Victor de Mello Lecture

The Victor de Mello Lecture was established in 2008 by the Brazilian Association for Soil Mechanics and Geotechnical Engineering (ABMS), the Brazilian Association for Engineering Geology and the Environment (ABGE) and the Portuguese Geotechnical Society (SPG) to celebrate the life and professional contributions of Prof. Victor de Mello. Prof. de Mello was a consultant and academic for over 5 decades and made important contributions to the advance of geotechnical engineering. Each year a worldwide acknowledged geotechnical expert is invited to deliver this special lecture. It is a privilege to have Dr. Harry G. Poulos (Coffey Geotechnics, Australia) delivering the second edition of the Victor de Mello Lecture. Dr. Poulos and Prof. de Mello were close friends for decades and in his lecture he reviews the contributions of the late Victor de Mello to foundation engineering and highlights the insights that he provided in a number of key areas.

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The de Mello Foundation Engineering Legacy

Harry G. Poulos

Abstract. This paper reviews the contributions of the late Victor de Mello to foundation engineering and attempts to highlight the insights that he provided in a number of key areas, including foundation design principles, the bearing capacity of shallow foundations, the axial load capacity of deep foundations, and the behaviour of foundations incorporating settlement reducing piles. In each case, de Mello challenged some of the existing concepts and as a consequence, subsequent research has clarified the profession's understanding and has led to the development and implementation of improved methods of design. Some examples of developments in the above areas, and their application to practice, are described.

Keywords: bearing capacity, design criteria, foundations, piles, piled raft, settlement.

1. Introduction

The late Victor de Mello was no ordinary man. He was not only one of the world's pre-eminent geotechnical engineers, but also a person with an enormous breadth and depth of knowledge, and with passionate but considered views of many aspects of human society and existence. As a consequence, he was a vibrant and stimulating colleague and friend. His personal qualities have been described fully and eloquently by Professor John Burland in his first de Mello Lecture (Burland, 2008) and I can only add that I was privileged, as was Professor Burland, to have the encouragement of this giant of our profession in the early stages of my career. He was extraordinarily well-read, both in his professional field and in many other areas of intellectual endeavour, and could debate with equal authority the finer points of soil behaviour and the competing virtues of various philosophers of the enlightenment.

De Mello was an expert in several areas of geotechnical engineering, and in particular, embankment dams, and his 1977 Rankine Lecture dealt with this topic in an authoritative and expansive way. He also had a major influence on foundation engineering, and it is this aspect that will be explored in this paper. In particular, there are two pivotal papers that will be referred to frequently here, his State of the Art Report at the 7th International Conference in Mexico City in 1969, entitled "Foundations of Buildings in Clay", and his General Report with Burland & Broms at the 9th International Conference in Tokyo entitled "Behaviour of Foundations and Structures". An indication of his breadth of reading is evidenced by the very large number of references in these papers, 344 in the first and 333 in the second. Mention can also be made here of his epic treatise on the Standard Penetration Test (de Mello, 1971) which contained no less than 353 references, his 1994 Terzaghi Oration at the 13th International Conference in New Delhi, and in a different vein, his paper in 2000 uniquely entitled "Overview of hypotheses not plucked or pursued. Merit recanting or rechanting?"

I will attempt in this paper to summarize the engineering philosophy of Victor de Mello and then to examine some areas within foundation engineering in which he made notable contributions, and in which he identified shortcomings. The questions that he raised have been addressed subsequently by both researchers and practitioners, and have led to a better understanding of foundation behaviour and to more robust practical methods of analysis and design. The areas that will be discussed include the bearing capacity of shallow foundations, the load capacity and settlement of piles under axial loading, and the behaviour and design of settlement reducing piles.

2. Some Aspects of the De Mello Philosophy

2.1. Broad views

Victor de Mello expounded his philosophical views on a number of issues, some related directly to foundation engineering, and some to broader issues of design, education and the role of the engineer in society. Burland (2008) succinctly summarized de Mello's philosophy in terms of five Design Principles (DP). These were oriented towards embankment dam design, but can perhaps be generalised as follows:

1. DP1 - Aim to design out any risk from behaviour triggered by local phenomena – *Robustness.*
2. DP2 - Use a dominant feature to cut across uncertainties – *Change the problem*
3. DP3 - Aim at homogenization – *Redundancy.*
4. DP4 - Minimize rapid uncontrolled loading – *Observational control.*
5. DP5 - Question each design assumption and the consequences of departure from it – *Ask "what if" questions.*

Beyond these broad design principles, there were a number of other viewpoints that de Mello expressed (often very forcefully), and a small selection of these views is presented below, based on his published papers. Most of the

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quotations are self-explanatory and require little or no comment.

2.2. False conclusions from data

De Mello was highly critical of people who drew inappropriate conclusions from available data, and illustrated his point with the following example (de Mello, 1984):

"Most persons die in bed; therefore bed is the single most dangerous place for humans".

2.3. Use and abuse of statistics

Statistics was viewed by de Mello as a useful tool but one that was frequently mis-used or abused. The following quotation sets out his views on this subject.

"We must shun statistics at random, and choose to apply statistical adjustments to our reasonable theories. The temporary application of a presumed theory does not preclude that it is not satisfactory, and consequently revising it, or even proposing an entirely different one; what cannot be condoned is the attempt to extract conclusions from data at random and spurious statistics, without any theory, however nominal, or any design and purpose, since such efforts prove sterile and may even lead to dangerous conclusions" (de Mello, 1984).

2.4. The costs of undue conservatism and the problem of codes

An enduring theme in de Mello's publications was his extreme distaste for excessive conservatism brought about by a lack of understanding of geotechnical and foundation behaviour and the shelter that codes and standards provided for those who lacked such understanding.

"Two fundamental challenges in geotechnical civil engineering have been neglected under the avalanche of the published word in scientific quantifications. One is the nurturing of past experience of individual cases. The other is the global resulting cost to society of the constructed facility, with due inclusion of the costs of risk and of discredited professional prestige" (de Mello, 1994).

"Do the learned writers of prescriptions and codes realize how much and how unjustifiably they increase the conservatism of driven piling?" (de Mello, 1995).

"Misunderstood pronouncements, and a few visible failures, have weighed a thousand times more than the trernendously more important silent record of cases that did not merit study or publication".

"How can committees, discussing Codes, lightly banter around with changes of FS values (*e.g.* from 1.5 to 2.0, or vice-versa) without any statistical data to evaluate the magnitudes of the consequences?" (de Mello, 1995).

2.5. The philosophy of design

"We recognise two distinct phases of study, firstly, the adjustment of parameters and computational models and methods, so as to be able to predict deformations or other behaviour reasonably. The second problem is one of decision: how acceptable are the displacements predicted or observed" (de Mello, 1983).

"Of the many absurdities in design practices, one lies in requiring the same FS per pile whether it is alone in supporting a column, or is one of a group for that task" (de Mello, 1995).

2.6. Role of computers and computations

"The computer has diverted a great proportion of attention from real-life field geotechnics – paper is easily generated and imprinted, and checking proof positive for mental models is simpler" (de Mello, 2004).

"Computations (analytic or numerical) are a means and not ends, in service of engineering" (de Mello, 1992).

2.7. Importance of knowing the ground conditions

"A prime requirement for foundation design and construction will always be a knowledge of the soil profile and groundwater conditions across the site. No amount of detailed laboratory testing or sophisticated analysis can compensate for such knowledge" (Burland *et al.*, 1977).

2.8. Professional communications

"Let us not make the mistake of speaking within our closed circle, to ourselves; it is to our clients that we must speak., and convincingly we must have the courage to separate some of the adulterated data that most often surround us" (de Mello, 1983).

2.9. Case histories

"Although we emphasize the importance of analysing case histories, in order to avoid chaotic conclusions, or conclusions dominated by subjective and/or wishful thinking, it is even more important to run such case history back-analyses objectively, expurgating the inexorable subjective and deterministic reasonings" (de Mello, 1983).

2.10. The failings of contemporary civil engineering education

"Am I becoming old and grouchy when I complain that universities are no longer producing the civil geotechnical engineers, but mostly young technocrats who are absolutely sure of their theories, and armed with computers, absolutely sure of their numbers, to several decimal places?" (de Mello, 1985).

2.11. Specifications

"It is fundamental to reject once and for all the often cited, and even lauded, method specification. It is illogical.

The only valid principle acceptable is the end product specification". (de Mello, 2000).

2.12. Lack of proper progress in geotechnical engineering

De Mello was passionate about the folly of pursuing unnecessary refinements that did not lead to material progress, but rather to the perpetuation of irrelevant problem-solving. The following quotation expresses very clearly his frustrations with the perceived lack of direction in progressing geotechnical engineering.

"For better setting our line of sight, it is imperative that we keep revising our origins and reappraising our goals of service to society. We move imperceptibly from finding adequate solutions to significant problems, to seeking illusory refinements of solutions, to finding problems in solution, and to seeking problems in problems. *Quo Vadis, Geotecnica*?" (de Mello, 1995).

3. Foundation Design Principles and Criteria

3.1. Introduction

De Mello thought carefully and critically about commonly used design principles, design methods and design criteria. As mentioned in Section 2.3 above, he was particularly hostile to the unthinking acceptance of the provisions of codes and standards that contained criteria that were excessively conservative or that were not consistent with practical experience. Some of his views on design principles and design criteria are summarized below, together with later views by other authors.

3.2. Design principles

A paper published in the Salas memorial volume in 2000 set out de Mello's views on the shortcomings of common design practices for piling. Among the issues upon which he commented were the following:

• The lack of benefit granted by codes to the design of pile groups, in comparison with single piles.

• The probability of failure or unsatisfactory behaviour decreased greatly with large groups, yet this was not taken into account in the codes of which he was aware. He therefore urged "earnest reconsideration of the historical arbitrary fixed FS numbers".

• "A building's performance doesn't know whether it is founded on footings, piles, piers or rafts; why is it that the settlement-limited codified prescriptions are so much tighter for piles than for footings?"

• Whatever the desirability may be, in 99% of practical cases, prior testing of preliminary piles is not feasible, in contrast to the recommendation of the ISSMFE subcommittee

• The standards for pile load testing lack rationality in the specified testing procedures. It is not necessary to wait for settlements to stabilize beyond the working dead load, as the emphasis is then on the pile capacity and checking the factor of safety. Accordingly, it would be more rational to employ a constant, and rapid, rate of penetration test, rather than a conventional incremental loading test.

It is clear that de Mello was greatly concerned about the lack of rationality of foundation design methods, and in particular, the rather ad-hoc choices that designers made for the factor of safety against failure. The following section describes one attempt to place this issue on a more rational and logical basis.

3.3. The de Mello principles applied in a design code

The recently – released Australian Piling Code, AS2159-2009, incorporates a risk assessment procedure for obtaining the partial factor of safety (or its reciprocal, the geotechnical strength reduction factor) when designing piles against failure. This code adopts a limit state approach, and the key requirement for the ultimate limit state (*i.e.* the design against geotechnical failure) requires the following condition to be satisfied:

$$R_{d,g} \geq E_d \tag{1}$$

where $R_{d,g}$ = design geotechnical strength of the pile and E_d = design action effect, *i.e.* the factored-up load combination.

$R_{d,g}$ is computed as follows:

$$R_{d,g} = \gamma_g R_{d,ug} \tag{2}$$

where $R_{d,ug}$ = ultimate geotechnical strength (capacity) of pile and γ_g = geotechnical reduction factor.

The geotechnical reduction factor is given by:

$$\gamma_g = \gamma_{gb} + (\gamma_{tf} - \gamma_{gb})K \geq \gamma_{gb} \tag{3}$$

where γ_{gb} = basic geotechnical strength reduction factor; γ_{tf} = intrinsic test factor: 0.9 for static load testing, 0.75, for rapid load testing, 0.8, for dynamic load testing of preformed piles, 0.75, for dynamic load testing of other than preformed piles, 0.85, for bi-directional load testing, and γ_{gb}, for no testing; K = testing benefit factor: $1.33p/(p + 3.3) \leq 1$, for static or rapid load testing, $1.13p/(p + 3.3) = 1$, for dynamic load testing, and p = percentage of the total piles that are tested and meet the specified acceptance criteria

The basic geotechnical strength reduction factor (γ_{gb}) is calculated using the following risk assessment procedure:

(a) Each risk factor shown in Table 1 is rated by the designer on a scale from 1 to 5 for the nature of the site, the available site information and the pile design and installation procedures adopted. This will produce an individual risk rating *(IRR)* according to the level of risk assessed by the designer, as set out in Table 2.

(b) The overall design average risk rating *(ARR)* is obtained using the weighted average of the product of all of the risk weighting factors (w_i) shown in column 2 of Table 2, times the relevant individual risk rating *(IRR)*, as follows:

$$ARR = \frac{\sum w_i IRR_i}{\sum wI} \tag{4}$$

(c) The basic geotechnical strength reduction factor (γ_{gb}) is then obtained from Table 3, depending on the level of redundancy in the piling system. Systems with a high degree of redundancy would include large pile groups under large caps, piled rafts and pile groups with more than 4 piles. Systems with a low level of redundancy would in-

Table 1 - Weighting factors and individual risk ratings for risk factors (AS2159-2009).

Risk factor	Weighting factor (w_i)	Typical description of risk circumstances for individual risk rating (IRR)		
		1 (Very low risk)	3 (Moderate)	5 (Very high risk)
Site				
Geological complexity of site	2	Horizontal strata, well-defined soil and rock characteristics	Some variability over site, but without abrupt changes in stratigraphy	Highly variable profile or presence of karstic features or steeply dipping rock levels or faults present on site, or combinations of these
Extent of ground investigation	2	Extensive drilling investigation covering whole site to an adequate depth	Some boreholes extending at least 5 pile diameters below the base of the proposed pile foundation level	Very limited investigation with few shallow boreholes
Amount and quality of geotechnical data	2	Detailed information on strength compressibility of the main strata	CPT probes over full depth of proposed piles or boreholes confirming rock as proposed founding level for piles	Limited amount of simple in situ testing (*e.g.*, SPT) or index tests only
Design				
Experience with similar foundations in similar geological conditions	1	Extensive	Limited	None
Method of assessment of geotechnical parameters for design	2	Based on appropriate laboratory or in situ tests or relevant existing pile load test data	Based on site-specific correlations or on conventional laboratory or in situ testing	Based on non-site-specific correlations with (for example) SPT data
Design method adopted	1	Well-established and soundly based method or methods	Simplified methods with well-established basis	Simple empirical methods or sophisticated methods that are not well established
Method of utilizing results of in situ test data and installation data	2	Design values based on minimum measured values on piles loaded to failure	Design methods based on average values	Design values based on maximum measured values on test piles loaded up only to working load, or indirect measurements used during installation, and not calibrated to static loading tests
Installation				
Level of construction control	2	Detailed with professional geotechnical supervision, construction processes that are well established and relatively straight forward	Limited degree of professional geotechnical involvement in supervision, conventional construction procedures	Very limited or no involvement by designer, construction processes that are not well established or complex
Level of performance monitoring of the supported structure during and after construction	0.5	Detailed measurements of movements and pile loads	Correlation of installed parameters with on-site static load tests carried out in accordance with this Standard	No monitoring

The pile design includes the risk circumstances for each individual risk category and consideration of all of the relevant site and construction factors.

clude isolated heavily loaded piles and piles set out at large spacings.

