Field measurements of CPT and pile base resistance in sand

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Abstract

A comprehensive database of load tests on closed-ended piles in sand has been assembled to examine the relationship between CPT resistance, q_c, and ultimate base capacity, q_b. The aim is to establish the origin of low reported values of q_b/q_c which contrast with continuum models that suggest $q_b = q_c$ for steady deep penetration. Partial embedment of the pile tip into a hard layer underlying weak material has been accounted for by weighting q_c. Partial mobilisation has been accounted for by defining failure according to a plunging criterion. When these two mechanisms are considered, the resulting values of q_b/q_c have a mean value of 0.90 and show no trend with pile diameter. The remaining slight underprediction of the 'continuum' model ($q_b = q_c$) could be attributed to the underestimation of plunging load in pile tests for which steady penetration is not reached. This conclusion challenges the diameter-based reduction factor on the ultimate end bearing capacity of closed-ended piles in sand recommended in the MTD design method proposed by Jardine & Chow (1996).

Introduction and background

A number of alternative methods exist to predict the unit base resistance, q_b, of a displacement pile in sand based on the results of a cone penetration test (CPT). The geometric similarity of piles and CPT instruments suggests that during steady penetration (or at the 'plunging' load[1] in a maintained load test), q_b should equal q_c, as is predicted by continuum analysis methods such as cavity expansion solutions (Randolph *et al.*, 1994) and the strain path method (Baligh, 1986). However, a number of authors have suggested that reduction factors should be applied to cone resistance, q_c, such that $q_b = \alpha\, q_c$ where $\alpha < 1$. These reduction factors can be linked to:

- Partial embedment (L/D)

  Since a pile has a greater diameter than a CPT instrument, a deeper embedment from the ground surface, or into a hard layer, is required to mobilise the 'full' strength of that layer. Prior to sufficient penetration, q_b will be less than q_c since the previous layer will still be 'felt' by the pile tip (eg. Meyerhof, 1976; Valsangkar & Meyerhof, 1977) This mechanism is illustrated in Figure 1.

  Also, since the L/D ratio of a CPT exceeds that of a pile, the ratio of shaft to base area is higher, and hence the ratio of Q_s/Q_b. Analysis of the interaction between the shaft and base offers a mechanism by which the surcharge on the soil surrounding the base of a CPT is higher than around the base of a pile, leading to a corresponding decrease in q_b/q_c (Winterkorn & Fang, 1975; Borghi et al, 2001).

- Local inhomogeneity

  Kraft (1990) proposes that a reduction factor should be applied to account for local inhomogeneities. It is argued that the probability of pile base resistance being reduced by a local region of weak soil is higher than that of a CPT due to the larger volume of soil

[1] 'Plunging' capacity is defined as the load at which continued penetration occurs without any further increase in resistance. Although not always reached in maintained load tests, this is a more fundamental measure of capacity than the load at a chosen a settlement criterion, of which there are many, and which are influenced by pile stiffness as well as strength.

under consideration. However, this argument could be reversed by considering the influence of local regions of hard soil.

- Absolute pile diameter

Jardine & Chow (1996), in the MTD (Marine Technology Directorate) design method for offshore piles, recommend a reduction factor on pile diameter. This was selected to provide a good fit with the database of load test results assembled by Chow (1996) (Figure 2). The origin of this scale effect is not linked to any mechanism, although it is suggested that shear bands may have an influence. The Chow (1996) database is reassembled in this paper and alternative conclusions are reached.

- Partial mobilisation

Lee & Salgado (1999) present reduction factors on CPT resistance to account for partial mobilisation of q_b by noting that the definition of q_b normally relates to a given settlement, rather than the 'plunging' load required for continued penetration. Finite element analysis is used to compare the proportion of ultimate pile capacity (which equals q_c, and is found by a cavity expansion method) mobilised at typical working settlements.

- Residual stresses

In addition, low *apparent* values of q_b arise if residual stresses are ignored. After the final blow or jacking stroke of installation the pile head rebounds. A larger displacement is required to unload the pile base than to reverse the shaft friction. Therefore, when the pile head reaches a state of equilibrium with the (zero) applied head load, the lower part of the pile remains in compression. A proportion of the base load is 'locked in', and balanced by negative shaft friction on the lower part of the shaft. It is common practice to re-zero pile instrumentation prior to a load test, to remove the influence of any instrument drift during driving. This leads to an under-prediction of base resistance and an over-prediction of shaft friction. Load tests on a jacked instrumented pile reported by Chow (1996) showed that approximately 50% of the ultimate base capacity was present as residual stress prior to load testing (Figure A.2). Load test results for displacement piles in which an initial base load of zero is reported should be treated with caution; a significant underestimate of q_b is likely.

In order to shed light on these possible differences between q_c and q_b, the database of compression load test results from closed-ended displacement piles in sand assembled by Chow (1996) has been re-evaluated from the original sources. The Chow database comprises open and closed-ended displacement piles in clay and sand. It has been selected as the basis for this paper since it represents the largest database of high quality pile load tests in the literature. This paper is concerned only with closed-ended piles in sand, for which field load test data from 28 pile tests at 12 sites was collated by Chow. For this paper, the original sources have been used to examine more closely the relationship between CPT and base resistance. The CPT soundings and load tests results are reproduced in Appendix 1.

Unit base resistance, q_b, has been evaluated according to two failure criteria: $D/10$ pile head settlement, and 'plunging' failure. 'Plunging' capacity is clearly defined in some tests, for which a constant penetration resistance was clearly reached. In other cases, where near-constant penetration resistance is reached, the maximum applied load has been chosen. This represents an under-estimate, which in most cases is by only a few percent if compared to an extrapolated curve. For each site the method of evaluating plunging capacity has been stated. CPT resistance, q_c, has been evaluated following Chow (1996) by averaging q_c over 1.5 pile diameters above and below the pile tip, with the exception of q_c at the Kallo and Lower Arrow Lake sites, for which a correction for partial embedment has been applied.

Site 1: Dunkirk (Chow, 1996) [DK]

Two compression tests on the Imperial College jacked instrumented pile are summarised in Table 1. It should be noted that q_c varies sharply (5-16 MPa) within +/- 0.2 metres of the level of test DK2/L1C (Appendix 1, Figure A.1), hindering selection of an appropriate value.

Test	*DK1/L1C*	*DK2/L1C*	*Source/notes*
Diameter (m)	0.1016	0.1016	D/10 = 10.16 mm
Pile tip depth (m)	7.40	5.96	Chow (1996)
q_c (av. +/- 1.5D) (MPa)	15.03	11.68	Chow (1996) Figure 7.4
Q_b (D/10 failure) (kN)	96	88	Chow (1996) Figure 7.30
q_b (D/10 failure) (MPa)	11.85	10.85	(DK2/L1C) and personal comm.
q_b/q_c (D/10 failure)	0.788	0.929	from Chow (2002) (DK1/L1C).
Q_b (plunging failure) (kN)	96	88	Q_b constant (+/- 5%) beyond
q_b (plunging failure) (MPa)	11.85	10.85	settlement of 4 mm
q_b/q_c (plunging failure)	0.788	0.929	
Chow (1996) interpretation			
q_c (av. +/- 1.5D) (MPa)	14.25	15	
Q_b (kN)	92 (s= 4.3mm)	79 (s= 3.0mm)	Failure defined as settlement, s,
q_b (MPa)	11.3	9.7	at τ=τ_{max} (i.e. D/35-D/20).
q_b/q_c	0.793	0.647	Q_b not fully mobilised.

Table 1. Dunkirk data.

Site 2: Labenne (Lehane, 1992) [LB]

Two compression tests on the Imperial College jacked instrumented pile are summarised in Table 2. Test LB2/L1C was conducted close to the base of a dense layer (Appendix 1, Figure A.3). Q_b was reducing sharply during installation to this depth. The load test became unstable after a settlement of 3.5 mm.

Test	*LB1/L1C*	*LB2/L1C*	*Source/notes*
Diameter (m)	0.1016	0.1016	D/10 = 10.16 mm
Pile tip depth (m)	5.95	1.83	Lehane (1992)
q_c (av. +/- 1.5D) (MPa)	4.1	6.2	Measured: Lehane (1992) Figure 6.2
Q_b (D/10 failure) (kN)	-	-	Values from Lehane (1992) Table
q_b (D/10 failure) (MPa)	4.7	4.3	6.2 for settlement of 20 mm.
q_b/q_c (D/10 failure)	1.15	0.69	Figure 6.16 indicates Q_b remained
			constant beyond s= 7 mm during
Q_b (plunging failure) (kN)	-	-	LB1/L1C; same Q_b used for D/10
q_b (plunging failure) (MPa)	4.7	4.3	and plunging failure.
q_b/q_c (plunging failure)	1.15	0.69	
Chow (1996) interpretation			
q_c (av. +/- 1.5D) (MPa)	4.7	6.2	
Q_b (kN)	36	37	Values differ from Lehane (1992)
q_b (MPa)	4.4	4.52	
q_b/q_c	0.936	0.729	

Table 2. Labenne data.

Site 3: Kallo (De Beer *et al.* 1979) [K]

6 compression load tests on Franki-piles with expanded concrete bases are reported, plus a large (250 mm diameter) CPT probe (Table 3). All tests were conducted at a shallow embedment (<1.6 m) into dense sand underlying soft clay and peat. The interface between

these strata lies at a depth of approximately 8.2 m, and is characterised by a ≈50-fold change in CPT resistance.

De Beer *et al.*'s paper focuses on the effect of such shallow embedment into a bearing stratum. This point is not considered by Chow (1996), who uses the Kallo data to validate the Jardine & Chow (1996) design approach which alternatively features a scale effect on absolute diameter (not normalised by embedment). The 'full' q_b available in the dense sand is not mobilised in the case of shallow embedment, since the overlying soft soil is still 'felt' by the pile base. The local q_c must be scaled down accordingly.

