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Comparison of pile design following two standards: EC7 and AS2159

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ABSTRACT: In normal design practice one method is chosen from the many available and applied to a site. In this paper two case histories are examined, where there is sufficient information to investigate and compare different design methods, including different methods of selecting design parameters. Because the projects are completed, there is information from boreholes and CPTs, from instrumented trial piles, working pile tests and dynamic load tests. Comparisons will include various design methods included in EC7, and also with AS 2159:2009.

1 INTRODUCTION

BS EN 1997-1 (BSI, 2010), or EC7 as it is commonly known, was published in 2004 after more than twenty years of drafting effort by a large group of people from many European countries, and was re-issued with corrigenda in 2009. It brought geotechnical engineering design in Europe into line with structural engineering design, by requiring the application of limit state design principles. In this paper we will look at the application of EC7 to the design of piles, where it was a welcome advance, since designing piled foundations using working stress design for a structure such as a cable stayed bridge being designed on limit state principles, had been extremely challenging.

In Australia, limit state design of piles has been incorporated into the standard, AS 2159 (Standards Australia, 2009), since 1995, and was revised in 2009. The opportunity will be taken to compare the two standards, in relation to two real projects.

2 PILE DESIGN TO EC7

Clause 7.4.2 gives a comprehensive list of matters to be taken into consideration in the design of piled foundations, including such things as installation effects, spacing and group behaviour, effect on adjacent properties, and chemical effects in the soil, amongst others. However, Clause 7.4.1 states:

"The design shall be based on one of the following approaches:

- the results of static load tests, which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience;
- empirical or analytical calculation methods whose validity has been demonstrated by static load tests in comparable situations;
- the results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations;
- the observed performance of a comparable pile foundation, provided that this approach is supported by the results of site investigation and ground testing."

2.1 *Pile design using the results of static load tests*

Taking design on the basis of static load tests first, the problem is, how is design on the basis of static load tests to be carried out? Bond and Harris (2008) refer to the fact that EC7 "places great emphasis on the use of static load tests, either as the primary design method or in providing validity to designs based on dynamic load tests or calculations". They then go on to recommend ISSMFE pile testing procedures, and to discuss how many tests should be carried out. However, they do not actually explain how static load testing could be used in practice to design a piled foundation. Bond and Simpson (2009) make many of the same points, but also include reference to the equations in EC7 which allow a characteristic value for ultimate pile resistance to be determined from test values. They include the equation for static load tests:

$$R_{c.k} = min\left\{\frac{(R_{c,m})_{mean}}{\xi_1}, \frac{(R_{c,m})_{min}}{\xi_2}\right\} \quad (1)$$

and discuss the values of ξ to be used, from EC7 and from the National Annex but, again, they do not actually explain how a pile may be designed using static load test results.

The best guidance has been found in Frank et al (2004) which was written by some of the key authors of the standard. Section 7.4 gives some very general explanation about design using static load test results, but the best information is in the worked example, Example 7.1. For this case a hypothetical foundation is to be designed for a major bridge, and the following information is available to the designer:

- Permanent Load 31 MN
- Accidental Load 16 MN
- The soil profile consists of 20 – 30 m of very soft clay, muddy sands, sands and clays, and, finally, sands and gravels at the level of the expected pile toe level
- Four static load tests were carried out on open ended driven piles, and of various lengths. The resulting ultimate resistances were rationalised to a length of 55.5 m and are included in Table 1.

Table 1: Measured (rationalised) ultimate pile resistances

Pile No	ULS Resistance (MN)
P8	14.0
P31	14.4
P79	12.1
P79b	13.9

In the hypothetical example the relevant correlation factors from Table A.9 of EC7 were applied to the mean and minimum resistances, in order to calculate a characteristic resistance, $R_{c,k}$=12.1 MN. The appropriate load factors were then applied to the permanent and accidental loads, and divided by the characteristic resistance to determine the number of piles required. This was checked for Load Combinations 1 and 2 of Design Approach 1, and also for Design Approach 2 and for accidental loads.

It is understood that this was only intended to be an illustrative example but, it is suggested, it is so far from reality as to actually be meaningless, because it does not give any real designer any assistance with designing a piled foundation using the results of static load tests. With over 40 years' experience of pile design and construction, the author is unable to conceive of a real project in which four static load tests are carried out, without even knowing how many piles are required on the project. As noted above, the pile lengths varied and were "rationalised to a length of 55.5 m", which begs the question as to how trial pile lengths were selected. In the event, the answer produced was that four piles were required, equal to the number of test piles! It also has to be noted that the example only works for purely vertical loads, which are extremely rare in practice. Most significant piled structures, be they moderate to high rise buildings, or infrastructure such as bridges, are subject to horizontal loads, which almost never are applied at the pile cap level. There are therefore moments to be combined with the vertical and horizontal forces, which means that the maximum pile load will be a function of its position, and that will in turn be a function of the number of piles. That will certainly be the case for a major bridge.

It is therefore believed that, for practical purposes, it is not possible to design piles purely on the basis of static load tests. When designing the foundations for high-rise buildings, it is necessary to iterate backwards and forwards with the structural engineers numerous times, in order to arrive at an optimised pile diameter and layout. This may involve several different load combinations, especially including wind loads on the high towers, and careful analysis of the stiffness of and bending moments in the mat foundation. Alternatively, in the design of foundations for balanced cantilever bridges, each extension of the balanced cantilever involves several new construction load cases, plus all the wind and flood load cases, together with the effects of age on the concrete (cracked modulus) and possible scour of the river bed. Many load cases need to be considered and, for the critical load cases, different layouts need to be tested using different numbers of piles of varying diameter, in order to arrive at an economic solution, which makes maximum reasonable use of available concrete stress.

