Earthquake Resistant Design of Foundations:
New Construction

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ABSTRACT

The first generic class of aseismic foundation design problem relates to the design of new foundations. Once earthquake risk and site effects have been evaluated the designer needs to proceed with the proportioning of the foundation. To date there is little in the way of code based recommendations to cover this. Eurocode 8 (the structural design code in the new Eurocode series) is an exception and contains an extensive section on the design of foundations to resist earthquake loading. This has been developed using the results of a number of special investigations, both laboratory and theoretical. This paper will address the background to the provisions in the Eurocode, will cover shallow foundations and deep foundations, and review differences between low level response for which the soil can be expected to remain elastic and other situations where nonlinear behaviour of the soil adjacent to the foundation occurs.

INTRODUCTION

At the dawn of the third millenium, aseismic design of foundations still remains a challenging task for the earthquake geotechnical engineer. Leaving aside the seismic retrofit of existing foundations, which is a more difficult issue, even the design of new foundations raises issues which are far from being totally resolved. One of the main reasons stems from the complexity of the problem which requires skills in soil mechanics, foundation engineering, soil-structure interaction along with, at least some knowledge of structural dynamics.

A parallel between static design and seismic design reveals some similarity but also very marked differences. In the early days, static design of foundations put much emphasis on the so-called bearing capacity problem (failure behavior); with the introduction of an appropriate safety factor, close to 3, the short term settlements were deemed to be acceptable for the structure. It is only with the increase in the understanding of soil behavior and the development of reliable constitutive models that sound predictions of settlements could be achieved. Not surprisingly, earthquake geotechnical engineers have focused their attention on the non linear behavior of soils and on the evaluation of the cyclic deformations of foundations. This was clearly dictated by the need for an accurate evaluation of the soil-structure interaction forces which govern the structural response. It is only during the last decade that seismic bearing capacity problems have been tackled. These studies have clearly been motivated by the foundation failures observed in the Mexico City (1985) and Kobe (1995) earthquakes.

These two aspects of foundation design have reached a state of development where they can be incorporated in seismic building codes; Eurocode 8 - Part 5 is certainly a pioneering code in that respect.

In this paper, the fundamental aspects underlying the earthquake resistant design of new foundations will be reviewed and their implementation in seismic building codes will be discussed.

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ASEISMIC FOUNDATION DESIGN PROCESS

The aseismic design process for foundations is a "very broad activity requiring the synthesis of insight, creativity, technical knowledge and experience" (Pender, 1995). Information is required and decisions have to be made at various stages including:
(i) the geological environment and geotechnical characterization of the soil profile;
(ii) the definition of the loads that will be applied to the foundation soil by the facility to be constructed;
(iii) information about the required performance of the structure;
(iv) investigation of possible solutions with evaluation of load capacity, assessment of safety factors and estimates of deformations;
(v) consideration of construction methods and constraints that need to be satisfied (finance and time);
(vi) exercise of judgment to assess potential risks.

Obviously the process described above is not a linear progression. Several iterations may be required, at least from step ii to step vi, before arriving at a feasible, reliable and economic design. In the following we will focus on steps ii and iv. We will assume that all the required information related to the soil characterization and structural performance is available. This in no way means that these two items are of secondary importance; the data listed under these items are probably the most difficult to assess and considerable experience is required as well as the exercise of judgment.

EVALUATION OF SEISMIC LOADS

Fundamentals of Soil Structure Interaction

The earthquake design loads applied to the foundation arise from the inertia forces developed in the superstructure and from the soil deformations, caused by the passage of seismic waves, imposed on the foundations. These two phenomena are referred in the technical literature as inertial and kinematic loading. The relative importance of each factor depends on the foundation characteristics and nature of the incoming wave field.

The generic term encompassing both phenomena is Soil-Structure Interaction (SSI). However, more often, design engineers refer to inertial loading as SSI, ignoring the kinematic component. This situation stems from the fact that:
- kinematic interaction may in some situations be neglected;
- aseismic building codes, except for very few exceptions like Eurocode 8, do not even mention it;
- kinematic interaction effects are far more difficult to evaluate rigorously than inertial interaction effects.

Figure 1 illustrates the key features of the problem under study (Gazetas-Mylonakis, 1998). It is presented in the general situation of an embedded foundation supported on piles but all the conclusions are valid for any foundation type.

The soil layers away from the structure are subjected to seismic excitation consisting of numerous incident waves: shear waves (S waves), dilatational waves (P wave), surface waves (R or L waves). The nature of the incoming waves is dictated by seismological conditions but the geometry, stiffness and damping characteristics of the soil deposit modify this motion; this modified motion is the free field motion at the site of the foundation. Determination of the free field motion is in itself a challenging task because, as pointed out by Lysmer (1978), the design motion is usually specified at only one location, the ground surface, and the complete wave field cannot be back-calculated from this incomplete information; that is the problem is mathematically ill posed. Assumptions have to be made regarding the exact composition of the free field motion and it can be stated that no satisfactory solution is available to date.

Let us now consider the motion around the structure and its foundation: the seismically deforming soil will force the piles and the embedded foundation to move, and subsequently the supported structure. Even without the superstructure, the motion of the foundation will be different from the free field motion because of the difference in rigidity between the soil on one hand, and the piles and foundations on the other hand; the incident waves are reflected and scattered by the foundation and piles which in turn are stressed developing curvatures and bending moments. This is the phenomenon of kinematic interaction.

The motion induced at the foundation level generates oscillations in the superstructure which develop inertia forces and overturning moments at its base. Thus the foundation, the piles, and eventually the surrounding soil experience additional dynamic forces and displacements. This is the phenomenon of inertial interaction.
Obviously the foundation, in a broad sense, must be checked for the combined inertial and kinematic loading.

