FOUNDATION SETTLEMENT ANALYSIS – PRACTICE VERSUS RESEARCH

The Eighth Spencer J. Buchanan Lecture

by

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Friday November 3, 2000

College Station Hilton
810 University Drive East
College Station, TX 77840

8th Spencer J. Buchanan Lecture, November 2000

Foundation Settlement Analysis – Practice Versus Research

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Abstract

This paper examines and evaluates some common methods of foundation settlement prediction in the light of recent research. Four common problems are considered: settlement of shallow foundations on clay, settlement of shallow foundations on sand, analysis of strip and raft foundations, and the settlement of pile groups. The outcome of the evaluation is a recommendation on whether a method should be adopted, adapted, or discarded. The crucial importance of appropriate assessment of the relevant geotechnical parameters is emphasized.

1. INTRODUCTION

Despite the significant volume of geotechnical research, particularly over the past three decades, there has been all too little effort made to try and evaluate the applicability of some of the commonly-used foundation design and analysis methods, in the light of the research findings. It would appear that the state of practice in traditional areas of foundation engineering generally lags far behind the state of the art. Cost and time pressures may preclude the use of modern techniques of analysis and design of foundations, and often result in the continued use of dated empirical procedures whose validity may be dubious.

The evaluation of commonly – used design procedures received a major boost in the 1970's with the Prediction Symposia organized by Lambe and his colleagues (1970,1973,1974). Such Symposia attempted to reveal the ability of geotechnical engineers to carry out accurate "Class A" predictions for a variety of practical circumstances i.e. true predictions made prior to the performance details being known. In general, this ability was, at best, variable, and at worst, depressingly inadequate. Subsequent Symposia (e.g. Briaud et al., 1994; Finno et al., 1989; Brand, 1990) have reinforced the findings of Lambe, and demonstrated that accurate prediction of the performance of piles and embankments is dependent as much on the experience of the predictor, and a good amount of luck, as on the adequacy of the method employed. The selection of geotechnical parameters also plays a major part in the success or otherwise of a prediction, and may outweigh or mask any shortcomings of the method used.

The main objective of this paper is to examine and evaluate some foundation settlement prediction procedures in the light of relatively recent research. Ideally, such an evaluation should consider both the theoretical "correctness" of the methods and also their applicability to practical cases. However, primary attention will be paid here to identifying the shortcomings and limitations of the methods when compared to modern theoretical approaches. Four common problems will be considered in this paper:

- Settlement of shallow foundations on clay
- Settlement of shallow foundations on sand
- Analysis of strip and raft foundations
- Settlement of pile groups

In each case, an attempt will be made to suggest a fate for the methods considered, in one of the following three categories: adopt, adapt, or discard.

2. ANALYSIS AND DESIGN METHODS

2.1 Desirable Attributes of Practical Analysis and Design Methods

Among the desirable attributes of practical methods of geotechnical analysis and design are the following:

- they should have a sound theoretical basis
- they should capture the major features of the problem being addressed and incorporate the important parameters
- they should be able to be applied to practice without requiring excessive computational resources
- the geotechnical parameters required in the methods should be able to be estimated by conventional field and laboratory tests
- they should be able to be checked with a simpler approach.

In this context, it is worth bearing in mind the following opinion of Burland (1989):
"Any design that relies for its success on precise analysis is a *bad* design".

2.2 Categories of Analysis and Design Methods

In assessing the relative merits of analysis and design methods, it is useful to categorize the methods in some way. It has been proposed previously (Poulos, 1989) that methods of analysis and design can be classified into three broad categories, as shown in Table 1.

Category 1 procedures probably account for a large proportion of the foundation design performed throughout the world. Category 2 procedures have a proper theoretical basis, but they generally involve significant simplifications, especially with respect to soil behaviour. The majority of available design charts fall into one or other of the Category 2 methods. Category 3 procedures generally involve the use of a site-specific analysis based on relatively advanced numerical or analytical techniques, and require the use of a computer. Many of the Category 2 design charts have been developed from Category 3 analyses, and then condensed into a simplified form. The most advanced Category 3 methods (3C) have been used relatively sparsely, but increasing research effort is being made to develop such methods, in conjunction with the development of more sophisticated models of soil behaviour.

From a practical viewpoint, Category 1 and 2 methods are the most commonly used. In the following sections, attention will be focussed on evaluating such methods with respect to more refined and encompassing methods, many of which either fall into Category 3, or have been derived from Category 3 analyses.

3. SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS ON CLAY

Estimation of settlement and differential settlement is a fundamental aspect of the design of shallow foundations. For foundations on clay, Table 2 summarizes some of the available techniques and their capabilities. The traditional approach, first developed by Terzaghi, employs the one-dimensional method in which the settlement is assumed to arise from consolidation due to increases in effective stress caused by the dissipation of excess pore pressures. Because of its still widespread use, it is of interest to examine the capabilities and shortcomings of the method, when compared with more complete two- and three-dimensional methods.

The one-dimensional method has the following limitations:

1. It assumes that the foundation loading causes only vertical strains in the subsoil
2. It assumes that all the settlement arises from consolidation, and that settlements arising from immediate shear strains are negligible
3. It assumes that the dissipation of excess pore pressures occurs only in the vertical direction; any lateral dissipation of excess pore pressures is ignored.

3.1 One-Dimensional versus Three-Dimensional Settlement Analysis

To examine the possible significance of these limitations, two very simple hypothetical examples are considered. The first involves a uniformly loaded circular footing resting on the surface of a homogeneous layer of overconsolidated clay, in which the soil stiffness is uniform with depth. The second involves the same footing on a layer of soft normally consolidated clay in which the soil stiffness increases linearly with depth, from a small initial value at the soil surface. The relationship between the one-dimensional compressibility m_v and the drained Young's modulus E'(for the three-dimensional analysis) is assumed to be that given by elastic theory for an ideal two-phase elastic soil skeleton, as is the relationship between the undrained Young's modulus E_u and E', i.e.

$$m_v = \frac{(1+\nu')(1-2\nu')}{(1-\nu')E'} \quad (1)$$

$$E_u = \frac{3E'}{2\ (1+\nu')} \quad (2)$$

where ν^1 = drained Poisson's ratio of soil skeleton.

Figure 1 shows the ratio of the one – dimensional settlement (excluding creep) to the correct three-dimensional total final settlement (Davis and Poulos, 1968). For the overconsolidated clay layer, the one-dimensional analysis gives a good approximation to the correct total settlement when the drained Poisson's ratio of the soil layer is less than about 0.35, even for relatively deep soil layers. The one-dimensional analysis tends to under-predict the settlement as the drained Poisson's ratio of the soil increases or the relative layer depth increases. For the soft clay layer, the one-dimensional analysis again gives a remarkably good approximation to the total final settlement if the drained Poisson's ratio of the soil layer is 0.35 or less.

Burland et al. (1977) provide a detailed discussion of the ratio of one-dimensional settlement to total settlement, and demonstrate that soil anisotropy can have some influence on this ratio. They also argue that, while S_{oed} is a good approximation to S_{TF} for stiff clays, it is more likely to approximate the final consolidation settlement S_{CF} for normally consolidated clays, because of the yielding of such a soil and the consequent irrecoverable strains.

Figure 2 shows the relative importance of immediate settlement as a proportion of the total final settlement. For the overconsolidated clay layer, immediate settlement can be a significant proportion of the total settlement and hence a one-dimensional analysis may give a misleading prediction of settlement, i.e. it suggests that all the settlement arises from consolidation, whereas a significant amount may be immediate settlement. On the other hand, for the soft clay layer, the ratio of immediate to total final settlement is considerably smaller, and one-dimensional theory may provide an adequate prediction of the final settlement. In the latter case, it is also often the case that the undrained Young's modulus is significantly greater than the theoretical value which would be implied by the relationship for an ideal elastic two-phase soil. Thus, the immediate settlement may, in reality, be an even smaller proportion of the total final settlement than is indicated in Figure 2.

3.2 Effects of Local Yield

The commonly-used methods of settlement analysis implicitly assume elastic behaviour of the soil over the range of stress applied by the foundation. While some allowance is made for nonlinear soil response by distinguishing between normally consolidated and over-consolidated states, and using different values of compressibility for each, no allowance is made for the development of local yield within the soil due to foundation loading. Davis and Poulos (1968) considered the conditions affecting the onset of local yield and showed that the applied loading at which local yield commenced was a function of both the factor of safety (i.e. the ratio of applied pressure q_u to ultimate bearing capacity q_u) and also the initial stress state. D'Appolonia et al. (1971) extended these concepts and developed correction factors for the effects of local yield on immediate settlement. In their approach, the immediate settlement S_I was given as:

$$S_i = S_{ielas} / F_R \tag{3}$$

where

S_{ielas} = immediate settlement computed from elastic theory

F_R = yield correction factor, a function of q/q_u and the initial stress ratio f, which is a function both of coefficient of earth pressure at rest and undrained shear strength. F_R is 1 for elastic conditions, and less than 1 when local yield occurs in the soil.

The effects of local yield on consolidation settlement and ratio of settlement do not appear to have been studied extensively. However, solutions presented by Small et al. and Carter et al. (1979) suggest that both the consolidation settlement and rate of settlement and the rate of settlement are not greatly affected by local yield within the soil, and that elastic theory can be used with sufficient accuracy for their estimation.

Thus, the main influence of local yield is on the immediate settlement S_i, and for category 2 methods of calculation, equation (3) can be used to estimate S_i.

In summary, it would appear that one-dimensional settlement theory can be adopted, but with some measure of adaptation. In particular, there is clear evidence that immediate settlements are important and cannot be ignored, especially for stiff clays. The effects of local yield on immediate settlement should also be considered. Based on the theoretical relationships discussed above, it would appear reasonable to adopt the following procedures if employing one-dimensional settlement theory:

(1) for stiff overconsolidated clays:

$$S_{TF} = S_{oed} \tag{4a}$$

$$S_{CF} = S_{oed} - S_i \tag{4b}$$

(2) for normally consolidated clays:

$$S_{TF} = S_i + S_{oed} \tag{5}$$

The predicted settlement for the second case may be conservative, but since S_i is often relatively small in comparison to S_{oed}, the potential extent of over-estimation is unlikely to be significant in most cases.

