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OSTERBERG CELL PILE LOAD TEST AT STYX MILL OVERBRIDGE

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INTRODUCTION

A bored pile constructed at the Styx Mill Bridge overpass, SH 1, in Christchurch, was load tested to determine axial compressive capacity. The testing technique involved using the Osterberg Load Cell, or O-cell. This innovative testing method is gaining widespread use worldwide because it provides a fast, effective, and economical means to conduct load tests on full-size deep foundations. The O-cell test at Styx Mill is the first in New Zealand and one of the objectives was to provide an opportunity for New Zealand engineers, contractors, owners, and other potential users of load test information to observe the test first-hand. The test was funded by Transit New Zealand and was conducted in conjunction with a workshop on bored pile design and construction organized by the authors at the University of Canterbury. Workshop attendees were on-site when the bored pile with the O-cell was installed, and were invited to attend the load test several days later. This report describes the site conditions, provides a brief background on the Osterberg-cell concept, describes the test conditions, and summarizes the results. Comparisons are provided between pile capacities predicted by conventional methods and the O-cell test results.

THE STYX MILL BRIDGE SITE

A cable tool borehole with standard penetration tests (SPT) at approximately one metre intervals was made close to the test pile location with the borelog shown in Figure 1. Generally, the upper 8.5 m of the subsurface stratigraphy at the test pile location consists of silts and sands with some peaty organics. These are underlain by sandy gravels to a depth of at least 17 m where the borehole was terminated. The water table is at 2.5 m.

A cone penetration test (CPT) was also made near to the test pile location and the resulting log is shown in Figure 2. Interpretation of the CPT data is given in Figure 3. The CPT stratigraphy is in good general agreement with the borelog showing layers of silts, silty sands, silty clays, sands and some peats in the upper 8 m. The sands and silty sands are shown to be generally loose condition becoming medium dense at depths approaching 8 m. Refusal was reached in gravel at 8.6 m and a qc value of 60 MPa.

The SPT blow count data are variable in the gravels below 8 m ranging from N = 39 (uncorrected) at 9.1 m to effectively zero where heaving of the hole is indicated.

TEST PILE CONSTRUCTION

The bored pile was constructed by Texco Drilling Ltd of Christchurch, during the period of May 5-10, 2002. The primary equipment used by Texco was a KATO bucket rig equipped with a casing oscillator (Figure 4). The method of construction involved supporting the hole with a steel casing and maintaining a sufficient head of water inside the casing to insure downward flow. This procedure prevented upward flow of groundwater at the tip of the excavation, which could affect pile end-bearing. First, a 600-mm diameter hole was predrilled to a depth of 7.0 m, followed by the insertion of a 625 mm O.D casing, 14.0 m in length. The oscillator was used to install the casing to a depth of 13.6 m and the remaining soil was excavated from inside the casing using a cleanout bucket. Excavation was completed prior to the date of the workshop field demonstration (9 May).

On the day of the demonstration, the rebar cage, with attached O-cell™ and instruments (Figure 5), was lowered into the shaft until the O-cell™ was at the target elevation. Concrete was to be placed into the shaft using a pump. Unfortunately, the pump tube became blocked soon after commencement of pumping. The reasons for the blockage included the large-size aggregate (20 mm specified) and the includion of some oversize oblong aggregate particles. Also, the pump line contained reducing connections which were obviously contributing to the difficulties.

Eventually, a decision was made to abandon the pour, and the cage was removed undamaged, the concrete removed from the hole, and the hole cleaned out. The hole was "rested" overnight with the drilling bucket left in the bottom of the hole to catch any suspended material and keep the bottom of the hole sound.

The following day (10 May), the rebar cage and O-Cell assembly were again lowered into the hole. Concrete was pumped successfully into the hole with the casing being pulled progressively by using the casing oscillator. On this occasion, Texco used a new galvanised steel tremie pipe and a concrete mix with maximum aggregate size of 12 mm was specified.

Assembly and installation of the O-cell™ and instrumentation was carried out under the direction and continuous supervision of Mr. Thomas Molnit of LOADTEST Asia Pte. Ltd. Figure 6 is a schematic diagram of the test pile configuration. Table A summarizes the bored pile as-built characteristics.

THE OSTERBERG LOAD CELL

The O-cell is a hydraulically driven, calibrated, sacrificial jacking device installed directly in the deep foundation. Since the device loads in two directions, upward against the side shear and downward against end-bearing, the O-cell makes it possible to obtain independent measurements of the two resistances. Because the O-cell derives it reaction forces from the deep foundation itself, no load testing beam, reaction piles, or other structural support are required. It is this ability to obtain separate measurements of side shear and end-bearing, combined with the cost advantages of eliminating a testing frame that have enabled the O-cell to replace conventional load testing for many applications. The O-cell also makes it possible to conduct pile load tests on many projects where any type of load test would not be economically feasible otherwise.

Instrumentation and measurements

Standard O-cell™ testing instrumentation included two bottom-plate telltales at the O-cell™ assembly with attached LVDTs (Linear Voltage Displacement Transducers) at pile top to measure expansion between lower and upper plates of the O-cell™ assembly. Compression of the pile above the O-cell™ assembly was measured using two traditional telltales extending above the top of pile. Telltale movements were monitored by LVDTs at the top of pile. Two LVDTs attached to a reference beam monitored the top of pile movement. The reference system was monitored for vertical movement using a digital level to a precision of 0.1 mm. A Bourdon pressure gage was used to measure the pressure applied to the O-cell™ at each load interval. The Bourdon pressure gage was used for operating the pump and for data analysis.

Test Procedure

The 200 mm diameter O-cell™ was pressurized in 15 equal loading increments to 30.3 MPa resulting in a bi-directional gross O-cell™ load of 597.8 kN. Loading was terminated after load interval 15 at which point the combined end bearing and lower side shear reached ultimate capacity. The O-cell™ was then depressurized in three decrements and the test was concluded. Load increments were applied using the Quick Load Test Method for Individual Piles (ASTM D1143 *Standard Test Method for Piles Under Static Axial Load*). Each successive load increment was held constant for eight minutes by manually adjusting the O-cell™ pressure. Approximately three minutes was used to move between increments. The data logger automatically recorded the instrument readings every 60 seconds, but herein only the 1, 2, 4 and 8 minute readings during each increment of maintained load are reported.

