Sven Hansbo Lecture:
Deep foundation design - Issues, procedures and inadequacies

Harry G. Poulos

Coffey Services, Australia. E-mail: harry.poulos@coffey.com

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ABSTRACT: This paper reviews the key issues that must be addressed in the design of deep foundations, especially for support of high-rise buildings. Common design criteria are then set out, and a three-stage design process is outlined. The importance of geotechnical site characterization is emphasised. Some of the available design tools are discussed, together with their limitations, and a comparison is made of analysis results from four different programs. Finally, some perceived inadequacies of common design procedures are discussed, including: ignoring foundation interactions, assuming a rigid raft, over-simplification of the ground profile, ignoring external ground movements, and ignoring kinematic seismic effects. Examples of the consequences of these inadequate procedures are illustrated via relevant case histories.

1. INTRODUCTION

Geotechnical engineers involved in the design of deep foundations for are increasingly leaving behind empirical methods and adopting state-of-the art procedures. Foundation and building behaviour is highly interactive, with the building loads influencing the foundation movements, which in turn influence the behaviour of the building. Foundation behaviour is mainly governed by the prevailing ground conditions, the foundation type, and the magnitude and distribution of the building loads. Foundation design should therefore be considered as a performance-based soil-structure interaction (SSI) issue and not limited to traditional empirically based design methods, such as a bearing capacity approach with an applied factor of safety.

The main elements in foundation design include the building loads, the ground conditions and the required building performance, as well as the other economic factors such as local construction conditions, cost and project program requirements

The critical factor in deep foundation design, especially for tall buildings, is often foundation settlement and lateral movement, rather than ultimate foundation stability, and so attention will be concentrated mainly on the prediction of foundation deformations. However, some issues related to geotechnical and structural strength of the foundation system itself will also be addressed.

2. DESIGN ISSUES

The following key issues need to be addressed in the design of deep foundations:

- Ultimate capacity and global stability of the foundation system under vertical, lateral and moment loading combinations.
- The influence of the cyclic nature of wind and earthquakes on foundation capacity and movements.
- Overall foundation settlements.

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- Differential settlements, both within the structure footprint, and between high-rise and adjacent low-rise areas.
- Possible effects of any externally-imposed ground movements on the foundation system, for example, movements arising from excavation and construction operations.
- Earthquake effects, including the response of the structure-foundation system to earthquake excitation, and the possibility of liquefaction in the soil surrounding and/or supporting the foundation.
- Dynamic response of the structure-foundation system to wind-induced forces.
- Structural design of the foundation system, including the load-sharing among the various components of the system (i.e. the piles and the supporting raft), and the distribution of loads within the piles.

3. DESIGN CRITERIA

Most modern methods of design now use limit state design concepts in which consideration is given to three main criteria:

- The ultimate limit state for geotechnical strength and stability;
- The ultimate limit state for structural strength and stability;
- The serviceability limit state.

The criteria for each of these aspects are discussed below.

3.1 Ultimate Limit State for Geotechnical Strength and Stability

In terms of limit state design using a load and resistance factor design approach (LRFD), the design criteria for the ultimate limit state for geotechnical design is as follows:

$$R_{dg} \geq E_d \tag{1}$$

where R_{dg} = design geotechnical strength = $\phi_g \times R_{ug}$; R_{ug} = ultimate geotechnical capacity; ϕ_g= geotechnical reduction factor; E_d = factored combination of loadings.

This criterion is applied to the entire foundation system. It is not considered to be good practice to apply the geotechnical criterion to each individual pile within the group, as this can lead to considerable over-design (Poulos, 1999).

The ultimate geotechnical capacity R_{ug} can be obtained from conventional methods of design, depending on the available geotechnical data. For example, various methods are discussed by Randolph (2003) and Poulos (2017).

The selection of suitable values of ϕ_g requires considerable judgement and should take into account a number of factors that may influence the foundation performance. As an example, the now-superseded Australian Piling Code AS2159-1995 specifies values of ϕ_g between 0.4 and 0.9, the lower values being associated with greater levels of uncertainty and the higher values being relevant when a significant amount of load testing is carried out.

A later version of this standard, AS2159-2009, employs a risk assessment approach to arrive at an appropriate geotechnical reduction factor, depending on a number of issues, as follows:

- The geological complexity of the site;
- The extent of ground investigation;
- The amount and quality of geotechnical data;
- Experience with similar foundations in similar geological conditions;
- The method of assessment of geotechnical parameters for design;
- The design method adopted;
- The method of utilizing the results of in-situ test data and pile installation data;
- The level of construction control;
- The level of performance monitoring of the supported structure during and after construction.

Each of these factors is given a subjective risk rating, ranging between 1 for very low risk, to 5 for very high risk. The individual risk ratings are weighted via an importance factor for that factor, and then an average risk rating (again between 1 and 5) is computed from the sum of the individual weighted risk factors. The higher the average risk rating, the lower is the geotechnical reduction factor. Some benefit is derived by having a high redundancy foundation system, for example, a large group of piles, or a piled raft foundation. Load testing provides further benefits and leads to a higher ϕ_g value, i.e. a less conservative design.

ϕ_g can typically range between 0.4, for conservative designs involving little or no pile testing and where uncertain ground conditions prevail, to 0.8, for cases in which a significant amount of testing is carried out and the ground conditions and design parameters have been carefully assessed.

The required load combinations for which the structure and foundation system have to be designed will usually be dictated by an appropriate structural loading code. In some cases, a large number of combinations may need to be considered.

In addition to the criterion in equation 1, it is considered prudent that an additional criterion should be imposed for the piled foundation of a tall structure to cope with the effects of repetitive loading from wind and/or wave action, as follows:

$$\eta R_{gs} \geq E_c \tag{2}$$

where R_{gs} = ultimate geotechnical shaft capacity; E_c = maximum half-amplitude of cyclic wind loading; η = cyclic load ratio.

This criterion attempts to avoid the full mobilization of shaft friction along the piles, thus reducing the risk that cyclic loading will lead to a degradation of shaft capacity. E_c can be obtained from computer analyses which gave the cyclic component of load on each pile, for various wind or seismic loading cases.

For the Emirates project in Dubai (Poulos and Davids, 2005), η was selected as 0.5, based on laboratory data from laboratory constant normal stiffness (CNS) tests.

3.2 Ultimate Limit State for Structural Strength and Stability

The design criterion for this limit state can be expressed as follows:

$$R_{ds} \geq E_d \tag{3}$$

where R_{ds} = design structural strength = $\phi_s \times R_{us}$; R_{us}= ultimate structural strength; ϕ_s = structural reduction factor.

This criterion is applied to the entire foundation system, and also to each individual pile within the system. R_{us} can be obtained from the estimated ultimate structural capacity via an appropriate structural analysis. The strength reduction factor is usually derived from the relevant design standards or codes.

3.3 Serviceability Limit State

The design criteria for the serviceability limit state can be stated as follows:

$$\rho_{max} \leq \rho_{all} \tag{4}$$

$$\theta_{max} \leq \theta_{all} \tag{5}$$

where ρ_{max} = maximum computed settlement; ρ_{all} = allowable foundation settlement; θ_{max} = maximum local angular distortion; θ_{all} = allowable angular distortion.

Values of ρ_{all} and θ_{all} depend on the nature of the structure and the supporting soil. Some suggested criteria have been reported by Zhang and Ng (2006) for deep foundations. Commonly specified values of maximum allowable settlement tend to be between 25 and 75 mm, depending on the nature of the building. Criteria specifically for very tall buildings do not appear to have been set, but it should be noted that it may be unrealistic to impose very stringent settlement criteria on very tall buildings on clay deposits, as they may not be achievable. For example, experience with tall buildings in Frankfurt Germany suggests that total settlements in excess of 100 mm can be tolerated without any apparent impairment of function.

Figure 1 shows a suggested approach to the acceptable angular distortion, θ_{all}, of structures, based on Juang et al. (2011). This figure shows that θ_{all} depends on the lateral strain to which the foundation is subjected, and that the probability of building damage increases significantly as the lateral strain increases. However, for most tall building foundations, the foundation system will be connected to a raft or slab which will largely inhibit lateral strains. A common criterion is θ_{all} = 1/500 (0.002), and Figure 1 suggests that, for this value, there is a 20% possibility that damage could occur.

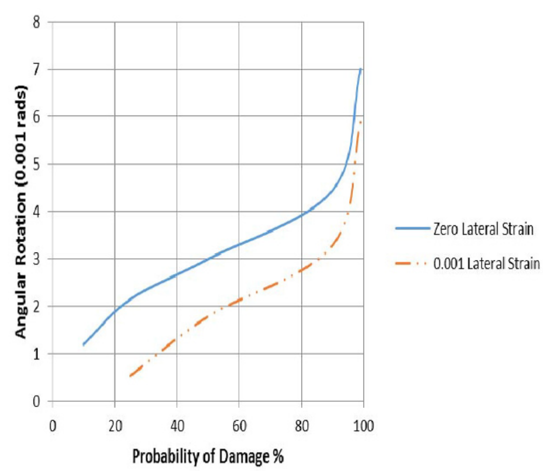

Figure 1. Angular distortion criteria (based on Juang et al. 2011)

It should also be noted that the allowable angular distortion, and the overall allowable building tilt, reduce with increasing building height, both from a functional and a visual viewpoint. It can also be noted that, in Hong Kong, the limiting tilt for most public buildings is 1/300 in order for lifts (elevators) to function properly.

4. DESIGN PROCEDURES

4.1 The Design Process

The following process can be employed for geotechnical assessment and deep foundation design:

- Geotechnical site characterization based on available ground investigation information and published data.
- Development of representative geotechnical model(s) for the site. For geologically complex sites, more than a single model may be required.
- Assessment of foundation requirements for ultimate limit state loads, including bearing capacity under vertical loadings and overall stability under combined loadings. These loadings, and those for serviceability, are provided by the structural designer.
- Assessment of foundation performance under serviceability loads (foundation settlements, differential settlements and lateral movements).
- Assessment of effects of cyclic loading on foundation capacity and deformations (including cyclic degradation).
- Assessment of loads and bending moments required for structural design of the foundation elements.
- Assessment of dynamic response (stiffness and damping) of the foundation system.
- Assessment of possible seismic effects, including site amplification, kinematic and inertial loadings on foundations, and liquefaction potential.
- Consideration of the effects of dewatering, excavation and other construction activities.
- Evaluation of load test data and modification, if necessary, of foundation design parameters.
- Evaluation of measured performance in relation to predicted performance.

It is sound practice for the geotechnical designer to work closely with the structural designer. The superstructure and the foundation are interacting components of a single system, and should not be treated as independent entities. Such interaction can lead to more effective structural design of the foundation elements, and also, in many cases, to more realistic loadings and foundation responses.