2.2 *Other pile design methods according to EC7*

Since design using the results of static load tests has been ruled out as normally impractical, the other recommended design methods will be detailed in relation to two projects, one in Bangkok using large diameter bored piles, and one in Queensland using precast concrete driven piles. It is noted that, for design by calculation, as opposed to design by static load test, Clause 2.4.1 requires the use of a "model factor", which seems to be a way of ensuring that a lumped factor of safety is still about 2.5, as it was in working stress design. This has also been referred to by Orr (2012) and Vardenaga et al (2012a), but seems to be unfortunate, since all the finesse gained by the use of partial factors to assess different degrees of uncertainty is once again lost to a lumped factor which incorporates a whole range of disparate uncertainties. It is suggested that the ULS calculation is, in fact, a true representation of that limit state, but that, in many real situations, it is actually the serviceability

limit state which needs to control the design, and requires careful consideration. Design methods appropriate to the SLS are described in Vardenaga et al (2012b), but this is a step for which we appear to be ill prepared. Design has been strength controlled for so long that our site investigation industry is much more attuned to measuring strength than it is to measuring stiffness, and results of various pile performance prediction events would suggest that we still have some way to go in reliably predicting performance (Fellenius et al 2014).

The model factor is also referred to in Clause 7.6.2.3 (2), which precedes both 7.6.2.3 (5) which mentions "profiles of tests", and 7.6.2.3 (8) which refers to use of characteristic strength values. In Frank et al (2004) the pile designs based on profiles of tests are referred to as "model piles". The model factors are not set out in EC7, but have been left to each Country to provide in their National Annex. According to Bond and Simpson (2009) the value of the model factor recommended in the UK National Annex for a case of no static load testing, $\gamma_{Rd} = 1.4$.

3 CASE HISTORY FROM BANGKOK

3.1 *Structural design and action effects*

In practice pile design is a far more complex process than suggested by the Example from the Designer's Guide. The structure in Bangkok comprised three towers, each of 70 floors so about 200 m in height, linked together and built on a single mat foundation, as shown in Figure 1. Bangkok soils consist of alternating layers of sands and clays, laid down in marine, deltaic and fluvial environments, with bedrock reckoned to be at depths of about 500 to1000 m (Balasubramaniam et al, 2004). This meant that the mat could not be a raft foundation, since the superficial layer of about 15 m of soft clay is incapable of providing a useful bearing capacity in the long term, and subject to significant surface settlement, typically about 20 mm per year, as a result of ongoing consolidation caused by underdrainage. Loads in the framed structure were developed during the design, arising from permanent load from the structure as it was progressively sized, live loads on the floors, which are reduced according to local codes for high-rise buildings as it is too conservative to assume full floor loading for every one of 70 floors, together with wind loads and earthquake loads in appropriate combinations.

Typical structural design makes use of numerical analysis, in which foundations such as piles are generally represented by linear elastic springs on a rigid base. The first step was to try to persuade the structural engineers that this model, which showed maximum pile load under the lift core, with a mat foundation dished like a saucer, was not a true representation of load transfer. Use was made of a simple PIGLET model, utilising both rigid and flexible foundation options, in order to try to bracket the behaviour. However the effect of the three linked towers was to create a structural stiffness which, when combined with the flexibility of the real soil, meant that the model suggested maximum pile loads at the corners, the opposite of the structural model. Compatibility was achieved by an iterative process, in which spring stiffnesses in the structural model were adjusted individually, based on the results of the PIGLET analysis, both spatially and according to deflection. The optimum number of 1 m diameter piles under the mat was determined as 399 spaced out on an equilateral grid, and the average ULS action effect was about 6,500 kN. However, when factored in accordance with EC7 this increased to about 9,670 kN, while in accordance with AS 2159 (AS 1170) it was about 8,830 kN, and the maximum ULS action effect was found to be about 11,000 kN, assuming a rigid mat foundation and some yielding of extremely loaded piles.

Figure 1. High rise building for case history from Bangkok

3.2 *Pile design methods*

3.2.1 *Pile design using empirical calculation methods based on static load tests [7.6.2.3]*

The empirical method used here was developed based on a large number of static load tests, carried out in the early 1990s as part of the expansion of urban infrastructure in Bangkok. About 24 tests were carried out on one project, with another 20 undertaken at the Second Bangkok International Airport Project between 2001 and 2005. These tests, many of which were instrumented in order to allow determination of shaft friction values related to displacement, were correlated with the SPT N value since this is still the most widely used test in Bangkok soils. Stroud (1974, 1988) recommended a factor f_1 such that:

$$s_u = f_1 \times N_{60} \quad (2)$$

and suggested that, for low plasticity clays, $f_1 = 5$. Balasubramaniam et al (2004), quoting Sambhandharaksa and Pitupakorn (1994), stated that for Bangkok clays the equivalent relationship is:

$$s_u = f_2 \times N \quad (3)$$

with $f_2 = 6.72$, but note that N_{60} is not commonly used in Bangkok. For pile adhesion this is used with an adhesion factor = 0.4, giving a shaft friction value of 2.68 x N. By comparison the Stroud (1974, 1988) value is commonly used with an adhesion factor of 0.5 (Vardanega et al, 2012a), giving a shaft friction value of 2.5 x N. Balakrishnan et al (1999) suggested a value for K_s of 2.3 for the Kenny Hill Formation in Kuala Lumpur based on static load tests, where:

$$q_{sf} = K_s \times N \quad (4)$$

Chang and Broms (1991) recommended $K_s = 2$ as a design value, and hence with some degree of conservatism, based on static load tests in Singapore. All of these values are very similar and suggest that $K_s = 2.5$ is probably a reasonable non-conservative design value, although for Bangkok soils the value of $K_s = 2.68$ has been verified by many static load tests.