The load test at Styx Mill was conducted under the supervision of Mr. Molnit on May 13, 2002. The authors of this report and personnel from Texco Drilling and several local consulting firms observed the test.

TEST RESULTS

Combined End Bearing and Lower Side Shear:

Maximum downward applied load during Stage 1 was 597.8 kN which occurred at load interval 15 (Figure 7). At this loading, the average downward movement of the O-cell™ base was 122.6 mm. The side shear capacity of the 1.8 m pile section below the O-cell™ is calculated to be 86.6 kN assuming a unit side shear value of 25.5 kPa and a nominal pile diameter of 600 mm. The maximum applied load to end bearing is then 511.2 kN and the unit end bearing at the base of the pile is calculated to be 1808.0 kPa at the above noted displacement.

Upper Side Shear:

Maximum upward applied *net load* was 541.0 kN which occurred at load interval 15 (Figure 7). At this loading, the upward movement of the O-cell™ top was 22.2 mm. Assuming a nominal pile diameter of 600 mm, the average unit side shear capacity of the 10.80 m pile section above the O-cell™ is calculated to be 25.5 kPa.

Equivalent Top Load:

Figure 8 presents the equivalent top-loaded load-settlement curves. The curve was generated by using the measured pile head movement and downward base of O-cell™ data. The curve can only be directly measured to a settlement of 22.9 mm. After 22.9 mm of movement, it is assumed that is the upper shear section of the pile carries no additional load and the equivalent top load settlement curve is calculated using only additional capacity from below the O-cell™ as measured. Note that, as explained previously, the equivalent top load curve applies to incremental loading durations of eight minutes. Creep effects will reduce the ultimate resistance of both components and increase pile top movement for a given loading over longer times. However, our experience suggests that such corrections are small and perhaps negligible for top loadings below the creep limit indicated in Figure 8.

COMPARISON TO CALCULATED CAPACITY

Bored pile compressive capacity is often analyzed for design purposes on the basis of published analytical and empirical methods and engineering judgment. A widely used method (as presented at the workshop) applicable to the Styx Mill site is presented here for comparison with the O-cell test results.

Compressive capacity from soil properties:

This approach involves estimating side and end-bearing resistances from soil shear strength properties. Soil strength properties are estimated from in-situ and laboratory test results. For side resistance, a cylindrical shear model is assumed as follows:

$$Q_s = \Sigma\, \pi B\, (K \tan\delta)\, (1/2\, \gamma z^2) \tag{1}$$

where: Q_s = total side resistance, B = pile diameter, K = coefficient of lateral earth pressure, δ = interface friction angle, γ = soil effective unit weight, and z = depth interval over which side resistance is calculated. The symbol Σ indicates summation of side resistances over a finite number of depth intervals. The soil profile is divided into intervals to account for changes in any of the variables (B, K, δ, γ) with depth. Values of each parameter are assumed to be constant over each depth interval. Physically, the three terms in Eq 1 represent the following characteristics: πB = pile perimeter, K tanδ = ratio of unit side resistance to vertical effective stress, and $\frac{1}{2}\gamma z^2$ = area beneath the curve of vertical effective stress versus depth. Values for the test pile are summarized below.

Soil Type	Depth Interval	Friction Angle ϕ (degrees)	Unit Wt. (kN/m^3)	K = (1 - sinϕ)
sandy silt	0-8.5 m	33	9.8	0.45
sand/gravel	8.5-13.6 m	40	9.8	0.36

The groundwater table is assumed to be at or close to the ground surface, giving a buoyant or submerged unit weight of approximately 9.8 kN/m^3. For the depth interval 0 to 8.5 m:

$$Q_{s,1} = \pi(0.625)\ (0.45 \tan 33)\ (1/2 \times 9.8 \times 8.5^2) = 203.1 \text{ kN} \tag{2}$$

For the depth interval 8.5 – 13.6 m:

$$Q_{s,2} = \pi(0.6)\ (0.36 \tan 40)\ [1/2 \times 9.8 \times (13.6^2 - 8.5^2)] = 327.6 \text{ kN} \tag{3}$$

Total side resistance:

$$Q_s = \Sigma\, Q_{s,1} + Q_{s,2} = 195.0 + 314.5 = \underline{\mathbf{530.7\ kN}} \tag{4}$$

End bearing or tip resistance is based on bearing capacity theory and is given by the following:

$$Q_{tip} = A_{tip}\, q_{ult} \tag{5}$$

where: Q_{tip} = total tip resistance and q_{tip} = unit ultimate tip resistance. O'Neill and Reese (1999) suggest an empirical relationship between unit tip resistance and Standard Penetration Test N-value, as follows:

$$q_{tip}\ (\mathrm{kN/m^2}) = 57.5\ N_{SPT} \tag{6}$$

In this expression q_{tip} corresponds to the unit tip resistance at a downward displacement corresponding to 5 percent of the pile diameter. For the test pile:

$$Q_{tip} = \pi/4\ (0.6)^2\ [57.5 \times 18] = 292.6\ \mathrm{kN} \ @\ 30\ \mathrm{mm\ displacement} \tag{7}$$

Combining side and tip resistances:

$$Q_{compression} = Q_s + Q_{tip} = 530.7 + 292.6 = \underline{\mathbf{823.3\ kN\ @\ \Delta = 30\ mm}} \tag{8}$$

Comparison of this estimate with the equivalent top load-settlement curve derived from the O-cell test (870 kN @ 30 mm settlement) suggests that the compressive capacity calculated using this approach is a conservative but reasonably accurate estimate for design purposes. Note that the submerged unit weight was used in the calculations above (since this would probably be assumed for design purposes) while the actual water table during construction and testing was approximately 2.5 m below the ground surface.

