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

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SUMMARY

This paper reviews 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, and others. It also points out areas where more research is needed to advance 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] presented a useful review of this. 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 are more detailed analyses 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.

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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 liquefable granular materials should be identified at this stage.

2.1 Field tests

It has long been recognized that soil dynamic properties depend on the level of strain induced. With the advent and significant development of field tests to evaluate soil shear modulus, it became important 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 are:
(a) Geophysical exploration from the 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 strain 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 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 with a certain strain level. Pressuremeter data, in terms of applied pressure and measured cavity strain, is often interpreted using an elasticity-based theory to estimate soil stiffness. However, 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. These 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 *in situ* dynamic stiffness, damping and strength of 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 analysis of a large number of geotechnical case histories [Tatsuoka and Kohata, 1995] has 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 the reason that laboratory stiffnesses are sometimes lower than those back-calculated from full-scale field 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 more rapid progress than field tests. Ingenious solutions and recent technological developments have been put into 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 a basis for 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 equipment. 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 do provide a solid basis for the assessment of dynamic properties. However, it is usual to find that comparison of results obtained with different methods for the same soil indicates considerable differences. Common errors or lack of precision in experimental measurements do not explain adequately the discrepancies [Gajo y Mongiovi, 1994]. Justification must be sought in the behavior of geomaterials, and not in the techniques, at least not directly, as 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 so that the sensitivity of the various parameters can be assessed under repeatable conditions. Additionally, determination of damping ratio is a relatively easy task in the laboratory, 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 the preparation method, which in general reduce 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; and that small sized 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 field wave velocities to be measured at not-so-small strains. Thus the field stiffness is obtained at strains larger than those developed, for example, in laboratory 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 appropriate to acknowledge that, due to the complex processes involved in soil profile formation, it is advisable to use geostatistical techniques to interpret and generalize, for example, soil deposit stratigraphies and CPT profiles [Auvinet, 1999].

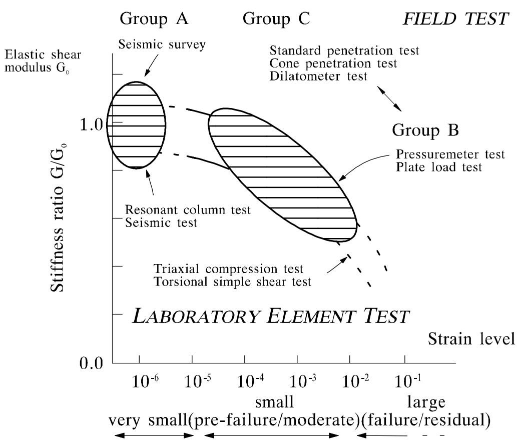

Figure 2.1 Comparative studies of nonlinear shear deformation, field tests vs. laboratory tests (after Tani, 1995)

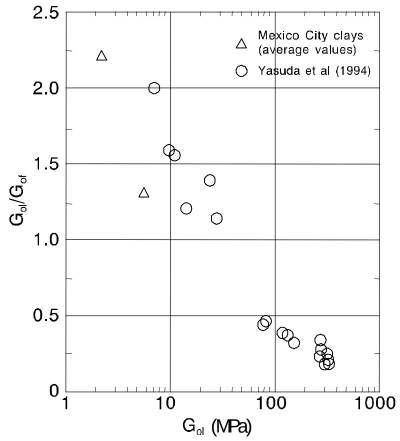

Figure 2.2 Soil-stiffness influence on lab/field (G_{ol}/G_{of}) shear modulus ratio (modified from Yasuda et al, 1994)

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 testing .