3.2.2 *Pile design using the model pile approach [7.6.2.3 (5)]*

Of the empirical methods included in EC7 the preferred approach is the use of "model piles". This involves taking each individual test location, such as a borehole, a CPT position or a series of pressuremeter tests, and designing a single pile, a "model pile" on the basis of the data provided. This is a method which the author used for many years in Bangkok, where it is particularly suitable. This is because it is widely known that Bangkok soils have very strong horizontal stratification, as a result of their deposition, over many cycles of marine regression, in a marine or fluvial environment leading to alternating layers of sands and clays over hundreds of metres of depth. Since the weak layers can occur at various depths, combining the data can lead to conservative estimates of characteristic strength related to the weaker strength values. On the other hand a pile, as a "rigid" vertical element, will tend to average the strength values out. It was found that "model piles" of the same length produced similar capacities from different boreholes as a result of this process.

On the subject site, eleven boreholes were available, as listed in Table 2.

Table 2: Borehole depths and calculated ultimate resistance

Borehole ID	Depth (m)	Ultimate geotechnical resistance (kN)
BH1	80	13269
BH2	80	14262
BH3	80	16620
BH4	80	13141
BH5	80	12464
BH6	80	15002
BH8	60	15366
BH9	60	16340
BH10	60	13183
BH11	60	14577
BH12	60	14264

These were used to create 11 model piles, in a spreadsheet incorporating conversion from SPT N value to shaft friction for both cohesive and granular soil layers. Assuming a 47 m long pile, the results were as shown in the table above, which have a mean of 14,408 kN and a minimum value of 12,464 kN. Table A.10 of EC7 gives $\xi_3 = 1.25$ and $\xi_4 = 1.08$ which leads to a characteristic ultimate geotechnical resistance = 11,526 kN.

3.2.3 *Pile design using the characteristic ground profile approach [7.6.2.3 (8)]*

In this method all of the data is considered, as plotted in Figure 2. It is apparent that there are roughly three depth zones below the upper 15 m which is the soft Bangkok marine clay, defined as 15 to 30 m, 30 to 60 m, and 60 to 80 m. These three layers were therefore treated separately. It was decided to use the method proposed by Bond (2011), in which the characteristic value of a parameter which varies with depth is defined by the 95% confidence level of the mean (the 50% fractile), and that proposed by Schneider (1997) in which the characteristic value is defined as half a standard deviation below the mean. For the former the relevant equations are:

$$X_{k,inf} = m_X - k_n s_X \quad (5)$$

Where m_x is the mean value, s_x is the standard deviation, and k_n is a statistical coefficient which depends on Student's t, as follows:

$$k_n = t_{n-1}^{95\%} \times \sqrt{1/n} \qquad (6)$$

where $t_{n-1}^{95\%}$ is Student's t-value for *(n-1)* degrees of freedom at a 95% confidence level.

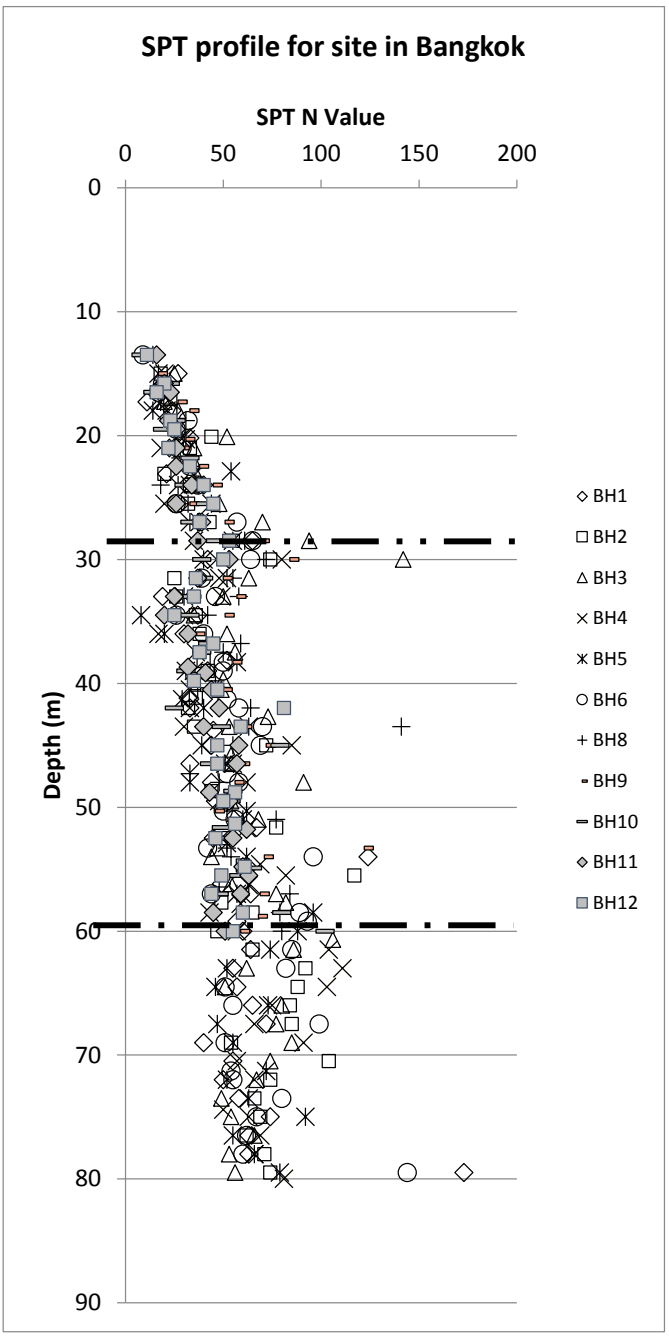

Figure 2. SPT data for first site

Figure 3 shows these two relationships plotted against depth, as well as the mean. It can just be made out that, because of the consistency of the data, the 50% fractile is less conservative than half a standard deviation below the mean for all three layers.