SUMMARY AND CONCLUSIONS

A 600 mm diameter bored pile was successfully installed to a depth of 13.6 m through variable weak silty sands, sands, silts, and peats into sandy gravels, using conventional drilling equipment and an oscillated steel casing.

A pile load test was successfully made using the Osterberg Cell (O-cell™) for the first time in New Zealand. O-cell load tests are made using a hydraulic cell installed at depth within a deep foundation eliminating the need for costly reaction frames and anchors. Side-resistance and end bearing are measured independently and there is virtually no limit to the size of foundation that can be tested..

Results of the load test confirmed that good tip bearing, in excess of calculated bearing capacity (1800 KPa in this case), can be achieved for bored piles constructed into gravels below the water table using the casing method. Good construction procedure is crucial to successful outcomes. Key steps in the procedure include: a) maintaining full water head in the casing above the water table during excavation, b) advancing the casing ahead of the augur, c) "resting the hole" by placing a bucket augur at the bottom of the hole overnight to collect soil falling out of suspension, d) using high slump (200 mm) concrete, and e) maintaining generous head of concrete in the hole at all times during casing withdrawal.

Good agreement was obtained between the load test result (860 KN load @ 30 mm settlement) and capacity calculation using a standard methodology (820 KN load @ 30 mm settlement).

Difficulties with obtaining appropriate concrete mixes for bored pile construction locally were highlighted. A high slump concrete (200 mm) that resists segregation and flows readily through a tremie pipe is essential. It is desirable that the concrete mix does not rely on super-plasticisers because of the rapid stiffening of such mixes and the relatively long time required to fill a bored pile. Discussion during the workshop and field demonstration suggested that the problem with local mixes results from over-washing of aggregates resulting in a lack of fines. The issue of special mix designs for bored piles warrants further investigation.

ACKNOWLEDGEMENTS

This load test was performed on a drilled pile constructed as a demonstration as part of a professional short course entitled "Bored Piles in Bad Ground" held 8 and 9 May 2002. The short course was co-sponsored by the University of Canterbury, Department of Civil Engineering and Centre for Continuing Education, the ADSC: The International Association for Foundation Drilling, Transit New Zealand, and the New Zealand Geotechnical Society.

Transit New Zealand provided the entire funding for the actual load test and made available the site for the demonstration pile.

Loadtest International Inc. supplied the Osterberg Cell, supervised installation, and performed the load test at a substantially discounted price.

Texco Drilling constructed the demonstration pile at direct cost only.

TABLE A
SUMMARY OF DIMENSIONS, ELEVATIONS & PILE PROPERTIES

Pile:
Nominal pile diameter (EL -1.00 m to -11.80 m) = 625 mm
Nominal pile diameter (EL -11.80 m to -13.50 m) = 625 mm
Nominal pile diameter (EL -13.50 m to -13.60 m) = 600 mm
Lower O-cells™: 50 = 200 mm
Length of side shear above break at base of O-cell™ = 10.80 m
Length of side shear below break at base of O-cell™ = 1.80 m
Side shear area above O-cell™ base = 21.2 m^2
Side shear area below O-cell™ base = 3.5 m^2
Pile base area = 0.3 m^2
Bouyant weight of pile above base of O-cell™ = 0.06 MN
Estimated pile stiffness, AE (EL -1.00 m to -11.80 m) = 6300 MN
Estimated pile stiffness, AE (EL -13.50 m to -13.60 m) = 5900 MN
Elevation of top of pile concrete = -1.00 m
Elevation of mud line = +0.00 m
Elevation of water table = -4.00 m
Elevation of base of O-cell™ (The break between upward and downward movement.) = -11.80 m
Elevation of pile tip = -13.60 m
Casings:
Elevation of top of temporary casing (625 mm O.D.) = +0.50 m
Elevation of bottom of temporary casing (625 mm O.D.) = -13.50 m
Compression Sections:
Elevation of top of telltale used for upper pile compression = +0.00 m
Elevation of bottom of telltale used for upper pile compression = -11.56 m
Miscellaneous:
Top plate diameter (30 mm thick) = 430 mm
Bottom plate diameter (30 mm thick) = 500 mm
ReBar size (9 No.) = M 20
Spiral size (300 mm spacing) = M 10
ReBar cage diameter = 500 mm
Unconfined compressive concrete strength = 20.0 MPa
O-cell™ telltales @ 0°, 180° with radius = 500 mm