![Soil-Pile-Structure System](image)

Figure 1: Sketch of the soil-structure interaction problem

The above decomposition of the problem into three tasks (site response analysis, kinematic interaction, inertial interaction) is convenient for highlighting the various contributions of each to the final result. It does not necessarily imply that the steps must be performed separately as a complete interaction analysis (usually with the finite element method) is conceptually possible. However from a design and practical standpoint the computation of the foundation seismic loads usually follows the three step approach.

A direct (or complete) interaction analysis is very time demanding and not well suited for design, especially in 3D, which requires that the steps described above under item ii to vi be repeated several times. The multistep approach reduces the problem to more amenable stages and does not necessarily require that the whole solution be repeated again if changes occur, let's say, in the superstructure. In addition, it has the advantage that some of the intermediate steps can be neglected, as shown later.

The multistep approach is not only attractive for illustrating the fundamental aspects of soil structure interaction, it is also of great mathematical convenience. This convenience stems, in linear systems, from the superposition theorem (Kausel and Roesset, 1974). This theorem states that the seismic response of the complete system of figure 2 can be computed in two steps:

- the kinematic interaction involving the response to base acceleration of a system which differs from the actual system in that the mass of the superstructure is equal to zero;
- the inertial interaction referring to the response of the complete soil-structure system to forces associated with accelerations equal to the sum of the base acceleration plus those accelerations arising from the kinematic interaction.

The later system is further divided into two consecutive steps:

- computation of the dynamic impedances at the foundation level;
- analysis of the dynamic response of the superstructure supported on the dynamic impedances and subjected to the kinematic motion, also called effective foundation input motion.
Provided each step described above is performed rigorously and under the restriction that the system remains linear, i.e. superposition is valid, the breakdown of the complete interaction analysis into consecutive steps is rigorous. For a mathematical description of the superposition theorem, the reader is referred to Kaukel and Roesset (1974) or Gazetas and Mylonakis (1998).

With this, now classical, theoretical background on soil-structure interaction, in mind, one can proceed to examine the practical implementation of SSI in the state of practice. We will also examine to what extent this state of practice could be improved at a minimal cost.

**Code Approach to Soil Structure Interaction Analyses**

In almost every seismic building code, the structure response and foundation loads are computed neglecting the soil-structure interaction, that is a fixed base analysis of the structure is performed. The belief is that SSI always plays a favorable role in decreasing the inertia forces; this is clearly related to the standard shape of code spectra which almost invariably possess a gently descending branch beyond a constant spectral acceleration plateau. Lengthening of the period, due to SSI, moves the response to a region of smaller spectral accelerations. However there is evidence that some structures founded on unusual soils are vulnerable to SSI. Examples are given by Gazetas and Mylonakis (1998) and Resendiz and Roesset (1985) for instance.

This has been recognized in some codes. Eurocode 8 states that "The effects of dynamic soil-structure interaction shall be taken into account in the case of:

- structures where P-8 effects play a significant role;
- structures with massive or deep seated foundations;
- slender tall structures;
- structures supported on very soft soils, with average shear wave velocity less than 100 m/s.

The effects of soil-structure interaction on piles shall be assessed..."

In addition, an annex to the code describes the general effect of SSI and a specific chapter analyzes its effects on piles and the way to deal with it. To the best of our knowledge Eurocode 8 is the only code which recognizes the importance of kinematic interaction for piled foundations, to quote:

"Bending moments developing due to kinematic interaction shall be computed only when two or more of the following conditions occur simultaneously:

- the subsoil profile is of class C (soft soil), or worse, and contains consecutive layers with sharply differing stiffness,
- the zone is of moderate or high seismicity, $\alpha > 0.10$,
- the supported structure is of importance category I or II."

Note that implicitly for "normal" soil profiles and ordinary buildings kinematic interaction need not be computed.

**Improved Evaluation of Seismic Loads**

**Linear Systems**

With the tremendous development of computer facilities, there does not seem to be any rational reason for neglecting soil-structure interaction. Most building codes now require that the structural response be evaluated using a multimodal analysis, as opposed to a former monomodal analysis, and this can be performed with most computer codes available on the market.

Referring to the multistep approach described previously, the last step of an SSI analysis (response of the structure connected to the impedances) can be performed on a routine basis provided that:

- the system remains linear;
- the kinematic interaction can be neglected;
- dynamic impedance functions are readily available.

Although the superposition theorem is exact for linear soil, pile and structure, it can nevertheless be applied to moderately non linear systems. This can be achieved by choosing reduced soil characteristics which are compatible with the free field strains induced by the propagating seismic waves: this is the basis
of the equivalent linear method, pioneered by Idriss and Seed (1968). This engineering approximation implies that all the soil non-linearities arise from the passage of the seismic waves and that additional non-linearities, developed around the edges of a mat foundation or along the piles shafts, are negligible. Experience shows that it is a valid approximation in many situations where large soil instabilities do not occur.
Figure 2: Superposition theorem for soil-structure interaction problem
For some situations, kinematic interaction can be neglected and the second step of the multistep approach can be bypassed. It must be realized however that, if kinematic interaction is thought to be significant, there is no simple means for evaluating it; as a matter of fact, evaluation of kinematic interaction is almost as difficult as solving the complete SSI problem. Obviously kinematic interaction is exactly zero for shallow foundations in a seismic environment consisting exclusively of vertically propagating shear waves or dilatational waves. Gazetas (1984) has demonstrated that when the piles are flexible with respect to the surrounding soil, kinematic interaction is significant for small to medium frequencies.

During the last decade, numerous solutions for the dynamic impedances of any shape foundations and of piles have been published (Gazetas, 1990). They are available for homogeneous soil deposits but also for moderately heterogeneous ones. In addition, simplified methods are available in the case of pile foundations to account for the group effect (Dobry and Gazetas, 1988).