Entering these into the same design spreadsheet as used before, to convert SPT N values to shaft friction and end bearing, and making the calculations for a 47 m long pile, gave a characteristic ultimate geotechnical resistance of $R_{c,k}$ = 13,254 kN using the 50%

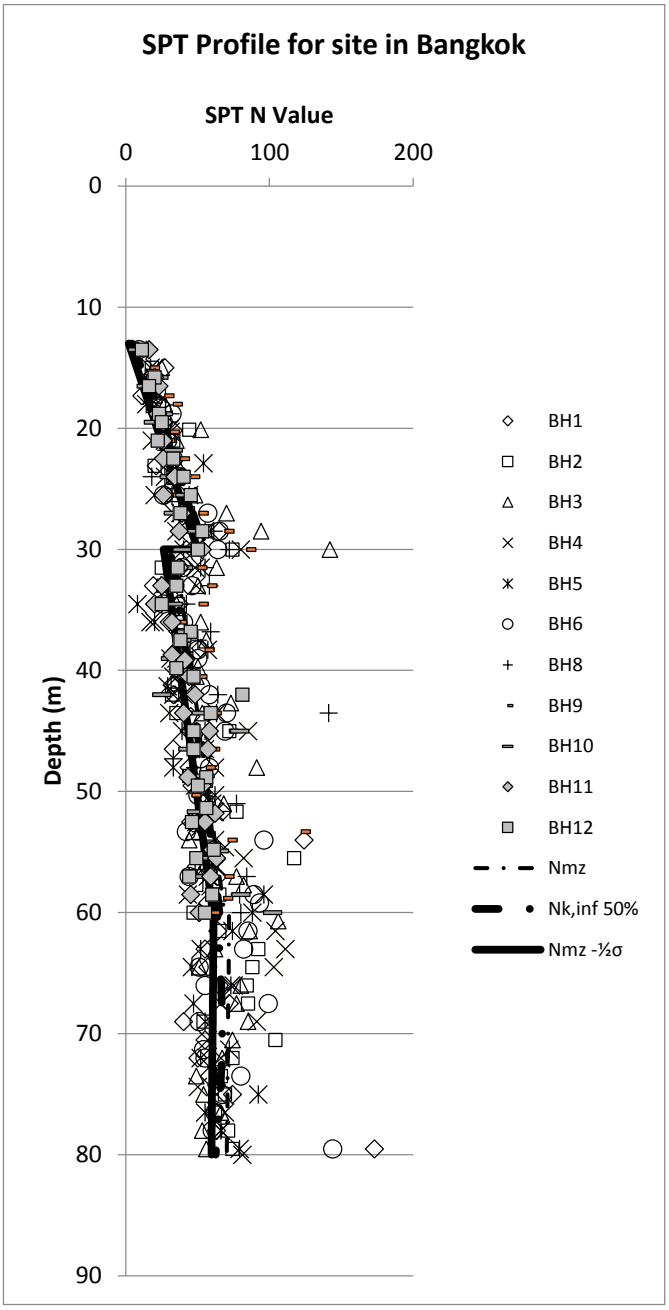

Figure 3. SPT data for first site with design lines

fractile, and $R_{c,k}$ = 11,787 kN using the characteristic as half a standard deviation below the mean.

3.2.4 *Pile design using dynamic load test results [7.6.2.4]*

On this site, four dynamic pile tests were also carried out on working piles, and the stress wave records were subjected to signal matching using the CAPWAP software. The results were as shown in Table 3.

These give a mean value of 15,557 kN, and a minimum of 15,277 kN. Table A.11 of EC7 gives ξ_5 = 1.6 and ξ_6 = 1.5, to be applied to the mean and minimum values respectively to determine a characteristic ultimate geotechnical resistance. However, a footnote to the table allows these to be multiplied by 0.85 if signal matching is used. This leads to modified values of

$\xi_5 = 1.36$ and $\xi_6 = 1.275$, which in turn produce a characteristic ultimate geotechnical resistance, $R_{c,k}$ = 11,439 kN.

Table 3: Dynamic pile test results after CAPWAP analysis

Pile No	Ultimate geotechnical resistance (kN)
#P101	15277
#214	15810
#224	15716
#349	15424

3.2.5 *Pile design using static load test results [7.6.2.2]*

In fact, in this case study, a static load test was carried out as a part of the design process for confirmation purposes, allowing a comparison to be made with this method also. The pile behaved better than had been anticipated by the design, such that at the maximum planned test load of 16,795 kN, restricted by the capacity of test frames, jacks and anchor piles, the settlement was only 29.2 mm. The Cemset program (Fleming 1992) was used to extrapolate the load-settlement curve to estimate the load corresponding to a pile head settlement equal to 10% of the pile diameter, a definition of ultimate geotechnical resistance in accordance with EC7. This figure was 20,630 kN. Based on Table A.9 of EC7 the factors ξ_1 and ξ_2 are both equal to 1.4 for a single value, with the result that $R_{c,k}$ = 14,736 kN.

3.3 *Comparison of methods*

It is therefore possible to compare directly the characteristic ultimate geotechnical resistances determined from the different methods for this single site. The results are shown in Table 4.

Table 4: Characteristic ultimate geotechnical resistances by different methods

Pile design method	EC7 Clause	$R_{c,k}$ (kN)
Required ultimate resistance		11,000
Model pile method	7.6.2.3 (5)	11,526
Ground profile method (1)	7.6.2.3 (8)	13,254
Ground profile method (2)	7.6.2.3 (8)	11,787
Dynamic pile tests method	7.6.2.4	11,439
Static load tests method	7.6.2.2	14,736

Ground profile method (1) uses the 50% fractile

Ground profile method (2) uses a characteristic strength half a standard error below the mean

The agreement between three of the methods is startling, with the maximum and minimum within 2% of the mean. The 50% fractile characteristic value was seen to be less conservative than the value based on Schneider, and that is reflected in a characteristic geotechnical resistance 14% above the mean value from the results above. The static load test gave the highest result, demonstrating that the design method of converting SPT N value to shaft friction and end bearing is still conservative, as one would hope, although it has been based on many previous tests.