In this paper a scaling procedure for two-layer soil based on the approach described by Meyerhof (1976) and Valsangkar & Meyerhof (1977) has been used to select an appropriate average q_c based on the two strata for a pile embedded at depth z_b into a hard stratum. The strata at Kallo have been idealised as having uniform q_c of 0.5 MPa and 25 MPa respectively, to allow this simple calculation method to be used (Appendix 1, Figure A.5). A linear variation in corrected q_c over 10 pile diameters beginning two diameters above the hard layer has been chosen, based on Meyerhof (1976) and Valsangkar & Meyerhof (1977) which indicate that the zone of influence extends between zero and four diameters above the strata interface (Equation 1, Figure 1).

It should be noted that the resulting values of mean q_c in Table 3 are very sensitive to the level at which the influence of the hard layer is first felt (taken as $2D$ in this case), due to the high strength differential at this site. Further discussion of this effect is included in the proceedings of the 1979 conference "Recent developments in the design and construction of piles", pp253-256.

$$q_{c,corrected} = q_{c,weak} + \frac{\left(q_{c,hard} - q_{c,weak}\right)\left(\frac{z_b}{D} + 2\right)}{10} \quad \text{for } -2 < \frac{z_b}{D} < 8 \qquad \text{Equation 1}$$

Test	*CPT250*	*I*	*II*	*III*	*IV*	*V*	*VII*	*Source/notes*
Diameter (m)	0.25	0.908	0.539	0.615	0.815	0.406	0.609	De Beer *et al.* (1979)
Pile tip depth (m)		9.69	9.71	9.82	9.80	9.33	9.37	Tables 1,2.
Embedment, z_b/D	5 (fig. 11)	1.41	1.97	2.06	1.60	3.22	2.25	De Beer *et al.* (1979) Tables 2,3
q_c (MPa)	17.65	8.68	10.0	10.2	9.14	13.0	10.7	Equation 1
Q_b (D/10 failure) (kN)	618.5	5800	2440	2890	4810	1390	2490	De Beer *et al.* (1979)
q_b (D/10 failure) (MPa)	12.6	8.96	10.7	9.73	9.22	10.7	8.55	Table 5. CPT250
q_b/q_c (D/10 failure)	0.71	1.03	1.07	0.95	1.01	0.82	0.80	from Figure 11
Q_b (plunging failure) (kN)	618.5	5800	2440	2890	4810	1390	2490	Extrapolation of load- settlement
q_b (plunging failure) (MPa)	12.6	8.96	10.7	9.73	9.22	10.7	8.55	curve indicates
q_b/q_c (plunging failure)	0.71	1.03	1.07	0.95	1.01	0.82	0.80	<10% additional capacity. D/10 values adopted (conservative)
Chow (1996) interpretation								Local q_c at pile tip
q_c **(NOT av. +/- 1.5*D*)** (MPa)	21*	24.1	30	24.5	22.1	24.5	25.5	from De Beer Figure
Q_b (kN)	618.5	5800	2440	2890	4810	1390	2490	11. No averaging.
q_b (MPa)	12.6	8.96	10.7	9.73	9.22	10.7	8.55	*CPT250 q_c misread:
q_b/q_c	0.6	0.37	0.36	0.40	0.42	0.44	0.34	Original ref: q_c = 20.2 MPa

Table 3. Kallo data.

Site 4: Hunter's Point (Briaud *et al.* 1989) [HP]

The maintained load test on a single closed-ended steel tubular pile hammer driven into sand reported by Briaud *et al.* (1989) is summarised in Table 4. The response is notably soft, with the D/10 capacity differing from the plunging load by 24% (Appendix A1, Figure A.8).

Test	*HP1*	*Source/notes*
Diameter (m)	0.273	$D/10$ = 27.3 mm
Pile tip depth (m)	7.78	
q_c (av. +/- 1.5D) (MPa)	7.2	Briaud *et al.* (1989) Figure 2.
Q_b (D/10 failure) (kN)	289	Briaud *et al.* (1989) Figures. 5, 7, 9.
q_b (D/10 failure) (MPa)	4.94	
q_b/q_c (D/10 failure)	0.69	
Q_b (plunging failure) (kN)	359	Briaud *et al.* (1989) p1123.
q_b (plunging failure) (MPa)	6.13	
q_b/q_c (plunging failure)	0.85	
Chow (1996) interpretation		
q_c (av. +/- 1.5D) (MPa)	7.2	
Q_b (kN)	289	
q_b (MPa)	4.94	
q_b/q_c	0.69	

Table 4. Hunter's Point data.

Site 5: Baghdad (Altaee *et al.* 1992, 1993) [BG]

Table 5 summarises compression tests on two driven square precast concrete piles. Correction for residual stresses was carried out in the original references following Fellenius (1989).

Test	*Pile 1*	*Pile 2*	*Source/notes*
Equivalent diameter (m)	0.285	0.285	$D/10$ = 28.5 mm
Pile tip depth (m)	11.0	15.0	
q_c (av. +/- 1.5D) (MPa)	6	6.6	Altaee *et al.* (1992), Figure 3a.
Q_b (D/10 failure) (kN)	342	465	Altaee *et al.* (1993), Table 5 gives Q_{tot}, Q_b for
q_b (D/10 failure) (MPa)	5.36	7.29	Q_{tot} = 1000, 1600kN on pile 1 & 2 respectively.
q_b/q_c (D/10 failure)	0.89	1.10	From Figure 5, at s= D/10, Q_{tot} = 950, 1550 kN
			respectively. Q_b has been factored accordingly.
Q_b (plunging failure) (kN)	396	480	Altaee *et al.* (1992), Figure 4: Q_{tot} =1100 kN at
q_b (plunging failure) (MPa)	6.21	7.52	s= 120 mm for pile 1. Q_b found by factoring as
q_b/q_c (plunging failure)	1.04	1.14	above. Pile 2 max load: 1600kN: (s= 33.2 mm)
			this (conservative) value used as plunging load.
Chow (1996) interpretation			
q_c (av. +/- 1.5D) (MPa)	7	7.4	
Q_b (kN)	370	400	
q_b (MPa)	5.8	6.27	
q_b/q_c	0.83	0.85	

Table 5. Baghdad data.

Site 6: Akasaka (BCP Committee 1971) [AK]

Three load tests on instrumented steel closed-ended piles from the research programme reported by the BCP Committee (1971) are included in the Chow (1996) database (Table 6). In tests 1C and 6B the pile was installed by jacking. Test 6C was hammer driven. The tests were conducted with the tip of the pile at a shallow embedment into a hard layer, although a clear transition into this stratum is not clear from the CPT profile (Appendix 1, Figure A.10). SPT N-values of 30 and >60 were recorded at depths of 10.5 and 12.5 m respectively. CPT probes ended (or reached refusal) at a depth of 11.5 m. Selection of an appropriate mean q_c is difficult, due to the high variation in q_c in the region 10-12 m depth. The values quoted in Table 6 were found by digitising the original reference and averaging over +/- 1.5 D.

A non-standard 43.7 mm diameter CPT probe was used. If this value is to be used in the Chow (1996) correlation for base resistance, the measured value of q_c should possibly be factored up by 1.05 in order to represent an appropriate value for a standard 35.7 mm diameter cone. This correction arises since the reduction factor in the Jardine & Chow (1996) design method for base resistance is calculated as 1-0.5 log (d_{CPT}/D), where d_{CPT} is the diameter of a standard cone. This adjustment is not explicitly made in the Chow (1996) database.

Test	*1C*	*6B*	*6C*	*Source/notes*
Diameter (m)	0.20	0.20	0.20	D/10 = 20 mm
Pile tip depth (m)	11.0	4.0	11.0	
q_c (av. +/- 1.5D) (MPa)	29.8	8.06	29.8	BCP committee (1971), Figure 2
Q_b (D/10 failure) (kN)	560	135	590	BCP committee (1971), Figures 8,9
q_b (D/10 failure) (MPa)	17.83	4.3	18.78	
q_b/q_c (D/10 failure)	0.60	0.53	0.63	
Q_b (plunging failure) (kN)	830	200	640	Pile head load continues to increase as pile enters denser material. Unloading cycles hide trend. Plunging load taken at s= D.
q_b (plunging failure) (MPa)	26.08	6.37	20.37	
q_b/q_c (plunging failure)	0.87	0.81	0.68	
Chow (1996) interpretation				
q_c (av. +/- 1.5D) (MPa)	30	7.85	30	
Q_b (kN)	525	125	561	
q_b (MPa)	16.71	3.98	17.86	
q_b/q_c	0.56	0.51	0.60	

Table 6. Akasaka data.

Site 7. Drammen (Gregersen *et al.* 1973) [D]

Two compression tests on an instrumented pre-cast cylindrical concrete pile are reported by Gregersen *et al.* (1973) (Table 7). Strain gauges were used to measure residual loads directly, although zero-drift was observed. During load testing, Q_s in compression appears to be 50-100% greater than in tension (Gregersen *et al.*, 1973, Figure 5), indicating that residual stresses may be present, leading to an underestimate of Q_b (and an over-estimate of Q_s in compression), as noted by Chow (1996). In addition, during each stage of the load test, shaft friction does not reach a limiting value even at high settlement. This suggests that some component of base resistance is included in the recorded shaft friction.

In this analysis, a simple attempt has been made to correct for residual stresses, by assuming that Q_s is equal in compression and tension. The small difference between Q_s in compression and tension attributed by De Nicola & Randolph (1993) to Poisson's strains in the pile has been ignored in this simple analysis. The plunging capacity is difficult to establish since regular unload-reload loops interrupt the development of ultimate load. The capacity is increasing at the end of each test. The maximum applied load has been used as plunging capacity, which is likely to be a 5-15% under-prediction of correct value.