Therefore provided all the aspects listed above are properly covered, seismic soil structure interaction can be covered at a minimal cost and reduces to the last step of the multistep approach: dynamic response of the structure connected to the impedance functions and subjected to the free field motion (equal to the kinematic interaction motion). However to be fully efficient, and to allow for the use of conventional dynamic computer codes, the impedance functions which are frequency dependent (figure 4) must be represented by frequency independent values. The simplest version of these frequency independent parameters are the so-called springs and dashpots. From the published results, it appears that only under very restrictive soil conditions (homogeneous halfspace, regular foundations) can these dynamic impedances be represented by constant springs and dashpots. Nevertheless, structural engineers still proceed using these values which, more than often, are evaluated as the static component (zero frequency) of the impedance functions.

Figure 3: Examples of cone models

However, fairly simple rheological models can be used to properly account for the frequency dependence of the impedance functions. These models can be developed using curve fitting techniques, or with physical insight, such as the series of cone models developed by Wolf (1994). Figure 3 shows examples of such models: figure 3a is the model proposed by De Barros and Luco (1990) based on a curve fitting technique; figure 3b is a class of cone models proposed by Wolf. With such models, which are most conveniently used in time history analyses, the actual dynamic action of the soil can be properly accounted for; even "negative stiffnesses", which are frequently encountered in layered soil profiles, can be apprehended with those models. As an illustrative example, figure 4 presents the application of model 3a to an actual bridge pier foundation; the foundation is a large circular caisson, 90 m in diameter, resting on a highly heterogeneous soft soil profile. The "exact" impedances were computed using a frequency domain finite element analysis. Note the very good fit achieved by the model (square symbols) even for the negative stiffness of the rocking component. Clearly, implementation of such simple rheological models does not impose a heavy burden to the analyst and represents a significant improvement upon the lengthy and tedious iteration process in which springs and dashpots are updated to become compatible with the SSI frequencies.
Figure 4: Rocking dynamic impedances - example

Before moving onto consideration of nonlinear SSI, there in one section of Eurocode 8 which provides for a transition between the linear elastic approach discussed above and non linear methods discussed below. Table 1 (taken from Eurocode 8) acknowledges that with increasing ground acceleration the soil adjacent to a shallow foundation will experience increasing shear strains and consequently the stiffness will decrease and the material damping increase. Table 1 suggests how the apparent average shear modulus and material damping of the soil adjacent will change with increasing peak ground acceleration and envisages that an elastic SSI calculation would be done with the modified values for the soil stiffness and damping. (Following this simplification there is, of course, no frequency dependence on the stiffness and damping parameters for the foundation).

Table 4.1: Average soil damping factors and average reduction factors (± one standard deviation) for shear wave velocity $v_s$ and shear modulus $G$ within 20 m depth ($v_{s,max} = \text{average } v_s \text{ value at small strain }(< 10^{-5})$, not exceeding 300 m/s. $G_{max} = \text{average shear modulus at small strain}$.)

<table>
<thead>
<tr>
<th>Ground acceleration ratio, $\alpha$</th>
<th>Damping factor</th>
<th>$\frac{v_s}{v_{s,\text{max}}}$</th>
<th>$\frac{G}{G_{\text{max}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,10</td>
<td>0,03</td>
<td>0,9(±0,07)</td>
<td>0,80(± 0,10)</td>
</tr>
<tr>
<td>0,20</td>
<td>0,06</td>
<td>0,7(± 0,15)</td>
<td>0,50(± 0,20)</td>
</tr>
<tr>
<td>0,30</td>
<td>0,10</td>
<td>0,6(+0,15)</td>
<td>0,35(+0,20)</td>
</tr>
</tbody>
</table>

Non Linear Systems

One of the main limitation of the multistep approach is the assumption of linearity of the system for the superposition theorem to be valid. As noted previously, some non linearities, such as those related to the propagation of the seismic waves, can be introduced but the non linearities specifically arising from soil-structure interaction are ignored. The generic term "non linearities" covers geometrical non linearities, such as foundation uplift, and material non linearities, such as soil yielding around the edges of shallow foundations, along the shafts of piles, and the formation of gaps adjacent to pile shafts. Those non linearities may be beneficial and tend to reduce the forces transmitted by the foundation to the soil and therefore decrease the seismic demand. This has long been recognized for foundation uplift for instance (see ATC 40).

Giving up the mathematical rigor of the superposition theorem, an engineering approximation to these aspects can be reached by substructing the supporting medium into two sub-domains (figure 5):
- a far field domain, which extends a sufficient distance from the foundation for the soil structure interaction non linearities to be negligible, non linearities in that domain are only governed by the propagation of the seismic waves,
- a near field domain, in the vicinity of the foundation where all the geometrical and material non linearities due to soil structure interaction are concentrated.

Figure 5: Conceptual subdomains for dynamic soil structure analyses

The exact boundary between both domains is not precisely known but its location is irrelevant for practical purposes. This concept of far field and near field domains can be easily implemented if one assumes that the degrees of freedom of the foundation are uncoupled: the far field domain is modeled with the linear (or equivalent linear) impedance function whereas the near field domain is lumped into a non-linear macro-element. A simplified rheological representation of this sub-structuring is shown in figure 6 (Pecker, 1998): the macro-element is composed of a finite number of springs and Coulomb sliders which are determined from curve fitting to the non-linear force-displacement (or moment-rotation) backbone curve, computed for instance with a static finite element analysis.

Figure 6: Non-linear rheological model for SSI

Damping in the near field domain arises only from material damping and obeys Masing's law; damping in the far field domain is of the viscous type. Calibration of this simplified rheological model against a rigorous 2D dynamic finite element analysis, including all the non linearities mentioned previously, shows very promising results. Figure 7 compares the overturning moments at the foundation level of a bridge pier
foundation computed with both approaches: not only the amplitudes are correctly matched but also the phases are preserved.

