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A new computer program for pile capacity prediction using CPT data

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Abstract. An interactive computer program "GLAMCPT" is developed for application in soil profiling and prediction of pile load capacity using cone penetration test (CPT) and laboratory soil test results. GLAMCPT calculates pile capacity according to 10 selected methods from European design codes, refereed international publications and recommendations of professional institutions. To demonstrate the capabilities of the program, a database of comprehensive ground investigation and full-scale pile tests in sand, at a Belgian site, is analysed using GLAMCPT. The database comprises 11 static tests and 12 dynamic tests on piles of different construction techniques, including driven pre-cast concrete piles and screwed cast in-situ piles, installed using 5 different procedures. Prior to pile installation, CPTs were carried out at each proposed pile location. Comparison of GLAMCPT predictions with the observed pile capacities reveals that the most accurate of the existing methods yields an average, μ, of predicted to observed pile head capacity $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ equal to 0.94. The most consistent method produces a coefficient of variation (COV) of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ equal to 0.1 and ranking index (RI) of 0.08. Parametric studies have been carried out using GLAMCPT to formulate an improved predictive method, which yielded: $\mu = 0.99$, COV = 0.07 and RI = 0.04.

Key words. computer software, cone penetration test, pile capacity, soil profiling.

1. Introduction

This paper contains numerical analyses of recently acquired data from 11 static and 12 dynamic tests on full-scale piles installed in sand using different techniques. Extensive ground investigation data are assembled from: (i) cone penetration tests (CPT); (ii) standard penetration tests (SPT); (iii) dilatometer tests (DMT); (iv) pressuremeter tests (PMT); and (v) borehole logs. The load tests were undertaken as part of a research programme by the Belgian Building Research Institute (BBRI) in the period from 2000 to 2002. The first author was invited by BBRI to participate in an international pile prediction event held in 2001. The event was convened subsequent to ground investigation and test pile installation, but prior to pile testing. Participants were allowed to use their preferred choice of ground investigation data and/or dynamic test results to predict the load capacities and load-settlement behaviour of the static test piles, and submit their report in advance of the static load testing activity.

According to the European Regional Technical Committee on piles (ERTC3 1997), *in-situ* tests are the most widely used methods of ground investigation, for purposes of pile foundation design. Laboratory soil testing is not targeted directly at facilitating pile design but rather to assist site characterisation.

A dual-purpose computer program "GLAMCPT" is developed for use in processing CPT data from any site in order to interpret the soil profile and predict pile load capacity. The main focus is on CPT because:

1. it is the most commonly used method for soil investigation in Europe, as reported by ERTC (1999) and De Cock (1998).
2. it is rapid, economical and generates continuous soil strength data, including pore water pressures, typically at 5 cm depth intervals so that the need for "undisturbed" soil sampling and time-consuming laboratory tests can be eliminated or minimised.
3. pile load capacity can be interpreted directly or indirectly from CPT data using a variety of calculation methods which can be refined through back-analysis.
4. soil parameters (such as relative density, over-consolidation ratio, stress–strain modulus, shear modulus, consolidation parameters, earth pressure coefficient) which are required in sophisticated methods of pile analysis, are difficult to measure accurately but can be interpreted from CPT results.

There are several methods for predicting pile load capacity directly from CPT results. According to Eurocode 7 (1997), the calculation method appropriate for a given situation should have been verified by static load tests in soil conditions comparable to the one at hand. Following this guideline, 10 methods recommended in national codes of practice, leading geotechnical journals and publications of professional institutions were coded for incorporation in GLAMCPT. The program is applied to analyse the vast CPT data and to predict the load capacities of the static test piles, using all 10 methods simultaneously.

In parallel, data from the dynamic pile tests are analysed using the commercial wave analysis program "CAPWAP" (Goble and Rausche, 1979) in order to predict the load capacity of the statically tested piles. Based on the static pile test results, the performance of the CPT and CAPWAP-based methods is assessed for the differently constructed piles. The prime objective is to develop an improved prediction method by studying some of the important factors influencing pile resistance mobilisation.

2. Database of soil data and test piles

The test area covers a 52 m by 24 m site in Limelette, 30 km south of Brussels, Belgium. A programme of ground investigation and pile testing was carried out by BBRI (Maertens and Huybrechts, 2003). The soil profile comprised: (a) 0.00–0.40 m: Recent FILL (b) 0.40–8.00 m: Quaternary sandy SILT or *silty* SAND (loam), and (c) 8.00–14.00 m: Tertiary *Bruxellian* and/or *Ledian* SAND. The water table was located at approximately 40 m depth.

Various methods of ground investigation were used, including (i) CPT (using mechanical and electrical cones), (ii) Standard penetration tests, (iii) Dilatometer tests, (iv) Pressuremeter tests, (v) Seismic refraction tests and (vi) boreholes laboratory soil testing. This paper focuses on the processing and application of CPT data to soil profiling and pile capacity prediction. The CPT data were acquired using standard electrical cones of 10 cm^2 base area with apex angles of 60°. At each pile location, CPT was carried out from ground level up to 15.95 m depth, with measurements recorded at 0.05 m depth intervals.

The layout of the test site is illustrated in Figure 1. A total of 32 piles, of 380–410 mm in diameter by 9.13–9.59 m in length were installed. These include piles $A1_{bis}$ and $C1_{bis}$, which were later constructed close to piles A1 and C1, respectively. The test programme comprised; (a) static tests on 11 piles, (b) dynamic tests on 12 piles and (c) quasi-static tests (commonly referred to as "statnamic" test) on 6 piles. This paper, focuses on the analysis of the static and dynamic test piles only. Pertinent attributes of the piles are given in Table 1. The piles were of 6 types: (1) pre-fabricated driven concrete piles and soil-displacement screwed, cast-in-place piles of types: (2) Fundex, (3) Olivier, (4) Omega, (5) DeWaal and (6) Atlas. The procedures for the installation of the 5 types of screwed cast-in-place piles were fully described by Huybrechts (2001). The elevation of a typical pile (pile A2) and the cone resistance (q_c) profile at that site is illustrated in Figure 2(a)–(b). In addition to pile head displacement sensors, a full-length extensometer was installed, contained in a 50 mm diameter central duct, in each static test pile. The extensometer reading measured the compression of the pile under load. Thus, the base movement (net settlement) at any load could be obtained by subtracting the pile compression from the pile head displacement.

For the 11 static test piles, the expected failure load Q_{max} was theoretically calculated by BBRI engineers based on Belgian pile testing experience. It was planned

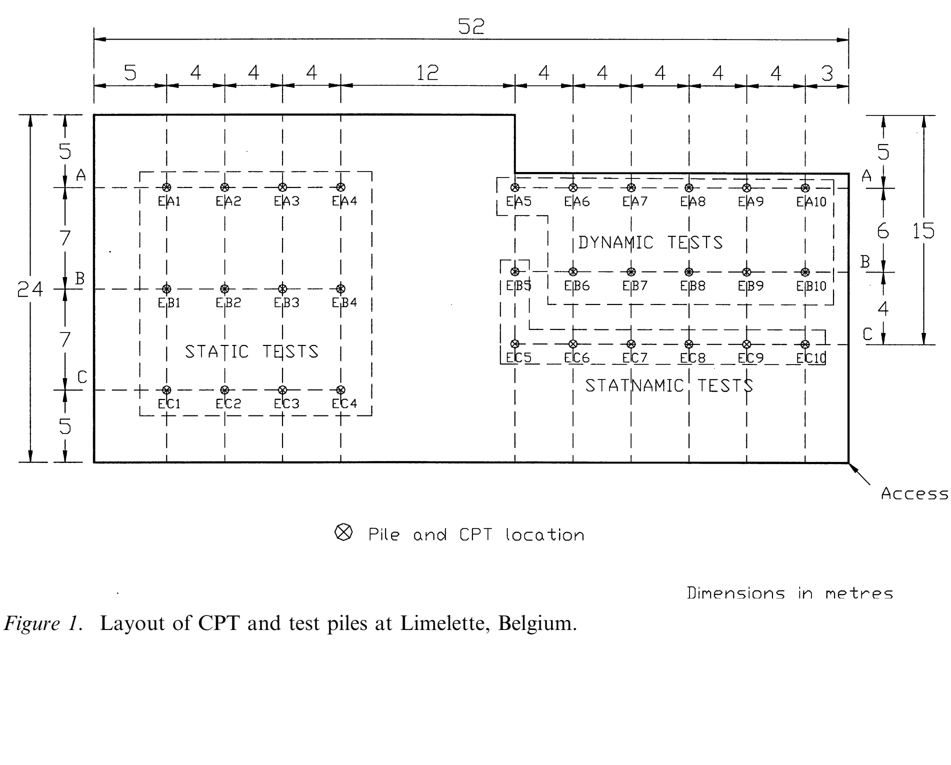

Figure 1. Layout of CPT and test piles at Limelette, Belgium.