Site 8. Arkansas (Mansur & Hunter, 1970; Coyle & Castello, 1981) [A]

Four of the compression load tests reported by Mansur & Hunter (1970) are included in the Chow (1996) database, using the corrections made for residual stresses by Coyle & Castello (1981) (Table 8). Borehole logs in Mansur & Hunter (1970) indicate SPT values in the range N= 32 to N= 50 for 0.8 feet penetration in the vicinity of the test piles. Considering this wide variation in SPT value, the Coyle & Castello (1981) values of relative density, D_r have been used to infer CPT resistance following Lunne & Christoffersen (1983), on the assumption that Coyle & Castello's inferred D_r values are based on additional site investigation data.

Load-settlement curves are not available for tests 1 & 3. The load-settlement curve for test 2 indicates a continuing increase in capacity beyond $s = D/10$, preventing reliable estimation of the 'plunging' load. Test 10 was halted prior to settlement of $D/10$ (Coyle & Castello extrapolate this curve to estimate $D/10$ capacity). Therefore, plunging load has not been estimated for this paper.

It should be noted that the instrumentation channels comprise 30% of the base area of piles 2 and 10. These channels were tapered close to the base, over a distance of 600 mm. The lowest strain gauges, which were used to estimate base resistance, lie half-way up this taper (Mansur & Hunter 1970, Figure 6). Therefore, an effective cross-sectional area comprising half of the instrumentation channel area in addition to the pile circular area has been used to calculate q_b in Table 8.

Test	*Pile A*	*Pile D/A*	*Source/notes*
Diameter (m)	0.28	0.28	
Pile tip depth (m)	8	16	
q_c (av. +/- 1.5D) (MPa)	2.80	5.10	Gregersen *et al* (1973) Figure 2
Q_b (D/10 failure) (kN)	161	211	Pile A tension test: Q_t = 92 kN @ s= 18 mm (end of test)
q_b (D/10 failure) (MPa)	2.61	3.43	Pile A compression test: Q= 253 kN @ s= D/10 (Figure 5)
q_b/q_c (D/10 failure)	0.93	0.67	Pile D/A tension test: Q_t = 240 kN @ s= D/10 (Figure 5)
			Pile D/A compression test: Q= 451 kN @ s= D/10 (Figure 5)
Q_b (plunging failure) (kN)	175	222	Pile A maximum applied load: Q= 267 kN (Figure 5)
q_b (plunging failure) (MPa)	2.84	3.61	Pile D/A maximum applied load: Q= 462 kN (Figure 5)
q_b/q_c (plunging failure)	1.01	0.71	
Chow (1996) interpretation			
q_c (av. +/- 1.5D) (MPa)	2.75	5	
Q_b (kN)	69	118	
q_b (MPa)	1.12	1.92	
q_b/q_c	0.41	0.38	

Table 7. Drammen data.

Test	*Pile 1*	*Pile 2*	*Pile 3*	*Pile 10*	*Source/notes*
Diameter (m)	0.324	0.406	0.508	0.406	Mansur & Hunter 1970
Pile circular area (m^2)	0.0824	0.1295	0.2027	0.1295	Table 2
Inst. channels area (m^2)	0.0221	0.0616	0.0353	0.0616	
Pile area (m^2)	0.0935	0.1603	0.2204	0.1603	Including 50% of the instrumentation channels
Pile tip depth (m)	16.18	16.09	16.15	16.15	Coyle & Castello (1981), Table 3
Relative density, D_r (%)	70	60	70	60	Ditto
Vert. eff. stress, σ'_{vo} (kPa)	151.4	147.5	150.9	147.8	Ditto
q_c (av. +/- 1.5D) (MPa)	16.47	12.51	16.45	12.52	From D_r and σ'_{vo}, Lunne & Christofferson (1983)
Q_b (D/10 failure) (kN)	757	1068	1424	712	Coyle & Castello (1981), Table 3
q_b (D/10 failure) (MPa)	8.01	6.66	6.46	4.44	
q_b/q_c (D/10 failure)	0.49	0.53	0.39	0.35	
Chow (1996) interpretation					
Pile area (m^2)	0.090	0.146	0.211	0.146	
q_c (av. +/- 1.5D) (MPa)	16.89	16.89	16.89	16.89	
Q_b (kN)	757	1068	1424	712	
q_b (MPa)	8.39	7.32	6.76	4.88	
q_b/q_c	0.50	0.43	0.40	0.29	

Table 8. Arkansas data.

Site 9: Hoogzand (Beringen *et al.* 1979) [G]

A single load test on a closed-ended pipe pile reported by Beringen *et al.* (1979) is summarised in Table 9. Chow (1996) notes that in the conference discussion, the authors state that residual loads were corrected for, even though the shapes of the shear stress distributions suggest otherwise. The compression shaft capacity is approx. 25% greater than the tension capacity, indicating that base resistance could be underestimated. Furthermore, a base load measurement of zero is recorded at the start of the compression load test, indicating that any residual load has been ignored (Appendix 1, Figure A.14). The base load increased beyond a value of 13.3 MPa at $D/10$ tip settlement to a load of 15.2 MPa at a settlement of $D/7$, when the test was halted. The value of q_b was continuing to increase steadily, so no plunging capacity has been inferred.

Test	*Closed-ended pile*	*Source/notes*
Diameter (m)	0.356	
Pile tip depth (m)	6.75	
q_c (av. +/- 1.5D) (MPa)	28.7	Digitised from Beringen *et al.* (1979) Figure 4.
Q_b (D/10 failure) (kN)	1330 (inferred from q_b)	
q_b (D/10 failure) (MPa)	13.3	Beringen *et al.* (1979) Figure 18
q_b/q_c (D/10 failure)	0.46	(*tip* settlement, not head settlement)
Chow (1996) interpretation		
q_c (av. +/- 1.5D) (MPa)	28.7	
Q_b (kN)	1324	
q_b (MPa)	13.3	
q_b/q_c	0.46	

Table 9. Hoogzand data.

Site 10: Hsin Ta (Yen *et al.* 1989) [HT]

Three load tests are reported on 609 mm diameter closed-ended pipe piles (Table 10). One test pile, designated TP4, was loaded in compression to failure. A borehole log at the location of TP4 indicates that the pile base was located within a 1.5 m thick layer of clay (Yen *et al.*, Figure 1). Boreholes corresponding to the other test pile locations (55 – 70 m distant) show that the depths at which clay is present vary across the site. CPT probes conducted for other test piles show a reduction in q_c to 2-3 MPa within the clay layers. However, the CPT probe closest to pile TP4 does not capture a reduction in q_c at the level of the pile base (despite the presence of a clay layer in the borehole log at TP4) and so may not give an appropriate value (Appendix 1, Figure A.15). The exact location of the CPT probe compared to pile TP4 and the borehole is not stated. The shape of the pile head load-settlement curve for TP4 shows the load at $D/10$ settlement to be comparable to plunging capacity.

Site 11: Seattle (Gurtowski *et al.* 1984) [S]

Two compression tests on octagonal concrete precast piles of nominal 24 inch (608 mm) diameter are reported (Table 11). Residual stresses are estimated from base load measurements of a nearby identical pile. This residual base load is approximately 12% of the back-analysed shaft capacity of the test piles. This is a surprisingly small proportion of shaft friction to have been retained after driving as a residual base load, suggesting this value is an underestimate. The piles were tested to a settlement of 2.5% of D, which could account for the low measured base resistance; $D/40$ has been used as the settlement criterion. CPT resistance was estimated following Burland & Burbidge (1985) (in Meigh, 1987). A mean value of N= 40 is found below 9 m depth (Gurtowki & Wu, 1984). Both piles are founded in silt and sand, for which Burland & Burbidge suggest q_c/N= 0.33, giving an estimate of q_c= 13.3 MPa (Table 11).

Test	*TP4*	*Source/notes*
Diameter (m)	0.609	
Pile tip depth (m)	34.25	
q_c (av. +/- 1.5D) (MPa)	7.9	q_c measured from Yen *et al.* Figure 1. Note: clay layer indicated in borehole log, but not evident in CPT record. Clay layers
Q_b (D/10 failure) (kN)	850	in nearby boreholes correspond to values of q_c = 2-3 MPa
q_b (D/10 failure) (MPa)	2.92	=> q_c may be 3-4 times over-estimated for test pile TP4.
q_b/q_c (D/10 failure)	0.37	
Q_b (plunging failure) (kN)	850	A Chin plot for pile TP5 (identical to TP4 but not in clay
q_b (plunging failure) (MPa)	2.92	layer and tested to only 20 mm settlement) indicates 27%
q_b/q_c (plunging failure)	0.37	greater capacity for TP5, suggesting that TP4 is influenced by a nearby clay layer (Yen *et al.*, Table 3).
Chow (1996) interpretation		
q_c (av. +/- 1.5D) (MPa)	8.03	
Q_b (kN)	890	
q_b (MPa)	3.06	
q_b/q_c	0.37	

Table 10. Hsin Ta data.

Test	*Pile A*	*Pile B*	*Source/notes*
Effective diameter (m)	0.61	0.61	Gurtowski & Wu (1984)
Pile tip depth (m)	29.9	25.6	
q_c (av. +/- 1.5D) (MPa)	13.3	13.3	SPT values converted following Burland & Burbidge (1985)
Q_b (D/40 failure) (kN)	-	-	
q_b (D/40 failure) (MPa)	3.83	3.21	Gurtowski & Wu (1984) table 1
q_b/q_c (D/40 failure)	0.29	0.24	
Chow (1996) interpretation			
q_c (av. +/- 1.5D) (MPa)	10.4	9.6	
Q_b (kN)			
q_b (MPa)	3.81	3.85	
q_b/q_c	0.37	0.40	

Table 11. Seattle data.