This model can be extended in a more rigorous way to account for the coupling between the various degrees of freedom of the foundation, especially between the vertical and rotational one when uplift occurs (Cremer et al, 2000).

![Comparison between finite element analysis and the non linear rheological model for the foundation](image)

Figure 7: Comparison between finite element analysis and the non linear rheological model for the foundation

**BEARING CAPACITY FOR SHALLOW FOUNDATIONS**

Once the forces transmitted to the soil by the foundation are determined, the design engineer must check that these forces can be safely supported: the foundation must not experience a bearing capacity failure nor excessive permanent displacements. At this point a major difference appears between static, permanently acting loads, and seismic loads. In the first instance excessive loads generate a general foundation failure whereas seismic loads, which by nature vary in time, may induce only permanent irrecoverable displacements. Failure can therefore no longer be defined as a situation in which the safety factor becomes less than unity; it must rather be defined with reference to excessive permanent displacements which impede the proper functioning of the structure. Although this definition seems rather simple and the methodology has been successfully applied to dam engineering (Newmark, 1965), its implementation in a code format is far from an easy task. One of the difficulties is to define what are acceptable displacements of the structure in relation to the required performance. Another difficulty obviously lies in the uncertainty linked to the estimation of permanent displacements.

**Fundamental Requirement of Code Approaches**

As an example of code documentation Eurocode 8 states that “The stability against seismic bearing capacity failure taking into account load inclination and eccentricity arising from the inertia forces of the structure as well as the possible effects of the inertia forces in the supporting soil itself can be checked with the general expression and criteria provided in annex F. The rise of pore water pressure under cyclic loading should be considered either in the form of undrained strength or as pore pressure in effective stress analysis. For important structures, non linear soil behavior should be considered in determining possible permanent deformation during earthquakes.”

More specifically, in most seismic codes the design engineer is required to check the following general inequality:

\[ S_d \leq R_d \]  

(1)
where $S_d$ is the seismic design action and $R_d$ the system design resistance. These two terms are explained below.

The design action represents the set of forces acting on the foundations. For the bearing capacity problem, they are composed of the normal force $N_{sd}$, shear force $V_{sd}$, overturning moment $M_{sd}$ and soil inertia forces $F$ developed in the soil. The actions $N_{sd}$, $V_{sd}$, and $M_{sd}$ arise from the inertial soil-structure interaction. The inertia force, $F = \rho a$ ($\rho$ mass density, $a$ acceleration), arises from the site response analysis and kinematic interaction. The term design action is used to reflect that these forces must take into account the actual forces transmitted to the foundation i.e. including any behavior and over-strength factors used in inelastic design.

The design resistance represents the bearing capacity of the foundation; it is a function of the soil strength, soil-foundation interface strength and system geometry (for instance foundation width and length).

Obviously, inequality (1) must include some safety factors. One way is to introduce partial safety factors, as in Eurocode 8. This is not the only possibility and some other codes, like the New Zealand one, choose the Load and Resistance Factored Method (LRFD) and factor the loads and resistance (Pender, 1999). The Eurocode approach is preferred because it gives more insight in the philosophy of safety; on the other hand it requires more experimental data and numerical analyses to calibrate the partial safety factors.

With the introduction of partial safety factors inequality (1) is modified as follows:

$$S_d (\gamma_r, \text{actions}) \leq 1 \frac{1}{\gamma_{Rd}} R_d \left( \frac{\text{strength parameters}}{\gamma_m}, \text{geometry} \right)$$

(2)

where "actions" represent the design action and "strength" the material strength (soil cohesion and /or friction angle, soil-foundation friction coefficient).

$\gamma_r$ is the load factor applied to the design action: $\gamma_r$ is larger than one for unfavorable actions and smaller than 1.0 for favorable ones.

$\gamma_m$ is the material safety factor used to reflect the variability and uncertainty in the determination of the soil strength. In Eurocode 8, the following values are used: 1.4 on the undrained shear strength and cohesion and 1.25 on the tangent of the soil friction angle or interface friction coefficient.

$\gamma_{Rd}$ is a model factor. It acts like the inverse of a strength reduction factor applied to the resistance in an LRFD code. This factor reflects the fact, that to evaluate the system resistance some approximations must be made: a theoretical framework must be developed to compute the resistance and like any model it involves simplifications, and assumptions which deviate from reality. It will be seen later on that the model factor is essential and can be used with benefit to differentiate a static problem from a seismic one.

### Theoretical Framework for the Pseudo-Static Bearing Capacity

Since the devastating foundation failures reported after the Mexico earthquake (Auvinet and Mendoza, 1986) a wealth of theoretical and experimental studies have been carried out to develop bearing capacity formulae which include the effect of the soil inertia forces (Sarma and Iossifelis 1990, Budhu and Al Karni, 1993, Richards et al 1993, Zeng and Steedman, 1998).

The theoretical studies mentioned above are based on limit equilibrium methods (Chen 1975, Salençon 1983); although they represent a significant improvement over the previous analyses which neglected the soil inertia forces, they suffer from limitations which restrict their use (Pecker, 1994):

- the horizontal accelerations of the soil and of the structure are assumed to have the same magnitude;
- the results are derived from an assumed unique failure mechanism which does not allow for foundation uplift;
- the methods only consider upper bound solutions without any indication on how close they are from the exact solution.