It is also highly desirable for the geotechnical designer to be involved in the measurements of foundation performance during and after construction, particularly settlements, to allow proper assessment of that performance in relation to design expectations. If there are major differences, then it may still be possible to make amendments to the foundation design if that is deemed to be necessary.

4.2 Stages of Design

The following design stages can be employed for foundation design:

- Concept Design;
- Detailed Design;
- Final Design.

These stages are described in more detail below, together with the activities that are required. The procedures employed for each stage should be consistent with the level of detail required. For example, sophisticated numerical analyses normally would not be appropriate for the concept design stage.

4.2.1 Concept Design

The aim of the Concept Design stage is to firstly establish the foundation system and to evaluate the approximate foundation behaviour. A preliminary ground model is developed, based on the available borehole information in the vicinity of the site, supplemented with any relevant published data and information from other sources.

In collaboration with the structural designers, a concept foundation layout is then developed and its performance under preliminary ultimate and serviceability loadings is assessed. Various foundation options are usually examined in this stage.

A Concept Design Stage report is prepared, summarizing the preliminary geotechnical model, the findings of the analyses undertaken, and details of the most feasible foundation options to be considered further.

4.2.2 Detailed and Final Phases of Design

In the Detailed Design stage, pile geotechnical capacities and stiffnesses are assessed for a range of pile diameters and preliminary pile layout options for various pile diameters. The foundation layout is adjusted and optimized to try and provide the most economical foundation system that satisfies the various design criteria.

The Final Design stage usually involves the use of a refined analysis to check the optimized solution developed in the Detailed Design phase. It provides the final values of predicted foundation performance and of pile stiffness characteristics that are then used by the structural designer.

4.3 Design Analyses

A summary of the analyses that are recommended to be carried out for building foundation design are shown in Table 1. These analyses involve various combinations of factored/unfactored geotechnical strengths and Ultimate Limit State (ULS) or Serviceability Limit State (SLS) loadings.

Table 1. Summary of Design Analyses

Case	Purpose	Factor applied	Load Case
i	Geotechnical Design Capacity	φ_g	ULS
ii	Structural Design Capacity	1.0	ULS
iii	Serviceability	1.0	SLS

It should be emphasized that when considering the structural design case, a geotechnical reduction factor should not be applied to the pile resistances, otherwise an unrealistic limit will be imposed on the computed forces and moments in the piles.

In addition, it should be recognized that the soil stiffness values used in the design analyses should be relevant to the loading condition being considered. Thus, for cases involving wind loading, short-term parameters should be used, whereas for long-term conditions under dead and live loading, long-term geotechnical parameters would be relevant.

Short-term soil stiffness parameters are generally larger than the corresponding long-term parameters, especially for fine-grained soils.

The above analyses should be applied to the entire foundation system, and will involve consideration of issues such as group efficiency and pile-soil-pile interaction. The criteria to be satisfied within each of the analyses are set out in Section 3 above.

4.4 Design Inputs

The required inputs for a satisfactory design to be undertaken should include, but not necessarily be limited to, the following:

- The design criteria that are being sought, such as allowable settlement, differential settlement and tilt;
- The key geotechnical parameters: these are set out in Section 5.1 below.
- The design loadings;
- Details of the analysis methods to be employed and justification of their relevance.

4.5 Design Outputs

The outcome of the geotechnical design process is usually a report and drawings that include, but are not limited to, the following items:

- The interpretation of the geological and geotechnical characteristics of the site;
- The geotechnical design parameters that have been adopted;
- The loadings for which the design has been undertaken;
- Details of the assessment of the Ultimate Limit State adequacy of the foundation system;
- Details of the assessment of the Serviceability Limit State adequacy of the foundation system. Desirably, these should also include verification of the outcome via an independent analysis, albeit perhaps via a simplified method;
- Values of the stiffness of the raft and of each pile within the foundation system. This is primarily required for the structural designer to input into the structural model to undertake a complete analysis of the structure-foundation system.

In providing equivalent spring stiffness values for the piles, an analysis of the pile group or piled raft system needs to be undertaken. In such an analysis, the following suggestions are offered:

- For the vertical springs, it is preferable to consider an average "working" load acting on each pile, so that representative linear spring stiffness values can be obtained.
- For the raft, to avoid undue complexity, an average spring stiffness (or modulus of subgrade reaction) can be computed on the basis of the ratio of average raft pressure to average raft settlement.
- For the lateral and rotational springs, again it is preferable to apply an average "working" lateral load to each pile, and assume that the pile cap is able to rotate.

5. GROUND CHARACTERISATION

The assessment of a geotechnical model and the associated parameters for foundation design should first involve a review of the geology and hydrogeology of the site to identify any geological features that may influence the design and performance of the foundations. A desk study is usually the first step, followed by site visits to observe the topography and any rock or soil exposures. Local experience, coupled with a detailed site investigation program, is highly desirable.

The site investigation is likely to include a comprehensive borehole drilling and *in-situ* testing program, together with a suite of laboratory tests to characterize strength and stiffness properties of the subsurface conditions. Based on the findings of the site investigation, the geotechnical model and associated design parameters are developed for the site, and then used in the foundation design process.

The in-situ and laboratory tests are desirably supplemented with a program of instrumented vertical and lateral load testing of prototype piles (e.g. bi-directional load cell tests such as the Osterberg Cell, Osterberg, 1989) to allow calibration of the foundation design parameters and hence, to better predict the foundation performance under loading. Completing the load tests on prototype piles prior to final design can provide confirmation of performance (i.e. pile construction, pile performance, ground behaviour and properties) or else may provide data for modifying the design prior to construction.

5.1 Key Parameters

For contemporary foundation systems that incorporate both piles and a raft, the following parameters require assessment:

- The ultimate skin friction for piles in the various strata along the pile.
- The ultimate end bearing resistance for the founding stratum.
- The ultimate lateral pile-soil pressure for the various strata along the piles
- The ultimate bearing capacity of the raft.
- The stiffness of the soil strata supporting the piles, in the vertical direction.
- The stiffness of the soil strata supporting the piles, in the horizontal direction.
- The stiffness of the soil strata supporting the raft.

It should be noted that the soil stiffness values are not unique values but will vary, depending on whether long-term drained values are required (for long-term settlement estimates) or short-term undrained values are required (for dynamic response to wind and seismic forces). For dynamic response of the structure-foundation system, an estimate of the internal damping of the soil is also required, as it may provide the main source of damping. Moreover, the soil stiffness values will generally tend to decrease as either the stress or strain level increases.

5.2 Methods of Parameter Assessment

The following techniques are used for geotechnical parameter assessment:

- Empirical correlations – these are useful for preliminary design, and as a check on parameters assessed from other methods.
- Laboratory testing, including triaxial and stress path testing, resonant column testing, and constant normal stiffness (CNS) testing.
- In-situ testing, including various forms of penetration testing, pressuremeter testing, dilatometer testing, and geophysical testing.
- Load testing, generally of pile foundations at or near prototype scale. For large diameter piles, or for barrettes, it is increasingly common to employ bi-directional testing to avoid the need for substantial reaction systems.

Detailed discussions of the methods of parameter assessment are available in several references, including Fleming et al. (2009), Tomlinson (2004), Poulos and Badelow (2015) and Poulos (2017).

5.3 Geophysical Testing

Geophysical testing is becoming more widely used in geotechnical investigations. At least three major advantages accrue by use of such methods:

- Ground conditions between boreholes can be inferred.
- Depths to bedrock or a firm bearing stratum can be estimated.
- Shear wave velocities in the various layers within the ground profile can be measured, and tomographic images developed to identify any vertical and lateral inhomogeneity.

From the measured shear wave velocity, v_s, the small-strain shear modulus, G_{max}, can be obtained as follows:

$$G_{max} = \rho v_s^{\,2} \tag{6}$$

where ρ = mass density of soil.

For application to routine design, allowance must be made for the reduction in the shear modulus because of the relatively large strain levels that are relevant to foundations under normal serviceability conditions. As an example, Poulos et al. (2001) have suggested the reduction factors shown in Figure 2 for foundations on clay soils, for the case where G_{max}/s_u = 500 (s_u = undrained shear strength).

This figure indicates that:

- The secant modulus for axial loading may be about 20-40% of the small-strain value for a practical range of factors of safety;
- The secant modulus for lateral loading is smaller than that for axial loading, typically by about 30% for comparable factors of safety.

An important outcome of the strain-dependence of soil stiffness is that the operative soil modulus below the foundation system will tend to increase with depth, even within a homogeneous soil mass.

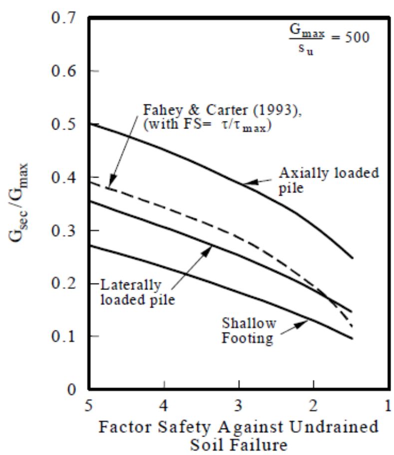

Figure 2. Example of secant shear modulus to small-strain value (Poulos et al 2001)

When modeling a foundation system using a soil model that does not incorporate the stress- or strain–dependency of soil stiffness, it is still possible to make approximate allowance for the increase in stiffness with increasing depth below the foundation by using a modulus that increases with depth. From approximate calculations using the Boussinesq theory to compute the distribution of vertical stress with depth below a loaded foundation, it is possible to derive a relationship between the ratio of the modulus to the small strain value, as a function of relative depth and relative stress level. Such a relationship is shown in Figure 3 for a circular foundation, with an overall factor of safety of 2 (a ratio p/p_u of applied pressure to ultimate pressure of 0.5). This may be used as an approximate means of developing a more realistic ground model for foundation design purposes. When applied to pile groups, the diameter can be taken as the equivalent diameter of the pile group, and the depth is taken from the level of the pile tips.

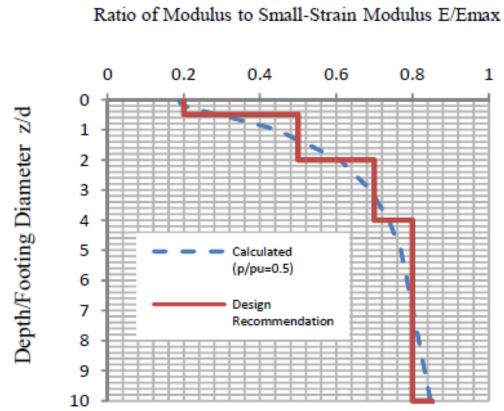

Figure 3. Ratio of operative modulus to small-strain modulus below circular foundation.