Table 1. Categories of Analysis and Design Methods

Category	Subdivision	Characteristics	Method of Parameter Estimation
1	--	Empirical - not based on soil mechanics principles	Simple in-situ or laboratory tests, with correlations
2	2A	Based on simplified theory or charts - uses soil mechanics principles - amenable to hand calculation; simple linear elastic or rigid plastic soil models	Routine relevant in-situ or laboratory tests - may require some correlations
	2B	As for 2A, but theory is non-linear (deformation) or elasto-plastic (stability)	
3	3A	Based on theory using site-specific analysis; uses soil mechanics principles, Theory is linear elastic (deformation) or rigid plastic (stability	Careful laboratory and/or in-situ tests which follow the appropriate stress paths
	3B	As for 3A, but non-linearity is allowed for in a relatively simple manner	
	3C	As for 3A, but non-linearity is allowed for via proper constitutive soil models	

Table 2. Some Methods of Settlement Analysis for Shallow Foundations

Method	Categ -ory	Immediate Settlement	Consolid'n Settlement	Settlement Rate	Creep Settlement
One dimensional $S_{TF} = S_{CF} = S_{oed}$	2	-	Y	Y	Y (can be incorporated)
Modified One Dimensional $S_{TF} = S_i + S_{oed}$	2	Y	Y	Y	Y (can be incorporated)
Skempton & Bjerrum (1957) $S_{TF} = S_i + \mu S_{oed}$	1-2	Y	Y	Y	-
Elastic Method $S_{TF} = S_i + (S_{TF} - S_i)$	2A	Y	Y	Y	Y (can be incorporated)
Modified Elastic $S_{TF} = S_i / F_R + (S_{TF} - S_i)$	2B	Y	Y	Y	
Elastic Finite Element	3A	Y	Y	Y	
Non-Linear Finite Element	3B, 3C	Y	Y	Y	Y (can be incorporated)

Note: S_{TF} = total final settlement S_{oed}= one-dimensional settlement (from oedometer)
S_i = immediate settlement μ = correction factor (Skempton & Bjerrum, 1957)
S_{CF} = final consolidation settlement
F_R=nonlinear correction factor (D'Appolonia et al.,1971)
Y = Yes

3.3 Case Study

The performance of the various methods of settlement analysis can be gauged by applying them to a real case history in which measurements of settlement are available. The case selected for consideration here is that published by Moore and Spencer (1969). This case involved a 2-storey brick structure erected in 1890 on a thick layer of compressible soil in South Melbourne. Figure 3 shows a plan of the building and a simplified representation of the soil profile. The average loading acting on the building was about 45kPa. Oedometer and laboratory stress path triaxial test data were obtained on the dominant clay and silt layer, and these data are also shown in Figure 3.

Moore and Spencer employed a number of methods of settlement calculation, ranging from one-dimensional settlement analysis to two different types of stress path analysis. Three types of oedometer test were carried out for use in the conventional oedometer analysis. The results of the calculations reported by Moore and Spencer are shown in Table 3, together with the measured settlement.

The effects of possible local yield have not been taken into account, and hence the immediate settlements are likely to have been under-estimated. It can be seen that the conventional one-dimensional analysis underestimates the settlement significantly, (depending on which type of oedometer data has been employed), as does also the Skempton and Bjerrum method. The elastic method of David and Poulos (1968) underestimates the settlement by only about 10%, while the Lambe stress path method gives a 23% underestimate.

While this case study can in no way be used to imply that a three-dimensional settlement analysis is always necessary, it does indicate that caution should be exercised in applying one-dimensional settlement analysis to foundations on soft clay. On the other hand, the earlier studies of Skempton, Peck and MacDonald (1955) and Skempton and Bjerrum (1957) provide evidence that one-dimensional analyses provide an adequate estimate of foundations on relatively stiff clays. Thus, the one-dimensional method deserves to be retained, but adapted, as a means of estimating the settlement of shallow foundations on clay.

3.4 Rate of Settlement

It is well known that three-dimensional effects may significantly accelerate the rate of settlement of foundations on clay, primarily because of the ability of excess pore pressures to dissipate horizontally as well as vertically. A number of approaches have been developed for estimating the rate of settlement under two and three-dimensional conditions, including the following:

1. Category 2 design charts for strip and circular foundations on a homogeneous layer e.g. Davis and Poulos (1972), Booker (1974); the former are derived from solutions of the simplified diffusion theory of consolidation, while the latter are based on the complete Biot theory.
2. Finite layer numerical solutions for strip, circular and rectangular footings on elastic clay layers e.g. as implemented by the computer program CONTAL (Small, 1998). This would be classified as a Category 3A approach.
3. Finite element numerical analyses for linear and non-linear soil layers e.g. Small et al. (1976); Sandhu and Wilson (1969).

Table 3.
Comparison of Total Settlements:
Boyd Domestic College Building
(After Moore and Spencer, 1969)

Method	Settlement mm
Observed	787
Conventional Oedometer	508
Oedometer with Back Pressure	404
Oedometer with Constant Rate of Strain	404
Skempton and Bjerrum (1975)	564
Davis and Poulos (1968)	709
Stress Path, Lambe (1964)	610

Note: Oedometer methods used stress distributions for 2-layer soil mass.

From a practical viewpoint, it may not always be feasible to employ a full two-or three-dimensional consolidation analysis. However, it is possible to ***adapt*** a one-dimensional consolidation analysis to take account of this effect by using an equivalent coefficient of consolidation, c_{ve}, which is obtained by multiplying the actual coefficient of consolidation c_v by a geometrical rate factor R_f:

$$c_{ve} = R_f \cdot c_v \tag{6}$$

R_f values can be derived from three-dimensional rate of settlement solutions, such as those derived by Davis and Poulos (1972). Figures 4 and 5 show values of R_f as a function of the layer depth to footing size ratio, for circular and strip foundations. In each case, three combinations of hydraulic boundary conditions at the top and base of the layer are considered: PTPB (permeable top, permeable base), PTIB (permeable top, impermeable base) and IFPB (impermeable top, permeable base). As the layer thickness increases relative to the footing size, the factor R_f increases, reflecting the acceleration of the rate of settlement due to lateral dissipation. It should be noted that for the case IFIB (impermeable footing, impermeable base), it is not possible to adapt a one-dimensional solution since the theoretical rate of settlement is always zero.

If the soil is anisotropic, then a further factor can be applied to the coefficient of consolidation, as presented by Davis and Poulos (1972).

An example of a comparison between solutions for the rate of settlement of a large flexible circular foundation (such as an oil tank) on a layered clay soil profile is shown in Figure 6. Three solutions are shown: a three-dimensional solution from a finite layer consolidation analysis using the program CONTAL (Small, 1998), a modified one-dimensional analysis in which the coefficients of consolidation have been multiplied by an R_f factor of 6 (see Figure 4 for the PTIB case), and a one-dimensional numerical analysis in which the original values of coefficient of consolidation are used. The modified one-dimensional solution is in reasonable agreement with the three-dimensional solution (although it suggests a somewhat more rapid rate of consolidation at larger times). The conventional one-dimensional numerical analysis significantly under-predicts the rate of settlement.

The author has used the above approach in several practical cases with good results, for example, for an embankment on clay (Poulos and Davis, 1975). A comparison between the measured rate of settlement and that calculated from a one-dimensional analysis using the equivalent coefficient of consolidation, shows quite reasonable agreement, certainly better than would have been obtained using a normal one-dimensional consolidation analysis. Thus, it would appear feasible in practical cases to adapt the one-dimensional consolidation analysis, as suggested above, if a proper three-dimensional analysis is either unwarranted or not available.

3.5 Creep and Secondary Consolidation

The existence of creep complicates the prediction of both the magnitude and rate of settlement of foundations on clay soils. Most practical methods of accounting for creep still rely on the early observations of Buisman (1936) that creep is characterized by a linear relationship between settlement and the logarithm of time. The gradient of this relationship is generally represented by the coefficient of secondary compression C_α, where:

$$C_\alpha = \Delta e / \Delta \log t \qquad (7)$$

and Δe = change in void ratio
t = time.

Mesri and Godlewski (1977) have found that C_α is related to the compression index of a soil, as indicated in Table 4. It should be noted that, for overconsolidated clays, the ratios in Table 4 apply to the recompression index; thus, the creep settlement rate is significantly smaller for an overconsolidated soil than for the same soil in a normally consolidated state.

Table 4.
Values of C_α/C_c for Geotechnical Materials
(Mesri et al., 1994)

Material	C_α/C_c
Granular soils, including	0.02 ± 0.01
rockpile	0.03 ± 0.01
Shale and mudstone	0.04 ± 0.01
Inorganic clays and silts	0.05 ± 0.01
Organic clays and silts	0.06 ± 0.01
Peat and muskeg	

The difficulty with applying the 'C_α' concept is that the time at which creep is assumed to commence is not well-defined. Considerable controversy exists on this point, with some researchers assuming that creep only commences at the end of consolidation (e.g. Mesri et al.,1994) while others contend that it takes place simultaneously with primary consolidation (e.g. Leroueil, 1996).

While various creep laws can and have been incorporated into consolidation analyses (eg. Gibson and Lo, 1961; Garlanger, 1972), it is very uncommon in practice for such analyses to be applied, even for one-dimensional problems.

From a practical viewpoint, the most convenient approach appears to be to add the creep settlement relationship to the conventional time-settlement relationship from consolidation theory, commencing at one of the following times:

1) a predetermined time after commencement of loading
2) after a predetermined degree of consolidation settlement
3) when the gradients of the primary settlement versus log time and the creep settlement versus log time relationships are equal.

While all three alternatives are arbitrary, the last of the three appears to be the easiest and most logical to apply.

Overall, it appears that, of all the aspects of settlement analysis, the issue of creep and secondary consolidation is the one in which least progress has been made in terms of fundamental understanding and in the incorporation of research into practice. In the absence of a more satisfactory approach, the method of Buisman may be adapted to provide a crude estimation of creep settlements.

4. SETTLEMENT OF SHALLOW FOUNDATIONS ON SAND

4.1 Previous Studies

A remarkable number of methods have been developed to estimate the settlement of shallow foundations on sand, yet consistent success in accurately predicting such settlements has remained largely elusive. These methods range from purely empirical (Category 1) methods developed originally for conservative footing design (Terzaghi and Peck, 1967), to complex Category 3 nonlinear finite element methods.

Many of the methods rely on in-situ SPT or CPT data, and hence it is not possible to satisfactorily examine the theoretical relationship between the various methods. More recently, Briaud et al (2000) have developed a method based on the use of the pressuremeter test curve to develop the full load-settlement curve to failure for a footing.

Assessments of the performance of various methods have generally been made on the basis of comparisons with measured settlements. At least two significant studies have been reported, one by Jeyapalan and Boehm (1986), and the other by Tan and Duncan (1991).

The study by Jeyapalan and Boehm (1986) involved the statistical analysis of 71 case histories for which settlements of footings on sand were reported, and the assessment of the relative accuracy of nine methods of settlement estimation. The methods of Schultze and Sherif (1973) and Schmertmann (1978) appeared to among the more dependable approaches.

Tan and Duncan (1991) carried out an assessment of the reliability of twelve methods of estimating footing settlement on sands by comparing calculated and measured settlements for 76 cases. Each of the methods was evaluated in terms of (1) accuracy (the ratio of average calculated to measured settlement), (2) reliability (the percentage of cases in which the calculated settlement equalled or exceeded the measured settlement, and (3) ease of use (the length of time required to apply the method. Table 5 summarizes the methods considered and the parameters used in each method. Figure 7 summarizes the findings on accuracy and reliability. Values of "accuracy" range from 1.0 (the ideal value) for Alpan's method to 3.2 for Terzaghi and Peck's method. Values of "reliability" varied from 34% for Schultze and Sherif's method to 86% for Terzaghi and Peck's method. In general, the methods which were less accurate (and more conservative) were more reliable in the sense that they underestimated the settlement relatively infrequently. Table 6 summarizes the hand computation times for a simple example. Those methods requiring correction of the SPT values generally involved the longer computation times. As concluded by Tan and Duncan, there is a tradeoff between accuracy and reliability in choosing a method of calculation.

4.2 Case Study

A comparison of the performance of a number of the methods has been made via a well-documented Prediction Symposium in which a number of people made "Class A" predictions of the settlement of footings on a natural sand profile (Briaud and Gibbons, 1994). The predictions were then compared with the actual settlement measurements.

Figure 8 summarizes the soil conditions near one of the footings tested (footing 1, nominally 3m by 3m in plan). The site consisted by layers of silty sand, underlain by a hard clay layer. A substantial amount of in-situ and laboratory data was obtained, some of which is shown in Figure 8.