The approach is based on an earlier paper that developed a reliability-based approach to pile capacity design (Poulos, 2004). It is considered that the approach incorporates a number of the aspects of foundation design philosophy that de Mello advocated, including:

- Proper consideration of the various geotechnical risks involved, including the site conditions, the design process and the construction procedure.
- The application of engineering judgement by the designer.
- Allowance for the benefits of doing pile load testing to reduce uncertainties.

3.4. Foundation settlement criteria for design

In his State of the Art paper in 1969, de Mello had commented that "a great number of truly outstanding cases of buildings and other projects successfully designed on clays, under conditions so adverse as to challenge responsibility to the point of daring, attest to the fact that there has been a very considerable progress in the field."

The subsequent paper by Burland *et al.* (1977) was highly influential in promoting a more rational approach to design criteria in relation to allowable foundation movements, and furthering the profession's appreciation of the importance not only of the type of structure, but also of the nature of the deformations. For example, following on the work of Burland & Wroth (1974), the 1977 paper emphasized that brick walls subjected to "hogging" deformations were more susceptible to damage than the same walls subjected to "sagging" movements.

They also summarized some of the available information relating building damage to foundation movements, including the following:

Skempton & MacDonald (1956) had recommended safe limits of total settlements of 40 mm for isolated foundations, and 40-65 mm for rafts, maximum differential settlements of 25 mm and a relative rotation (angular distortion) of 1/500. In sands, settlement takes place rapidly under load, and therefore these criteria may be conservative. Indeed, no cases of damage to buildings founded on sand had been reported up to that time.

Table 2 - Individual Risk Rating (IRR).

Risk level	Individual risk rating (IRR)
Very low	1
Low	2
Moderate	3
High	4
Very high	5

For buildings on isolated foundations on clay, some cases of slight damage had been reported for total settlements in excess of 150 mm and differential settlements in excess of 50 mm.

For buildings founded on rafts in clay, no damage had been reported for total settlements less than 250 mm and differential settlements less than 125 mm.

The movements quoted above are well in excess of the allowable values that are commonly adopted for foundation design, and prompted the authors to question "who is limiting the settlements and why."

More recent work by Zhang & Ng (2006) has confirmed that the conclusions reached by Burland *et al.* (1977), and their recommendations are summarized in Table 4. Even these recommendations may be somewhat conservative in light of the fact that a number of buildings in Frankfurt founded on piled rafts in clay have settled well in excess if 100 mm without any visible signs of distress.

4. Bearing Capacity of Shallow Foundations

4.1. Introduction

In his state-of-the art lecture at the 7th International Conference in 1969, de Mello introduced a degree of scepticism in relation to the theory of bearing capacity of a shallow foundation, and wrote as follows: "Notwithstanding the great importance of the determination of the ultimate bearing capacity of a foundation, it is evident that the theo-

Table 3 - Basic geotechnical strength reduction factor (γ_{gb}) for average risk rating.

Range of average risk rating (*ARR*)	Overall risk category	γ_{gb} for low redundancy systems	γ_{gb} for high redundancy systems
ARR ≤1.5	Very low	0.67	0.76
1.5 < *ARR* ≤2.0	Very low to low	0.61	0.70
2.0 < *ARR* ≤2.5	Low	0.56	0.64
2.5 < *ARR* ≤3.0	Low to moderate	0.52	0.60
3.0 < *ARR* ≤3.5	Moderate	0.48	0.56
3.5 < *ARR* ≤4.0	Moderate to high	0.45	0.53
4.0 < *ARR* ≤4.5	High	0.42	0.50
> 4.5	Very high	0.40	0.47

Table 4 - Suggested serviceability criteria for structures (Zhang & Ng, 2006).

Quantity	Value	Comments
Limiting tolerable Settlement mm	106	Based on 52 cases of deep foundations. Std. Deviation = 55 mm. Factor of safety of 1.5 recommended on this value
Observed intolerable Settlement mm	349	Based on 52 cases of deep foundations. Std. Deviation = 218 mm
Limiting tolerable angular distortion rad	1/500 1/250 (H < 24 m) 1/330 (24 < H < 60 m) 1/500 (60 < H < 100 m) 1/1000(H > 100 m)	Based on 57 cases of deep foundations. Std. Deviation = 1/500 rad From Chinese Code (MOC, 2002) H = building height
Observed intolerable angular distortion rad	1/125	Based on 57 cases of deep foundations. Std. Deviation = 1/90 rad

retical solutions to the problems are still subject to discussion, both in comparison between them, and in comparisons with controlled tests designed to check their validity". This scepticism proved to be well-founded, as subsequent work demonstrated significant dispersion of theoretical solutions and also disturbing differences between theoretical and measured behaviour. Some of these differences are discussed below.

4.2. Conventional theory

The traditional Terzaghi bearing capacity theory (Terzaghi, 1943) expresses the ultimate bearing capacity, q_u, of a shallow footing as follows:

$$q_u = c.\,N_c + 0.5\gamma B.N_\gamma + \gamma D.\,N_q \tag{5}$$

where c = soil cohesion, γ = soil unit weight, B = footing width, D = depth of embedment of base of footing below surface, N_c, N_γ and N_q are bearing capacity factors that depend on the angle of internal friction γ of the soil.

Terzaghi derived the bearing capacity factors from a limit equilibrium analysis. Subsequently, Davis & Booker (1971) obtained solution for the bearing capacity factors N_c, N_γ and N_q from plasticity theory, and compared these with the traditional Terzaghi theory. As shown in Fig. 1, the Terzaghi theory overestimates the bearing capacity factors considerably as compared with the more rigorous

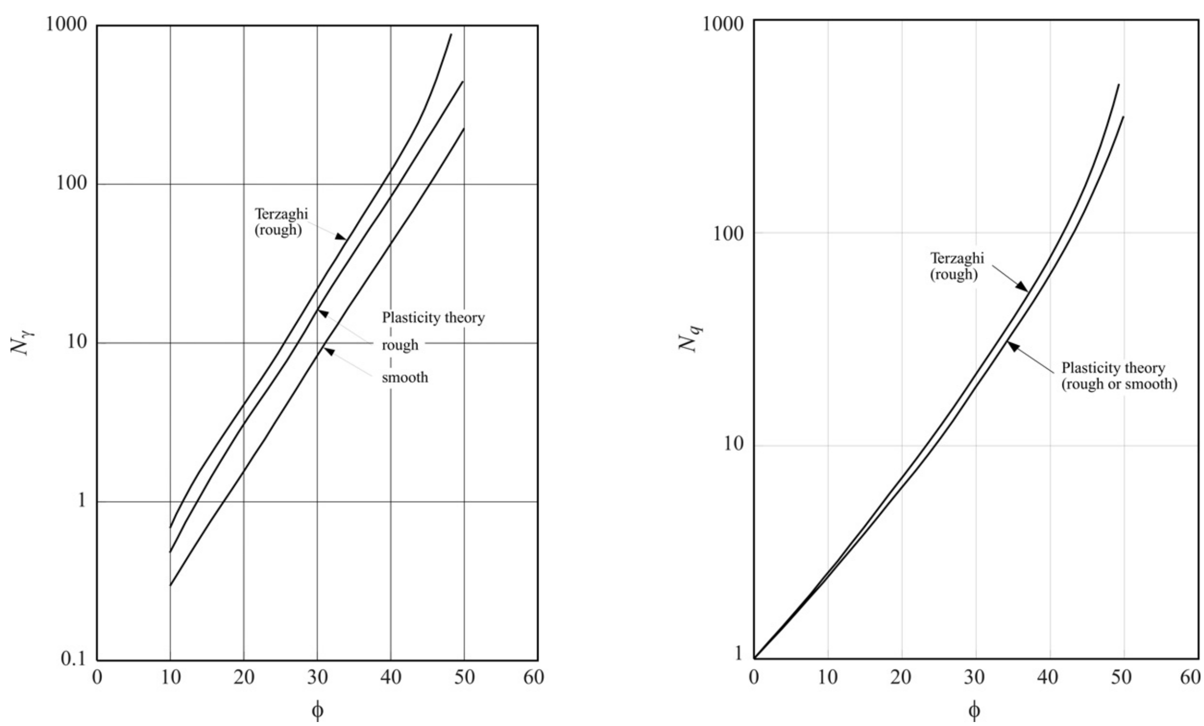


Figure 1 - Comparison between Terzaghi (1943) and Davis & Booker (1971) solutions for shallow footing bearing capacity.

plasticity solutions of Davis & Booker, with the difference being particularly marked for the factor N_γ for a smooth footing.

The superposition of the three components of bearing capacity in Eq. (3) has been recognised as being an approximation and Poulos *et al.* (2001) point out that the highly non-linear behaviour of real soils may mean that the superposition is at best approximate. They also note that while the traditional bearing capacity approach is based on plasticity theory, there is a significant amount of empiricism to allow for practical complicating factors that make a rigorous solution intractable or very difficult to obtain, for example, the effects of footing shape, load inclination, and soil surface inclination.

4.3. Effects of soil compressibility

A further issue was raised by Vesic (1973) who demonstrated the critical importance of soil compressibility in determining foundation bearing capacity. While the traditional bearing capacity theories for a rigid plastic material might be satisfactory for stiff clays under undrained conditions, they could seriously over-predict the bearing capacity of footings on relatively compressible soils such as loose calcareous sediments. Vesic introduced compressibility correction factors for the traditional bearing capacity factors that were a function of the rigidity index I_r, defined as follows:

$$I_r = G / (c + q.\tan\gamma) \tag{6}$$

where G = soil shear modulus, c = cohesion, q = vertical pressure, γ = angle of internal friction.

Terzaghi had in fact recognised this shortcoming in describing the mechanism of "local shear" failure for compressible sands. He recommended that, in such cases, a reduced angle of friction of about 2/3 of the actual friction angle be employed. An illustrative case of the importance of soil compressibility was presented by Poulos & Chua (1985) who compared the bearing capacity of a shallow circular model footing on silica sand and then the same footing on calcareous sand. The calcareous sand had a much greater compressibility, as indicated by the load-settlement curves in Fig. 2. Figure 3 shows the measured bearing capacity as a function of the relative density of the soil. The more compressible calcareous soil has a markedly smaller bearing capacity than the silica sand at the same relative density.

Figures 4 and 5 compare the measured bearing capacities with three different computed values:

• That computed from Terzaghi's conventional rigid plastic theory (general shear), using the measured angle of internal friction;

• That computed from Terzaghi's bearing capacity, using a friction angle reduced to 2/3 of the measured value;

• That computed from cavity expansion theory.

These comparisons show that, for both the silica sand and the calcareous sand, Terzaghi's conventional theory significantly over-estimates the bearing capacity, whereas the latter two methods of calculation give a more satisfactory level of agreement with the measurements.

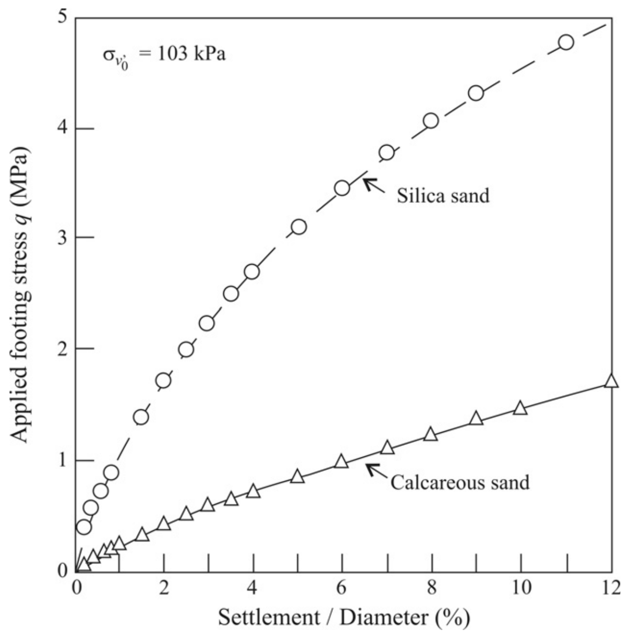


Figure 2 - Load-settlement curves for model footing on silica sand and calcareous sand (Poulos & Chua, 1985).

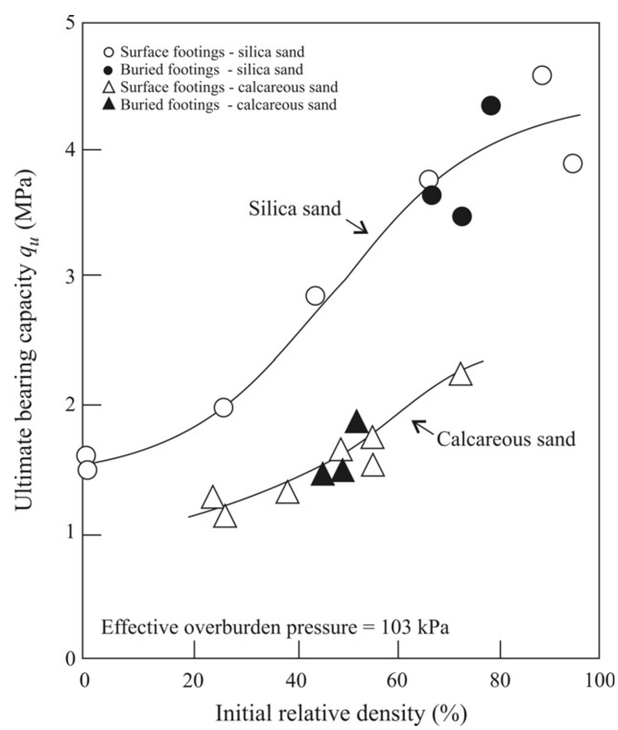


Figure 3 - Bearing capacity of model footings on silica sand and calcareous sand (Poulos & Chua, 1985).

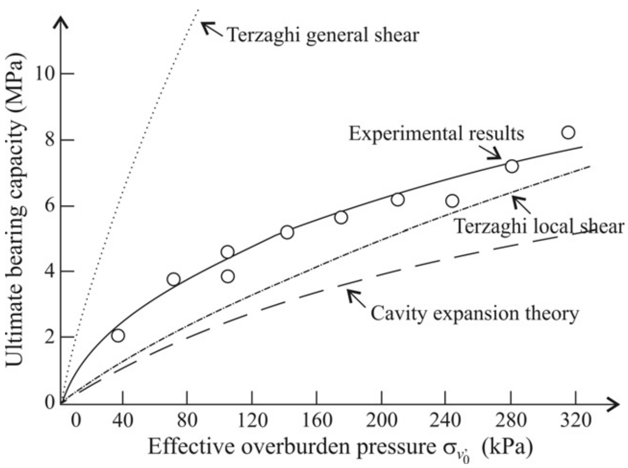


Figure 4 - Comparison between measured and calculated bearing capacity of footing on silica sand (Poulos & Chua, 1985).