6. DEEP FOUNDATION INTERACTIONS

Deep foundation systems involve interactions through the ground in which the systems are located. Such interactions can be an important influence on the foundation behaviour, and a brief review will be given below of interactions that occur in two common foundation systems: pile groups, and piled rafts.

6.1 Pile Groups

When analyzing a group of piles, consideration should be given to the interaction between the piles in the group. Under vertical loading, such interaction leads to an increase in settlement as compared with a single isolated pile under the same average load.

One of the common means of analyzing pile – pile interaction is via the interaction factor method described by Poulos and Davis (1980). In this method, referring to Figure 4, the settlement w_i of a pile i within a group of n piles is given as follows:

$$w_i = \sum_{j=1}^{n} (P_{av} S_1 \alpha_{ij}) \tag{7}$$

where P_{av} = average load on a pile within the group; S_1 = settlement of a single pile under unit load (i.e., the pile flexibility); α_{ij} = interaction factor for pile i due to any other pile (j) within the group, corresponding to the spacing s_{ij} between piles i and j.

Eq. 7 can be written for each pile in the group, thus giving a total of n equations, which together with the equilibrium equation, can be solved for two simple cases:

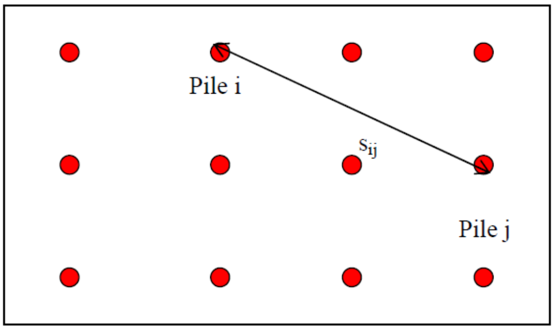

Figure 4. Superposition via the interaction factor method – plan of pile group

- Known load on each pile, in which case the settlement of each pile can be computed directly. In this case, there will usually be differential settlements among the piles in the group.
- A rigid (non-rotating) pile cap, in which case all piles settle equally. In this case, there will be a uniform settlement but a non-uniform distribution of load in the piles.

In contrast to the original approach suggested by Poulos (1968) in which the interaction factors are applied to the whole of the single pile settlement, Mandolini and Viggiani (1997) and Randolph (1994) have argued that the interaction factor should only be applied to the elastic component of settlement of an adjacent pile, since the plastic component of settlement is due to a localized phenomenon and is not transmitted to the adjacent piles. In this case, the settlement of a pile i in the group is then given by:

$$w_i = \sum_{j=1}^{n} (P_{av} S_{1e} \alpha_{ij}) \tag{8}$$

where S_{1e} is the elastic flexibility of the pile.

By further assuming that the load-settlement behaviour of the pile is hyperbolic, Mandolini and Viggiani (1997) expressed the interaction factor, α_{ii}, for a pile i due to its own load as:

$$\alpha_{ii} = 1/(1 - R_f P / P_u)^q \tag{9}$$

where R_f = hyperbolic factor (taken as unity); P = load on pile i; P_u = ultimate load capacity of pile i; q = analysis exponent = 2 for incremental non-linear analysis and 1 for equivalent linear analysis.

This approach is recommended as being more logical than the original approach.

6.1.1 A Note on Interaction Factors

The interaction factors can be computed by various methods, including via the use of elastic theory using Mindlin's equations in a boundary element analysis. However, the resulting interaction factors may over-estimate the interaction effects for larger spacings for the following reasons:

- For soil profiles in which the ground is not homogeneous and whose stiffness increases with depth, the interaction factors are smaller than those for a corresponding homogeneous soil mass;
- In large pile groups, interaction effects may be reduced by the presence of piles between the two piles being considered.
- In real soils, the stiffness increases with decreasing strain level away from the pile. Thus, assuming that the soil between two piles is homogeneous will trend to over-estimate the interaction effects.

From a practical viewpoint, it is therefore important to estimate the interaction factors by taking account of the non-homogeneity of the soil with depth, and by allowing for stiffer soil between the piles than at the pile circumference, for example, via the approximate approach suggested by Poulos (1988). In addition, an upper limit should be placed on the maximum spacing at which interaction occurs; experience suggests that a maximum spacing of the lesser of 20 diameters or the pile length, appears to be reasonable.

6.2 Piled Rafts

Within a piled raft foundation, there are four interactions that need to be considered, rather than just the one mentioned above. These interactions are:

- Pile-pile interaction;
- Pile-raft interaction;
- Raft – pile interaction;
- Raft – raft interaction.

Approximate methods of accounting for pile-raft and raft-pile interactions are discussed by Poulos (1994). These interactions are often overlooked, especially by structural designers who adopt a subgrade reaction analysis for the raft behavior, and who may also consider the piles as independent springs. The consequences of ignoring these interactions will be examined later in this paper.

7. DESIGN TOOLS

7.1 Concept Design

For the concept design phase, one can make use of spreadsheets, MATHCAD sheets, or simple hand or computer methods which are based on reliable but simplified methods. It can often be convenient to simplify the proposed foundation system into an equivalent pier and then examine the overall stability and settlement of this pier. For the ultimate limit state, the bearing capacity under vertical loading can be estimated from the classical approach in which the lesser of the following two values is adopted:

- The sum of the ultimate capacities of the piles plus the net area of the raft (if in contact with the soil);
- The capacity of the equivalent pier containing the piles and the soil between them, plus the capacity of the portions of the raft outside the equivalent pier that are ion contact with the ground.

In using the equivalent pier method for assessment of the average foundation settlement under working or serviceability loads, the elastic solutions for the settlement and proportion of base load of a vertically loaded pier (Poulos, 1994) can be used, provided that the geotechnical profile can be simplified to a soil layer overlying a stiffer layer. It should be recognized that such simplified methods cannot readily consider the effects of lateral and moment loading, which can have a significant effect on foundation design. Such loadings are generally dealt with during detailed and final design.

For these detailed and final design stages, more refined techniques are generally required than for preliminary design, and the programs used should ideally have a number of capabilities.

For overall stability, the program should be able to consider:

- Non-homogeneous and layered soil profiles;
- Non-linearity of pile and, if appropriate, raft behaviour;
- Geotechnical and structural failure of the piles (and the raft);
- Vertical, lateral and moment loading (in both lateral directions), including torsion;
- Piles having different characteristics within the same group.

For serviceability analysis, the above characteristics are also desirable, and in addition, the program should have the ability to consider:

- Pile-pile interaction, and if appropriate, raft-pile and pile-raft interaction;
- Flexibility of the raft or pile cap;
- Some means by which the stiffness of the supported structure can be taken into account.

There do not appear to be any commercially available software packages that have all of the above desirable characteristics, other than three-dimensional finite element packages such as PLAXIS 3D or ABAQUS, or the finite difference program FLAC3D.

The pile group analysis programs REPUTE, PIGLET and DEFPIG have some of the requirements, but fall short of a number of critical aspects, particularly in their inability to include raft-soil contact and raft flexibility.

Some proprietary programs, such as GARP (Small and Poulos, 2007) remove some of these limitations, and such programs are useful tools for the detailed design stage, provided their limitations are recognised and (if possible) compensated for.

7.2 Comparison of Some Design Software Packages

Pirello and Poulos (2013) have compared four different pile analysis programs, PIGLET (Randolph (2004), CLAP (a development of the program DEFPIG (Poulos, 1990), REPUTE (GeoCentrix, 2013)) and PLAXIS 3D. One of the cases they considered was the complex foundation system involving 172 piles for the proposed Incheon Tower in South Korea. The same parameters were employed in each of the programs used.

Figure 5 shows an artist's impression of the completed tower.

The foundation plan for the Incheon Tower is shown in Figure 6; further details can be found in Abdelrazaq et al (2011).

The four programs were used with the following combination of loads:

- Vertical load = 6560.4 MN
- Lateral Load (x-direction) = 149 MN
- Bending moment (x-direction) = 21600 MNm
- Lateral Load (y-direction) = 114.6 MN
- Bending moment (y-direction) = 12710 MNm
- Torsional load = 1996 MNm.

The loads were applied at the centre of the foundation layout for CLAP, PIGLET and REPUTE. For PLAXIS 3D, moment and torsion could not be applied directly, and so these loadings were represented by equivalent equal and

Figure 5. Incheon Tower (artist's impression).

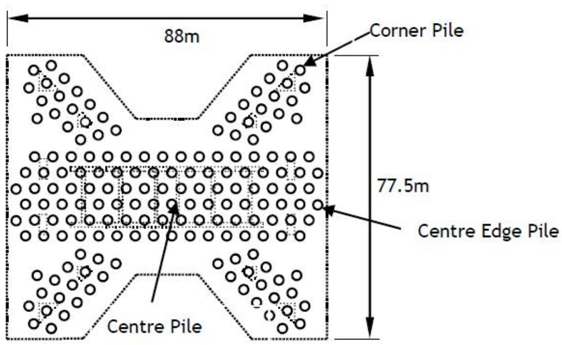

Figure 6. Pile Layout Plan Incheon Tower (Abdelrazaq et al., 2011).

opposite point loads, as discussed by Poulos et al. (2011).

Table 2 shows the results of the analyses. It should be noted that the PIGLET results quoted in the original paper were in error, and Table 2 shows the corrected values. In Table 2, the symbols are as follows:

S_c = central settlement, ρ_x and ρ_y = horizontal displacements in the x- and y-directions, θ_x and θ_y are rotations in the x- and y-directions, θ_z = torsional rotation, $P_{\max}$ = maximum axial pile load, $M_{x\max}$ and $M_{y\max}$ = maximum pile moments in x- and y-directions, $M_{z\max}$ = maximum torsional moment.

Table 2 indicates that all four programs give similar results for the key aspects of behaviour. The maximum moments given by PIGLET and REPUTE tend to be smaller those for the other two programs, but overall, the agreement is reasonable and suggests that all programs provide an adequate basis for foundation design, provided that the geotechnical parameters are assessed appropriately.

Table 2. Comparison of Solutions for Incheon Tower

Quantity	*CLAP*	*PIGLET*	*REPUTE*	*PLAXIS3D*
S_c (mm)	53	58	55	56
ρ_x (mm)	19	20	21	19
ρ_y (mm)	15	15	18	15
θ_x (rad)	.0002	.0002	.0002	.0002
θ_x (rad)	.0002	.0002	.0000	.0002
θ_z (rad)	.0004	.0001	.0003	.0003
P_{max} (MN)	84.6	83.5	84.8	83.0
H_{xmax} (MN)	2.7	4.3	3	2.5
H_{ymax} (MN)	2.6	3.1	2.8	2.2
M_{xmax} (MNm)	22.9	18.5	21.4	20
M_{ymax} (MNm)	22.9	13.3	18.5	21
M_{zmax} (MNm)	3.7	0.5	1.0	2.5

8. SOME INADEQUACIES OF COMMON DESIGN PROCEDURES

This section examines some aspects of common deep foundation design that the author considers may be inadequate. The following aspects are considered:

- Ignoring foundation interactions;
- Ignoring the beneficial effect of the raft;
- Assuming a rigid cap or raft;
- Over-simplification of the geotechnical profile;
- Ignoring the beneficial effects of basement walls;
- Ignoring the effects of ground movements;
- Ignoring kinematic effects in seismic design.
- Assuming purely elastic behaviour of the pile material.