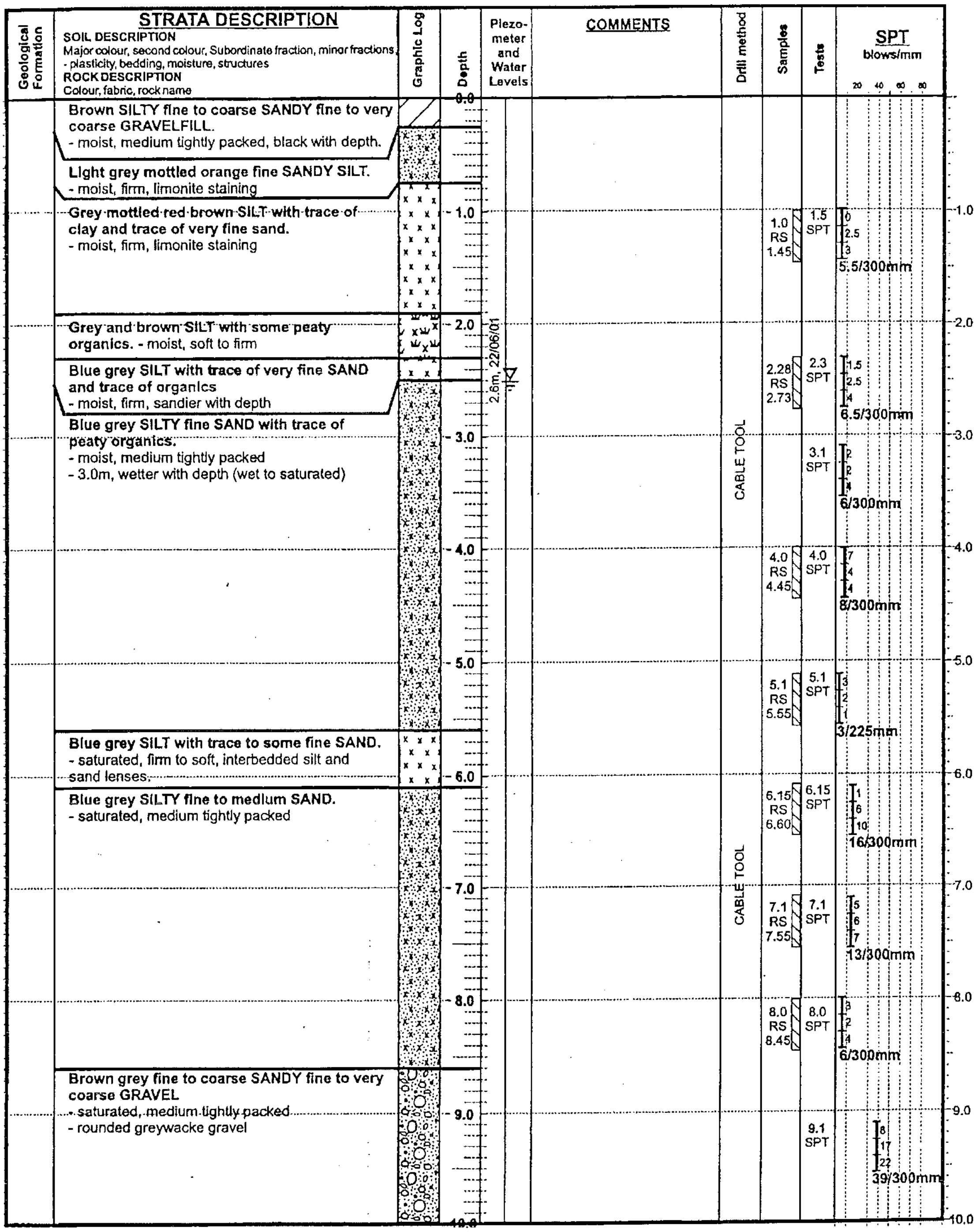
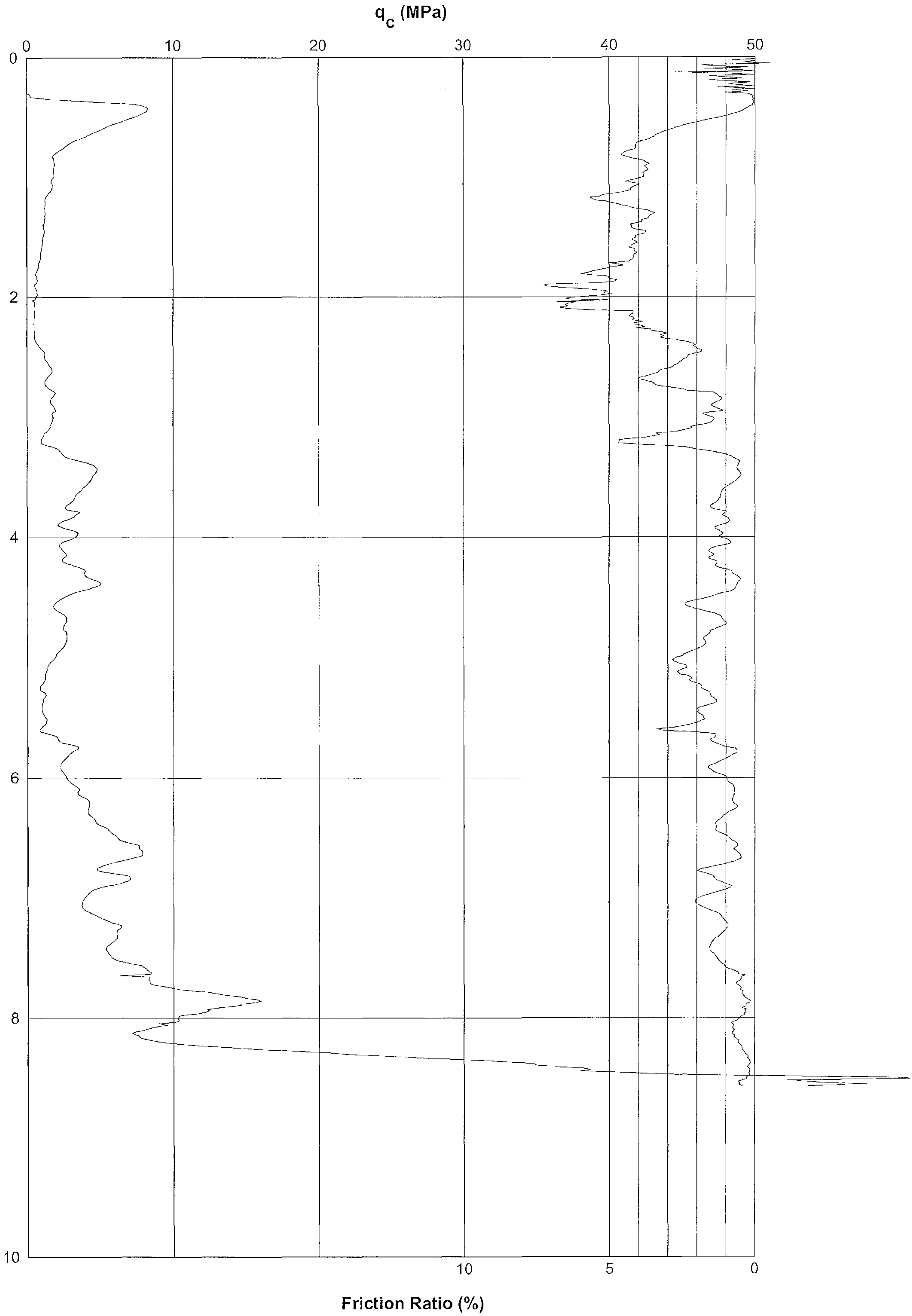


