

**SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
AND IN-SITU SOIL TESTING
LEAMINGTON FERRY DOCK
LEAMINGTON, ONTARIO**

MTO CONTRACT NO. 2007-3410

Geocres Number: 40J2 - 111

Report to

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1 INTRODUCTION

Thurber Engineering (Thurber) has been retained by Delcan Corporation to provide an independent determination of the subsurface stratigraphy and geotechnical parameters as they pertain to the design of a temporary protection system for strengthening of the Leamington Dock. The purpose of the supplemental investigation is to determine the in-situ strength and stiffness parameters of the various soils adjacent to an existing sheet pile wall that comprises the current ferry berth and between concrete caissons recently installed as part of a proposed toe pin strengthening system.

At the time of the investigation, a series of 914 mm diameter caissons spaced 2.5 m apart and measuring approximately 12 m in length have been constructed. At the time of the current site investigation, the 1.0 m deep, horizontal concrete cap beam connecting the caissons had not yet been constructed.

Geotechnical investigations had previously been carried out as part of the preliminary and detailed design phases for design and construction of the dock strengthening works. Foundation Reports by Golder Associates Ltd. (2004), Peto MacCallum Ltd. (2007) and CT Soil & Materials Testing Inc. (2008) have been provided to Thurber prior to commencing the site investigation.

The previous investigations by Golder (2004) and Peto (2006) provided a range of geotechnical parameters (friction angle, unit weight, cohesion, shear strength and coefficient of subgrade reaction) based on conventional field and selected laboratory testing. The subsequent investigation carried out by CT Soils (2008) involved the use of a flat plate dilatometer in conjunction with the SPT to further assess the in-situ strength and stiffness of the various layers of soil.

Thurber's scope of work involved drilling of three boreholes to approximately 15 m in depth. Two of the boreholes were located in the lake and the third was advanced through the existing wharf. At two of the locations Self Boring Pressuremeter (SBPM) and static seismic Cone Penetration Tests (CPTu) were conducted. One location only involved the advancement of the a CPT(u) test in addition to a sampled borehole. The two SBPM boreholes were advanced near two of the boreholes carried out during previous investigations in order to obtain an independent

measurement of the insitu-strength and stiffness parameters for the various soil layers. One of the boreholes drilled in the lake was located close to Golder BH 102 (2004) and the second borehole drilled through the wharf was located near CT Soils BH 2M. The final borehole was drilled in the lake near CT Soils BH 3S. The current and previous borehole locations are shown on the Borehole Locations and Soil Strata drawing in Appendix C.

This report presents the results of the geotechnical investigation and in-situ testing carried out by Thurber.

2 INVESTIGATION PROCEDURES

2.1 General

The site work was carried out between April 13 and 21, 2009 and consisted of the drilling and sampling of 3 boreholes, 3 static cone penetration boreholes and 2 self-boring and pre-bored (high pressure) pressuremeter holes.

The inland boreholes were advanced a minimum of 2.1 m away from a series of boreholes drilled previously by Golder Associates Ltd. (2004) and by C.T. Soils & Materials Testing Inc. (2008). The boreholes drilled east of the wharf (within Lake Erie) were advanced at a location equidistant between the previously installed caissons. The caisson positions were determined based on caisson number markings located on the wharf concrete wall cap. All in-water boreholes were advanced from slightly above the wharf surface, through a temporary cantilevered platform constructed at each borehole location (see Figure 1 in Appendix D).

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing attached in Appendix C of this report. All of the borehole locations were surveyed using a differential GPS (dGPS) unit and corrected to NAD83 UTM (Zone 17) coordinates. The elevations of the boreholes were estimated based on physical measurements taken in the field relative to the lake level and hourly lake elevations measured approximately 16 km west at the Kingsville Ferry dock.

2.2 Field Investigation

The boreholes were advanced using a truck mounted CME-75 drill rig and either a combination of hollow stem augers with NW casing or a combination of HW and NW casing and wash boring methods. Samples of the overburden material were recovered at regular intervals of depth using a 50 mm outside diameter split spoon sampler in conjunction with the Standard Penetration Test (SPT).

In cohesive materials, the undrained shear strength was assessed using a standard MTO 'B' vane, CPT(u) and the SBPM mentioned above.

The field work was supervised on a full-time basis by a member of our engineering staff, who located the boreholes in the field and directed the drilling, sampling and in-situ testing operations.

All boreholes were logged in the field. Soil samples were identified, placed in labelled, air tight containers and transported back to our laboratory for further examination and testing.

Results of the field drilling, sampling and geotechnical lab testing are presented on the Record of Borehole sheets in Appendix A and on the graphs in Appendix B.

2.3 Borehole Locations

Within the wharf area (Thurber BH 09-01 N, S and C); hollow stem augers and HW casing were advanced a total depth of 6.1 m (El. 169.9 m) through the granular fill to the surface of the native soils. The sampled borehole (09-01N) and the CPT(u) borehole (09-01S) were positioned on either side of the SBPM borehole (09-01C). Borehole 09-01N was drilled and sampled to a total depth of 13.5 m below the surface of the wharf (El. 162.5 m) and was terminated within a hard glacial till layer. The CPT(u) borehole (09-01S) was advanced to a depth of 12.0 m below the surface of the wharf (El. 164.0 m) and the SBPM borehole (09-01C) was advanced to a depth of 15.3 m (El. 160.7 m). Following completion of the three boreholes advanced through the wharf, large voids were observed below the surface of the asphalt pavement within the granular fill. The voids were backfilled with uncompacted granular material provided by the contractor. It is understood that as part of the wharf reconstruction, the asphalt was to be removed by the contractor (at a later date) and any voids would be backfilled with a low strength unshrinkable fill. Throughout the drilling activities, the surface of the granular material was observed to settle and was replaced with additional granular material from time to time.