Site 12: Lower Arrow Lake (McCammon & Golder 1970) [E]

A compression load test was conducted on a steel pipe pile driven open-ended with regular coring of the soil plug (Table 12). The pile was filled with a concrete plug after first being loaded to measure shaft friction alone. The tip of the pile was embedded a short distance into a layer of fine dense silty sand (SPT *N*-value 49) overlain by clayey silt (SPT *N*-value 8) (McCammon & Golder, Figure 2).

The borehole log indicates that the dense sand layer begins at a depth of 144 feet, although the driving record of the pile does not show a significant increase in resistance at this point. Instead, a sharp increase in driving resistance is apparent at around 149 feet, although it is not clear whether this is prior or subsequent to construction of the concrete plug. During further driving of the now closed-ended pile a sharp increase in driving resistance commensurate with the transition into dense sand is apparent at a depth of 153 feet.

The site cross-section shows the top of the dense layer to be sloping at a gradient of 1:8, but the borehole location is not shown. Were the borehole to lie 50 feet ‘uphill’ of the test pile, the sand layer could lie at a depth of 149 feet at the pile location rather than the 144 feet shown in the borehole log, as could be tentatively assumed from the driving record. This would place the pile tip at an embedment of 6 feet, or 3 pile diameters, into the dense sand layer, for which some correction due to partial embedment into the bearing stratum should be applied (Equation 1, Figure 1).

CPT data is not available, so SPT values have been converted following Burland & Burbidge (1985). For the clayey silt layer, N= 8 and q_c/N = 0.2, giving an estimate of q_c= 1.6 MPa. For the fine dense silty sand, N= 50 and q_c/N= 0.4, giving q_c= 20 MPa. Using Equation 1, an appropriate mean value of q_c at an embedment of 3 pile diameters into the dense sand is 10.8 MPa.

Base capacity is derived by subtracting the shaft capacity measured in the initial open-ended test from the total load measured after construction of the concrete plug. The 500 ton capacity of the loading rig was reached at a pile head settlement of 2.5 inches (D/10 = 2.4 inches). Extrapolation of the load-settlement curve suggests plunging load was almost reached; D/10 values have been used as a conservative estimate.

Test	*Closed-ended pile*	*Source/notes*
Diameter (m)	0.61	2 feet (McCammon & Golder, 1970)
Pile tip depth (m)	47.24	155 feet (McCammon & Golder, 1970)
q_c (MPa)	10.8	SPT values converted following Burland & Burbidge (1985) and averaged for partial embedment following Equation 1.
Q_b (D/10 failure) (kN)	2781	From q_b
q_b (D/10 failure) (MPa)	9.58	McCammon & Golder (1970) Table 3
q_b/q_c (D/10 failure)	0.89	(1 tsf = 95.8 kN/m^2)
Q_b (plunging failure) (kN)	2781	From q_b
q_b (plunging failure) (MPa)	9.58	McCammon & Golder (1970) Table 3
q_b/q_c (plunging failure)	0.89	(1 tsf = 95.8 kN/m^2)
Chow (1996) interpretation		
q_c (av. +/- 1.5D) (MPa)	20.62	
Q_b (kN)	2750	
q_b (MPa)	9.41	
q_b/q_c	0.46	

Table 12. Lower Arrow Lake data.

Discussion

The load test data of q_b/q_c as interpreted by Chow (1996) to validate the Jardine & Chow (1996) design method for base resistance on closed-ended piles in sand is shown in Figure 2. The same data interpreted as described in this paper is shown in Figures 3 and 4, for which D/10 settlement and 'plunging' have been used to define failure respectively. The scale effect on absolute diameter is not apparent when the data are interpreted as described in this paper. Instead, q_b is typically slightly lower than q_c, but no trend with diameter is evident.

The outlying points on Figure 2, for which q_b/q_c < 0.5, comprise data from sites for which q_c has been estimated from SPT data, with the exception of the data point for Drammen for which residual loads are not fully accounted for. The selection of alternative empirical SPT-CPT correlations can alter the position of these points by a factor 2 in either direction. A more stringent acceptance criterion for pile tests to be included in this database would be to exclude sites for which actual CPT data is not available.

When considering only the load tests for which a 'plunging' capacity can be identified, the only data point for which q_b/q_c < 0.6 is from Hsin Ta. However, this test pile was located in a clay layer which is not captured in the CPT profile. If this result is ignored, a mean value of

q_b/q_c = 0.90 is found from the data set of 20 piles. If this relationship were used as a basis for the prediction of q_b at plunging failure, a mean ratio of predicted to measured capacity of 1.02 is found, with a standard deviation of 0.17 and a coefficient of variation (COV) of 0.17. This fit to the database in this paper is comparable with the fit between the Chow (1996) database and the Jardine & Chow (1996) design method for the base resistance of closed-ended piles in sand, using $q_b/q_c = 1 - 0.5 \log (D/d_{cpt})$, for which COV= 0.18.

This exercise demonstrates that databases of pile load test data should be treated with caution, and care should be taken to establish the methods used to extract the underlying load test data and ground conditions. However, the differences between Figures 2, 3 and 4 are not random, and cannot be entirely attributed to ambiguous historical field records. The majority of field records of low q_b/q_c which form the basis of the apparent scale effect on diameter evident in Figure 2 can be attributed to other factors:

- Partial embedment

  The load tests conducted at Kallo, Lower Arrow Lake and Akasaka comprise piles which are shallowly embedded in dense sand. At this shallow embedment the 'full' capacity of the dense stratum is not mobilised, and the pile tip 'feels' the overlying weak soil. Laboratory tests have shown that this effect can extend to an embedment of several pile diameters and can be accounted for using a correction of the form of Equation 1, illustrated in Figure 1 (Meyerhof, 1976; Valsankar & Meyerhof, 1977).

  Partial embedment is probably responsible for many further examples of recorded low values of q_b/q_c during pile load tests beyond the data assembled in this paper. Piles bearing in dense sand are usually installed only to a shallow embedment to prevent pile tip damage and driveability problems.

  Noting that several diameters of penetration are required to fully mobilise the strength of the hard layer, engineers are correct to design with $q_b/q_{c,local} < 1$ in these cases, and will observe the same in load tests. However, this should not be mistaken for a scale effect on absolute diameter, but relates to partial embedment. Installing the pile deeper into the bearing stratum would yield increased $q_b/q_{c,local}$ and higher capacity.

- Residual stresses

  The load test data from Seattle, Hoogzand, Drammen and Baghdad are influenced by residual stresses, in that the measurement of base resistance began from a zero value at the start of the load test (i.e. zero head load), even though some base resistance would have remained locked in by negative shaft friction.

  - The Baghdad data was corrected for residual base load by the original authors, and shows values of q_b/q_c close to unity.
  - The Drammen data has been corrected in this paper using a simple method yielding values of q_b/q_c between 0.7 and 1 compared to an uncorrected value of 0.4.
  - Chow (1996) notes that the Hoogzand data shows slight evidence of residual stress errors. Although the original authors discuss zero drift and residual stresses, since the base load is recorded as zero at the start of the load test, any residual base load has been ignored. Plunging failure was not reached during this test.
  - The Seattle data is corrected for residual base load by the original authors using measurements from a nearby identical pile. However, the recorded value of 12% of the shaft friction appears low, casting doubt upon their degree of correction.

- Partial mobilisation

Plunging capacity was reached prior to a settlement of $D/10$ for 60% of the piles. The piles at Baghdad, Drammen, Hunter's Point and Akasaka showed differences between $D/10$ and plunging capacity. For a D/10 failure criterion, these sites show a mean q_b/q_c of 0.75, which rises to 0.89 for a plunging failure criterion. When assessing pile capacity according to the $D/10$ displacement failure criterion, the value is influenced by pile stiffness for this subset of 40% of the piles, with the chosen figure depending on the degree of partial mobilisation. For the remaining 60% of the database, the pile stiffness is sufficiently high to have no influence on the chosen value since the plunging capacity is reached prior to $D/10$ settlement.

In this paper, these three mechanisms have been accounted for by:

- Calculating appropriate values of q_b/q_c when the pile tip is at a shallow embedment in a bearing stratum by using Equation 1 to include the weakening contribution of the overlying layer when selecting q_c (Kallo and Lower Arrow Lake sites)
- Accounting for residual base load by using tension tests to estimate the compressive shaft capacity (Drammen site)
- Assessing pile capacity based on plunging load. Although this value is often not reached during load tests and requires a larger safety factor in design, it is a clear definition, and prevents pile stiffness clouding the measurement of ultimate pile strength, as is the case with a settlement criterion.

Following this methodology, it has been found from the database of field load tests assembled by Chow (1996), that no scale effect on q_b/q_c with absolute pile diameter is evident. Instead, plunging base resistance for this set of pile load test results is best estimated as 90% of q_c (corrected for partial embedment), and is independent of diameter.

This conclusion indicates that the ratio q_b/q_c is influenced by two of the mechanisms described in the introduction to this paper: partial embedment and partial mobilisation. An appropriate value of q_c at the pile tip to account for partial embedment can be selected by suitable consideration of the low values of q_c in the overlying weak layer. It should be noted that the strength differential between soft and hard layers is typically high, making the corrected value of q_c very sensitive to the weighting technique. Partial mobilisation can be accounted for by defining q_b as the plunging capacity, and selecting design safety factors (or more correctly *mobilisation* factors) appropriately. After removing these two effects, q_b is on average 10% lower than q_c. This effect could be attributed to local inhomogeneity, base-shaft interaction, or more probably to the conservative definition of plunging capacity as the maximum applied load in the load tests for which steady penetration under constant load was not reached.

