EXPERIMENTALLY DERIVED RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND DRIVEN PILE SETS

A Project D report submitted in partial fulfilment of the requirements
for the Degree of Master of Engineering

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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.

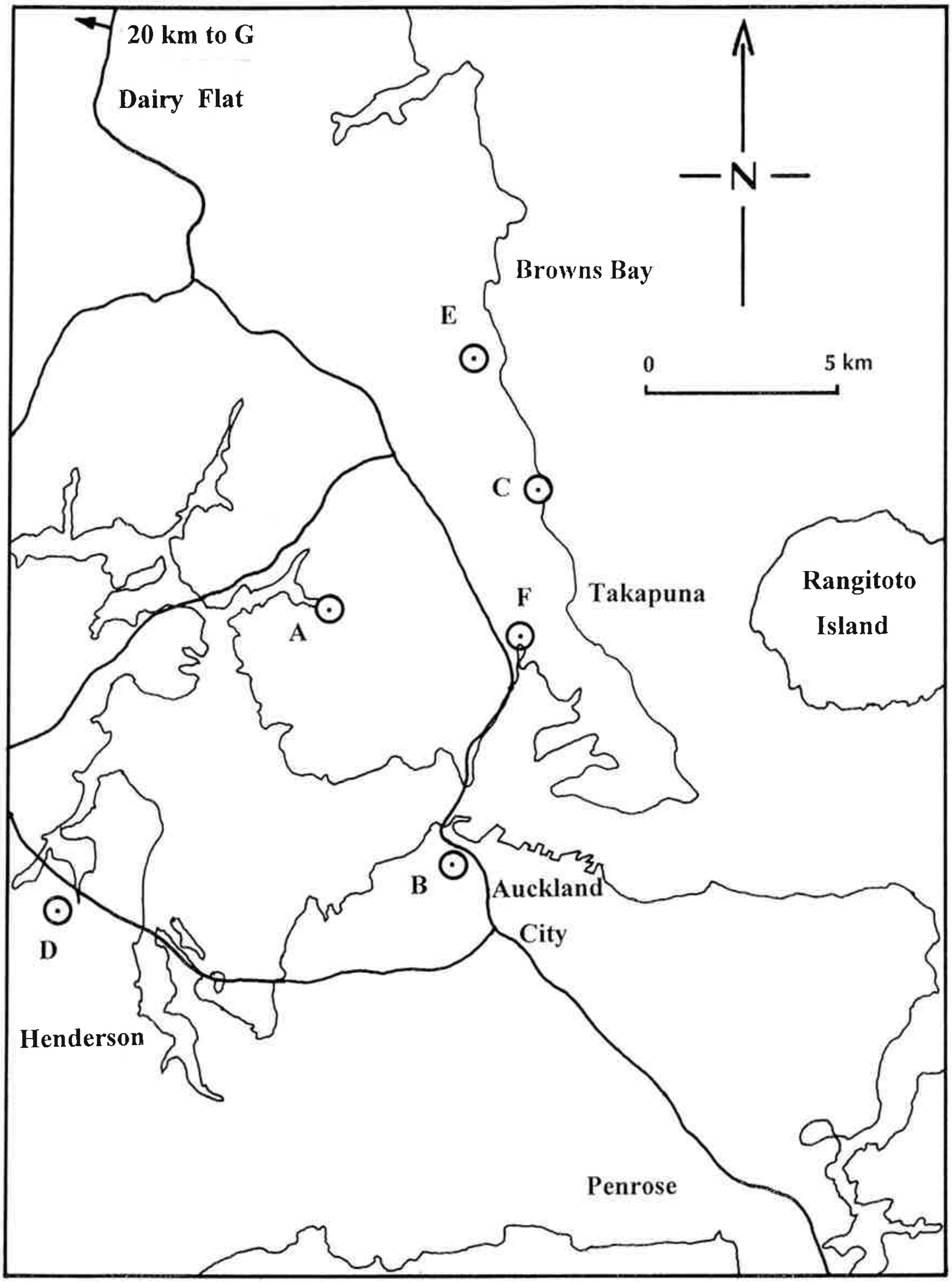

Fig. 2.1 Locations of Pile Test Sites

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.)