Table 1. General attributes: static and dynamic test piles at Limelette, Belgium

Pile No.	Pile type ϕ_{shaft}, ϕ_{base}	Installation date	ϕ_{shaft} (m)	ϕ_{base} (m)	Exposed length L_o (m)	Embeded length L_s (m)	Pile base depth (m)	Test	Measured head capacity, (kN) (static load test)
A1	Fundex 38/45	6-6-2001	0.390*	0.450	1.17	8.50	9.50	Not tested	–
A1bis	Fundex 38/45	11-6-2001	0.390*	0.450	1.22	8.59	9.59	Static	2950
C1	Fundex 38/45	6-6-2001	0.390*	0.450	1.24	8.44	9.44	Not tested	–
C1bis	Fundex 38/45	11-6-2001	0.390*	0.450	1.23	8.65	9.65	Not tested	–
A2	Olivier 36/51	12-6-2001	0.550*	0.550*	1.20	8.45	9.45	Static	3218# (3000)
C2	Olivier 36/51	12-6-2001	0.550*	0.550*	1.30	8.38	9.38	Static	2967# (2500)
A3	Omega 41/41	19-6-2001	0.410	0.410	1.18	8.45	9.45	Static	2750
C3	Omega 41/41	19-6-2001	0.410	0.410	1.21	8.45	9.45	Static	2650
A4	DeWaal 41/41	26-6-2001	0.410	0.410	1.11	8.53	9.53	Static	2375
C4	DeWaal 41/41	26-6-2001	0.410	0.410	1.22	8.52	9.52	Static	2250
B3	Atlas 36/51	29-6-2001	0.510	0.510	1.22	8.58	9.58	Static	3325
B4	Atlas 36/51	29-6-2001	0.510	0.510	1.16	8.58	9.58	Static	3250
B1	Prefab 35/35	20-6-2001	0.446	0.395	1.20	8.51	9.51	Static	2600
B2	Prefab 35/35	20-6-2001	0.446	0.395	1.19	8.57	9.57	Static	3400
A8	Fundex 38/45	7-6-2001	0.390*	0.450	1.63	8.82	9.62	Dynamic	–
A10	Fundex 38/45	7-6-2001	0.390*	0.450	1.66	8.78	9.58	Dynamic	–
A7	Olivier 36/51	11-6-2001	0.550*	0.550*	1.63	8.44	9.24	Dynamic	–
A9	Olivier 36/51	11-6-2001	0.550*	0.550	1.59	8.45	9.25	Dynamic	–
A6	Omega 41/41	19-6-2001	0.410	0.410	1.75	8.70	9.50	Dynamic	–
B10	Omega 41/41	19-6-2001	0.410	0.410	1.68	8.65	9.45	Dynamic	–
B7	DeWaal 41/41	26-6-2001	0.410	0.410	1.66	8.92	9.72	Dynamic	–
C9	DeWaal 41/41	26-6-2001	0.410	0.410	1.68	8.65	9.45	Dynamic	–
B6	Atlas 36/51	28-6-2001	0.510	0.510	1.58	8.53	9.33	Dynamic	–
C10	Atlas 36/51	28-6-2001	0.510	0.510	1.84	8.52	9.32	Dynamic	–
B8	Prefab 35/35	20-6-2001	0.446	0.395	2.29	8.71	9.51	Dynamic	–
B9	Prefab35/35	20-6-2001	0.446	0.395	2.30	8.70	9.50	Dynamic	–

* When measurement from the auger differed from theoretical value, the measured values have been taken.

Pile head capacity not reached but extrapolated using the method of Chin (1972) and reduced by 10% as recommended by Darrag (1987). Values in parentheses are maximum loads applied in test.

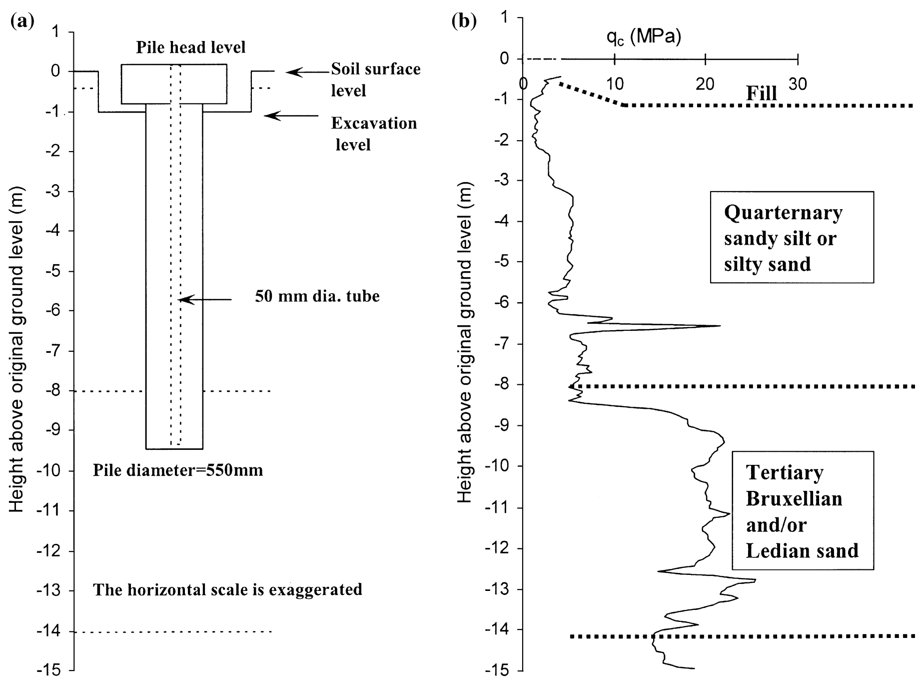

Figure 2. (a) Elevation of typical pile (pile A2) (b) Typical q_c profile (site A2).

to apply load in such a way that the maximum pile head resistance could be mobilised within 8–10 equal load increments. At each step, load was maintained for 60 min, with no intermediate unloading cycles. The criterion for ultimate load capacity was either (i) a pile head settlement rate of 0.6–0.8 mm/min under constant load or (ii) a pile head settlement equivalent to 15% of the nominal base diameter. For pile head settlements greater than 25 mm, when the pile head settlement rate was less than 0.6–0.8 mm/min, the next load increment was halved in order to refine that area of the load-settlement curve. Upon meeting any of the failure criteria, the pile was unloaded in 4 equal decrements, each one being maintained for 10 min.

A 4-ton crane-operated drop hammer was used to apply a series of blows to each of the 12 dynamic test piles. In most cases, a drop height sequence of: 0.4, 0.8 and 1.2 m was applied. Dynamic measurements of strain and acceleration were acquired using the "Pile Driving Analyser (PDA)" monitoring system described by Likins (1984), while displacements were recorded using a laser system supplied by a specialist contractor, A 0.4 m diameter head was cast on top of the 10 screwed cast-in-place piles before dynamic testing and transducers were attached 0.8 m from the top of the pile head. Stress wave data from each blow applied to the pile were monitored and electronically captured by PDA. It was checked that, for a given pile, analysis by CAPWAP produced consistent results for several randomly chosen blow counts. Therefore, prediction of load capacity using CAPWAP was based on the data from a specific blow number.

The acquired strain and acceleration data, from the dynamic test piles, were analysed using CAPWAP, in order to predict the capacities of the statically loaded piles of the same types, as defined in Table 2. Since BBRI merely indicated to the authors that static load tests were carried out between 24th August 2001 and 15th October 2001, it is not possible to define the precise time difference between installation and testing, for a given pile. Nevertheless, as evident from Table 2, for a given pile type there is not a great difference between the ages at the time of dynamic and static load tests. Therefore, pile set-up is not likely to cause much error in predicting the capacities of the static test piles using the data from the dynamic test piles.

3. A new computer program for interpreting CPT data and predicting pile capacity

The authors have developed an interactive program GLAMCPT for use in soil profiling and pile capacity prediction based on CPT and borehole data from any site. Currently, the capability of the program is being extended to include pile capacity and settlement prediction using data from SPT (standard penetration tests), pressuremeter and laboratory soil tests. A simplified flow chart for GLAMCPT program is given in Figure 3(a)–(d), The various steps involved in the flow chart are shown in the following figures capturing the user interfaces:

Steps in soil profiling	*Steps in capacity analysis*
Steps (i)–(ii): Fig.4(a)	Steps (1)–(6): Fig.7
Step (iii): Fig.4(b)	Step (7): Fig.4(b)
Step (iv): Fig.4(c)	Steps (8)–(10): Fig.8
Step (v): Fig.5	Step (11): Fig.9
Step (vi): Fig.6	Step (12): Fig.10(a)
	Step (13): Fig.10(b)

Table 2. Schedule of application of dynamic pile test to predict static pile capacity prediction

Data from dynamic test pile number	Used in CAPWAP to predict static capacity of static test pile number
A8 and A10 (Fundex)	Al_{bis}(Fundex)
Age at time of dynamic test = 139 days	Age at time of static test = 74–125 days
A7 and A9 (Olivier)	A2 and C2 (Olivier)
Age at time of dynamic test = 136 days	Age at time of static test = 73–124 days
A6 and B10 (Omega)	A3 and C3 (Omega)
Age at time of dynamic test = 128 days	Age at time of static test = 66–117 days
B7 and C90 (De Waal)	A4 and C4 (De Waal)
Age at time of dynamic test = 121 days	Age at time of static test = 59–110 days
B6 and B10 (Atlas)	B3 and B4 (Atlas)
Age at time of dynamic test = 119 days	Age at time of static test = 56–107 days
B8 and B9 (Precast driven)	B l and B2 (Precast driven)
Age at time of dynamic test = 127 days	Age at time of static test = 65–116 days