At the same time numerous studies have been initiated in France and Europe with the objective of providing more general solutions (Pecker and Salençon 1991, Dormieux and Pecker 1994, Salençon and Pecker 1994 a-b, Paolucci and Pecker 1997, PREC8, 1996). The solutions were developed within the
framework of the yield design theory (Salençon 1983, 1990): the loading parameters N, V, M and F are considered as independent loading parameters thereby allowing for any combination of actions to be analyzed; many different kinematic mechanisms are investigated and lower bound solutions are also derived to (i) obtain the best possible approximation to the bearing capacity, (ii) bracket the true value to obtain a quantitative measure of the goodness of the solution. It is interesting to note that the results have been later completed by additional lower bound solutions which confirm the merit of the upper bound solutions and help to narrow the gap between upper and lower bound solutions (Ukritchon et al, 1998 ). Finally the results, mainly based on the upper bound solutions are cast in the general format (Pecker, 1997):

$$\phi (N, V, M, F) \leq 0$$  \hspace{1cm} (3)

where $\phi (\cdot) = 0$ (figure 8) defines in the loading parameter space the equation of a bounding surface.

Inequality (3) expresses the fact that any combination of the loading parameters lying outside the surface corresponds to an unstable situation; any combination lying inside the bounding surface corresponds to a potentially stable situation. The word potentially is used to point out that no assurance can be given since the solutions were derived from upper bound solutions. Indications on the merit of the solutions is obtained by comparison with the lower bound solutions and the model safety factor of Eq.(2) is introduced to account for that uncertainty. The uncertainty is twofold: the solution is obtained from an upper bound approach and, although various kinematic mechanisms were investigated, their number remains necessarily limited when a comprehensive implementation of the upper bound theorem would require that all the conceivable mechanisms be investigated.

These results are put in a simple mathematical expression and implemented in the current version of Eurocode 8 (Annex F) and are applicable to cohesive and purely frictional materials. The equation, (Pecker, 1997), is:

$$\frac{(1 - eF)^{\sigma} (\beta V)^{\sigma}}{(\bar{N})^{\sigma}
\left[(1 - mF^{k})^{\gamma} - \bar{N}\right]^{\bar{p}}} + \frac{(1 - lF)^{\varepsilon M} (\gamma M)^{\varepsilon M}}{(\bar{N})^{\varepsilon}
\left[(1 - mF^{k})^{\gamma} - \bar{N}\right]^{\bar{p}}} - 1 \leq 0$$

\hspace{1cm} (4)

where: $\bar{N} = \frac{\gamma_{rd} N_{sd}}{N_{max}}$, $\bar{V} = \frac{\gamma_{rd} V_{sd}}{N_{max}}$, $\bar{M} = \frac{\gamma_{rd} M_{sd}}{B N_{max}}$.

$N_{max}$ is the ultimate bearing capacity of the foundation under a vertical centered load, $N_{sd}$, $V_{sd}$, and $M_{sd}$ are the design action effects at the foundation level, B the foundation width, and $\gamma_{rd}$ the material safety factor. The soil inertia forces are accounted for by the normalized parameter $\bar{F}$ equal to $p a B / c_u$ for cohesive soils and to $a / g \tan \phi$ for frictional soils. The other parameters entering equation (4) are numerical parameters derived by curve fitting to the "exact" bearing capacity, the values of which can be found in Pecker, 1997.

Figure 8: Bounding surface for cohesive soils
Evaluation of Permanent Displacements

As noted previously and as recommended in Eurocode 8, in seismic situations, the permanent displacements should be evaluated. However such an evaluation is anything but an easy task. Probably the most rigorous approach would be to use a global model (finite element model) including both the soil and the structure. Obviously, the results depend on the non linear constitutive relationship used to model the soil behavior and are only meaningful if a realistic model is used. Owing to this constraint, to computer limitations, and to the required skill from the analyst in geotechnical engineering, structural engineering, soil-structure interaction and numerical analysis, such an approach is seldom used in everyday practice.

The alternative approach, once the seismic forces are known, is to rely on a Newmark type of approach (Newmark, 1965). The bounding surface defined by Eq.(3) is used as the surface defining the onset of permanent displacements. Sarma and Lissifelis (1990), Richards et al (1993) used the Newmark's approach assuming that the soil moves together with the foundation in a rigid body motion. The method has been further extended by Pecker and Salençon (1991) considering a deformable soil body corresponding to the assumed kinematic failure mechanism. Using the kinetic energy theorem, these authors computed the foundation angular velocity, and by integration over time, the foundation permanent rotation. When applied to actual case histories the method proved to be reliable (Pecker et al, 1995).

A potential use of the method can be found for the development of a code like approach. Computed permanent displacements develop when the resultant of the design action lies outside the bounding surface: the larger the distance to the bounding surface, the greater the displacements.

This can be expressed mathematically by writing that for such situations:

\[ S_d = \lambda \cdot R_d \]  

with \( \lambda > 1 \) ; \( \lambda = 1 \) corresponds to the onset of permanent displacements.

Comparing Eq.(5) to Eq.(2), it is readily apparent that allowing \( S_d \) to reach regions outside the bounding surface is equivalent to specifying a model safety factor \( \gamma_{rd} \) smaller than 1.0. Therefore, \( \gamma_{rd} \) can be used, in addition to reflecting the uncertainties in the model, to relax the constraint that at any time the resistance shall be larger than the action, recognizing the fundamental difference between a static problem and a seismic one in which forces vary in time.

This approach has been implemented in Eurocode 8 and the tentative values proposed in its Annex F are intended to allow for the development of permanent displacements in potentially non dangerous materials (medium to dense sand, non sensitive clay). These values range from 1.0 (medium dense to dense sands, non sensitive clays) to 1.5 (loose saturated sands) with intermediate values of 1.15 for loose dry sands. If this phenomenon were disregarded, \( \gamma_{rd} \) values would always be larger than 1.0 (in the range 1.2 to 1.5).

In the case of non sensitive clays further justification for setting \( \gamma_{rd} \) equal to 1.0 is the observation that shallow foundations in clay have generally been observed to perform well under seismic loading. As mentioned above, a reason for this may be the enhanced undrained shear strength available under rapid loading (Romo 1995, and Ahmed-Zeki et al 1999).