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 duration of dynamic loading. The influence of other factors like the overconsolidation ratio, effective stress-strength parameters and time-sustained loading are 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 suggests 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 this 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 by 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 changes, 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 plasticity clays which may accumulate large 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] and dynamic pore water pressures are negligible for shear stresses less than 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 Cohesionless Soils

It is recognised that the cyclic behaviour of granular soils is very complex, and as such it is very difficult to take into account many of these features. On the basis of observed field and laboratory soil responses, it may be argued that for foundation analyses under dynamic loading, consideration of the tendency for dilation or compression would suffice for most practical cases. Thus special attention should be paid to soil relative density. When granular soils exhibit dilative behaviour then negative pore water pressures develop for undrained conditions, leading to effective stress increases that improve bearing capacities. On the other hand, if the granular materials exhibit contractive behaviour then positive pore water pressures ensue when loaded under constant volume conditions. Accordingly, there is a decrease in effective stresses with a consequent loss in bearing capacity. 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 behaviour and soil fatigue effects apply also to granular soils .

Therefore, foundation engineers should be more concerned with granular soils that have a tendency to decrease in volume during earthquake shaking. It is recommended that, for this condition, instead of designing for such an unfavourable situation, it would be better to apply the most appropriate 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 influences on the seismic wave 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 gathering 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 (18.08º Lat N, 102.94º Long W; M_s=8.1) and 1999 Tehuacán (18.20º Lat N, 94.47º 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 behaviour of soils in the stratigraphy, including fatigue-type effects of 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 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 nearer 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-normalised 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.

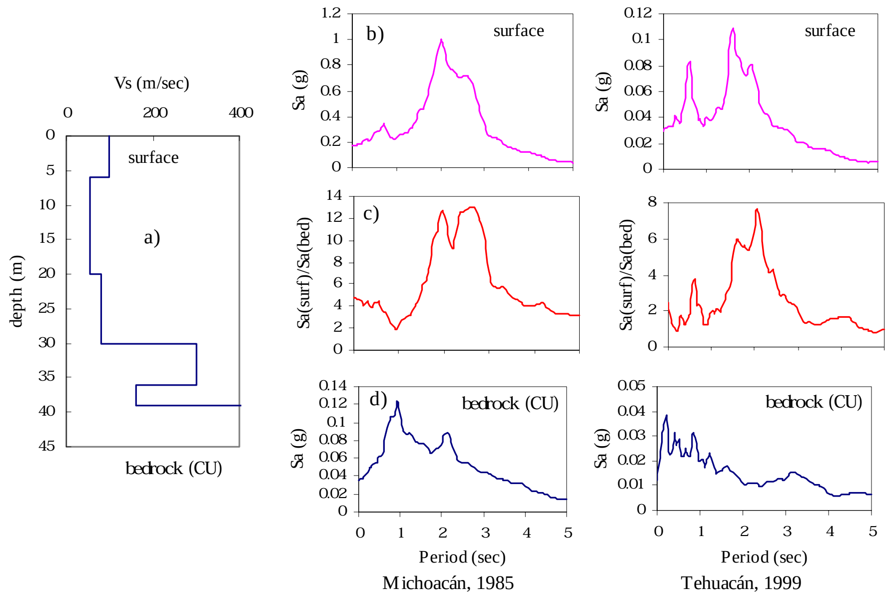

Figure 4.1. SCT site response to two earthquakes

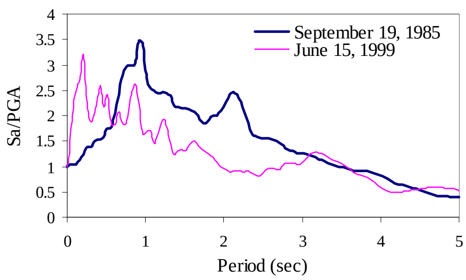

Figure 4.2 Seismological effects on rock-like motions

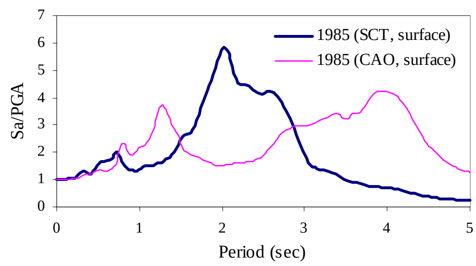

Figure 4.3 Geotechnical and geologic effects

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 rigid 10 m deep 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.