Each inadequate aspect will be considered in turn, with examples given of the possible consequences.

8.1 Example of Ignoring Interaction Effects

As an example of the effect of ignoring interaction within a piled raft system, the case is considered of a high-rise building in Doha, Qatar, an impression of which of shown in Figure 7.

The tower was designed to have a central high-rise tower 510 m tall, which was to be surrounded by a low-rise podium area. The foundation system was designed as a piled raft.

Figure 7. Doha tower (artist's impression)

The foundation system is shown in Figure 8 and consisted of the following components:

- 525 piles, with diameters of 1.0, 1.2 and 1.5 m.
- Piles founded at four different levels: RL -26 m, -29 m, -54 m, and -60 m.
- A raft thickness of 4.0 m for the majority of the foundation footprint.
- Locally thickened areas of the raft beneath the lift over-run and core-wall areas of 6 m, 8.3 m and 12 m thickness.
- A raft thickness of 0.8 m below the outer podium area.

The geotechnical model developed for the site was based on available in-situ and laboratory test data, and is shown in Table 3. The column loads were applied as uniformly distributed loads over the base area of the columns for the assessment of the pile loads and as one uniformly distributed load over the whole area of the tower footprint (0.95 Mpa) for the assessment of the settlement and pile stiffness values. The serviceability assessment used the dead load plus the live load.

Analyses were undertaken to compute the settlement and pile load distribution within the foundation system, taking account of the flexibility of the raft foundation. The computer program GARP (Small and Poulos, 2007) was employed, using a finite element formulation to model the raft

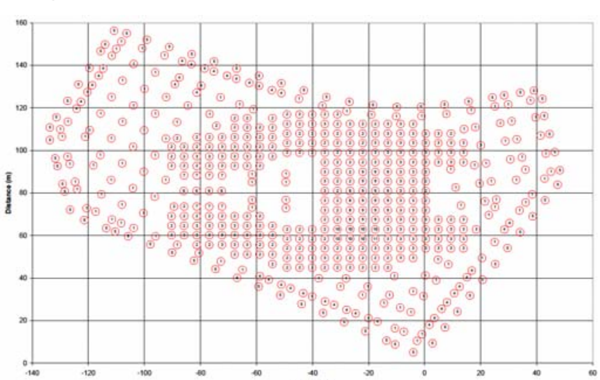

Figure 8. Foundation layout

Table 3. Geotechnical model for Doha site

Stratum	*Top RL (m)*	*Thickness (m)*	*E'$_v$ (Mpa)*	*E'$_h$ (MPa)*	*f_s (kPa)*	*f_b (kPa)*
Simsima L'stone	-18	3.5	2500	1750	600	-
Midra Shale	-21.5	3.0	700	490	525	-
Rus (1)	-24.5	75.5	500	350	425	5.9
Rus (2)	-100	Large	1000	-	-	-

and idealizing the piles as non-linear interacting springs. A raft thickness of 4 m was used in the analyses, with the finite element mesh for the raft having a total of 945 elements and 3006 nodes. The analyses carried out are listed in Table 4.

Table 4. Summary of analyses for Doha Tower

Run No.	Details
Q1	Normal analysis – all interactions included
Q2	Zero pile-pile interactions, but raft-raft, pile-raft and raft-pile interactions included
Q3	Zero pile-pile, pile-raft and raft-pile interactions; only raft-raft interaction accounted for

The computed maximum settlement for the three cases in Table 4 are shown in Figure 9. It can be seen that ignoring the pile-pile interactions reduces the maximum settlement from 81 mm to 41 mm, and ignoring the raft-pile and pile–raft interactions as well further reduces the maximum settlement to 20 mm. Thus, the settlement could be underestimated by a factor of 4 in this case if no consideration is given to the interactions among the piles and with the raft. Unfortunately, such an approach is not uncommon among designers who are focused primarily on the structure itself.

The effect of ignoring interactions on the maximum rotation and the maximum raft bending moment in the x-direction are shown in Figures 10 and 11. The computed rotation becomes smaller if the interactions are ignored, with the maximum computed rotation decreasing by about 25% if all interactions are ignored. In contrast, the effect of ignoring interactions on the maximum bending moment is less marked.

Figure 12 compares the calculated maximum axial load in any of the piles. Ignoring all interactions (other than raft-raft) leads to a significant increase in the maximum pile load almost 25%), and consequently, to more stringent requirements for reinforcement of the piles.

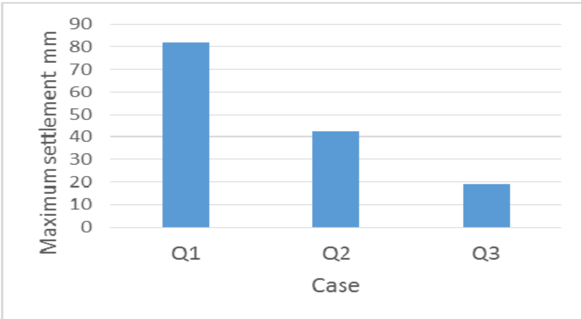

Figure 9. Effect of ignoring foundation component interactions on maximum settlement

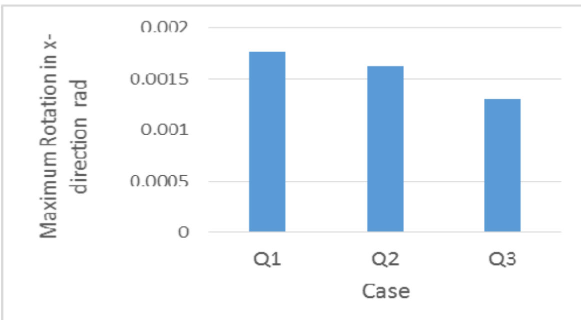

Figure 10. Effect of ignoring foundation component interactions on maximum x-rotation

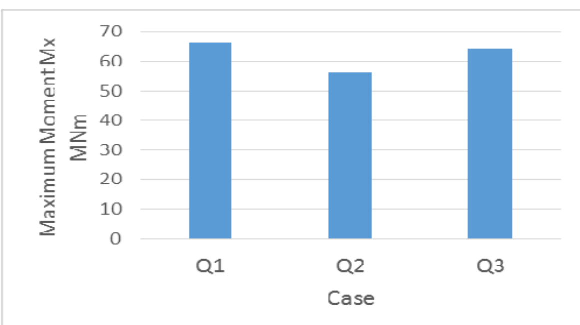

Figure 11. Effect of ignoring foundatiuon component interactions on maximum x-moment

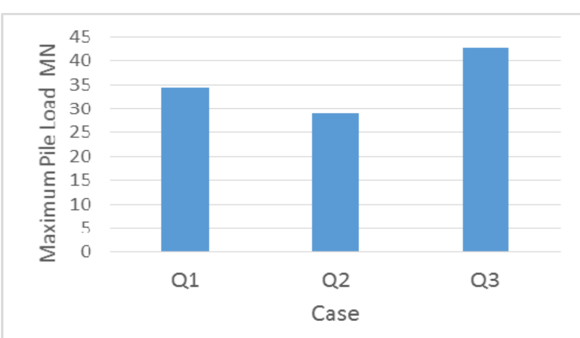

Figure 12. Effect of ignoring foundation component interactions on maximum axial pile load

Clearly, it is vitally important not to ignore the interactions that exist within a pile raft foundation system. To do so gives an unconservative estimate of settlement and differential settlement, but a conservative estimate of axial pile loads.

8.2 Ignoring the Presence of the Raft

It is not uncommon for foundation designers to ignore the effect of raft-soil contact and to assume that the piles carry the entire structural load.

For the same case as considered above, ignoring the effect of the raft-soil contact can be simulated by setting the limiting pressure on the base of the raft to (almost) zero. All the load is then carried by the piles, which are now free-standing. Table 5 compares various aspects of the computed behavior of the foundation system. The following characteristics are noted when the presence of the raft is ignored:

- The computed maximum settlement is increased dramatically.
- The maximum rotation is increased dramatically.
- The maximum pile load is almost doubled.
- The maximum bending moment in the raft is increased, but by a more modest amount than the other characteristics.

It seems clear that such a design, based on this over-conservative approach, would be inadequate and would almost certainly not satisfy the serviceability criteria, despite the presence of 525 piles in the system. However, by taking rational account of the presence of the raft, the settlements and rotations of the foundation are much more likely to be acceptable and to satisfy the serviceability design criteria.

Table 5. Effect of Ignoring the Raft

Value	Allowing for raft	Ignoring the raft
Max settlement (mm)	81.8	174.6
Min settlement (mm)	7.6	7.2
Max x-rotation rad	0.00176	0.01260
Max x-moment (MNm)	66.3	70.3
Min x-moment (MNm)	-46.2	-45.0
Max pile load (MN)	34.4	67.2
% load on raft	24.6	0

8.3 Assuming a Rigid Pile Cap or Raft

When designing or analysing pile groups or piled rafts, it is common to make the simplifying assumption that the pile cap or raft is perfectly rigid. Because rafts in some modern high-rise buildings can be as thick as 5-6 m, a rigid raft assumption may at first sight seem very reasonable. However, making this common assumption can lead to misleading outcomes, as it tends to over-estimate the loads in the outer piles within the system and under-estimate the loads in inner piles. As a consequence, the computed values of pile head stiffness may also be affected.

This leads on to the following important question: how thick does a raft have to be to be considered as rigid? To answer this question, recourse may be made to the work of Brown (1969), who considered the behaviour of a flexible circular raft on a finite elastic layer. Brown defined the relative flexibility of the raft via a factor *K*, given by:

$$K = \frac{E_r(1-\nu_s^{\,2})\left(t/a\right)^3}{E_s} \qquad (10)$$

where E_r = Young's modulus of raft; ν_s= Poisson's ratio of soil; t = raft thickness; a = raft radius; E_s = Young's modulus of soil.

Brown's results indicated that a raft could be considered as perfectly flexible if $K \leq 0.01$, and virtually rigid if $K \geq 10$.