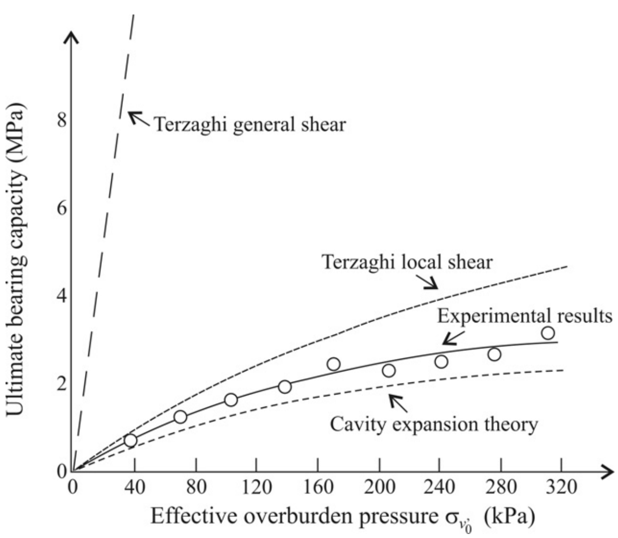


Figure 5 - Comparison between measured and calculated bearing capacity of footing on calcareous sand (Poulos & Chua, 1985).

4.4. Combined vertical, lateral and moment loadings

The Terzaghi equation does not directly consider the effects of horizontal or moment loading, and is confined to purely vertical applied load on a shallow footing. A variety of approximations have been developed to cater for combined loading, and a review of some of these was made by Poulos *et al.* (2001). An equation describing the failure locus in terms of all three components of the load was proposed by Taiebat & Carter (2000a) and was expressed as follows:

$$f=\left(\frac{V}{V_u}\right)^2+\left(\frac{M}{M_u}\left(1-\alpha_1\,\frac{H\,M}{H_u\,|M|}\right)\right)^2+\left|\left(\frac{H}{H_u}\right)^3\right|-1=0 \quad (7)$$

where V_u, M_u and H_u are the ultimate vertical, moment and horizontal load capacities of the footing, and α_1 is a factor that depends on the soil profile.

For a homogeneous soil, a value of $\alpha_1 = 0.3$ provides a good fit to the bearing capacity predictions from the numerical analysis. The three-dimensional failure locus described by Eq. (7) will not tightly match the numerical predictions over the entire range of loads, especially around the abrupt changes in the failure locus that occur when the horizontal load is high. However, overall the approximation is satisfactory, conservative and sufficient for many practical applications.

For a footing subjected to eccentric vertical loading, there is no exact expression to evaluate the effects of eccentricity of the load applied to a foundation. However, the effective width method is commonly used in the analysis of foundations subjected to eccentric loading (*e.g.*, Vesic, 1973; Meyerhof, 1951, 1953). In this method, the bearing capacity of a foundation subjected to an eccentrically applied vertical loading is assumed to be equivalent to the bearing capacity of another foundation with a fictitious effective area on which the vertical load is centrally applied.

Comparisons presented by Poulos *et al.* (2001) showed that the effective width method, commonly used in the analysis of foundations subjected to eccentric loading, provides good approximations to the collapse loads, and that its continued use in practice therefore appears justified.

4.5. Differences between theory and experiment

According to the classical bearing capacity theory, the bearing capacity q_u of a footing of width B on the surface of a soil layer with zero cohesion is given by:

$$q_u=\gamma B N_\gamma/2 \quad (8)$$

where γ = soil unit weight and N_γ = bearing capacity factor depending on the friction angle γ.

This equation implies that the larger the footing width B, the larger is the unit bearing capacity q_u. Unfortunately, there is now considerable evidence that demonstrates that this theoretical conclusion is not borne out in practice. For example, Decourt (2008) has re-plotted data from tests on footings of various sizes and found that, when normalized with respect to settlement/diameter (S/B), the load-settlement curves are unique and not dependent on the footing size nor on the relative density (Fig. 6). Similar conclusions have been reached from recent centrifuge test carried out on model footings by Gavin *et al.* (2009). In Fig. 7, the ratio of bearing pressure to cone resistance is plotted against S/B, and again, a relatively unique relationship is derived, regardless of footing size (for prototype footings ranging between 1 and 3 m in width). Akbas & Kulhawy (2009) have arrived at similar conclusions to those of Decourt and Gavin *et al.*

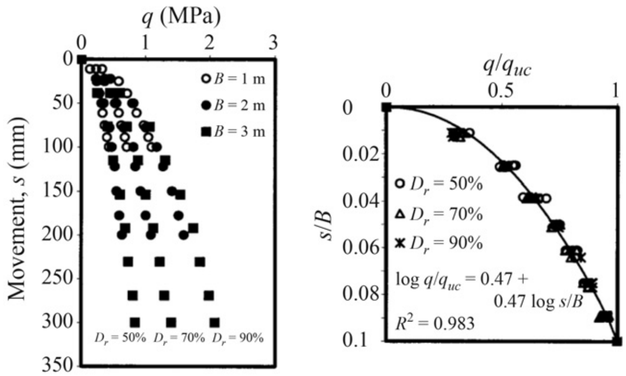


Figure 6 - Load-settlement curves for footings of various diameter on sand (Decourt, 2008).

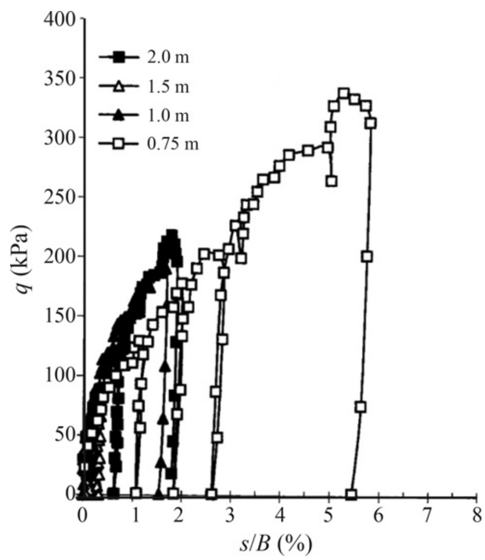


Figure 7 - Load-settlement curves for footings on sand at shenton park, site C (Gavin *et al.*, 2009).

4.6. Summary

De Mello's doubts in 1969 regarding the applicability of Terzaghi's theory to practice appear to have been well-founded. Experience now demonstrates that:

• The original Terzaghi bearing capacity factors were not entirely accurate;

• Soil compressibility plays a major role in bearing capacity and the use of the original rigid plastic theory may tend to overestimate bearing capacity significantly for granular soils.

• The "N_γ" term in the Terzaghi bearing capacity equation implies that the bearing capacity of a surface footing increases in proportion to its size. However, this does not appear to be the case in reality.

It is interesting to note that, 31 years after his 1969 classic paper, de Mello bemoaned the persistent adherence by the geotechnical profession to conventional bearing capacity theories, as follows: "My questions and objections to be raised in these matters are unfortunately repeated from a distant candid outcry (de Mello, 1969). I appeal for an unabashed abandonment of plasticity theory solutions, their postulates and results to be courageously recanted".

5. Axial Load Capacity of Pile Foundations

5.1. Introduction

The 1969 General Report by de Mello highlighted a number of important issues that were emerging in relation to the axial load capacity of piles. These issues included the following:

• The pile installation method can have a significant effect on the axial capacity;

• The displacement required to mobilize the ultimate shaft resistance is independent of pile diameter, whereas that required to mobilize the base resistance is roughly proportional to pile diameter.

• The ultimate skin friction of piles in sand does not increase linearly with depth, as would be inferred from conventional methods of calculation. Rather, the work of Vesic (1965) indicated that a limiting average skin friction would be reached at some depth, typically 10-20 diameters.

• The shaft friction in compression is different from that in tension, which had been frequently overlooked in attempts to establish skin friction values from field tests.

• It is desirable to develop a load-settlement curve for a pile, not only an estimate of the ultimate load. Emerging methods of analysis, such as those published by Seed & Reese (1955) and Poulos & Davis (1968) were mentioned.

For piles in clay, de Mello reproduced data from Kerisel (1965) that related the ratio of ultimate skin friction (f_s) to undrained shear strength (s_u), as a function of s_u. This ratio (which de Mello referred to as β but is more commonly given the symbol α) was recognized by de Mello as "a rough indication which must be subject to "a considerable latitude of judgement".

De Mello concluded that there was a need to develop improved approaches to the estimation of pile shaft friction in place of the rudimentary methods existing at that time. Some of these developments are outlined below.

5.2. Methods of estimation of pile shaft friction

5.2.1. Total stress approach

One of the traditional methods of estimating the ultimate shaft friction in compression, f_s, involves the use of the total stress ("alpha") method for piles in clay soils. This method relates f_s to the undrained shear strength s_u as:

$$f_s = \alpha s_u \tag{9}$$

where α = adhesion factor.

Poulos *et al.* (2001) summarize several approaches for assessing the adhesion factor α, most of which involve relating α to s_u; for example, Kulhawy & Phoon (1993) suggest the following approximation:

$$\alpha = 0.21 + 0.26\,(p_a/s_u)\,(\leq 1.0) \tag{10}$$

where p_a = atmospheric pressure.

It must be admitted that relatively limited progress has been made with total stress approaches since de Mello's report, the possible exception being the approach developed by Fleming *et al.* (1992) in which α is related not to s_u but to the ratio of undrained shear strength to vertical effective stress, s_u/σ_v':

$$\alpha = (s_u/\sigma_v)^{0.5}\,(s_u/\sigma_v)^{-0.5} \text{ for } (s_u/\sigma_v') \leq 1 \tag{11}$$

$$\alpha = (s_u/\sigma_v)^{0.5}\,(s_u/\sigma_v)^{-0.25} \text{ for } (s_u/\sigma_v') \geq 1 \tag{12}$$

5.2.2. Effective stress approaches

The effective stress ("beta") method can be applied for piles in any soil type. f_s is related to the in-situ effective stresses as follows:

$$f_s = K_s \tan\delta\,\sigma'_v \tag{13}$$

where K_s = lateral stress coefficient; δ = pile-soil friction angle; σ'_v = effective vertical stress at level of point under consideration.

Several of the more recent effective stress methods have employed cavity expansion theories in an attempt to model the effects of installation and subsequent loading of the pile (for example, Randolph *et al.*, 1979; Carter *et al.*, 1979b). While the results of such studies have been illuminating and have indicated the important effects of initial installation and subsequent dissipation of excess pore pressures, they appear to have had relatively little impact on design practice, due largely to the need to have reasonably detailed knowledge of the initial stress conditions within the soil, as well as the soil strength and compressibility characteristics. A detailed and intensive discussion of effective stress approaches to estimating the ultimate shaft friction is given by O'Neill (2001).

An alternative approach has been adopted by a number of researchers, in which attempts have been made to develop more reliable methods of estimating the lateral stress coefficient K_s. Notable among such methods is the approach of Jardine & Chow (1996), who have related K_s to the cone resistance, the distance from the pile tip, and the dilatant increase in normal stress during pile loading. Different expressions have been derived for driven piles in sand and clay soils, and the case of open-ended piles has also been considered. These expressions have been based on carefully instrumented pile data and a close appreciation of the fundamental behaviour of soils and pile-soil interfaces. Alternative methods have been developed more recently and these are summarised conveniently by Seo *et al.* (2009). Most of these recent methods have been developed for the offshore industry and involve the use of data from cone penetration testing. Where such data are available, comparisons between measured and computed shaft friction values indicate more satisfactory agreement than with the earlier procedures.

Seo *et al.* (2009) have presented an interesting comparison of the computed shaft capacities for an H-pile in a layered soil profile consisting of interbedded clays, silts and sands. The comparison is shown in Table 5, together with the measured shaft capacity. The computed values are for the assumption that the friction is mobilized around the outer shaft perimeter, rather than around the full interface contact perimeter. It can be seen that five of the seven methods considered tend to over-estimate the shaft capacity, and that there is a factor of almost 3 between the largest and smallest estimates of capacity.

Thus, despite almost 40 years of research and application, there is still great uncertainty in predicting the shaft capacity of a single pile in a realistic layered soil profile.

In addition, a number of issues raised by de Mello in 1969 still remain to be clarified for practical pile designers in relation to the ultimate shaft friction on piles. Such issues include the following:

- Does a limiting value of f_s actually exist, especially for piles in sandy soils?
- How does the value of f_s in uplift compare to the value of f_s for compression?
- Can laboratory testing be used to provide a more reliable estimate of f_s?

The results of recent research over the past decade or so can shed some light on these issues.

5.3. Limiting f_s values for piles in sandy soils

The concept of limiting ultimate shaft resistance in sandy soils was developed by Kerisel (1961), Vesic (1967) and BCP (1971). It arose from tests on instrumented piles in which it appeared that the average ultimate shaft friction reached a limiting value for depths in excess of between 5 and 20 pile diameters from the top of the pile. This was attributed to an arching phenomenon around the shaft, and led to the adoption of a practice of specifying limiting f_s val-

Table 5 - Measured and computed shaft capacities for an H-pile in layered soil (after Seo *et al.*, 2009).

Prediction method	Shaft capacity (kN)
Fleming *et al.* (1992) & API (1993)	1314
Foye *et al.* (2009) and API (1993)	1724
Aoki & Velloso (1975) – SPT	1179
Aoki & Velloso (1975) – CPT	868
Bustamante & Gianeselli (1982) – CPT	638
NGI (Clausen *et al.*, 2005) – CPT	1281
ICP (Jardine *et al.*, 2005) - CPT	1228
Measured value	1053

ues in design (*e.g.*, Vesic, 1969; Meyerhof, 1976; Poulos & Davis, 1980).

The existence of such a limiting value has been questioned critically by a number of authors subsequently (*e.g.*, Kulhawy, 1984; Fellenius, 1984). The apparent limiting values of f_s have been attributed to at least two factors:

- The existence of residual stresses in the piles before the measurements of shaft resistance were made. This leads to the shaft friction in the lower part of the pile appearing to be lower than the true value;

- The overconsolidation of the soil near the surface, which gives rise to higher values of in-situ lateral stress, and hence values of shaft resistance. The effects of overconsolidation become less with increasing depth, and hence the rate of increase of shaft resistance with depth becomes less.

