site improvement to eliminate the possibility of positive pore pressure generation, or use good common sense and find, if possible, a better foundation site. On the other hand, if the granular material has a dense-type response, the stability of the building-foundation system becomes secondary and the engineer should focus on potential permanent displacements induced by the design earthquake.

4. SEISMIC ENVIRONMENT

The evaluation of site-specific ground motions involves a number of steps that include the identification of potentially active sources in the region, the evaluation of the seismicity associated with individual sources, the estimation of travel-path influence on the seismic waves characteristics as they propagate from the source to the particular rock site, the computation of the dynamic response of soil deposits and of soil-structure systems, and the assessment of their stability when subjected to the design-level seismic environment. The first three steps, which are closely related to geological and geophysical processes, are treated in depth in other state-of-the-art papers in this World Conference. Therefore, only the aspects related with soil site, soil-structure interaction and foundation-stability assessment will be discussed herein.

4.1 Local Site Effects

The influence of ground deposits on bedrock movements depends on seismological aspects, geologic conditions, site-geotechnical characteristics and site-geometrical peculiarities. Table 4.1 lists the main factors that contribute to site effects. Detailed reviews of local site effects are given elsewhere [Aki, 1988; Somerville, 1998].

4.1.1 Field Evidence

The ever-increasing awareness of the importance that instrumental information has on improving our understanding on how soil deposits affect ground motions, has driven the installation of many accelerometers throughout the world. This has permitted to gather an extensive collection of ground motion records on a great variety of soil-site conditions that has contributed to an enhanced understanding of local effects for a wide variety of seismological, geological, geotechnical and geometrical conditions. Examples that show how these factors may affect site ground motions are given in figures 4.1 to 4.5. Figures 4.1 and 4.2 depict the acceleration spectra of the surface horizontal ground motions recorded at SCT site, located on the soft clay deposits of Mexico City and at rock-like site (CU), during the 1985 Michoacán (18.08° Lat N, 102.94° Long W; M_s=8.1) and 1999 Tehuacán (18.20° Lat N, 94.47° Long W; M_s=6.7) earthquakes. For site locations see figure 6.3.

Table 4.1 Main factors that influence site-effects

Seismological	- Intensity and frequency characteristics of bedrocks seismic environment - Duration of bedrock motions
Geological	- Local geologic structure - Underlying rock type - Soil deposit thickness - Stratigraphical characteristics - Soil types in the stratigraphy
Geotechnical	- Elastic vibration characteristics of the soil deposit - Impedance contrast between the bedrock and overlying soil materials - Nonlinear behavior of soils in the stratigraphy, including fatigue-type effects by shaking duration
Geometrical	- Non horizontal soil-deposit layering - Topography of underlying bedrock - Basin configuration - Other inclusions that lead to two and three dimensional geometries

Response spectra of figure 4.1 show the combined influence of the seismological, geological and geotechnical factors on site effects. It is seen that in addition to amplifying the maximum ground accelerations, the surface ground spectral accelerations are enhanced. To separate the effect of each of the three geophysical factors, the information in figure 4.1 is reinterpreted as follows. To appreciate the seismological influence on rock motions, the acceleration response spectra are normalized by their peak ground acceleration, PGA, and plotted as indicated in figure 4.2. Normalization eliminates the intensity factor of the motions recorded on rock during both events. Thus, the differences observed between the spectral curves reflect the effect of the energy-release source and wave paths (followed from the epicenter to the site) discrepancies between both earthquakes. It is evident that the event from the closer source (Tehuacán) has a higher frequency content, as it would be expected.

Ground motions that have been recorded at so many sites worldwide have shown beyond any doubt that soil type and stratigraphic characteristics modify appreciably the rock motions characteristics. As an example of the importance of this local site effect, figure 4.3 shows a comparison between the ground surface-normalized spectral accelerations of two clayey sites (SCT and CAO) in Mexico City. They clearly indicate that even for relatively close sites having similar geotechnical conditions, their responses are significantly different. This particular example, alerts us of the potential mistakes that can be made when motions from a particular site are used as input excitation for the seismic design of a foundation-building system on a not-far-away site having look-alike geotechnical conditions. In this case, the differences in spectral values are mainly explained on the basis of soil thickness variations.

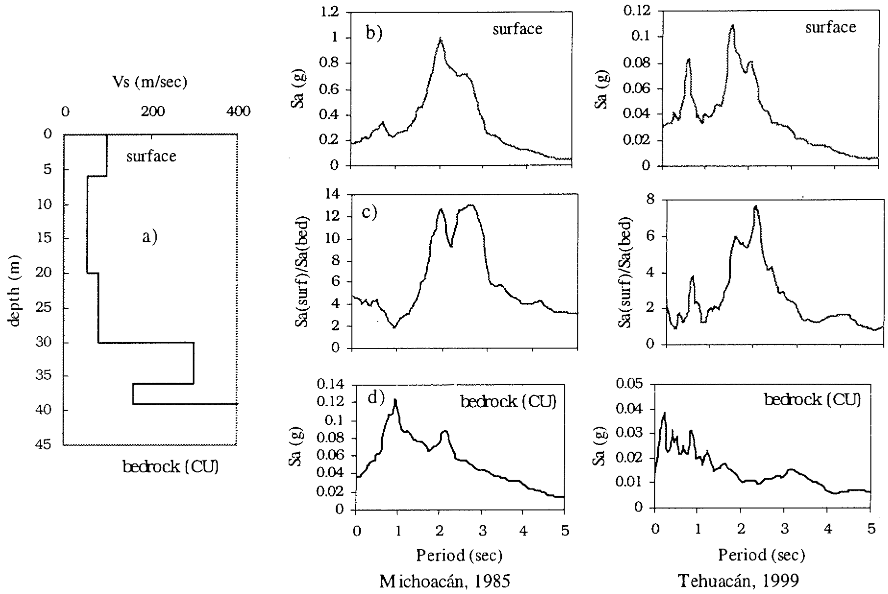

Figure 4.1. SCT site response to two earthquakes

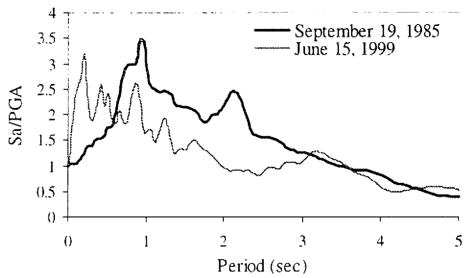

Figure 4.2 Seismological effects on rock-like motions

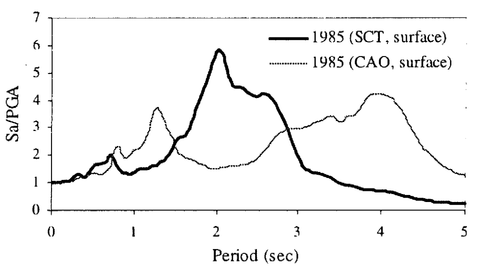

Figure 4.3 Geotechnical and geologic effects

Usually the excitation is represented by a response spectrum (or an accelerogram) specified at the surface of the free-field. This assumption neglects ground motion-severity decrease with depth that for some soft soils stratigraphies may be appreciable, particularly within the top few meters, as depicted in figure 4.4 for the SCT site for the September 14, 1995 seismic event (16.31° Lat N, 98.88° Long W, M_s=7.2). It is seen that for periods ranging from 1.5 to 2.5 sec the depth-attenuation effect is highly significant for this particular site. This suggests that foundation designs in earthquake prone areas should give due consideration to this fact. It may be argued that a rigid foundation, as compared with the stiffness of the volume of soil it replaces, seated at some depth, will decrease (even without considering interaction effects) the severity of the motions that are transmitted to the structure as compared to an equivalent more flexible foundation, with the ensuing benefits on safety and economy. Figure 4.4 clearly shows that a 10 m-deep seated rigid foundation (i.e. box foundation) would be the best choice for a building having natural periods in the 1.5–2.5 sec range. However, outside this range the seismic attenuation benefits are not decisive when selecting the foundation.

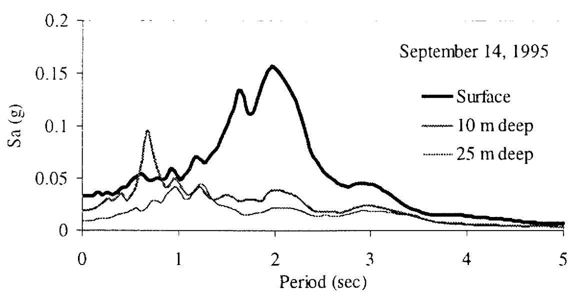

Figure 4.4 Spatial variation of ground motions SCT

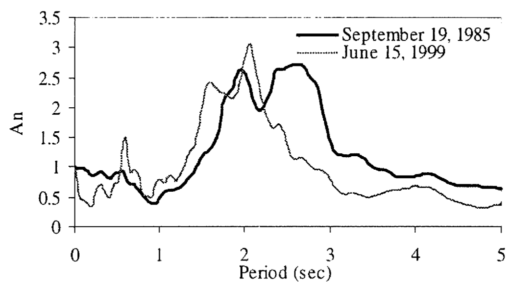

Figure 4.5 Nonlinear effects on ground motions

Another geotechnical factor that is usually neglected when designing foundation-building systems is the influence that nonlinear soil behavior has on the ground motion characteristics. Indeed, instrumental information as well as laboratory test results show that soil stiffness decreases and soil damping increases as the intensity of the seismic excitation grows. Accordingly, the natural period of a soil deposit will become larger and the intensity of ground motions will be attenuated. Therefore, when defining the input excitation for the aseismic design of a foundation-building system these two aspects should be accounted for. The amount of soil softening and motion damping are a function of soil type, being, in general, larger for sands and lower for highly plastic clays. As an example of the nonlinear-soil behavior effect, figure 4.5 compares the empirical amplification functions of figure 4.1c [(Sa(surf)/Sa(bed)], normalized by their corresponding amplitude at period T=0, yielding the normalized amplification functions A_n. These functions distinctively show that higher periods and lower peak amplitudes develop as the severity of the seismic motion increases. This is a clear manifestation of the nonlinear behavior that even the extremely-high-plastic clays of Mexico City may undergo when subjected to severe motions.