However, there are a number of factors included in EC7 which have not yet been taken into account. It is noted that they are not applied consistently. For example, the model factor is to be applied to characteristic values determined using ground test results, including model piles, but not to designs based on static load tests or dynamic load tests. It is also noted that the correlation factors for use with static load tests, and those for use with ground profiles, can be multiplied by 1.1 if the structure has enough stiffness to redistribute loads, but the same is not applied to designs based on dynamic load tests or ground test results other than model piles. The result is that Table 4 is modified as shown in Table 5.

Table 5: Characteristic ultimate geotechnical resistances when various additional factors have been applied

Pile design method	EC7 Clause	$R_{c,k}$ (kN)
Required ultimate resistance		11,000
Model pile method	7.6.2.3 (5)	12,464
Ground profile method (1)	7.6.2.3 (8)	9,467
Ground profile method (2)	7.6.2.3 (8)	8,419
Dynamic pile tests method	7.6.2.4	11,439
Static load tests method	7.6.2.2	16,209

It is immediately apparent that there is far less agreement between the different methods, and it is much harder to explain the differences. Since both the model pile method and the ground profile method are based on the same correlation between SPT N value and shaft friction, based on the same set of pile load test data, the significantly more conservative designs based on the ground profile method are very hard to justify.

4 CASE HISTORY FROM QUEENSLAND

4.1 *Structural design and action effects*

In this case the structure was a fluid storage tank associated with a water treatment plant. The pile load was determined from permanent load of the structure, plus permanent live load from the water in the tank, which was to be normally full. Various other load cases were considered, together with negative shaft friction as a result of consolidation of the near surface soft clays under new fill, and the result was a required ultimate geotechnical strength of 1,995 kN. As in the previous case study, the relationship between structural design, involving all the various imposed loads in different combinations with appropriate factors, and geotechnical design which accounts for soil structure interaction, and deformation within the soil particularly for large pile groups, is complex and required close cooperation and iterations between the designers. 270 mm square precast concrete piles were

to be used, and about 300 were required to support the structure.

4.2 *Pile design methods*

4.2.1 *Pile design using empirical calculation methods based on static load tests [7.6.2.3]*

The geotechnical investigations had involved a number of phases, over a number of years, as set out in Table 6.

Although a total of 13 CPT tests and 23 boreholes were carried out, only the last three boreholes in the later years were deep enough to be used for pile design. A weak alluvial layer in the top 10 m would not provide shaft friction support, but would in fact create negative shaft friction loads on the piles.

Because of a lack of static load testing results available for Queensland soils, the same relationship between SPT N value and shaft friction was used as was derived in Bangkok. This is clearly very poor in relation to site specific correlations, or even to correlations based on the same geology but, in some circumstances such as this may become necessary and will have to be taken into account in the selection of appropriate factors.

4.2.2 *Pile design using the model pile approach [7.6.2.3 (5)]*

In Figure 4 all of the SPT N values from the three boreholes are plotted. The individual boreholes were used to create three "model piles". The ultimate geotechnical resistances were calculated as shown in Table 7.

Table7: Ultimate geotechnical resistances from model pile approach

BH ID	Ultimate Geotechnical Resistance (kN)
BH101	2728.6
BH102	2904.5
BH103	2830.3

These in turn led to a mean value of 2821 and a minimum value of 2729 kN. For three model piles the correlation factors from Table A.10 of EC7 are ξ_3 = 1.33 and ξ_4 = 1.23. The resulting characteristic ultimate geotechnical resistance was $R_{c,k}$ = 2,121 kN.

4.2.3 *Pile design using the characteristic ground profile approach [7.6.2.3 (8)]*

Figure 5 shows the mean line after removal of data for the upper soft soils, with the inferior characteristic and the line half a standard deviation below the mean. The formulae from these two lines have then been entered into the same design spreadsheet as previously with the following results. The characteristic ultimate geotechnical resistance using the 50% fractile was $R_{c,k}$ = 2,947 kN, and using the values half a standard deviation below the mean was $R_{c,k}$ = 2,379 kN.

Table 6: Available ground investigation information

2006		2009		2010		2010	
BH ID	Depth (m)	BH ID	Depth (m)	BH ID	Depth (m)	BH ID	Depth (m)
BH01	5	BH01	3	BH01	23.95	BH101	38.5
BH02	5	BH02	3	BH02	23.75	BH102	40.06
BH03	5	BH03	3	CPT1	15.4	BH103	40.4
BH04	5	BH04	3	CPT2	15.73		
BH05	15.45	BH05	3	CPT3	15.08		
BH06	15.45	BH06	3	CPT4	14.85		
BH07	20.95	BH07	3	CPT5	15.08		
BH08	18.45	BH08	19.5	CPT6	15.23		
		BH09	18	CPT7	15.58		
		BH10	18	CPT8	15.25		
		CPT1	17	CPT9	15.38		
		CPT2	14.53	CPT10	21.35		
		CPT3	15.63				