Attempts to reproduce theoretically the apparent limiting shaft friction have been unsuccessful, although a reduction in the rate of increase of shaft resistance has been obtained by consideration of the effects of compressibility of the soil, and the reduction of the soil friction angle (and hence the interface friction angle) with increasing effective pressure and depth.

The conclusion to be drawn from research into this aspect is that a limiting value of f_s probably does not exist, although the rate of increase of f_s with depth is not linear. However, from the viewpoint of practical design, the adoption of a suitable limiting value of f_s is a conservative approach which at least avoids predicting unrealistically large shaft friction values at great depths within a sandy soil.

5.4. Shaft resistance in uplift and compression

It is generally accepted that the uplift shaft resistance for piles in clay is similar to that for compressive loading. However, there is conflicting evidence in relation to piles in sand, with some early researchers indicating similar values for both compression and uplift, while others found the values in uplift to be less than in compression.

A significant advance in understanding of this problem was made by de Nicola & Randolph (1993) who showed that the ratio of the uplift resistances in uplift and compression, f_{su}/f_{sc}, was dependent on the relative compressibility of the pile, via the Poisson effect. The relationship they derived is as follows:

$$\frac{f_{su}}{f_{sc}} = \left\{1 - 0.2\log_{10}\left[\frac{100}{L/d}\right]\right\}(1 - 8\eta - 25\eta^2) \quad (14)$$

where L = pile length; d = pile diameter; η = dimensionless compressibility factor = $\nu_p.\tan\sigma.(L/d).(G_{av}/E_p)$; ν_p = pile Poisson's ratio; σ = pile-soil interface friction angle; G_{av} = average soil shear modulus along pile shaft; E_p = Young's modulus of pile material. For piles in medium dense to dense sands, this ratio typically ranges between 0.7 and 0.9, but tends towards unity for relatively short piles.

5.5. Use of laboratory testing for f_s

It has generally been accepted by practitioners that there is no suitable laboratory test which can be used reliably to measure the ultimate shaft friction f_s. However, there has been a significant development over the past 10-15 years in direct shear testing of interfaces, with the development of the "constant normal stiffness" (CNS) test (Ooi & Carter, 1987; Lam & Johnston, 1982). The basic concept of this test is illustrated in Fig. 8, and involves the presence of a spring of appropriate stiffness against which the normal stress on the interface acts. This test provides a closer simulation of the conditions at a pile-soil interface than the conventional constant normal stress direct shear test. The normal stiffness K_n can be "tuned" to represent the restraint of the soil surrounding the pile, and is given by:

$$K_n = \frac{4G_s}{d} \quad (15)$$

where G_s = shear modulus of surrounding soil; d = pile diameter.

The effects of interface volume changes and dilatancy can be tracked in a CNS test, and the results are particularly enlightening when cyclic loading is applied, as they demonstrate that the cyclic degradation is due to the reduction in normal stress arising from the cyclic displacements applied to the interface.

Some success has been achieved in applying CNS testing to the estimation of skin friction f_s for large diameter piles in Middle East soft carbonate rocks. Figure 9 shows comparisons between values of ultimate static shaft friction from CNS tests and measured mobilized values of shaft friction from full-scale pile load tests for the Emirates Towers (Poulos & Davids, 2005). There is a tendency for the CNS data to be somewhat higher than the measured mo-

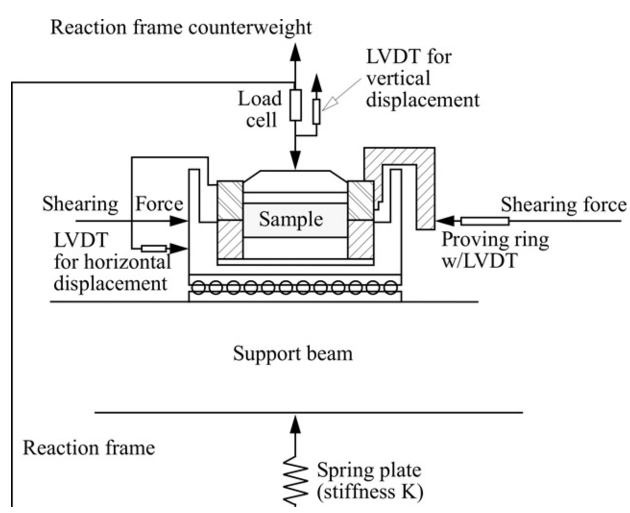


Figure 8 - Constant normal stiffness direct shear apparatus (Tabucanon *et al.*, 1995).

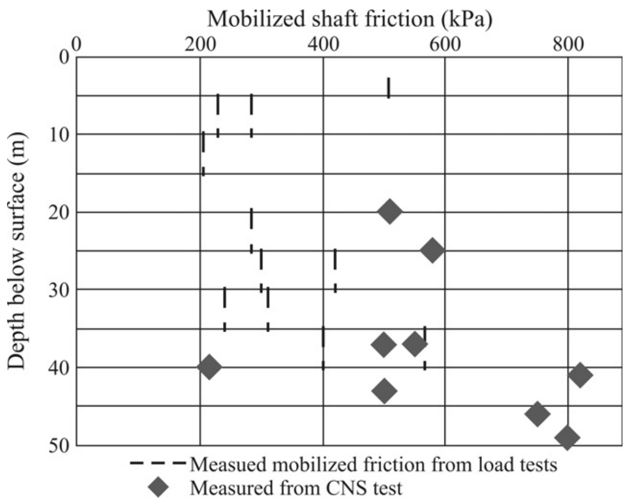


Figure 9 - Shaft friction data from Emirates project, Dubai (after Poulos & Davids, 2005).

bilized values, but it must be pointed out that the full pile capacity had not been mobilized when the maximum test load was reached. Hence, the actual ultimate shaft friction values may well have been similar to those measured from the CNS testing. In any case, as a consequence of both the laboratory testing and the subsequent pile load tests, the design values of shaft friction were increased considerably over the values that had previously been adopted in Dubai.

5.6. Methods of estimation of pile end bearing

In the total stress approach, the ultimate end bearing resistance f_b is given by:

$$f_b = N_c \; s_u \tag{16}$$

where N_c = bearing capacity factor.

This approach is almost universally used for piles founded in clay, but clearly is inapplicable to piles founded in granular materials or rock. For piles in granular materials, or for long-term bearing capacity generally, an effective stress approach must be used, and the following approximate relationship is commonly adopted:

$$f_b = \sigma_v' . N_q \tag{17}$$

where σ_v' = vertical effective stress at level of pile base and N_q = bearing capacity factor.

Figure 10 reproduces a figure that appeared in the classic text by Lambe & Whitman (1969) and demonstrated an alarming spread of theoretical solutions for the bearing capacity factor N_q for deep foundations. For a typical angle of internal friction of 35 degrees, this factor could vary between about 53 and 380, depending on whose theory was employed. Perhaps as a consequence of this gross uncertainty with the theoretical basis of calculation, let alone the issue of appropriate geotechnical parameter selection, researchers have attempted to develop methods of end bearing capacity estimation that bypass the theory. A valuable

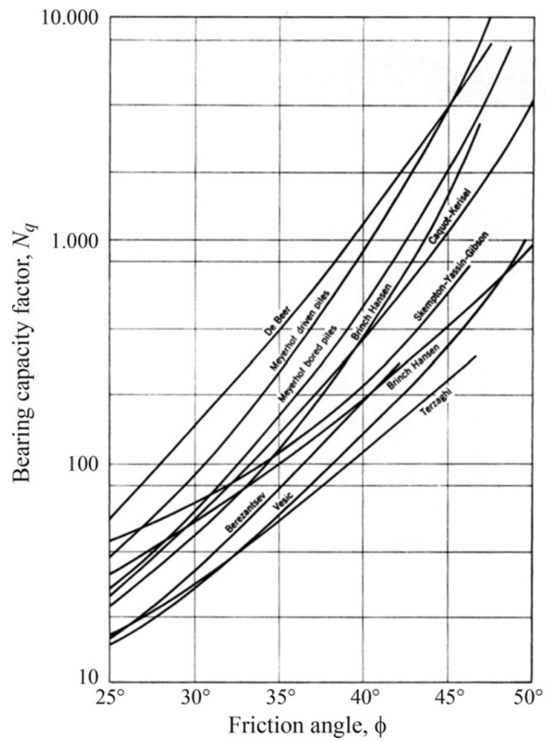


Figure 10 - Variability of theoretical solutions for bearing capacity factor N_q (Lambe & Whitman, 1969).

summary of some of these approaches is given by Seo *et al.* (2009), and again, many of these methods require cone penetration test (CPT) data.

For the same soil profile considered for shaft friction comparisons, Seo *et al.* (2009) compared the computed end bearing capacities from a number of methods for a steel H-pile, using the gross cross-sectional area of the pile in the calculations. Table 6 compares the computed end bearing values, and the measured value for a settlement of 10% of the equivalent pile diameter. It can be seen that there is a considerable scatter of the computed values and that most of the methods (except that of Jardine *et al.*, 2005) overestimate the end bearing capacity. Clearly, while there have been considerable advances in our understanding of the mechanics of pile-soil interaction, there is still a considerable uncertainty attached to our ability to predict the most fundamental characteristic of a pile, its ultimate axial load capacity.

5.7. Load-settlement curve estimation

5.7.1. Single piles

In1969 de Mello had commented on the need to develop methods of load-settlement estimation. Over the following four decades, some advances have been made in this regard, but it is interesting that the method of Seed & Reese (1955), utilizing the load transfer (or "t-z") curve concept, remains firmly embedded as one of the most commonly used approaches. Over, the past forty years, advances have

Table 6 - Measured and predicted ultimate base capacities (Seo *et al.*, 2009).

Prediction method	Base capacity using gross cross-sectional area (kN)
Fleming *et al.* (1992)	1409
Aoki & Velloso (1975) –SPT	1488
Aoki & Velloso (1975) – CPT	1306
Bustamante & Gianeselli (1982) – CPT	1260
NGI (Clausen *et al.*, 2005) - CPT	1096
Fugro (Kolk *et al.*, 2005) – CPT	1257
UWA (Lehane *et al.*, 2005) – CPT	1375
ICP (Jardine *et al.*, 2005) – CPT	853
Foye *et al.* (2009) - CPT	1204
Measured (at 10% base diameter settlement)	906

been made in the means of developing the "t-z" curves, progressing from the purely empirical methods of Coyle & Reese (1966), through the method published by Kraft *et al.* (1981) that utilized some aspects of elastic theory, to the relatively sophisticated approaches described by Randolph (2003) via his RATZ analysis. This program combines parabolic models for the shaft and base resistance responses with elastic compression of the pile, to compute the overall pile head load-settlement relationship for the pile. Figure 11 shows a satisfactory comparison between measured and predicted load-settlement behaviour for a single pile within a silty sand and sand site (Deeks *et al.*, 2005).

It is also possible to obtain good agreement between computed and measured load-settlement behaviour using a modified boundary element technique that utilizes elastic theory for the soil, but impose limiting values of shaft and base resistances, and assumed hyperbolic relationships between the local Young's modulus and local stress level. Figure 12 shows an example of a "Class A" prediction for a large diameter bored test pile for the Emirates twin tower project in Dubai, using the modified boundary element approach (Poulos & Davids, 2005). The agreement with the measured load-settlement behaviour is reasonably good, although the measured axial capacity and stiffness of the pile are clearly greater than those predicted from the design parameters.

A further development of elastic theory has been proposed by Mayne & Elkahim (2002) and Mayne & Zavala (2004), in which the elastic solutions for pile head settlement are combined with a modulus degradation function developed by Fahey & Carter (1993), namely:

$$E/E_0 = [1 - f(P/P_u)^g] \tag{18}$$

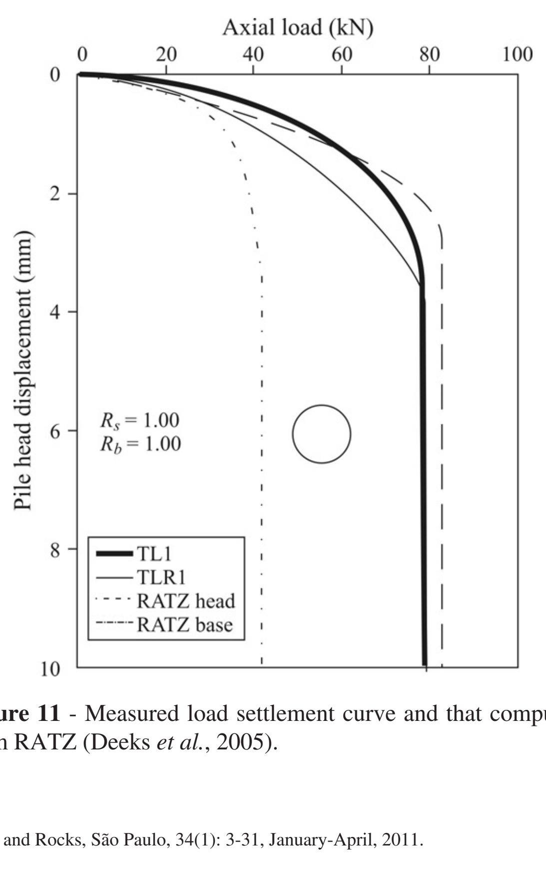


Figure 11 - Measured load settlement curve and that computed from RATZ (Deeks *et al.*, 2005).

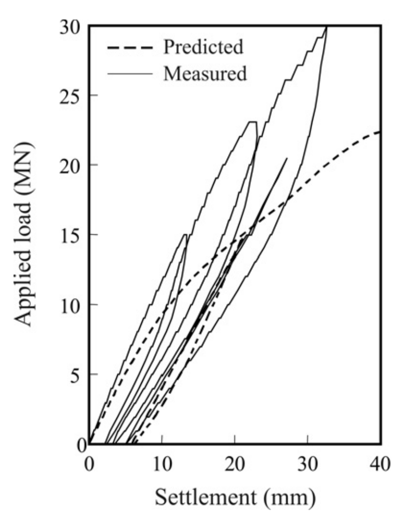


Figure 12 - Comparison between predicted and measured load-settlement behaviour for test pile at Emirates project, Dubai (Poulos & Davids, 2005).

where E_0 = small-strain Young's modulus, E = Young's modulus for an applied load P, P_u = ultimate axial load capacity, and f and g are parameters, generally taken as $f = 1$ and $g = 0.3$.

Figure 13 reproduces the measured and computed load-settlement curves for a case considered by Mayne & Elhakim (2002) in which the small-strain Young's modulus was derived from shear wave velocity measurements within the soil. The agreement can be seen to be very good, both for the overall load-settlement behaviour and for the individual shaft and base load versus settlement curves.

It appears that various methods of estimating the load-settlement behaviour of single piles have been developed since 1969, and that, provided appropriate values of pile shaft friction and end bearing, and soil stiffness, are used, these analyses can give a reasonable prediction of load-settlement behaviour.