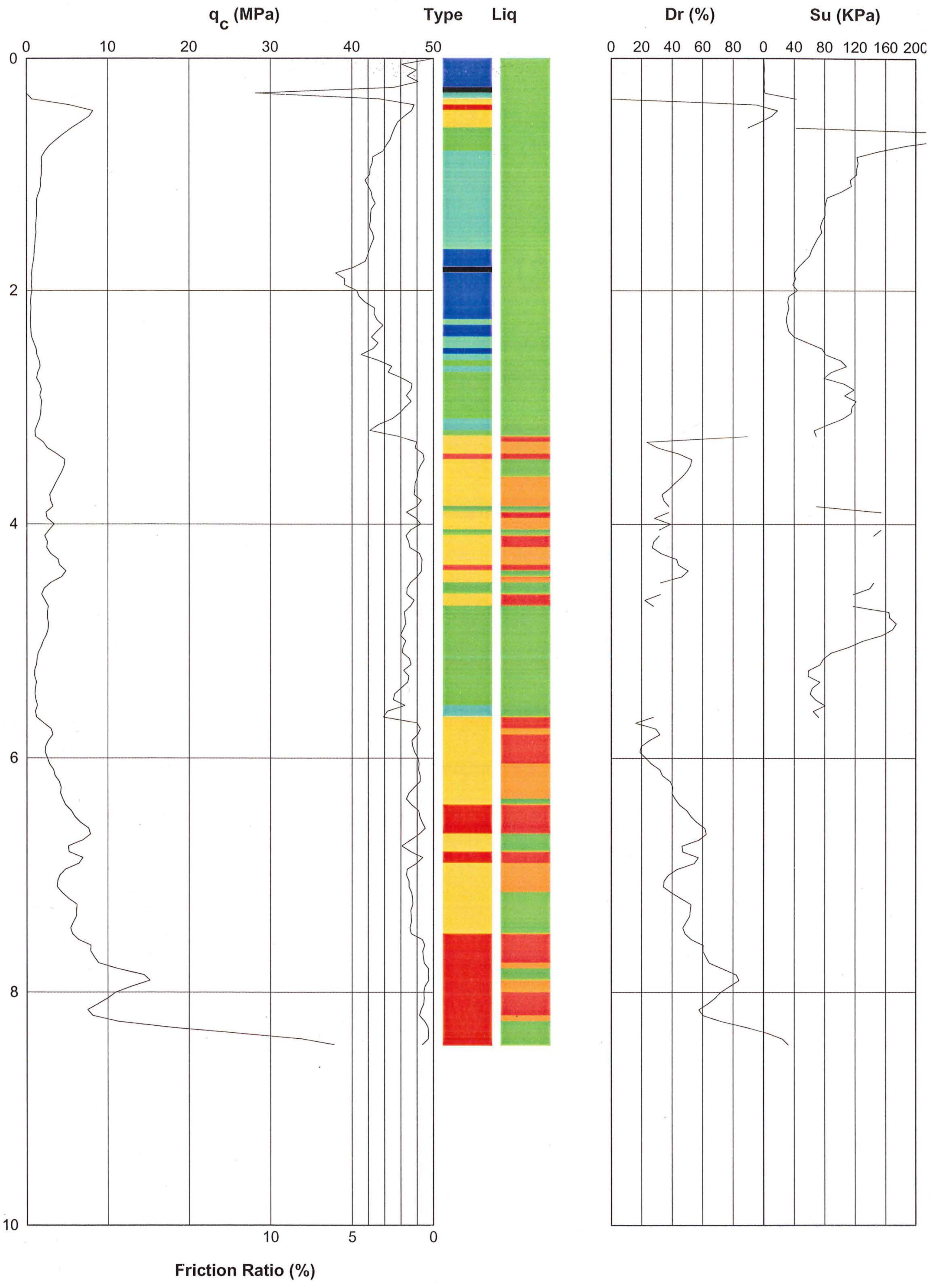
Figure 1. Borelog for Styx Mill Site.

Geological Formation	STRATA DESCRIPTION SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures ROCK DESCRIPTION Colour, fabric, rock name	Graphic Log	Depth	Piezometer and Water Levels	COMMENTS	Drill method	Samples	Tests	SPT blows/mm 20 40 60 80	
	Brown grey fine to coarse SANDY fine to coarse GRAVEL - saturated, medium tightly packed - rounded greywacke gravel - 10.0 - 11.0m, looser at depth		10.0					10.1 SPT	6 6 13 19/300mm	10.0
			11.0			CABLE TOOL		11.0 SPT	no result due to heave (0.4m)	11.0
	- 12.0m, more tightly packed with depth		12.0					12.1 SPT	3.5 7.5 11 18.5/300mm	12.0
			13.0					13.0 SPT	no result due to heave (0.3m)	13.0
	- 14.0m, progressively sandier with depth		14.0					14.0 SPT	no result due to heave	14.0
			15.0			CABLE TOOL		15.0 SPT	6 7 6 13/300mm	15.0
	- 16.5m, less gravel, sandier, some silt lenses with depth		16.0							16.0
	End of BH2 at 17m		17.0							17.0
			18.0							18.0
			19.0							19.0
			20.0							20.0

Figure 1 (Continued) Borelog for Styx Mill Site.



q_c (MPa)
0
10
20
30
40
50
0
2
4
6
8
10
10
5
0
Friction Ratio (%)



SITE INVESTIGATION LIMITED Figure 3. CPT interpretation

CPT ANALYSIS NOTES

Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance and friction ratio only. No normalisation for overburden stress is applied.