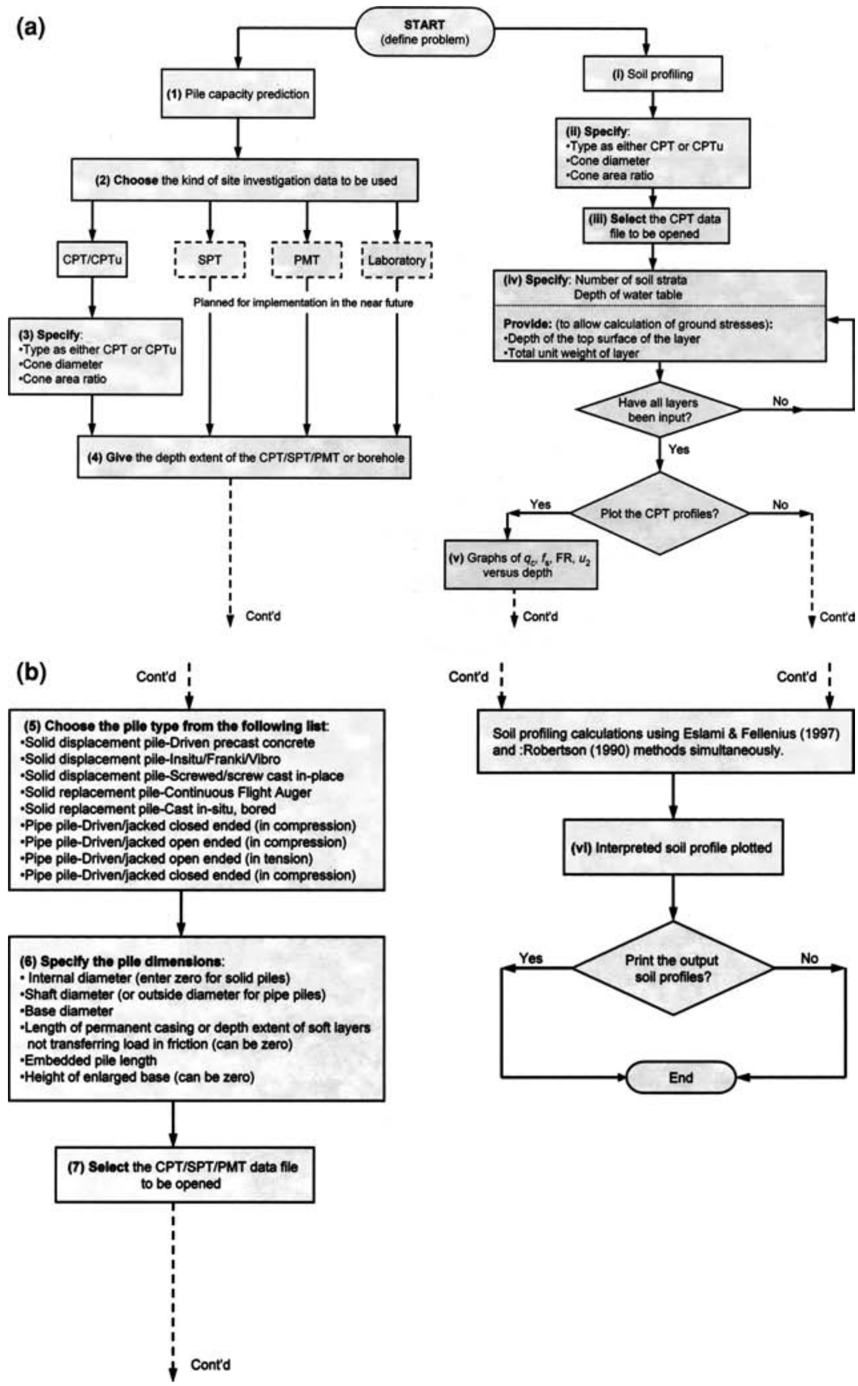

Figure 3. (a) Flowchart for GLAMCPT program, (b) continuation of GLAMCPT flowchart, (c) continuation of GLAMCPT flowchart, (d) continuation of GLAMCPT flowchart.

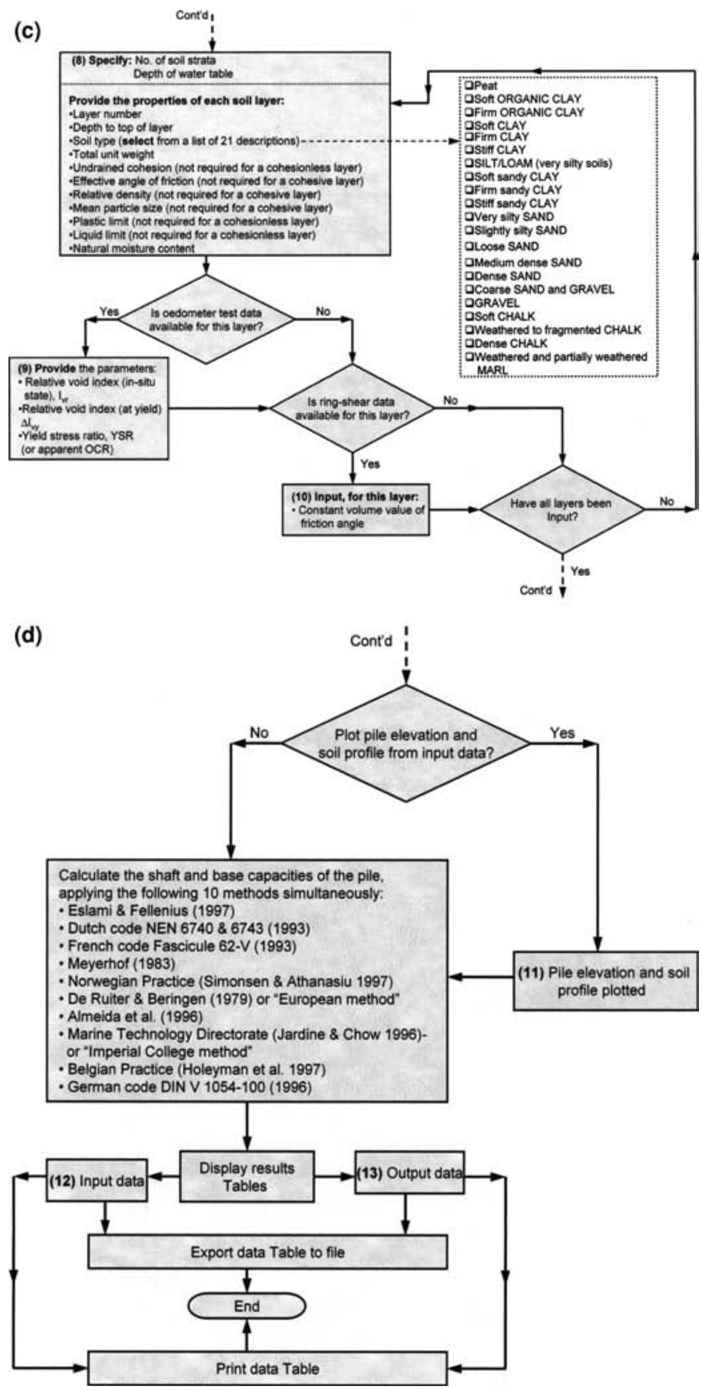

Figure 3. (Continued)

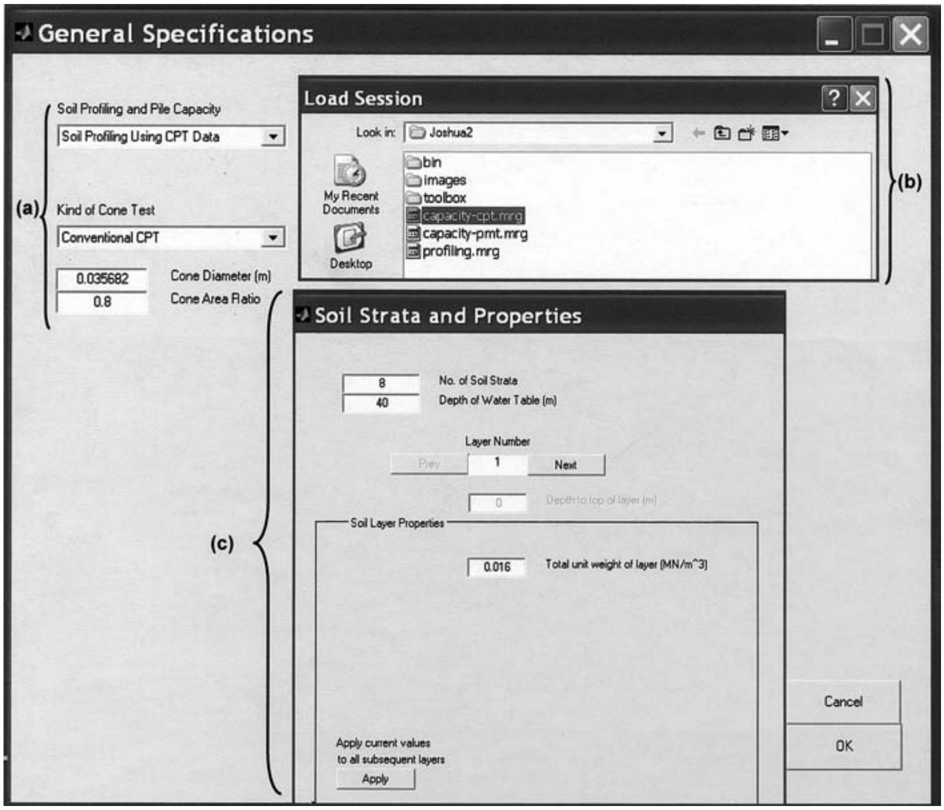

Figure 4. (a) General settings for soil profiling, (b) opening the CPT data file into GLAMCPT (for soil profiling or capacity prediction), (c) additional ground infomlation for soil profiling.

If, in Figure 4(a) or Figure 7, the kind of cone test is selected as CPT, then GLAMCPT expects the CPT text file (Figure 4b) to comprise 3 columns: Depth (m), cone resistance (MPa) and sleeve friction (MPa). If the kind of cone test is chosen as piezocone (CPTu) then the CPT text file must have 4 columns: Depth (m), cone resistance (MPa), sleeve friction (MPa) and pore water pressure, u_2 (measured on the cone shoulder).