(5) During pile installation the pile sets, hammer drop heights, pile diameters (top and bottom) and lengths were all accurately measured.

(6) The formed building platform on which piles were driven was accurately levelled relative to the positions from which boreholes were drilled.

(7) When piles were driven through high strength material lubricants (eg. water), were not used to reduce the driving resistance.

(8) The piles were driven continuously without delays which would otherwise give an apparent set improvement with time as excess pore water pressures dissipated and shaft skin friction took effect.

(9) The pile ends were square and the hammer drops axial.

From the raw data base of 1000 test piles available for potential inclusion in the study, it was found that only 31 piles met all the above acceptance criteria. The project analysis uses test data from these piles.

4.3 Pile Set Measurement

At the seven sites used in this study pile set measurements were recorded using the techniques normally used by each of the piling contractors. On some sites cardboard set cards were attached to piles and a datum beam held close to the pile allowing a pencil mark to be made during driving. On other sites a chalk mark was made directly onto the pile.

All piles were driven to either a pre-defined depth below ground level, refusal or a minimum set in accordance with NZS:3604 Appendix D driving requirements. Consequently temporary compression measurements were not always recorded.

5. TEST RESULTS

5.1 Introduction

Borehole soil testing data together with data recorded from pile testing in accordance with Section 4 is presented and discussed.

5.2 Soil Testing Data

The results of field shear strength testing using a Pilcon shear vane are given in Table 5.1. Peak and remoulded shear strengths are recorded for the level at which sets were measured. Soil sensitivity ratios of peak to remoulded shear strength are also given.

Peak and remoulded shear strengths were measured over the driven depth of each pile shaft and the mean shear strength calculated applying a weighting for the thicknesses of the various layers identified on the borehole logs. Any other tests which were undertaken on the soil at the pile base level are also included.

5.3 Pile Driving Data

Table 5.2 records the data obtained from each pile driving test. Pile materials comprised predominantly Tanilised timber (26 piles) with 2 piles being spiral welded steel tube and 3 piles using steam cured reinforced concrete.

The diameters of pile tops and bases were measured together with pile lengths allowing pile masses to be calculated. Pile densities were assumed to be 600 kg/m^3 for timber (following discussions with pile installers (Ref. 4 & 6)), 7850 kg/m^3 for steel and 2400 kg/m^3 for reinforced concrete. Hammer masses recorded were as given by each contractor. Driven pile depths, hammer drop height and driving sets per were recorded by the author or staff working under his direction.

Test Ref No	Site Pile Ref No	Dist. from Pile to BH	Fnd.Level on Borelog	Fnd.Level Su (Peak) Corr. (kPa)	Fnd.Level Su (Rem) Corr. (kPa)	Fnd.Level Su (Pk+Rem)/2 Corr. (kPa)	Fnd.Level Sensitivity Ratio	Fnd.Level Other Tests	Mean Shaft Su (Peak) Corr. (kPa)	Mean Shaft Su (Rem) Corr. (kPa)
1	#1	0.5m to BH1	1.2	140	69	105	2.0		145	80
2	#1	1.5m to BH2	2.6	100	43	72	2.3		94	43
3	#2	3.9m to BH2	1.6	95	54	75	1.8		87	43
4	#8	2.0m to BH2	2.3	108	46	77	2.3		98	45
5	#14	2.9m to BH2	3.2	54	27	41	2.0		85	41
6	#17	2.6m to BH1	2.3	92	48	70	1.9		89	47
7	#24	1.6m to BH1	2.8	111	54	83	2.1		70	37
8	#25	2.8m to BH1	2.4	111	54	83	2.1		64	34
9	#29	1.6m to BH1	3.0	73	41	57	1.8		72	38
10	#30	0.8m to BH1	3.1	73	41	57	1.8		73	39
11	#31	2.6m to BH1	2.9	73	41	57	1.8		72	38
12	#5	2.9m to BH5	2.7	132	79	106	1.7		128	70
13	#13	2.4m to BH5	2.1	132	71	102	1.9		128	74
14	#14	1.0m to BH5	2.0	132	71	102	1.9		127	74
15	#15	1.6m to BH5	2.0	132	71	102	1.9		127	74
16	#16	0.4m to BH5	2.6	132	79	106	1.7		126	74
17	#19	2.2m to BH5	2.0	132	71	102	1.9		127	74
18	#20	2.0m to BH5	2.0	132	71	102	1.9		127	74
19	#21	2.6m to BH5	2.2	132	71	102	1.9		128	74
20	#25	2.9m to BH2	2.7	154	-	-	-	WC 42	145	-
21	#27	3.7m to BH2	2.9	200 *	-	-	-		148	-
22	#B 21	1.8m to BH3	3.4	30	12	21	2.5		95	47
23	#B 21	1.8m to BH3	8.4	52	-	-	-	WC 41 LL 63, PI 38	58	26
24	#33	2.9m to BH 1	4.3	250 *	-	-	-		140	64
25	#P 7	4.8m to BH3	15.2	83	35	59	2.4	WC 56	92	30
26	#P 7	4.8m to BH3	19.7	107	55	81	1.9		95	36
27	#P 7	4.8m to BH3	23.7	160	63	112	2.5	WC 47	100	39
28	#1	0.6m to BH1	2.0	73	46	60	1.6		158	59
29	#1	0.6m to BH1	3.0	103	38	71	2.7		141	51
30	#2	2.6m to BH1	2.0	73	46	60	1.6		158	59
31	#2	2.6m to BH1	3.0	103	38	71	2.7		141	51