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988)

- high susceptibility
- medium susceptibility
- low susceptibility

High risk is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium risk is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

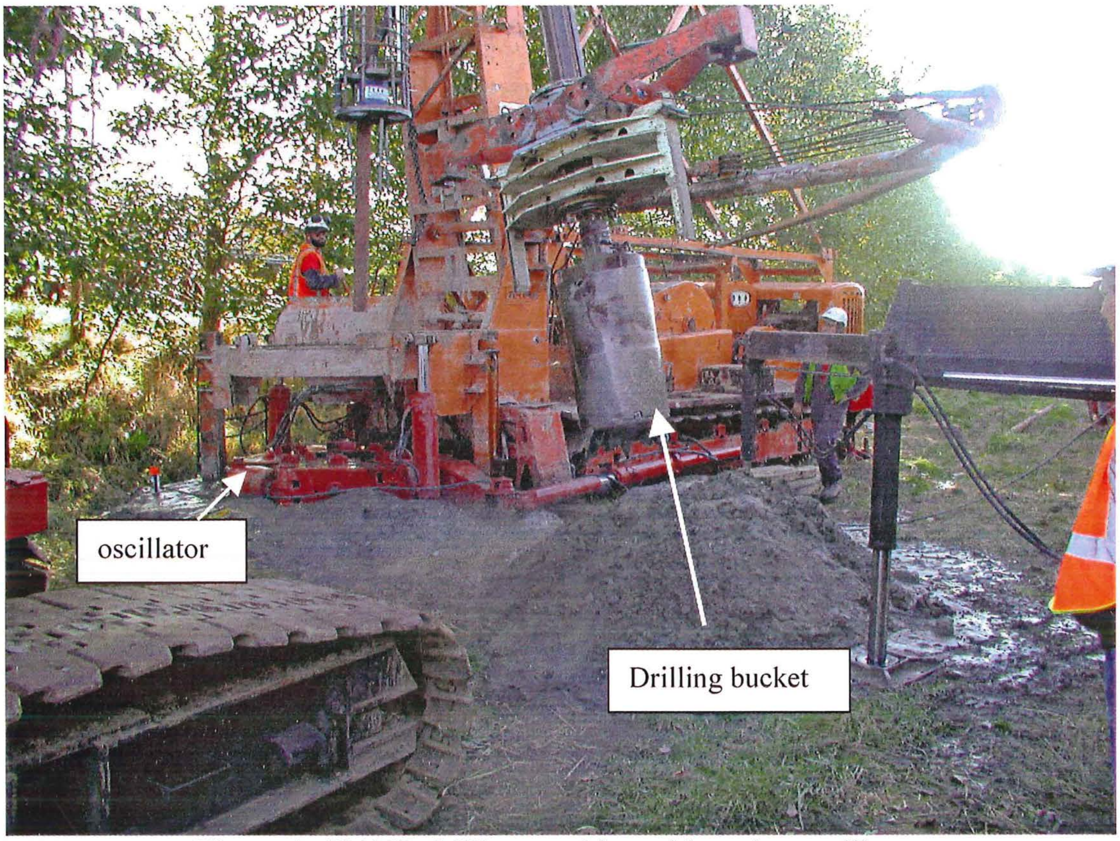


Figure 4. KATO drilling machine with casing oscillator.

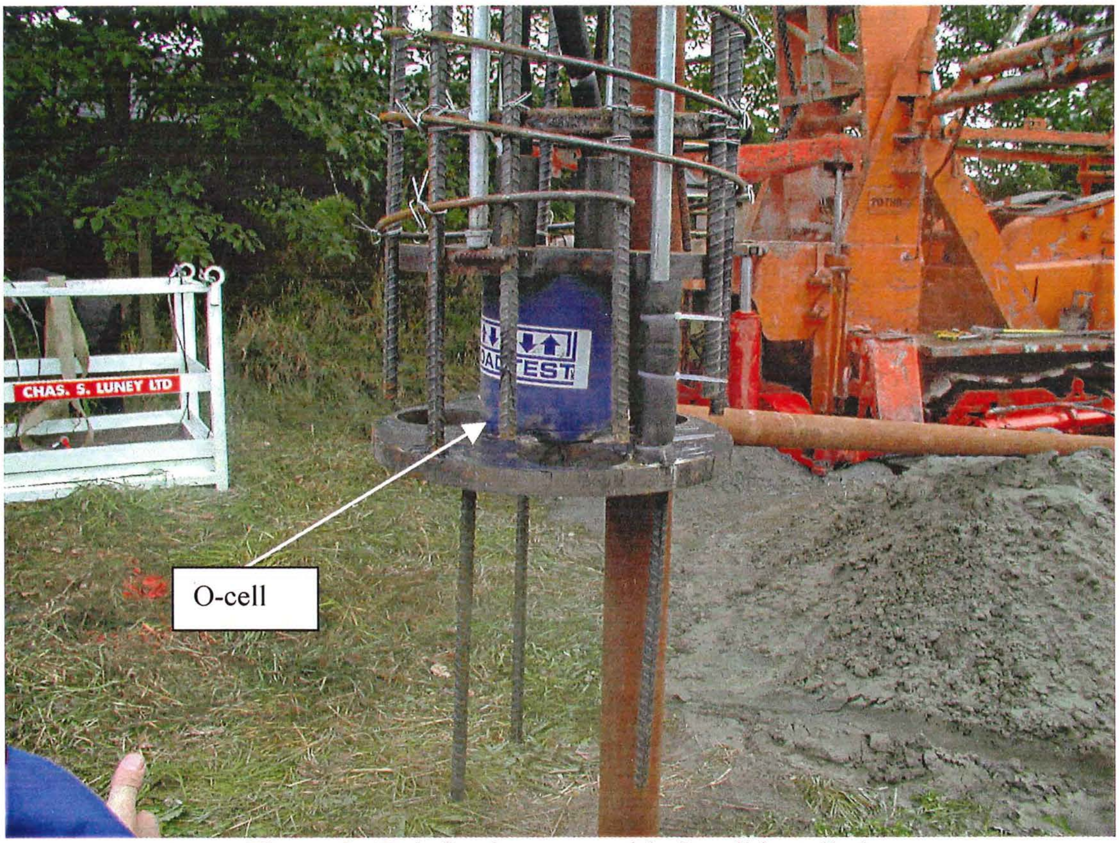


Figure 5. Reinforcing cage with O-cell installed.