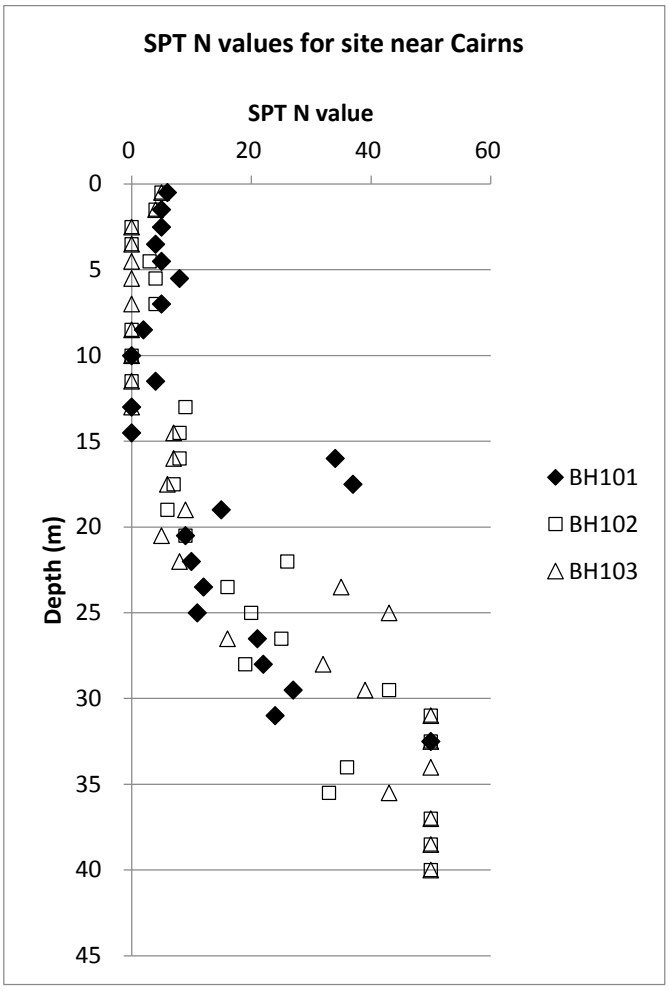

Figure 4. SPT data for second site

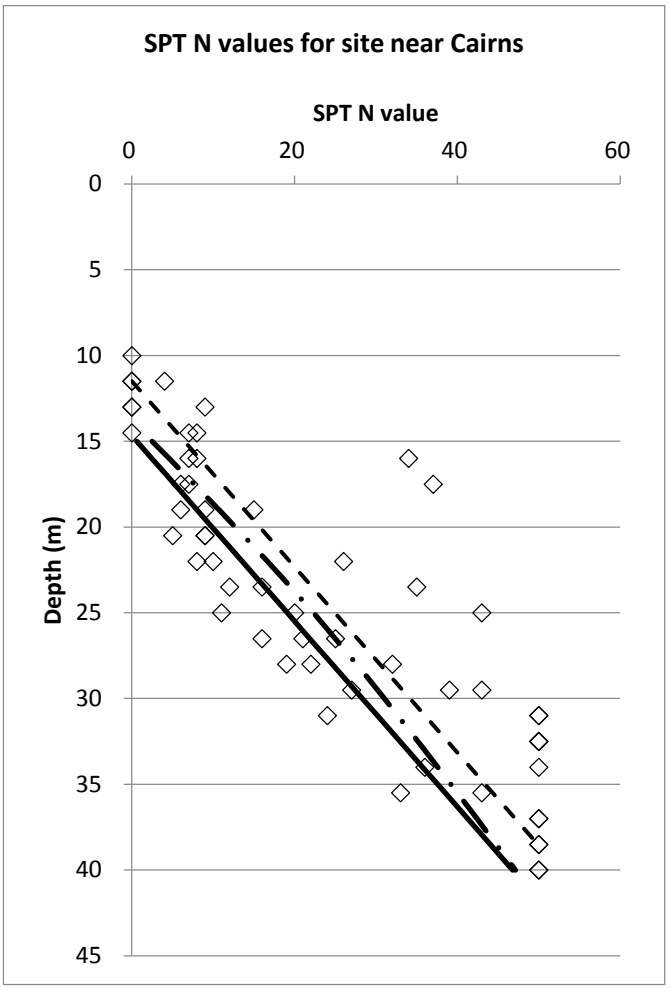

Figure 5. Selected SPT data for second site with design lines

4.2.4 *Pile design using dynamic load test results [7.6.2.4]*

On this site 15 dynamic pile tests were carried out, and the ultimate geotechnical resistances determined from the wave equation analysis (Case Method) were as given in Table 8.

Table 8: Ultimate geotechnical resistances from dynamic pile tests

Pile ID	DPT results	Pile ID	DPT results
CP21	2568	P36	2599
CP27	2581	P59	2712
CP32	2728	P95	2610
CP73	2554	P99	2665
CP79	2839	P113	2821
CP84	2541	P114	2867
IW4	2534	UP4	2650
P12	2453		

For 15 dynamic pile tests the correlation factors from Table A.11 of EC7 are ξ_5 = 1.42 and ξ_6 = 1.25, which lead to a characteristic ultimate geotechnical resistance $R_{c,k}$ = 1,869 kN.

This value is clearly low, and reflects the large factors that need to be applied to wave equation analysis evaluations because of uncertainties over critical issues such as selection of damping factors which have a major impact on the outcome. As previously, Table A.11 of EC7 also allows a reduction of these correlation factors by 0.85 if signal matching is used, such as CAPWAP. On this site 5 of the DPT results were subject to CAPWAP analysis. The correlation factors increase because of the smaller number of tests, but this is more than offset by the reduction to 85%. The appropriate correlation factors are ξ_5 = 1.275 and ξ_6 = 1.1475 and these lead to a characteristic ultimate geotechnical resistance = 2,017 kN.

The results are summarised in Table 9.

Table 9: Summary of ultimate geotechnical resistance values by different methods

Pile design method	EC7 Clause	$R_{c,k}$ (kN)
Required ultimate resistance		1,995
Model pile method	7.6.2.3 (5)	2,121
Ground profile method (1)	7.6.2.3 (8)	2,947
Ground profile method (2)	7.6.2.3 (8)	2,379
Dynamic pile tests method (1)	7.6.2.4	1,869
Dynamic pile tests method (2)	7.6.2.4	2,017

Unfortunately no static load tests were carried out on this site to give that added correlation. Nevertheless it can be seen that the design satisfied the ULS inequality in every case except for the use of dynamic pile testing without signal matching.

As with the first case study, we have not yet applied the additional factors, such as model factors and superstructure stiffness factors, recommended by EC7. Again these are applied inconsistently, with characteristic ultimate geotechnical strengths increased due to load sharing only for the model pile method, since there are no static load tests, and reduced because of the application of the model factor, for the model pile method and both ground profile methods, but not for the dynamic pile test methods. The results are as set out in Table 10.