Unresolved Issues and Further Developments

One of the strong assumptions underlying the seismic bearing capacity checks, is the independence between the computed design actions and soil yielding. Except for the sophisticated approaches involving the partition in near and far fields, the design actions are computed assuming quasi-linear foundation behavior. However it is recognized that partial yielding of the foundation may affect the forces.

Attempts have been made by Nova and Montrasio (1991) for monotonic static loading based on the concept of a macro-element modeling the soil and foundation; the constitutive law for the macro-element is rigid plastic strain hardening with non associated flow rule. That concept of macro-element expressed in global variables at the foundation level has been extensively used in mechanics but seldom applied to soil-structure interaction. Paolucci (1997) and Pedretti (1998) have extended the method to seismic
loading. These last two studies definitively prove that yielding of the foundation cannot be ignored in the evaluation of the design action.

A more general formulation has been proposed recently by Cremer et al (2000). The developed macro-element taking advantage of the partition between near field and far field describes the cyclic behavior of the foundation, reproduces the material non-linearities under the foundation (yielding) as well as the geometrical non-linearities (uplift), and accounts for the wave propagation in the soil. The strength criterion for the macro-element is represented by the bounding surface defined by the bearing capacity formula and a non-associated flow rule with kinematic and isotropic hardening is used to compute the pre-failure displacements; the plastic model is coupled with an uplift model to integrate the influence of soil yielding on the uplift. Although still under development the model shows some promising capabilities and should represent a step forward in the evaluation of permanent seismic displacements of shallow foundations.

STIFFNESS AND CAPACITY OF PILE FOUNDATIONS

As for shallow foundations, the ultimate capacity of the piles, or pile groups, has to be checked once the applied inertia loads acting at the pile cap are known. Two failure modes must be examined: bearing failure with a vertical force exceeding the available tip and shaft resistance and lateral failure when the available lateral resistance is mobilized along part the pile shaft. The latter failure mode is more likely to occur although bearing capacity failures of floating piles have been observed, for instance in Mexico City in the 1985 earthquake.

As well as capacity, the stiffness of the pile, or pile group, needs to be evaluated for the calculation of kinematic and inertial interaction. Kinematic effects are known not to have a great effect on pile head motions. The pile head lateral and rotational stiffness is highly nonlinear.

Single Pile Stiffness

Dealing first with the stiffness and considering initially a single vertical pile, we need the stiffness of the pile head to vertical, lateral and moment loading; in addition we need to be able to estimate the response of the pile shaft to soil movements which occur in kinematic interaction. The linear response is well documented. Two models are commonly used: the elastic continuum and the Winkler spring. There is an important difference between the vertical stiffness of a pile, particularly if the length of the pile is modest and it bears on a firm stratum, and the lateral stiffness. The axial stiffness of a vertical pile is larger than the lateral stiffness, it may involve soil-pile interaction over the full length of the pile shaft, whereas the lateral stiffness mobilizes a relatively short portion of the pile shaft.

Elastic Continuum Model

The deflected shape of a pile which has horizontal lateral load applied at the ground surface extends a distance of several pile diameters into the soil profile. Beneath this there is negligible lateral displacement. The length over which the lateral displacement occurs is known as the active length and it is a function of pile diameter, the elastic modulus of the soil, and the ratio of pile modulus to soil modulus. Expressions for the active length for both static and dynamic lateral loading are given by Gazetas (1991). If the length of pile shaft is greater than this amount then the components of the pile head stiffness matrix are independent of the length of the pile shaft. Gazetas (1991) also gives expressions for the components of the pile head stiffness matrix for a variety of soil modulus distributions with depth; these are included in Eurocode 8. A very important finding about the dynamic lateral pile head stiffness is that the values are only slightly affected by loading frequency, thus a very good first approximation to the pile head stiffness components is given by the static stiffness of the pile. For dynamic loading there is, of course, damping. It has been found that for frequencies less than the natural period of the clay layer there is no radiation damping so the only damping is material damping in the soil adjacent to the pile. For frequencies higher than the natural frequency of the layer radiation damping, which increases with increasing frequency, is added to the material damping. Gazetas (1991) gives expression for the frequency effect on pile head stiffness and also expressions for the radiation damping. Thus, as stated above, simple equations are available for the elastic dynamic pile head
stiffness coefficients which are needed for inertial interaction. The potential limitation that these are available only for homogeneous or simple variations of soil modulus with depth is not such a problem as the depth of soil involved in the interaction is limited by the active length of the pile shaft. Gazetas also provided simple expressions for the vertical stiffness and radiation damping for piles. However, the appropriateness of these may be limited by layering in the soil profile.

Winkler Spring Model

When dealing with layered soil profiles the elastic continuum model in not applicable, but, fortunately, the Winkler model is useful for this case. This model consists of a series of independent horizontal (lateral) or vertical (axial) springs distributed along the beam (pile) length. It has long been used in offshore engineering to compute the pile head deflection and settlement under static loading at the pile head (Matlock and Reese, 1960). Based on field experiments, the approach models the soil response at a particular depth in terms of a p-y curve for lateral loading or t-z curve for axial loading; p and y (t, z) denote the resultant lateral (axial) soil reaction per unit length of pile and the associated lateral (axial) pile deflection. Matlock and Reese give a method of estimating the spring stiffness based on laboratory test data on soil samples. Another approach is given in the French code using a tri-linear backbone curve for static design of foundations (Fascicule 62, 1992). The slopes of the three portions are related to the pressuremeter modulus (Menard's modulus). The two break points in the distributed force per unit length of the pile shaft occur at p_{D} and p_0 D, where p_c and p_0 are the creep and limiting pressures measured in a pressurometer test and D the pile diameter. In this approach a reduction is applied to the parameters near the ground surface because it is very unlikely that the soil near the ground surface contributes fully to the pile resistance. A further alternative, (Pender, 1993), uses an empirical approach to estimate the initial spring stiffness, evaluates the maximum lateral pressure from the shear strength of the soil, and has a hyperbolic transition between these two limits. For dynamic loading the soil-pile interaction is modeled not only with distributed springs but also with dashpots, which are frequency dependent. Many different methods are available to determine the springs and dashpots in the linear, or quasi linear, case (Gazetas et al, 1992; Makris and Gazetas, 1992; Kavvadas and Gazetas, 1993).