* Estimated

Table 5.1 Soil Testing Data

Test Ref No	Pile Type	Pile base Dia. (mm)	Pile top Dia. (mm)	Pile Length (Total, m)	Pile Mass (kg)	Driven Depth (m)	Hammer Mass (kg)	Drop Height (m)	Set/Blow Rec. (mm)
1	Timber	140	155	2.0	21	1.2	180	1.0	8.5
2	Timber	145	165	3.0	34	2.6	320	1.5	30
3	Timber	145	165	3.0	34	1.6	320	1.5	35
4	Timber	160	180	3.6	49	2.0	320	1.5	27
5	Timber	150	175	3.6	45	2.0	320	1.5	45
6	Timber	145	170	3.6	42	1.8	320	1.5	40
7	Timber	175	200	4.2	70	2.0	320	1.5	30
8	Timber	170	195	4.2	66	2.0	320	1.5	25
9	Timber	170	195	4.2	66	2.2	320	1.5	30
10	Timber	170	195	4.2	66	2.1	320	1.5	35
11	Timber	170	195	4.2	66	2.0	320	1.5	35
12	Timber	175	195	3.0	49	2.0	275	1.8	20
13	Timber	150	165	2.4	28	1.9	275	1.8	18
14	Timber	150	165	2.4	28	1.8	275	1.8	21
15	Timber	150	165	2.4	28	1.8	275	1.8	20
16	Timber	175	195	3.0	49	2.4	275	1.8	11
17	Timber	150	165	2.4	28	1.8	275	1.8	24
18	Timber	150	165	2.4	28	1.8	275	1.8	25
19	Timber	175	195	3.0	49	2.0	275	1.8	25
20	Timber	175	200	4.2	70	3.4	275	1.8	5.5
21	Timber	175	200	4.2	70	3.6	275	1.8	7
22	Steel Tube	250	250	11.0	780	3.0	3100	1.2	200
23	Steel Tube	250	250	11.0	780	8.0	3100	1.2	144
24	Timber	150	175	4.2	53	3.9	560	0.9	10
25	Concrete	275 x 275	275 x 275	18.0	3267	14.0	3200	1.0	75
26	Concrete	275 x 275	275 x 275	27.0	4901	18.5	3200	1.0	47
27	Concrete	275 x 275	275 x 275	27.0	4901	22.5	3200	1.0	28
28	Timber	200	225	4.2	90	2.0	545	0.9	22
29	Timber	200	225	4.2	90	3.0	545	0.9	16
30	Timber	200	225	4.2	90	2.0	545	0.9	26
31	Timber	200	225	4.2	90	3.0	545	0.9	15

Table 5.2 Pile driving Data

6 ANALYSIS OF RESULTS

6.1 Introduction

In order to evaluate the results from Section 5 using data from piles of different diameter driven with varying energy levels, the measured sets have been normalised using three alternative methods:

6.2 Gross Energy Input

In this method recorded pile sets are normalised by applying a factor to give an energy input per unit area of pile base equivalent to the minimum which would be achieved under NZS:3604 Appendix D.

