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Axial capacity of driven piles in deltaic soils using CPT

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ABSTRACT: The prediction of axial pile capacity is a complex engineering problem. Traditional methods of data collection and subsequent analyses are frequently in error when compared to full-scale load tests. Cone penetration testing (CPT) provides a means by which continuous representative field data may be obtained.

This paper compares the predictions from thirteen axial pile capacity methods with the results obtained from eight full-scale pile load tests on six different piles. The piles were steel pipe piles driven into deltaic soil deposits. The thirteen prediction methods, separated into direct and indirect classes, used data obtained from the CPT as input for analyses.

A brief evaluation of each method investigated is presented and the preferred method(s) of analyses are identified.

1 INTRODUCTION

The continued growth of many large Canadian cities has led to increased construction of larger more complex structures on sites with difficult ground conditions. In Vancouver, as in other metropolitan areas, large structures such as major highway bridges, cement processing plants, storage tanks and high-rise residential and commercial complexes are now being constructed on deltaic and alluvial deposits. Essentially the delta region of the Fraser River is located immediately to the south of Vancouver. The delta region is covered by a thin veneer of clays, silts and peats up to about 6 m in thickness, which is underlain by a tidal-flat deposit of sands and silts to a maximum thickness of 30 m, which in turn is underlain by stratified marine delta deposits ranging from silty clays to clean sands up to 300 m in thickness. The groundwater table is at or near the ground surface. In these Fraser River deltaic soil deposits, piled foundations are used extensively to support large structures. The recent construction of the Alex Fraser bridge and corresponding highway extensions are excellent examples. In order that a piled foundation may be designed safely and economically, its behaviour under load must be accurately predicted and ideally, a full-scale pile load test should be perfomed. Full-scale load tests are, however, very expensive and are therefore often impractical. Predictive methods for pile capacity require an accurate assessment of the properties of the soil into which the pile is to be placed. The CPT offers an excellent means to accurately obtain the required properties, especially in soft deltaic soils.

A total of thirteen static axial pile capacity methods were used to predict the results obtained from eight full-scale pile load tests on six different piles. These methods, separated into direct and indirect classes, used data obtained from the cone penetration test (CPT). The CPT is a fast, economic and repeatable in-situ test, especially in loose and soft deltaic sediments. The CPT can also be considered to be a model displacement pile.

This paper summarizes the results of the eight full-scale pile load tests and compares these results with the CPT predictions using the thirteen methods.

2 TEST SITE

The test site is located on Lulu Island which is within the post-glacial Fraser River delta (Fig. 1).

The surficial geology of the Lulu Island region is typical of a former marine environment no longer dominated by tidal action. A summary of the soil profile at

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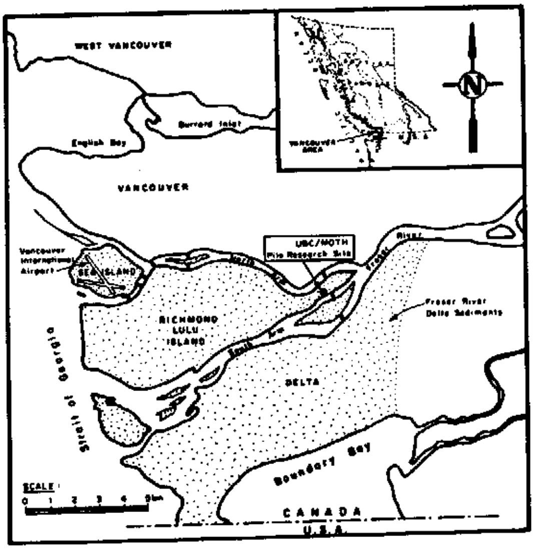

Fig. 1. General location of research site.

the test site to a depth of 75 m based on sampling and CPT is shown in Fig. 2. Below a surface layer of fill there is a prevalent deposit of organic silty clays to a depth of about 15 m that has been laid down in a quiescent swamp or marsh environment. Below this upper layer, a medium dense sand deposit, locally silty, prevails to a depth of 30 m. This sand deposit is indicative of a high energy depositional period and most likely represents a former channel bank of the Fraser River. Underlying the sand, to a depth of up to 150 to 200 m (Blunden, 1975), exists a normally consolidated clayey silt deposit containing thin sand layers. Below a depth of about 60 m the sand layers are more prevalent and thicker (up to 1 m thick). The non-uniformity of the deposits below 30 m indicates a depositional history most likely consisting of alternating turbulent and quiescent environments associated with either tidal flat facies, marginal bank or an alluvial floodplain depositional environment. The CPT profile in Fig. 2 presents a clear picture of the stratigraphic detail at the test site.