Conclusions

The comprehensive database of load tests on closed-ended piles in sand presented by Chow (1996) has been reassembled from the original sources to examine the relationship between CPT resistance, q_c, and base capacity, q_b. In contrast to continuum analyses which predict that $q_b = q_c$ during steady penetration, reduction factors are often recommended such that $q_b/q_c < 1$ for design.

Two mechanisms to explain these reduction factors are partial embedment of the pile into the bearing stratum and partial mobilisation of base resistance. In this analysis, partial embedment has been accounted for by weighting q_c to account for overlying weak layers in the case of piles shallowly embedded into a bearing stratum. Partial mobilisation has been accounted for by defining failure according to a plunging criterion.

The resulting values of q_b/q_c have a mean value of 0.90 and show no trend with pile diameter, for the 20 load tests in which plunging load was identified and reliable values of q_c were available. This slight underprediction of the 'continuum' model ($q_b= q_c$) could be attributed to the underestimation of plunging load in pile tests for which steady penetration was not reached. This outcome challenges the advice in the MTD design method (Jardine & Chow, 1996) which proposes a reduction in ultimate end bearing capacity in sand based on pile diameter.

Notation

D	Pile diameter
N	SPT value
Q_s	Total shaft friction
Q_b	Total base resistance
q_b	Unit base resistance
q_c	(Unit) CPT tip resistance
$q_{c,local}$	(Unit) CPT tip resistance at pile base level (no weighting with depth)
s	Pile head settlement
z	Depth
z_b	Depth of embedment into hard layer

References

Altaee A., Fellenius B.H., Evgin E. 1992. Axial load transfer for piles in sand. I. Tests on an instrumented precast pile. Canadian Geotechnical Journal 29:11-20

Altaee A., Fellenius B.H., Evgin E. 1993. Load transfer for piles in sand and the critical depth. Canadian Geotechnical Journal 30:455-463

Baligh M.M. 1985 Strain path method. ASCE Journal of Geotechnical Engineering 111(9):1108-1136

BCP Committee 1971. Field tests on piles in sand. Soils and Foundations. 11(2):29-49

Beringen F.L., Windle D. & Van Hooydonk W.R. 1979. Results of loading tests on driven piles in sand. Proc. Conference on Recent Developments in the Design and Construction of Piles. ICE, London 213-225

Borghi X., White D.J., Bolton M.D. & Springman S. (2001) Empirical pile design based on CPT results: an explanation for the reduction of unit base resistance between CPTs and piles. Proc. 5th Int. Conf. on Deep Foundation Practice, Singapore. pp 125-132

Briaud J-L. 1988. Evaluation of cone penetration test methods using 98 pile load tests. Proc. Int. Symposium on Penetration Testing, ISOPT-1, Orlando. 2:687-697

Briaud J-L, Tucker L.M. & Ng E. 1989. Axially loaded 5 pile group and a single pile in sand. Proc. 12th International Conference on Soil Mechanics and Foundations Engineering, Rio de Janeiro. (2):1121-1124

Burland J.B. & Burbidge M.C. 1985. Settlement of foundations on sand and gravel. Proc. Institution of Civil Engineers. 78(1):1325-1381

Chow F.C. 1996. Investigations into the behaviour of displacement piles for offshore foundations. PhD dissertation, University of London (Imperial College)

Chow F.C. 2002. Personal communication.

Coyle H.M. & Castello R.R. 1981. New design correlations for piles in sand. ASCE Journal of Geotechnical Engineering. 197(GT7) 965-985

De Beer E.E., Lousberg D., De Jonghe A., Carpentier R. & Wallays M. 1979. Analysis of the results of loading tests performaed on displacement piles of different types and sizes penetrating at a relatively small depth into a very dense layer. Proc. Conf. on Recent Developments in the Design and Construction of Piles, ICE, London. 199-211.

De Nicola A. & Randolph M.F. 1993. Tensile and compressive shaft capacity of piles in sand. ASCE. Journal of Geotechnical Engineering 119(12): 1952-1973

Fellenius B.H. 1989. Prediction of pile capacity. Proc. Symposium on Predicted and Observed Behaviour of Piles. ASCE Special Geotechnical Publication 23:293-302

Gregersen O.S., Aas G. & Dibiagio E. 1973. Load tests on friction piles in loose sand. Proc. 8th Int. Conf. Soil Mechanics & Foundation Engineering, Moscow (2):109-117

Gurtowski T.M. & Wu M-J. 1984. Compression load tests on concrete piles in alluvium. Proc. Symposium on the Analysis and Design of Pile Foundations, San Francisco, ASCE. pp. 138-153.

Jardine R.J. & Chow F.C. 1996. New design methods for offshore piles MTD Publication 96/103, Marine Technology Directorate, London

Kraft L.M. 1990. Computing axial pile capacity in sands for offshore conditions. Marine Geotechnology 9:61-72

Lehane B.M. 1992. Experimental investigations of pile behaviour using instrumented field piles. PhD dissertation, Imperial College, University of London.

Lunne T. & Christofferson H.P. 1983. Interpretation of cone penetrometer data for offshore sands. Proc. Offshore Technology Conference, OTC4464, Houston 181-192.

Mansur C.I. & Hunter A.H. 1970. Pile tests- Arkansas River project. ASCE J. Soil Mechanics & Foundations Division. 96(SM5):1545-1582

McCammon N.R. & Golder H.Q. 1970. Some loading tests on long pipe piles. Geotechnique 20(2):171-184

Meyerhof G.G. 1976. Bearing capacity and settlement of pile foundations. ASCE Journal of Geotechnical Engineering. 102(GT3)197-228

Randolph M.F., Dolwin J. & Beck R. 1994. Design of driven piles in sand Géotechnique 44(3):427-448

Valsangkar A.J. & Meyerhof G.G. 1977. Bearing capacity of piles in layered soils. Proc. 8th Int. Conf. Soil Mechanics & Foundation Engineering, Moscow. (1):645-650

Winterkorn, A.F. & Fang, S.Y. 1975. Foundation Engineering Handbook, Van Nostrand Reinhold Co. New York.

Yen T.L, Lin H, Chin C-T & Wang R-F. 1989. Interpretation of instrumented driven steel pipe piles'. In: Proc. Congress on Foundation Engineering- Current Principles and Practice, Illinois, ASCE. pp1293-1308.

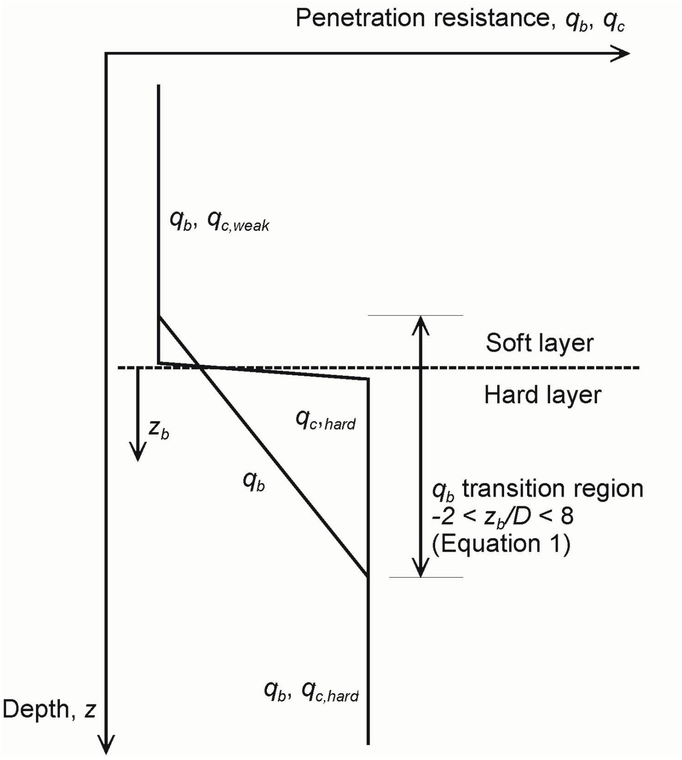

Figure 1. Partial embedment reduction factor on base resistance.

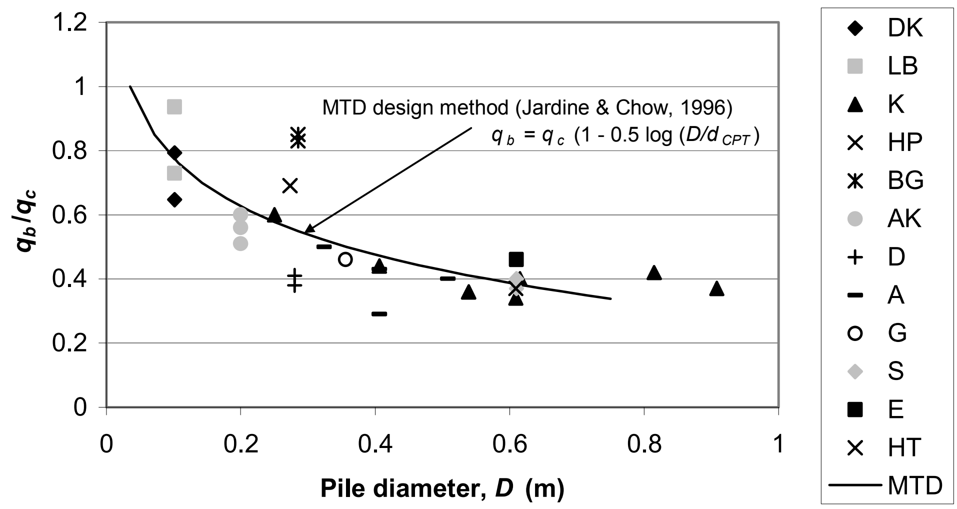

Figure 2. Normalised pile base resistance vs. pile diameter (Chow, 1996; Failure: D/10 settlement)