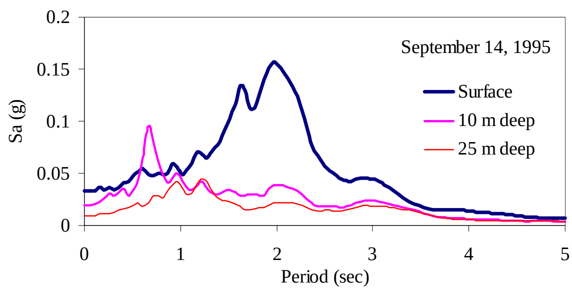

Figure 4.4 Spatial variation of ground motions SCT

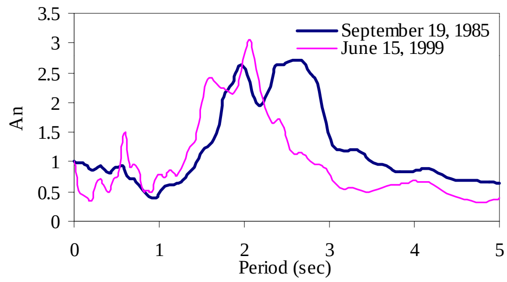

Figure 4.5 Nonlinear effects on ground motions

Another geotechnical factor that is usually neglected when designing foundation-building systems is the influence that nonlinear soil behaviour 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 nonlinear-soil behaviour, figure 4.5 compares the empirical amplification functions of figure 4.1c [(Sa(surf)/Sa(bed)], normalised with respect to the amplitude at period T=0, giving the normalised amplification functions A_n. These functions show that at longer periods the severity of the seismic motion is greater for lower peak amplitudes. This is a clear manifestation of the nonlinear behaviour that even the extremely highly plastic clays of Mexico City may undergo when subjected to severe motions.

Conceptually, the effects of nonlinear behaviour of soils may be beneficial. Indeed, one of the lessons derived from the behaviour 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 flexed. 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 nonlinear soil effects may be able to mitigate the damage 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 damage [Romo, 1990; 1995]. Needless to say, this proposition should be investigated further.

The influence of surface topography on ground motions is not well understood. In many cases important discrepancies between theoretical and observed responses of hills support this statement. Whatever the physical explanation for this, the analysis of recordings obtained from dense arrays of instruments coupled with geophysical and geotechnical surveys, would improve our knowledge of this problem and 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 design. Therefore, most current site response procedures 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 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 inhomogeneities, 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 analysed 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 behaviour, and ii) the half space that is outside of the generalised soil-structure interface, that is modelled 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 analysed simultaneously. It is important to recognise 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 wave energy. To minimise this energy-reflection problem, and at the same time to keep the model within a reasonable size, numerous energy-absorbing boundaries have been developed [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 analyse 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 best known and used to analyse 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] which uses 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, a hybrid mathematical model that combines the spectral and finite element spatial discretisation 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 behaviour. This computational tool is capable of modelling the complete seismic problem that spans from seismic source to structural response.

The predictive capabilities of many of the above mentioned procedures have been evaluated throughout comparisons with actual seismic responses of soil-structure systems. In what follows, the response if the Bernardo Quintana building on soft clay in Mexico City 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.