In the Prediction Symposium, a total of 31 persons made predictions, using a wide variety of methods. However, for the present exercise, the author has made his own application of a number of the methods to Footing 1, as well as presenting the original prediction made for this footing. An exception is the result of a finite element analysis, which was carried out by one of the other predictors.

Table 7 summarizes the results of the author's calculations for the settlement at a load of 4000 kN (corresponding to a factor of safety against ultimate failure of about 2·5). The following observations can be made from Table 7:

1. all methods over-estimated the footing settlement
2. the elastic-based methods, based on CPT, SPT and pressuremeter data, all give reasonable predictions
3. the Terzaghi and Peck method, which is meant to provide a conservative footing design to ensure a settlement less than 25mm, gives a predictably conservative settlement estimate
4. the author's original prediction, based on an elastic analysis with a strain-dependent Young's modulus, overpredicts the settlement significantly
5. the finite element method, using a nonlinear constitutive soil model, overpredicts the settlement substantially.

While it is again imprudent to draw firm conclusions on the basis of such limited comparisons, it does appear reasonable to suggest that the simple elastic-based methods (including Schmertmann's method) appear capable of providing reasonable estimates of footing settlement. The key to success lies more in the appropriate choice of the shear or Young's modulus of the sand than in the details of the method employed. Such methods therefore deserve to be retained and adopted. On the other hand, the more complex finite element methods appear to require far more development before being able to be used with confidence. Indeed, from a practical viewpoint, there may be relatively few cases in which such analyses are warranted.

5. ANALYSIS OF RAFT AND STRIP FOUNDATIONS

5.1 Introduction

The analysis and design of raft and strip foundations usually involves the following assessments:

- bearing capacity under the design loadings
- settlements and differential settlements
- bending moments and shears for the structural design of the foundation.

Attention will be focussed here on the latter two aspects. Ideally, analyses should take account of the stiffness of the raft or strip, together with the stiffness of the structure being supported. Such structure-foundation-soil interaction analyses, while becoming more common with major structures, are still the exception rather than the rule, and most analyses ignore the effects of superstructure stiffness.

5.2 Subgrade Reaction versus Elastic Continuum Soil Models

Table 8 summarizes and categorizes a number of methods commonly used for the analysis and design of raft and strip foundations. All but the simple rigid footing approximation give settlements and differential settlements, as well as moments and shear forces. The majority of these methods consider the stiffness of the raft or strip, and differ primarily in the manner in which the supporting soil is modelled. There are two usual methods of modelling of the soil:

1. by use of the subgrade reaction method, in which the soil is modelled as a series of independent springs (often called the "Winkler spring model" after one of the originators of the concept)
2. by use of elastic continuum theory, in which the soil is modelled as an elastic continuum.

The first approach has long been favoured by many structural and foundation engineers because of its theoretical convenience, and because, prior to the computer age, analytical solutions were available for strip foundations resting on a Winkler soil model. However, despite its theoretical convenience, the Winkler soil model has a number of important limitations which are not always appreciated. These include the following:

1. A Winkler soil model only deflects if a pressure is applied to it. Thus unloaded areas in a Winkler soil model do not deflect, and hence there is no stress transmission or interaction within the soil
2. A Winkler soil responds to loading only in the direction of that loading. Thus, vertical loading will produce only vertical displacements, and no horizontal displacements, and vice-versa
3. A Winkler soil is usually characterised by the modulus of subgrade reaction, which has units of force/length^3. The modulus of subgrade reaction is NOT a fundamental soil parameter, but is dependent on the dimensions of the foundation.

A Winkler soil model cannot incorporate properly the effects of soil layering since it does not allow stress transmission. The assessment of the modulus of subgrade reaction for a layered soil profile therefore involves considerable uncertainty which is sometimes resolved by resorting to elastic theory to obtain an equivalent value.

Table 5. Variables Used in Methods of Estimating Settlements of Footings on Sand (Tan and Duncan, 1991)

Method (reference)	N	N_{cor}	q_c	B	D_W	D_f	γ_t	L	T	Soil Type	Str. Hist	Time
	Variables Used											
Alpan (1964)		Y		Y	Y	Y	Y				Y	
Burland and Burbridge (1985)	Y			Y	Y	Y	Y	Y	Y	Y	Y	Y
D'Appolonia & D'Appolonia (1970)	Y			Y	Y	Y			Y			
Duncan & Buchignani (1976)	Y			Y						Y		Y
Meyerhof (1956)	Y			Y								
NAVFAC (1982)	Y			Y	Y							
Parry (1971)	Y			Y	Y				Y			
Peck & Bazaraa (1969)		Y		Y	Y	Y	Y			Y		
Peck, Hanson, Thornburn (1974)		Y		Y	Y	Y	Y					
Schmertmann (1978)			Y	Y	Y	Y	Y					Y
Schultz & Sherif (1973)	Y			Y		Y			Y			
Terzaghi and Peck (1967)	Y			Y	Y					Y		

N = SPT Blow Count
q_c = Cone Penetration Test tip resistance
D_f = depth of footing below ground surface
T = thickness of sand layer below footing
Time = duration of loading
N_{cor} = SPT Blow Count corrected for overburden pressure

B = footing width
γ_t = total unit weight of sand
Soil Type = silty or clean sand
D_W = depth of water table
L = footing length
Stress Hist. = max. previous load

Y = Yes

Table 6. Computation Times for Methods Based on SPT Blow Count
(Tan and Duncan, 1991)

Method	Computation Time (minutes)
Alpan (1964)	29
Burland & Burbridge (1985)	14
D'Appolonia & D'Appolonia (1970)	8
Duncan & Buchignani (1976)	9
Meyerhof (1956)	6
NAVFAC (1982)	8
Parry (1971)	9
Peck & Bazaraa (1969)	25
Peck, Hanson, Thornburn (1974)	25
Schultze & Sherif (1973)	6
Terzaghi & Peck (1967)	11

Table 7. Summary of Calculated & Measured
Settlement of 3m Square Footing

Method	Settlement for P = 4MN	Notes
Terzaghi & Peck (1957)	39	Av. N = 20
Schmertmann (1978)	28	
Burland & Burbridge (1985)	21	Average value (range 10-58 mm)
Elastic Theory, using E_s = 3N MPa	18	Decourt (1989)
Elastic Theory, using PMT data	24	Reload modulus values
Strain-dependent modulus	32	Poulos (1996), Class A prediction
Finite Element Analysis	75	Chang (1994), Class A prediction, using constitutive soil model
Measured	14	After 30 minutes.

Table 8. Method of Analysis of Raft and Strip Foundations

Method	Category	Remarks	Typical References
Rigid footing assumption	1	Does not give settlements	Bowles (1984)
Strip on Winkler Soil	2A	Closed form solutions	Bowles (1984)
Strip on Elastic Soil	2A	Approximate equations for deep layer	Vesic (1961)
Design Charts for Strip on Elastic Soil	2A	Concentrated loadings, deep layer	Brown (1975)
Design Charts for Raft on Elastic Soil	2A	Uniform loadings only, finite layer	Fraser & Wardle (1976); Brown (1969)
Strip on Elastic Soil or Winkler Soil	3A	Computer program GASP	Poulos (1991)
Raft on Winkler Soil	3A	Computer program based on finite elements	Bowles (1984)
Raft on Elastic Soil	3A	Finite elements for raft	Wood (1977)
Raft on Nonlinear Soil	3B	Approx. allowance for local soil yield and raft lift-off; program GARP	Poulos (1994a)

The first two limitations are at variance with our knowledge of real soil behaviour, while the third has led to some significant difficulties, with inadequate designs arising from the use of subgrade reaction moduli which have not been corrected for the footing dimensions.

It is of interest to examine the relationship between solutions for a loaded strip foundation on Winkler and elastic continuum soil models. Brown (1977) has presented comparisons between the computed bending moments for a strip footing subjected to increasing numbers of concentrated loads. The relative stiffness of the strip, K, is defined as follows:

$$K = 16\, EI\, (1-\nu_s^2) / \pi\, E_s\, L^4 \qquad (8)$$

where
- EI = bending stiffness of strip
- E_s = Young's modulus of soil
- ν_s = Poisson's ratio of soil
- L = length of strip.

The Young's modulus and modulus of subgrade reaction values have been chosen such that the settlements of a rigid strip with a single central load are equal.

Figure 9 shows the comparison for a single central load and reveals quite reasonable agreement for a variety of relative stiffness values K of the strip. Figures 10 and 11 show similar comparisons for 3 and 5 loads equally spaced along the strip. The differences between the solutions becomes greater as the number of loads increases, and the general "dishing" effect which the elastic model reveals is not exhibited by the Winkler model, because the latter cannot consider interaction and stress transmission through the soil. In the extreme case of a uniform loading along the entire strip, the Winkler soil model predicts ZERO bending moment at all points in the strip, whereas the elastic model gives significant moments. In general, it may be concluded that the subgrade reaction approach may provide reasonable estimates of bending moment (and shear force) for strips subjected to isolated concentrated loads, but it becomes increasingly unsatisfactory as the loading becomes more distributed in nature.

5.3 The Analysis of a Raft as a Series of Strip Footings

It is common design practice to analyse a raft foundation by dividing it up into a series of strip footings and analysing each strip as an independent foundation subjected to the loadings applied on that strip. A simple example of this procedure is illustrated in Figure 12. While convenient, this procedure has a number of limitations, including:

- the strip method cannot give torsional moments in the raft
- there will generally be an incompatibility between the computed settlements at the junction of the intersecting strips.

Assuming the case shown in Figure 12, and an elastic continuum soil model, Table 9 compares the key performance characteristics computed from the strip analysis and that computed from a proper analysis of the raft as a plate. The strip solutions have been obtained from the computer program GASP (Poulos, 1991) while the raft solutions are from the program GARP (Poulos, 1994).

Two solutions from the strip analysis are shown, one in which the strip sections are assumed to be isolated independent strips, and the other in which the effects of loads on the raft area outside the strip is taken into account (the 'interacting strip' solution). Assuming that the GARP analysis is the 'benchmark' solution, the following observations are made:

a) the analysis using isolated independent strips underestimates both the settlement and bending moments
b) the interacting strip solution gives a good estimation of the maximum settlement, but under-estimates the minimum settlement
c) the interacting strip solution tends to under-estimate the maximum bending moments.

Overall, the performance of the strip analysis is disappointing and of some concern since it tends to err on the unconservative side as far as bending moments and structural design are concerned, although conversely it tends to be conservative when estimating the differential settlement between the columns in the case considered. In general, it would appear that strip analyses used to be viewed with caution, and it may be appropriate for some further research to be carried out in order to develop better procedures of adaptation of the strip method to raft analysis.

Table 9. Comparison of Computed Performance of Raft

Quantity	Calculated Value		
	Raft Analysis elastic cont'm soil	Strips with Extl. Areas	Isolated Strips
Settlement at EC mm centre col.	88.8	88.4	68.2
Settlement at A mm outer col.	75.2	55.0	33.6
M_{XX} at AC MNm/m	2.90	1.83	1.57
M_{yy} at AC MNm/m	2.40	1.08	1.12
M_{xx} at EA MNm/m	0.22	0.18	0.16
M_{yy} at A MNm/m	0.32	0.21	0.19

5.4The Effects of Structure-Foundation-Soil Interaction

It has been recognised for many years that the stiffness of a structure will affect the distribution of settlements along a strip or raft foundation, and that in turn, the distribution of structural loads and moments will be affected by the foundation flexibility. Methods of incorporating the foundation-soil interaction into a settlement analysis have been described by several authors, including, Lee and Brown (1972), Lee (1975) and Poulos (1975). In general, it has been found that the stiffness of the structure generally leads to a reduction in the differential settlements, compared to the usual methods which take the structural loads as being constant and statically determinant. An excellent example of the improvement in differential settlement prediction which may result from incorporating the structural stiffness is presented by Lopes and Gusmao (1991). For a 15 storey apartment building in Brazil, supported by a system of strip footings, the settlement distribution is predicted more closely if the stiffness of the structure is included in the settlement analysis (see Figure 13).