Conceptually, the effects of nonlinear behavior of soils may be beneficial. Indeed, one of the lessons derived from the behavior of buildings on embedded stiff foundations, during the September 19, 1985 seismic event, was that soil foundation plastic deformations acted as an energy-dissipation mechanism that in many cases limited building damage [Romo, 1990]. On the other hand, constructions on flexible foundations were more susceptible to damage and in several instances were a direct cause of building collapses, because their bases twisted and bended. Similar conclusions have been reached by Trifunac and Todorovska [1998] for the Los Angeles-Santa Monica region during the 1994 Northridge earthquake. These data seem to indicate that a strong foundation coupled with the soil nonlinear effects may be able to mitigate the damages a building might undergo in severe seismic events. As will be discussed later, stiff foundations, particularly when they are deeply embedded, enhance the nonlinear response of soils due to kinematic interaction. This may be viewed as if the foundation were acting as an active isolation mechanism that helps to limit building damages [Romo, 1990; 1995]. Of course, the premises from which this proposition is inferred should be further investigated.

The influence of surface topography on ground motions is not well understood. In many cases important discrepancies between theoretical and observed responses in a number of hills support this statement. Whatever the physical explanation for this, continued research involving field dense arrays to obtain earthquake recordings coupled with geophysical and geotechnical surveys, would improve our knowledge on this problem and would generate a wealth of information to evaluate existing numerical techniques [Bard, 1999].

4.1.2 Analytical procedures

Advances in numerical methods and computational capabilities allow, at least in principle, evaluation of free field-site specific motions using a model that includes the source of the seismic event. However, the limited knowledge regarding source parameters and regional geology makes the solution of this problem highly uncertain in the frequency range of interest for foundation designs. Therefore, most current procedures of site response analysis attempt to forecast time and spatial variations of ground motions from a single specified seismic environment acting at some control point within the site.

Analytical procedures to study ground response have evolved from one dimensional to three dimensional approaches. Linear, piece-wise linear and true nonlinear soil modeling is presently feasible. In principle, it is then possible to analyze any practical problem. However, despite the impressive advances in numerical-computing capabilities, it seems unlikely that most of these tools will soon reach the practitioner. The main restrictions of 2- and 3-D techniques to find their way to practical applications are that it is very difficult (if not impossible) to accurately define three (and even two) dimensional geometries and soil properties of a specific valley, and there is not a universally accepted procedure to quantify the duration of the motion and to incorporate it in earthquake design problems. Both aspects should be addressed and work in these areas should be identified.

Because of their simplicity and the experience accumulated by their ample use throughout the years, the 1-D procedures are the most commonly used in engineering practice to evaluate site-specific ground motions. For wide valleys with relatively shallow deposits, where material stiffness increases with depth, the assumption of vertically propagating seismic waves through horizontally layered deposits is reasonable. This has been demonstrated by a large number of cases where linear and piece-wise continuous one dimensional approaches have reproduced with reasonable accuracy recorded ground motions on a wide variety of soil materials [i.e. Rosenblueth, 1952; Idriss and Seed, 1968; Romo and Jaime, 1986; Seed et al, 1994]. To overcome some difficulties with equivalent linear soil response methods to model strong shaking, one dimensional nonlinear time domain procedures have been used particularly when earthquake-induced pore water pressures (and their dissipation) play an important role in the seismic behavior of soil deposits [i.e. Martin, 1975; Lee and Finn, 1991; Li et al, 1992].

Seismologists and earthquake engineers have devoted great efforts to investigate the influence of boundary conditions on the seismic response of confined valleys. Their analytical results have shown that if the bottom of the valley is concave (upwards) and surface waves develop at the valley-boundary edge, then surface ground motions may be amplified and duration increased [i.e. Aki, 1988 and 1993]. Many analytical procedures have been developed over the last three decades to compute the response of valleys with simple geometries and homogeneous materials [i.e. Trifunac, 1971; Sanchez-Sesma, 1983]. To account for irregularities in the valley geometry and soil inhomogeneties, numerical methods such as finite differences, finite elements, spectral elements or hybrid procedures have to be used [i.e. Alterman and Karal, 1968; Lysmer and Drake, 1971; Sánchez-Sesma, 1983; Bielak et al, 1991; TRISSE, 1999].

4.2 Soil-Structure Interaction Effects

The fundamental objective of a dynamic soil-structure interaction study is to estimate the motions of one or more foundation buildings at a specific site, from a known free field seismic environment. Accordingly, a complete interaction analysis necessarily involves firstly the determination of the temporal and spatial variations of the free field motions and secondly, the evaluation of the motions of the foundation-building system placed in the free field seismic environment.

The interaction between a vibrating foundation-building system and its supporting medium produces basically two mechanisms that modify free field ground motions. One is due to the base shears and overturning moments induced by the structure's own vibration which, in turn, give rise to soil deformations of increasing magnitude as soil compressibility becomes higher. This mechanism is usually referred to as inertial interaction. The other, known as kinematic interaction, develops when any or a combination of the following conditions exist: i) embedded foundation elements are stiffer than the surrounding soil, ii) inclined wave trains impinge on the foundation, and iii) ground motions are incoherent. The influence of these two interaction mechanisms can be analyzed using either the substructure (impedance or continuum) technique or the complete (direct) approach.

In the substructure procedure the soil-structure system is usually divided in two parts: i) a finite region which encircles all the geometric irregularities, the structure, and the nearby soil that might experience inelastic behavior, and ii) the half space that is outside of the generalized soil-structure interface, that is modeled with frequency dependent impedance functions. See Gazetas [1991] for a complete account of these functions.

On the other hand, the complete method incorporates the soil and the foundation-building system in a sole model, which is usually developed by means of finite elements, and analyzed simultaneously. It is important to recognize that, as it is not feasible to cover the complete layered half space with discrete elements, an artificial boundary should be included in the model to account for the missing layered medium on the exterior of the interaction region. Artificial boundaries may reflect, into the foundation-building system, appreciable amounts of the outwardly propagating waves energy. To minimize this energy-reflection problem and at the same time to keep the model within a reasonable size, numerous energy-absorbing boundaries have been brought about [i.e. Lysmer and Waas, 1972; Kausel, 1974].

Substructure and complete finite-element procedures are equivalent and if implemented consistently, identical results should be obtained. Thus, both approaches are capable of capturing the relevant issues of soil-structure interaction phenomena. Taking advantage of this fact, many researchers have used finite element methods with transmitting boundaries to analyze the layered half space and develop various alternatives for the substructure methods. For embedded foundations, this method can handle the problem via the rigid boundary [Kausel and Roesset, 1974; Luco et al, 1975], flexible boundary [Gutiérrez, 1976] and flexible volume methods [Lysmer, 1978]. Of the three approaches, the last one seems to be the most efficient [Tabatabaie-Raissi, 1982].

Most of the theoretical developments have been implemented in computer codes. Among the most known and used to analyze soil-structure problems are FLUSH [Lysmer et al, 1975], SASSI [Lysmer et al, 1981; Ostadan, 1983] which use finite-element methods and CLASSI [Luco et al, 1989] that makes use of boundary elements to compute foundation-soil impedances. Wolf and Darve [1986] developed a procedure that uses boundary elements for an elastic layered far-field region together with a nonlinear model of the soil and structure near-field zone.

A comprehensive investigation, supported by the European Commission, included the development of mathematical models, formulation of practical guidelines and laboratory tests on large scale soil-foundation models [TRISSE, 1999]. As a result of this project, an hybrid mathematical model that combines the spectral and finite element spatial discretization techniques was developed and encoded in the numerical tool named AHNSE. It can handle three-dimensional problems of wave propagation and soil-structure interaction. Also it accounts nonlinear soil behavior. This computational tool is capable of modeling the complete seismic problem that spans from seismic source to structural response.

The predicting capabilities of many of the above mentioned procedures have been evaluated throughout comparisons with actual seismic responses of soil-structure systems. In what follows, a case of a building on soft clay is presented as an example of the importance that seismic instrumentation and monitoring has on the final model development stages and evaluation of its reliability on building response calculations. Well documented case histories provide useful information that can be used to evaluate building and soil material properties by solving the inverse problem. It should be stressed, however, that because only a small number of locations are usually monitored compared with the degrees of freedom in the soil-structure system, identification of material characteristics corresponds to an indeterminate problem, and any resolution is a best fit to the data in one sense or another.

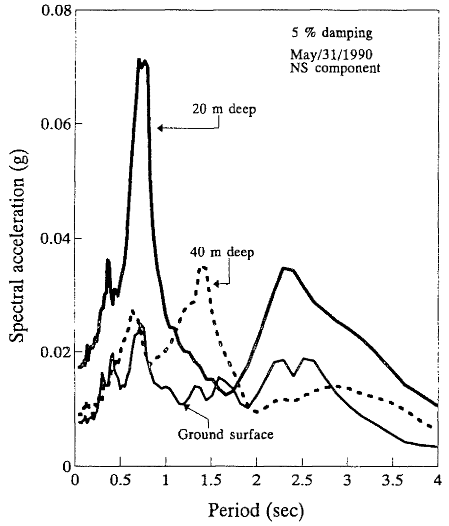

Figure 4.6 Motions recorded at building site

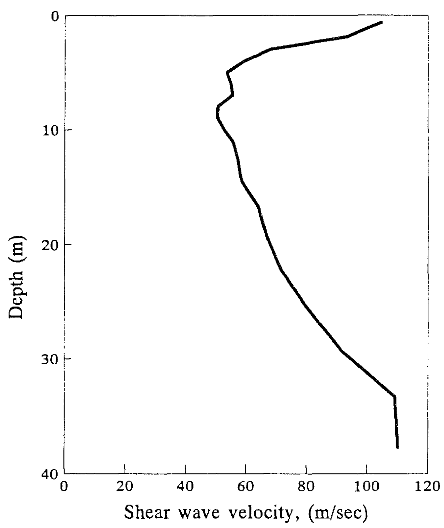

Figure 4.7 Site shear wave profile

The case history refers to Bernardo Quintana building in Mexico City. It is an eight story concrete structure supported by a rigid box that is embedded about eight meters into the soil deposit. Although the 1985 Michoacán event did not caused structural damage whatsoever, it was necessary to bring up the foundation and structure to the 1987 Federal District Building Code. Ambient vibration studies performed before and after the rehabilitation works showed that the foundation-building frequency increased from 1.11 to 1.68 Hz in the transversal direction and from 0.86 to 1.19 Hz in the longitudinal one [Rodríguez, 1992]. A vertical array of accelerometers was installed some 10 m from the foundation.