For the BBRI test piles, the CPTs did not include pore water pressure measurements and therefore the cone area ratio was not required in the calculations for load capacity. At present, only circular cross-section piles, solid or hollow, can be analysed by GLAMCPT. The program calculates pile shaft and base capacities using the following 10 direct/indirect CPT methods simultaneously:

(1) Eslami & Fellenius (1997)
(2) Dutch code NEN 6740 & 6743 (Everts and Luger, 1997)
(3) French code: Fascicule 62-V (1993)
(4) Meyerhof (1983) This gives two alternative formulae for shaft resistance: one based on cone resistance and another based on sleeve friction.

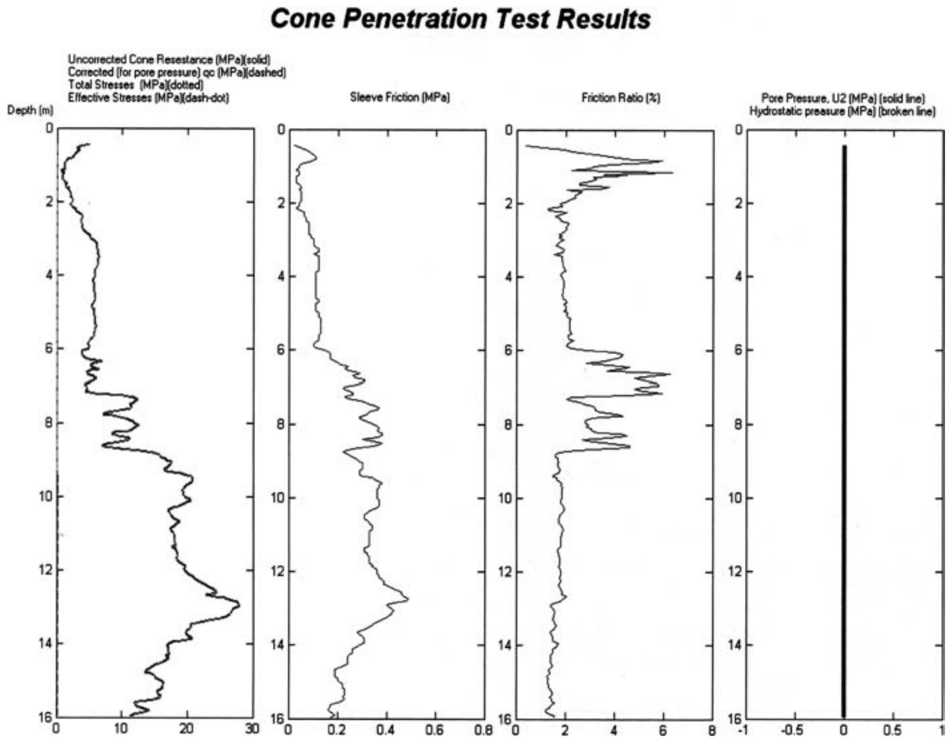

Figure 5. GLAMCPT generated plots of CPT input data.

(5) Norwegian Practice (Simonsen and Athanasiu, 1997)
(6) De Ruiter and Beringen (1979) or "European method"
(7) Almeida et al. (1996)
(8) Marine Technology Directorate (Jardine & Chow, 1996)
(9) Belgian Practice or national application document for Eurocode 7 (Holeyman et al., 1997)
(10) German practice DIN V 1054-100 (Katzenbach and Moorman, 1997)

For the present CPT and test piles, which were installed in sand, Almeida et al. (1996) method is not applicable. This method is used only when piezocone (CPTu) data is available and the pile is formed in clay. For open-ended piles in compression loading, GLAMCPT takes into account the influences of plugging and coring mechanisms on base resistance mobilisation, following the procedures of the Marine Technology Directorate method.

4. Computer analysis of dynamic test pile data

It was considered that a detailed matching of the stress-wave data would predict static capacity more reliably than the existing pile driving formulae (such as AASHTO, 1990, Olson and Flaate, 1967 and Mansur and Hunter, 1970). The monitored data

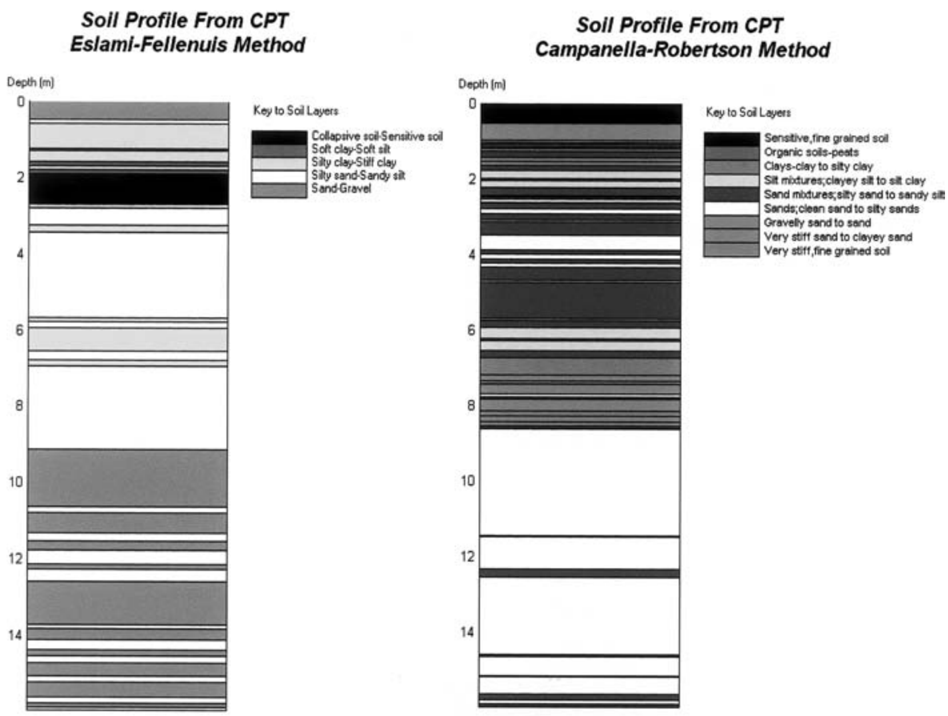

Figure 6. Output of soil profiling by GLAMCPT.

from dynamic load tests included pile head force, velocity and displacement. Additional characteristics such as wave propagation speed, density and pile impedance were also supplied in electronic format. All the wave data were acquired based on a nominal wave speed of 4×10^3 m/s. The measurement section was located in the pile head concrete, for which the compressive strength was determined to be 40–53 MPa. This figure was obtained from tests on specimen concrete cylinders according to the Belgian Standard NBN 15-001. For CAPWAP analysis, the pile shaft and surrounding soil were divided into 9 segments of approximately 1.0 m thick. The PDA test file captured data from all blows but only one of these was selected for CAPWAP analysis. The test file, logged by PDA, was automatically read into the CAPWAP program. By optimising the parameters for each soil segment along the pile shaft, the stress-wave data were matched by iteration until the computed and measured pile head velocities showed acceptable agreement. Table 3 summarises the input data, model soil-pile parameters and the computed outputs of shaft and base capacities from two of the dynamic test piles (piles A6 and A7).

5. Results from GLAMCPT and CAPWAP analyses

For the 11 static test piles analysed, the results of shaft and base capacities computed by GLAMCPT, using the 9 indirect/direct CPT methods and CAPWAP methods are

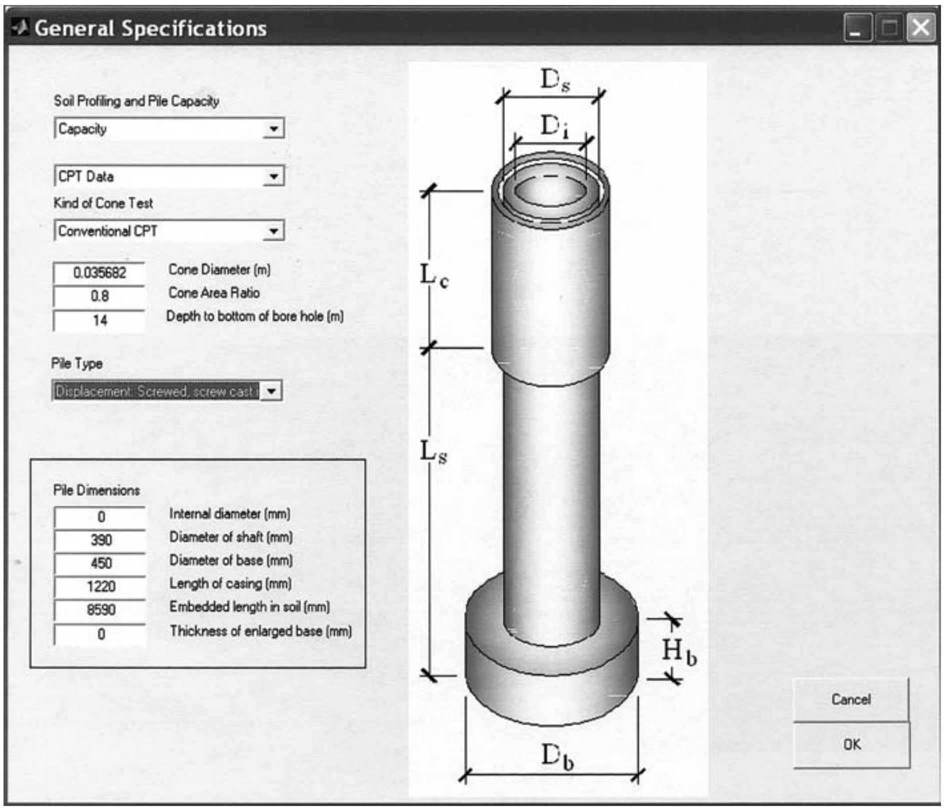

Figure 7. Specifications for pile capacity analysis.

presented in Table 4. Also included in the table are pile test results and capacity values extrapolated using Chin (1972) method (discussed later). In the Meyerhof's method, the reported shaft capacity values were calculated using sleeve friction values. It was found that the alternative of calculating shaft capacity from cone resistance gave nearly 5 times lower shaft capacity values, for all the static test piles.

