EXPERIMENTALLY DERIVED RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND DRIVE PILE SETS

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A Project D report submitted in partial fulfilment of the requirements
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by
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ABSTRACT

This report presents the results of field testing relating to the driving of foundation piles using a variety of standard commercially available plant under normal operating conditions.

Piles were driven into cohesive soils in which shear strength testing was undertaken. The driving energy used and the base area of each pile were normalised on the basis of the energy input per unit area of pile base relative to the standard conditions of NZS:3604 Appendix D.

A relationship between shear strength and normalised pile set has been derived. A practical application of the relationship is presented as an aid to foundation design practitioners.

Ultimate pile capacities were evaluated on a theoretical basis and compared with pile testing undertaken for the preparation of NZS:3604 Appendix D. Recommendations are made regarding the use of static and dynamic pile capacity formulae.

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1. INTRODUCTION

1.1 Background

Since the introduction of NZS:3604 in 1978 many thousands of light timber framed structures have been successfully erected with foundations constructed using driven piles installed in accordance with Appendix D of that standard.

Appendix D specifies pile dimensions, minimum pile driving energy input and for design purposes gives a table of maximum pile sets of 25 mm, 50 mm, or 100 mm for different combinations of floor, wall and roof loadings together with joist spans and pile spacings. (Ref. 7)

Designers of these structures have always faced a difficulty in predicting the depths at which the pile driving set criterion will be achieved especially in deeply weathered soils. If piles are too shallow and the design set is not achieved, additional longer piles must be driven or a closer pile spacing used. If piles are too long material wastage occurs when pile tops are cut off.

In practice the initial piles driven on a site are effectively test piles. Following their driving pile lengths and spacings are modified as required. Most piling contracts however are let on a lump sum basis requiring a reasonable estimate of pile lengths to be made at the time of tendering.

Economies could be achieved in the design of bearers and the selection of pile spacings if information was made available to the designers enabling them to estimate the founding depths at which different pile sets would occurred.

A simple relationship has not been available whereby soil testing data from a typical geotechnical investigation could be related to the design pile set criterion of NZS:3604 giving an anticipated pile founding depth.

1.2 Objectives

The initial objectives of this research were:

(1) To perform a series of pile driving tests using a variety of standard commercially available plant under normal operating conditions.

(2) To drive piles comprising differing materials, diameters and lengths into cohesive soils of widely ranging strength characteristics.

(3) To define a series of acceptance criteria to ensure that only high quality data is used in the analysis.

(4) To derive a relationship between a soil parameter readily measurable by field testing, and pile sets normalised to NZS:3604 Appendix D driving criteria.

During the course of the research information became available leading to a broadening of the objectives :

(5) To compare the ultimate capacity of driven piles using dynamic analysis, static analysis, and relationships derived from pile load testing for NZS:3604.

2. TEST SITES

2.1 Introduction

Pile testing was performed under the author's direction in conjunction with geotechnical investigations and the construction of foundations on residential and commercial projects in the Auckland area. These projects were constructed over several years and were undertaken as a normal part of the consulting engineering work in which the author is engaged. During the period of enrolment for Project D further testing was undertaken to augment the earlier material.

Pile testing data was obtained from the raw test results of about 1000 driven piles installed at over 30 sites. Following the implementation of the pile test acceptance criteria (outlined in Section 4.2) this data base was reduced to 31 piles located at seven sites in the Auckland area.

2.2 Locations

The localities of the seven sites in the Auckland area from which pile driving test data was obtained for use in this study are shown on Fig. 2.1. General information about each site is given in Table 2.1.

2.3 Borehole Drilling

At each locality a geotechnical investigation was undertaken in conjunction with the foundation design for the particular structures to be erected. Boreholes were drilled and subsoil information obtained from which pile design criteria were evaluated.

The borehole drilling methods were chosen according to the anticipated soil conditions and pile founding depths. At all sites except D and F hand augered boreholes of 70 mm diameter were drilled until a target depth of usually 5 m was reached or the high strength of the soil prevented further augering.