The response spectra of the recorded motion during a mild earthquake are included in figure 4.6. It may be noted that the spectral ordinates are increased as seismic waves move upwards from 40 m to 20 m. However, from this depth to the ground surface they are significantly attenuated. This attenuation is very significant and for some period intervals, surface motions are even lower than the corresponding motions at 40 m of depth. This significant attenuation has been shown to be due to the kinematic interaction developed between the deep box foundation and the surrounding soil [Romo and Bárcena, 1994]. It is worth mentioning that for shallow raft-type foundations it has been observed that not only the attenuation effects are much less significant, but free field ground motions may be amplified at some frequency intervals [Romo, 1991].

In order to evaluate the capabilities of a finite element random procedure, the soil-foundation-building system was analyzed using as input motion the acceleration response spectrum of the movements recorded at 40 m deep. The input control point and the boundary between the discrete model and the half space were considered at this depth. A local shear wave velocity profile is given in figure 4.7. The equivalent approach was used to account for any nonlinear behavior that could had developed, particularly near the foundation-soil interface. Figure 4.8 shows comparisons between observed and computed response spectra at ground surface and at 20 m of depth.

The results indicate that when the response-controlling parameters are properly modeled, finite element procedures (and others) may represent valuable tools to evaluate the influence of soil-structure interaction on free field ground movements and define the input motion for building seismic analysis. Foundation engineers may, indeed, take advantage of analytical procedures to design the most appropriate foundation system for soil and building specific conditions.

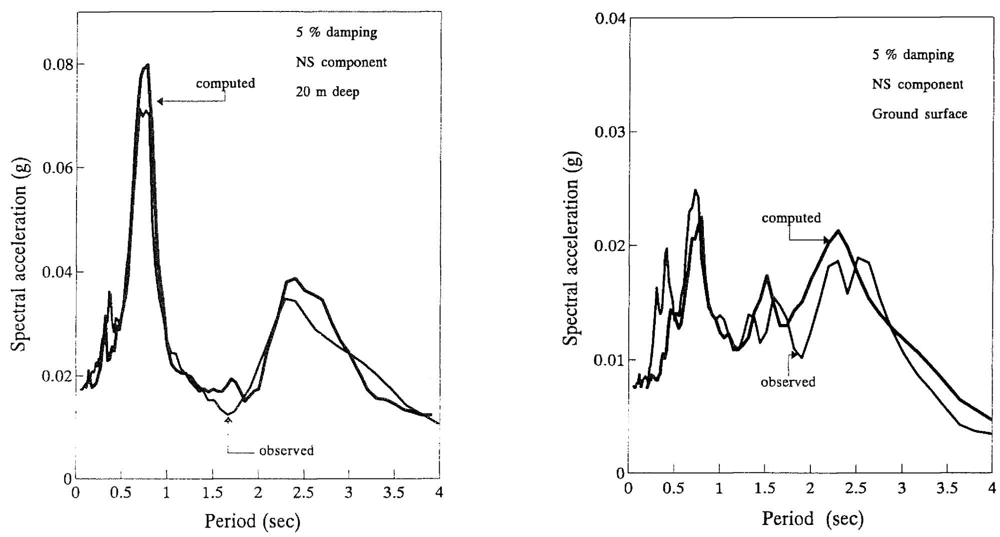

Figure 4.8 Measured and computed responses at building site for May 31, 1990 earthquake

From analytical studies with this finite element procedure of structures founded on the soft clays of Mexico City, the following general conclusions have been reached: i) for box-type foundations, the deeper the embeddment is the greater the attenuation effects on free field motions are; ii) friction pile-raft foundations have a negligible influence on the horizontal component of free field ground motions, however as indicated in section 6.2, piles modify significantly the vertical ground motion; iii) deep box foundations have a similar effect regarding building-input seismic energy attenuation as building-base isolators, thus it seems that kinematic interaction effects may be taken to the advantage of the foundation designer to mitigate, in a natural way, the severity of the motions the building might be subjected to during severe seismic events; iv) as the soil nonlinear effects become more significant, the building-base isolation effect is more notorious and so, the motion attenuations are larger; and v) it seems that the interaction-attenuation effects are more significant at periods similar to the rigid-base natural period of the structure.

5. SEISMIC DESIGN ISSUES

5.1 General Considerations

The design of a foundation involves in general a process that includes the following steps:

- Project information on superstructure, restriction on settlements, performance requirements
- Specification of geological and geotechnical environment
- Site investigation, idealization of the stratigraphy and determination of soil properties
- Definition of applied loads, including the seismic forces
- Visualization of the type of foundation, or diagnosis of the problem
- Design analysis and safety verifications according with building codes
- Considerations on excavation and construction procedures.

In fact this process is not carried out as a unique sequence, but it has a cyclic and iterative nature. The goal is to provide, in a technical and economic way, and from the point of view of construction, a foundation system, fulfilling the functional and seismic environmental requirements. Design analysis involves calculations that are carried out to assess the likely performance of the visualized foundation. If this predicted behavior is not in agreement with the requirements, some characteristics of the initial guess of foundation, such as size or amount of piles, are modified and the above mentioned steps should be repeated.

Besides of the uncertainties about the behavior of some foundation systems under seismic loads, other doubtful situations are faced for their design. Among them, uncertainties arise in finding adequate information about the subsoil conditions and its properties, as well as in the definition of the applied dynamic loads on the foundation (a highly random field of waves is generated during an earthquake). Thus, combining these uncertainties and the remarkable advances on analytical tools, it has been proposed to carry design analyses following three levels, in accordance with the complexity of the foundation problem [Pender, 1995]:

Level 1: Bearing capacity determination is carried out with traditional methods. For the transient displacement assessment during seismic loading, the soil is assumed to remain elastic.

Level 2: "Engineering" methods that involve the real behavior of cyclically loaded soil. The expected strain levels are accounted for on the soil stiffness. Insight for these methods comes from methods of levels 1 and 3. Special attention is paid to back analysis of the observed earthquake response of foundations.

Level 3: Full analysis is undertaken considering properly the dynamic loading, nonlinear soil properties, generation of dynamic excess pore pressure, strain softening, and the complexities of the soil-structure interaction.

The level 2 methods are potentially the most useful tools in solving geotechnical problems when designing typical buildings, although further development is needed precisely in the methods of this level. Difficulties in the full characterization of soil parameter values limit the practical use of complete analyses of level 3. The complexity of these rigorous theoretical approaches makes them accessible to researchers rather than to practicing engineers. Geoseismic instrumentation on prototypes, as it is highlighted herein, may be an excellent way to evaluate all these methods.

5.2 Assessment of Dynamic Bearing Capacity

Evaluations of foundation bearing capacity under seismic loading are usually performed with pseudostatic procedures built on conventional bearing capacity theories for sustained load [i.e. Richards et al, 1993; Pecker and Salençon, 1991; Sarma and Iossifelis, 1990]. In addition to superposing the pseudostatic-load system to the pre-earthquake forces acting on the foundation, most procedures also include the inertia of the potential sliding soil mass in the equilibrium analyses by multiplying the weight of this soil wedge times a seismic coefficient that is usually deduced from the peak ground surface acceleration likely to develop under the design earthquake. In the authors' opinion, use of maximum absolute ground accelerations to model earthquake-induced inertial effects, aside from being conservative, it is inconsistent with the physics of the problem. In fact, the purpose of considering inertia-like forces is to account, in a simple manner, the soil shear stresses caused by the passage of the seismic waves that superposed to the stresses induced by the total load system acting on the foundation conform the stress state to be used in bearing capacity evaluations. Now, it is known that in flexible materials the wave passage-induced shear stresses at any depth are given by the differential ground motions between the top and bottom of the soil element being considered. Thus, to comply with this fact, when modeling soil mass inertial effects by means of seismic coefficients, these should be computed from the relative motions of the upper most point on the failure surface with respect to the deepest point on this surface.

It becomes obvious that seismic coefficients computed in this fashion are bound to be much lower than those derived from total ground accelerations. Therefore, soil mass inertial contribution to the overall design force is less important and so it is its influence on "seismic" bearing capacity. Other aspects that are not considered by pseudostatic procedures and may affect bearing capacity evaluations, are related with soil nonlinear (and soil-foundation interaction) effects on the magnitude of building inertia forces, as discussed in chapter 4.

Pseudostatic methods are conservative as long as soil undrained strength does not drop due to earthquake loading. Accordingly, this potential loss on soil resistance should be accounted for when these procedures are used to evaluate foundation bearing capacity. A practical way to deal with this problem is to use in the capacity analyses the residual strength of the soils. Although this assumption may lead to conservative solutions, it is a safe one particularly when pre-earthquake shear stresses are larger than soil residual strengths.

Studies oriented to overcome some of these shortcomings have led to the establishment of an equation of motion for the building-foundation-sliding mass system, considered as a rigid body [Romo and García, 1994]. This method differs from all of the above in the sense that is a truly dynamic approach and thus capable of defining time-varying safety factors. Parametric studies have indicated that soil mass inertial effects are negligible unless the shear stresses induced bring the static safety factor below unity. Also, simple correlation between the static safety factor and earthquake-induced displacements were established [Romo and Díaz, 1997].

5.3 Earthquake-Induced Displacements

All procedures mentioned above to evaluate safety factors have been extended to compute permanent displacements due to seismic loading. All of them use a Newmark-type of approach for this purpose. Some of these procedures are now being used in practical applications without giving proper consideration to their limitations.