The criterion for rigidity can be facilitated by assuming that the factor *K* also applies to a rectangular raft having an area equal to that of the circular raft. If the average dimension of the raft is *B*, so that the area is B^2, then the requirement for rigidity can be approximated as follows:

$$\left(t/B\right)_{rigid} \approx \sqrt{p.\,[K_{rigid}/\left(E_r/E_s\right).(1-\nu_s^{\,2})]^{1/3}} \qquad (11)$$

where K_{rigid} = value of *K* for a rigid raft, i.e. 10.

A similar equation can be written for the relative thickness, $(t/B)_{flex}$, when a raft is perfectly flexible, by substituting, in Eq. 11, the value of *K* for a flexible raft (i.e. 0.01) instead of that for a rigid raft.

Figure 13 plots the relationship between the relative raft thickness, *t/B*, for both rigid and flexible rafts, for typical values of E_r (30000MPa) and ν_s (0.3). Rafts with a *t/B* value on or above the line for a rigid raft would be classed as rigid, those falling on or below the line for a flexible raft would be flexible, while those falling between the lines for rigid and flexible rafts would be classed as partially flexible.

The following points can be noted:

- The value of $(t/B)_{rigid}$ for a rigid raft increases as the soil modulus increases.
- Even for very soft soils, for example E_s = 10 MPa, $(t/B)_{rigid}$ is about 0.25. Thus, for an average dimension of 50 m, the raft would need to be about 12.5 m thick to be truly rigid.

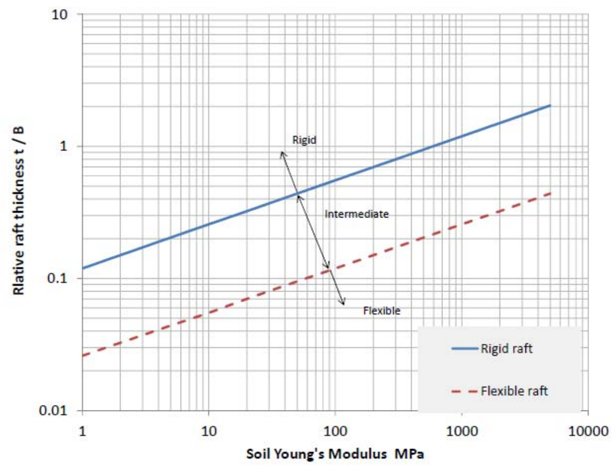

Figure 13. Thickness requirements for rigid and flexible rafts

- For a very stiff soil layer, for example, E_s = 500 MPa, $(t/B)_{rigid}$ is almost 1.0. Thus, for an average dimension of 50 m, the raft would need to be about 50 m thick!
- For a more common raft thickness of 3 m, a raft with an average dimension of 50 m would have *t/B* = 0.06, and this would be almost perfectly flexible even for a soft soil, and certainly perfectly flexible for the very stiff soil.

It therefore seems clear that pile caps and piled rafts supporting high-rise structures are likely to tend towards the perfectly flexible category.

As an example of the effects of assuming a rigid pile cap, the case of the 151 storey Incheon Tower, shown in Figure 5, has been considered.

The detailed design analyses were carried out using the program CLAP (Combined Load Analysis of Piles) for the ultimate limit state load cases (ULS) and the program GARP (Small and Poulos, 2007) for serviceability (SLS) loadings. As part of the design process, estimates were required of the maximum axial loads in each pile within the foundation system, and initially, the program CLAP was used. CLAP implicitly assumes that the raft supporting the piles is rigid, and as a consequence, the computed axial loads on some piles were found to be very large.

To investigate the effect of the rigid raft assumption, the foundation performance was re-assessed using GARP, taking the flexibility of the raft into account. The serviceability load case (i.e dead and live loads) was considered and the loads were applied at column and core locations.

Table 6 presents a summary of foundation settlement, axial loads and stiffness on the corner, centre edge and centre piles of the foundation (see Figure 6). The maximum predicted settlement occurred within the heavily loaded core area, while the computed pile stiffness values were greatest for the outer piles. As the analysis considered non-linear pile behaviour, the outer piles had a higher initial stiffness, but this stiffness degraded more rapidly under increasing loading than the central piles.

Table 6. Summary of foundation performance

		Rigid Raft	Flexible Raft
Pile Load (MN)	Centre Pile	24	49
	Centre Edge Pile	65	33
	Corner Pile	85	43
Pile Stiffness (MN/m)	Centre Pile	511	726
	Centre Edge Pile	1418	932
	Corner Pile	1604	1292
Raft Settlement (mm)	Maximum	52	67
	Minimum	26	28

Considering a rigid raft for the foundation, the total and differential settlements were predicted to be smaller, with higher pile head loads for corner and centre-edge piles, thus resulting in higher vertical pile stiffness values, especially on the outer piles, when compared with those for a flexible raft.

When the flexibility of the raft was incorporated, the pile load distribution was found to be more uniform, with slightly higher pile loads being predicted at the centre of the foundation where the heavily loaded core is located. The loads on piles for a rigid raft case were approximately two times the loads for a flexible raft, except for the centre piles.

It is interesting to refer to Figure 13 to assess the relative flexibility of the 5.5 m thick raft. The average dimension of the foundation was about 70 m, so that the ratio t/B was about 0.08. The average Young's modulus within a depth equal to this dimension was about 275 MPa, and for this modulus, the value of t/B for a rigid raft would be about 0.75, i.e. the raft would need to be about 52.5 m thick. In fact, even for a flexible raft, the value of t/B from Figure 13 would be about 0.17, so that the raft, with a t/B of less than half this value, could be classed as perfectly flexible. Based on the assessment, it is concluded that it is important to model the flexibility of the raft to avoid having to design for unrealistically large loads in the outer piles within the group, and also to obtain more realistic distributions of settlement within the foundation footprint.

8.4 Over-Simplification of the Geotechnical Profile

Over-simplification of the geotechnical profile can occur for several reasons, including:

- Inadequate ground investigation to an appropriate depth which will be influenced by the foundation;
- The use of simplified analysis and design tools that do not readily allow for variable ground conditions below the founding level of the foundation system;
- Inadequate attention given to the potential variability of ground stiffness with depth, even in a relatively homogeneous geo-stratum (refer to Figure 3).

Two examples will be given below to illustrate the consequences of ground profile over-simplification.

8.4.1 Assumed Uniform Conditions below Foundation Level

It is not uncommon for foundation analysts to assume that the ground conditions below the pile tips remain constant and extend to relatively large depths. The consequences of this assumption may be that pile-pile interactions are over-estimated. Such an over-estimation was experienced by the author in relation to the foundation design for the Emirates twin towers in Dubai (Poulos and Davids, 2005). These towers are shown in Figure 14.

Figure 14. The Emirates Towers soon after completion of construction

Predictions of the behavior of a single test pile were found to be in reasonable agreement with the measured behavior of test piles. On this basis, the parameters developed for a single pile were used to predict the settlement of the piled rafts supporting the two separate towers.

Conventional pile capacity analyses were used to assess the ultimate geotechnical capacity of the piles and raft. In additional to the conventional analyses, more complete analyses of the foundation system were undertaken with the computer program GARP (Poulos, 1994). This program utilized a simplified boundary element analysis to compute the behaviour of a rectangular piled raft when subjected to applied vertical loading, moment loading, and free-field vertical soil movements. The raft was represented by an elastic plate, the soil was modelled as a layered elastic continuum, and the piles were represented by hyperbolic springs which can interact with each other and with the raft. Beneath the raft, limiting values of contact pressure in compression and tension were specified, so that some allowance could be made for non-linear raft behaviour. In addition to GARP, the program DEFPIG (Poulos and Davis, 1980) was used for the pile stiffness values and pile-pile interaction factors, and for computing the lateral response of the piles.

For the analysis of settlements under the design loads, the same values of Young's modulus were used as for the single piles.

Measurements were available only for a limited period during the construction process and these are compared with the predicted time-settlement relationships in Figure 15 for typical points within the Hotel Tower. To the author's disappointment, it was found that, for both towers, the actual measured settlements were significantly smaller than those predicted, being only about 25% of the predicted values after 10-12 months.

The disappointing lack of agreement between measured and predicted settlement of the towers prompted a "post-mortem" investigation of possible reasons for the poor predictions. At least four such reasons were examined:

- Some settlements may have occurred prior to the commencement of measurements;
- The assumed time-load pattern may have differed from that assumed;
- The rate of consolidation may have been much slower than predicted;
- The interaction effects among the piles within the piled raft foundation may have been over-estimated.

Of these, based on the information available during construction, the first three reasons did not seem likely, and the last was considered to be the most likely cause. Calculations were therefore carried out to assess the sensitivity of the predicted settlements to the assumptions made in deriving interaction factors for the piled raft analysis with GARP. In deriving the interaction factors originally used, it had been assumed that the soil or rock between the piles had the same stiffness as that around the pile, and that the rock below the pile tips had a constant stiffness for a considerable depth. In reality, the ground between the piles is likely to be stiffer than near the piles, because of the lower levels of strain, and the rock stiffness below the pile tips is likely to increase significantly with depth, both because of the increasing level of overburden stress and the decreasing level of

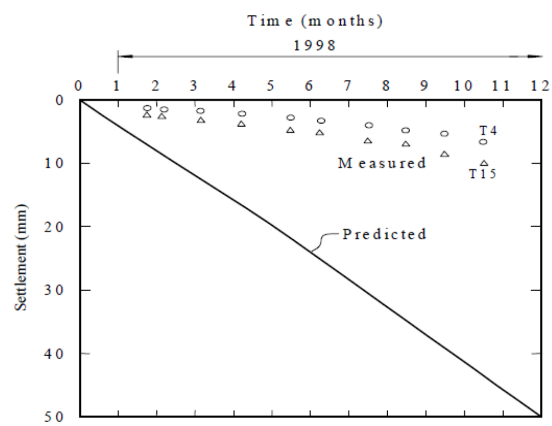

Figure 15. Predicted and measured time-settlement behaviour of Hotel tower

strain. The program DEFPIG was therefore used to compute the interaction factors for a series of alternative (but credible) assumptions regarding the distribution of stiffness both radially and with depth. The ratio of the soil modulus between the piles to that near the piles was increased to 5, while the modulus of the material below the pile tips was increased from the original 70 MPa to 600 MPa (the value assessed for the rock at depth). The various cases are summarized in Table 7.

Figure 16 shows the computed relationships between interaction factor and spacing for a variety of parameter assumptions. It can be seen that the original interaction curve used for the predictions lies considerably above those for what are considered (in retrospect) more realistic assumptions. Since the foundations analyzed contained many piles, the potential for over-prediction of settlements is considerable, since small inaccuracies in the interaction factors can translate to large errors in the predicted group settlement. In addition, Al-Douri and Poulos (1994) indicate that the interaction between piles in calcareous deposits may be much lower than those for a laterally and vertically homogeneous soil. Unfortunately, this experience was not incorporated in the Class A pile group settlement predictions for the towers.