In the Dutch and European methods, an OCR-dependent upper limit is imposed on the calculated base resistance. On invoking the prescribed limit, the base capacity of pile C4 is calculated to be 377 and 147 kN using the Dutch and European methods, respectively. If no upper limit were to be applied, both of the Dutch and European methods would predict the base capacity of pile C4 as approximately 1400 kN. This figure is closer to 1675 kN obtained from the Eslami–Fellenius method. Therefore, the observed underprediction by the European and Dutch methods can be attributed to the low reduction factors for base resistance.

It is observed that the three most accurate methods: Meyerhof and Eslami–Fellenius and Belgian practice show consistent disparities between their shaft and base capacities predictions. For the 11 piles analysed, the ratios of shaft capacity to head capacity are:

Eslami–Fellenius	*Meyerhof*	*Belgian practice*
22%–27% (mean = 25%)	61%–73% (mean = 66%)	20%–30% (mean = 23%)

Since the piles were not fully instrumented, the actual shaft and base resistances mobilised at any stage of loading are unknown. Therefore, the shaft and base resistances had to be interpreted from the observed load-settlement data. It was assumed that the load transfer of both the shaft and base could be represented by hyperbolic functions. Hirayama (1990), Kim et al., (1999), O'Neill and Hassan (1994), Carrubba (1997) and Fleming (1992) demonstrated that hyperbolic functions of different forms are appropriate to represent load transfer behaviour of most pile types.

For each of the 11 static test piles graphs of Δ_b (base settlement) versus Δ_b/P_h (base settlement divided by head load) were plotted. Two typical ones (Omega and De Waal types) are shown in Figures 11–12. According to Chin (1972), provided the pile mobilises significant proportions of both the shaft and base resistance capacities, the variation of Δ_b (vertical axis) with Δ_b/P_h (horizontal axis) can be idealised as bilinear. Further, the reciprocals of the gradients of the first (starting at the origin) and second trend-lines are equal to the shaft and head capacities, respectively. Using this method, the interpreted shaft and head capacities of the 11 test piles are included in Table 4. The ratios of shaft capacity to head capacity (from Chin's method) lie in

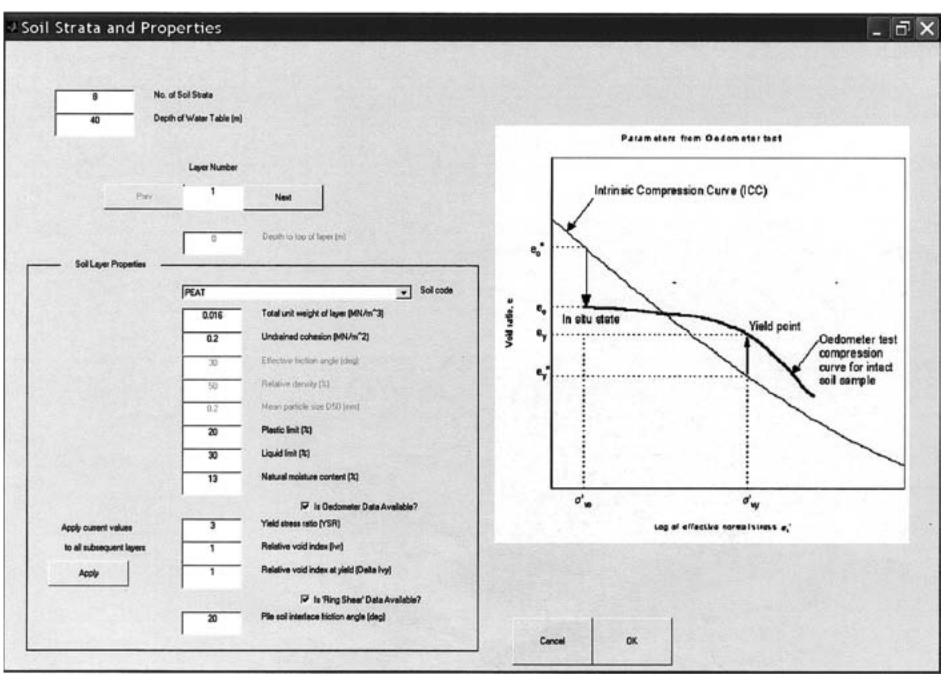

Figure 8. Inputs of soil layer properties for pile capacity prediction.

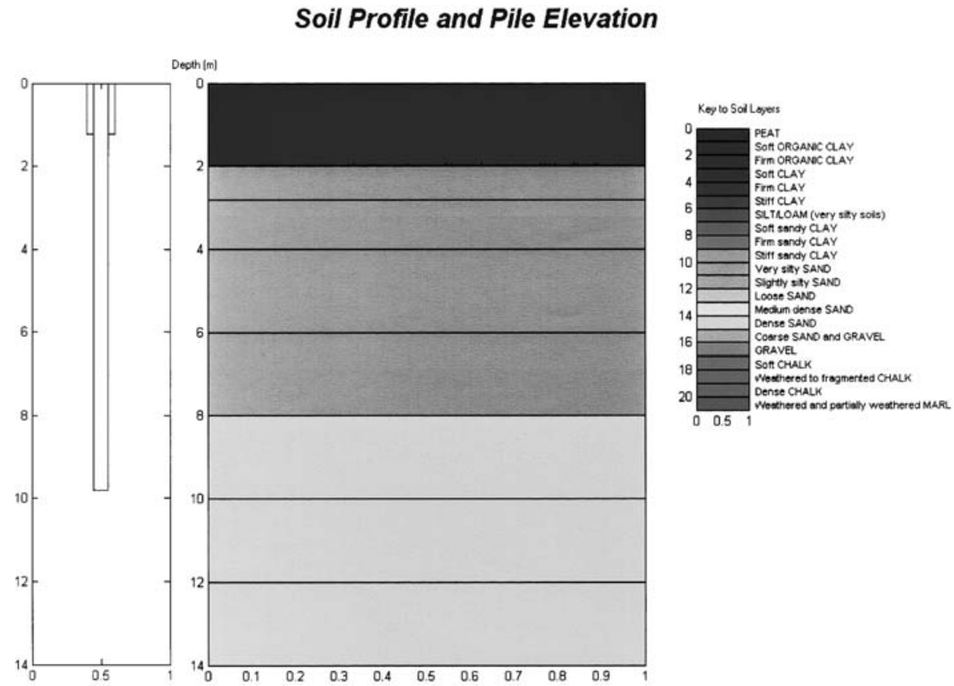

Figure 9. Pile elevation and soil profile generated by GLAMCPT.

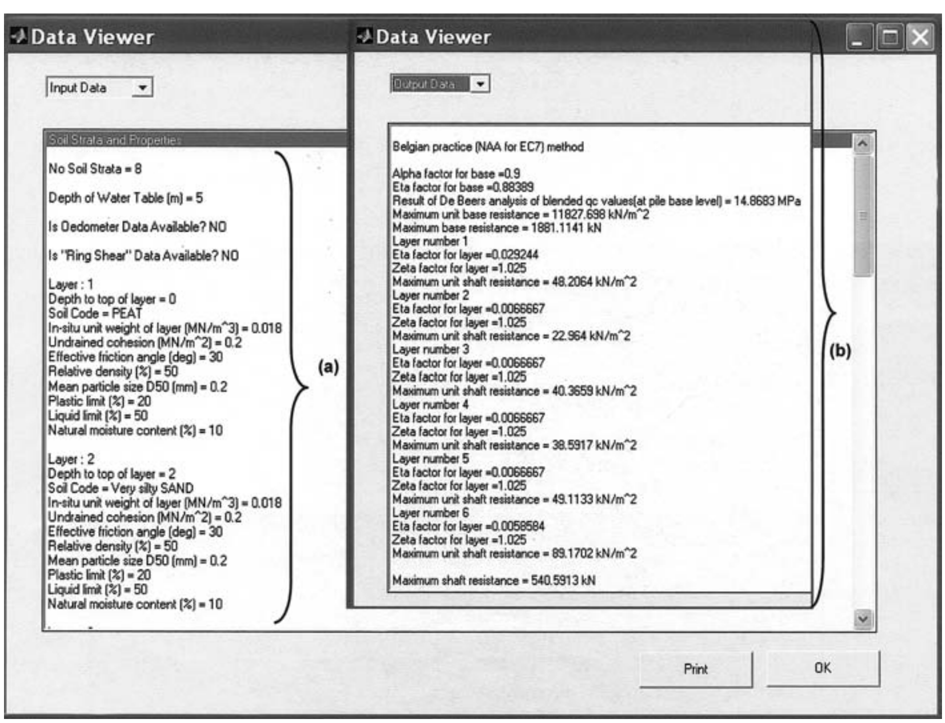

Figure 10. (a) Readout of the input data, (b) Readout of the output data from capacity calculations.