The Winkler model is also used for calculating kinematic interaction between a pile and a soil profile. The difference between Matlock and Reese's model and this dynamic model stems from the location of the applied excitation; in the dynamic model (figure 9) the extremities of the springs and dashpots are connected to the free field where the soil response, computed independently, is imposed; in the static model those extremities are fixed. These calculations are presented by Kavvadas and Gazetas (1993) and Makris and Gazetas (1992). Tabesh and Poulos (1999) suggest that quite useful design information can be obtained for kinematic effects using an essentially static analysis of the pile shaft provided information about the free field displacement profile is available; they use a one dimensional site response analysis to obtain this.

Gapping

Yet another effect which has been modeled using the Winkler approach is gapping which is observed adjacent to the top of pile shafts when piles in cohesive soil are subject to earthquake loading. The formation of the gap is modeled by having Winkler springs on both sides of the pile shaft and monitoring the load in each spring. When the load is about to become tensile it is detached from the shaft. During a cycle in the next direction the position of the spring relative to the pile shaft is monitored and it is reattached at the appropriate moment. A very simple summary of the results of these calculations is that the gap extends to a depth about equal to the active length of the pile shaft. When this occurs the lateral stiffness of the pile is, not surprisingly, about half that when the springs are attached to both sides.

Raked piles

Raked piles are frequently observed to suffer substantial head damage during earthquakes. This occurs when the axial stiffness of the pile is much greater than the lateral stiffness (Pender, 1993). In this situation even a small angle of rake gives horizontal and moment stiffness terms of the pile head stiffness matrix considerably larger than those for the vertical pile. The consequence is that when a horizontal displacement
load is applied at the pile head it sustains large forces. This is a design problem that can be accommodated simply enough by a proper formulation of the pile head stiffness.

Figure 9: Dynamic Winkler model for soil-pile interaction analyses

Free field motion

Pile motion

Layer j

Soil / Pile

Vertical Shear waves

Nonlinear Stiffness

Along the same lines as for the shallow foundation, the Winkler model can be extended to nonlinear situations. The springs and dashpots of figure 9 model the far field domain. The near field domain is modeled with non linear, hysteretic, springs. The back bone curve for these springs would be the p-y curves, or any experimentally based curve for which the series of springs and sliders of figure 6 constitute a piecewise linear approximation. (Note that the initial tangent stiffness of the p-y curves should be removed from the near field stiffness since it is already accounted for in the far field spring). The gapping mentioned above is a nonlinear phenomenon and the Winkler model was used to calculate it. Dobry and Gazetas (1984) provide a simplified method for estimating nonlinear pile head stiffness and damping in layered soil deposits; they use a Winkler approach to estimate the pile head lateral stiffness and suggest a method for estimating the damping based on the deflected shape of the pile shaft. An alternative to calculations with a nonlinear Winkler spring model is equations given by Davies and Budhu (1986) and Budhu and Davies (1987) which give nonlinear relations between lateral load and pile head displacement and rotation. These
relations were developed for static lateral loading of piles; the justification for using them in a dynamic context is finding that the pile head stiffness is not greatly affected by frequency.

**Single Pile Capacity**

The vertical capacity can be estimated using standard methods. For dynamic loads the capacity is likely to be greater than static because of enhanced soil strength mentioned above. Similarly there are standard methods for estimating lateral capacity. However for earthquake loading there are likely to be simultaneous vertical and lateral cyclic loads so the question of the capacity under this combined loading arises. As the lateral capacity is generated in the upper part of the pile shaft, over a length similar to the active length used for stiffness, one could allocate this length to generating lateral resistance — even if this is likely to be expensive in most cases. Another issue is the effect of the cyclic vertical capacity on the pile shaft. It is well known that severe cyclic degradation of the pile shaft capacity can occur, although this is unlikely to be serious as long as there is no reversal of shear stress along the pile shaft.

**Piles in Liquefied Soil Deposits**

Inspired by the large number of observations of damage to piles by the flow of liquefied sand there has been extensive investigation of this problem in Japan. The results of this work are discussed by Yasuda and Berrill (2000) elsewhere in this volume.

**Pile Group Stiffness**

Once again the need is to consider stiffness for inertial and kinematic interaction as well as group capacity.

No special kinematic interaction calculation is necessary for pile groups as it has been found from numerical investigations that kinematic effects for pile groups are little different from those for individual piles.

In evaluating the dynamic stiffness of a pile group the interactions between the piles are important, in just the same way as they are for the static stiffness of a pile group. Early investigations of this using boundary element calculations (Kaynia and Kausel, 1982) revealed frequency dependence rather more complex than that for a single pile. Fortunately, a quite remarkable simplification was discovered (Dobry and Gazetas, 1988) which enables the dynamic stiffness components of a pile group to be evaluated by means of simple formulae, the essence of which is representing the interaction between the piles in the group by the propagation of cylindrical waves. Thus the expressions for the interaction coefficients between the piles are given in terms of pile spacing, excitation frequency, and the velocity of waves through the soil between the piles. If one models a building on a pile group foundation as an equivalent single degree of freedom system using the Dobry and Gazetas equations to handle the pile group, then it is found that the stiffness of the system exhibits much less frequency dependence than the pile group alone, Pender (1993). If the response spectrum method is used to estimate the maximum building displacement it is found that the values obtained using simply the static stiffnesses are very similar to those obtained using the frequency dependent stiffness and damping terms for the pile group.