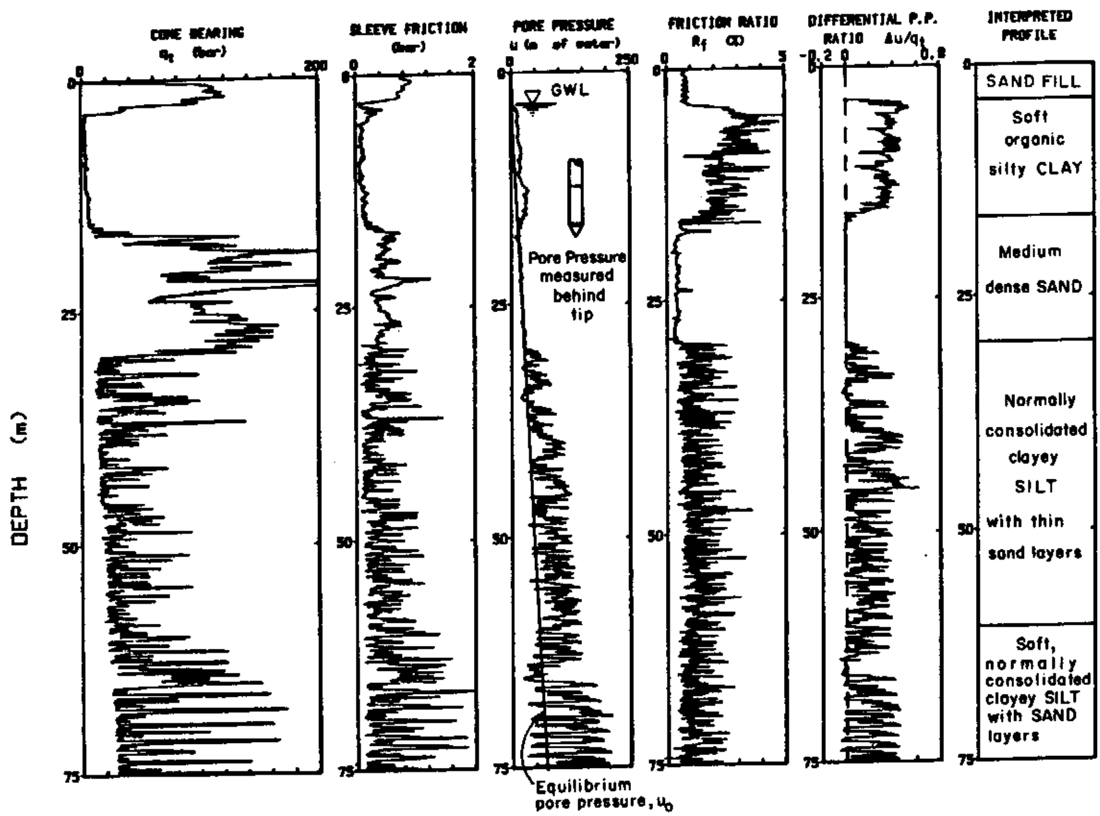

Fig. 2. Soil profile for pile research site. (1 bar = 100 kPa)

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Across the entire site, 2 to 4 m of non-homogeneous fill exists at the surface. For the purpose of facilitating in-situ testing, making pile driving possible, and studying lateral pile behaviour, the fill material was removed in the general area of the research piles. This material was replaced with clean sand.

Six pipe piles were driven (four 324 mm dia., 9.5 mm wall thickness; one 324 mm dia., 11.5 mm thickness, one 915 mm dia., 19 mm thickness) at the site. The relative embedments and pile tip conditions of the piles are shown in Fig. 3. The five smaller piles were placed and tested under the supervision of University of British Columbia (UBC) personnel. The large pile was placed and tested under the supervision of the B.C. Ministry of Transportation and Highways (MOTH). Pile No. 1 had a larger diameter sleeve for the first 2 m to remove any frictional resistance in the upper sand fill (see Fig. 3).