5.7.2. Pile groups

In their 1977 state-of-the-art paper, Burland *et al.* (1977) commented that the settlement of pile groups was at that time commonly calculated from the assumption that end bearing piles are rigidly supported at the toe and that floating piles are rigidly supported at the centre of the lower third point. Since then, there have been significant developments in the prediction of the settlement of pile groups, and a number of methods are now available for practical application. A review of some of these methods has been made by Randolph (1994), Mandolini *et al.* (2005) and Poulos (2006), among others. In general, the prediction of pile group settlement is less satisfactory than for single piles, because pile group settlement is influenced not only by the shaft and base load transfer characteristics, but also by pile-soil-pile interaction, which is dependent on a number of factors, including pile spacing and configuration and the nature of the ground profile below the piles. An example of a satisfactory single pile settlement prediction, but an unsatisfactory pile group settlement prediction, is given by Poulos & Davids (2005).

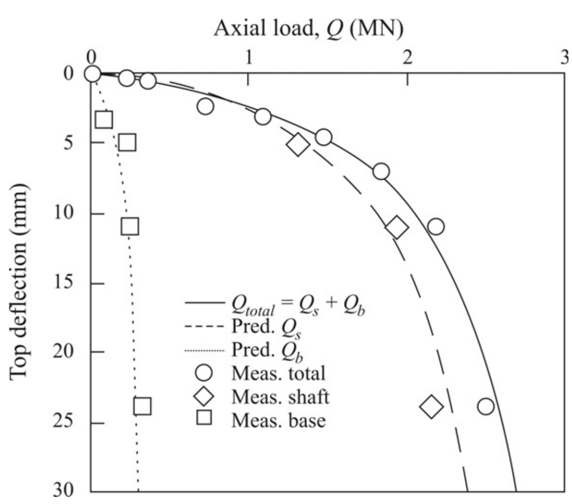


Figure 13 - Comparison between measured and calculated load-settlement behaviour for bored pile at Opelika site, Alabama (Mayne & Elhakim, 2002).

There is now an increasing tendency for full three-dimensional finite element analyses to be applied to pile group settlement problems. Thus, future advances may well require more focus on proper ground characterisation and soil modelling, than on the further development of numerical techniques themselves.

5.8. Summary

Considerable research has been carried out since 1969 to improve our ability to predict pile capacity and load-settlement behaviour. Regrettably, it is not possible to claim complete success in this endeavour, as the accurate prediction of axial pile capacity remains rather elusive, despite the increased understanding of pile-soil interaction and the increased sophistication of some of the more recent methods of calculation. While some success has been achieved in predicting the load-settlement behaviour of single piles, accurate prediction of the settlement of pile groups, particularly if the piles are floating, also remains elusive. Given the high degree of sophistication that it is now possible to bring to bear on pile prediction tasks, it appears likely that the lack of consistent success may be due more to the deficiencies in characterising the ground profile, than to deficiencies in the methods of calculation.

6. Settlement Reducing Piles

6.1. Introduction

Burland *et al.* (1977) drew attention to the concept of settlement reducing piles, and commented that it should be possible to carry a substantial part of the vertical load from a pile cap or raft in the soil between the piles. They emphasized that the number of piles required to reduce settlements to an acceptable level will often be relatively small and hence the spacing of the piles within a piled raft may be relatively large. The following quotation is still as relevant today as it was in 1977:

"Traditionally engineers engaged in a pile group design have asked themselves "How many piles are required to carry the weight of the building?" When settlement is the controlling factor in the choice of piles designers should perhaps be asking the question: 'How many piles are required to reduce the settlements to an acceptable amount?' The number of piles in answer to the second question is invariably less than in answer to the first question, provided it is accepted that the load carrying capacity of each pile will probably be fully mobilized".

In many countries today, pile group design is still governed by the first question, but increasingly it is recognized that the second question is now the key design issue. This section will review, relatively briefly, some of the developments in piled raft analysis and design that have occurred over the past 33 years, and will outline some cases in which the piled raft concept has been used successfully.

6.2. Foundation concept and alternative design philosophies

Piled raft foundations utilize piled support for control of settlements with piles providing most of the stiffness at serviceability loads, and the raft element providing additional capacity at higher load levels after the capacity of the piles has been fully utilized. A geotechnical assessment for design of such a foundation system therefore needs to consider not only the capacity of the pile elements and the raft elements, but their combined capacity and their interaction under serviceability loading.

Randolph (1994) has defined clearly three different design philosophies with respect to piled rafts:

• The "conventional approach", in which the piles are designed as a group to carry the major part of the load, while making some allowance for the contribution of the raft, primarily to ultimate load capacity.

• "Creep Piling", in which the piles are designed to operate at a working load at which significant creep starts to occur, typically 70%-80% of the ultimate load capacity. Sufficient piles are included to reduce the net contact pressure between the raft and the soil to below the preconsolidation pressure of the soil.

• Differential settlement control, in which the piles are located strategically in order to reduce the differential settlements, rather than to substantially reduce the overall average settlement.

In addition, there is a more extreme version of creep piling, in which the full load capacity of the piles is utilized, *i.e.* some or all of the piles operate at 100% of their ultimate load capacity. This gives rise to the concept of using piles primarily as settlement reducers, while recognizing that they also contribute to increasing the ultimate load capacity of the entire foundation system.

Clearly, the latter approaches are most conducive to economical foundation design. However, it should be emphasized that the design methods to be discussed allow any of the above design philosophies to be implemented.

Figure 14 illustrates, conceptually, the load-settlement behaviour of piled rafts designed according to the various strategies. Curve O shows the behaviour of the raft alone, which in this case settles excessively at the design load. Curve 1 represents the conventional design philosophy, for which the behaviour of the pile-raft system is governed by the pile group behaviour, and which may be largely linear at the design load. In this case, the piles take the great majority of the load. Curve 2 represents the case of creep piling where the piles operate at a lower factor of safety, but because there are fewer piles, the raft carries more load than for Curve 1. Curve 3 illustrates the strategy of using the piles as settlement reducers, and utilizing the full capacity of the piles at the design load. Consequently,

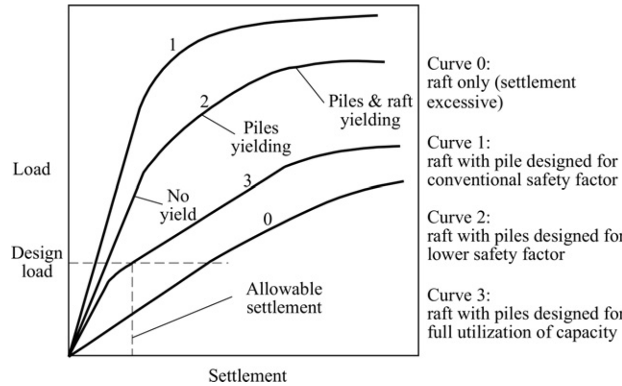


Figure 14 - Load – settlement curves for various piled raft design philosophies.

the load-settlement may be nonlinear at the design load, but nevertheless, the overall foundation system has an adequate margin of safety, and the settlement criterion is satisfied. Therefore, the design depicted by Curve 3 is acceptable and is likely to be considerably more economical than the designs depicted by Curves 1 and 2.

6.3. Favourable and less favourable circumstances for piled rafts

The most effective application of piled rafts occurs when the raft can provide adequate load capacity, but the settlement and/or differential settlements of the raft alone exceed the allowable values. Poulos (2001) has examined a number of idealized soil profiles, and found that the following situations may be favourable:

• Soil profiles consisting of relatively stiff clays

• Soil profiles consisting of relatively dense sands.

An example of the application of the piled raft concept in such circumstances was described by de Mello (1972) who developed a scheme for adding piles to control the differential settlement of a heavily loaded building.

Conversely, there are some situations which are less favourable, including:

• Soil profiles containing soft clays near the surface.

• Soil profiles containing loose sands near the surface.

• Soil profiles which contain soft compressible layers at relatively shallow depths.

• Soil profiles which are likely to undergo consolidation settlements.

• Soil profiles which are likely to undergo swelling movements due to external causes.

In the first two cases, the raft may not be able to provide significant load capacity and stiffness, while in the third case, long-term settlement of the compressible underlying layers may reduce the contribution of the raft to the long-term stiffness of the foundation. The latter two cases should be treated with caution. Consolidation settlements (such as those due to dewatering or shrinking of an active clay soil) may result in a loss of contact between the raft and the soil, thus increasing the load on the piles, and leading to increased settlement of the foundation system. In the case of swelling soils, substantial additional tensile forces may be induced in the piles because of the action of the swelling soil on the raft. Theoretical studies of these latter situations have been described by Poulos (1993) and Sinha & Poulos (1997).

6.4. Design issues and the design process

As with any foundation system, a design of a piled raft foundation requires the consideration of a number of issues, including:

1. Ultimate load capacity for vertical, lateral and moment loadings;
2. Maximum settlement;
3. Differential settlement;
4. Raft shears and moments, for the structural design of the raft;
5. Pile loads and moments, for the structural design of the piles.

In much of the available literature, emphasis has been placed on the bearing capacity and settlement under vertical loads. While this is a critical aspect, and is considered in detail herein, the other issues must also be addressed. In some cases, the pile requirements may be governed by the overturning moments and shear forces applied by wind loading, rather than the vertical dead and live loads.

It is suggested that a rational design process for piled rafts involves three main stages:

• A preliminary stage to assess the feasibility of using a piled raft, and the required number of piles to satisfy design requirements.

• A second stage to assess where piles are required and the general characteristics of the piles.

• A final detailed design stage to obtain the optimum number, location and configuration of the piles, and to compute the detailed distributions of settlement, bending moment and shear in the raft, and the pile loads and moments.

The first and second stages may involve relatively simple calculations which can usually be performed without a complex computer program. Poulos (2001) gives details of some methods that may be employed for each of the above design stages.

Once the preliminary stage has indicated that a piled raft foundation is feasible, and an indication has been obtained of the likely piling requirements, it is necessary to carry out a more detailed design in order to assess the detailed distribution of settlement and decide upon the optimum locations and arrangement of the piles. The raft bending moments and shears, and the pile loads, should also be obtained for the structural design of the foundation.

The detailed stage will generally demand the use of a suitable computer program which accounts in a rational manner for the interaction among the soil, raft and piles. The effect of the superstructure may also need to be considered. Several methods of analyzing piled rafts have been developed, and some of these have been summarized by Poulos *et al.* (1997) and Mandolini *et al.* (2005). It has been found that, despite some differences among the various methods, most of those which incorporate nonlinear behaviour give somewhat similar results, although there are significant differences among the computed raft bending moments. However, it would appear that, provided the analysis method is soundly based and takes into account the limited load capacity of the piles, similar results may be expected for similar parameter inputs.

6.5. Some characteristics of piled raft behaviour

Poulos (2001) has examined some of the characteristics of behaviour of piled rafts and the effect of the following factors on this behaviour:

1. The number of piles
2. The nature of the loading (concentrated versus uniformly distributed)
3. Raft thickness
4. Applied load level.

The following important points have been noted for practical design:

• Increasing the number of piles, while generally of benefit, does not always produce the best foundation performance, and there is an upper limit to the number of piles, beyond which very little additional benefit is obtained.

• The raft thickness affects differential settlement and bending moments, but has little effect on load sharing or maximum settlement.

• For control of differential settlement, optimum performance is likely to be achieved by strategic location of a relatively small number of piles, rather than using a large number of piles evenly distributed over the raft area, or increasing the raft thickness.

• The nature of the applied loading is important for differential settlement and bending moment, but is generally not very important for maximum settlement or load-sharing between the raft and the piles.

A particularly interesting example demonstrating the "law of diminishing returns", as applied to piled raft foundations, is described by Mandolini *et al.* (2005). They examined the effects of reducing the number of piles for the foundation of a pier of the Garigliano bridge in Italy. The conventional design approach required the addition of 144 piles to satisfy bearing capacity requirements. However, they found that a very similar settlement performance could be obtained with a significantly smaller number of piles, as shown in Fig. 15. Both their computer analysis, utilizing the program NAPRA, and a simple hand calculation method (PDR) described by Poulos (2000) showed that the settlement of the piled raft (expressed in dimensionless form in terms of the settlement of the raft alone) would be virtually unaffected if the number of piles was halved to 72. There would also be virtually no change in the load sharing between the raft and the piles.

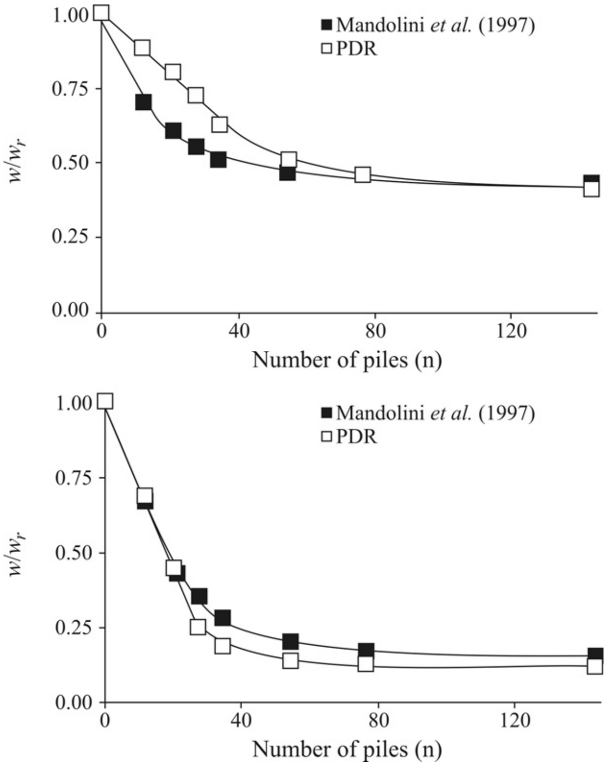


Figure 15 - The effect of number of piles on the relative settlement and load sharing (Mandolini et al., 2005).

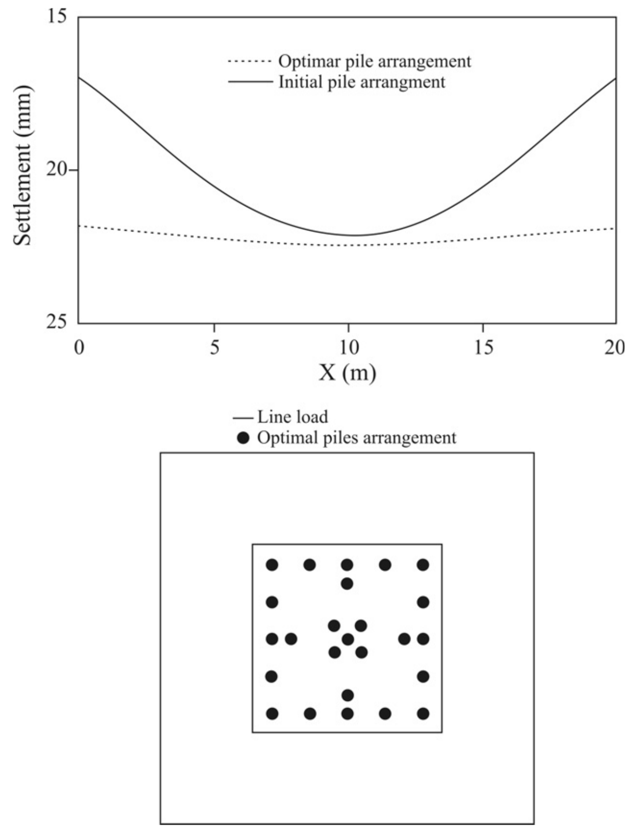


Figure 16 - The effect of pile configuration on the settlement profile below a piled raft (Fadaee & Rowhani, 2006).