Indeed, there are a few issues that are not properly modeled by pseudostatic procedures. For example, it is known that the magnitude of the load affects the location of the critical failure surface, hence during seismic loading this position should hop around randomly. This poses a conceptual problem to pseudostatic methods because they implicitly assume one critical failure surface throughout shaking duration. In view of this phenomenon, it would seem that current procedures are bound to overestimate soil degradation and pore water pressures because cyclic shear stresses are considered to be acting on the same surface throughout earthquake shaking duration. Another aspect that may not be considered by pseudostatic approaches is the effect of building tilt as a consequence of foundation differential settlements induced by earthquake loading. This may turn out to be a significant shortcoming because P-delta effects increase the overturning moments that significantly affect overall stability (and permanent displacements) of the foundation [Romo and García, 1994]. Accordingly, initial adverse conditions caused by static differential settlements cannot be considered either. This limitation could be partially relieved by modifying the eccentricity of the load as a function of the building tilt, which would lead to an iteration procedure. Finally, available pseudostatic procedures are restricted to slab-type foundations.

In an attempt to overcome some of these shortcomings, Romo and García [1994] extended their approach to allow updating of the location of the potential failure surface, inclusion of P-delta effects and consideration of soil-pile interaction by means of tangential and normal springs. Parametric analyses with this approach have indicated that the most dominant parameters are the static factor of safety, building height and the time-varying horizontal building inertia. It was found that there exist thresholds for each of these parameters that define whether the foundation settlements would be approximately uniform, or large differential settlements that might lead to a general soil failure would develop. From the sensitivity analyses it was possible to establish that number and length of piles, depth of foundation embedment and soil strength help to decrease foundation settlements and make them more uniformly distributed. Despite this procedure follows more closely the physics of the problem and has yielded acceptable results when compared with actual cases, there are still many unanswered questions that warrant further investigations.

5.4 Pile Foundation

In Pender [1995] a thorough review of the many procedures for the analysis of pile foundations and design issues is given. Therefore, only few additional comments are included herein.

When dynamic analyses of piles are performed, it is important that modeling includes the following aspects: i) the variations in soil properties with depth, ii) the nonlinear stress-strain behavior of soil at pile-soil interface, including soil gapping, slippage along this interface and liquefaction, iii) the changes in ground motions with depth, iv) the three dimensional displacement pattern when pile pushes into the soil, and v) the effect of geometric damping.

In their state-of-the-art summary, Martin and Lam [1995] conclude that beam on nonlinear Winkler foundation models are adequate for most practical situations. Since the pioneering work of Matlock et al [1978] that developed the computer code SPASM8, many researchers have contributed in the improvement of this procedure [i.e. Bray et al, 1995].

Most of this class of methods considers the pile as a linear elastic beam element. To overcome this limitation, advanced structural analysis have been used to incorporate the nonlinear response characteristics of the pile into the soil-pile system [Prakash et al, 1993].

5.5 Reliability

Perhaps the most important objective in foundation design is to assure foundation safety and reliability. Its design is usually formulated under conditions of uncertainty because the loading process, and structural and soil properties are usually not known with precision. Also the information and the relationships utilized in the design process are in most situations approximate. In the face of such uncertainties, complete assurance of safety and reliability would be difficult to achieve.

Uncertainty has traditionally been considered implicitly through the use of a factor of safety (or load resistance factors and resistance reduction coefficients). However, since safety factors results from engineering judgement, it is a difficult matter to define what constitutes an adequate safety factor. Therefore, safety and reliability may be assured only with a tolerable risk or probability of failure.

Over the life of a foundation, one or more potential models of failure may be critical to its safety and reliability. Accordingly, within its useful life such modes of failure should be examined. In this regard, the lifetime maximum load would be of special concern in the evaluation of its safety against major damage or collapse, whereas the operational loads would be of importance in considering cumulative damage, such as fatigue, within the anticipated design life [Ang, 1973]. Reliability evaluations carried out by Auvinet and Rossa [1991] for compensated and friction pile foundations in Mexico City clays, have shown that they present a low reliability, mainly for slender buildings. This agrees, at least qualitatively, with the high damage endured by friction-pile-foundations in Mexico City [Mendoza and Auvinet, 1988], during the September 19, 1985 Michoacan seismic event.

Because of the practical difficulty of determining the correct probability distribution, nonparametric reliability methods have been developed [i.e. Rosenblueth and Esteva, 1971], where the concept of safety index has been introduced as the sole measure of reliability. For the purpose of formulating consistent code provisions for design, where the required level of safety of foundation components can be calibrated [Lind, 1971] with existing codes, the safety index would be sufficient to provide the necessary consistency in the code format.

5.6 Approaches and provisions given by building codes

After the analysis and design of a foundation have been concluded, its safety should be verified according with codes and standards provided by a governmental agency, or any other responsible body. Knowing that in many instances a foundation is subjected to its most critical condition when an intense earthquake occurs, any code must explicitly include the pertinent provisions and verifications in order 1) to assure that a foundation can resist without damage minor earthquakes occurring several times during its operation life; and 2) to reduce to a very low probability the collapse of the foundation and structure, assuring so no loss of human lives, even for a major earthquake with small probability of occurrence.

Riddell and de la Llera [1996] consider that although one should not expect codes to guarantee this survivability condition, according to the current state of knowledge, rational procedures should be available to ensure safety against collapse. It is clear that in general the level of protection is linked to economy, and the society of each country, region or city should define the price willing to be paid to ensure its protection. However, to the best of the authors' knowledge, none of the codes in the world has an explicit cost-performance approach. That level of protection is usually determined through the occurrence of seismic events. In accordance with the response of foundations of a region to earthquakes, the code regulations are evaluated and adjusted. Usually, when failures and the degree of damage is not acceptable, the provisions are indurated, with the consequent increment in cost. In these cases, code-policy-makers should be open to make the necessary code-adjustments as results of new investigations and experiences on various subjects of earthquake engineering become available. Consequently, a construction code should be a flexible-dynamic norm instead of an immovable mandatory document.

Nowadays, there is a worldwide tendency toward unified codes where the safety factor as a measure of stability is being replaced by load and resistance factors. The latter reduces the bearing capacity of the foundation system and the former increases the design loads. Although there are some differences, most modern codes specify that the reduced bearing capacity of the foundation be higher than the factored combination of loads. In addition to verifying limit states of failure, in most modern codes limit states of serviceability must be satisfied when designing a foundation.

Code design spectra consider elastic soil behavior and are specified at the surface of the free field. It seems that only the Uniform Building Code [in Seed and Moss, 1999] includes in its definition the influence of soil nonlinear behavior but none considers the seismic intensity attenuation with ground depth. These two aspects should be duly considered in forthcoming construction code updatings. Another step forward in improving seismic codes would be achieved by introducing the concept of performance-based seismic design. However, this is a major challenge because it implies reliable estimations of damage and displacements of the structural system, and both are strongly influenced by shaking duration.

It has been recently suggested that to properly account for the potential seismic threat for structures, 2D and 3D wave propagation effects should be included when defining seismic design spectra [TRISSE, 1999]. Similarly, in dense urban areas on soft ground deposits, the seismic risk of buildings may be increased by the surficial waves, generated by the interaction phenomenon, that superpose on the primary incoming train waves [Romo, 1991].

6. NEW TRENDS IN SEISMIC DESIGN ANALYSES

6.1 Centrifuge and Shaking Table Techniques

Centrifuge technology has evolved significantly during the last decade and has established itself as a reliable experimental technique to test geotechnical models. Much of this trust was gained throughout the coordinated research that was carried out within the VELACS project in the period 1989-1993.

It should be understood that regardless the technique used, a model does not represent all features of the prototype being considered during a particular design procedure study or under construction. However, it can provide indications about performance and contribute to determining different critical scenarios, that are of great value when designing foundations acted upon by static and dynamic loading. Thus, it is not surprising that earthquake centrifuge models are now contemplated in design processes for large-scale projects where the application of conventional earthquake approaches is uncertain. A recent case that illustrates the potential contributions of this testing technique to design analyses refers to the foundation of the Rion Antirion bridge in Greece [Garnier and Pecker, 1999].

The main use of centrifuge modeling has been oriented, though, to generate realistic data of the seismic response of a broad variety of foundations on sandy soils and clayey materials. Also, it has been widely used to validate numerical codes, particularly those that deal with dynamic effective stresses, soil degradation, ground failure and soil-structure interaction.

Shaking table technology has been around for many years and has been used to study the seismic behavior of prototype-scale foundation models. However, its popularity among geotechnical earthquake engineers decreased steadily from the early 80's to mid 90's. This had to do mainly with the problems posed by the reflection of waves at the lateral boundaries of the container model. To avoid spurious waves impinging in the foundation, laminar containers, which are an extension of those used in centrifuge testing, that closely reproduce level ground seismic conditions were developed. This single action has signified an important improvement that has spurred the use of shaking tables to study earthquake geotechnical problems. Recent applications to investigate pile-soil dynamic interaction effects [Meymand et al, 1999; Tsukamoto et al, 1999] and the response of gravity quay walls [Iai and Sugano, 1999] are clear examples of shaking-tables present capabilities.

6.2 Instrumentation of Foundations

For many years foundation analysis has been considered as one of the geotechnical problems better understood and, hence, easier to solve. However, recent seismic events [i. e. Mexico City, 1985; Loma Prieta, 1989; Northridge, 1994; and Kobe, 1995] have clearly shown that the seismic behavior of soil-foundation systems is far from being fully comprehended.

To meet the safety and cost requirements of a good design, the engineer must be able to quantify accurately the input loading, to evaluate properly soil behavior under this loading and to make reliable assessments of the soil-foundation system response.