3 AXIAL PILE LOAD TESTS

The 'Quick Load Test Method' of axial loading (similar to ASTM D1143-81 Section 5.6) was used with the axial load being applied in roughly 5% increments of the anticipated failure load for the piles shown in Fig. 3. The 'Quick Load Test Method' was used to minimize time-dependent effects. A summary of the pile driving and testing schedule is presented in Table 1.

Table 1. UBC/MOTH pile driving and testing schedules.

Pile/ Test No.	Pile Length (m)	Driving Date(s)	Testing Date(s)
1	14.3	19 AUG 85	09 NOV 85
2	13.7	16 AUG 85	01 MAR 85
3	16.8	16 AUG 85	09 NOV 85
4	23.2	16 AUG 85	01 MAR 85
5	31.1	15 AUG 85; 16 AUG 85	22 SEP 85; 06 OCT 85
A	67.0	10,11,13,16, 17 APR 84	09 MAY 84
B	78.0	11 MAY 84	01 JUN 84
C	94.0	09 JUN 84	29 JUN 84

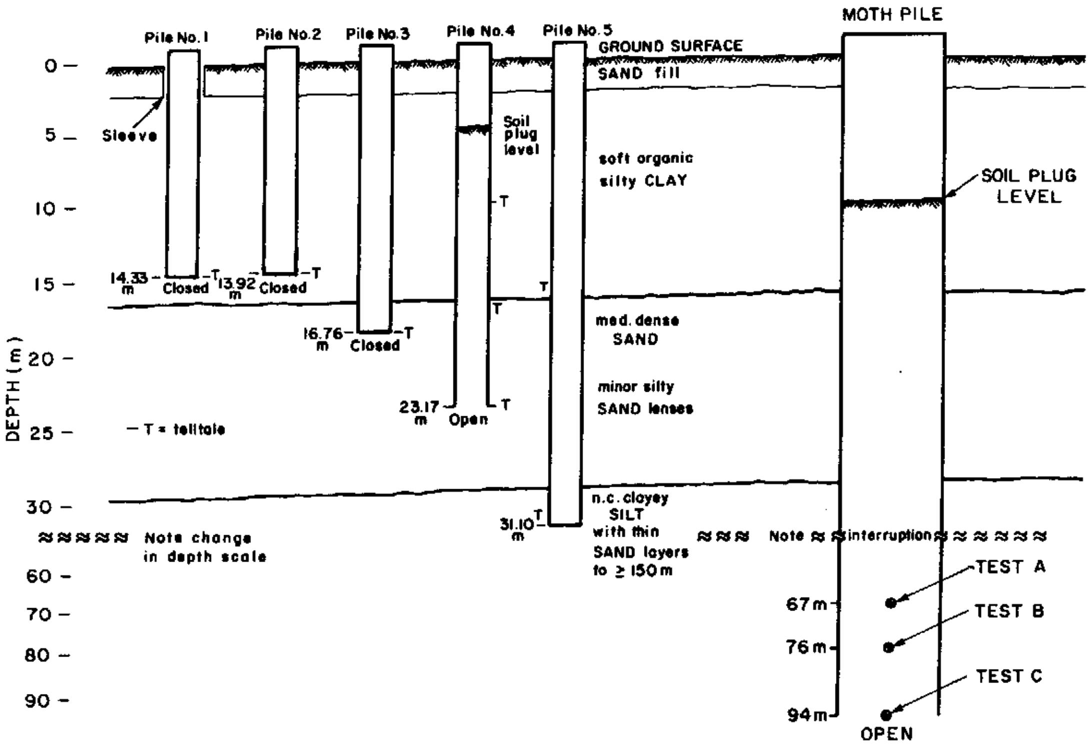

Fig. 3. Schematic of UBC/MOTH test pile embedments.