Table 3. CAPWAP analysis of typical dynamic test pile data (piles A6 and A7)

Pile No.	CAPWAP input data		Soil model parameters/ extensions		Pile model	Shaft capacity F_{us} (kN)	Base capacity F_{ub} (kN)	Pile head P_{uh} (kN)
			Case damping factor					
Pile A6	(a) Embedded length:	8.7 m	Skin	Toe	Top segment length (m) = 1.05			
	(b) Shaft perimeter:	1.288 m	0.066	0.525	Top impedance (kN/m/s) = 1355			
OMEGA	(c) Base area:	1320 cm^2	Unloading quake (% of loading quake)		Pile damping (%) = 2.0			
	(d) Blow No. (PDA file):	blow 7	Skin	Toe	Time increment (m/s) = 0.256	909	575	1484
D_b = 0.41 m	(e) Observed final set:	1.31 mm	20	65	Computed final set = 7.56 mm			
D_s = 0.41 m	(f) Wave prop. speed:	4108 m/s	Reloading level (% of R_u)		CAPWAP match quality = 3.346			
L_s = 8.7 m	(g) Comp. strength:	52.3 MPa	Skin	Toe				
			100	100				
CPT:EA6			Unloading level (% of R_u)					
			Skin	Toe				
			0					
			Resistance gap (mm) (incl. in toe quake) = 0.010					
			Soil plug weight (kN) = 14.08					
Pile A7			Case damping factor					
	(a) Embedded length:	8.44 m	Skin	Toe	Top segment length (m) = 1.08			
OLIVIER	(b) Shaft perimeter:	1.950* m	1.033	0.210	Top impedance (kN/m/s) = 2420			
	(c) Base area:	2376 cm^2	Reloading level (% of R_u)		Pile damping (%) = 2.0			
D_s = 0.55 m	(d) Blow No. (PDA file)	blow 11	Skin	Toe	Time increment (ms) = 2420	1091	1255	2346
D_b = 0.55 m	(e) Observed final set:	2.44 mm	100	100	Computed final set = 2.441 mm			
L_s = 8.44 m	(f) Wave prop. speed:	4077 m/s	Unloading level (% of R_u)		CAPWAP match quality = 4.15			
	(g) Comp. strength:	49.8 MPa	Skin	Toe				
CPT: EA7			0					
			Soil plug weight (kN) = 18.79					

Table 4. Shaft and base capacity predictions from 9CPT methods, CAPWAP and Chin's (extrapolation) method

Pile	Predicted capacities	German practice	Belgian practice	Marine Technology Directorate	Norwegian practice	French code	Eslami-Fellenius	Dutch code	European method	Meyerhof	CAPWAP-(mean of 2)	Chin (1972)	Load test result
Albis	Shaft (kN)	591	541	801	556	491	657	471	297	1902	1282.5	2693	*
	Base (kN)	354	1900	1091	2007	1208	1951	2114	1581	1033	863.5	562	*
	Total (kN)	945	2441	1892	2563	1699	2608	2585	1878	2935	2146	3255	2950
A2	Shaft (kN)	879	682	1083	680	670	821	599	380	2529	1292.5	2251	*
	Base (kN)	708	2526	1941	3009	1840	2566	2180	2423	908	1075	1275	*
	Total (IN)	1587	3208	3024	3689	2510	3387	2779	2803	3437	2367.5	3526	3218
A3	Shaft (kN)	664	499	803	520	491	563	402	266	1743	817	1470	*
	Base (kN)	388	1661	1280	1648	1091	1722	1217	1353	851	598	1913	*
	Total (kN)	1052	2160	2083	2168	1582	2285	1619	1619	2594	1415	3383	2750
A4	Shaft (kN)	663	490	831	520	503	550	389	262	1815	990.5	1300	*
	Base (kN)	402	1743	1312	1738	1119	1759	1193	1326	898	669.5	1727	*
	Total (kN)	1065	2233	2143	2258	1622	2309	1582	1588	2713	1660	3027	2375
B1	Shaft (kN)	661	446	831	496	525	573	703	283	1896	721.5	2035	*
	Base (kN)	371	1799	1501	1591	1045	1648	1241	1241	912	1064	944	*
	Total (kN)	1032	2245	2332	2087	1570	2221	1944	1524	2808	1785.5	2979	2600
B2	Shaft (kN)	761	519	999	641	594	640	771	330	2301	721.5	2284	*
	Base (kN)	406	2026	1523	1888	1071	1948	1039	1041	1095	1064	1762	*
	Total (IN)	1167	2545	2522	2529	1665	2588	1810	1371	3396	1785.5	4046	3400
B3	Shaft (kN)	590	683	907	583	684	708	461	379	2679	986	2317	*
	Base (kN)	693	2769	1864	3284	1865	2572	1182	1316	1443	964	1874	*
	Total (kN)	1283	3452	2771	3867	2549	3280	1643	1695	4122	1950	4191	3325
B4	Shaft (kN)	595	596	913	583	691	791	564	405	2473	986	2711	*
	Base (kN)	693	2613	1720	3284	1820	2562	1083	1206	1352	964	931	*
	Total (kN)	1288	3209	2633	3867	2511	3353	1647	1611	3825	1950	3642	3250

C2	Shaft (kN)	592	578	993	587	719	613	431	332	2259	1292.5	1849	*
	Base (kN)	806	2943	1950	3821	1910	2061	1101	1226	1058	1075	1324	*
	Total (kN)	1398	3521	2943	4408	2629	2674	1532	1558	3317	2367.5	3173	2967
C3	Shaft (kN)	465	465	779	452	535	569	392	286	1820	817	1079	*
	Base (kN)	448	1840	1220	2128	1062	1512	724	806	1042	598	2172	*
	Total (KN)	913	2305	1999	2580	1597	2081	1116	1092	2862	1415	3251	2650
C4	Shaft (KN)	587	470	822	488	535	537	377	278	1770	990.5	1897	*
	Base (KN)	459	1947	1218	2220	1042	1675	689	768	1135	669.5	838	*
	Total (KN)	1046	2417	2040	2708	1577	2212	1066	1046	2905	1660	2735	2250

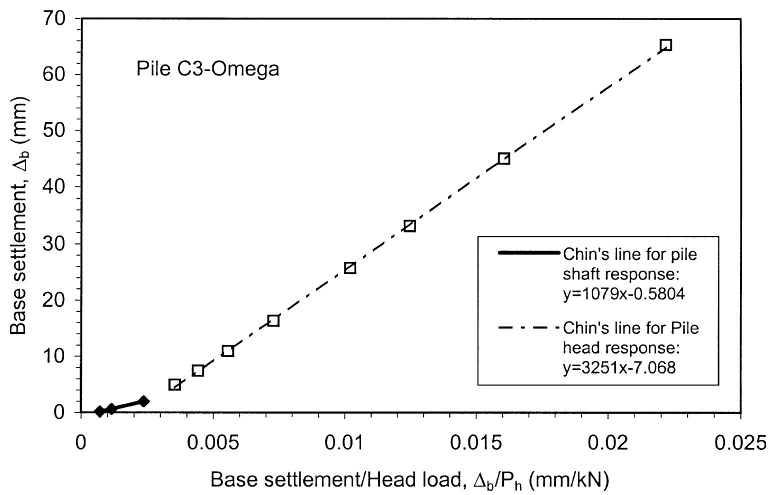

Figure 11. Interpretation of capacities by Chin's method-pile C3.

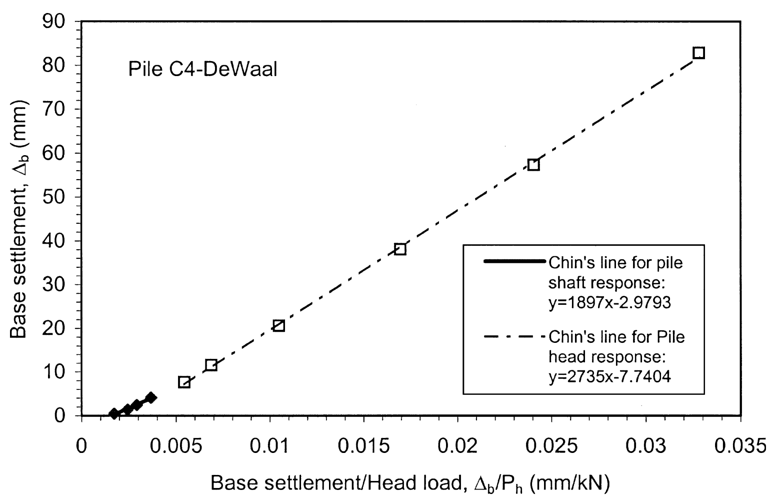

Figure 12. Interpretation of capacities by Chin's method-pile C4.

the range 33% to 82% with a mean of 59%. This compares favourably with the results from Meyerhof's method where the ratio is 61%–73% (mean = 66%). Hence, of the 3 "best" methods (Meyerhof, Eslami–Fellenius and Belgian practice), it is Meyerhof's method that represents the actual pile behaviour most accurately.

6. Formulation of an improved method (UoG method)

Using GLAMCPT and the pile test data, a series of parametric studies have been carried out in an attempt to increase the accuracy and reliability of the method proved to be best so far; that is Meyerhof's method. This led to the development of an alternative (UoG) method of base capacity prediction by taking into consideration the following factors:

(a) The depth interval, over which cone resistance values are averaged in order to, calculate the unit pile base resistance, represents the assumed extents of the failure zone around the pile base (the influence zone). Accuracy of the influence zone is particularly important in heterogeneous soils, where the cone resistance profile often exhibits many peaks and troughs. Without paying attention to soil heterogeneity, most of the CPT-based methods discussed assume a fixed influence zone, for example: (i) Eslami–Fellenius: 8D above to 4D below, (ii) Meyerhof: 4D above to 1D below, (iii) French: 1D above to 1D below, (iv) Dutch and European: 8D above to (0.7D to 4D) below.
(b) The CPT methods assume that the influence zone is controlled only by the pile diameter, thus important factors such as soil strength, failure mechanism and soil heterogeneity are ignored.
(c) The assumed extent of the influence zone of only 1D below pile base (in French and Meyerhof's methods) is probably inadequate to account for either a punching shear mechanism or a general failure mechanisms. In the first mechanism, the vertical failure surfaces may be deep penetrating while in the second mechanism, the curved failure surfaces may be relatively longer than predicted by classical "slip surface" theories.