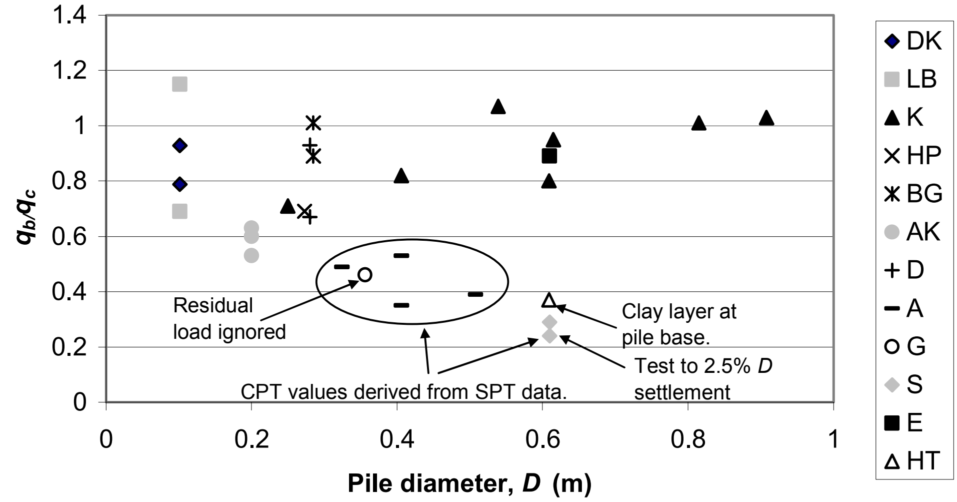

Figure 3. Normalised pile base resistance vs. pile diameter (White, 2003; Failure: D/10 settlement)

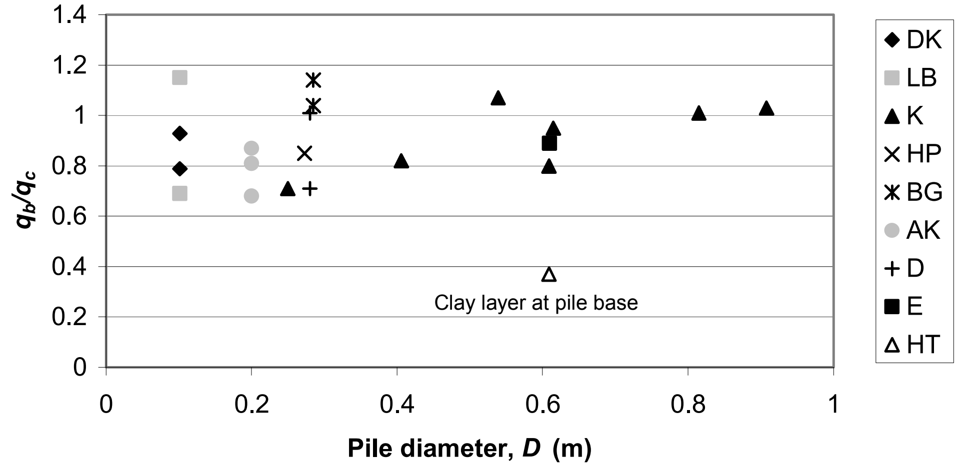

Figure 4. Normalised pile base resistance vs. pile diameter (White, 2003; Failure: plunging load)

Appendix 1: Cone penetration data and load test results

If available in the original reference, the cone penetration data and base load-settlement results from each site are reproduced in this Appendix. The 'design' cone resistance, q_c, taken as a local average (+/- 1.5 D) or using Equation 1 for partial embedment as described previously, is indicated on each load-settlement curve.

Site	*CPT profile, q_c*	*Base load –settlement curves, q_b-s*
Dunkirk (Chow (1996)	Figure A.1	Figure A.2
Labenne (Lehane, 1992)	Figure A.3	Figure A.4
Kallo (De Beer *et al*., 1979)	Figure A.5	Figure A.6
Hunter's Point (Briaud *et al*., 1989)	Figure A.7	Figure A.8
Baghdad (Altaee *et al*., 1992, 1993)	Figure A.9	Not given in original reference
Akasaka (BCP Committee, 1971)	Figure A.10	Figure A.11
Drammen (Gregersen *et al*., 1973)	Figure A.12	Failure load corrected for residual load in this paper. Original uncorrected data not shown.
Arkansas (Mansur & Hunter, 1970)	SPT	Original data uncorrected for residual load- not shown.
Hoogzand (Beringen *et al*., 1979)	Figure A.13	Figure A.14
Hsin Ta (Yen *et al*., 1989)	Figure A.15	Not given in original reference
Seattle (Gurtowski *et al*., 1984)	SPT	Not given in original reference
Lower Arrow Lake (McCammon & Golder, 1970)	SPT	Base capacity estimated in original reference by comparing open and close-ended tests

Table A.1. List of CPT profiles and base load-settlement figures.

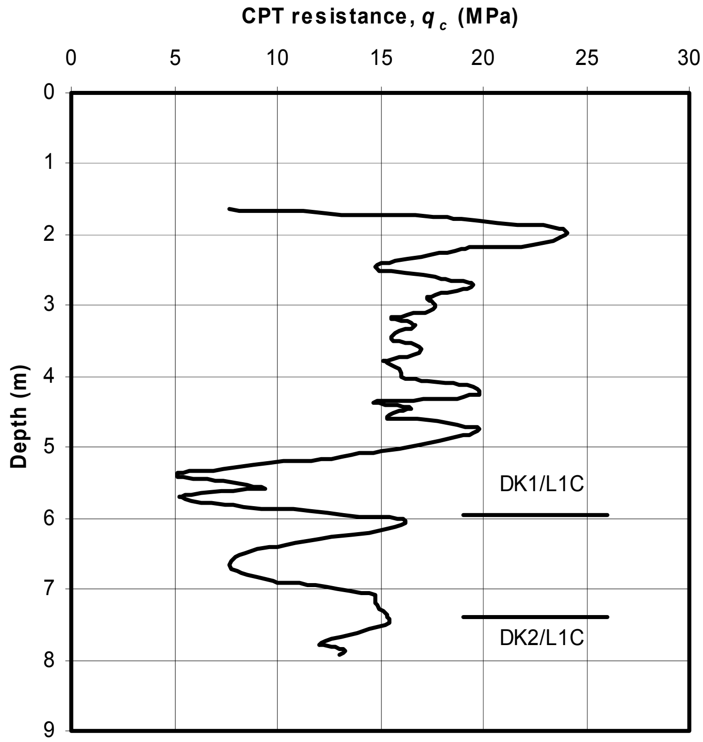

Figure A.1. Dunkirk CPT profile (after Chow, 1996)

Figure A.2. Dunkirk base load-settlement response (after Chow, 1996; Chow 2002)

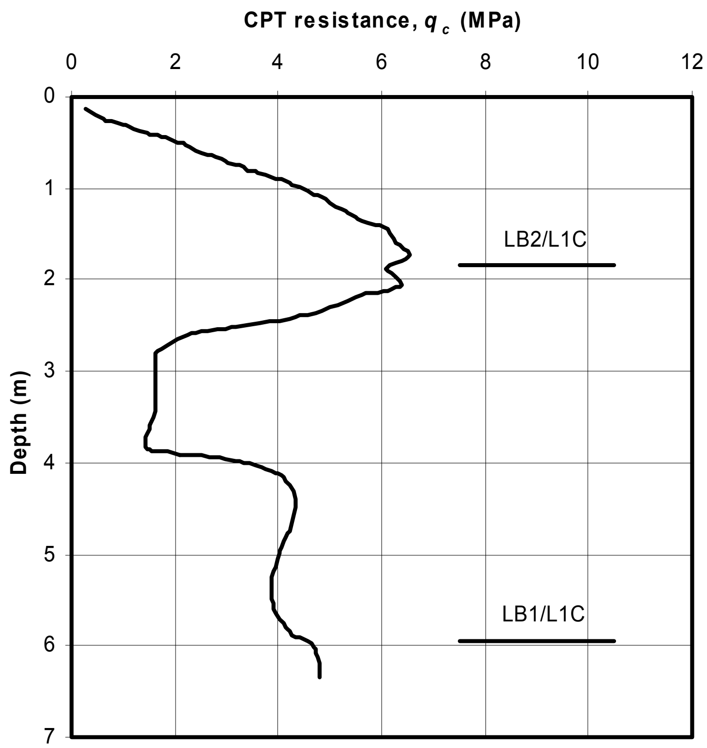

Figure A.3. Labenne CPT profile (after Lehane, 1992)

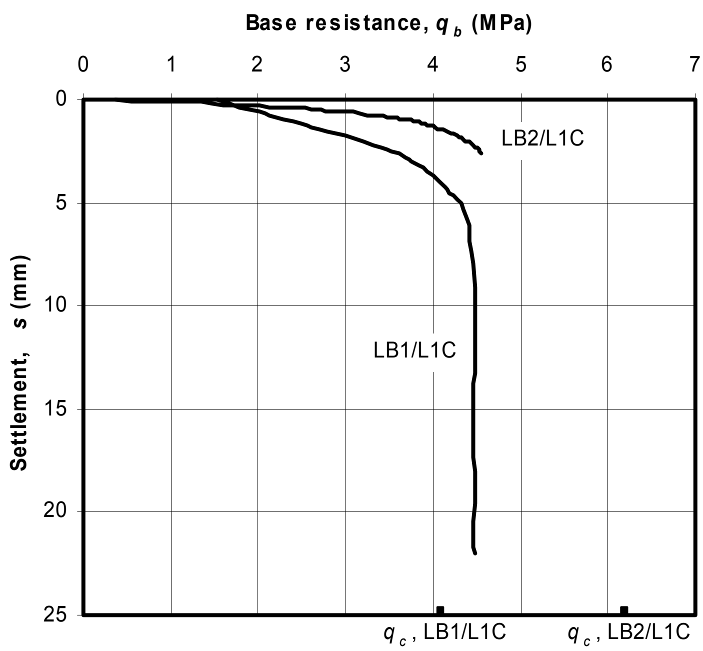

Figure A.4. Labenne load-settlement response (after Lehane, 1992)