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Analysis of the results from axially loaded vertical test piles is more complicated than generally realized (Brierley et al, 1978). For a pile (generally assumed to be stronger than the soil), the ultimate failure load is reached when the pile plunges; i.e., rapid settlement occurs under sustained or only slightly increased load. This definition, however, is often inadequate because plunging requires very large displacements and is often less a function of the pile-soil system and more a function of the capacity of the man-pump system (Fellenius, 1980). To be useful, a failure definition should be based on a simple mathematical rule that can generate repeatable results independent of the individual using the method and of the scale chosen for plotting the load test data. For example, Fig. 4 shows the results of a hypothetical pile load test plotted to different scales. The hypothetical test pile could be interpreted, based on a visual inspection of the results, as a predominantly friction or 'floating' pile (upper figure) or a predominantly end bearing pile (lower figure). A popular method of interpreting axial pile load test data is that by Davisson (1973) and involves a simple graphical manipulation of the theoretical elastic compression line for the pile in question. Davisson's method (1973) has been used in this study to determine failure loads. Fellenius (1980) studied nine commonly used failure criteria and found Davisson's method to be among the most conservative.

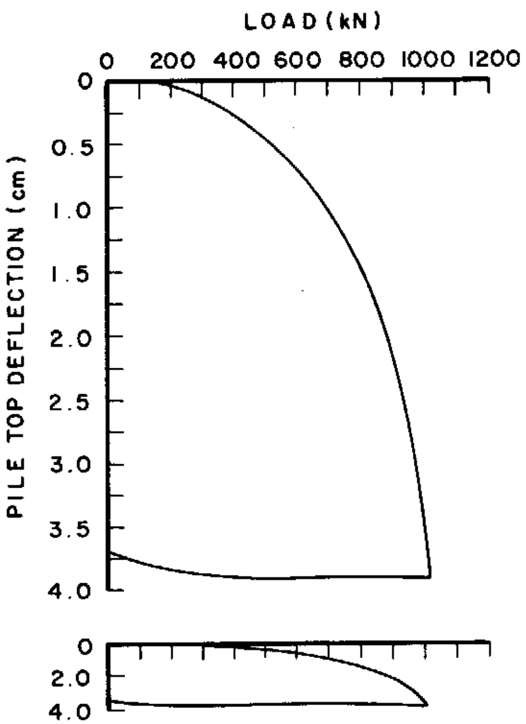

Fig. 4. Load-displacement diagram of hypothetical test pile drawn to two different scales.

Fig. 5(a) presents a summary of the load-displacement test results for the five smaller piles. Based on the telltale data piles 1, 2 and 5 are interpreted as predominantly shaft resistance piles whereas piles 3 and 4 had significantly larger contributions to their total capacity from end bearing. Pile No. 4 could not be loaded to failure, but the load-deflection diagram was based on the combined results of the other pile test results. Fig. 5(b) presents a summary of the load-displacement results for the larger pile. The larger pile was tested at three depths (67, 78 and 94 m) as shown in Fig. 3. All three test results (Fig. 5b) indicate that the larger pile has a large shaft resistance component. The reduction in measured load observed for the larger pile at 67 m and 94 m depth occurred because, with rapid axial deflections, the hydraulic jacks were unable to sustain the load. Full details of the test program for the 915 mm pile is given by Robertson et al (1985). A summary of the axial load testing is presented in Table 2.

Table 2. Full details of the overall test program are given by Davies (1987).

Pile/ Test No.	Length (m)	Diameter (m)	Wall Thickness (mm)	L/D	Open/ Closed Ended	Capacity (kN) (Davisson, 1973)
1	14.3	0.324	9.5	44	C	170
2	13.7	0.324	9.5	42	C	220
3	16.8	0.324	9.5	52	C	610
4	23.2	0.324	9.5	72	O	1200
5	31.1	0.324	11.5	96	C	1070
A	67.0	0.915	19	73	O	7500
B	78.0	0.915	19	85	O	7000
C	94.0	0.915	19	103	O	8000

4 PREDICTION OF STATIC AXIAL PILE CAPACITY

The prediction methods will be separated as follows:
1) direct methods
2) indirect methods.

The term "direct method" is applied to any static prediction method that uses CPT data (tip resistance, q_c, and/or sleeve friction,