Seismic loading acting upon a soil-foundation system results from the interplay of earthquake incoming waves with building-swaying-produced waves. The complex foundation vibration patterns that result from this interaction are difficult (if not impossible) to predict because they depend on many factors (that are interrelated) such as wave-train characteristics, building-foundation vibration patterns, soil-foundation interaction, soil behavior (elastic/inelastic), site geological and geotechnical characteristics, and pre-earthquake foundation conditions. Furthermore, in dense urban zones the incoming wave patterns can be modified as compared with commonly assumed isolated-single-foundation-building conditions, due to their interaction with waves radiating away from nearby soil-foundation systems.

Accordingly, if foundation seismic loads cannot be quantified adequately, reliable evaluations of soil behavior and soil-foundation response are, in principle, near-impossible tasks. Additionally, soil dynamic properties are usually determined from laboratory tests on nominal undisturbed element samples subjected to loads with simple wave forms and having boundary conditions far from matching the in situ ones.

Although there have been many important developments in analytical tools and laboratory testing techniques, there is still a wide gap between soil-foundation modeling and reality. Given the complex interplay among foundation-performance controlling factors, it seems that to improve our knowledge on these matters and narrow the modeling-reality breach a significant number of soil-foundation systems with varying characteristics should be instrumented with sensitive devices to measure loading effects on the foundation soil, on the various components of the foundation, and at the soil-foundation interface. It is important that continuous monitoring be carried out from the early construction stages and throughout foundation operation. Of course, a detailed account of loading sequence has to be recorded. Having a cause-effect continuous history, it would be possible to make a significant step forward in understanding the physics of the problem. Thus, any mathematical model derived from this type of information would necessarily be closer to reality.

Until now, a relatively small number of foundations have been instrumented worldwide. Most of the instrumentation have been designed to measure load-transfer mechanisms in piles, soil pore water pressures, settlements and soil-foundation contact pressures, under static or monotonically increasing loads. However, there are not many cases where these and other parameters can be also monitored during earthquake shaking. This is a serious limitation for foundations in seismic zones. In the next paragraphs the main lessons that have been drawn from the continuous monitoring of a box-friction-pile foundation on the Mexico City soft soil deposits are stated.

Soil conditions in Mexico City offer serious challenges to geotechnical engineers not only because of the low strength and high compressibility characteristics of the clayey materials, but also due to the general ground subsidence induced by water withdrawal from the relatively shallow aquifers. Thus, proper foundation designs for medium- to high-rise buildings that need deep foundations, have to satisfy overall stability and, at the same time, to minimize building emergence relative to the sinking adjacent ground. The former is achieved using long end bearing piles, but emergence is not avoided. Aside from aesthetical considerations, ground-foundation relative displacements bring about serious disruptions to building municipal services and important damages to nearby light buildings. Furthermore, large ground settlements may reduce the lateral capacity of piles at their upper part under dynamic loading, and increase pile vertical load due to negative skin friction.

To minimize the deleterious effects of ground subsidence, the concept of yielding piles was developed and applied by Mexican engineers ever since some 50 years ago [i. e. Zeevaert, 1957]. The basic idea was to use friction piles to reduce building settlements due to the overload and, at the same time, follow regional subsidence. This meant to design friction piles with near-to-one safety factors. Solutions of this type have been widely used with success in Mexico City for many years. Unfortunately, this solution being a clever answer for foundations subjected only to sustained vertical loads, it was not apt (when high slab-soil contact pressures existed) for earthquake loading as was dramatically exhibited by the September 19, 1985 seismic event [Mendoza and Auvinet, 1988]. From the many reasons that were brandished to explain the poor behavior of many box-friction pile foundations, it became evident the lack of a clear understanding as to how pile-soil-box load transfer mechanisms developed during sustained and dynamic loading. Accordingly, the instrumentation installed in one of the foundations of a urban bridge support was designed mainly to better our knowledge on this matter, but also to learn more about dynamic pore pressure generation in highly plastic clays. Additional lessons that are expected to be extracted from this case history are on earthquake-induced permanent settlements, soil-structure interaction effects and long-term foundation behavior. A detailed description of soil stratigraphy, foundation characteristics, geotechnical and seismic instrumentation peculiarities of this case history is given in Mendoza and Romo [1996, 1998] and Mendoza et al [1999]. These references also include the main lessons derived from this case history related to pre-earthquake conditions.

Since the end of bridge construction, three seismic events have been recorded at the foundation site. Two of them (January 11, 1997 and July 19, 1997) originated along the Pacific coast (18.09° Lat N, 102.86° Long W; 16.00° Lat N, 98.23° Long W, with magnitudes 7.3 and 6.3 respectively). The third earthquake was the Tehuacán event. Time histories of accelerations on the foundation and free field (ground surface and 60 m deep), of pile loads, slab-soil contact pressures and pore water pressures were recorded during shaking.

Fourier spectra of above mentioned earthquakes show that the horizontal orthogonal foundation responses are alike, although the bridge axis component is somewhat more intense. The shape of Fourier spectra of the vertical components are similar, in general, but have a higher frequency content. All spectra show a distinctive peak at 0.25 Hz that corresponds to the elastic natural period of the soil deposit [Mendoza et al, 1999].

To evaluate pile-slab load transfer mechanisms during shaking, slab-soil pressure cells were located as close as possible from instrumented piles. With the purpose of illustration, pile load and soil slab contact pressure time variations recorded on pile P41 and cell C1 are plotted in figure 6.1. From these time histories it is seen that during the action of the earthquake both pile and slab indeed carry some loading. Most interesting is that both time histories have similar traces: peaks and valleys show up practically at the same times. This indicates that pile and slab responses are in phase and support the hypothesis of considering slab and pile contributions in seismic design analyses of mixed-type foundations. For the case included, the maximum transient load variation on the pile was 50 kN which represents a little less than 10% of the pre-earthquake loading condition. The peak transient pressure oscillation was near 5 kPa which is equivalent to nearly 40% of the acting pressure before the seismic event. Although in absolute terms the pressure increase is not important, percentage wise it is significant. This points out that when the static design allows appreciable slab-soil contact pressures, during earthquake shaking these may exceed with relative ease the yielding stress of the soil causing, as a result, large settlements. Extreme seismic or static contact pressures may lead to slab bearing capacity failure. This is particularly risky when piles are designed to their limit capacity for foundation settlement control. Finally, it is worth pointing out that some piles showed a slight loading decrease during earthquake shaking, but after the seismic event ceased, they recuperated their pre-earthquake charge. The opposite phenomenon was observed on slab-soil contact pressures.

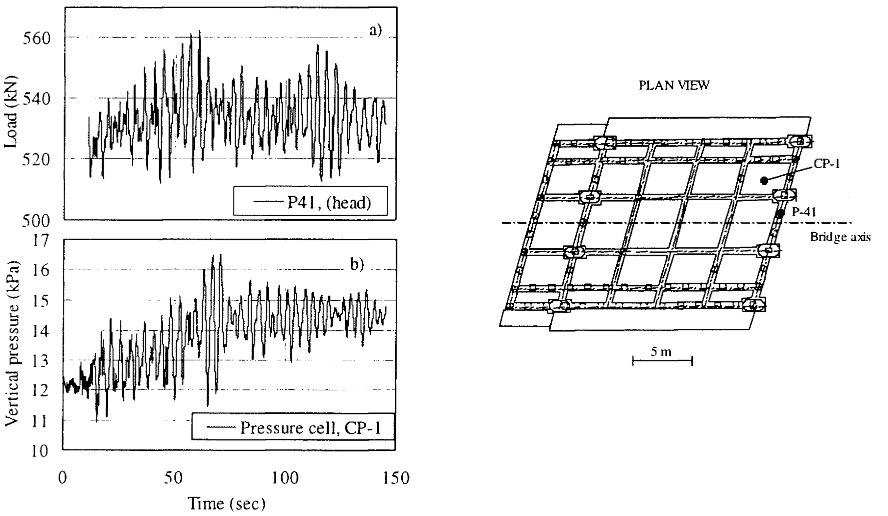

Figure 6.1 Pile load and raft-soil contact pressure time histories

As an illustration of the characteristics of earthquake-induced soil pore water pressures, in figure 6.2 their time variation throughout the earthquake is presented. This piezometer is located at a depth of 27 m in a clayey layer without a granular bulb, and its location in plan is noted in this figure. When compared with the accelerogram included in the figure, it is clear that accelerations (or equivalently shear stresses) and pore water pressures have time signatures almost identical. Also, no cumulative (an hence residual) pore pressures develop. These observations suggest that the soil responded in a quasi-linear fashion and lend support to previous studies in the sense that seismically-induced pore water pressures are of little concern when designing foundations on highly plastic clays, although for sensitive and low plasticity clays this may not be the case [Yasuhara, 1995]. From the free field and foundation responses (not shown herein) during the Tehuacán earthquake, it was clear that soil-foundation interaction effects on free field motions were negligible (as stated previously) in the horizontal components. However, the interaction influence was very important in the vertical component. For example, the vertical peak ground acceleration was attenuated by a factor of 0.14 by the presence of the foundation. This is understandable on the grounds that flexible piles follow horizontal ground movements, but in the vertical direction because of their larger stiffness strongly interact with the neighbouring soil, leading to significant amounts of energy dissipation along their pile shafts. Accordingly, it should be expected that important transient shear stresses would develop at pile-soil interfaces and, consequently, additional cyclic forces will be induced. These forces, that can reach high values in foundations near seismic sources, must be considered in the design of raft-pile foundations in earthquake prone zones.