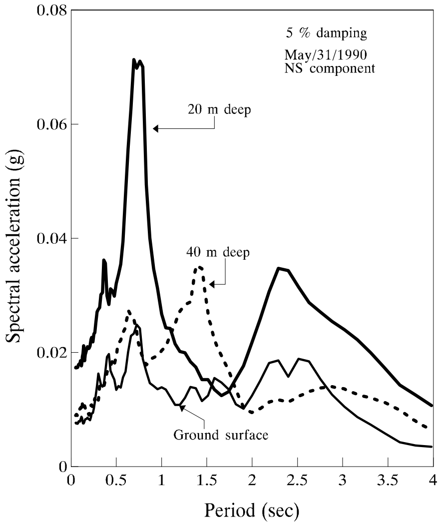

Figure 4.6 Motions recorded at building site

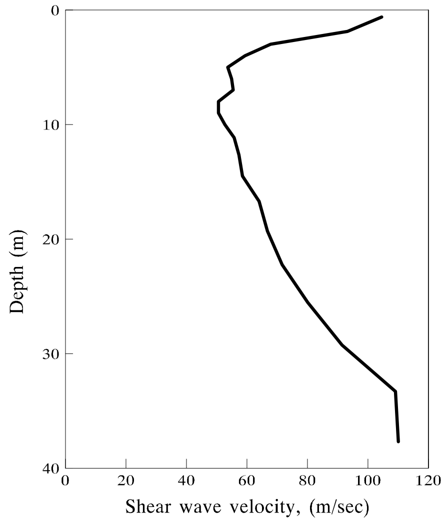

Figure 4.7 Site shear wave profile

The Bernardo Quintana building 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 caused no 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 a depth of 40 m. 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 develop, particularly near the foundation-soil interface. Figure 4.8 shows comparisons between observed and computed response spectra at the ground surface and a depth of 20 m. 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.

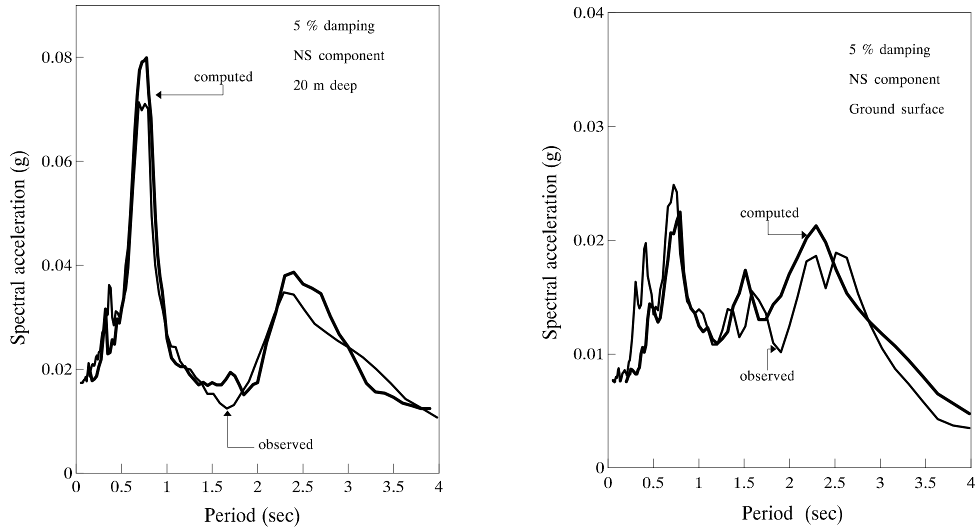

Figure 4.8 Measured and computed responses at the Bernardo Quintana building site for May 31, 1990 earthquake

Analytical studies, with this finite element procedure, of structures founded on Mexico City soft clays, lead to the following general conclusions: i) for box-type foundations, the deeper the embeddment the greater the attenuation of the free field motions; 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, thus it seems that kinematic interaction effects may be of advantage to the foundation designer in mitigating, in a natural way, the severity of the motions a building might be subjected to during severe seismic events; iv) as nonlinear soil effects become more significant, the base isolation effect becomes more pronounced and the attenuation of the motion increases; and v) it seems that the interaction-attenuation effects are more important at periods similar to the rigid-base natural period of the structure.

5. SEISMIC DESIGN ISSUES

5.1 General Considerations

In general, the design of a foundation involves a process including the following steps:

- Assembly of project information on superstructure, restriction on settlements, performance requirements
- Statement of geological and geotechnical environment
- Site investigation, idealisation of the stratigraphy and determination of soil properties
- Definition of applied loads, including the seismic forces
- Visualisation of the type of foundation, or diagnosis of the problem
- Design analysis and safety verifications according to building code requirements
- Consideration of excavation and construction procedures.