Shear strengths were measured at regular intervals down the boreholes using a Pilcon shear vane. Soil samples were taken for further examination and laboratory testing. Most boreholes were extended using a Scala penetrometer until effective refusal with this device was reached (three consecutive blow counts of ten or more blows per 50 mm increment).

At sites D and F the site geology indicated that the depth to the underlying basement materials was in the 20 m to 30 m range, consequently machine boreholes were drilled. Continuous coring was taken with Pilcon shear vane testing in each core run prior to extrusion. In the higher strength or sandy materials standard penetration tests (SPT) were undertaken. Pocket penetrometer testing was also used.

The logs of boreholes from sites A to F are attached in Appendix A.

2.4 Soil Conditions

At all sites except D and F soil types comprised residual silts and clays formed by weathering of the Waitemata Formation . Undrained shear strengths (s_u) in these materials at the level at which sets were measured varied from 54 kPa to 250 kPa with remoulded shear strengths of 27 kPa to 79 kPa. Sensitivity ratios ranged from 1.6 to 2.7 indicating that the soils were of low sensitivity.

At sites D and F soil types comprised Pleistocene alluvium in the form of silts and clays. Undrained shear strengths (s_u) at the pile tips varied from 30 kPa to 160 kPa with remoulded shear strengths of 12 kPa to 63 kPa. Sensitivity ratios ranged from 1.9 to 2.5 indicating that these soils were also of low sensitivity.