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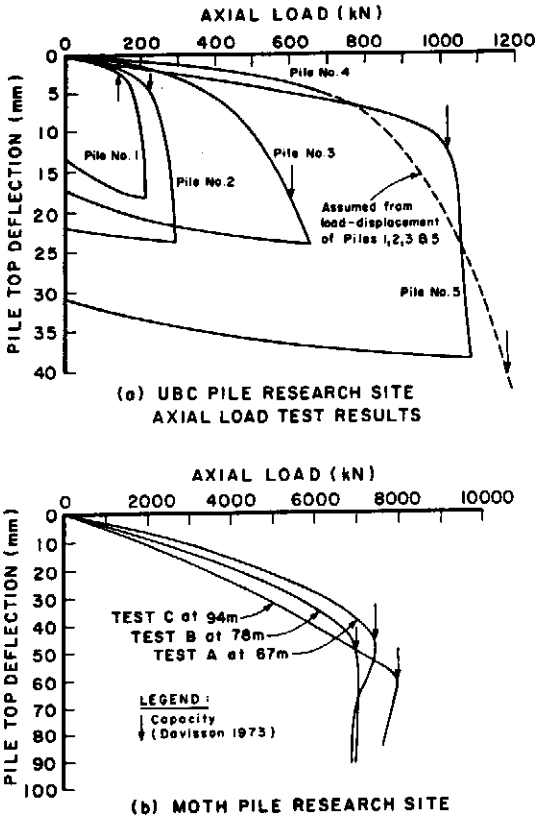

Fig. 5. Summary of axial load test results.

f_s, depending upon method used) directly by the use of theoretical and/or empirical scaling factors without the need to evaluate any intermediate values (coefficients of earth pressure, bearing capacity factors, friction angle, etc.). The scaling factors, in all cases, resemble the original work of de Beer (1963). De Beer (1963) suggested that if a probe of zero diameter penetrates a soil layer, the device would "feel" the entire effect of the lower soil layer immediately upon penetration. However, if a large diameter pile were pushed into the layer, the point resistance would not equal that of the zero diameter probe until the pile reached a greater depth. This depth is often termed the critical depth. De Beer showed that it is reasonable to assume that the pile resistance curve between the layer interface and the critical depth varies linearly; thus, the pile resistance at any intermediate depth could be determined if both the idealized penetration resistance curve and the critical depth were known. Although it is not possible to use a probe of zero diameter, the standard electric cones (35.7 mm in diameter) can be assumed to approximate this condition, especially for large diameter piles. This concept is complicated in highly layered materials

Table 3. Prediction methods evaluated

Direct Methods	References	Notes
1. Schmertmann and Nottingham CPT	Schmertmann (1978)	Proven CPT Method (q_c & f_s used)
2. de Ruiter and Beringen CPT	de Ruiter and Beringen (1979)	European (Fugro) (q_c & f_s used)
3. Zhou et al CPT	Zhou et al (1982)	Chinese Railway Experience (q_c & f_s used)
4. Van Mierlo and Koppejan CPT	Van Mierlo and Koppejan (1952) and Begemann et al (1982)	Original Dutch (q_c only used)
5. Laboratoire Central des Ponts et Chaussées CPT (LCPC)	LCPC-Bustamante and Gianesalli (1982)	French Method (q_c only used)
6. Belgian CPT	W.F.Van Impe (1986)	Belgian Method (q_c only used)
Indirect Methods	**References**	**Notes**
7. API RP2A	American Pet. Inst. (1980)	Offshore
8. Dennis and Olson	Dennis and Olson (1983a & b)	Modified API
9. Vijayvergiya and Focht	Vijayvergiya and Focht (1972)	"λ"Method
10. Burland	Burland (1983)	"β" Method
11. Janbu	Janbu (1976)	NIT
12. Meyerhof Conventional	Meyerhof (1976)	Original Bearing Theory
13. Flaate and Selnes	Flaate and Selnes (1977)	NGI

where layer thicknesses are less than the critical depth for the large diameter pile. In these situations the full penetration resistance may be mobilized on the cone but may not be realized for the pile before the influence of another layer is felt. The way in which the different direct methods define the critical depth and layering effects for both sleeve friction and point resistance is, for the most part, what separates the methods available.

An "indirect method" is taken to refer to static prediction methods that require intermediate correlations in order to predict pile capacity from CPT data. It must be realized that, unlike the direct methods, most indirect methods were not formulated specifically for use with CPT data. As such, any discrepancies between the predicted and measured pile capacities using the indirect methods may not be due solely to problems inherent to these methods. Table 3 lists the thirteen predictive methods evaluated. The first six methods are direct methods whereas the remaining seven are indirect methods. In each case the CPT data used was that shown in Fig. 2.