In the UoG method, the location of the influence zone (i.e. the depth interval over which cone resistance values are to be averaged) is first determined, taking into account effects of the pile diameter and the shear strength of the sand in the vicinity of the pile base. The influence zone is assumed to be represented by a logarithmic spiral, where the shear lines extend to the surface of the pile at a certain distance above the pile base level. This is illustrated by the normalised plots in Figure 13. Using the (classical) logarithmic spiral:

$$r = ae^{b\theta} \tag{1}$$

(where $[r,\theta]$ is the polar co-ordinate of a point on the spiral; a = value of r when $\theta = 0$; and b = constant), Eslami and Fellenius (1997) suggested that:

(a) the deepest point of the shear lines below the pile base can be determined by maximising the projection of the radius of the logarithmic spiral on the vertical axis, Hence, from (1), the shear lines may be expressed as follows:

$$r = De^{\pi \tan \phi} \tag{2}$$

in which r = radius of logarithmic spiral, D = pile diameter, ϕ = angle formed by a radius and the tangent of the spiral at the point of intersection of the radius and the spiral.

(b) the angle ϕ may be taken as the average effective angle of friction of the sand layer existing beneath the pile base.

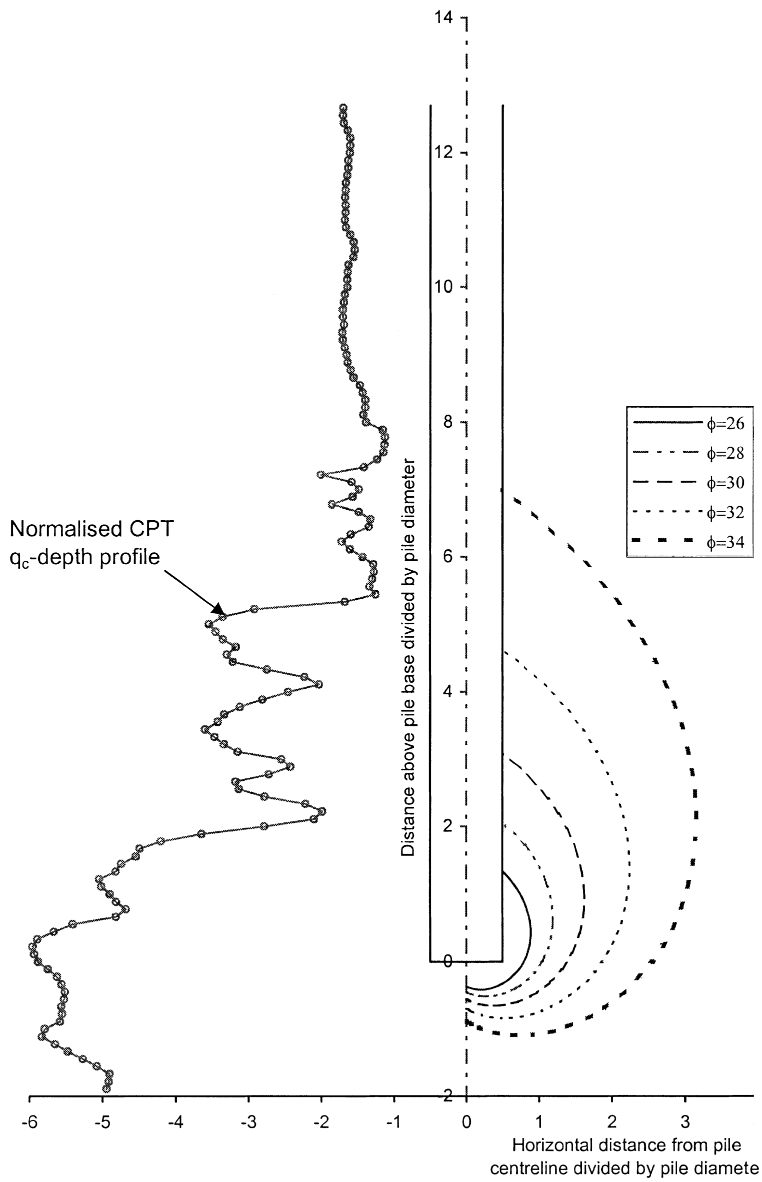

Figure 13. Influence of angle of friction angle on the shearing zone around pile base.

For arbitrary ϕ values lying in the range 26° to 34°, the computed shear lines for homogeneous soil are plotted in Figure 13. Also included in the figure is a normalised plot of the cone resistances from the site of pile $A1_{bis}$. The figure shows that the influence zone can extend to between 2D and 8D above the pile base, depending on the value of ϕ. In comparison, the depth of the influence zone below the pile base, for assumed homogeneous soil, is comparatively less sensitive to ϕ and is up to 1.5D.

In the UoG method, a weighted (with respect to strata thicknesses) average ϕ value is computed internally within the program (GLAMCPT) for the soil layers located between 8D above pile base and 1.5D to 4D below pile base. Hence, given the pile diameter, the UoG method determines the height of the influence zone above the pile base, using the logarithmic spiral defined by the weighted average value of ϕ. Although the logarithmic spiral analysis indicated the lower extent of the influence zone to be 1.5D below pile base, a higher figure of 4D is adopted by the UoG method because: (1) the existence of a weak stratum just beneath the 1.5D level will lead to lower pile base capacity than would have been predicted assuming the extent of the influence zone is only up to 1.5D below pile base level, and (2) other than a punching failure, the shear lines below the pile base follow a curved path which is longer than 1.5D.

For base capacity, the UoG method uses the form of expression proposed by Meyerhof, however the depth interval over which cone resistances are averaged is allowed to vary with pile diameter, soil shear strength and CPT q_c profile. The formula used for shaft capacity in the UoG method is the same as that proposed by Meyerhof.

7. Comparison between the suggested and existing CPT methods of capacity prediction

In order to assess the validity of the predictive methods, it is imperative to quantify statistically the performance of each method, using the pile test results as benchmarks. For each prediction method, the ratio predicted to measured pile head capacity $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ was calculated. Hence the natural logarithms of $P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}$ were also formed. From these data, the following indices are calculated: (i) mean, μ, (ii) standard deviation, SD, (iii) Coefficient of variance, COV and (iv) Ranking Index, RI (Jardine and Chow, 1996); where

$$COV = \frac{SD}{\mu} \tag{3}$$

$$\mathrm{RI} = \left|\mu\left\{\ln\left(\frac{P_{\mathrm{uh(p)}}}{P_{\mathrm{uh(m)}}}\right)\right\}\right|. + SD\left\{\ln\left(\frac{P_{\mathrm{uh(p)}}}{P_{\mathrm{uh(m)}}}\right)\right\} \tag{4}$$

in which the first term is the absolute value of the mean of $\ln\,[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$, where ln = natural logarithm. The second term is the standard deviation of the natural logs of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$. For the generally skewed statistical spread of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ values, RI gives a representative quality index. RI, COV and SD should all be as low as possible while μ should be as close to unity as possible. The accuracy of a method is represented by μ which is a measure of the ability of the method to predict the measured pile capacity. On the other hand, the consistency of a method is indicated by the scatter of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ values around the mean and is therefore represented by SD.

Gauss distribution plots for $[P_{uh(p)}/P_{uh(m)}]$ are illustrated in Figure 14. None of the distributions is a perfectly symmetrical U-shape, however it is seen that the methods that exhibit the least degree of skewing are: (i) the German practice; (ii) new UoG method; (iii) Eslami–Fellenius; and (iv) Meyerhof methods, in the specified order.

The performance of the predictive methods, according to the four statistical indices: μ, SD, COV and RI are depicted in Figures 15–18. It should be borne in mind that the pile test data involved here are completely independent from the databases used by various authors to develop the existing CPT methods of pile capacity prediction. Therefore in that sense, these comparisons represent a valid evaluation of the methods. Table 5 compares the performance of the prediction methods, based on the values of μ, SD, COV and RI from 11 test piles. It is found that, in terms of 3 of the 4 indices (μ, COV and RI), the UoG proposed method ranks in the first place. In terms of standard deviation (SD), the German practice ranks highest, although it is also the lowest ranking in terms of μ and RI indices. This shows that the method in German practice is highly reliable but over-conservative probably due to high factors of safety built into the method.

8. Conclusions

A computer program "GLAMCPT" was developed with a general scope for soil classification and pile capacity prediction direct from the results of *in-situ* and laboratory ground investigation. The program incorporates 10 direct and indirect CPT-based methods of pile analysis. A variety of pile types can be analysed using GLAMCPT, including displacement and replacement piles, of solid or hollow

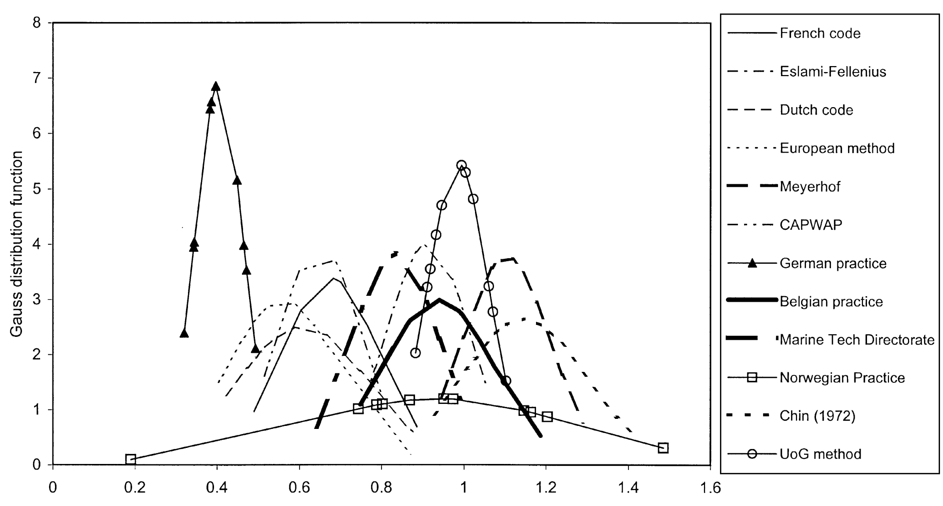

Figure 14. Gauss distribution functions for $[P_{uh(p)}/P_{uh(m)}]$ values from 11 piles (various predictive methods).