Revised settlement calculations, on the basis of these interaction factors, gave the results shown in Figure 17. The interaction factors used clearly have a great influence on the predicted foundation settlements, although they have almost no effect on the load sharing between the raft and the piles. For Case 4, the maximum settlement is reduced to 29% of the value originally predicted, while the minimum settlement was only 25% of the original value. If this case were used for the calculation

Curve No.	Modulus of Layer below MPa	Modulus of Soil between Piles to Near-Pile Values
1	80	1.0
2	80	5.0
3	200	5.0
4	600	5.0
5	600	1.0

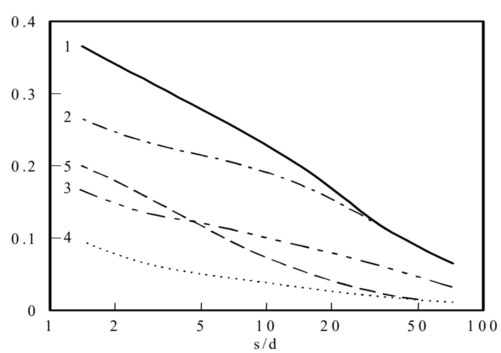

Figure16. Sensitivity of computed interaction factors to analysis assumptions

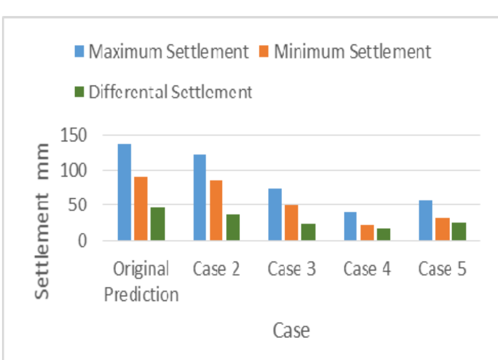

Figure 17. Effect of ground profile assumption on computed settlements

Table 7. Summary of Revised Calculations for Hotel Tower

Case	Modulus below 53 m (MPa)	Ratio of max. to near-pile modulus
Case 1: (Ori. cal.)	80	1
Case 2	80	5
Case 3	200	5
Case 4	600	5
Case 5	600	1

of the settlements during construction, the settlement at Point T15 after 10.5 months would be about 12 mm, which is in reasonable agreement with the measured value of about 10 mm.

This project demonstrated the vital importance of proper characterization of the ground, not only along the piles, but also beneath the piles. Especially for foundation systems (as is typical of tall buildings), the assumptions made about ground stiffness at depth can have a profound effect on the computed settlements. In addition, if use is made of a method of analysis which involves interaction factors, such assumptions will also influence the computed values of interaction factor.

8.4.2 Effect of Ignoring Compressible Underlying Layers

Golder and Osler (1968) have described an interesting case of a series of furnace foundations on piles, which were founded at a relatively high level, well above a deep layer of compressible Leda Clay. Figure 18 shows the stratigraphy of the site and some of the key engineering properties revealed by the investigations. The configuration of the pile group is also shown in this figure. A number of the original boreholes extended to depths up to 236 feet (72 m) without encountering bedrock.

A load test was carried out on a pile similar to that used for the furnace foundations. At a typical working load of 75 US tons (668 kN), the measured settlement was about 0.04 inches (1.0 mm). Applying normal pile group settlement theory to this result, it might have been expected that the settlement of a 32-pile furnace group would have been of the order of 3 to 6 mm. 15 years of settlement records were available for a bank of five furnaces, and these measurements enabled some interesting conclusions to be drawn regarding the sources of settlement of the foundation. Figure 19 reproduces the measured settlements over the bank of five furnaces, and reveals the following interesting characteristics:

- The maximum settlement nearly 15 years after construction was about 73 mm and was continuing to increase;
- The measured settlements were an order of magnitude greater than those which may have been expected simply on the basis of the pile load test;
- The settlement of the end furnaces (Furnace 1 and Furnace 5) was clearly affected by the other furnaces, and showed a significant tilt.

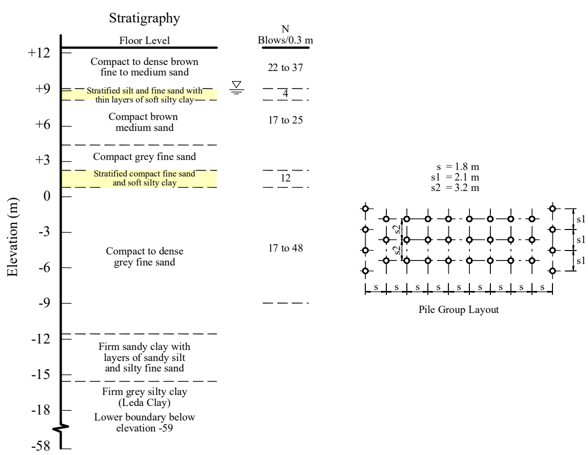

Figure 18. Stratigraphy and pile group layout for furnace foundation (Golder and Osler, 1968).

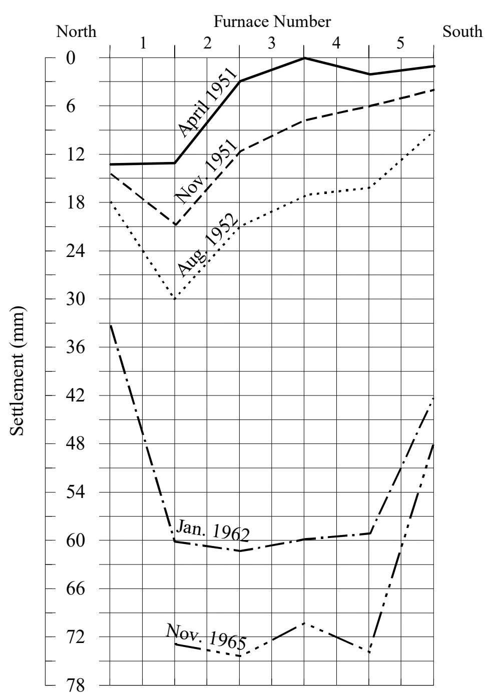

Figure 19. Settlement observations along north-south line through furnaces 1 to 5 (Golder and Osler, 1968).

It was estimated by Golder and Osler that, taking into account the settlement of the compressible layers below the pile tips, the anticipated final settlement of the end furnace (No.1) could be of the order of 87 mm, consisting of 10 mm of pile group settlement, 13 mm consolidation of the silty clay layer below the pile tips, and 64 mm from the deep Leda clay.

This case clearly demonstrates the importance of taking account of the compressibility of underlying compressible layers below the pile tips, and of also considering the interaction among adjacent foundations. It also highlights the potential dangers of relying solely on the results of a single pile load test to predict pile group behaviour, without a proper appreciation of the ground conditions.

8.5 Ignoring the Beneficial Effects of Basement Walls

Many structures, especially tall buildings, incorporate a basement into the substructure to provide parking and storage facilities. In the design of foundation systems, the effect of the basement is often ignored when assessing the foundation capacity and stiffness, even though the basement may extend to a considerable depth below the surface.

Chow and Poulos (2019) have explored the effects of a basement on the capacity and stiffness of a piled or piled raft system, using the three-dimensional finite element program PLAXIS 3D. A numerical example was presented to illustrate the effects of a basement wall on the capacity and stiffness of the foundation system. The wall was assumed to be rectangular in shape, of plan dimensions B_r x L_r, and embedded to a depth of D_r below the ground surface, as shown in Figure 20. The direction of lateral loading was parallel to the dimension L_r.

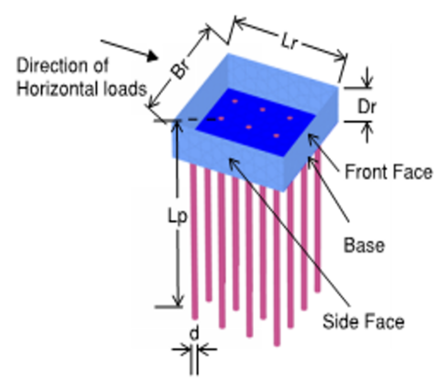

Figure 20. Geometry of basement wall and foundation system

A square raft of 16 m x 16 m in dimension was assumed to be supported by a 4 x 4 pile group with a centre-to-centre spacing of 4 m, embedded in a deep uniform stiff clay profile. The piles had a diameter of 1 m and a length of 24 m. The piled raft was assumed to be rigidly connected to the basement walls.

The foundation system was subjected to rapidly applied loading, such that the soil would have an undrained behavior with an undrained shear strength of 80 kPa. A constant shear resistance of 50 kPa was assumed along basement walls and the beneath the underside of the raft. The ultimate skin friction (f_s) on the piles was assumed to be 56 kPa based on the α-method, $f_s = \alpha s_u$. The ultimate bearing capacity (f_b) and lateral yield pressure were assumed to be $9s_u$ = 720 kPa. The parameters used for the analysis are summarized in Table 8.

Figure 21 illustrates the foundation layout in plan and section of foundation. The finite element mesh used for the analysis employed a total of 19836 elements and 29568 nodes. The soil was modeled as a homogenous continuum obeying the Mohr Coulomb criterion. The piles were modeled by embedded beam elements with interface elements, while the raft and basement walls were modeled by plate elements. In order to simulate the slip along the raft base and basement walls, a thin layer of 0.1 m thick with the strength as specified (base shear of raft and shear resistance along basement wall) was introduced underneath the raft and adjacent to the basement walls.

Table 8: Parameters for problem considered

Parameter		Value
Young's Modulus of Clay (MPa)	E_v (vertical)	50
	E_h (horizontal)	35
Undrained Shear Strength of Clay, s_u (kPa)		80
Ultimate Skin Friction (kPa)		56
Ultimate End Bearing (kPa)		720
Young's Modulus of Pile (MPa)		30,000
Young's Modulus of Raft (MPa)		30,000
Thickness of Raft (m)		1.2
Base Shear along raft (kPa)		50
Young's Modulus of Basement Wall (kPa)		30,000
Thickness of basement wall (m)		0.5
Shear Resistance along Basement Wall		50

The depth of the basement wall depth varied from 0 to 10 m and the foundation system was subjected to three load cases:

- uniform vertical loading,
- uniform horizontal loading, and bending moment applied at the centre of the foundation.

Figure 22 presents the percentage increase in vertical and lateral capacity computed by Chow and Poulos (2019), as compared with those of the piled raft with no wall. The results indicate that with wall embedment, both the vertical and lateral capacity of the foundation system increase. The horizontal capacity increases dramatically, while the vertical capacity increases relatively modestly.