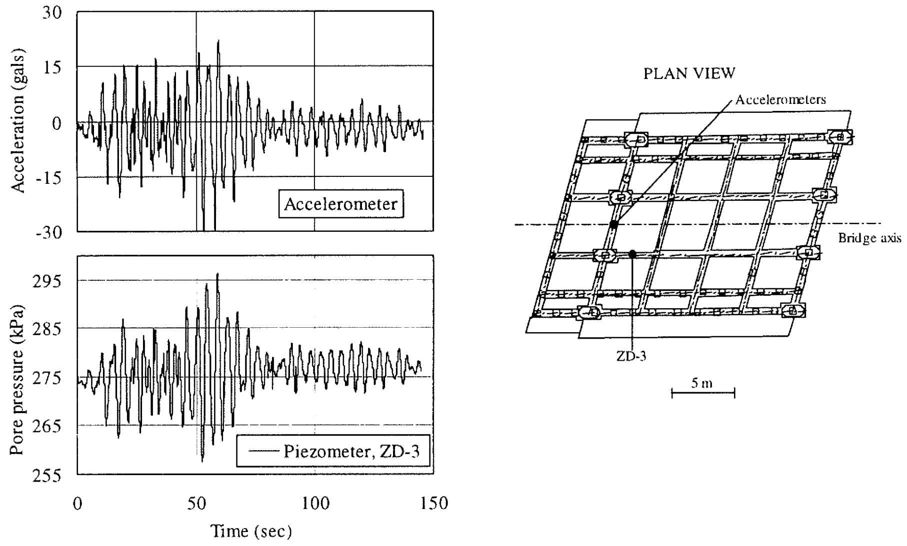

Figure 6.2 Acceleration and pore water pressure time histories

6.3 Knowledge-Based Procedures

There is a growing number of research engineers that are looking for new alternatives to solve earthquake-related problems. In the past, knowledge-based techniques such as expert systems have been used for this purpose. However, their success has been restrained because their output relies heavily on the knowledge of the experts consulted. Another option that has been rapidly emerging for the analysis of engineering problems is called Artificial Neural Networks (ANNs). It simulates the way Biological Neural Networks (BNN) learn and process the information that human body receptors perceive from the environment.

Engineering tasks are primarily concerned with analysis, design, system identification, diagnosis, prediction, control, planning and scheduling. These drudgeries may be classified as either mapping from cause to effect for estimation and prediction or inverse mapping from effects to possible causes. The nature of ANNs is to map from one space of input patterns to a space of output patterns. In this sense, ANNs are just another tool to solve these type of problems.

In a recent paper [Romo, 1999], it was shown that solution of complex problems related with earthquake geotechnical engineering is attainable. Results show that ANNs are, in general, more accurate predictors than analytical tools. This outcome is not fortuitous, but is backed up by the fact that knowledge-based procedures are universal approximators and as such they may be reasonably be considered as methods that will closely capture the laws of mechanics that the actual phenomenon obeys. Accordingly, no implicit assumptions are involved in developing ANNs as is required in analytical procedures. In this sense, knowledge-based procedures are potentially more reliable and general than mathematically-based numerical methods. As an example of the predictive capabilities of ANNs, comparisons between observed and computed response spectra at various sites in Mexico City and for different seismic events are included in figure 6.3. Matching is nearly perfect (correlation = 0.97). This case (among many others) makes evident the usefulness of field instrumentation as a needed procedure to originate information upon which alternate seismic analyses methods based on ANNs may be built up.

This technique has also been used to model the seismic response of clays, sands and gravels, to establish correlation between field-determined soil parameters (i.e. cone penetration resistance versus shear wave velocity), and to evaluate soil-foundation dynamic interaction, and so on. ANNs, coupled with fuzzy logic, can be used to develop control systems for active earthquake-energy-dissipator devices [Ghaboussi and Joghataie, 1995], earthquake-detection alarms, and structural damage identification, among many other earthquake-related problems.

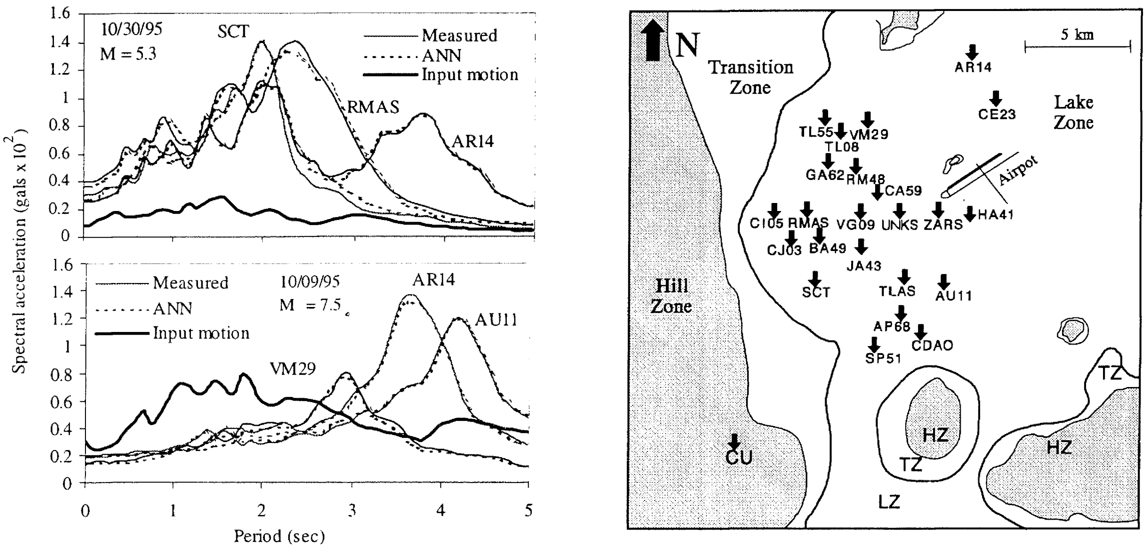

Figure 6.3 Measured and ANN predicted responses at various sites in Mexico City, as indicated in the geotechnical zoning

The success of ANNs resides in the fact that they can recognize patterns within vast data sets and then generalize them into recommended course of action. Thus, to develop a reliable ANN it is necessary to have a comprehensive database. In some instances when the information is limited, analytical tools can be used to enrich the database or a well designed ANN may be integrated into the analytical procedure to model, for instance, nonlinear behavior of soils.

6.4 Assessment of Soil Parameters from Ground Motions

Deployment of downhole arrays of strong motion instruments in many soil deposits and earth structures, has made possible the compilation of very valuable information containing, ideally, the input motion to the system and its response. These cause-effect data, coupled with system identification (SI) procedures, yield an alternative for the evaluation of soil dynamic properties. For rock-size particles, it seems that SI is the most plausible procedure to define their dynamic characteristics.

Although SI is conceptually a reliable approach, there are two major issues whose correct treatment is essential for the results to be of practical value. First, it is required that the finite set of parameters determines completely the mathematical model to be used. And, secondly, it is necessary to identify these parameters on the basis of the observed behavior of the system. It should be recognized that identification of nonlinear relationships between measured input and output of a given system is not an easy task. Accordingly, SI is usually focused on estimating a linearized model which is equivalent, in some sense, to the original nonlinear system. Instead of using a mathematical model, at present time ANN are beginning to be used as some sort of SI models.

There are some examples where SI has been used in earth and rockfill dams [i.e. Seed et al, 1973; Abdel-Ghaffar and Scott, 1979; Romo et al, 1981]. More recently, using the motions recorded in downhole arrays at Lotung site and Mexico City, a series of studies have been undertaken to asses the nonlinear response of soils with good results [i.e. Chang et al, 1991; Zeghal and Elgamal, 1995; Taboada et al, 1999]. Earthquake-induced liquefaction has also been studied [Zeghal and Elgamal, 1994].

7. CONCLUSIONS

It is recognized that analysis and design of foundations involve the interplay of a wide variety of aspects related with disciplines such as seismology, geology, soil and rock dynamics, and applied mechanics. Therefore, foundation engineers should be well aware of the advances and technical developments in these fields, or to be properly advised in these matters in order to accomplish cost-efficient and safe designs.

Even though there have been huge advances in most of the fields included in this keynote paper, there is still much to do in the development of procedures to evaluate seismic bearing capacity of, and earthquake-induced permanent displacements in shallow and deep foundations. Simple procedures to determine foundation seismic loading for practical applications are urgently needed. These may be developed from the available numerical methods that have proven to reproduce accurately the seismic response of instrumented prototypes.

Small-sample laboratory techniques have advanced at large strides mainly due to the transference of technology from control and electronic disciplines. However, there is room for improvement on the modeling of loading and boundary conditions existing in the prototype. Resurgence of shake tables and development of centrifuges have helped enormously in closing the representativeness gap, but they still face some technical limitations and the principles of similarity not always may be proved rigorously. These restraints must be clearly recognized by the end users.

In most countries the final design and construction of foundations have to comply with construction codes. Accordingly, these documents should incorporate the last developments in foundation earthquake engineering. However, it should be stressed that the reliability of any advance must be proved before it is included in code provisions. The present tendency in many countries is to produce unified codes where it is intended to harmonize all requirements in terms of safety, which represents a step forward in construction code development. Furthermore, the profession should make efforts to implement performance-based codes.

Improved knowledge about the mechanisms that govern the seismic response of foundations may be acquired from well instrumented prototypes. Monitoring systems should include equipment to record the cause-effect duality for static and seismic conditions. Evidently, this sort of information is of great value to asses the predicting capabilities of existing analytical tools and to develop new ones. Unfortunately, the scarcity of soil-foundation systems with proper seismic instrumentation has limited full achievement of these tasks.

Information gathered from several case histories has led to the integration of comprehensive data bases that are being used in knowledge-based procedures such as artificial neural networks to model phenomena related to foundation response behavior. In view that neural networks are universal estimators, it may be argued that well designed networks (aside from being more efficient) are potentially more accurate than analytical tools that necessarily include simplifying assumptions compelled by the phenomenon-comprehension level of the developer. Similarly, this cause-effect duality is being used to evaluate soil dynamic properties using SI techniques.

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9. REFERENCES

ICRAGEESD = International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Rolla, Prakash S. (ed.), University of Missouri-Rolla.

IS-Hokkaido'94 = International Symposium on Pre-failure Deformation Characteristics of Geomaterials, Sapporo Hokkaido, Shibuya, S., Mitachi, T. and Miura, S. (eds.), Balkema.

ICEGE = International Conference on Earthquake Geotechnical Engineering, Lisboa, Sêco e Pinto, P. (ed.), Balkema.

Abdel-Ghaffar, A.M. and Scott, R.F. (1979), "Shear moduli and damping factors of earth dam", *Journal of the Geotechnical Engineering Division*, ASCE, 105, GT12, pp1405-1426.