Lee (1975) has studied the effects of raft flexibility on the column loads in two-dimensional and three-dimensional structural frames, and has found that increasing raft flexibility leads to a more uniform distribution of structural loads than is the case for a rigid foundation (the usual case assumed by structural analysts). Lee also found that the use of the Winkler soil model predicted the reverse trend, and attributed this incorrect trend to the different settlement profiles which emerge from the subgrade reaction theory. Lee made the following observation: "With the advent of large high speed computers, the justification for the Winkler model is removed, and it is clear that it is now only of historical importance…this is no real reason for its continued use". In the intervening 25 years, computer power has increased by orders of magnitude, yet there is still an unfortunate but widespread persistence with the Winkler concept because of its convenience and simplicity. The price of this simplicity is high, given the potential for unreliable and unrealistic results and the enduring problem of assessing an appropriate modulus of subgrade reaction. The time has come for the Winkler concept to be consigned to history, and not to be perpetuated in modern-day structural and geotechnical analyses.

6 SETTLEMENT OF PILE GROUPS

Table 10 summarizes a selection of the many methods available for estimating the settlement of pile groups. Further details of some of these methods are given by Poulos and Davis (1980), Fleming et al. (1992), Poulos (1993, 1994) and Randolph (1994).
Attention here will be concentrated on a comparison between solutions from the interaction factor method, the equivalent pier method, and the equivalent raft method. Two idealised problems have been analysed, as illustrated in Figure 14:

1) a floating or friction pile group, where the founding layer is underlain by a layer of different stiffness (Figure 5a)
2) an end-bearing pile group which is founded on a stratum which is stiffer than the overlying soil (Figure 5b).

For each problem, an examination is made of the influence of the number of piles and the relative stiffness of the two layers on the group settlement predicted by the three methods. The interaction factor method has been implemented via the computer program DEFPIG (Poulos, 1990).

Figure 15 plots the settlement of the floating pile group as a function of the number of piles in the group, for three values of E_2/E_1 and for typical pile spacing and pile-soil parameters. For small numbers of piles, the equivalent raft method tends to underestimate settlement, as compared with the DEFPIG (interaction factor) analysis. However, for nine or more piles, the three methods generally agree well. There is a tendency for the equivalent pier method to underpredict the settlement if the underlying layer is relatively stiff but overall, it appears that the approximate methods are capable of providing a reasonable prediction of the settlement of floating pile groups.

Figure 16 compares solutions for the settlement of an end bearing group, for the case $E_2/E_1 = 5$. The equivalent raft method here significantly overpredicts the settlement for relatively small numbers of piles, but provides a satisfactory solution for 16 or more piles. Conversely, the equivalent pier method tends to underpredict the settlement as the number of piles in the group increases.

In summary, the foregoing comparisons suggest that:

1) for groups containing a relatively small number of piles, the interaction factor method or the equivalent pier method can be used with some confidence.
2) for groups containing more than about 16 piles, the equivalent raft method (implemented as described in this paper) can provide a useful approach for group settlement prediction.

The inaccuracies involved in the use of the equivalent pier or equivalent raft methods are likely to be significantly less than the uncertainties involved in assessing the geotechnical parameters.
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Table 10. Some Methods of Settlement Analysis for Pile Groups

Method	Category	Settlement	Diff. Settlem't	Pile Loads	Settl. Rate	Notes
Settlement Ratio $\rho_G = R_S \cdot \rho_1$	1	Y	-	-	-	R_S from empirical expressions (Skempton, 1953; Meyerhof, 1959) for sands
Meyerhof (1976) $\rho_G = \frac{0.9q\sqrt{B}\ I\ mm}{N}$	1	Y	-	-	-	q=net press.(kPa) B=group width(m) N=av. SPT within depth of B below piles I=(1-D′/8B)≥0.5
Equivalent Raft (Tomlinson, 1993; Poulos, 1993)	2	Y	Y	-	Y	Various modifications are available eg. Hirayama 1995.
Equivalent Pier (Poulos, 1993)	2	Y	-	-	Y?	Treat group as single pier of piles and soil
Settlement ratio $\rho_G = R_S \cdot \rho_1$	2	Y	-	-	-	R_S from elastic analysis; can approximate as $R_S = n^w$
Interaction Factor (Poulos & Davis, 1980)	3	Y	Y	Y	-	Can be implemented via programs such as DEFPIG & PIGLET
Boundary Element e.g. Banerjee & Driscoll (1976); Poulos & Hewitt, (1986)	3	Y	Y	Y	-	Implemented by programs such as PGROUP
2-D Finite Element	3	Y	Y	Y	Y*	Can idealize as plane strain or axi-symmetric
3-D Finite Element	3	Y	Y	Y	Y*	Various soil and pile models can be implemented

* Requires modelling of pore pressures and consolidation behaviour of soil.

6.1 Case Study

Goosens and Van Impe (1991) have described a case involving a block of 40 cylindrical reinforced concrete silos, each 52 m high and 8 m in diameter, that covered a rectangular area 34 m by 84 m. A 75 m high tower block was also constructed adjacent to the silos. The silos were built on a 1.2 m thick foundation slab which was supported by a total of 697 driven cast in situ reinforced concrete piles. The pile length was 13.4 m and the shaft diameter was 0.52 m. The diameter of the expanded base was variable, but was judged to have an average value of 0.8 m. The average pile load under operating conditions was about 1.3 MN.

Figure 17 shows a simplified geotechnical profile near the centre of the site, average values of the static cone resistance, and assessed values of Young's modulus of the various layers.

In the DEFPIG analysis, a ratio of 3 for small strain modulus to near-pile modulus was assumed, and the computed settlement ratios for 16 and 25 piles were extrapolated to give a value for 697 piles. The settlement ratio exponent was found to be 0.743, thus giving a settlement ratio of about 130 for the 697 piles. At the average load per pile of 1.3 MN, the computed single pile settlement was 5.0 mm, which is rather greater than the average measured value, from two pile load tests, of about 2.8 mm. The computed average group settlement was thus 650 mm. Because settlements were measured only along the outside of the silo, it was necessary to correct the computed average settlement to obtain the settlement along the outer edge of the group. On the assumption that the foundation is sufficiently large to behave as a flexible area, the settlement of the central edge is about two-thirds of the average value; consequently, the estimated settlement at the centre of the outer edge was 440 mm. Smaller settlements could have been predicted had a larger ratio of small strain to near-pile moduli been adopted, but there was little basis for assessment of this ratio in this case.

This case has also been considered by Mandolini and Viggiani (1997), using an analysis similar in principle to that used in DEFPIG. However, there were some important differences in their analysis, in particular, the use of a hyperbolic nonlinear analysis for the single pile based on a low-strain shear modulus, a different approach to the computation of the interaction factors, and the use of a maximum spacing beyond which interaction effects do not occur. The settlement profile computed by Mandolini and Viggiani is also shown in Figure 18, and is found to be in good agreement with the measured profile. The authors therefore conclude that "a proper application of the interaction factors method can give satisfactory results even for large groups".

While the work of Mandolini and Viggiani is encouraging, it also demonstrates that the success of some Category 3 methods may well depend on the assumptions employed and the choice of parameters, and that they will not necessarily always give a closer prediction of settlements than simpler equivalent raft and equivalent pier methods. It would appear that the latter methods can be retained and adopted, (and perhaps adapted as suggested by Hirayama, 1995, to improve their performance). Interaction factor methods also deserve to be retained, particularly as they provide a means of predicting both settlements and pile loads. However, some adaptation of the original method (such as that described by Mandolini and Viggiani, 1997) may prove beneficial and may result in settlement estimates which are less conservative than the original approach, and more in agreement with measured behaviour.

7. THE PIVOTAL ROLE OF PARAMETER ASSESSMENT

It is axiomatic that an essential ingredient for successful settlement prediction is the selection of appropriate geotechnical parameters. It is the author's experience that settlement predictions are far more sensitive to the geotechnical parameters and site characterization than to the method of analysis (e.g. Poulos, 1989). Limitations of space preclude a detailed discussion of methods for assessing parameters, but there appear to be number of "maxims" which appear worthy of recognition. These include the following:

1) all practical methods of settlement analysis involve simplification of the soil behaviour to enable calculations to be made readily e.g. the idealization of soil as an elastic material. The parameters describing this simplified behaviour must therefore not be considered to be "constants"
2) because soil behaviour is generally non-linear and highly dependent on the effective stress state and the stress path, tests to assess the simplified parameters should be carried out using appropriate initial stresses and applied stress paths. For example, the stress-path method of Lambe (1964) and the laboratory triaxial test procedure proposed by Davis and Poulos (1968) attempt to subject a laboratory

sample to a stress history similar to that in the field

3) extreme caution must be exercised when applying elastic theory to soils for the purpose of predicting settlements. First, the distinction between undrained and drained behaviour needs to be made for clay soils. Second, an elastic modulus derived for the settlement analysis of a spread footing may not be appropriate for the settlement analysis of a pile, because of the differences in the stress history caused by pile installation and by the predominant load transfer mechanism of the pile being shear rather than normal stress.
4) when assessing soil parameters from in-situ tests, it is desirable for the stress path followed by the test to be similar to that for the foundation being considered. Thus, for example, the results of a plate bearing test would be more relevant to a shallow foundation than to a pile foundation
5) different values of the equivalent elastic modulus may be relevant to different components of the settlement. As an example, Figure 19 shows that there may be four distinct values of modulus for a pile or pile group. The behaviour of a single pile is likely to be affected primarily by the soil modulus values immediately adjacent to and below the pile, whereas a pile group may be influenced considerably by the soil modulus values away from and below the pile tips
6) empirical correlations between simple in-situ tests (e.g. SPT, CPT) and soil deformation parameters are often valuable for preliminary estimates of settlement. Because such tests do not generally follow the correct stress path, the potential for inaccurate settlement predictions is generally greater than if more appropriate in-situ or laboratory testing is carried out. It must be borne in mind that most empirical equations are dependent on both soil type and on foundation type, and that indiscriminate use of a correlation may lead to unsatisfactory results. As an example, the correlation developed by Schmertmann (1978) for the Young's modulus applies to shallow foundations on sand, but would be quite inappropriate for piles in clay.

Burland (1989) emphasizes the existence and significance of non-linearity of soil behaviour, even when the strains are small. He also summarizes backfigured undrained Young's modulus values from various categories of field problems, and shows that, for a given value of load factor, the value of apparent Young's modulus is highly problem-dependent. Significantly higher values are obtained for pile foundations than for shallow footings or strutted excavations, or values from laboratory triaxial tests.