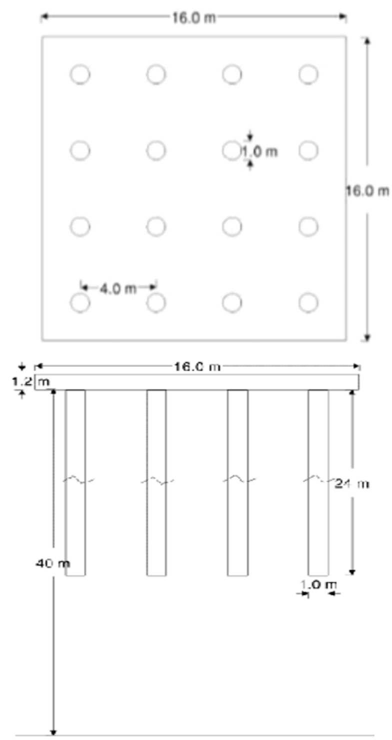

Figure 21. Details of foundation system analyzed

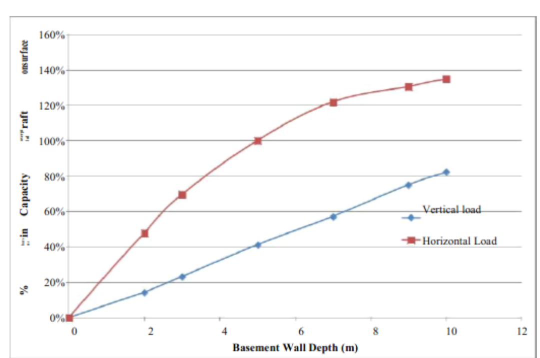

Figure 22. Percentage increase in capacity due to the piled raft and basement wall embedment (Chow and Poulos, 2019)

The author has also attempted to undertake simplified analyses by representing the basement walls as a series of equivalent piles having the same axial stiffness and bending stiffness as the wall. The piles had a diameter equal to the thickness of the wall and a length equal to that of the wall. The parameters of the equivalent piles were such that they provided an equal axial and lateral capacity as that of the walls.

For axial loading, the program GARP was employed (Small and Poulos, 2007), while for lateral and moment loading, the program CLAP (a development of the DEFPIG program (Poulos, 1990)) was used. In the latter cases, the pile cap is assumed to be rigid.

Figure 23 shows, for typical working load levels, the percentage reduction in responses, i.e maximum settlement under vertical load, lateral deflection under lateral load, and rotation under moment loading. It can be seen that in In all three cases, there is a reduction in response with increasing wall depth, with the lateral deflection and rotation experiencing the largest reductions. For the lateral loading, the rate of reduction of lateral deflection decreases once the wall depth exceeds about 6 m. This tends to reflect the fact that the wall has an effective length which, when exceeded, results in little or no additional reduction in deflection.

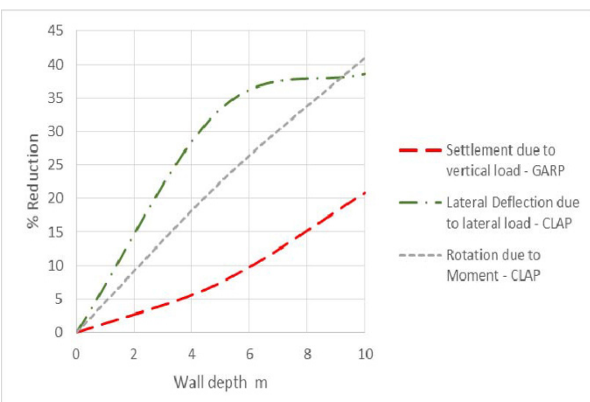

Figure 23. Reduction in responses due to basement wall

For axial loading, the reduction in maximum settlement is accompanied by an increase in the differential settlement, since the walls act to "hold up" the outer edges of the raft. However, for the case analyzed, the increase in differential settlement is relatively modest, being only about 12% for a 5 m deep wall, and about 20% for a 10 m deep wall.

At the same time, there is also a modest re-distribution of axial loads, as shown in Figure 24. The loads in the corner and mid-side piles tend to reduce, while the load on the centre piles increases. The piles nearest the corner of the raft are most affected, with a decrease in axial load of about 15% being experienced for the 10 m deep wall.

The presence of the walls has a "shielding" effect on the foundation piles and thus tends to reduce the bending moments developed within these piles. Table 9 shows the computed percentage reduction in pile head bending moments

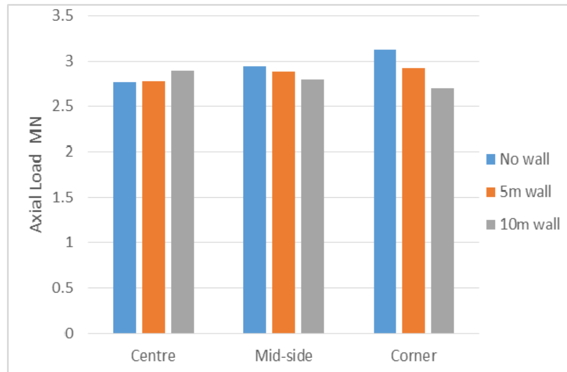

Figure 24. Effect of basement walls on axial load distribution

due to the wall, for both horizontal and moment loading. This reduction is most significant for walls up to about 5-6 m deep, and the effect diminishes thereafter. The reductions in pile head moment suggest that the requirements for structural design of the piles are reduced because of the presence of the walls.

Table 9. Percentage reduction in average pile head bending moment

Wall depth m	% reduction in head moment due to horizontal loading	% reduction in head moment due to moment loading
0	0	0
5	44	30
10	45	37

The implications of the above analysis results are as follows:

- For both vertical and horizontal loads, the displacements decrease as the depth of basement walls increases.
- The basement walls provide additional vertical and horizontal resistance to the foundation system, and thus can provide a larger margin of safety against failure (especially horizontal) than a piled raft without embedment.

- The basement walls provide additional rotational stiffness to the foundation system thus contributing to the reduction of its angular rotation.
- The induced bending moments within the foundation piles tend to be reduced significantly (in this example), with most of the benefit arising from relatively shallow walls. There is also a modest re-distribution of axial load within the foundation piles.

It should be recognized that the larger the area of the foundation footprint, the smaller will be the effect of the basement walls surrounding the foundation system. Nevertheless, it seems highly desirable to incorporate the basement walls into the foundation design to avoid undue conservatism in relation to the forces and bending moments in the piles, and on the other hand, to avoid under-estimating differential settlements in the vicinity of the walls.

8.6 Ignoring External Ground Movements

In contemporary urban environments, it is not unusual for excavations of tunnelling works to be carried out in proximity to planned or existing deep foundations. If such works are known or anticipated, then it is possible to make allowances in the design of the deep foundations. If they are carried out after construction of the deep foundations, then it is necessary to assess the impact of such works on the integrity and movement of the existing foundations.

Consideration is given here to the effects of tunnelling adjacent to a deep foundation, as shown in Figure 25. The ground movements due to the tunnel can be estimated via the equations developed by Loganathan and Poulos (1998) and the effect of these movements on piles can be analysed as per the approach described by Chen et al (1999).

For the case shown in Figure 25, a volume loss of 1% is assumed for the 6 m diameter tunnel, and the centreline of the tunnel is assumed to be 6 m from the centreline of the pile, which is 36 m long and 1.2 m in diameter. The pile head is assumed to be fixed into a pile cap so that rotation is restrained. A horizontal load of 1 MN and a vertical load of 5 MN are applied at the pile head.

Pile-soil interaction analyses have been carried out to compute the induced horizontal displacement, bending moment, vertical settlement

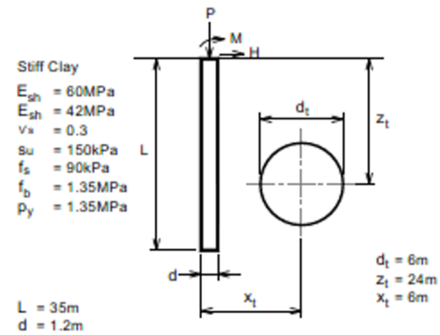

Figure 25. Example of a pile near a new tunnel

and axial load in the pile. These are shown in Figures 26 to 29 in turn.

From these figures, the following observations can be made:

- The maximum horizontal displacement occurs near the level of the tunnel invert, and is significantly greater than the pile head displacement;
- The maximum moment at the pile head is little affected by the tunnel, but there is a significant additional moment developed near the level of the tunnel invert. In many cases, this would be at a lower level than that at which the reinforcement is often terminated;
- The tunnelling causes the pile head settlement to almost double in this case;
- There is a “downdrag” component of axial force near the level of the tunnel invert, although in this case, the maximum axial force is still the applied force at the pile head.

This simple example illustrates that potentially harmful effects can arise from tunnel construction near existing piles. In particular, large bending moments can be developed in the pile near the level of the tunnel invert, which may be below the usual cut-off level for bored pile reinforcement. Consequently, if the possibility of future tunnelling operations is recognized, then full-length reinforcement may need to be provided.

Only a single pile is considered in this example, but group effects can in fact be slightly beneficial and result in a modest reduction of the moments and axial forces induced in the pile.