Piles installed in accordance with NZS:3604 1990 Appendix D Section 5.1 are required to be driven:

> *"..by a hammer having a mass M of not less than 200 kg falling freely through a distance h of not less than 480/M metres (where M is in kilograms)... The free fall of the hammer has been defined so as to ensure that the hammer will deliver to the top of the pile not less than 4800 J of energy per blow.* (Ref. 7)

This is a gross energy input and was based on the original 1974 research data for Appendix D in which the hammer mass was 450 lb (204 kg) and the drop height 94.5 in (2.40 m). (Ref. 2)

$$W.g.h = 204 \times 9.81 \times 2.40 = 4803 \text{ J}$$

A tractor mounted fence post driving rig was used with the hammer being lifted by a wire cable attached to a pulley system activated by a hydraulic ram. This was a similar system to the hydraulic release mechanism outlined in section 3.3.2 above. The net energy delivered to the pile head was assessed applying an 80% efficiency factor to the hammer drop height. (Ref. 2)

In normalising the sets on a gross energy input basis the gross energy delivered to the head of each pile tested in the current study was divided by 4800 to give the Gross Energy Ratio in Table 6.1.

Allowance has been made for the variation in pile base area by dividing the base area of each pile by the minimum pile area allowed in NZS:3604 (a 140 mm dia. pile driven small end first), to give the Area Ratio in Table 6.1.

The above two ratios are combined by dividing the Area Ratio by the Gross Energy Ratio to give the Normalising Factor (Gross) which is applied to the recorded sets giving the Set/Blow Norm (Gross). The resulting variation in shear strength with normalised set is plotted in Fig. 6.1 giving a relationship with a comparatively small scatter.

Regression analysis was undertaken as detailed in Appendix B with the resulting best fit curve:

$$s_{U\,(PEAK)} = 240\ e^{-0.02322\ SET}$$

or in terms of set:

$$set = 236 - 43.1 \log_e s_{U\,(PEAK)}$$

This relationship is plotted on Fig. 6.1 and gives 86.21% explained variation with comparatively low residuals.

The above formulae are limited to a reliable range of about 30 kPa and 200 kPa peak corrected shear strength. The set is normalised as explained previously.

Other relationships which are presented in Sections 6.3 and 6.4 do not achieve as good a correlation. Appendix C details the practical application of the upper formula with typical examples working through the Normalising Factor (Gross).

Test Ref No	Hammer Mass (kg)	Drop Height (m)	Gross Energy Input (J)	Gross Energy Ratio	Area Ratio	Normalising Factor (Gross)	Set/Blow Rec. (mm)	Set/Blow Norm. (mm) (Gross)
1	180	1.0	1766	0.37	1.00	2.72	8.5	23
2	320	1.5	4709	0.98	1.07	1.09	30	33
3	320	1.5	4709	0.98	1.07	1.09	35	38
4	320	1.5	4709	0.98	1.31	1.33	27	36
5	320	1.5	4709	0.98	1.15	1.17	45	53
6	320	1.5	4709	0.98	1.07	1.09	40	44
7	320	1.5	4709	0.98	1.56	1.59	30	48
8	320	1.5	4709	0.98	1.47	1.50	25	38
9	320	1.5	4709	0.98	1.47	1.50	30	45
10	320	1.5	4709	0.98	1.47	1.50	35	53
11	320	1.5	4709	0.98	1.47	1.50	35	53
12	275	1.8	4856	1.01	1.56	1.54	20	31
13	275	1.8	4856	1.01	1.15	1.13	18	20
14	275	1.8	4856	1.01	1.15	1.13	21	24
15	275	1.8	4856	1.01	1.15	1.13	20	23
16	275	1.8	4856	1.01	1.56	1.54	11	17
17	275	1.8	4856	1.01	1.15	1.13	24	27
18	275	1.8	4856	1.01	1.15	1.13	25	28
19	275	1.8	4856	1.01	1.56	1.54	25	39
20	275	1.8	4856	1.01	1.56	1.54	5.5	8
21	275	1.8	4856	1.01	1.56	1.54	7	11
22	3100	1.2	36493	7.60	3.19	0.42	200	84
23	3100	1.2	36493	7.60	3.19	0.42	144	60
24	560	0.9	4944	1.03	1.15	1.11	10	11
25	3200	1.0	31392	6.54	4.91	0.75	75	56
26	3200	1.0	31392	6.54	4.91	0.75	47	35
27	3200	1.0	31392	6.54	4.91	0.75	28	21
28	545	0.9	4812	1.00	2.04	2.04	22	45
29	545	0.9	4812	1.00	2.04	2.04	16	33
30	545	0.9	4812	1.00	2.04	2.04	26	53
31	545	0.9	4812	1.00	2.04	2.04	15	31