It has been found that the performance of a piled raft foundation can be optimized by selecting suitable locations for the piles below the raft. In general, the piles should be concentrated in the most heavily loaded areas, while the number of piles can be reduced, or even eliminated, in less heavily loaded areas (Horikoshi & Randolph, 1998). An interesting example of pile location optimization is presented by Fadaee & Rowhani (2006), who considered a square raft with a square line load as shown in Fig. 16. The authors compared the computed distribution of settlement for two pile arrangements: 25 piles uniformly distributed across the raft, and the arrangement concentrated in the vicinity of the line load. This figure compares the computed settlement distributions, and clearly demonstrates a dramatic reduction in differential settlement with the latter pile arrangement.

Some useful further insights into piled raft behaviour have been obtained by Katzenbach *et al.* (1998) who carried out three-dimensional finite element analyses of various piled raft configurations. They used a realistic elasto-plastic soil model with dual yield surfaces and a non-associated flow rule. They analyzed a square raft containing from 1 to 49 piles, as well as a raft alone, and examined the effects of the number and relative length of the piles on the load-sharing between the piles and the raft, and the settlement reduction provided by the piles. An interaction diagram was developed, relating the relative settlement (ratio of the settlement of the piled raft to the raft alone) to the number of piles and their length-to-diameter ratio, *L/d*. For a given number of piles, the relative settlement was found to reduce as *L/d* increases. It was also found that there is generally very little benefit to be obtained in using more than about 20 piles or so, a conclusion which is consistent with the results obtained by Poulos (2001).

An interesting aspect of piled raft behaviour, which cannot be captured by simplified analyses, is that the ultimate shaft friction developed by piles within a piled raft can be significantly greater than that for a single pile or a pile in a conventional pile group. This is because of the increased normal stresses generated between the soil and the pile shaft by the loading on the raft. The results obtained by Katzenbach *et al.* (1998) indicate that the piles within the piled raft foundation develop more than twice the shaft resistance of a single isolated pile or a pile within a normal pile group, with the centre piles showing the largest values. Thus, the usual design procedures for a piled raft, which assume that the ultimate pile capacity is the same as that for an isolated pile, will tend to be conservative, and the ultimate capacity of the piled raft foundation system will be greater than that assumed in design.

6.6. Some applications of piled rafts

There are many examples of the successful use of piled rafts in practice, several of which are described in the book by Hemsley (2000). Some other cases are described briefly below.

6.6.1. Residential buildings, Sweden

An early case demonstrating the "law of diminishing returns" was provided by Hansbo (1983) who presented time-settlement curves for two similar buildings, one on 228 piles and the other on 104 piles. The first foundation system was designed as a conventional piled foundation while the second was designed using the "creep piling" concept of piled raft behaviour, as described by Burland *et al.* (1977). As shown in Fig. 17, the settlements of the two buildings were very similar, clearly indicating that the conventional design approach did not lead to any improvement in performance, despite it being more than double the cost of that using the creep piling concept.

6.6.2. Westendstrasse1, Frankfurt, Germany

The case of the Westendstrasse 1 building in Frankfurt was examined by Poulos *et al.* (1997). Figure 18 shows a plan of the tower and the 40 bored piles on which the tower was founded, and which supported an average applied pressure of about 323 kPa. Comparisons were made between the measured values of settlement and pile load, and those computed from a variety of methods, Fig. 19 shows these comparisons, from which the following conclusions can be drawn from this case:

• The measured maximum settlement is about 105 mm, and most methods tend to over-predict this settlement. However, most of the methods provide an acceptable design prediction.

• The piles carry about 50% of the total load. Most methods tended to over-predict this proportion, but from a design viewpoint, most methods give acceptable estimates.

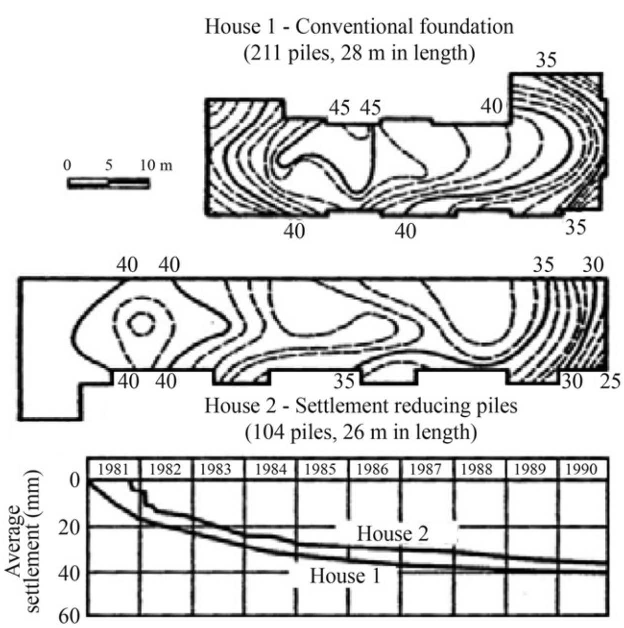


Figure 17 - Settlements for two adjacent residential buildings – (Hansbo, 1983).

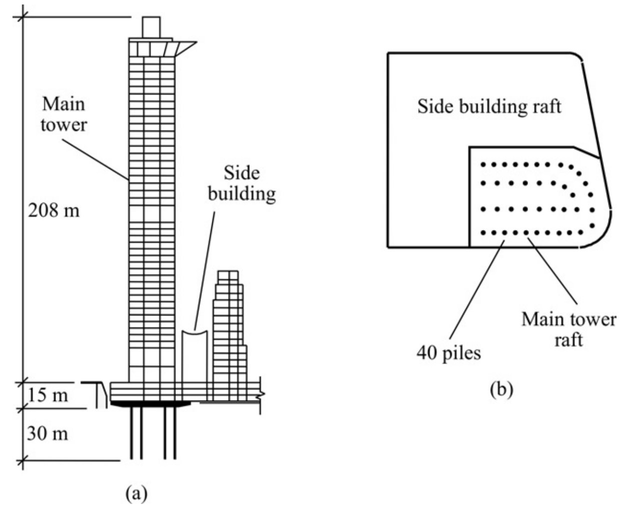


Figure 18 - Westendstrasse 1 building, Frankfurt, Germany (Franke *et al.*, 1994).

• All methods capable of predicting the individual pile loads suggest that the load capacity of the most heavily loaded piles is almost fully utilized; this is in agreement with the measurements.

• There is considerable variability in the predictions of minimum pile loads. Some of the methods predicted larger minimum pile loads than were actually measured.

This case history clearly demonstrates that the design philosophy of fully utilizing pile capacity can work successfully and produce an economical foundation which performs satisfactorily. The available methods of performance prediction appear to provide a reasonable, if conservative, basis for design in this case.

6.6.3. High-rise buildings on the Gold Coast, Australia

Badelow *et al.* (2006) (Table 7) have described two cases of high-rise buildings in which the original foundation designs were carried out ignoring the presence of the raft. The first building comprised a 30 storey 176 unit residential tower located in Surfers Paradise, Queensland, where the site was underlain by alluvial sand and clay sediments, below which there was a residual soil stratum of silty clay overlying meta- siltstone rock. The second case involved a 23 storey residential tower with three levels of basement located at Tweed Heads. This site was again underlain by alluvial sand and clay layers overlying a residual silty clay layer which in turn overlaid siltstone bedrock. In both cases, the founding conditions were favourable for piled rafts.

The foundations were re-designed taking account of the presence of the raft, and Table 6 compares the original and revised designs. This table shows that significant construction cost and time savings were achieved by the use of piled raft foundation systems as alternatives to conven-

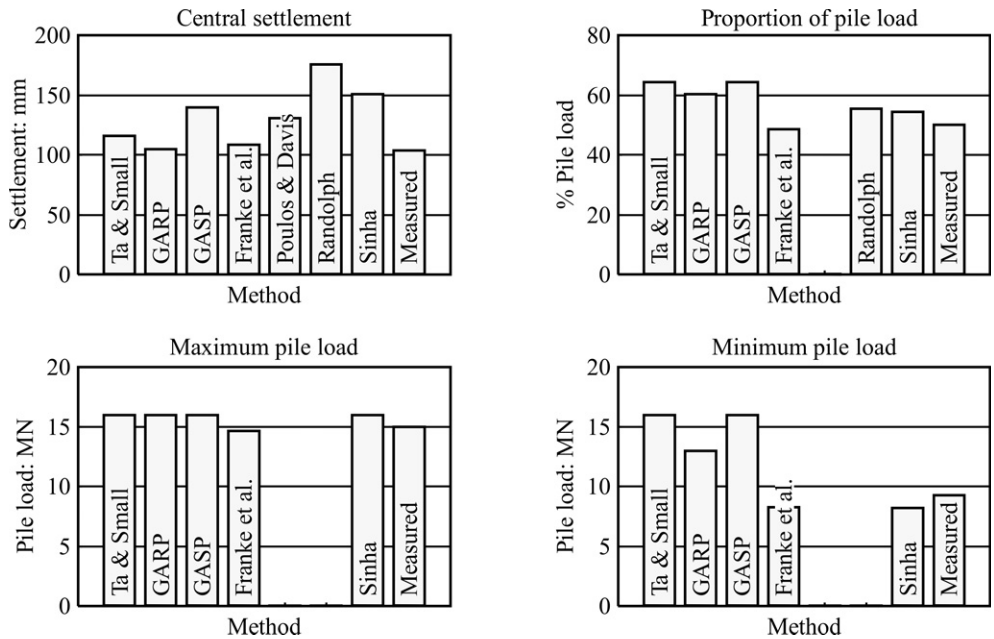


Figure 19 - Comparison of analysis methods for piled raft foundation, Westendstrasse 1 (Poulos *et al.*, 1997).

tional fully piled systems. The adoption of a piled raft resulted not only in a reduction in the number of piles required, but also in the length of the piles. In the second case, the overall foundation performance was improved because the differential settlements were reduced.

6.6.4. The Burj Dubai (Burj Khalifa)

The current world's tallest building is the Burj Dubai, re-named the Burj Khalifa at its official opening on January 4th 2010. This building is founded on a piled raft, and the design process for this foundation has been described by Poulos & Bunce (2008). Figure 20 shows a plan of the foundation, which consists of a raft 3.7 m thick and 196 piles, 1.5 m in diameter and about 50 m long, founded in a weak calcareous rock. The design of the foundation was found to be governed primarily by the tolerable settlement of the foundation rather than the overall allowable bearing capacity of the foundation. The capacity of the piles was assessed to be derived mainly from the skin friction developed between the pile concrete and rock, although limited end bearing capacity would be provided by the very weak to weak rock at depth.

The estimated maximum settlement of the tower foundation, calculated using various analysis tools, are in reasonable agreement, with the most comprehensive methods predicting a maximum long-term settlement of the order of 75-80 mm, which was considered to be within acceptable limits.

The settlements measured during construction for one of the wings of the "tripod" foundation are shown in Fig. 21 and are consistent with, but smaller than, those predicted.

Figure 22 shows contours of measured settlement. The general distribution is similar to that predicted by the various analyses.

As of mid-2009, when almost all the dead load was applied to the foundation, the maximum measured settlement was about 44 mm. On the basis of these measurements, it was estimated that the long-term settlement of the

Table 7 - Summary of Gold Coast case studies (Badelow *et al.*, 2006).

Case	Original foundation design	Revised piled raft foundation	Performance
30 storey tower, Surfers Paradise	Over 140 bored piles founded on rock at depths of 35-40 m	0.8 m thick raft on 123 0.7 m diameter CFA piles founded on stiff clay at 18 m	Saved 2767 m of pile length, and costs of about A$500,000. Maximum settlement predicted < 50 mm, maximum differential settlement < 1/400
23 storey tower, Tweed Heads	437 0.7 m and 0.9 m bored piles founded into weathered rock, and 0.45 m thick slab	0.45 m thick raft, locally thickened to 0.8 m under heavily loaded core areas, on 186 0.5 m diameter piles, and 46 0.9 m diameter CFA piles, founded on weathered rock	Savings of about A$500,000. Significant improvement in foundation performance, in terms of differential settlements between columns. Maximum predicted settlement < 50 mm

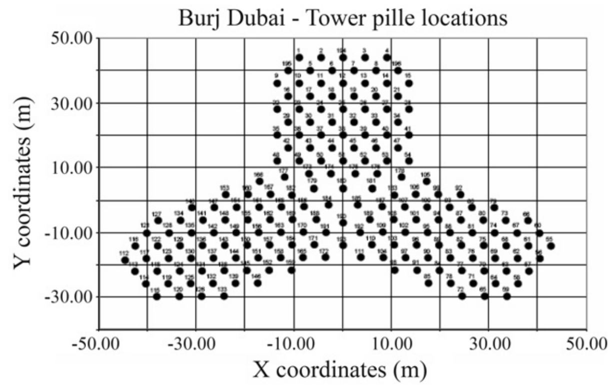


Figure 20 - Foundation layout for Burj Dubai.

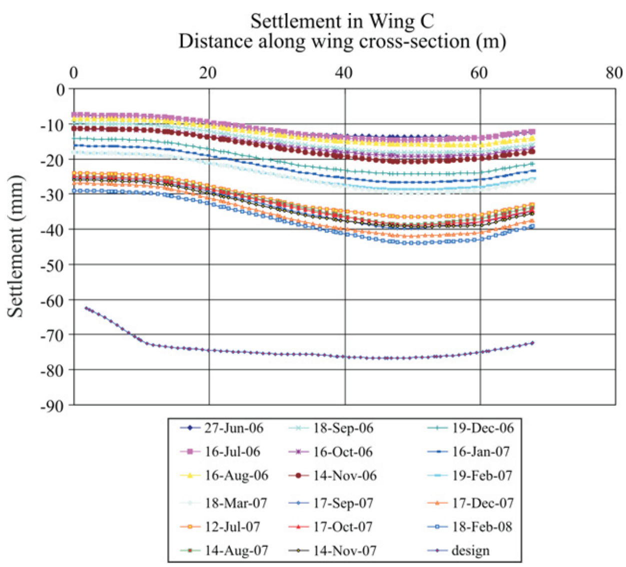


Figure 21 - Measured and computed settlements – wing C.