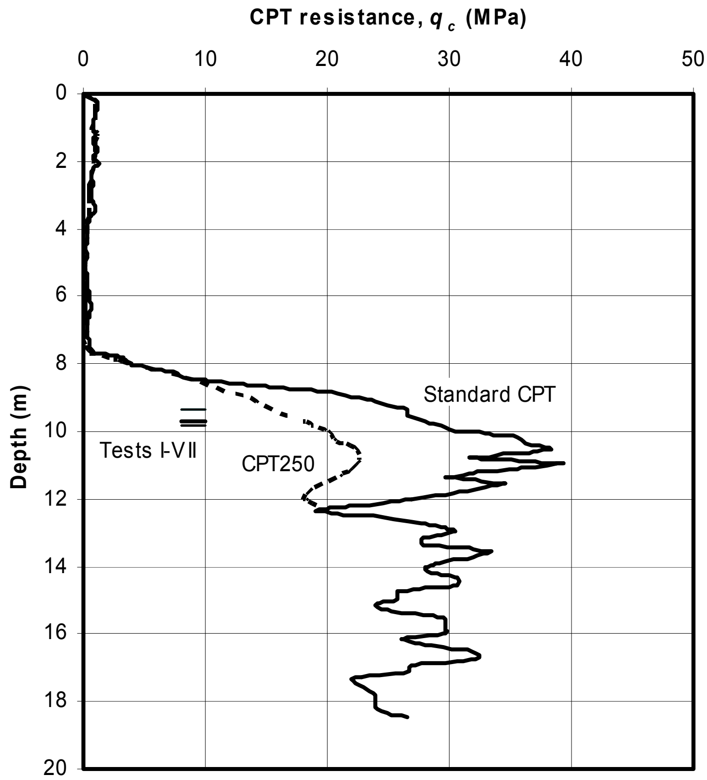

Figure A.5. Kallo CPT profile (after De Beer *et al*., 1979)

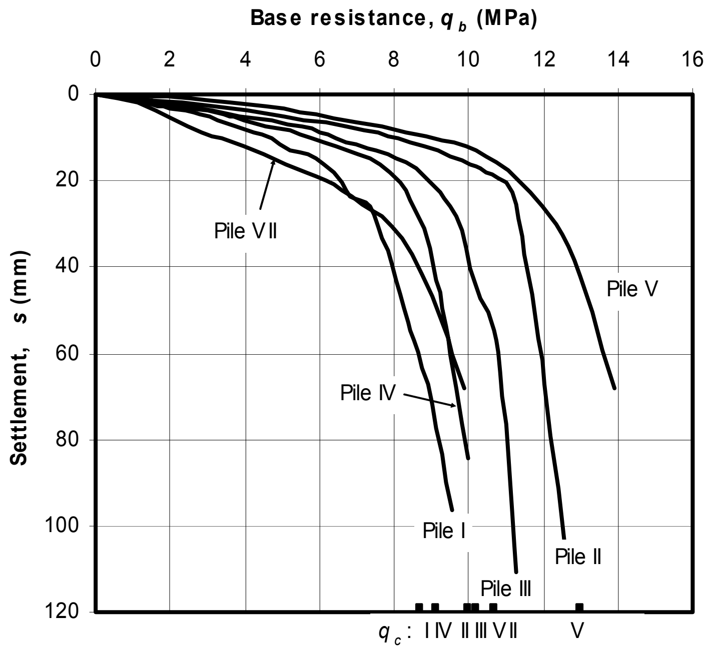

Figure A.6. Kallo base load-settlement response (after De Beer *et al*., 1979)

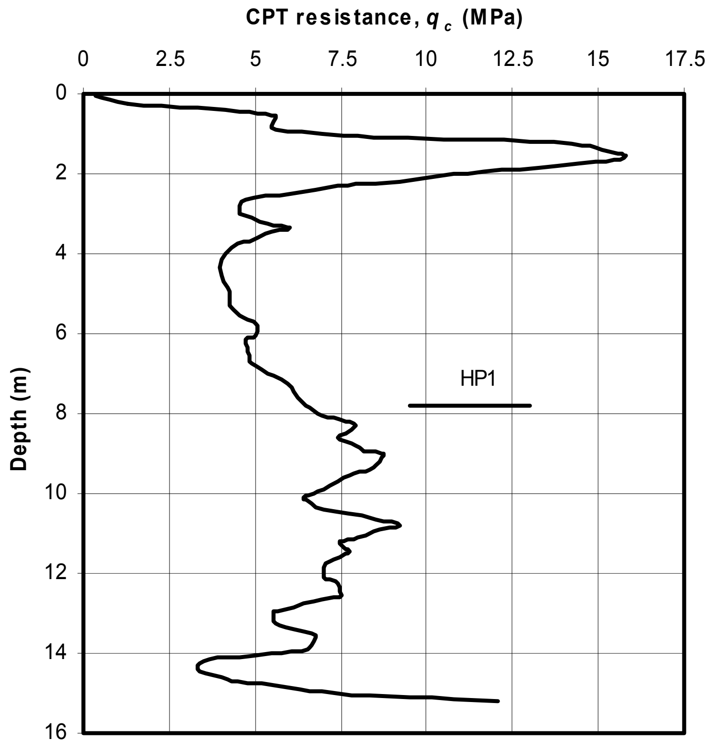

Figure A.7. Hunter's Point CPT profile (after Briaud *et al.*, 1989)

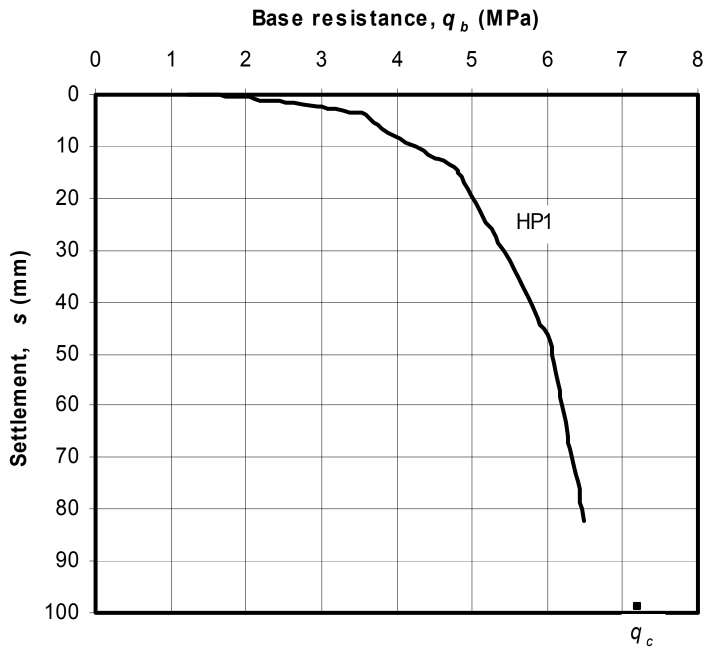

Figure A.8. Hunter's Point base load-settlement response (after Briaud *et al.*, 1989)

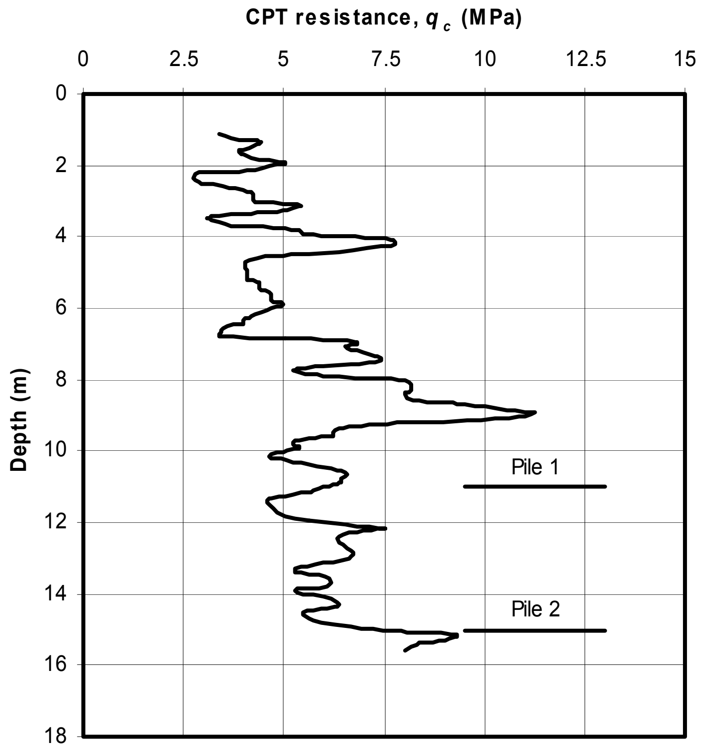

Figure A.9. Baghdad CPT profile (after Altaee *et al.*, 1992)