Most pile prediction methods are relatively difficult and time consuming to implement without the aid of a computer. This is especially true when near continuous CPT data is used. For each of the prediction methods used in this study a computer program was written using commercially available spreadsheet software. The spreadsheet is seen as a powerful engineering computational tool that is well suited to geotechnical engineering design. The spreadsheet is particularly well adapted for performing sensitivity analyses and therefore rapid evaluation of input parameters. Perhaps the greatest attraction of using spreadsheets, however, is that the programmer/operator requires little computer programming background.

Examples of predicted and measured pile capacities for one of the direct and one of the indirect methods are shown in Figs. 6 and 7, respectively. Fig. 6 shows the LCPC CPT (French) method and Fig. 7 shows the Dennis and Olson (modified API) method. Note that the LCPC method predicts the capacity of both the smaller piles and the tests on the larger pile with excellent

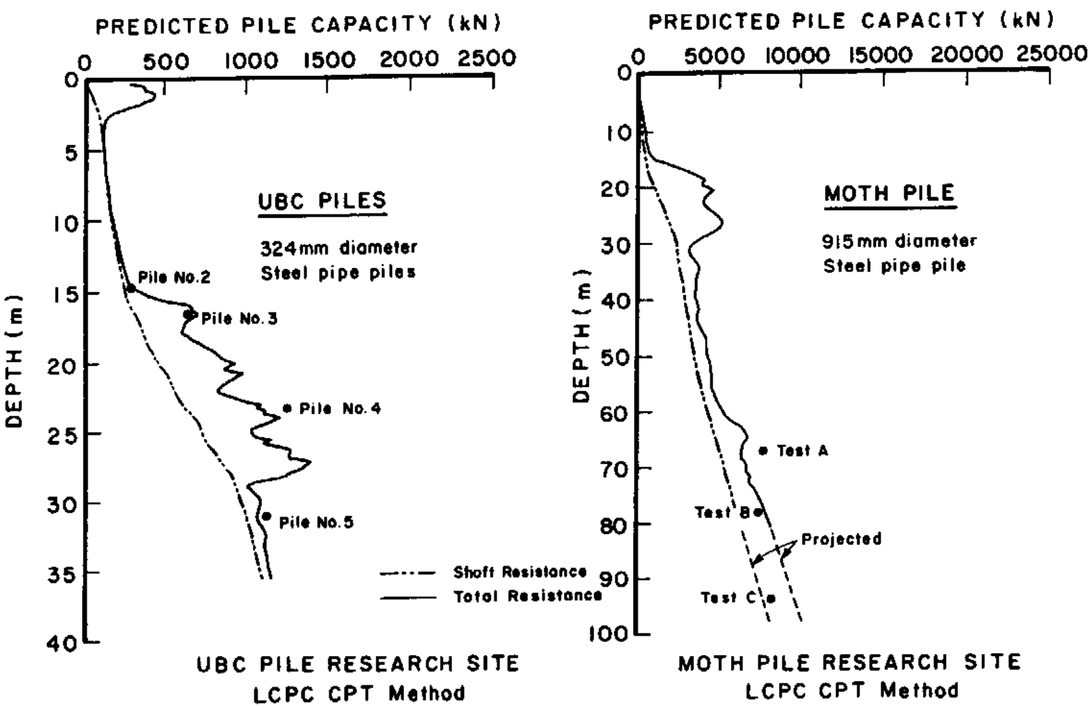

Fig. 6. Predicted axial capacity vs. depth for UBC and MOTH pile test sites using LCPC method.

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agreement. The Dennis and Olson method predicts the capacities of the smaller piles quite well but significantly overpredicts the capacities of the tests on the larger pile. This trend is seen for all of the indirect methods evaluated. Results for pile 1 are not included in Figs. 6 and 7 because the predicted pile capacities shown include the resistance in the upper sand fill which was not acting on pile 1. To predict the capacity of the 915 mm diameter pile at depths greater than 75 m the CPT profile was projected assuming a continued linear increase (see Fig. 2). Available borehole data to a depth of 130 m suggests this is a reasonable assumption.