**Nonlinear Effects**

Calculations using the above formula show that when a pile group is subject to a dynamic horizontal shear force the distribution of load between the piles is a function of the position of the pile in the group. Once again this mirrors the known behavior of pile groups subject to static shear forces. It has been observed that when 3x3 pile groups are subject to static lateral loads that for lateral displacements up to about 3% of the pile diameter the distribution of load between the piles follows the predictions calculated using elastic interaction coefficients. However, as the load is increased the piles in the leading row carry a larger proportion of the load and the rear a smaller fraction. This is known as shadowing and its existence for static loading can also be expected for dynamically loaded groups.
Dynamic testing of full scale pile groups has been very rare. Recent work (Halling et al, 2000) in which a 3x3 pile group was loaded statically and dynamically, with both sinusoidal and impulse excitation, reveals the major effect that a single loading event can have on the subsequent behavior of a pile group. The piles for this group were embedded in clay so part of the explanation might be related to the formation of gaps along the pile shafts.

Both of these observations indicate that nonlinear effects are as significant for pile groups as for individual piles. Regrettably no simple methods for estimating these effects are known to the authors.

Pile rafts

A pile raft derives part of its stiffness and capacity from the group of piles and the remainder from interaction between the underside of the raft and the soil. For pile groups which have piles passing through sand and bearing on firmer strata no interaction can be assumed with the underside of the raft as some settlement of the sand is expected, and has been observed in past earthquakes. For pile rafts in clay the interaction with the underside of the foundation arises but there is no simple method for estimating these effects. However, it is possible to place bounds on the stiffness contributions. This has been done (Pender, 1994) and it was found that the interaction between the underside of the raft and the supporting soil is likely to be most significant for the vertical stiffness and least significant for moment loading.

Pile Group Capacity

Under earthquake loading a pile group may be subjected to horizontal shear and moment loading in addition to changes in any static vertical load it will be required to carry. This suggests a surface similar to that in figure 8 for a shallow foundation. Such a treatment has not been developed and would involve interactions between the vertical capacity of the pile group, shear capacity and moment capacity, further the moment capacity of the pile shaft is also involved. In many applications the interaction between vertical and moment capacity is likely to be more critical as the moment resistance, or the greater part of it, comes from the axial capacity of the piles. Some preliminary work has been done on this for 2x2 and 3x3 pile groups by Pender (1993). Any moment capacity using the axial capacity of the pile shafts is subject to the potential degradation of the capacity mentioned above for single piles. This is not likely to be serious as long as there is no reversal of the shear stress on the pile shaft; whether or not reversal occurs is dependent on the ratio of the static vertical load carried by the group to the cyclic moment. Horizontal shear can often be equilibrated with lateral earth pressures at the sides of the raft rather than being transferred to the pile shafts.

Unresolved Issues and Future Developments

Probably the two most pressing issues here are those mentioned above: the load distribution in a group and shadowing, and the effect of earthquake loading on the subsequent dynamic stiffness of pile groups.

The most important way in which understanding of the dynamic response of pile group foundations can be enhance is by well documented testing of prototype scale pile groups – an expensive process.

CONSTRUCTION DETAILING

Although the safety of a constructed facility does not rely only upon a blind application of seismic codes and standards which are used for its design and construction, those documents help significantly to minimize the most commonly encountered causes of deficiencies and failures.

Because all phenomena described previously cannot be analyzed with the necessary mathematical rigor and are not often relevant to even sophisticated calculations, construction detailing must always be enforced in seismic design of foundations. This is one of the major merits of seismic codes.
Many of these detailing practices, which are found in the most recent codes, are little more than common sense and by no means, the points raised herein constitute an exhaustive list. However based on the authors' experience, they represent the most common mistakes made in design by non-experienced designers:

- Foundations must not be located close to (or across) major active faults. Ground motions in the near field are far from being predictable and attempts to design buildings to accommodate such movements, especially the static co-seismic displacements associated with fault rupture, are almost hopeless.
- Liquefiable deposits and unstable slopes must always be treated before construction. Even if a piled foundation can survive the cyclic deformations of a liquefied deposit, the quasi static displacements imposed by the post-earthquake ground flow are an order of magnitude larger and cause distress of the foundation as evidenced in the Kobe earthquake.
- The foundation system under a building must be as homogenous as possible unless construction joints are provided in the structure. In particular, for individual footings, the situation where some of them rest on a man-made fill and some on in-situ soils must always be avoided. It is also highly desirable that the foundations respect the symmetries of the building.
- The choice of the foundation system must always account for possible secondary effects such as settlements in medium-dense or loose dry sands, the post-earthquake consolidation settlements of clay layers, the settlements induced by the post-earthquake dissipation of pore pressures in a non liquefiable sand deposit. Raft foundations or end bearing piles are to be preferred whenever the anticipated magnitude of the settlements is high or when they can be highly variable across the building.
- Individual footings must always be linked with tie beams at the foundation level. These longitudinal beams must be designed to withstand the differential settlements between the footings.
- Piles must be reinforced along their whole length, even if calculations do not require reinforcement. Special care must be given to the connections with the raft or to soil layers interfaces when two layers in contact present marked differences in stiffness. For instance, the connection with the raft can be detailed to allow for a plastic hinge as allowed in Eurocode 8.
- Inclined piles must preferably be avoided: they can be subjected to parasitic bending stresses due to soil densification following an earthquake, they induce large forces onto pile cap and, if their arrangement is not symmetric, permanent rotations may develop due to different stiffness of the pile group in each direction of loading.

REFERENCES