Table 10: Summary of ultimate geotechnical resistances after additional factors

Pile design method	EC7 Clause	$R_{c,k}$ (kN)
Required ultimate resistance		1,995
Model pile method	7.6.2.3 (5)	1,667
Ground profile method (1)	7.6.2.3 (8)	2,105
Ground profile method (2)	7.6.2.3 (8)	1,699
Dynamic pile tests method (1)	7.6.2.4	1,869
Dynamic pile tests method (2)	7.6.2.4	2,017

The obvious result is that now three of the methods fail to meet the required ultimate geotechnical strength, and this is considered to be unduly conservative.

5 PILE DESIGN METHODS ACCORDING TO AS 2159-2009

Australia has had a limit state design code for piling since 1985. This has used the resistance factor method applied to the overall resistance, rather than the strength factor method applied to individual components of strength, and has applied a geotechnical strength reduction factor, φ_g, selected by the designer. In the 1995 version there was a table, Table 4.1, which allocated a range of values to φ_g according to design method. These ranged from $0.7 < \varphi_g < 0.9$, for piles subject to static load testing, through $0.5 < \varphi_g < 0.85$ for dynamic load tests, to $0.4 < \varphi_g < 0.55$ for static analyses or pile driving formulae.

One of the problems with this method was that, even though guidance was given in Table 4.2 as to from where in the given range the reduction factor should be selected, based on quality and quantity of data and methods available, it was inevitably found that designers employed by clients in conventional roles would be more conservative than those employed by contractors in Design & Construct roles.

An attempt was made to reduce this discrepancy in the 2009 revision of the standard, and also to encourage static load testing, which has become extremely rare in Australia. The new derivation of the geotechnical resistance factor is now a risk based approach, in which nine influencing attributes are each evaluated on a scale of 1 to 5, 1 representing the lowest risk, and appropriate weightings are applied to each attribute.

The average risk rating, ARR, is then defined as the sum if the individual ratings each multiplied by its weighting, divided by the sum of the weightings. The ARR is then used to determine a basic geotechnical strength reduction factor, φ_{gb}, from which the final value of φ_g is determined based on the amount of static or dynamic load testing to be carried out. The appropriate geotechnical strength reduction factor is then applied to the design ultimate geotechnical strength, $R_{d,ug}$, to give the design geotechnical strength, $R_{d,g}$. The design ultimate geotechnical strength is to be determined from:

a. Analysis using data from a site investigation
b. Analysis based on dynamic data obtained during installation of test or working piles (pile driving formula, wave equation analysis, closed form solutions or signal matching)
c. Analysis of data collected during pile installation
d. Analysis using data from a static, rapid or bi-directional load test

Standard formulae for shaft and base resistance are recommended, using average shaft friction in compression, $f_{m,s}$ and base resistance, f_b, with the comment:

"In assessing $f_{m,s}$ and f_b, consideration shall be given to the pile type, the method of installation, the soil type and other factors which may influence $f_{m,s}$, and f_b, such as the installed condition of the shaft and base."

This gives no clear guidance as to what strength values should be used, and Vardenaga et al (2012a) commented "It does seem curious that partial factors can be assigned without knowledge of how conservatively engineers treat their soil data." It is noted that AS 2159 assigns a geotechnical strength reduction factor to the second decimal place, while Vardenaga et al selected a 25th percentile as the appropriate strength when others might well have chosen half a standard deviation below the mean, or even the mean, expecting the strength reduction factor to deal with uncertainty.

It is interesting that AS 2159 does not refer anywhere to "characteristic" strength for soil or rock, and reserves the term only for concrete. It does introduce a term, R_{ug}, within the definitions which is described as the "Ultimate geotechnical strength of a pile. This is estimated either by calculation ($R_{d,ug}$) or by test ($R_{t,ug}$)" and also as "The resistance developed by an axially or laterally loaded pile or pile group at which static equilibrium is lost or at which the supporting ground fails."

Since this value can only be *estimated* by design or by test, it would appear to be a notional ultimate strength. It is only subsequently used in relation to jacked piles and pile testing. Although guidance is given on the determination of $R_{d,ug}$, which is taken to be a good estimate of R_{ug}, there is much less help with $R_{t,ug}$. This will be discussed again later.

There is also a catch within AS 2159-2009, which is in the chapter on testing, Section 8, rather than in the design chapter where all the other material related to φ_g can be found. Clause 8.2.4 (c) states:

"Where the basic geotechnical strength reduction factor is greater than 0.4, the following testing shall be undertaken:

(i) In the absence of tests to verify design ultimate geotechnical strength, testing shall be performed to verify pile serviceability for all foundations with average risk rating of 2.5 or greater. The relevant acceptance criteria nominated in Clauses 8.4.3 and 8.5.2 shall apply. The minimum rate of testing will depend on the average risk rating, as tabulated in Table 8.2.4(A).

(ii) Testing shall be performed to verify the integrity of pile shafts. Assessment of pile shaft integrity may be by high-strain dynamic pie testing (see Clause 8.5), or other methods of integrity testing (see Clause 8.8).

(iii) (This clause sets out the percentage of piles for integrity testing, according to ARR)."

Thus, for any pile with a geotechnical strength reduction factor greater than 0.4, at least integrity testing is required, and, if ARR > 2.5 (i.e. the risk rating is higher than very low or low), proof load testing is also required. The corollary is that, if no testing is carried out, φ_g cannot be greater than 0.4.

6 CASE HISTORY FROM BANGKOK

6.1 *Structural design and action effects*

Using the load factors of AS 1170 (Australian Standards 2002), the design action effect was determined to be about 8830 kN. For no pile testing, with $\varphi_g = 0.4$ this is equivalent to $R_{d,ug} \geq 22{,}075$ kN. Based on the calculations considered earlier with regard to EC7, this would clearly be an uneconomic design.

With a significant amount of site investigation, including boreholes to 80 m, extensive experience in the Bangkok soils over many years an many load tests, the calculated ARR = 1.76, leading to $\varphi_g = 0.70$ for high redundancy systems, such as 399 piles under a single mat. If four dynamic load tests are carried out on bored piles, this increases to 0.71, and if a single static load test is carried out, it increases to 0.72. Thus the required $R_{d,ug} \geq 12{,}265$ kN.