8. CONCLUSIONS

This paper has reviewed methods of settlement analysis for various types of foundations and has attempted to assess the capabilities of conventional methods of analysis and design in the light of more modern methods developed from research over the past two to three decades. Based on this review, it has been suggested that the conventional methods of settlement analysis should be adopted, adapted or discarded. The results of this assessment may be summarized as follows:

a) Methods which may be adopted -
   - Schmertmann's method for settlement of shallow footings on sands
   - elastic method for settlement of shallow footings on sands
   - elastic analysis of raft foundations
   - equivalent raft analysis for pile groups
   - equivalent pier analysis for pile groups

b) Methods which should be adapted -
   - one-dimensional settlement analysis of shallow footings on clay (make allowance for immediate settlement)
   - one-dimensional rate of settlement analysis for shallow footings (make allowance for three-dimensional geometry and soil anisotropy)
   - linear creep/secondary settlement versus log time relationship (need to consider carefully when creep commences)

- strip analysis for rafts (allow for loaded areas outside the strip section analyzed)

c) Methods which may need to be discarded -

Methods for shallow foundations based on subgrade reaction concepts; while they may sometimes give satisfactory results for isolated loadings, they can be misleading for uniform loadings and may also create difficulties with the selection of modulus of subgrade reaction because of its dependence on the foundation dimensions.

It must be stated that the above assessments contain a certain element of subjectivity. Also, it is vital to recognize that the ultimate success of settlement prediction depends as much (if not more) on appropriate modelling and parameter selection than on the method of analysis used.

In conclusion, it is sobering to recall the following comments of Terzaghi (1951):
"....foundation engineering has definitely passed from the scientific state into that of maturity.......one gets the impression that research has outdistanced practical application, and that the gap between theory and practice still widens".

The gap to which Terzaghi referred is far greater now, almost fifty years later, and it would seem appropriate that a major effort be mounted for the beginning of the new millennium to assess the current state of practice in various aspects of foundation engineering, and incorporate relevant aspects of modern research and state of the art knowledge into practice.

9. ACKNOWLEDGEMENTS

The author is fortunate to have had the privilege of collaborating with a number of distinguished colleagues who have contributed significantly to the state of knowledge of foundation settlement analysis, and in particular, the late Professors E.H. Davis and J.R. Booker, the late Dr P.T. Brown, and Professor T William Lambe. Grateful acknowledgement is also given to Professor J.P. Carter and Associate Professor J.C. Small for their generous assistance in the preparation of this paper.

10. REFERENCES

Alpan, I. (1964). Estimating Settlements of Foundations on Sands. *Civ. Eng. And Public Works Review.*

Banerjee, P.K. and Driscoll, R. (1976). Three-Dimensional Analysis of Raked Pile Groups. *Proc. ICE,* Pt. 2, 61: pp. 653-671.

Bjerrum, L. , Johnson, W. and Ostenfeld, C. (1957). The Settlement of a Bridge Abutment on Friction Piles. *Proc. 4th ICSMFE, London,* 2:pp. 14-19.

Booker, J.R. (1974). The Consolidation of a Finite Layer Subject to Surface Loading. *Int. J. Solids and Structs.,* Vol. No.10, pp. 1053-1065.

Bowles, J.E. (1984). *Foundation Analysis and Design. 4th Ed.* McGraw Hill, New York.

Brand, E.W. (1990). Predicted and Observed Performance of an Embankment Built to Failure on Soft Clay. *Geotech. Eng.,* 22(1): pp. 23-42.

Briaud, J-L. and Gibbens, R.M. (1994). Test and Prediction Results for Five Large Spread Footings on Sand. *Geot. Spec. Pub. 41, ASCE,* pp. 92-128.

Briaud, J-L., Hossain, K., Barfknecht, J. and Lee, J.H. (2000). Load Settlement Curve Method for Spread Footings in Sand. *Report to Texas Trans. Institute, Texas A & M University, College Station, Texas.* 67pp.

Brown, P.T. (1969). Numerical Analyses of Uniformly Loaded Circular Rafts on Elastic Layers of Finite Depth. *Geotechnique,* 19(2): pp. 301-306.

Brown, P.T. (1975). Strip Footings with Concentrated Loads on Deep Elastic Foundations. *Geotech. Eng.,* 6(1): pp. 1-13.

Brown, P.T. (1977). Strip Footings. *Geotechnical Analysis and Computer Applications;* Lecture 7,

University of Sydney.
Buisman, A.S. (1936). Results of Long Duration Settlement Tests. *Proc. 1st ICSMFE, Cambridge, Mass.,* 1: pp. 103-106.
Burland, J.B. (1989). Ninth Laurits Bjerrum Memorial Lecture: Small is Beautiful – the Stiffness of Soils at Small Strains. *Can. Geot. Jnl., 26(4):* pp. 499-516.
Burland, J.B., Broms, B.B. and de Mello, V.F.B. (1977). Behaviour of Foundations and Structures. *Proc. 9 ICSMFE, Tokyo,* Vol.2, pp. 495-546.
Burland, J.B. and Burbridge, M.C. (1985). Settlements of Foundations on Sand and Gravel. *Proc. I.C.E.*, Part 1, pp. 1325-1381.
Carter, J.P., Booker, J.R. and Small, J.C. (1979). The Analysis of Finite Elasto-Plastic Consolidation. Int. JNAMG, Vol. 3: pp. 107-129.
Chua, K.M., Xu, L., Pease, E. and Tamare, S. (1994). Settlement of Test Footings: Predictions from the University of New Mexico. *Geot. Spec. Pub. 41, ASCE,* pp. 240-244.
D'Appolonia, D.J. and D'Appolonia, E. (1970). Closure to Settlement of Spread Footings on Sand, *JSMFED*, ASCE, 96(SM2): pp. 754-762.
D'Appolonia, D.J., Poulos, H.G. and Ladd, C.C. (1971). Initial Settlement of Structures on Clay. *JSMFD, ASCE,* 97(SM10): pp. 1359-1397.
Davis, E.H. and Poulos, H.G. (1968). The Use of Elastic Theory for Settlement Prediction Under Three-Dimensional Conditions. *Geotechnique,* 18(1): pp. 67-91.
Davis, E.H. and Poulos, H.G. (1972). Rate of Settlement Under Three Dimensional Conditions. *Geotechnique,* 22(1): pp. 95-114.
Decourt, L. (1989). SPT – State of the Art Report. *Proc. 12 ICSMFE, Rio de Janeiro,* 4: pp. 2405-2416.
Finno, R. (Ed.) (1989). *Predicted and Observed Axial Behaviour of Piles.* ASCE Spec. Pub. 23, 385p.
Fleming, W.G.K., Weltman, A.J., Randolph, M.F. and Elson, W.K. (1992). *Piling Engineering (2nd Ed.).* Surrey Univ. Press.
Fraser, R.A. and Wardle, L.J. (1976). Numerical Analysis of Rectangular rafts on Layered Foundations. *Geotechnique,* 26(4): pp. 613-630.
Gibson, R.E. and Lo, K.Y. (1961). A Theory of Consolidation for Soils Exhibiting Secondary Consolidation. *Acta Polytech. Scand.*, p. 296.
Goossens, D.and van Impe, W.F. (1991). Long Term Settlements of a Pile Group Foundation in Sand, Overlaying a Clayey Layer. *Proc. 10th Eur. Conf. SMFE, Florence,* 1: pp. 425-428.
Hain, S.J. and Lee, I.K. 1974). Rational Analysis of Raft Foundations. *Jnl. Geot. Div., ASCE,* 100(GT7).
Jeyapalan, J.K. and Boehm, R. (1986). Procedures for Predicting Settlements in Sands. *Proc.Spec. Conf. On Settl. Of Shallow Founds. on Cohesionless Soils, Geot. Spec. Pub.* 5 ASCE, pp. 1-22.
Lambe, T.W. (1964). Methods of Estimating Settlement. *Jnl. SMFD, ASCE,* 90(SM5): pp. 47-71.
Lambe, T.W. (1973). Predictions in Soil Engineering. *Geotechnique,* 23(2): pp. 149-202.
Lambe, T.W. , Wolfskill, L.A. and Wong, I.H. (1970). Measured Performance of a Braced Excavation. *JSMFD, ASCE,* 96(SM3): pp. 817-836.
Lambe, T.W. et al. (1974). *Proceedings of Foundation Deformation Prediction Symposium.* MIT, Cambridge, Mass., 2 Vols.,482p.
Lee, I.K. (1975). Structure-Foundation-Supporting Soil Interaction Analysis. *Soil Mechanics, Recent Developments, Ed. S. Valliappan et al., Univ. NSW, pp.*255-294.
Lee, I.K. and Brown, P.T. (1972). Structure and Foundation Interaction Theory. *Jnl. Struct. Divn., ASCE,* 98 (ST11).
Leroueil, S. (1996). Compressibility of Clays: Fundamental and Practical Aspects. *Jnl. Geot. Eng., ASCE,* 122 (7): pp. 534-543.

Lopes, F.R. and Gusmao, A.D. (1991). On the Influence of Soil-Structure Interaction in the Distribution of Foundation Loads and Settlements. *Proc. 10th Eur. Conf. SMFE, Florence,* 2: pp. 475-478.
Mandolini, A. and Viggiani, C. (1997). Settlement of Piled Foundations. *Geotechnique,* 47(4): pp. 791-816.
Mesri, G. and Godlewski, P.M. (1977). Time-and Stress-Compressibility Interrelationship. *Jnl. Geot. Eng., ASCE,* 103(GT5): pp. 417-430.
Mesri,G., Lo, D.O.K. and Feng, T-W. (1994). Settlement of Embankments on Soft Clays. *Geot. Spec. Pub. 40, ASCE,* 1: pp. 8-56.
Meyerhof, G.G. (1956). Penetration Tests and Bearing Capacity of Cohesionless Soils. *Jnl. SMFD, ASCE,* 82(SM1): pp. 1-19.
Meyerhof, G.G. (1965). Shallow Foundations". *JSMFD, ASCE,* 91(SM2): pp. 21-31.
Meyerhof, G.G. (1976). Bearing Capacity and Settlement of Pile Foundations. *J. Geotech. Eng., ASCE,* 102(GT3): pp. 195-228.
Meyerhof, G.G. (1979). Soil-Structure Interaction and Foundations. General Report. *Proc. 6th Panamerican Conf. SMFE, Lima, pp.* 109-140.
Moore, P.J. and Spencer, G.K. (1969). Settlement of Building on Deep Compressible Soil. *JSMFD, ASCE,* 95(SM3): pp. 769-790.
Morton, K and Au, E. (1974). Settlement Observations on Eight Structures in London. *Proc. Conf. Settl. Of Structures, Cambridge,* Pentech Press, pp. 183-203.
NAVFAC (1982). *Soil Mechanics-Design Manual 7.1.* US Dept. Navy, US Govt. Printing Office, Washington.
Parry, R.H.G. (1971). A Direct Method of Estimating Settlements in Sands for SPT Values. *Proc. Symp. Interaction of Struct. and Found.*, Birmingham, pp. 29-32.
Peck, R.B. (1977). Case Histories in Soil Mechanics. *Proc. 9th ICSMFE, Tokyo, Spec. Case History Volume,* pp. 1-14.
Peck, R.B. and Bazaraa, A.R.S. (1969). Discussion to Settlement of Spread Footings on Sand. *JSMFD, ASCE,* 95(SM3): pp. 905-909.
Peck, R.B, Hansen, W.E. and Thornburn, T.H. (1974). *Foundation Engineering. 2nd Edition.* John Wiley, New York.
Poulos, H.G. (1975). Settlement Analysis of Structural Foundation Systems. *Proc. 4th SE Asian Conf. SMFE, Kuala Lumpur, pp.* 4-54 – 4-62.
Poulos, H.G. (1989). Pile Behaviour – Theory and Application. *Geotechnique,* 39(3): pp. 365-415.
Poulos, H.G. (1991). Analysis of Piled Strip Foundations. *Comp. Methods and Advs. In Geomechs., Ed. G. Beer et al., Balkema,* 1: pp. 182-191.
Poulos, H.G. (1993). Settlement of Bored Pile Groups. *Deep Founds. on Bored and Auger Piles, ed. W. van Impe, Balkema,* pp. 103-117.
Poulos, H.G. (1994a). An Approximate Numerical Analysis of Pile-Raft Interaction. *IJNAMG,* 18: pp. 73-92.
Poulos, H.G. (1994b). Settlement Prediction for Driven Piles and Pile Groups. *Spec. Tech. Pub. 40, ASCE,* 2: pp. 1629-1649.
Poulos, H.G. (1996). Measured and Predicted Settlement of Shallow Foundations on Sand. *Proc. 7th ANZ Conf. Geomechs., Adelaide,* pp. 686-692.
Poulos, H.G. and Davis, E.H. (1980). *Pile Foundation Analysis and Design.* John Wiley, New York.
Poulos, H.G. and Hewitt, C.M. (1986). Axial Interaction Between Dissimilar Piles in a Group. *Proc. 3rd Int. Conf. Num. Meths. Offshore Piling, Nantes,* pp. 253-270.
Randolph, M.F. (1994). Design Methods for Pile Groups and Piled Rafts. *SOA Report, 13 ICSMFE, New Delhi,* 5: pp. 61-82.
Sandhu, R.S. and Wilson, E.L. (1969). Finite Element Analysis of Seepage in Elastic Media. *J. Eng. Mech. Div. ASCE,* 95, (3), pp. 641-652.
Schmertmann, J.H., Hartmann, J.P. and Brown, P.R. (1978). Improved Strain Influence Factor Diagram. *Jnl. Geot. Eng. Div., ASCE.* 104(GT8): pp. 1131-1135.