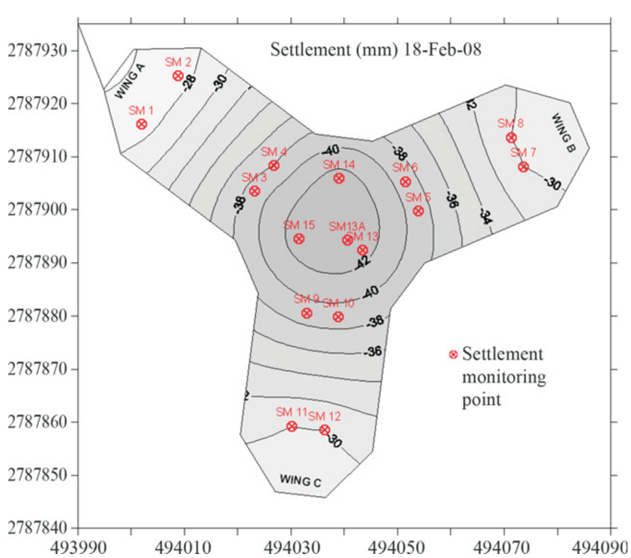


Figure 22 - Measured settlement contours for the Burj Dubai (now the Burj Khalifa).

foundation would be of the order of 55 mm, somewhat less than the predicted 75 mm.

Overall, the performance of the piled raft foundation system has exceeded expectations to date.

6.6.5. Piled rafts on very soft soils

As mentioned earlier, soft clay sites do not provide ideal ground conditions for piled rafts, but nevertheless, it is sometimes necessary to cope with such circumstances. As pointed out by Poulos (2005), possible foundation solutions may include:

• A compensated raft foundation;

• A piled raft foundation;

• A compensated raft foundation.

Compensated piled rafts involve the excavation of soil, before or after piles are installed, in order to reduce the net increase in load applied by the foundation to the underlying soft soil. The removal of soil reduces the vertical effective stress in the soil, thus putting it in an over- consolidated state and reducing its compressibility. The subsequent loadings of the foundation will therefore tend to cause less settlement than if no excavation of the soil had been carried out.

The key issues to be addressed in the design of compensated piled rafts are as follows:

• The maximum depth to which an excavation can be carried out.

• The effect of the overconsolidation caused by the excavation on the stiffness and ultimate load capacity of the raft.

• The effect of the overconsolidation on the stiffness and ultimate load capacity of the piles.

As a first approximation, it would appear reasonable to make the following assumptions with respect to raft behaviour to allow for the possible effects of excavation:

• The modulus of the soil used to compute the raft stiffness is the unload/reload value until the average contact pressure below the raft reaches the "preconsolidation" pressure, *i.e.* the footing pressure required to cause virgin (first-time) loading of the footing to occur. For average contact pressures in excess of this "preconsolidation pressure", the first loading modulus value is used.

• The ultimate bearing capacity of the raft is unaffected by the excavation process, other than for the effect of embedment, which will tend to increase its capacity.

The possible effects of excavation on the soil modulus around the piles have been ignored, since the process of pile installation generally causes a significant "preloading" of the soil around and below the pile shaft. Moreover, the simplifying assumption is made that the ultimate axial capacity of the piles is also unaffected by excavation.

6.6.6. Application to La Azteca building case

The case of the La Azteca building was described by Zeevaert (1957) (Fig. 23). The building exerted a total average loading of about 118 kPa, and was located on a deep highly compressible clay deposit which was also subjected to ground surface subsidence arising from groundwater extraction. The building was founded on a compensated piled raft foundation, consisting of an excavation 6 m deep with a raft supported by 83 concrete piles, 400 mm in diameter, driven to a depth of 24 m (*i.e.* the piles were about 18 m long below the raft).

Figure 22 shows details of the foundation, the soil profile, the settlement computed by Zeevaert, and the measured settlements. The settlement without piles computed by Zeevaert (from a one-dimensional analysis) was substantial, but the addition of the piles was predicted to reduce the settlement to less than half of the value without piles. The measured settlements were about 20% less than the calculated settlements, but nevertheless confirmed the predictions reasonably well.

An approximate analysis by the author was applied to this case, excluding the effects of ground settlements, which were not detailed by Zeevaert in his paper. The following approach was adopted:

1. The one dimensional compressibility data presented by Zeevaert was used to obtain values of Young's modulus of the soil at various depths, for the case of the soft clays in a normally consolidated state. A drained Poisson's ratio of 0.4 was assumed. The modulus values thus obtained were typically very low, of the order of 0.5-1.0 MPa, and lower than would have been anticipated on the basis of the measured shear strength of the clay.

2. The bearing capacity of the raft was estimated from the shear strength data provided by Zeevaert, and was found to be about 200 kPa. This represented a factor of safety of about 1.7 on the average applied loading of 118 kPa.

3. The settlement of an uncompensated raft was computed using these modulus values together with conventional elastic theory. A very large settlement, in excess of 2.3 m, was obtained for the final settlement.

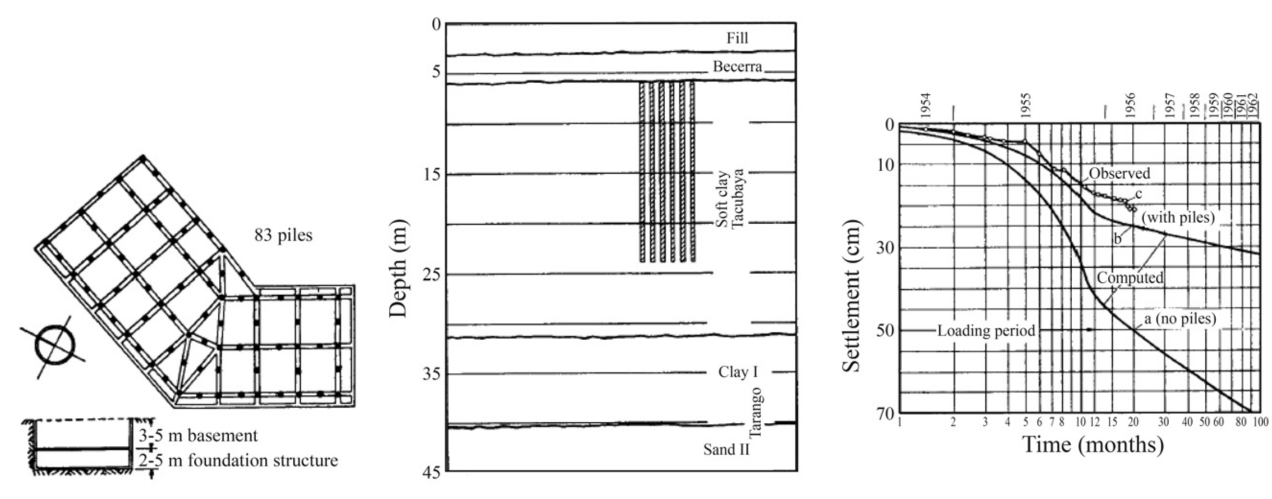
1. The settlement of a compensated raft was computed, assuming a 6 m depth of excavation, and assuming that the soil modulus values for the overconsolidated state were 10 times those for the normally consolidated state (based on the oedometer data presented by Zeevaert). The additional raft pressure to recommence virgin loading conditions, p_{ec}, was taken to be zero. A settlement of the order of 988 mm was thus computed.

2. From the pile load tests reported by Zeevaert, values of the single pile capacity and stiffness were obtained, these being about 735 kN and 25 MN/m respectively.

3. For the 83 piles used in the foundation, the group stiffness was computed by using the approximation of Poulos (1989) and applying a factor of 9.1 (the square root of the number of piles, *i.e.* $83^{0.5}$) to the single pile stiffness. A group stiffness of about 230 MN/m was calculated.

4. The average settlement of the foundation for an uncompensated piled raft was computed, using the equations developed by Randolph (1994) for the piled raft stiffness. A settlement of about 1.08 m was obtained. The analysis indicated that, in this case, the raft would carry only about 4% of the load under elastic conditions, and that the capacity of the piles would be mobilized fully under the design load of about 78 MN.

5. The effects of carrying out a 6 m deep excavation (as was actually used) was simulated by reducing the thickness of the soil profile accordingly, and again assuming that, for the raft, the soil Young's modulus for the overconsolidated state was 10 times that for the normally consolidated state (based on the laboratory oedometer data published by Zeevaert). The stiffness of the raft was thus in-



a) Foundation plan & pile layout b) Subsoil profile c) Settlement curves

Figure 23 - Details of La Azteca building on compensated piled raft (Zeevaert, 1957).

creased significantly, leading also to a significant increase in the stiffness of the piled raft foundation, to about 300 MN/m. The raft, at the design load, was found to carry about 40% of the total load, and the computed settlement under that load was reduced to about 280 mm.

The analysis results are summarized in Table 8. It can be seen that the settlement of the compensated piled raft is about 26% of the settlement of the piled raft without compensation, 29% of the settlement of the compensated raft alone, and only about 12% of the value for the uncompensated raft. Zeevaert's calculations gave larger settlements than those computed above, being about 1000 mm for the compensated raft alone, and about 370 mm for the compensated piled raft. This represented a reduction in settlement of about 63% in using the compensated piled raft rather than the compensated raft alone. This compares reasonably well to the 71% reduction in settlement computed from the present approach. It is also interesting to note that the measured settlements about 2 years after the commencement of construction were about 20% less than those predicted by Zeevaert. At that stage, the measured settlement was about 205 mm and the computed settlement from Zeevaert was 250 mm, *i.e.* about 68% of the final predicted settlement. Assuming a similar rate of settlement, the prediction made by the author's approach for the settlement after 2 years would be about 192 mm, in fair agreement with, but somewhat less than, the measured 205 mm.

Clearly, the combined use of piles and compensation via excavation, leads to a foundation that provides a superior performance to that of an uncompensated piled raft or a compensated raft alone.

6.6.7. Piled raft cases in Malaysia

Tan *et al.* (2004, 2005) have described the application of conventional piled raft foundations to cases in Malaysia involving a series of 2-storey and 5-storey apartment buildings founded on a relatively deep layer of soft silty clay. The soil profile consisted of 25-30 m of very soft to firm silty clay with some intermediate sandy layers, underlain by silty sand. Figures 24 and 25 show the variation of compressibility and strength parameters with depth at the site.

The site was subjected to filling of 0.5 to 1 m in thickness, together with temporary surcharging having heights varying from 2 m to 5 m. After the subsoil had achieved a specified percentage of settlement, the surcharging fills were removed and construction of the foundation system was commenced.

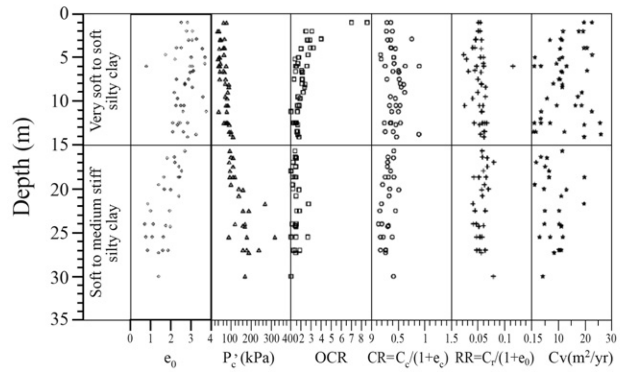


Figure 24 - Compressibility parameters for Klang clay (Tan *et al.*, 2004, 2005).

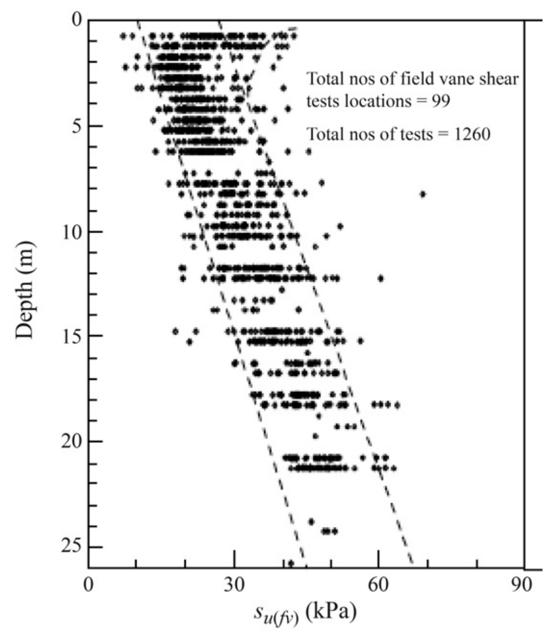


Figure 25 - Undrained strength and sensitivity of Klang clay (Tan *et al.*, 2004, 2005).

For the 2-storey buildings, piled raft foundations were used with relatively short friction piles of equal length. For the 5-storey buildings, piled rafts were also used, but the pile lengths were considerably longer and the pile length was varied, depending on the location.

Table 8 - Summary of computed average settlements.

Case	Computed average final settlement (mm)	Ratio of settlement to settlement of compensated raft
Raft alone, no compensation	2342	2.37
Raft alone, with compensation	988	1.0
Piled raft, no compensation	1084	1.10
Piled raft, with compensation	283	0.29

In their analysis of the foundation systems, Tan *et al.* used a combination of techniques to estimate the overall settlement behaviour and the pile-soil interaction. The overall settlement behaviour was computed from the conventional Terzaghi one-dimensional settlement analysis while the pile-soil interaction analysis involved iterative application of a simplified pile group analysis based on the work of Randolph & Wroth (1979), together with a commercially available finite element analysis of the raft slab.

2-Storey Buildings

For the 2-storey buildings, the column loads ranged from 10 kN to 360 kN, and the line loadings from the brick walls were from 9 kN/m to 16 kN/m. A uniform live loading of 2.5 to 3.0 kN/m^2 was assumed to act over the ground floor raft.

The foundation system consisted of a 150 mm thick raft slab thickened to a total of 600 mm over strips 350 mm wide below the column locations. 150 mm square reinforced concrete piles, 9 m long, were located below the columns (Fig. 26).

Settlements were monitored over a 6-month period, from the completion of construction of the ground floor columns to the commencement of installation of the architectural finishes. Figure 27 shows typical time-settlement relationships for one of the buildings. During the observation period, the settlements increased relatively rapidly with time, due to the increasing loads applied during construction, and at the end of the observation period, the maximum settlement was about 17 mm, with a maximum angular distortion of only 1/2850.

5-Storey buildings

For the 5-storey buildings, the column loadings ranged from 100 to 750 kN, and the line load from the brick walls was 9 kN/m. A uniform live loading of 2.7 kPa was assumed for the ground floor. The primary design criterion was to limit the angular distortion to a maximum of 1/350 to prevent cracking in walls and partitions.