In fact, this process is not a once-through 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 trial foundation. If this predicted behaviour is not in agreement with the requirements, some characteristics of the initial guess, such as size or number of piles, are modified and the above steps are repeated.

Besides of the uncertainties about the behaviour of some foundation systems under seismic loads, other doubtful situations are faced for their design. Among them, uncertainties arise in finding adequate information about subsoil conditions and properties, as well as in the definition of the applied dynamic loads on the foundation as a highly random field of waves is generated during an earthquake. Thus it has been suggested that there are three levels of design analysis [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 behaviour 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 in methods of this level. Difficulties in the full characterisation 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 practising engineers. Geoseismic instrumentation on constructed foundations, 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 superimposing 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 induced by the passage of the seismic waves which when superimposed on the stresses induced by the static load system acting on the foundation give 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 modelling 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, the inertial contribution of the soil mass to the overall design force is less important and so 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 section 4.

Pseudostatic methods are conservative as long as the undrained shear strength of the soil does not drop due to earthquake loading. Accordingly, this potential loss of 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 the residual strength of the soil in the capacity analyses. Although this assumption may lead to conservative solutions, it is a safe one particularly when pre-earthquake shear stresses are larger than residual soil 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 correlations between the static safety factor and earthquake-induced displacements were established [Romo and Díaz, 1997].

5.3 Earthquake-Induced Displacements

All the procedures mentioned above for evaluating 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 the position of this could move around randomly. This poses a conceptual problem to pseudostatic methods because they implicitly assume one critical failure surface throughout the duration of the shaking. In view of this, it seems that current procedures are bound to overestimate soil degradation and pore water pressure increases 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, that 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 embeddment and soil strength help to decrease foundation settlements and make them more uniformly distributed. Even though this procedure follows more closely the mechanics 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 [1993] gave a thorough review of design issues and many procedures for the design analysis of pile foundations. Therefore, only few additional comments are included herein.

When dynamic analyses of piles are performed, it is important that modelling includes the following features: i) the variation in soil properties with depth, ii) the nonlinear stress-strain behaviour of soil at the pile-soil interface, including soil gapping, slippage along this interface and liquefaction, iii) 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 these methods consider the pile as a linear elastic beam element. However, 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. 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 utilised in the design process are in most situations approximate. In the face of such uncertainties, complete assurance of safety and reliability is difficult to achieve.

Uncertainty has traditionally been considered implicitly through the use of a factor of safety (or load 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 have a low reliability, especially 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 i) to assure that a foundation can resist without damage minor earthquakes occurring several times during its operation life; and ii) 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 acceptable price to ensure 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 are not acceptable, the provisions are made more demanding, 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 also be satisfied when designing a foundation.

Code design spectra consider elastic soil behaviour 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 behaviour but none considers the seismic intensity attenuation with ground depth. These two aspects should be duly considered in forthcoming construction code updating. 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. Furthermore, it has been suggested recently that to account properly for potential seismic threats to 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 for testing geotechnical models. Much of this trust was gained throughout the co-ordinated 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 indicating critical scenarios, that control the design of 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 modelling has been oriented, though, to generate realistic data on 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 behaviour 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 is an important improvement and 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 the present capabilities shaking-tables.

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 behaviour 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 behaviour 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 interrelated factors such as wave-train characteristics, building-foundation vibration patterns, soil-foundation interaction, soil behaviour (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, dynamic soil 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 modelling and reality. Given the complex interplay among foundation-performance controlling factors, it seems that to improve our knowledge on these matters and narrow the modelling-reality gap 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 the 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 nature 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 has 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 extraction of groundwater 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 minimise building emergence relative to the sinking adjacent ground. The former is achieved using long end bearing piles, but emergence is not avoided. Aside from aesthetic considerations, ground-foundation relative displacements bring about serious disruptions to building services and cause important damage to nearby light buildings. Furthermore, large ground settlements may reduce the lateral capacity of the upper part of piles under dynamic loading, and increase pile vertical load due to negative skin friction.