Fig. 8 summarizes the results of all the methods in the form of bar charts. For reasons mentioned previously, the results for pile No. 1 are not included. Note that both the direct and indirect methods provided reasonable predictions of the measured capacities of the smaller piles. The direct methods, the Zhou et al method to a lesser extent, also predicted the capacity on the larger pile quite satisfactorily. However, without exception, the indirect methods had predictions that were significantly in error and non-conservative when compared to the measured results for the large pile. Since the indirect methods generally did reasonably well in predicting the capacity of the smaller piles, and since the piles are all in the same deltaic soil deposits, the results suggest that scale effects are extremely important for the large diameter pile. Most of the indirect methods are empirical in nature and based upon observed results from piles considerably smaller than 915 mm in diameter and 100 m in length.

5 SUMMARY AND CONCLUSIONS

This paper briefly compares thirteen axial pile capacity methods with the results from eight full-scale pile load tests on six different piles. The piles were steel pipe piles driven into deltaic soil deposits. The length to diameter ratios (L/D) for the piles ranged from 40 to 100. The measured axial capacities ranged from 170 kN to 8,000 kN in soils that included organic silt, sand and clay.

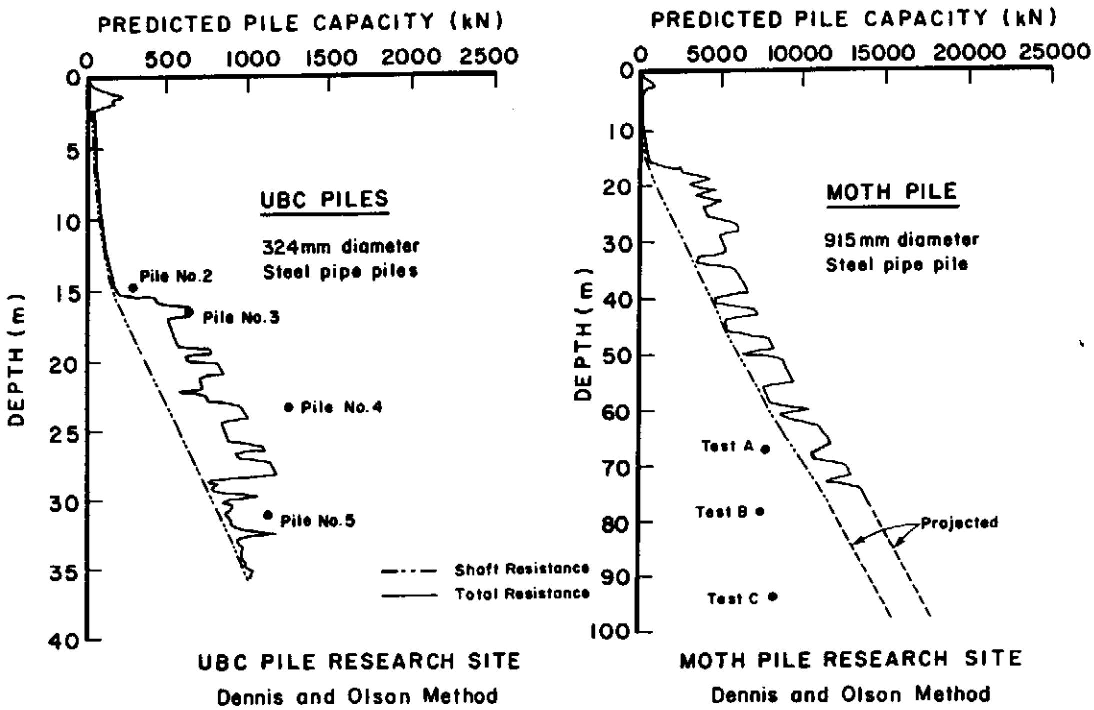

Fig. 7. Predicted axial capacity vs. depth for UBC and MOTH pile test sites using Dennis and Olson method.