The foundation system developed consisted of a 300 mm thick raft with thickened strips 350 mm wide by 700 mm deep, supported by 200 mm square section reinforced concrete piles with lengths varying from 18 m to 24 m. The longer piles were located below the central portion of the buildings, as shown in Fig. 28. In this case, the designers followed the principle set out by Reul & Randolph (2004) of reducing the differential settlements by concentrating the stiffness provided by the piles towards the centre of a loaded area.

Settlements were monitored at various locations over a 10 month period from when the building had reached the 3rd floor to more than 6 months after completion of the building. Figure 29 shows the measured time-settlement behaviour of the various locations. These measurements revealed that, while the observed settlements were relatively large, the maximum angular distortions over the period of measurement were of the order of 1/1000.

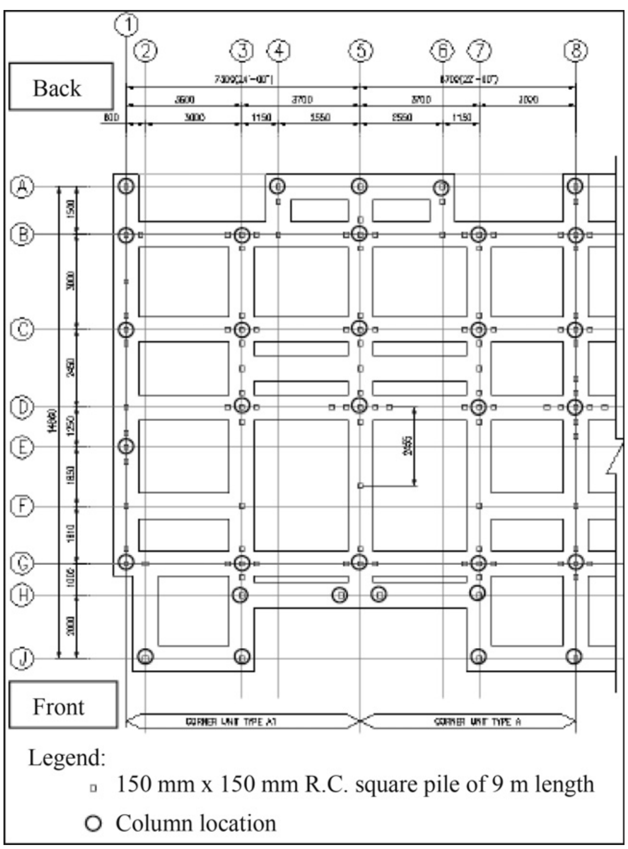


Figure 26 - Typical column and pile layout for 2-storey building (Tan *et al.*, 2004, 2005).

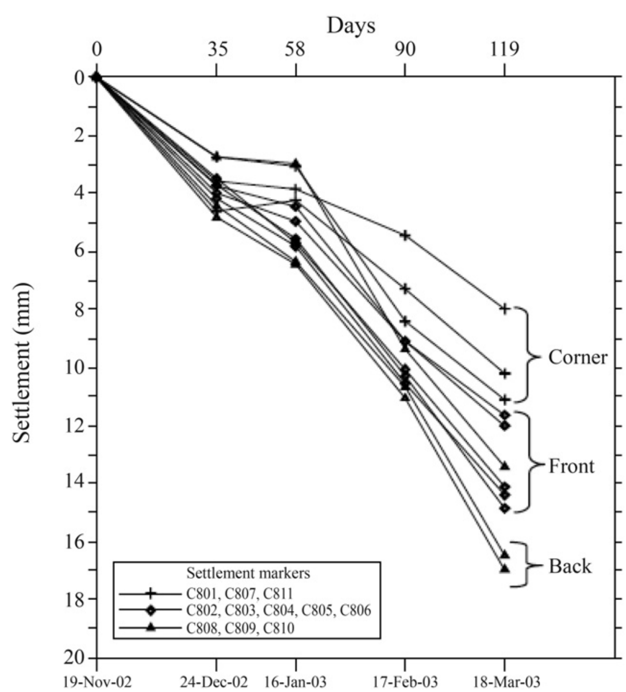


Figure 27 - Time-settlement monitoring results for typical 2-storey block (Tan *et al.*, 2004, 2005).

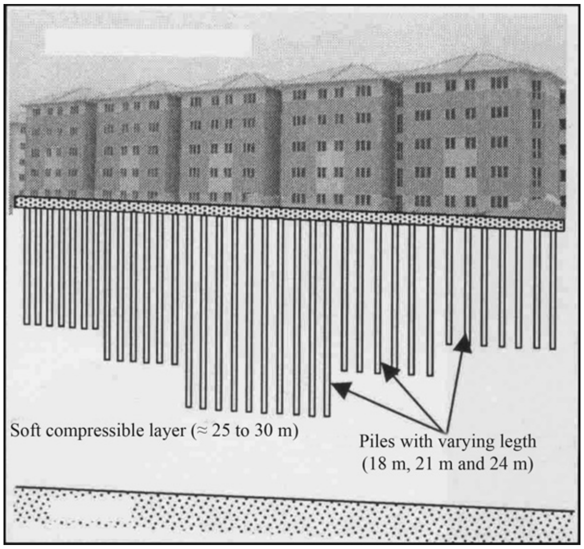


Figure 28 - Foundation system for 5-storey blocks with variable pile lengths.

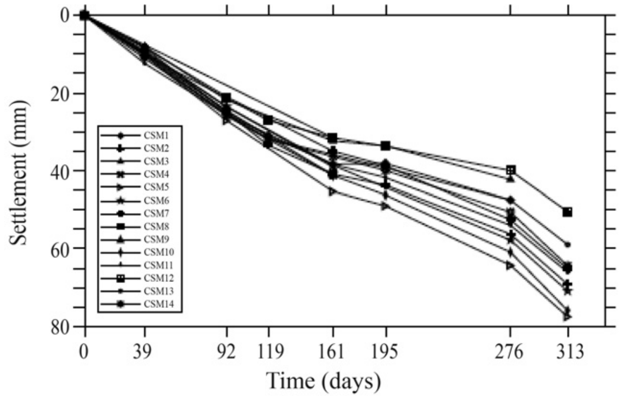


Figure 29 - Time-settlement measurements for 5-storey blocks.

Analysis of Malaysian Cases

The Malaysian cases have been analysed using a simplified analysis for pile rafts (Poulos, 20001). On the basis of the available information, the following assumptions have been made in the analyses:

1. The undrained shear strength s_u of the clay increases linearly with depth, according to the relationship $s_u = 16 + 1.6z$ kPa, where z = depth below ground surface in metres.

2. The thickness of the soft compressible clay is 30 m.

3. The long-term drained Young's modulus for the clay = $100s_u$ for calculating the settlement the raft and $200s_u$ for the calculation of pile settlements.

4. The average loading applied to the foundation by the buildings is 25 kPa for the 2-storey buildings and 62.5 kPa for the 5-storey buildings (*i.e.* 12.5 kPa per storey).

5. The raft for the 2-storey buildings is rectangular, with dimensions 80 m by 15 m.

6. The raft for the 5-storey buildings is rectangular, with dimensions 75 m by 25 m.

7. The settlement ratio for the pile groups, R_s, is approximated as $n^{0.5}$, where n = number of piles (Poulos, 1989).

8. Interaction among adjacent blocks is ignored.

Table 9 summarizes the results of the calculations for the average settlement of each building when supported by the piled raft system actually used. Also shown in this table are the settlement computed for a raft without piles, and the settlement computed if no account is taken of the presence of the raft. It can be seen that the use of piles in conjunction with the raft has resulted in a substantial reduction in the settlement, by a factor of about 3, as compared to the case of the raft alone, and by about 30%-40% compared to the piles without the raft.

Table 9 also shows the range of measured settlement reported by Tan *et al.* (2004, 2005) at the end of the settlement observation periods. The computed settlements for the piled raft are of a similar order to those measured, bearing in mind that the measured settlements were still increasing significantly with time when the observations ceased. The cases reported by Tan *et al.* therefore clearly demonstrate the feasibility of employing piled raft systems to support structures on soft clays.

6.7. Summary

The concept of settlement reducing piles, advocated by Burland *et al.* (1977) has become recognized as a potentially economical and effective type of foundation which has been used successfully in a variety of ground condi-

Table 9 - Summary of computed and measured settlements for buildings in Malaysia.

Case	Settlement mm	
	2-storey buildings	5-storey buildings
Calculated final average settlement for raft without piles	128	329
Calculated final average settlement for piles without raft	63	132
Calculated final average settlement for piles with raft	43	99
Range of measured settlements at end of monitoring period	8-22	50-78

tions. It is not uncommon for savings in the cost of the foundations of about 30% to be achieved by using piled rafts instead of conventional fully piled solutions.

Several of the world's tallest buildings in the Middle East are founded on this type of foundation, while many buildings in Frankfurt have functioned successfully on piled rafts, despite the fact that total settlements in excess of 100 mm have occurred. Piled rafts are most effective when the ground conditions near the underside of the raft are favourable and allow the raft to develop considerable stiffness and bearing capacity. However, in recent years, they have also found application in very soft clays. The cases in Malaysia demonstrate that low-to-medium rise buildings on very soft clays can be supported by piled raft foundations in which the raft is relatively thin, and the piles are engineered to obtain acceptable settlement and differential settlement performance.

Compensated piled raft foundations can be an effective foundation solution for very soft soils and have been used successfully in Mexico City. They combine the relief of overburden stress as a result of excavation, with the additional capacity and stiffness that can be provided by combining piles with a mat or raft foundation.

7. Conclusions

Victor de Mello developed a philosophy of foundation design that incorporated both common sense and sound theory. He questioned a number of conventional design approaches and pointed out their shortcomings. In particular, he was highly critical of codes that were poorly conceived and inflexible, and that led to uneconomical designs. The shortcomings that he identified in design methods included the following:

1. The use of traditional bearing capacity theory to estimate the ultimate load capacity of shallow foundations. Subsequent research has found that the traditional rigid plastic theory can be unconservative in that the effects of soil compressibility can reduce the bearing capacity very markedly, and that the theoretical size effect for foundations on sand (in which larger footings can develop larger bearing capacities) is not borne out in practice. However, on the positive side, the simple procedures adopted in practice to handle eccentric loading and applied moment appear to have been verified by subsequent research using sophisticated three-dimensional numerical analysis.

2. The commonly used "α method" for estimating the ultimate shaft friction of piles in clay is not always reliable. While considerable research has been carried out since 1969 to improve our ability to predict pile capacity and load-settlement behaviour, the accurate prediction of axial pile capacity remains rather elusive, despite the increased understanding of pile-soil interaction and the increased sophistication of some of the more recent methods of calculation. While some success has been achieved in predicting the load-settlement behaviour of single piles, accurate prediction of the settlement of pile groups, particularly if the piles are floating, also remains elusive. Given the high degree of sophistication that it is now possible to bring to bear on pile prediction tasks, it appears likely that the lack of consistent success may be due more to the deficiencies in characterising the ground profile, than to deficiencies in the methods of calculation.

3. The concept of settlement reducing piles, advocated by Burland *et al.* (1977) has become recognized as a potentially economical and effective type of foundation which has been used successfully in a variety of ground conditions. It is not uncommon for savings in the cost of the foundations of about 30% to be achieved by using piled rafts instead of conventional fully piled solutions. There is also potential for a compensated piled raft foundation to reduce both the absolute settlement and the differential settlement between the foundation and the surrounding soft soil. It therefore provides a means of developing a foundation that works and settles "with the ground", rather than one which "fights the ground".

It is sobering to reflect on the almost despairing question asked by de Mello in 1995 "*Quo vadis, Geotecnica?*" It may be argued that our capacity to solve numerical and analytical problems in geotechnical engineering has developed enormously in the 15 years since he asked that question. Yet, it may also be argued that our ability to make realistic predictions of foundation performance has barely improved. This lack of progress may be attributed to a number of factors, but perhaps the most pertinent of these are:

1. The enduring difficulty of carrying out adequate ground investigations to properly characterise a site. Despite the ground conditions often being the most potent risk factor in an engineering project, ground investigation is still generally treated as a commodity to be obtained at the cheapest price, rather than as a vital component of the engineering design process.

2. The difficulty of quantifying the soil and rock properties, taking into account the multitude of geological, environmental and geotechnical features that influence the ground behaviour. There is likely to be an optimal level of characterization that can be sought, perhaps analogous to the story of Goldilocks and the three bears. There can be too little effort expended ("the porridge is too cold") and so key aspects of behaviour are overlooked or not described adequately. There can be too much effort expended ("the porridge is too hot"), in which enormous effort is expended on every conceivable type of in-situ and laboratory test, and then tries to incorporate every conceivable physical phenomenon into the ground model. In such cases, the translation of the results into a practical ground model is either too lengthy or else it may still miss the key features of the problem. Then there is the optimal solution ("the porridge is just right") in which experience and judgement are combined with sound in-situ and laboratory testing to produce an adequate ground model that suits the key features of the prob-

lem, without trying to cover irrelevant aspects of behaviour.

3. The enduring difficulty of honestly evaluating our ability to do "Class A" predictions. De Mello commented on, and despaired of, the lack of success of experts in prediction events. It would no doubt be gratifying to him if there was a concerted effort made to make performance measurements and comparisons between anticipated and measured behaviour, a routine part of the construction and operation process.

4. Perhaps because of the increasingly large number of geotechnical researchers and the current publishing imperative, much of the geotechnical research is directed towards what may be termed "the last 2%" of a problem, *i.e.* the refinement of analyses and design procedures that are more than adequate for practical purposes. What is needed far more in research is another concerted effort to close the gap between theory and practice and to identify what combinations of ground investigation quantification and design method give reliable outcomes. It may well be time to heed de Mello's pleas and discard some of the old traditional theories (for example, the Terzaghi bearing capacity theory) and to stop the perpetuation of teaching of such theories simply because they appear in text books that have been written without an adequately critical appraisal of their applicability to geotechnical reality.

It would be highly instructive for students to be directed back to the writings of such giants of the profession as Terzaghi, Casagrande, Taylor, Skempton and de Mello. There they would find a great deal of wisdom and guidance that would assist them in understanding what is significant in a geotechnical problem and what is not. Then, in combination with properly digested and calibrated modern theories and design methods, they could achieve improved capabilities in designing foundation according to the 5 design principles of de Mello set out in Section 2.

It may be appropriate to conclude by appreciating the broader legacy that Victor de Mello left to our profession and recalling the following words from his Presidential address at the 1985 International Conference in San Francisco:

"Engineering uses art and science, intuition, and of course, the rational analyses of the day: all these are means. But the end is creativity, often inventiveness, ingenious. Engineering is the end product of design + construction + operation, a live function to be continually reviewed and revised in order to preserve or enhance the intent. As a community of engineers, we must urgently repel the widespread notion of our acting on certainty, and providing static, permanently valid projects".

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