To minimise the deleterious effects of ground subsidence, the concept of yielding piles was developed 50 years ago and applied by Mexican engineers ever since [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, although this solution is a clever answer for foundations subjected only to sustained vertical loads, it is not apt when high slab-soil contact pressures exist 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 behaviour 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, and also to learn more about dynamic pore pressure generation in highly plastic clays. Additional lessons are expected to be extracted from this case history on earthquake-induced permanent settlements, soil-structure interaction effects and long-term foundation behaviour. 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 (18.09º Lat N, 102.86º Long W; 16.00º Lat N, and 98.23º 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 for the earthquake motions 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.

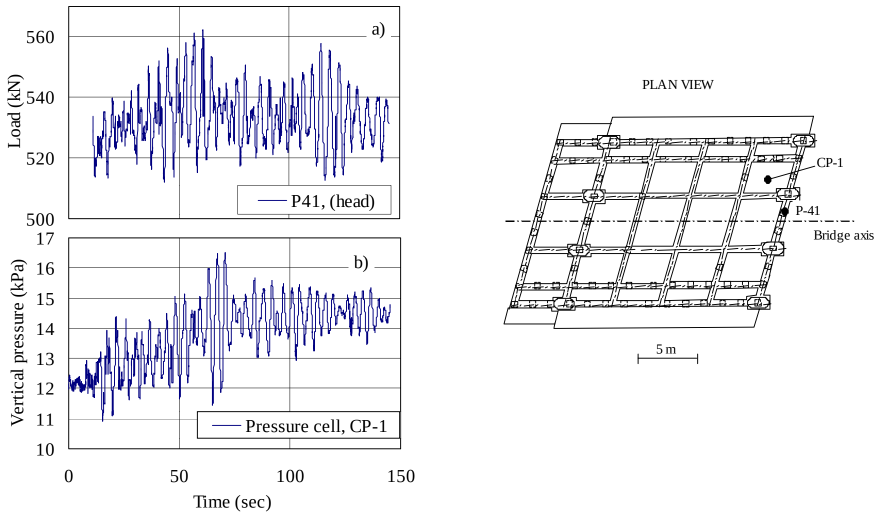

Figure 6.1 Pile load and raft-soil contact pressure time histories

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 which are 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 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.