6.2 *Geotechnical design*

The geotechnical design could be carried out by the model pile procedure of EC7, as was the author's standard practice, which would have led to a mean design ultimate geotechnical strength = 14,408 kN or a minimum = 12,464 kN. This satisfies the inequality. The four dynamic pile test results then all confirm the design, with a mean ultimate geotechnical strength = 15,557 kN and a minimum = 15,277 kN. The static load test, with a predicted ultimate geotechnical strength = 20,630 kN, also confirms the design.

7 CASE HISTORY FROM QUEENSLAND

7.1 *Structural design and action effects*

AS 2159-2009 does not require negative shaft friction to be taken into account at the geotechnical ultimate limit state, although it must be considered in relation to the structural ultimate limit state and at the serviceability limit state. The argument is that, if the ultimate geotechnical limit state were to occur, then any soil-pile movement which had mobilised negative shaft friction would be reversed. As a result the action effect, $E_d = 1{,}500$ kN. The piles were installed under a D&C contract allowing the piling contractor to select the geotechnical strength reduction factor. As stated previously, without any testing at all this would have been restricted to 0.4 by Clause 8.2.4 (c), and this would have required $R_{d,ug} \geq 3{,}750$ kN which would be very unlikely to be economic, especially for driven piling where dynamic load testing is readily available at little extra cost. In the event the contractor/designer opted for 10% pile testing allowing them to select a $\varphi_g = 0.74$, requiring $R_{d,ug} \geq 2{,}027$ kN.

7.2 *Geotechnical design*

Using the same geotechnical design method as for the EC7 design, the three model piles gave ultimate geotechnical resistances of 2,729, 2,904 and 2,830 kN, which would have been more than adequate. Fifteen dynamic pile test results with a mean of 2,654 kN and a minimum of 2,453 kN would also have confirmed the design, as would the five CAPWAP results with a mean of 2,572 kN and a minimum of 2,539 kN.

In the event, when the design was submitted to the structural engineer for comment, a number of queries were raised, mainly on structural issues. The contractor/designer then took the unusual step of withdrawing the calculations on the basis that the dynamic pile testing had proven the design, because the minimum test value was higher than the required ultimate geotechnical strength. However, this then created the dilemma that a design based on a "characteristic" strength was not available, and it was necessary to

consider, based on the dynamic load test results available, what results might have been obtained if all piles had been tested. No guidance is available for this situation in AS 2159-2009, as mentioned above.

It is a matter of statistics, and one approach would be to use the factors in EC7 applied to minimum and mean strengths. This gives a characteristic strength, as before, of 1,869 kN based on the dynamic load tests, and 2,017 based on the CAPWAP results, neither of which is enough. The problem is that, with the use of the geotechnical strength reduction factor and the correlation factors together, there is almost certainly too much conservatism built into the design. It also highlights the problems that can occur when mixing elements from more than one code.

8 COMPARISON OF CODES

It may be useful to compare the numerical results obtained by use of the two different codes. This is attempted in Table 11 below, and it appears that, for the same pile size and length. The EC7 values are more conservative than the AS 2159 values. However caution is advised, as it is not a matter of simply looking at the numbers. The ultimate geotechnical resistance values from EC7 are characteristic values, whereas there is no equivalent in AS2159. It must also be noted that these values are used in the basic inequality, of the form:

$$F_{c;d} \leq R_{c;d} \tag{7}$$

and that the magnitude of the design action effect may vary.

It also needs to be noted that these comparisons have been made at the ultimate limit state, which is often the first state to be examined, probably because of history in working stress design. AS2159 specifically requires examination of the serviceability limit state, whereas EC7 states in a note under 7.6.4.1 (2) that "for piles bearing in medium-to-dense soils …. The safety requirements for the ultimate limit state are normally sufficient to pre vent a serviceability limit state in the supported structure."

9 CONCLUSIONS

- The design of piled foundations in real situations is more complex than is implied by some of the published data, including examples of how to use design standards such as EC7.
- Although design of piled foundations using the results of static load tests directly is the preferred option according to EC7, the method is not likely to be practical for the design of real foundations.
- The design of piled foundations by empirical methods based on the results of static load tests is a practical solution, which can provide credible results.
- These methods can include the use of "model piles", where an empirical design is based on each individual investigation position, such as a borehole or an in situ test (e.g. CPT) profile.
- They can also involve methods in which characteristic strength profiles are determined from a number of boreholes, and these have been shown to give compatible results.
- With regard to site characterisation, the use of the model pile approach or the use of characteristic strength profiles gave very similar results when rational methods were used to produce the strength profiles.
- The design of piled foundations using the results of dynamic load tests has been shown to produce results which are compatible with the above methods, but this would still require the installation of a number of piles ahead of the main works in order to establish the design.
- The application of some of the additional factors according to EC7, such as the "model factor" and the foundation stiffness factor, appear to be somewhat inconsistent and, in some cases, unnecessary.
- Both foundations considered, one for bored piles and one for driven piles, appeared to be able to meet the requirements of both EC7 and AS2159-2009.

Table 11: Comparison of the results from the two codes

Site	With or without testing	Code	
		EC7 – $R_{c,k}$ (kN)	AS2159 – $R_{d,ug}$ (kN)
Bangkok	No testing	11,526	12,464
	With static load testing	11,434	15,277
	With dynamic testing	14,736	20,630
Queensland	No testing	2,121	2,379
	With dynamic testing	2,017	2,017

10 ENDNOTE

It should be noted that, while the author has a keen interest in EC7 and has followed its development over several decades, he does not use it on a regular basis nor is he subject to any National Annex, which is a crucial part of the application. Therefore the factors referred to, and comments made upon them, are from the base document and many of these will have been modified by a National Annex. The exception is the Model Factor, for which no value is given in the base document, so the value of 1.4 has been taken from the UK National Annex.

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