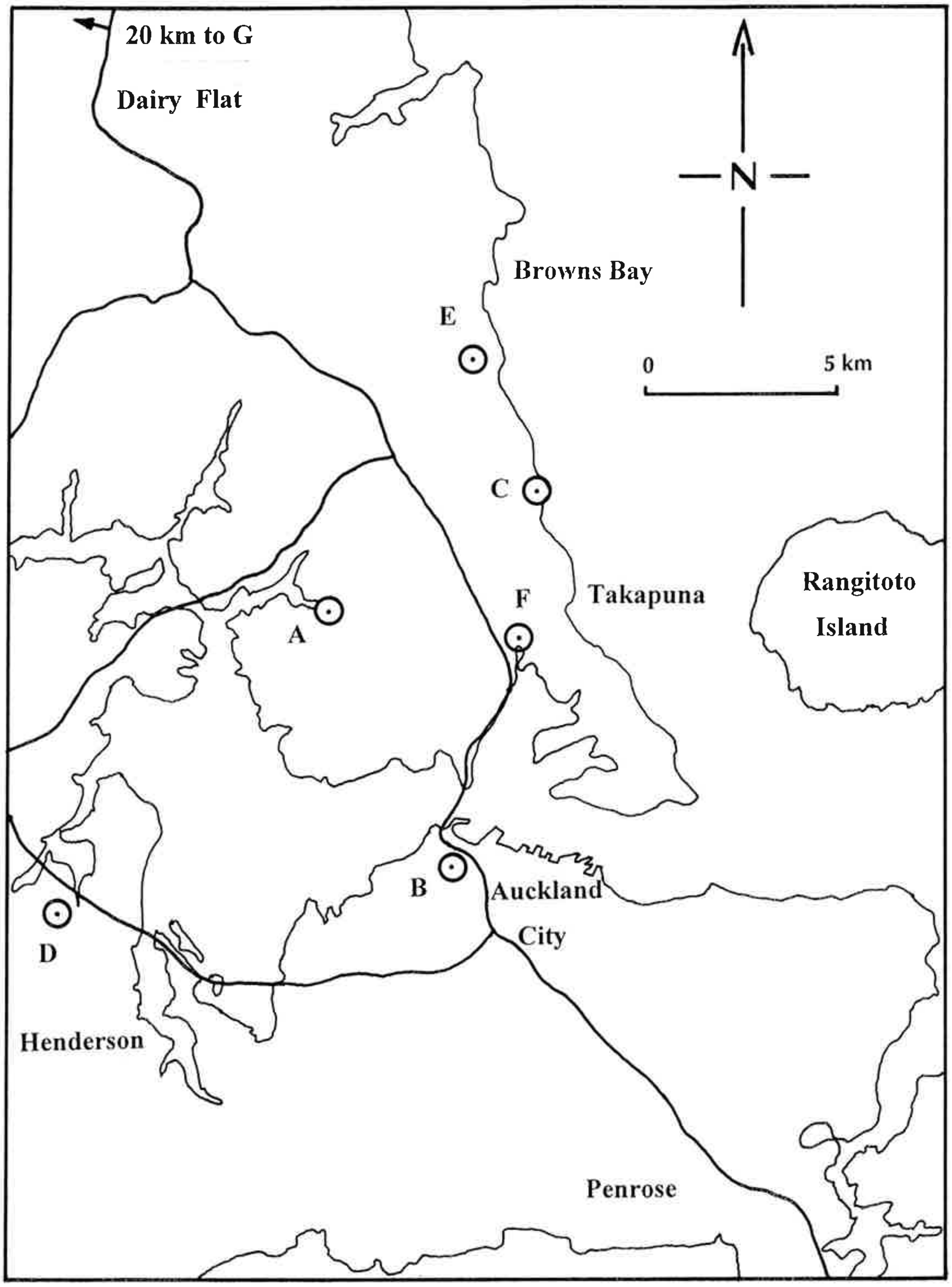

Fig. 2.1 Locations of Pile Test Sites

Test Ref No	Site Location	Soil Type	Soil Description
1	A - Stephanie Cl. Glenfield	Residual Waitemata	Silt some sand, trace clay, sl.-mod. plastic, light grey with orange staining.
2	B - Melford St. St. Marys Bay	Residual Waitemata	Silt, clayey white- grey orange.
3	"	Residual Waitemata	"
4	"	Residual Waitemata	"
5	"	Residual Waitemata	"
6	"	Residual Waitemata	"
7	"	Residual Waitemata	Silt clayey, white grey yellow brown.
8	"	Residual Waitemata	"
9	"	Residual Waitemata	Clay silty, yellow white grey.
10	"	Residual Waitemata	Clay sl. silty, white grey orange.
11	"	Residual Waitemata	"
12	C - The Esplanade Castor Bay	Residual Waitemata	Clay silty, light grey mottled orange.
13	"	Residual Waitemata	"
14	"	Residual Waitemata	"
15	"	Residual Waitemata	"
16	"	Residual Waitemata	"
17	"	Residual Waitemata	"
18	"	Residual Waitemata	"
19	"	Residual Waitemata	"
20	"	Residual Waitemata	Silt sl. clayey, yellow brown limonite staining, occasional grey mottles.
21	"	Residual Waitemata	"
22	D - Lincoln Rd. Lincoln	Pleistocene Alluvium	Clay silty, highly plastic, light grey, orange mottles.
23	"	Pleistocene Alluvium	Silt sandy, low plasticity, light grey, orange mottles.
24	E - Seaton Rd. Murrays Bay	Residual Waitemata	Silt sandy, sl. plastic, light grey, yellow brown.
25	F - Barry's Point Rd. Takapuna	Pleistocene Alluvium	Clay, trace sand, highly plastic, grey with black organic fragments.
26	"	Pleistocene Alluvium	Clay, trace sand, highly plastic, greenish-grey with black organic fragments.
27	"	Pleistocene Alluvium	Clay , highly plastic when remoulded, grey.
28	G - Pinchgut Rd. Kuakapakapa	Residual Waitemata	Silt trace sand, non plastic, lt.grey orange mottles, weathered siltstone gravel.
29	"	Residual Waitemata	Silt trace sand, non plastic, lt.grey orange mottles, rootlets.
30	"	Residual Waitemata	Silt trace sand, non plastic, lt.grey orange mottles, weathered siltstone gravel.
31	"	Residual Waitemata	Silt trace sand, non plastic, lt.grey orange mottles, rootlets.

Table 2.1 General Site Information

3 PILE DRIVING EQUIPMENT

3.1 Introduction

The piles installed at the seven sites in this study were driven by standard commercially available equipment. Five different pile driving companies were involved and each rig was built to suit a particular process and the ease and economy with which piles could be installed by the specialist contractors in competitive tendering situations.

3.2 Pile Driving Rigs

The types of vehicle on which the rigs were mounted included the following:

(1) Four wheel drive trucks and tractors with hammer weights of up to 320 kg. The light weight of these machines and their manouverability allowed access to difficult sites but generally limited their use to domestic situations requiring smaller piles.