Schultze, E. and Sherif, G. (1973). Prediction of Settlements from Evaluated Settlement Observations for Sand. *Proc. 8th ICSMFE, Moscow*, Vol. 1.3, pp. 225-230.
Simons, N.E. and Som, N.N. (1970). Settlement of Structures on Clay with Particular Emphasis on London Clay. *CIRIA REPORT 22,* July.
Skempton, A.W. and Bjerrum, L.(1957). A contribution to the Settlement Analysis of Foundations on Clay. *Geotechnique,* 7(3): pp. 168-178.
Skempton, A.W., Peck, R.B. and MacDonald, (1955). Settlement Analyses of Six Structures in Chicago and London. *Proc. ICE, London,* 4(1): pp. 525-544.
Small, J.C., Booker, J.R. and Davis, E.H. (1976). Elasto-Plastic Consolidation of Soil. *Int. J. Solids & Structs.,* 12, pp. 431-448.
Small, J.C. (1998). CONTAL Users Guide. *Centre for Geot. Research, Univ. of Sydney.*
Tan, C.K. and Duncan, J.M. (1991). Settlement of Footings on Sands- Accuracy and Reliability. *Proc. Geot. Eng. Congress, ASCE, Geot. Spec. Pub. 27.* Vol. 2, pp. 446-455.
Terzaghi, K. (1951). The Influence of Modern Soil Studies on the Design and Construction of Foundations. *Building Research Congress,* London, pp. 68-74.
Terzaghi, K. and Peck, R.B. (1967). *Soil Mechanics in Engineering Practice. 2nd. Ed.,* John Wiley, New York.
Tomlinson, M.J. (1995). *Foundation Design and Construction.* Longman Scientific and Technical, Harlow.
Wood, L. A. (1977). The Economic Analaysis of Raft Foundations. *Int. J. NAMG,* 1: p. 377.
Vesic, A.S. (1961). Bending of Beams Resting on Isotropic Elastic Solid. *Jnl. Eng. Mechs. Div., ASCE,* 87(EM2): pp. 35-53.

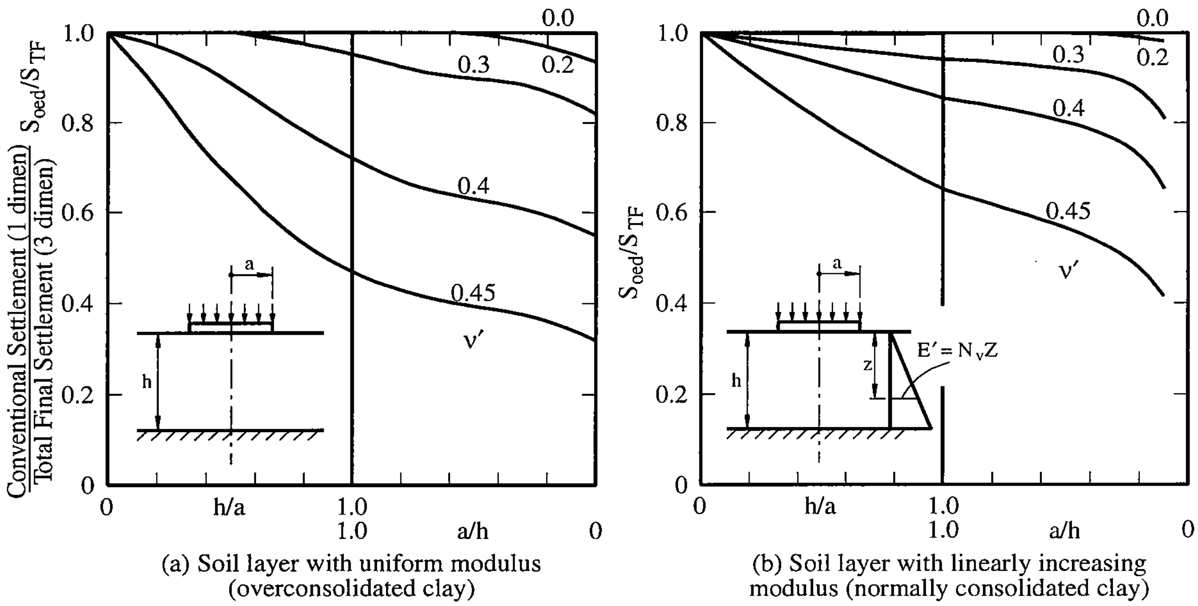


Figure 1. Theoretical ability of one-dimensional analysis to predict total final settlement.

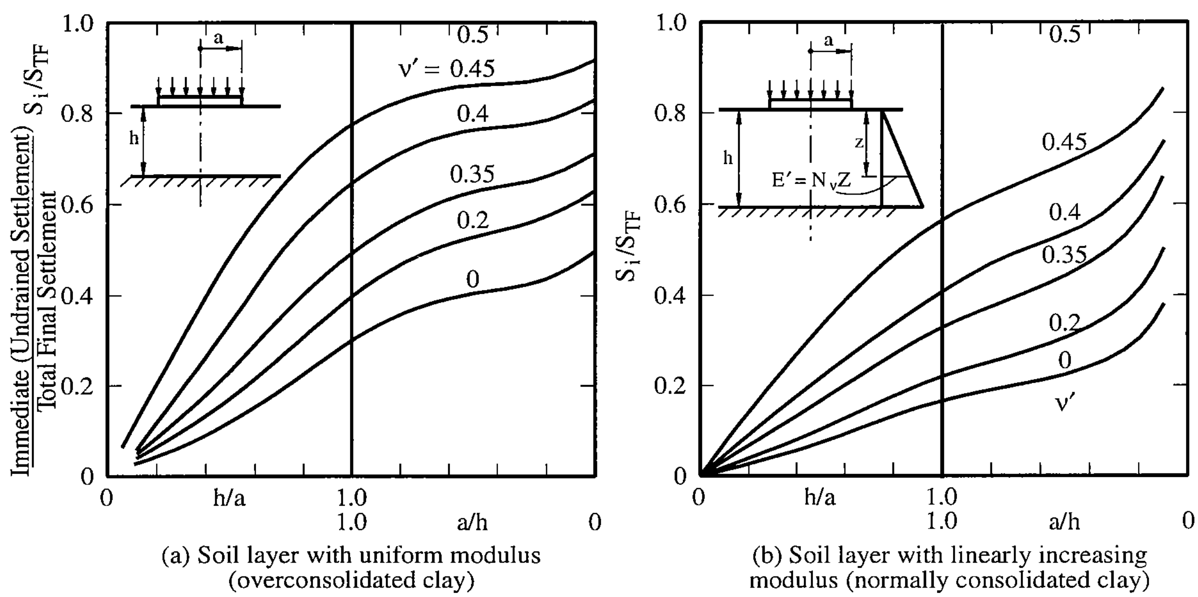
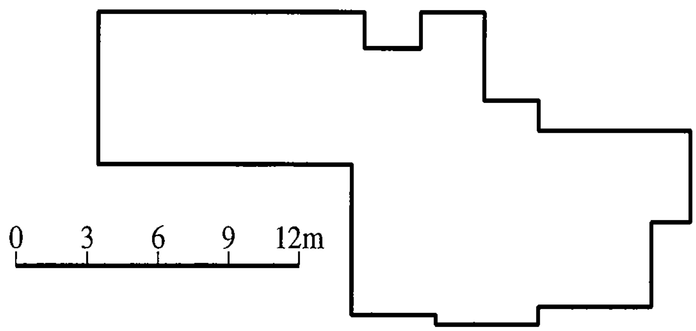
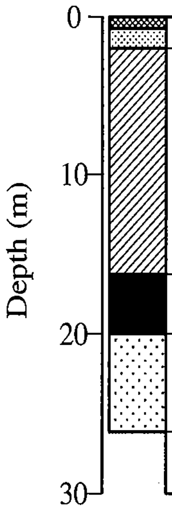


Figure 2. Theoretical relative importance of immediate settlement.



Plan outline of building



Soil	Nat w/c %	LL	PL	Stress Path Triaxial tests	Oedometer Tests (conv'l) m_v m^2/MN
Fill, firm silty sand	26 - 33	26	23	--	--
Soft black clays & silts, some organic matter & shell fragments	49 - 90	91 - 98	30 - 47	E_u = 34.74 MPa E' = 0.52 MPa ν' = 0.40 A = 0.92 (at repres. depth of 8.5 m)	1.32 0.50
Soft peaty sandy silt	40 - 69	83	38 - 44	--	--
Grey silty sand	24 - 33	29	21	--	0.15
Med-coarse sand & gravel	--	--	--		

Figure 3. Boyd Domestic College Building, South Melbourne
(After Moore & Spencer, 1969).

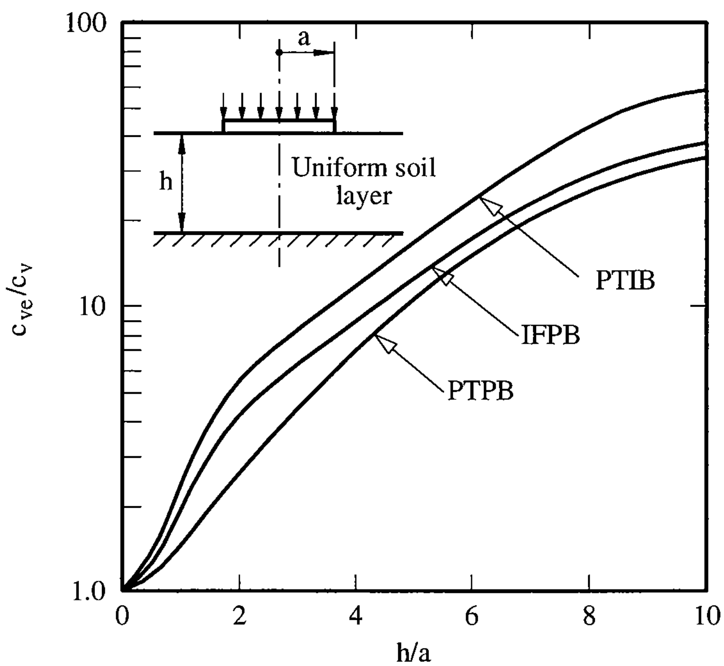


Figure 4. Equivalent coefficient of consolidation for 1-D analysis of rate of settlement - circular footing.