Table 6.1 Normalised Sets (Gross Energy Basis)

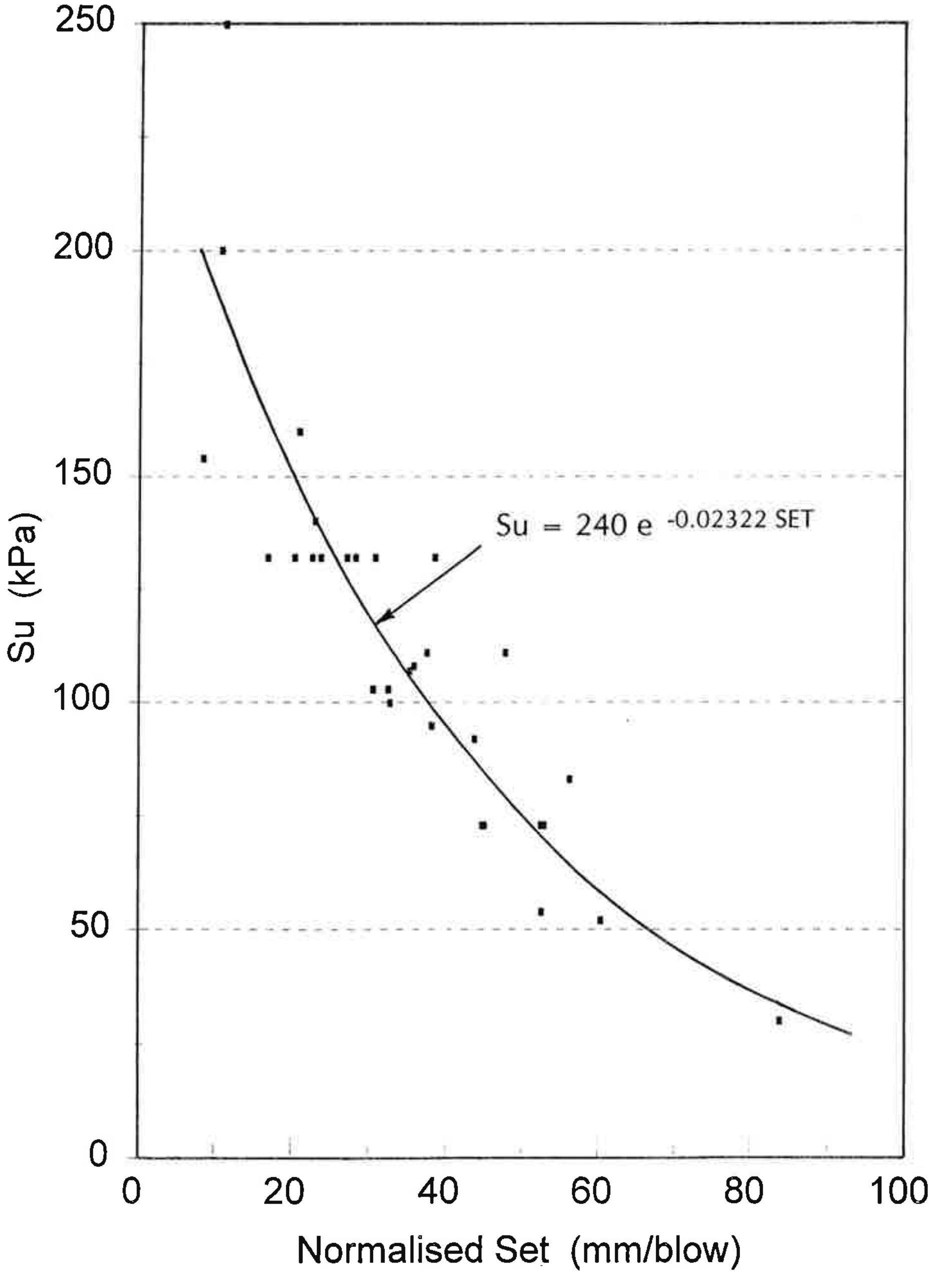

Fig. 6.1 Shear Strength vs. Normalised Set(Gross Energy Basis)