(2) Track mounted hydraulic excavators with hammer weights of up to 560 kg. The masts on these larger machines were capable of accommodating longer piles of larger diameter capable of resisting greater loads.

(3) Track mounted crane rigs of up to 35 tonne with hammer weights of up to 3200 kg. These large rigs could instal piles in sections of up to 12 m length which were then connected to the previously driven section using the appropriate method for the type of pile material. The application of these large piles is usually limited to commercial sites.

3.3 Driving Hammers

At all test sites pile driving rigs used drop hammers for driving the piles. Two different types of hammer release mechanism were used, clutch released or hydraulic released.

3.3.1 Clutch Release Mechanisms

On sites A, D, F and G, hammer weights ranged from 180 kg to 3200 kg and were lifted by a wire cable and winch. When a clutch on the winding drum was released the falling hammer dragged the cable reversing the direction of the drum and allowing the hammer to fall onto the pile head .

The use of this type of driving method presented the following difficulties:

(1) To control the drop height the operator read markings on the mast of the rig often at steep angle up from the position where the controls were located. The operator adjusted the hammer lift height by eye estimating the difference between the top and bottom of the pile stroke, (usually within a tolerance of about 5% to 10%).

(2) At the bottom of each stroke the operator applied the brake on the winding drum, preventing the drum from over running and tangling the cable, then disengaged the clutch to lift the hammer for the next stroke. These operations took a fraction of a second to perform. If the brake was applied before the instant of hammer impact the hammer acceleration would be reduced before hitting the pile but sufficient stretch in the cables would take place to still allow the impact to occur.
The timing of the brake application could therefore vary the energy input considerably. When an accurate set measurement was required however, operators usually concentrated closely to give the maximum energy input.

3.3.2 Hydraulic Release Mechanisms

On sites B, C and E, hammer weights ranged from 275 kg to 560 kg and a variation on the above driving method was used. A double acting hydraulic ram pulled the cable raising the hammer. At the end of a one metre stroke the hydraulics returned the ram quickly to its initial position causing minimal resistance to the cable and hammer. The design of the hammer assembly allowed for the set of the pile following the previous hammer blow maintaining a standard one metre hammer drop height.

3.4 Pile Helmets and Driving Caps

With the smaller timber piles the driving hammers fell directly onto the pile head without the use of pile helmets or driving caps. These piles were not subject to hard driving onto bedrock and consequently the pile heads were not usually damaged by this installation method. Where minor damage did occur sufficient excess pile length was usually available to enable the pile top to be sawn off.

In the case of the steel tube piles a short dolly with timber packing over a steel cap was used. The timber packing was replaced after driving every few piles.

Driving of the reinforced concrete piles was achieved using a short helmet with a synthetic lurethane laminated packer. This packer cushions the blow on the concrete and unlike timber will not char when subjected to the heat generated by prolonged driving. (Ref. 10 & 13)

4. PROCEDURE

4.1 Introduction

This section describes a set of criteria applied to the raw pile data base to determine the suitability of data for inclusion in the study. The methods used for pile set measurement are also presented.

4.2 Pile Test Acceptance Criteria

In order to ensure that high quality data was used in the analysis, a set of acceptance criteria was formulated with respect to pile installation and soil testing results. To be included in the data base of the study each of the following nine pile test acceptance criteria conditions had to be met:

(1) Piles needed to be located within 3 metres of a borehole, unless multiple boreholes drilled on the site indicated that minimal lateral variability of soil conditions occurred in the area where the piles were installed.

(2) The soil conditions over the depth where the set was measured were found to be uniform in the adjacent borehole.

(3) Boreholes were tested at regular intervals using a hand held Pilcon Shear Vane calibrated in accordance with BS1377 within the previous 12 months.

(4) Piles driven into cohesive materials only were considered (silts and clay). (Data from non-cohesive soils in which scala penetrometer testing was undertaken was initially included but discounted due to inconsistency of results. Scala testing often has additional friction on the rods during testing and results can vary laterally especially when the depth is approaching an interface with stronger underlying materials.)