Aki, K. (1988), "Local site effects", *Earthquake Engineering and Soil Dynamics II*, Von Thum, J. L. (ed.), Geotechnical Special Publication No. 20, ASCE, pp103-155.

Aki, K. (1993), "Local site effects on weak and strong ground motions", *Tectonophysics*, 218, 1-3, pp93-111.

Alterman, Z.S. and Karal, F.C. (1968), "Propagation of elastic waves in layered media by finite difference methods", *Bulletin of the Seismological Society of America*, 58, pp367-398.

Ang, A.H-S (1973), "Structural risk analysis and reliability-based design", *Journal of the Structural Division*, ASCE, 99, ST9, pp1891-1910.

Auvinet, G. and Rossa, O. (1991), "Reliability of foundations on soft soils", *Proceedings, CERRA-ICASP6*, 2, Mexico City, pp768-775.

Auvinet, G. (1999), Personal Communication.

Bard, P. Y. (1999), "Strong motion and site effects: General Report", *2nd ICEGE.*

Bielak, J., MacCamy, R.C., McGhee, D.S. and Barry, A. (1991), "Unified symmetric BEM-FEM of site effects on ground motions – SH waves", *Journal of Engineering Mechanics*, ASCE, 117 (10), pp2265-2285.

Bray, J.D., Espinoza, R.D., Soga, K. and Taylor, R. L. (1995), "GeoFeap: Geotechnical finite element analysis program", *Report No. UCB/GT/95-05*, University of California, Berkeley.

Bray, J.D., Riemer, M.F. and Gookin, W.B. (1999), "On the dynamic characterization of soils", *Proc. 2nd ICEGE*, 3, pp847-856.

Burland, J.B. (1989), "Small is beautiful-The stiffness of soil at small strains", Ninth Laurits Bjerrum Memorial Lecture, *Canadian Geotechnical Journal*, 26, 4, pp499-516.

Burland, J.B. (1995), "Closing ceremony", *Proc. 1st IS-Hokkaido'94*, 2, pp703-705.

Clarke, J. (ed.) (1992), *Large-scale pile tests in clay*, Thomas Telford, London.

Chang, C.-Y., Mok, C.M., Power, M.S., Tang, Y.K., Tang, H. T. and Stepp, J.C. (1991), "Development of shear modulus reduction curves based on Lotung down-hole ground motion data", *2nd ICRAGEESD*, 1, pp111-118.

Dobry, R. and Vucetic, M. (1987), "Dynamic properties and seismic response of soft clay deposits", *Proc. Int. Symp. on Geotechnical Engrg. of Soft Soils*, Mexico City , Mex. Soc. for Soil Mechanics, 2, pp48-85.

Gajo, A. and Mongiovi, L. (1994), "The effect of measure accuracy in the interpretation of dynamic tests on saturated soils", *Proc. IS-Hokkaido'94,* 1, pp163-168.

Garnier, J. and Pecker, A. (1999), "Use of centrifuge tests for the validation of innovative concepts in foundation engineering", *2nd ICEGE,* 1, pp433-439.

Gazetas, G. (1991), Foundation vibration, Chapter 15, *Foundation Engineering Handbook*, Fang, H.-Y. (ed.), Van Nostrand, 2nd edition.

Ghaboussi, J. and Joghataie, A. (1995), "Active control of structures using neural networks", *Journal of Engineering Mechanics*, ASCE, 11, 4, pp555-567.

Gutiérrez, J.A. (1976), "A substructure method for earthquake analysis of structure-soil interaction", *Report No. EERC/79-9*, University of California, Berkeley, April.

Hardin, B.O. and Black, W.L. (1969), Closure to "Vibration modulus of normally consolidated clays", *Journal of Soil Mechanics and Foundations Division*, ASCE, 95, SM6, pp1531-1537.

Hardin, B.O. and Drnevich, V.P. (1972), "Shear modulus and damping in soils: Design equations and curves", *Journal of Soil Mechanics and Foundations Division*, ASCE, 98, SM7, pp667-692.

Iai, S. and Sugano, T. (1999), "Soil-structure interaction studies through shaking table tests", *2nd ICEGE,* 3, pp927-940.

Idriss, I.M. and Seed, H.B. (1968), "Seismic response of horizontal soil layers", *Journal of Soil Mechanics and Foundations Division*, ASCE, 94, SM4, pp1003-1031.

Idriss, I.M., Dobry, R. and Singh, R.D. (1978), "Nonlinear behavior of soft clay during cyclic loading", *Journal of Geotechnical Engineering Division*, ASCE, 104, GT12, pp1427-1447

Isenhower, W.M. and Stokoe, K.N. (1981), "Strain-rate dependent shear modulus of San Francisco Bay Mud", *Proc. 1st ICRAGEESD*, 2, pp597-602.

Kausel, E. (1974), "Forced vibrations of circular foundations on layered media", *Soils Publication No. 336*, Department of Civil Engineering, MIT, Cambridge, Massachusetts, January.

Kausel, E. and Roesset, J. (1974), "Soil-structure-interaction problems for nuclear containment structures", *ASCE Power Division Specialty Conference*, Denver, August.

Kramer, S.L., von Laum, F.Y. and Sivaneswaran, N. (1992), "Strain-controlled, variable frequency cyclic loading system of soft soils", *Geotech. Testing Journal*, ASTM, 15, 3, pp264-270.

Lee, M.K.W. and Finn, W.D.L. (1991), "DESRA-2C: Dynamic effective stress response analysis of soil deposits with energy transmitting boundary including assessment of liquefaction potential", University of British Columbia, Faculty of Applied Science, 34 p.

Li, X.S., Wang, Z.L. and Shen, C.K. (1992), "SUMDES: A nonlinear procedure for response analysis of horizontally-layered sites subjected to multi-directional earthquake shaking", Dept. University of California, Davis, 81 p.

Lind, N.C. (1971), "Consistent partial safety factors", *Journal of the Structural Division*, ASCE, 97, ST6, pp1651-1670.

Lo Presti, D.C.F., Pallara, O. and Puci, I. (1995), "A modified commercial triaxial testing system for small strain measurements: Preliminary results on Pisa clay", *Geotech. Testing Journal*, ASTM, 18, 1, pp15-31.

Luco, J.E., Wong, H.L. and Trifunac, M.D. (1975), "A note on the dynamic response of rigid embedded foundations", *Earthquake Engineering and Structural Dynamics*, 4, pp119-128.

Luco, J.E. and Wong, H.L. (1982), "Response of structures to non-vertically incident seismic waves", *Bulletin of the Seismological Society of America*, 72, 1, pp265-302.

Lysmer, J. and Drake, L.A. (1971), "The propagation of Love waves across non horizontally layered structures", *Bulletin of the Seismological Society of America*, 61, pp1233-1252.

Lysmer, J. and Waas, G. (1972), "Shear waves in plane infinite structures", *Journal of Engineering and*

Mechanics Division, ASCE, 98, pp85-105.
Lysmer, J., Udaka, T., Tsai, C.-F. and Seed, H.B. (1975), "FLUSH: A computer program for approximate 3-D analysis of soil-structure interaction problems", *EERC-UCB Report 75-30*, University of California, Berkeley.
Lysmer, J. (1978) "Analytical procedures in soil dynamics", *Report No. EERC 78/29*, University of California, Berkeley, December.
Lysmer, J, Tabatabaie-Raissi, M., Tagirian, F., Vahdami, S. and Ostadan, F. (1981), "SASSI: A system for analysis of soil-structure interaction", *UCB/GT81-02*, University of California, Berkeley.
Martin, G.R. and Lam, I.P. (1995), "Seismic design of pile foundations: structural and geotechnical issues", *3rd ICRAGEESD*, 3, pp1491-1515.
Martin, P. (1975), "Nonlinear methods for dynamic analysis of ground response", PhD dissertation, Department of Civil Engineering, University of California, Berkeley.
Matlock, H., Foo, S., Tsai, C-F and Lam, I.P. (1978), "SPASM8: A dynamic beam-column program for seismic pile analysis with support motion", Fugro Inc.
Mendoza, M.J. and Auvinet, G. (1988), "The Mexico earthquake of September 19, 1985-Behavior of building foundations in Mexico City", *Earthquake Spectra Journal*, 4, 3, pp835-853.
Mendoza, M.J. and Romo, M.P. (1996), "Geoseismic instrumentation of a friction pile-box foundation", CD-*Proc. 11th World Conference on Earthquake Engrg.*, No. 1892, Acapulco.
Mendoza, M.J., Gutiérrez, C.A. and Domínguez, L. (1997), "Dynamic shear modulus of soils determined in the laboratory (resonant column) and the field (suspension logging)", *Proc. 11th Nat. Congress on Seismic Engineering*, Veracruz-Mexico, Mexican Soc. for Earthquake Engng., 1, pp637-648 (in Spanish).
Mendoza, M.J. and Romo, M.P. (1998), "Performance of a friction pile-box foundation in Mexico City", *Soils and Foundations*, 38, 4, pp239-249.
Mendoza, M.J., Romo, M.P., Orozco, M. and Domínguez, L. (1999), "Observed seismic response of a friction pile-box foundation", *Proc. 2nd ICEGE*, 1, pp395-400.
Meymand, P.J. Lok, T.M., Pestana, J.M., Riemer, M.F. and Seed, R.B. (1999), "Large-scale shaking table model tests and analyses of seismic soil-pile-superstructure interaction", *Proc. 2nd ICEGE*, 1, pp341-352.
Ohta, Y. and Goto, N. (1976), "Estimation of S-Wave velocity in terms of characteristic indices of soil", Butsuri-Tanko (*Geophysical Exploration*; in Japanese), 29, 4, pp34-41.
Ostadan, F. (1983), "Dynamic analysis of soil-pile structure systems", Ph.D. Dissertation, University of California, Berkeley.
Ovando, E. and Romo, M. P. (1990), "Evaluation of Mexico City clay shear wave velocities from cone penetration tests", *Sismodinámica*, 2, pp107-123 (in Spanish).
Pecker, A. and Salençon, J. (1991), "Seismic bearing capacity of shallow strip foundations on clay soils", *Proc. of the Intl. Workshop on Seismology and Earthquake Engng.*, CENAPRED, Mexico City, pp287-304.
Pender, M.J. (1995), "Earthquake resistant design of foundations", Keynote address: *Pacific Conf. on Earthquake Engrg., PCEE95*, Melbourne.
Prakash, V., Powell, G. and Campbell, S. (1993), "DRAIN-2DX: Base program description and user guide", *Report No. UCB/SEMM-93/M*, University of California, Berkeley.
Richards, R.Jr., Elms, D.G. and Budhu, M. (1993), "Seismic baring capacity and settlements of foundations", *Journal of Geotechnical Engineering.*, ASCE, 119, 4, pp662-674.
Riddell, R. and De la Llera, J.C. (1996), "Seismic analysis and design: Current practice and future trends", CD-*Proc. 11th World Conference on Earthquake Engineering*, Acapulco, Paper No. 2010, Elsevier.
Rodríguez, N. (1992), "FICA, Activities Report 1987-1992", Fundación ICA (in Spanish).
Romo, M.P., Ayala, G., Reséndiz, D. and Díaz, C. (1981), Response analysis of El Infinernillo and La Villita dams, Chapter 6, 15, Federal Commission of Electricity.
Romo, M.P. and Jaime, A. (1986), "Dynamic characteristics of Valley of Mexico clays and ground response analyses", Internal Report, Institute of Engineering, UNAM, April (in Spanish).
Romo, M.P. (1987), "Foundation engineering in Mexico City: Seismic aspects", *Proc. Int. Symp. on Geotech. Engineering of Soft Soils*, Mexico City, Mexican Soc. for Soil Mechs., 2, pp213-224.
Romo, M.P., Jaime, A. and Taboada, V.M. (1989), "Cyclic behavior of Mexico City clay", Internal Report, Institute of Engineering, UNAM (in Spanish).
Romo, M.P., (1990), "Seismic behavior of Mexico City clay and its influence on foundation engineering", *Proc. of the Symp. The Subsoil of the Valley of Mexico and its Relation to Foundation Engineering: Five Years After the Earthquake*, Mexican Soc. for Soil Mech., pp83-94 (in Spanish).
Romo, M. P. (1991), "Soil-structure interaction in Mexico City", Conferencia de ingreso como Académico de número, Academia Mexicana de Ingeniería, Alternativas Tecnológicas, mayo 15 (in Spanish).
Romo, M. P. and Bárcena, A. (1994), "Dynamic soil-structure interaction analyses in Mexico City", *Series del Instituto de Ingeniería, No. 565*, December (in Spanish).
Romo, M.P. and García, J.A. (1994), "A simple procedure to evaluate earthquake-induced displacements in