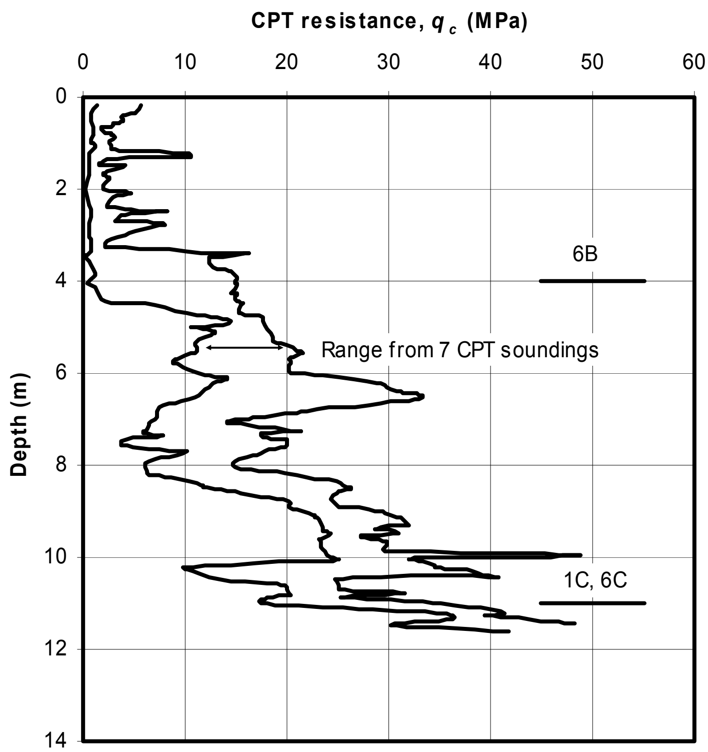

Figure A.10. Akasaka CPT profile (after BCP Committee, 1971)

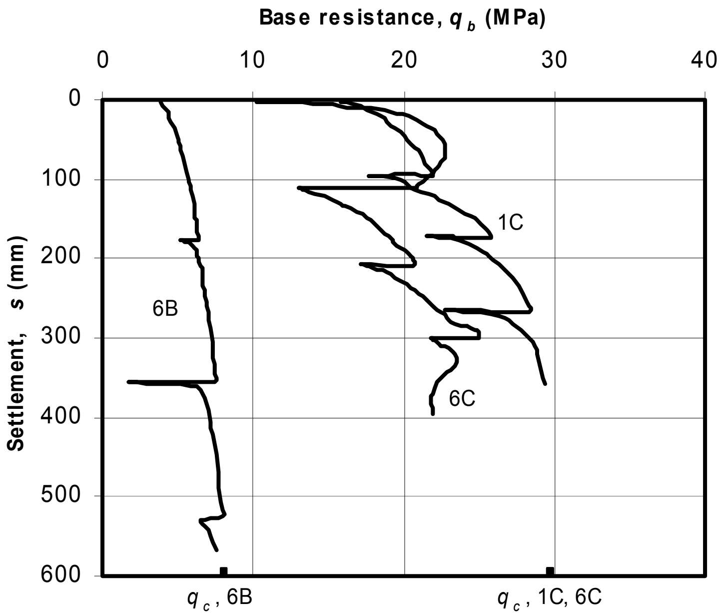

Figure A.11. Akasaka base load-settlement response (after BCP Committee, 1971)

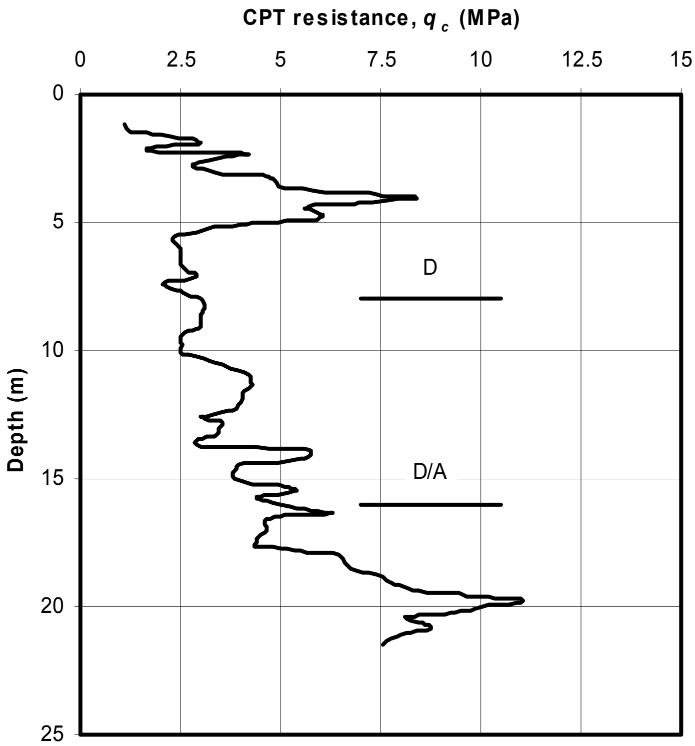

Figure A.12. Drammen CPT profile (after Gregersen *et al.*, 1973)

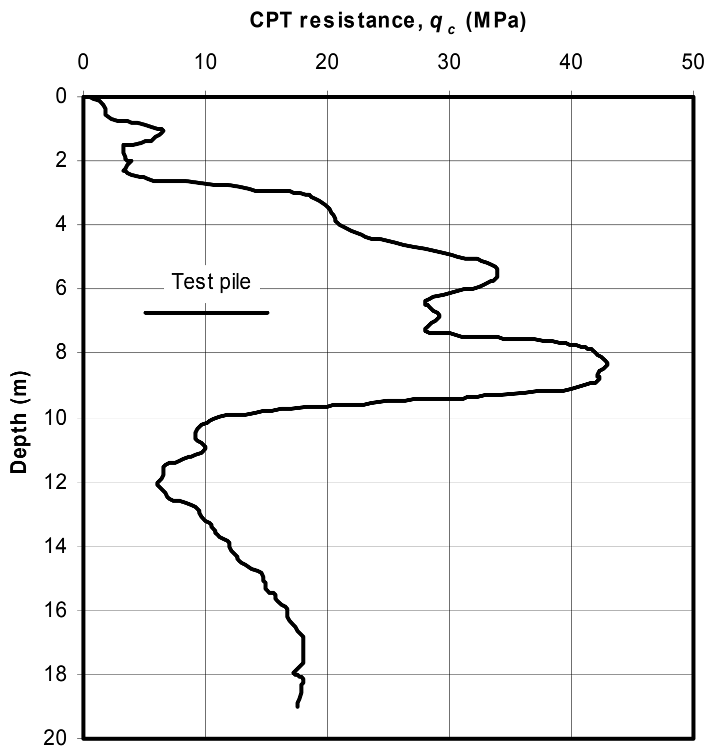

Figure A.13. Hoogzand CPT profile (after Beringen *et al.*, 1979)

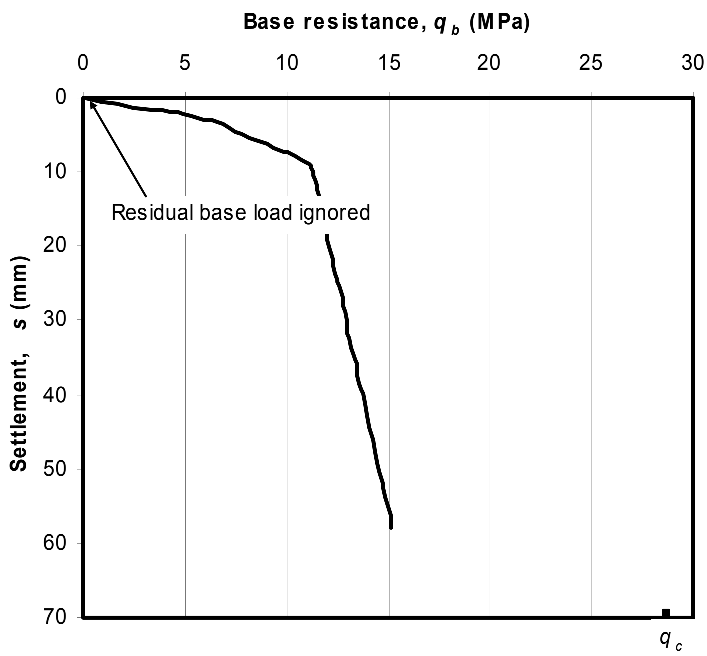

Figure A.14. Hoogzand base load-settlement response (after Beringen *et al.*, 1979)

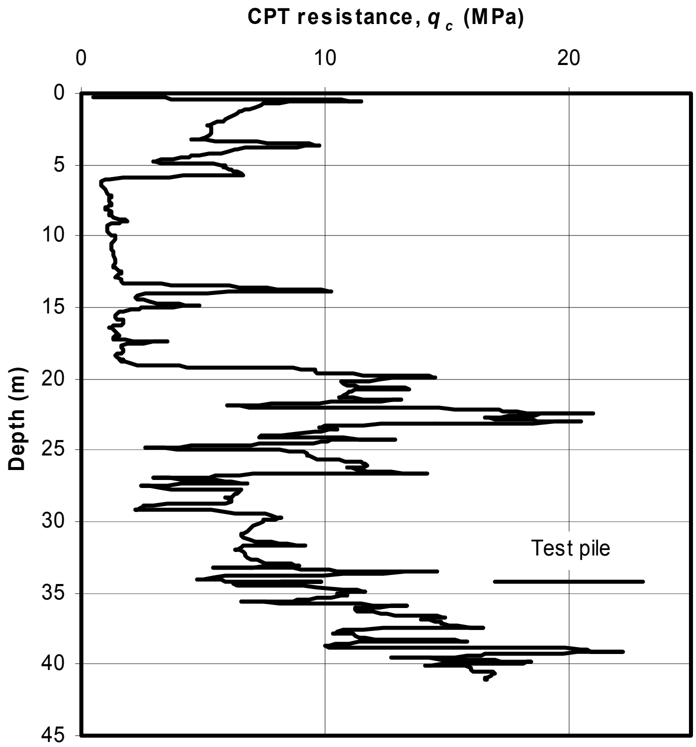

Figure A.15. Hsin Ta CPT profile (after Yen *et al.*, 1989)