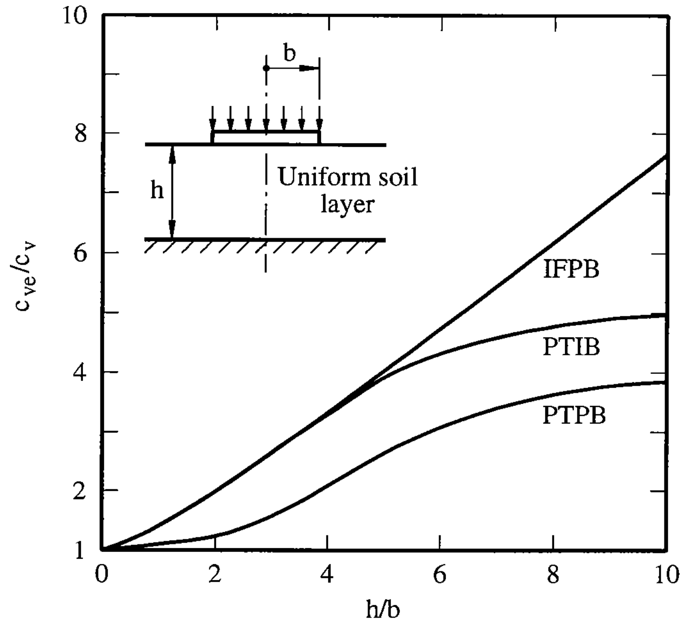
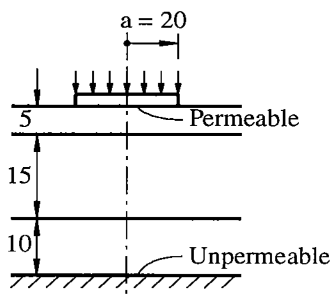


Figure 5. Equivalent coefficient of consolidtion for 1-D analysis of rate of settlement - strip footings.



E′ MPa	ν′	m_v m^2/MN	c_v m^2/yr	k m/yr
20	0.35	0.0312	80	0.0249
10	0.4	0.0467	20	0.0093
40	0.3	0.0186	60	0.0111

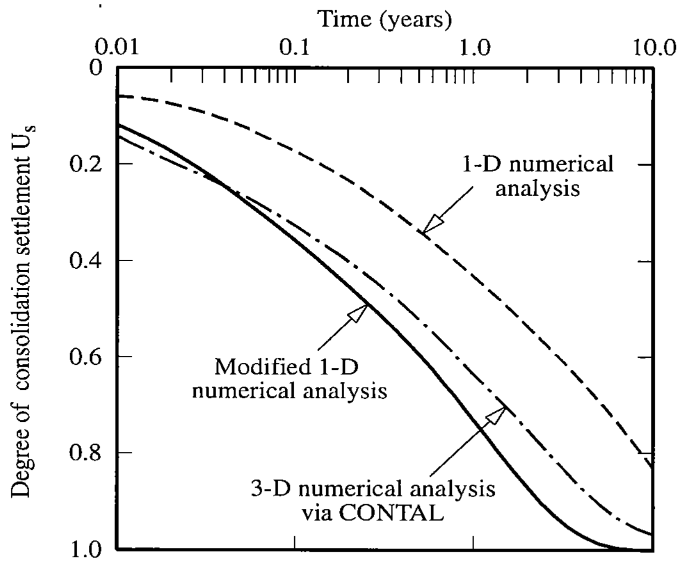


Figure 6. Example of comparison between 3-D & modified 1-D rate of settlement analyses.

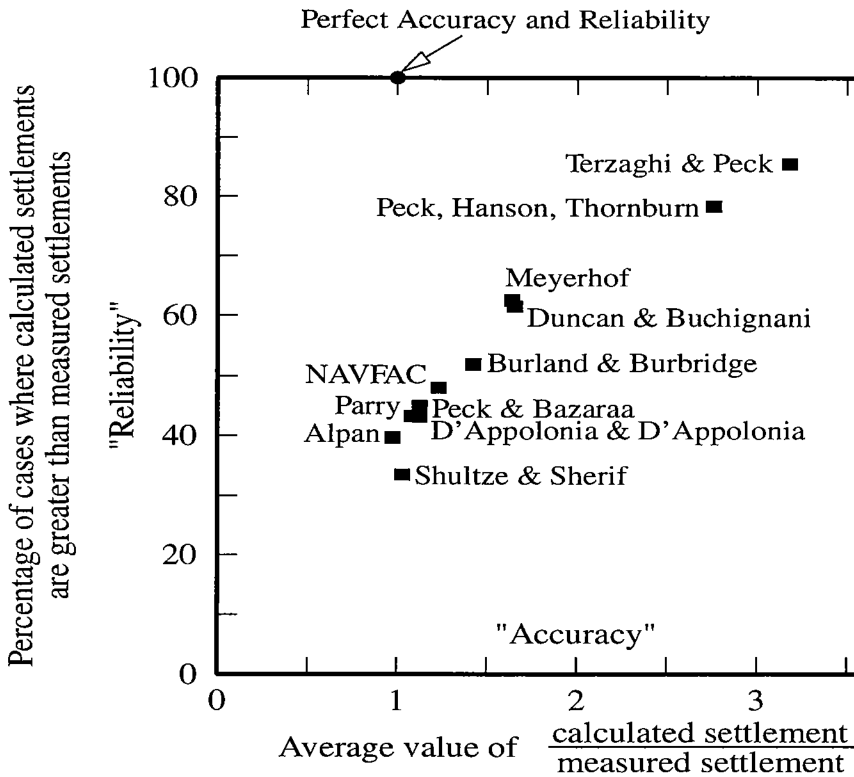


Figure 7. Relationships between accuracy and reliability for eleven methods based on SPT blow count. (Tan and Duncan, 1991).

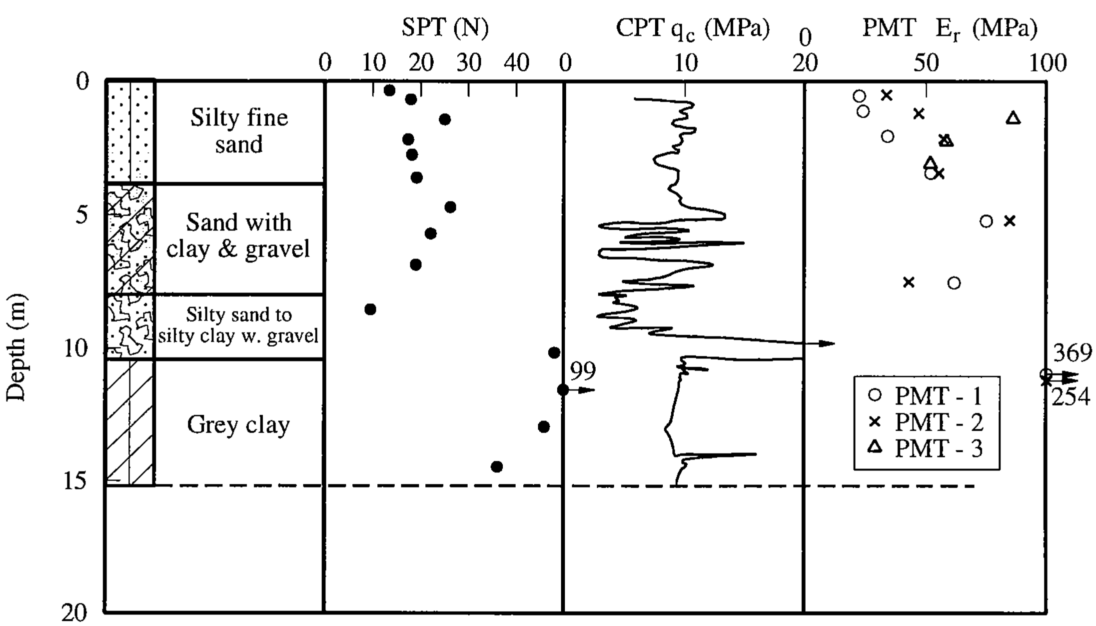


Figure 8. Summary of soil conditions near Footing 1.

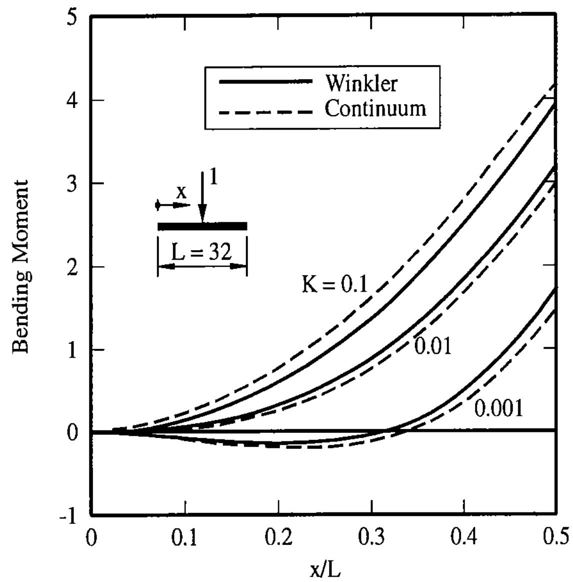


Figure 9. Bending moment distributions using Winkler and continuum soil models (Brown, 1977).

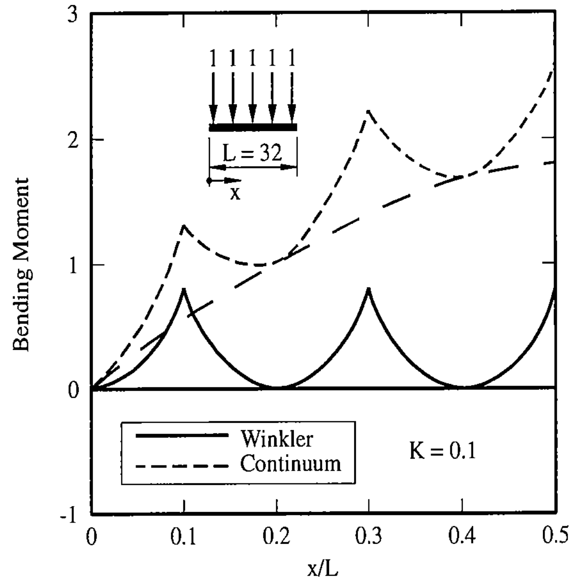


Figure 11. Bending moment distributions using Winkler and continuum soil models (Brown, 1977).

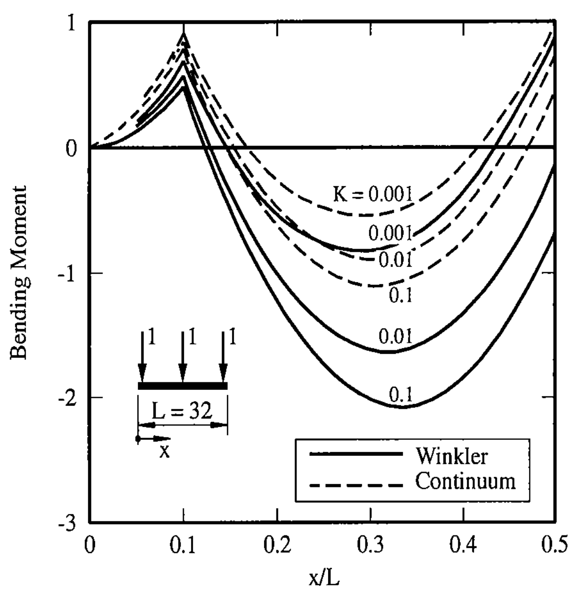


Figure 10. Bending moment distributions using Winkler and continuum soil models (Brown, 1977).