shallow and deep foundations", Internal Report, Institute of Engineering, UNAM, December (in Spanish).
Romo, M.P. (1995), "Clay behavior, ground response and soil-structure interaction studies in Mexico City", *Proc. 3rd ICRAGEESD*, 2, pp1039-1051.
Romo, M.P. and Ovando, E. (1995), "Geoseismic zonation in Mexico and its applications for design spectra computations", State-of-the-Art Paper, *Proc. 10th Panamerican Conf. on Soil Mech. and Found. Engng.*, Orozco, C. (ed.), Guadalajara, Mexico, Mexican Soc. for Soil Mechs., 4, pp558-614 (in Spanish).
Romo, M.P. and Díaz, R.R. (1997), "Earthquake-induced displacements in shallow foundations: A method of analysis", Internal Report, Instituto de Ingeniería, UNAM, January (in Spanish).
Romo, M.P. (1999), "Earthquake Geotechnical Engineering and Artificial Neural Networks", *4th Arthur Casagrande Lecture*, Foz do Iguaçu, Brasil.
Romo, M.P. and Ovando, E. (1999), "P-Y curves for piles under seismic lateral loads", *Geotechnical and Geological Engineering*, 16, pp251-272.
Rosenblueth, E. (1952), "Theory of the seismic design over soft deposits", Ediciones ICA, México B, 14 (in Spanish).
Rosenblueth, E. and Esteva, L. (1971), "Reliability basis for some Mexico Codes", *Proc., ACI Symposium on Probabilistic Design of RC buildings*.
Sánchez-Sesma, F.J. (1983), "Diffraction of elastic waves by three-dimensional surface irregularities", *Bulletin of the Seismological Society of America*, 73, pp1621-1636.
Sarma, S.K. and Iossifelis, I.S. (1990), "Seismic bearing capacity factors of shallow strip footings", *Géotechnique*, 40(2), pp265-273.
Seed, H.B. and Idriss, I.M. (1970), "Soil moduli and damping factors for dynamic response analyses", *Report No. EERC-70-10*, University of California, Berkeley, California.
Seed, H.B., Lee, K.L., Idriss, I.M. and Makdisi, F. (1973), "Analysis of slides in the San Fernando Dam during the earthquake of Feb. 9, 1971", *Report No. EERC 73-2*, University of California, Berkeley.
Seed, R.B., Dickenson, S.E. and Mok, C.M. (1994), "Site effects on strong shaking and seismic risk: Recent developments and their impact on seismic design codes and practice", *Proc. of Structure Congress XII*, ASCE, 1, pp573-578.
Seed, R.B. and Moss, R.E.S. (1999), "Recent advances in US codes and policy with regard to seismic geotechnics", *2nd ICEGE*, 3, pp1111-1116.
Shibuya, S., Tatsuoka, F., Teachavorasinskum, S., Kang, X.J., Abe, F., Kim, Y-S. and Park, C-S. (1992), "Elastic, deformation properties of geomaterials", *Soils and Foundations*, 32-3, pp26-46.
Shibuya, S., Toshiyuki, T., Fukuda, F. and Degoshi, T. (1995), "Strain-rate effects on shear modulus and damping of normally consolidated clay", *Geotech. Testing Journal*, ASTM, 18, 3, pp365-375.
Somerville, P.G. (1998), "Emerging art: earthquake ground motions", *Proc. Geotechnical Earthquake Engineering in Soil Dynamics III*, P. Edited by Dakoulas, M., Yegian and R. D. Holtz, Geotechnical Special Publication No. 75, ASCE, 1, pp1-38.
Stokoe, K.H., II, Hwang, S.K., Lee, J.N-K. and Andrus, R.D. (1994), "Effects of various parameters on the stiffness and damping of soils at small to medium strains", *Proc. 1st IS-Hokkaido'94*, 2, pp785-816.
Sun, J.I., Golesorkhi, R. and Seed, H.B. (1988), "Dynamic moduli and damping ratios for cohesive soils", *Report No. EERC 88-15*, Univ. of California, Berkeley.
Tabatabaie-Raissi, M. (1982), "The flexible volume method for dynamic soil-structure interaction analysis", PhD Thesis, University of California, Berkeley, January
Taboada-Urtuzuástegui, V.M., Martínez, H. and Romo, M.P. (1999), "Evaluation of dynamic soil properties in Mexico City using downhole array records", *Soils and Foundations*, 39, 5.
Tani, K. (1995), "General report: Measurement of shear deformation of geomaterials – Field tests", *Proc. 1st IS-Hokkaido'94*, 2, pp1115-1135.
Tatsuoka, F. and Kohata, Y. (1995), "Stiffness of hard and soft rocks in engineering applications", *Proc. 1st IS-Hokkaido'94*, 2, pp947-1063.
Teachavorasinskum, S., Shibuya, S., Tatsuoka, F., Kato, H. and Horii, N. (1991), "Stiffness and damping of sands in torsion shear", *Proc. 2nd ICRAGEESD*, 1, pp103-110.
Trifunac, M.D. (1971), "Surface motion of a semi-cylindrical alluvial valley for incident plane SH waves", *Bulletin of the Seismological Society of America*, 61, pp1661-1667.
Trifunac, M.D. and Todorovska, M.I. (1998), "Nonlinear soil response as a natural passive isolation mechanism -The 1994 Northridge, California, earthquake", *Soil Dynamics and Earthquake Engineering*, 17, 1, pp41-51.
TRISEE (1999), "3D site effects and soil-foundation interaction in earthquake and vibration risk evaluation", edited by Faccioli, E., Paolucci, R. And Vanini, M., European Commission, Directorate General XII for Science, Resaerch and Development.
Tsukamoto, Y., Ishihara, K., Yamabe, S. and Hyodo, J. (1999), "Behavior of piles in the liquefied deposit undergoing lateral spreading", *Proc. 2nd ICEGE*, 1, pp377-388.
Vucetic, M. Lanzo, G. and Doroudian, M. (1998), "Damping at small strains and cyclic simple shear tests",

Journal of Geotechnical Engineering, ASCE, 124, 7, pp585-594.

Wolf, J.P. and Darve, G.R. (1986), "Nonlinear soil-structure interaction analysis based on boundary element method in time domain with application to embedded foundations", *Journal of Earthquake Engineering and Structural Dynamics*, 14, pp385-400.

Yasuda, S., Nagase, H., Oda, S., Masuda, T. and Morimoto, I. (1994), "A study on appropriate number of cyclic shear tests for seismic response analyses", *1st IS-Hokkaido'94*, 1, pp197-202.

Yasuhara, K. (1995), "Consolidation and settlement under cyclic loading", State-of-the-Art report, *IS-Hiroshima*, May.

Zeevaert, L. (1957), "Compensated friction-pile foundation to reduce the settlement of buildings on the highly compressible volcanic clay of Mexico City", *Proc. 4th ICSMFE*, London, 2, pp81-86.

Zeghal, M. and Elgamal, A.-W. (1994), "Analysis of site liquefaction using earthquake records", *Journal of Geotechnical Engineering*, ASCE, 120(6), pp996-1017.

Zeghal, M. and Elgamal, A.-W. (1995), "Lotung downhole array II: Evaluation of soil nonlinear properties", *Journal of Geotechnical Engineering*, ASCE, 121(4), pp363-377.