8.7 Ignoring Kinematic Seismic Effects

Inertial lateral loads are imposed on a foundation

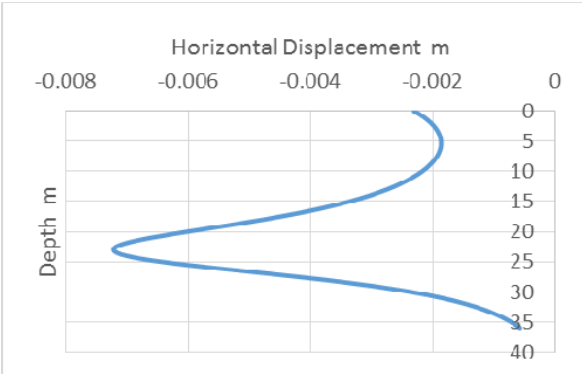

Figure 26. Horizontal displacements in pile due to applied load and tunnel

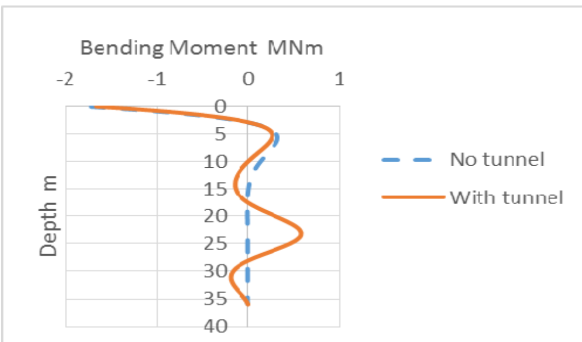

Figure 27. Bending moments in pile due to applied load and tunnel

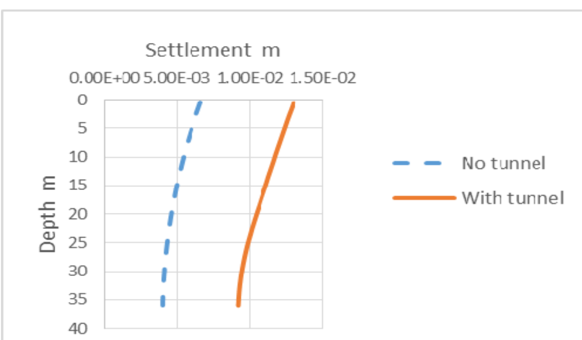

Figure 28. Vertical settlement of pile due to applied load and tunnel

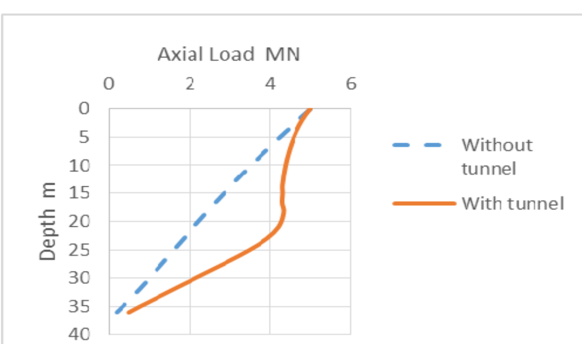

Figure 29. Axial forces in pile due to applied load and tunnel

system when the supported structure responds to seismic loading. The influence of such inertial effects on the seismic response of pile foundations is well-recognized, and depends on the frequency content of the earthquake and the natural period of the pile-soil-cap-mass system. However, an earthquake also imposes kinematic actions on a deep foundation system due to the movement of the ground in response to the seismic excitation, and the consequent pile-soil interaction.

Mylonakis et al. (1997) have identified the following characteristics:

- Inertial bending can be significant, especially in the upper part of the piles, when the dominant period of the earthquake is similar to the fundamental period of the soil-pile-structure system.
- Kinematic bending can be significant when the dominant period of the soil motions are similar to the natural period of the soil strata.

The three most likely areas of damage of a pile are the pile head, interfaces between layers of different stiffness, and the pile toe. Pile head damage is most likely in homogeneous strata while damage at strata interfaces is most likely when there is a marked stiffness contrast between the soil layers. The kinematic bending strains at the pile toe may be significant when the toe is restrained.

To facilitate an understanding of the relative importance of inertial and kinematic effects, analyses have been performed on the fixed head single pile shown in Figure 30. The analysis has been carried out via the pseudo-static approach described by Tabesh and Poulos (2001), so that the results provide an envelope of maximum bending moments and shears along the pile. It is assumed that the site is subjected to the 1994 Northridge earthquake with a maximum bedrock acceleration of 0.2 g. Three cases have been considered:

- A pile subjected to kinematic effects, with no vertical load/cap mass;
- A pile with a lateral inertial load of 0.2 MN and no kinematic effects;
- A pile with the same lateral inertial load as in the second case, but where the kinematic ground movements are included in the analysis.

Figure 31 shows the computed distributions of bending moment along the pile. Two key points emerge from this figure:

- If kinematic effects are ignored, and only inertial (lateral load) effects are considered, the maximum moment at the pile head can be seriously under-estimated.

- If only inertial effects are considered, the moment at depths in excess of about 7 m becomes insignificant, but with the kinematic effects incorporated, there is a significant moment between depths of about 7 to 10 m, i.e. in the vicinity of the interface between the softer upper layer and the stronger lower layer.

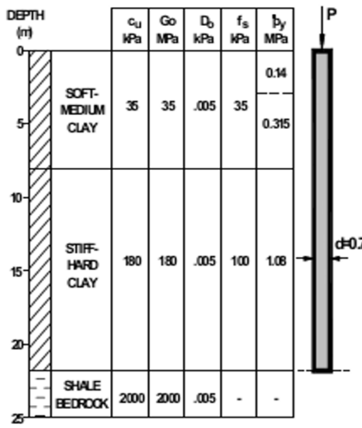

Figure 30. Example for effect of kinematic loading

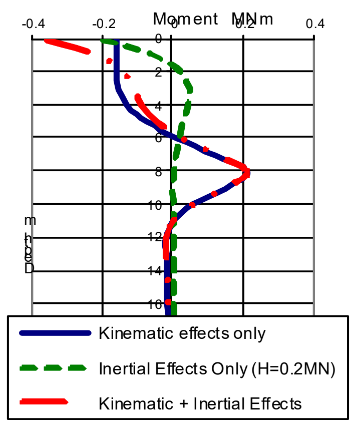

Figure 31. Computed bending moment distributions

The importance of considering both kinematic as well as inertial effects is clearly emphasized in this example. As with the example of ground movements due to tunneling, it is possible that full-length reinforcement may be required for bored piles, especially if significant layering of the soil profile exists and there is a distinct stiffness contrast between adjacent layers.

8.8 Assumption of Elastic Behaviour of Piles

In designing piles for lateral loading, it is common to assume that the piles themselves remain elastic during the entire loading process. While this may be a reasonable assumption for steel piles, it may be an over-simplification for concrete piles. A typical moment-curvature relationship for a bored pile is shown in Figure 32, and it can be seen that there is significant non-linearity in the relationship.

The effect of such pile non-linearity has been examined in a number of papers, for example, Ashour et al (2001), Kramer and Heavey (1988), Hsueh et al (2004). Kramer and Heavey have shown that even a simple elastic-plastic relationship for the pile can provide results that are in reasonable agreement with measurements of lateral pile behaviour. Figures 33 and 34 show an example of the improved agreement with observed behaviour when a simple non-linear elastic-plastic model is used for the pile.

A further example of the effects of using a non-linear pile model are shown in Figure 35 for a 0.76 m diameter bored pile in stiff clay (Ashour et al, 2001). Here, the load-deflection curve for a non-linear pile model is shown together with solutions for various values of the bending stiffness (*EI*) of an elastic pile. A would be expected, at low loads, the solution for the non-linear pile is the same as that for a linear pile with the same initial bending stiffness. However, as the load increases, there is a gradual transition from the initial curve for a linear pile across curves for decreasing values of EI.

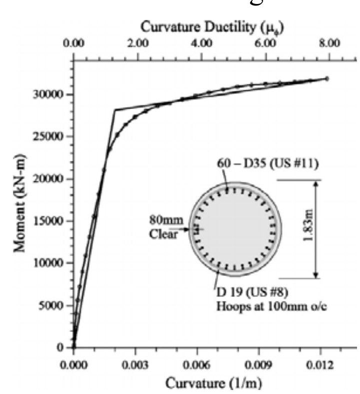

Figure 32. Typical moment – curvature relationship for bored pile

As a practical approximation, it may be feasible to use such a series of load-deflection curves to construct a non-linear curve, by starting the transition once the initial cracking moment of the pile section is reached, and then gradually moving across the curves for decreasing bending stiffness. From Figure 36, it may be expected that, for relatively low levels of lateral load, the use of a concrete modulus perhaps 1/3 to 1/2 of the small-strain value might provide an adequate result, but in any case, a measure of judgement is required to adopt an equivalent modulus to represent the non-linearity of the pile behaviour in bending.

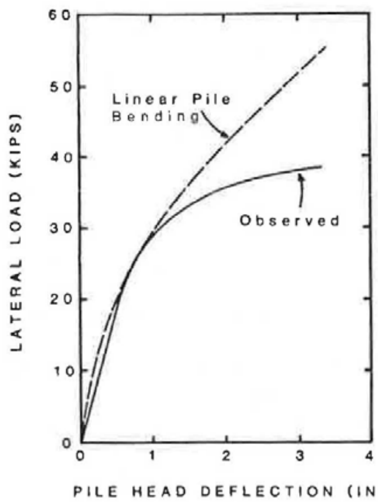

Figure 33. Comaprison between linear model and observed behaviour (Kramer and Heavey, 1988)

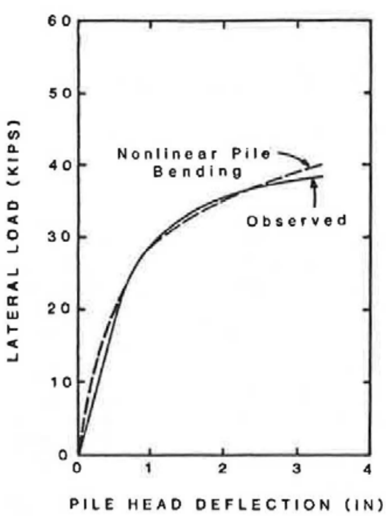

Figure 34. Comaprison between non-linear model and observed behaviour (Kramer and Heavey, 1988)

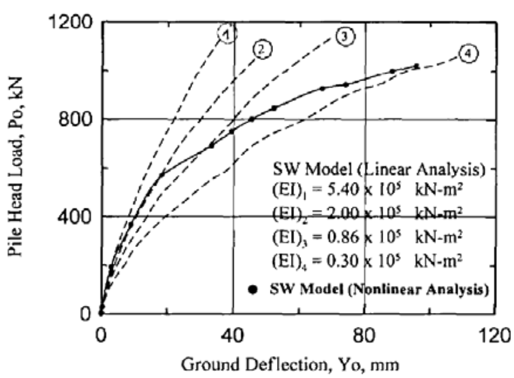

Figure 35. Load-deflection curves for non-linear pile model and for various *EI* values for a linear model (Ashour et al., 2001)

9. CONCLUSIONS

A three-phase process for the design of deep foundations has been outlined. The design issues that need to be considered have been discussed and then some of the available design tools have been compared. It has been found that similar results can be obtained from all four programs considered, so that the key to successful design may be related more to the assessment of geotechnical design parameters than to the specific design software adopted.

A number of aspects of design which are seen to be inadequate, or else in need of improvement, are examined, and of these factors may lead to inaccurate predictions of deep foundation behaviour.

The following factors are found to have the potential to lead to unconservative designs:

- Ignoring foundation interactions;
- Ignoring the effects of ground movements;
- Ignoring the kinematic effects of ground movements in seismic design;
- Ignoring the non-linearity of the pile material.

Conversely, the following factors may lead to conservative, and hence unnecessarily expensive, design outcomes:

- Ignoring the beneficial effect of the raft;
- Assuming a rigid cap or raft;
- Over-simplification of the ground profile;
- Ignoring the beneficial effects of basement walls.

With the design tools that are now available, all of the above perceived inadequacies can be addressed satisfactorily. The main challenges that remain in relation to foundation design are the recognition and modelling of the factors that can influence deep foundation behaviour, and the ever-present challenge of appropriate assessment of the relevant geotechnical parameters.

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