Osterberg Cell Load-Movement Curves

Preliminary Test Pile 1 - Styx Mill Bridge - Christchurch, New Zealand

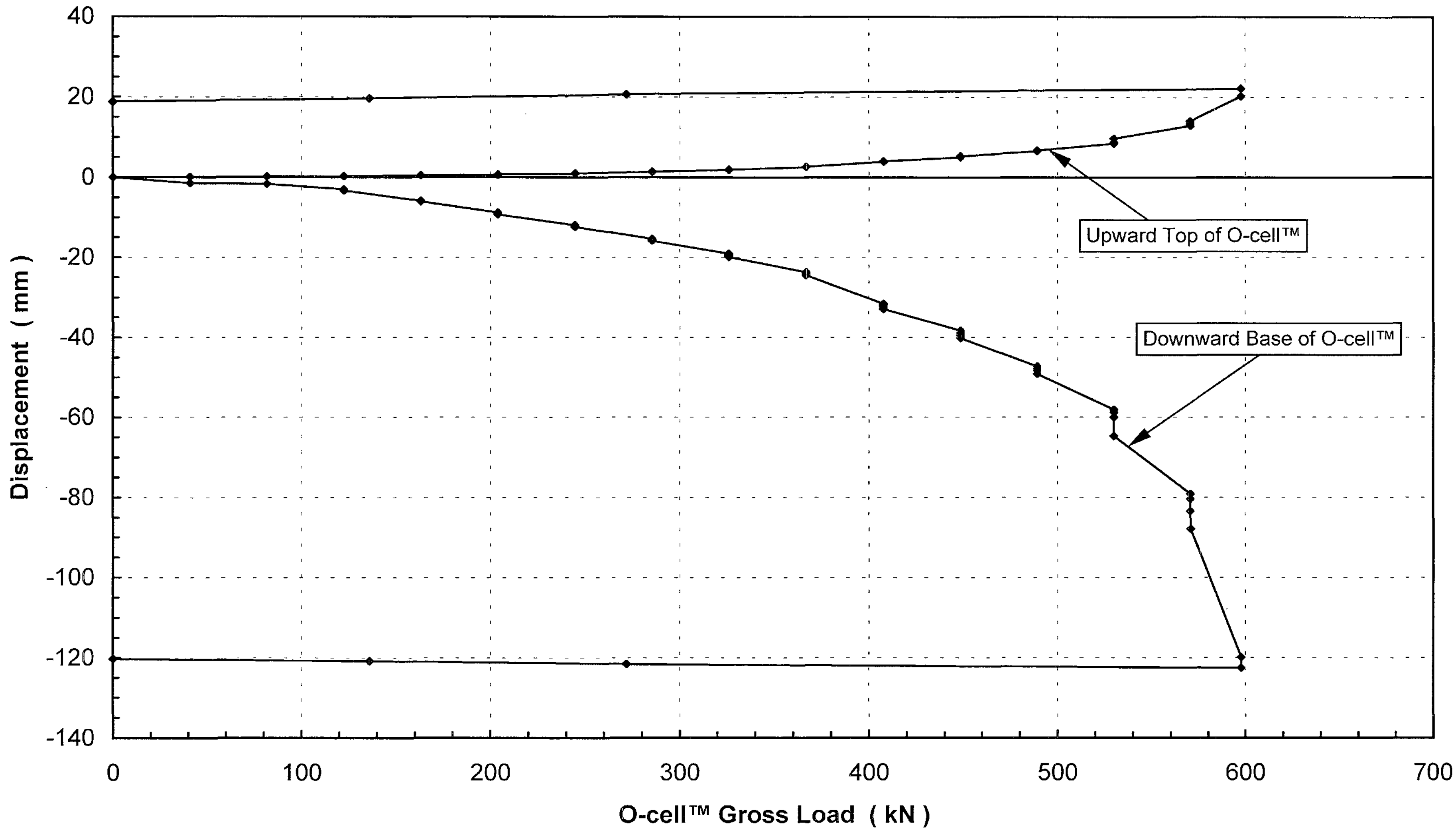


Figure 7

Equivalent Top Load-Settlement Curves

Preliminary Test Pile 1 - Styx Mill Bridge - Christchurch, New Zealand

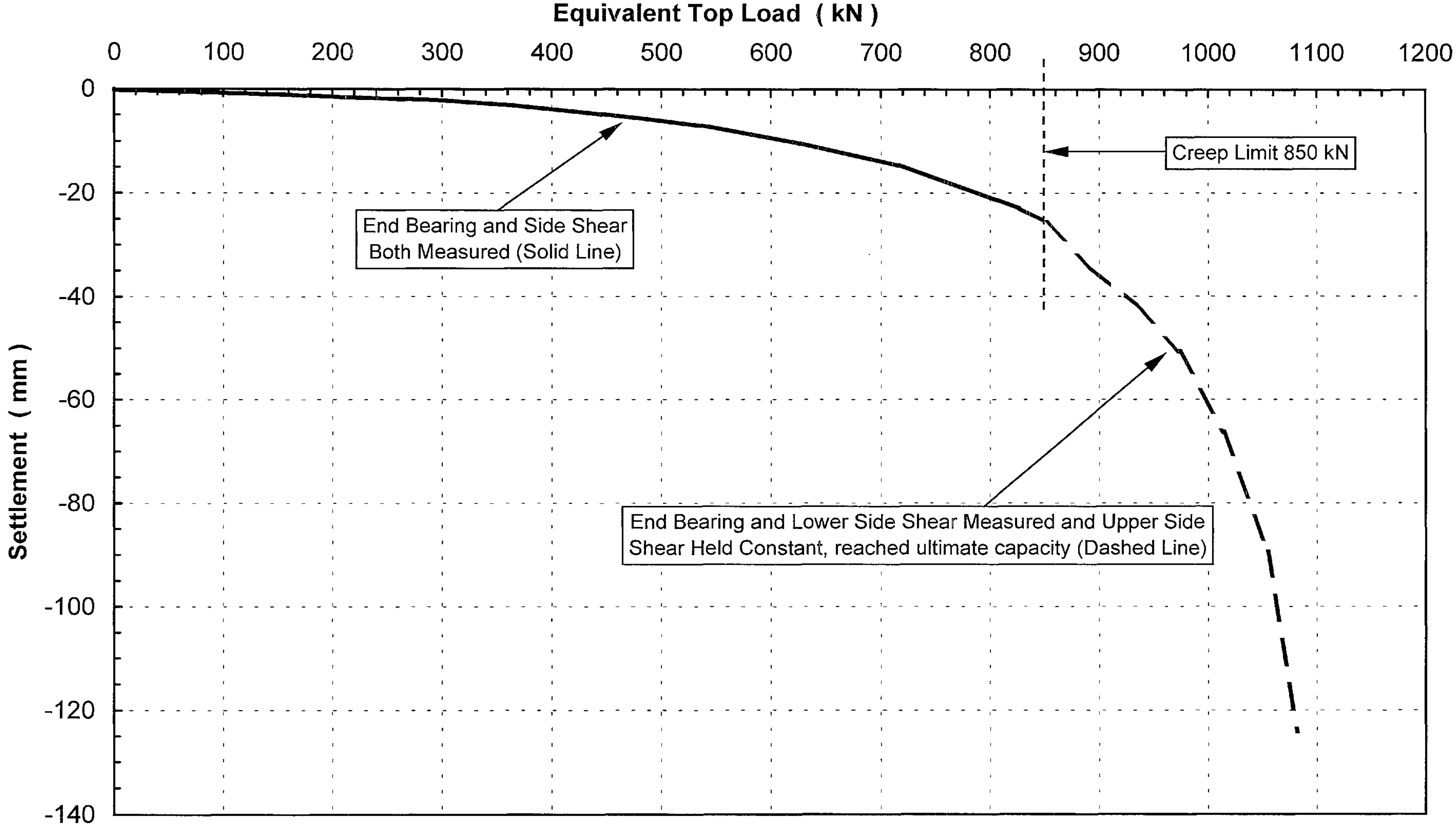