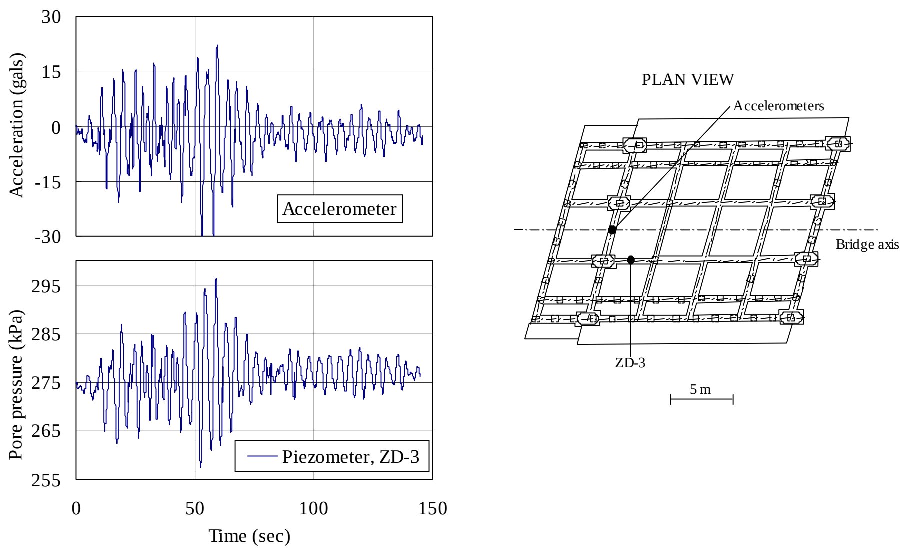

Figure 6.2 Acceleration and pore water pressure time histories

6.3 Knowledge-Based Procedures

There is a growing number of research engineers who 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 developing for the analysis of engineering problems is the so-called Artificial Neural Network (ANN). It simulates the way Biological Neural Networks (BNN) learn and process the information that human body receptors receive 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, the ANN is another tool to solve these mapping problems.

In a recent paper [Romo, 1999], it was shown that the 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. The matching is nearly perfect (correlation = 0.97). This case (among many others) makes evident the usefulness of field instrumentation as a needed procedure to gain 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 correlations 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.

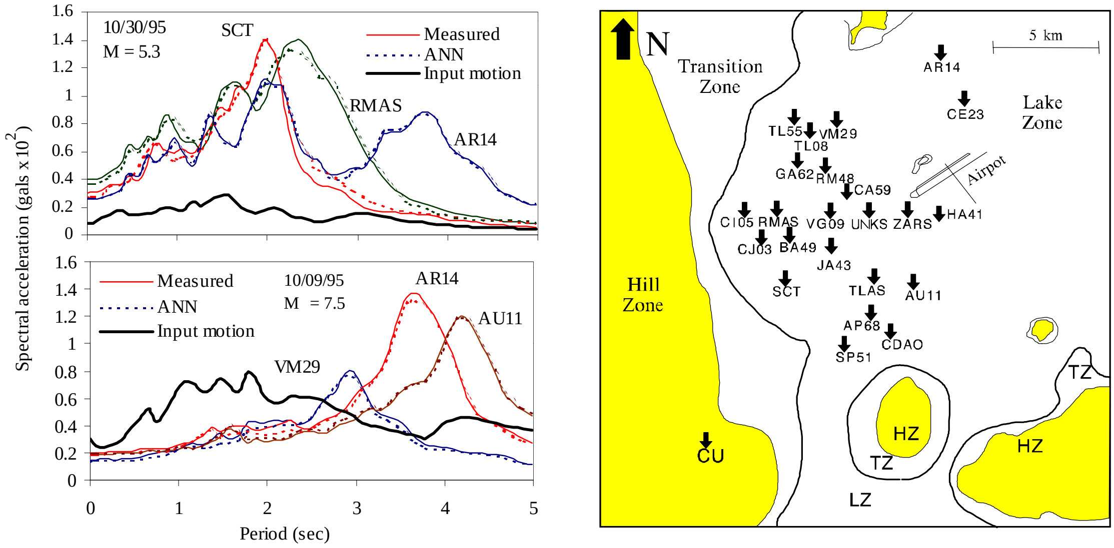

Figure 6.3 Measured and ANN predicted responses at various sites in Mexico City, as indicated in the geotechnical zoning

The success of ANNs resides in the fact that they can recognise patterns within vast data sets and then generalise them into recommended courses 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 behaviour 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 dynamic soil 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 behaviour of the system. It should be recognised 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 linearised model which is equivalent, in some sense, to the original nonlinear system. Instead of using a mathematical model, at the 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 recognised that analysis and design of foundations involve the interplay of a wide variety of related disciplines such as seismology, geology, soil and rock dynamics, and applied mechanics. Therefore, foundation engineers need to 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 been shown to reproduce accurately the seismic response of instrumented prototypes.

Small-sample laboratory techniques have advanced in large strides, mainly due to the transference of technology from control and electronic disciplines. However, there is room for improvement on the modelling of loading and boundary conditions existing in the prototype. Resurgence of shake tables and the 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 satisfied rigorously. These restraints must be clearly recognised 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 latest 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 harmonise all requirements in terms of safety, represents a step forward . 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 behaviour. As 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 system identification techniques.

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