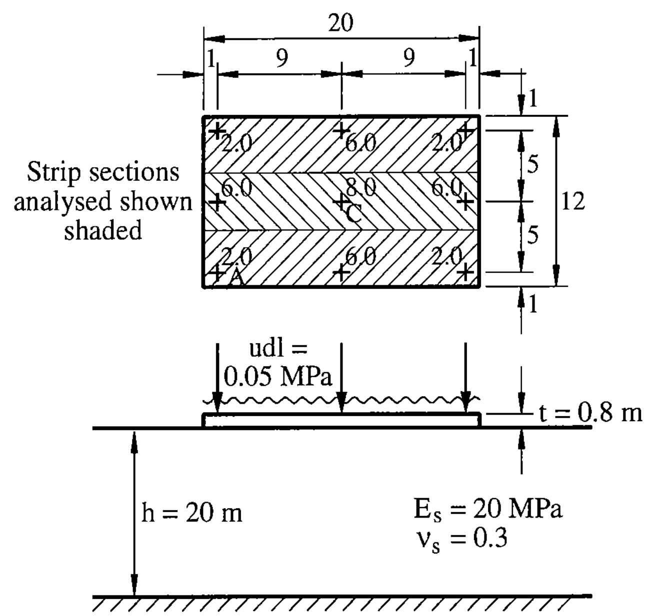


Figure 12. Example of raft analysis.

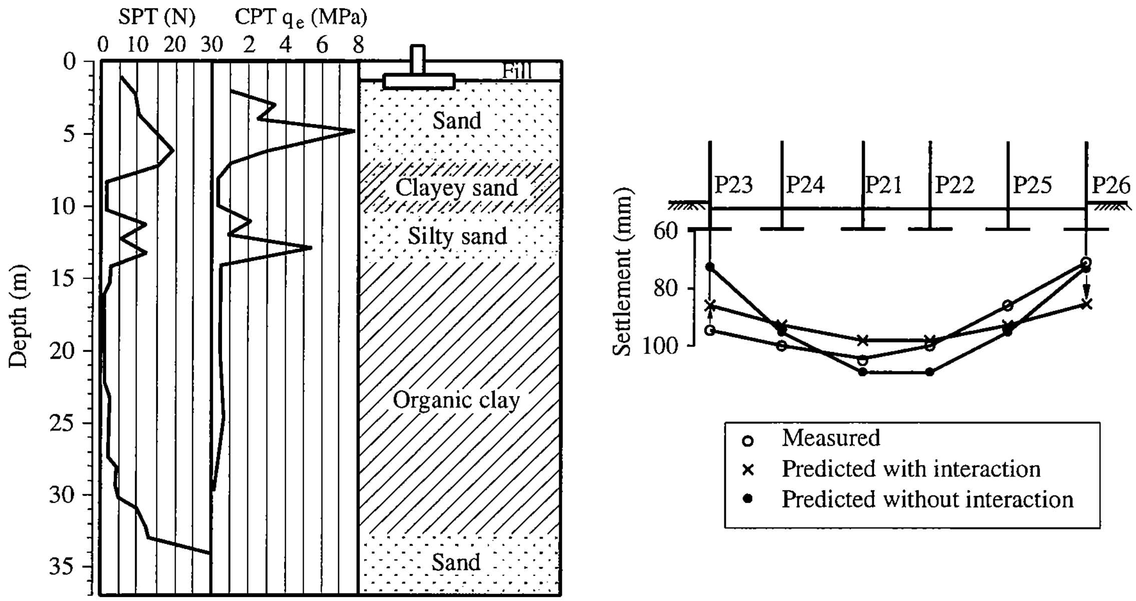
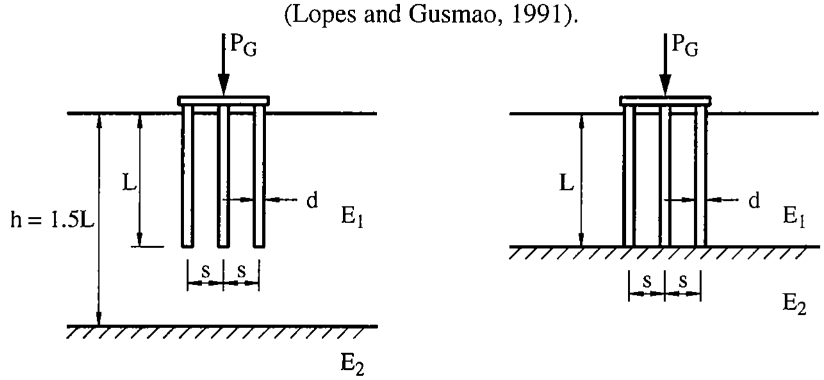


Figure 13. Effect of including structure-foundation interaction on predicted settlements (Lopes and Gusmao, 1991).



(a) Floating pile group

(b) End bearing pile group

Figure 14. Pile group cases analyzed.

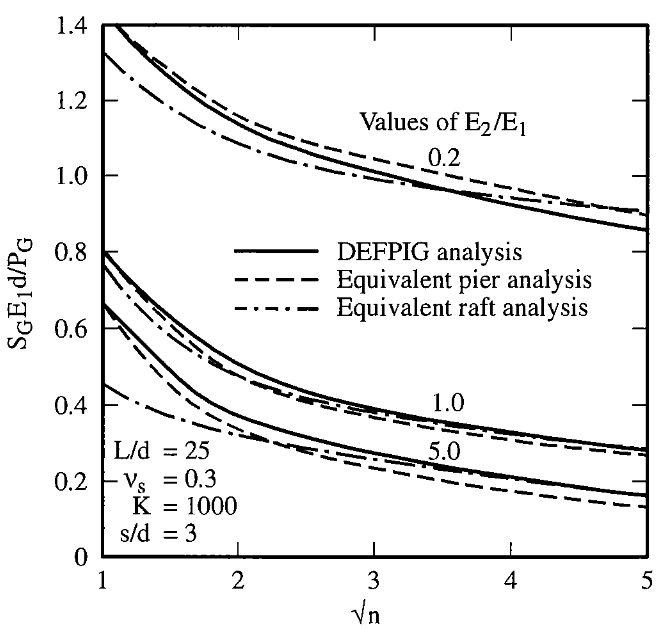


Figure 15. Comparison between solutions for total final settlement. Floating pile group in soil underlain by different layer.

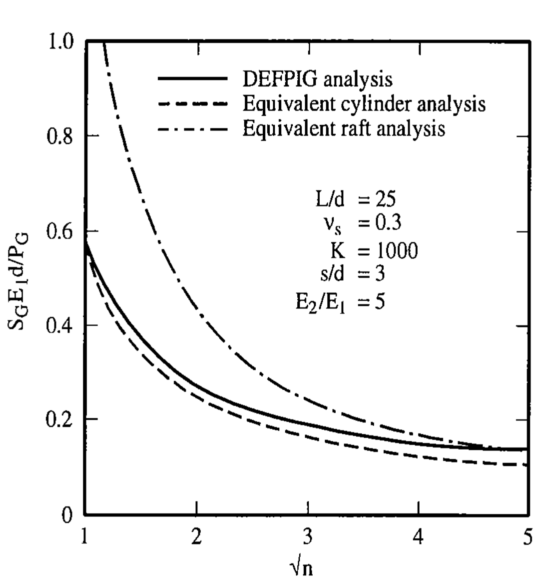


Figure 16. Comparison between solutions for total final settlement. End bearing group on stiffer layer.

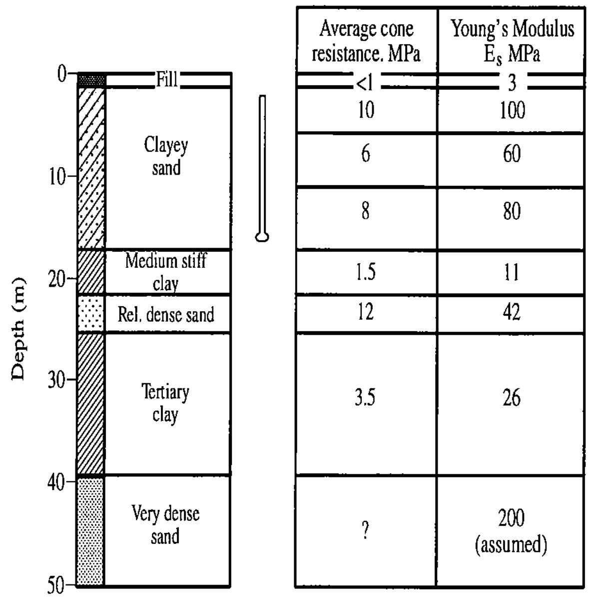


Figure 17. Geotechnical profile and model for case of Goosens and Van Impe (1991).

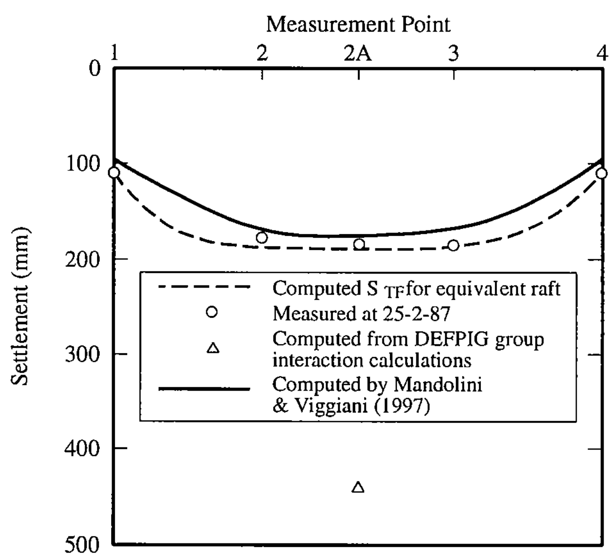


Figure 18. Computed and measured settlement along edge of silo.

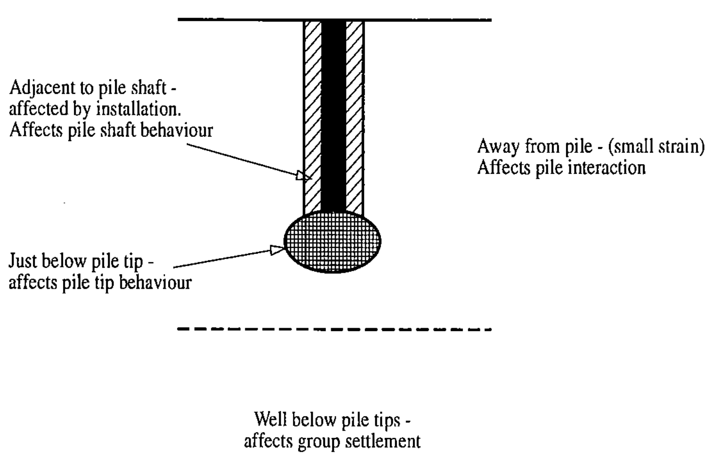


Figure 19. Different soil modulus values around a pile.