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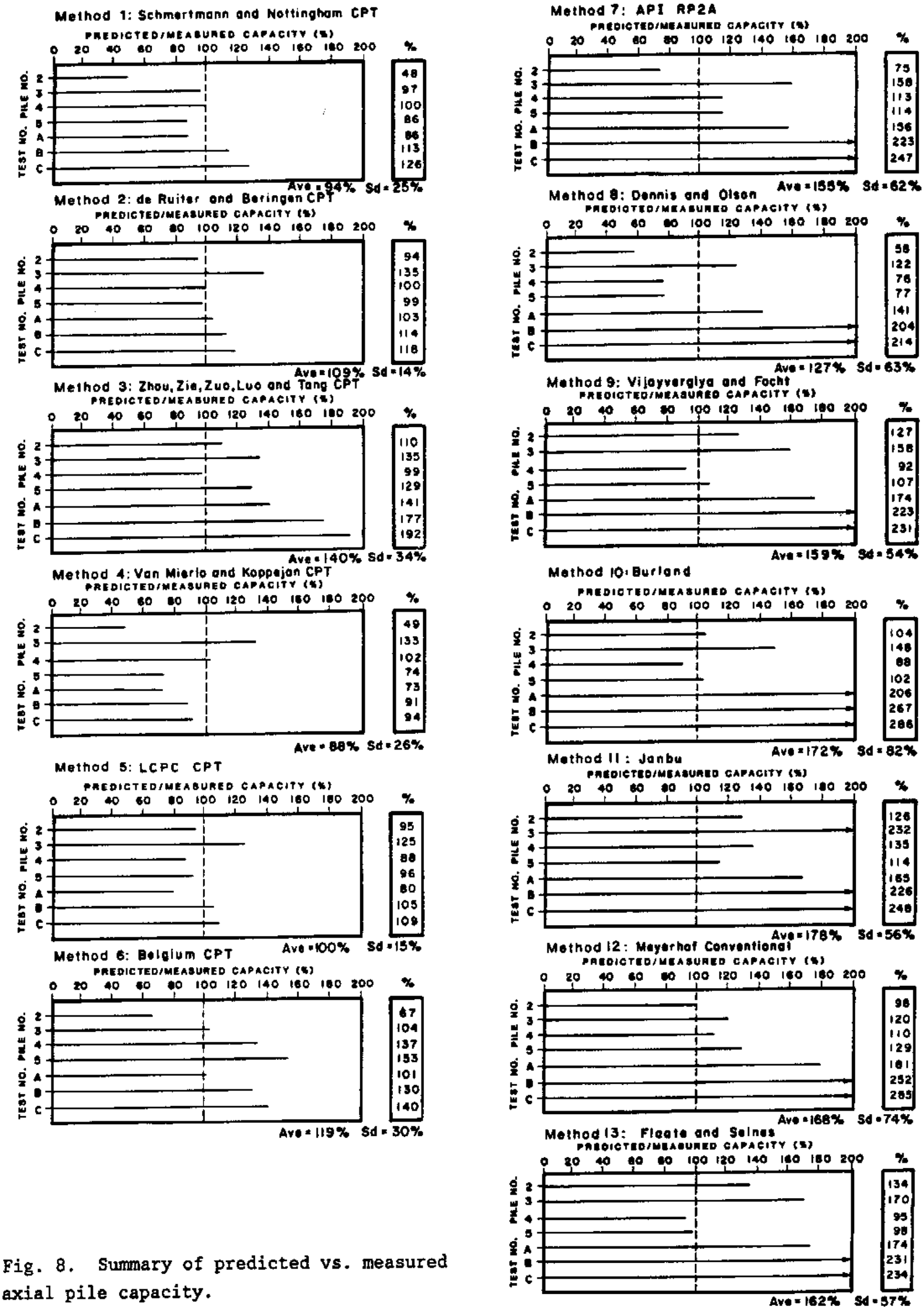

Fig. 8. Summary of predicted vs. measured axial pile capacity.

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CPT data was used for the prediction of pile capacity for the thirteen methods evaluated. The direct methods, which incorporate CPT-pile scaling factors, provided the best predictions for the piles and methods evaluated. Based on the results of this study the following three direct methods are preferred:

1. LCPC CPT (Bustamante and Gianesalli, 1982)
2. de Ruiter and Beringen CPT (1979)
3. Schmertmann and Nottingham CPT (1978)

For the piles tested, the LCPC (French) method is shown to be the best method with a maximum error of about 25%, an average error of 0%, and a standard deviation (Sd) of 15%. In addition, the LCPC does not directly require the CPT sleeve friction value other than to define soil type. This is a desirable feature since the cone bearing is generally obtained with more accuracy and confidence than the sleeve friction.

When piles are required to be supported in soft deltaic soils the CPT can be a highly economical method of providing extensive subsurface information to predict axial pile capacity.

The results of this study indicate that indirect CPT methods to predict axial pile capacity may significantly overpredict the capacity of large diameter, long piles (L/D > 75) supported in soft clayey soils.

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