6.3 Net Energy Input

The normalised set in Fig. 6.1 does not take account of the large difference in net energy transmitted to each pile and available to cause the driving action. In this method the previous approach is extended by making an allowance for the variation in efficiency factors for hammer drop heights, coefficients of restitution and hammer blow efficiencies. These factors together determine the net energy input at the pile top which can then be compared with the standard energy available under NZS:3604 driving conditions to produce a set which has been normalised on a net energy basis.

In Table 6.2 an 80% efficiency factor has been applied to all hammer drop heights. There is some debate from pile driving companies that this is too conservative especially with the hydraulically operated release mechanisms. In the absence of any specific test data on individual rigs the general value given in literature was adopted. (Ref. 1)

The coefficient of restitution (e) varies with the type of pile driving hammer, pile material and pile head condition (helmet, packing, dolly, driving cap, etc.). Three different values of (e) were applied to the piles used in the study as follows:

(1) Drop hammers striking the tops of timber piles, $e = 0.25$.

(2) Drop hammers driving steel tube piles with a short dolly and timber packing over a steel cap, $e = 0.32$.

(3) Drop hammers driving reinforced concrete piles with a short helmet and synthetic lurethane synthetic packer, $e = 0.40$. (Ref. 13)

Blow efficiency (n) calculations then follow using the relationship:

$$n = \frac{W + Pe^2}{W + P}$$

where: W = hammer weight

P = pile weight

e = coefficient of restitution

(Ref. 1)

The Net Energy Input is calculated from:

$$E_{NET} = \text{hammer mass} \times g \times \text{drop height} \times \text{hammer efficiency} \times \text{blow efficiency}$$

The Net Energy Ratio is the ratio of the Net Energy Input for each pile to the Net Energy Input for a standard 140 mm dia. pile driven to NZS:3604 conditions as given in Section 6.2 above. It is assumed that this standard pile is of 2.7 m length (the mean pile length covered by the standard), and that the taper complies with NZS:3605 1977, *Specification For Load Bearing Round Timber Piles and Poles.* (Ref. 8). At a density of 600 kg/m^3 the pile mass is 28.15 kg giving the following net energy calculation using the above equation:

$$E_{NET\ 3604} = 200 \times 9.81 \times 2.4 \times 0.8 \times \frac{200 + 28.15 \times 0.25^2}{200 + 28.15}$$

$$= 3331 \text{ J}$$

The Net Energy Ratio and the Area Ratio are combined to give the Normalising Factor (Net) which is applied to the recorded sets giving the Set/Blow Norm (Net).

The resulting variation in shear strength with normalised set is presented graphically in Fig. 6.2. Attention is drawn to the locations of the numbered points corresponding to the four piles with greatest driven depth, (more than 4 m). Three of these points have moved a significant distance from the best fit line when the normalising factor includes the net energy, (compared with Fig. 6.1). Reference to Fig. 6.2 shows that for these piles the Net Energy Ratio has almost halved from the previous gross energy ratio. This is due mainly to the low blow efficiencies used (0.49 to 0.58) compared with the other piles (0.81 to 0.92), and increased coefficient of restitution (0.40) compared with (0.25 to 0.32). These variations occur partly as a result of the large pile masses involved (up to 4901 kg).