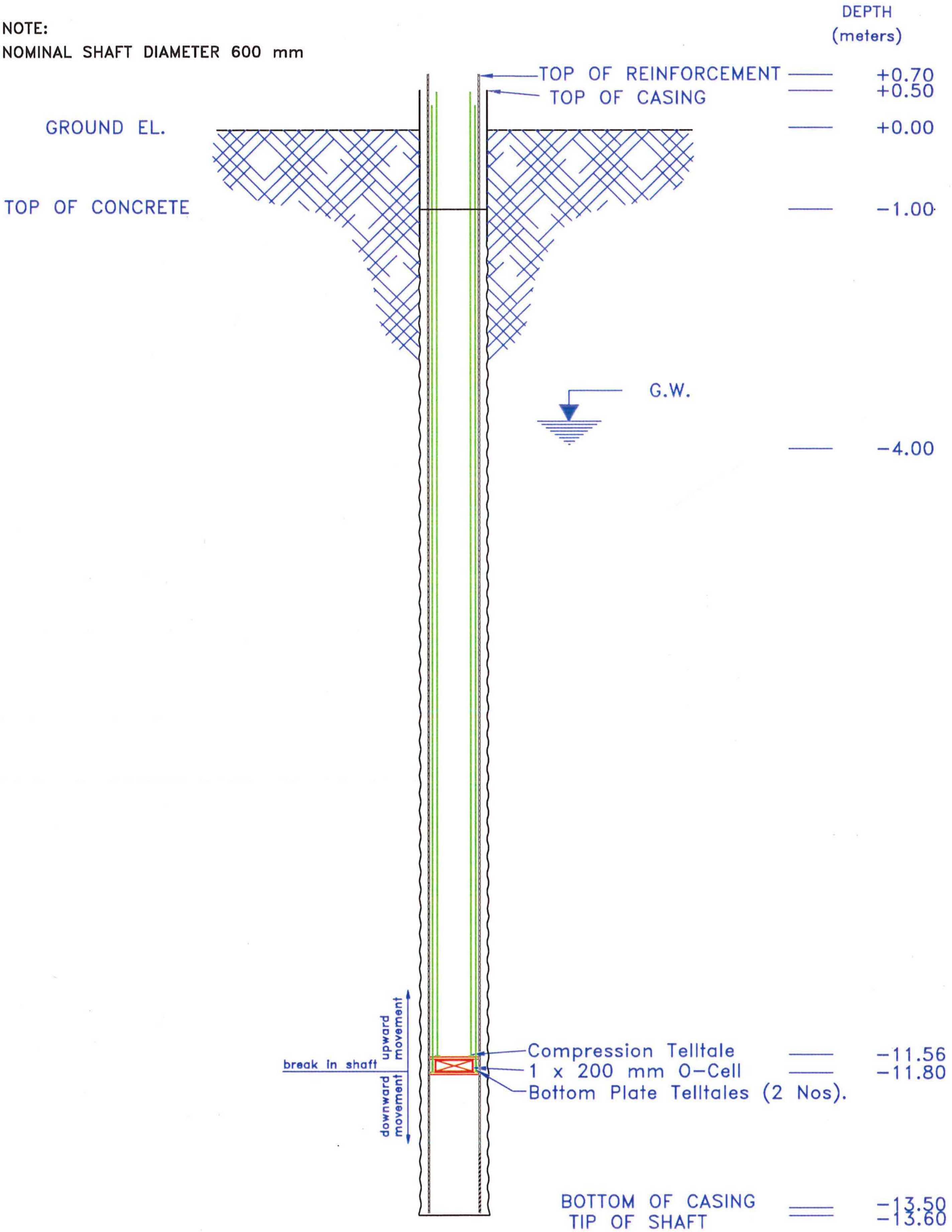


Figure 8

NOTE:
- NOMINAL SHAFT DIAMETER 600 mm



LOADTEST INC.	79 Kampong Bahru Road Singapore 169377 Phone: (+65) 6377-5665 Fax: (+65) 6377-3359	SCHEMATIC SECTION OF PRELIMINARY TEST PILE 1	LT-2203 Styx Mill Bridge New Zealand FIGURE A