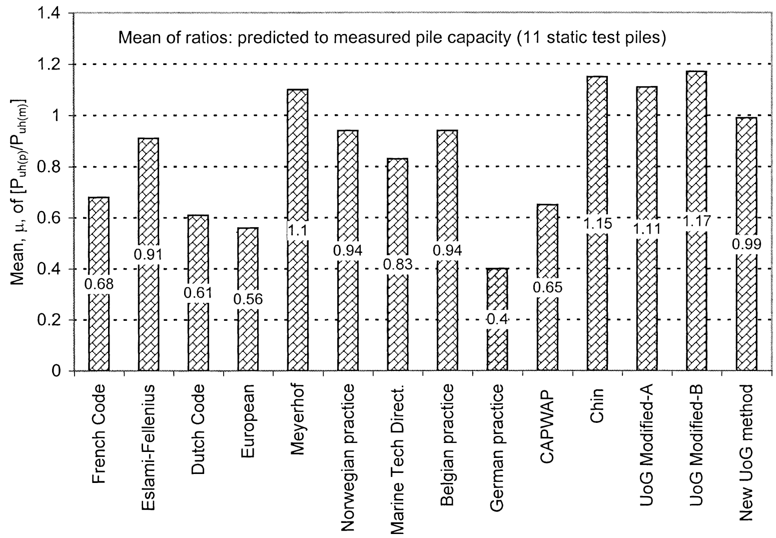

Figure 15. UoG method compared with existing others (mean values of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ from 11 piles).

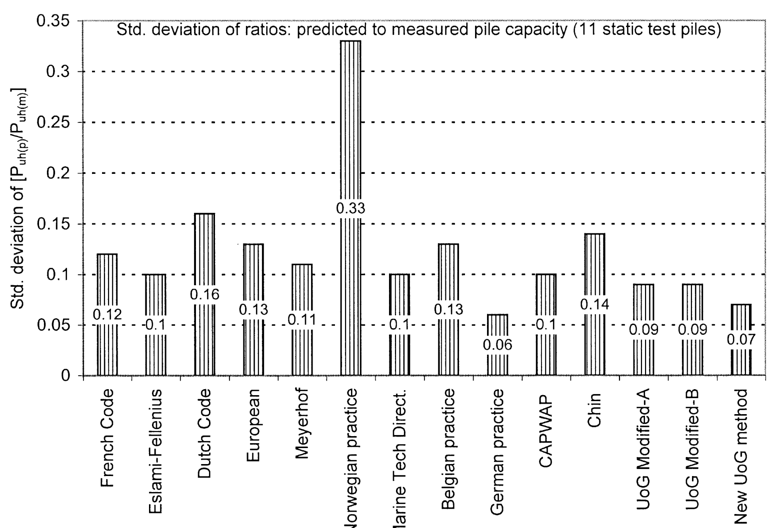

Figure 16. UoG method compared with existing others (standard deviation of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ from 11 piles).

cross-sections. The program automatically corrects the CPT input data, for pore pressures and overburden stresses, and interprets all the parameters required for pile capacity prediction, e.g. over-consolidation ratio (OCR) and horizontal earth pressure coefficient.

A database comprising 23 static and dynamic test piles, installed using different techniques, 11 CPT logs and one borehole log were compiled. This case history

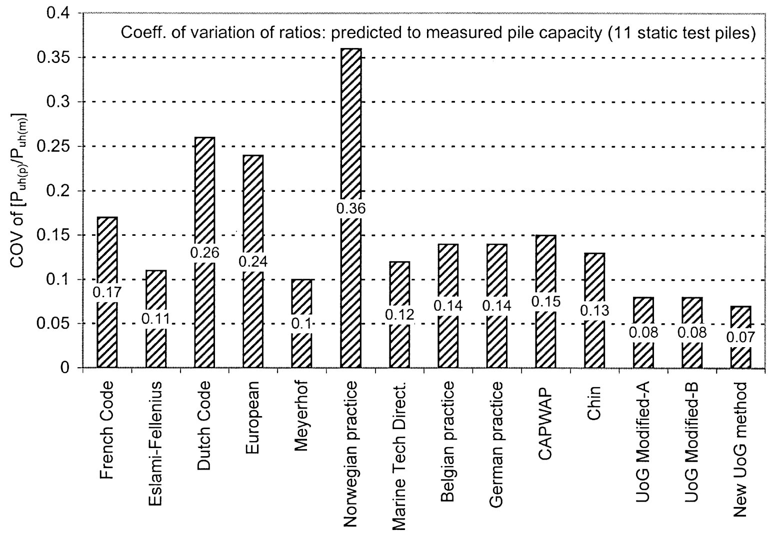

Figure 17. UoG method compared with existing others (coefficient of variation of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ from 11 piles).

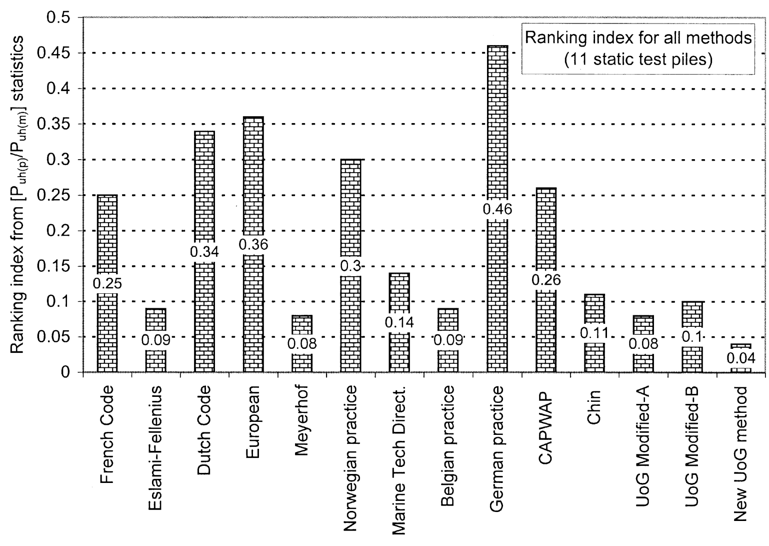

Figure 18. UoG method compared with existing others (ranking index of $[P_{\mathrm{uh(p)}}/P_{\mathrm{uh(m)}}]$ from 11 piles).

is totally independent of the databases used by others in the development of the CPT-based predictive methods. GLAMCPT was used to evaluate the CPT based methods by predicting the load capacities of 11 statically tested piles of 6 different types. The capacities of the same piles were also predicted using a stress wave analysis program. The predicted load capacities of the static test piles were compared with the load test results.

Table 5. Capacity predictions for the 11 static test piles: comparison of statistical indices of $[P_{uh(p)}/P_{uh(m)}]$

μ ranking	SD ranking	COV ranking	RI ranking
1. UoG method	1. German practice	1. UoG method	1. UoG method
2. Belgian practice	2. UoG method	2. UoG Modified-A	2. UoG Modified-A
3. Norwegian practice	3. UoG Modified-A	3. UoG Modified-B	3. Meyerhof
4. Eslami-Fellenius	4. UoG Modified-B	4. Meyerhof	4. Eslami-Fellenius
5. Meyerhof	5. Eslami-Fellenius	5. Eslami-Fellenius	5. Belgian practice
6. UoG Modified-A	6. CAPWAP	6. Marine Technology Directorate	6. UOG Modified-B
7. Chin	7. Marine Technology Directorate	7. Chin	7. Chin
8. UoG Modified-B	8. Meyerhof	8. German practice	8. Marine Technology Directorate
9. Marine Technology Directorate	9. French Code	9. Belgian practice	9. French Code
10. French Code	10. Belgian practice	10. CAPWAP	10. CAPWAP
11. CAPWAP	11. European	11. French Code	11. Norwegian practice
12. Dutch code	12. Chin	12. European	12. Dutch code
13. European	13. Dutch code	13. Dutch code	13. European
14. German practice	14. Norwegian practice	14. Norwegian practice	14. German practice

It was established that, in terms of accuracy, the best of the existing prediction methods is the Belgian practice, which gave the ratio of predicted to observed pile head capacity $\left[P_{uh(p)}/P_{uh(m)}\right]$ equal to 0.94. In terms of the standard deviation of $\left[P_{uh(p)}/P_{uh(m)}\right]$ values, the best of the existing methods is the German practice (SD = 0.06), however this method is seen to be over-conservative therefore under-predicting pile capacity by as much as 60%, Based on a defined ranking index, which includes both accuracy and reliability, Meyerhof's method is proven overall to be the best of the current methods, giving RI = 0.08. Some of the methods seriously under-predict pile capacity to the extent that $\left[P_{uh(p)}/P_{uh(m)}\right] = 0.35$. It was shown that, in the European and Dutch methods, the use of OCR-dependent upper limits of base resistance leads to consistent underestimation of pile base capacity in sand.

With the aid of GLAMCPT program, an improved (UoG) method was developed, which takes into account the influence of soil strength and pile diameter on the extent of the rupture surface around a pile. Application of the UoG method to the 11 static test piles yielded: $\mu = 0.99$, SD = 0.07, COV = 0.07 and RI = 0.04. These figures demonstrate that the proposed method gives more accurate predictions of total pile capacity than the existing methods evaluated here, for the given ground conditions.

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