Test Ref No	Coeff. of Rest. (e)	Blow Eff (n)	Net Energy Input (J)	Net Energy Ratio	Area Ratio	Normalising Factor (Net)	Set/Blow Rec. (mm)	Set/Blow Norm. (mm) (Net)
1	0.25	0.90	1277	0.38	1.00	2.61	8.5	22
2	0.25	0.91	3427	1.03	1.07	1.04	30	31
3	0.25	0.91	3427	1.03	1.07	1.04	35	36
4	0.25	0.88	3296	0.99	1.31	1.32	27	36
5	0.25	0.88	3331	1.00	1.15	1.15	45	52
6	0.25	0.89	3354	1.01	1.07	1.07	40	43
7	0.25	0.83	3134	0.94	1.56	1.66	30	50
8	0.25	0.84	3161	0.95	1.47	1.55	25	39
9	0.25	0.84	3161	0.95	1.47	1.55	30	47
10	0.25	0.84	3161	0.95	1.47	1.55	35	54
11	0.25	0.84	3161	0.95	1.47	1.55	35	54
12	0.25	0.86	3338	1.00	1.56	1.56	20	31
13	0.25	0.91	3547	1.06	1.15	1.08	18	19
14	0.25	0.91	3547	1.06	1.15	1.08	21	23
15	0.25	0.91	3547	1.06	1.15	1.08	20	22
16	0.25	0.86	3338	1.00	1.56	1.56	11	17
17	0.25	0.91	3547	1.06	1.15	1.08	24	26
18	0.25	0.91	3547	1.06	1.15	1.08	25	27
19	0.25	0.86	3338	1.00	1.56	1.56	25	39
20	0.25	0.81	3147	0.94	1.56	1.65	5.5	9
21	0.25	0.81	3147	0.94	1.56	1.65	7	12
22	0.32	0.82	23927	7.18	3.19	0.44	200	89
23	0.32	0.82	23927	7.18	3.19	0.44	144	64
24	0.25	0.92	3637	1.09	1.15	1.05	10	11
25	0.40	0.58	14457	4.34	4.91	1.13	75	85
26	0.40	0.49	12352	3.71	4.91	1.32	47	62
27	0.40	0.49	12352	3.71	4.91	1.32	28	37
28	0.25	0.87	3340	1.00	2.04	2.04	22	45
29	0.25	0.87	3340	1.00	2.04	2.04	16	33
30	0.25	0.87	3340	1.00	2.04	2.04	26	53
31	0.25	0.87	3340	1.00	2.04	2.04	15	31

Table 6.2 Normalised sets (Net Energy Basis)

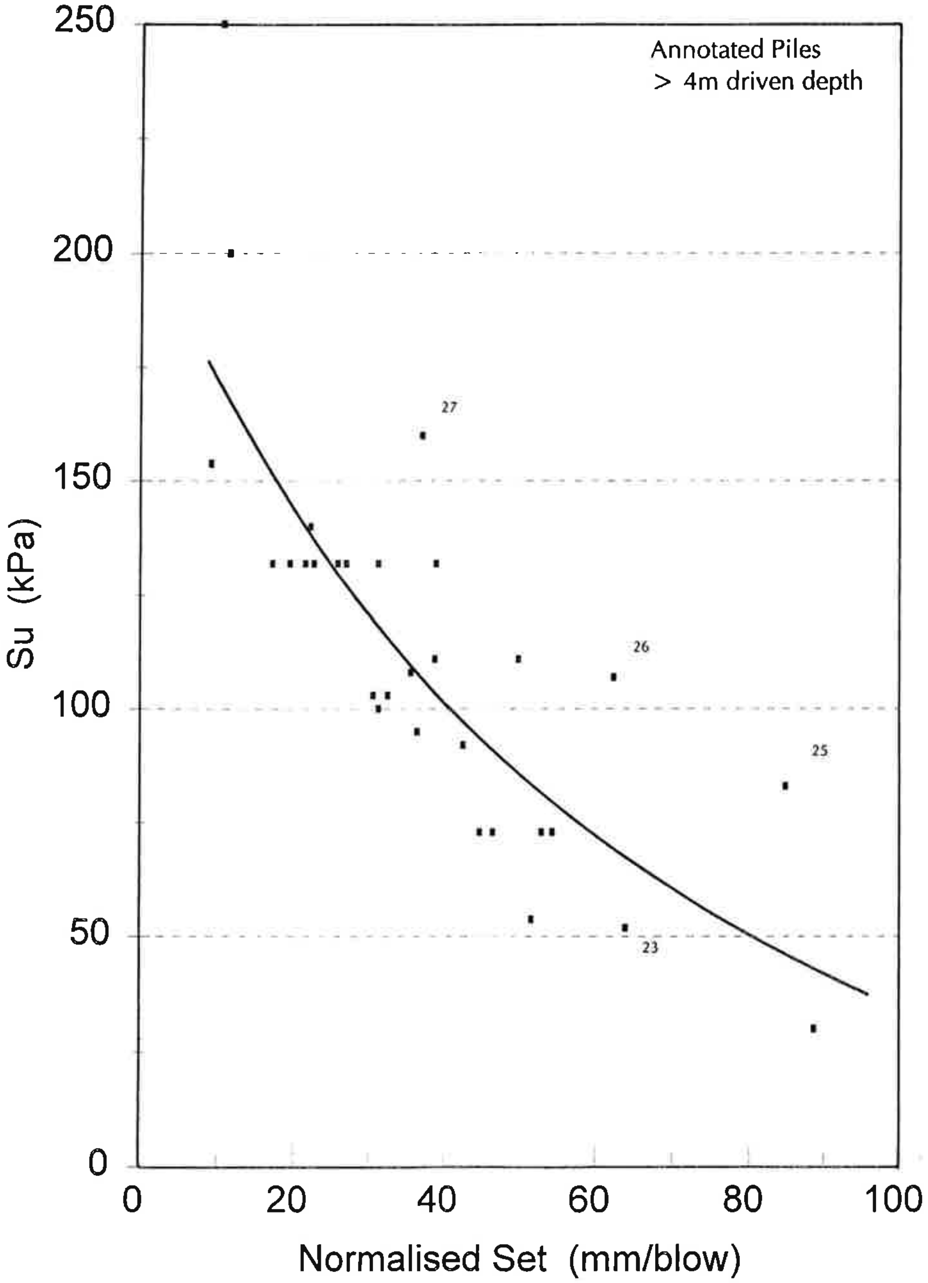

Fig. 6.2 Shear Strength vs. Normalised Set (Net energy Basis)

6.4 Shaft Resistance

In this method an allowance is made for the energy absorbed during driving due to skin friction on the pile shaft.

During the driving of the longer piles (8 m to 22.5 m, Test Ref No 23, 25, 26, 27) it was noted that as the pile bases passed through layers of comparatively low strength the sets did not increase as much as expected, indicating that shaft skin friction was absorbing energy.

In the analysis of ultimate pile bearing capacity the skin friction component is taken as the mean shaft peak shear strength weighted by an adhesion factor alpha, which Flaate has evaluated as a function of s_u (peak) and plasticity index PI, (as reported in Ref. 9). The alpha factor allows for the amount by which the soil adjacent to a pile shaft regains strength after the completion of driving and adheres to the shaft providing skin friction.

The author suggests however that during the driving process the soil adjacent to the pile shaft will be in a highly disturbed state and the driving resistance provided by shaft adhesion will be a function of s_u (remolded) rather than s_u (Peak).

In tables B1, B2, B3 and B4, in Appendix B, a series of alpha factors, 5%, 10%, 20% and 50% have been applied to the mean shaft s_u (remolded) acting on the pile giving a pseudo-static shaft skin friction during driving. This has been added to the assumed pile base resistance during driving (9 $s_{u(peak)}$ Ap) to give the total driving capacity. The previous Normalising Factor (Gross), derived in section 6.2, is then reduced by the proportion of the total driving capacity attributable to shaft skin friction, to give the Final Normalising Factor (Gross). This in turn is applied to the recorded pile set per blow to give the Set/Blow (SF Norm) which is plotted as the four graphs shown on Fig. 6.3.

The above procedure is repeated in tables B5, B6, B7 and B8 with the same alpha factors but using the Normalising Factor (Net) which incorporates the net energy delivered to the pile top. These results are plotted as the four graphs on Fig. 6.4.

A comparison within each family of these graphs shows that with increasing alpha factors the data points move closer to the y-axis, as would be expected. The effect is greater on deeper piles given their larger shaft areas. The degree of scatter in the data however increases with the alpha factor.

A comparison between these two families of graphs shows that normalising on a net energy basis displaces the curves further along the x-axis and introduces more scatter due mainly to the deeper piles as discussed at the end of Section 6.2 above.