In the lake bed area near Golder BH 102 (Thurber BH 09-02 N, C and S), only two borehole locations were accessible because of the existing concrete cap/bumper system. Therefore the CPT(u) borehole and the sampled borehole were positioned in the same general location. In order to obtain the best possible in-situ data, the CPT(u) (09-02N) was first advanced to refusal or a depth of 13.1 m (El. 163.1 m) midway between caissons Nos. 23 and 24. The sampled borehole (09-02C) was then moved slightly south towards caisson No. 24 and drilled and sampled to a depth of 19.5 m (El. 156.7 m) below the top of the temporary deck. The pressuremeter borehole (09-02S) was advanced to a depth of 14.7 m (El. 161.5 m) from midway between caissons Nos. 24 and 25. In all three cases, the depth to the surface of the mudline was measured as 8.8 m from the top of the temporary deck.

In the southern part of the wharf (Thurber BH 09-03 N and S) only a sampled borehole and a CPT(u) borehole were advanced. In the case of the sampled borehole, the borehole was extended to a total depth of 18.9 m below the top of the temporary deck (El. 157.3 m) and the CPT(u) borehole was advanced to a depth of 13.1 m (El. 163.1 m). Borehole 09-03S was positioned midway between caissons Nos. 33 and 34 and Borehole 09-03N was positioned midway between caissons Nos. 32 and 33. In both cases, the HW casing was set at a depth of 9.1 m from the top of the deck to the surface of the mudline (El. 197.1 m).

The total depth and final elevations of each of the boreholes drilled at the site are summarized below in Table 2.1.

Table 2.1
Total Borehole Depth and Elevations

Borehole No.	Borehole Type	Location	Total Depth (m)	Borehole Termination Elevation (m)
09-01N	Sampled	Wharf	13.5	162.5
09-01C	SBPM	Wharf	15.3	160.7
09-01S	CPT(u)	Wharf	12.0	164.0
09-02N	CPT(u)	Lake	13.1	163.1
09-02C	Sampled	Lake	19.5	156.7
09-02S	SBPM	Lake	14.7	161.5
09-03N	CPT(u)	Lake	13.1	163.1
09-03S	Sampled	Lake	18.9	157.3

3 IN-SITU TEST PROCEDURES

3.1 Standard Sampling and Shear Vane

Samples of the fill and native materials were recovered from SPT tests at close intervals throughout the entire depth of each borehole in accordance with ASTM D1586-08. Each split spoon sample was driven 0.6 m and the recovered material was visually examined prior to continuation of the borehole. When cohesive materials were encountered, shear vane tests were carried out to measure the in-situ undrained shear strength and the sensitivity of the soil. The shear vane was advanced 0.3 m into the undisturbed material and the soil sheared at a constant rate in accordance with ASTM D2573-08. An image of the shear vane used during the investigation is shown in Figure 3 in Appendix D at the end of the report.

Once the initial (in-situ) shear strength was obtained, the remoulded strength and sensitivity of the cohesive material were measured. The shear vane was rotated several times to ensure complete failure of the soil surrounding the blade and allowed to 'rest' for one minute prior to measuring the remoulded shear strength.

3.2 CPT(u)

The cone penetrometer was pushed continuously in 0.5 m intervals by the drill rig. During each interval, the cone depth, tip resistance (Q_c), friction, inclination and pore pressure (P_w) was recorded every 50 mm and uploaded to a data acquisition system. An image of the cone penetrometer used during the investigation is shown in Figure 4 in Appendix D at the end of the report. At each interval, shear wave tests were conducted and the pore pressure dissipation was monitored at each interval in Borehole 09-01S and every second interval (1 m) in Boreholes 09-02N and 09-03N.

Shear wave tests were carried out on land using a steel beam positioned beneath the rear outriggers of the drill rig and the shear wave was generated by the impact of a sledge hammer on each end of the beam. An image of the inland beam arrangement is shown in Figure 5 in Appendix D. Each test required impact on both sides of the beam to create ‘mirrored’ shear waves monitored at the downhole geophone located in the instrument. The shear waves were generated in the water by lowering the steel beam to the floor of the lake (mudline) and striking a rod and anvil at the deck surface. The rod was attached to the beam by a swivel that allowed the rod and anvil to move to either side of the downhole instrument. Images of the in-water beam and sounding system are shown on Figures 6 to 8 in Appendix D.

The CPT(u) was advanced to refusal in each of the three locations which was typically between 12 to 13 m below the wharf/temporary deck surface. It should be noted that refusal in 09-01S may have occurred on a cobble based on the overall termination depth of the borehole when compared to the total depth of the sampled borehole (09-01N) and the pressuremeter borehole (09-01C).

3.3 Pressuremeter

The SBPM was advanced in two locations, first from within the wharf at Borehole 09-01C (near CT-2M) and from between two caissons in the lake at 09-02S near GAL 102. In each case, HW casing was set to the top of the native soil through a granular fill (wharf) or at the mudline through the water (09-02S). The SBPM was then advanced into the native materials an initial depth of 1.5 m to minimize the risk of blowout near the top of the first test interval. Once the initial test zone had been reached, the SBPM was gradually inflated using nitrogen gas and the expansion (strain) of the cavity was monitored by three arms positioned at 120° intervals from one another. During the loading of the sidewalls of the borehole, the applied stresses and strains were continuously recorded by a data acquisition system. The pressure was reduced (unloaded) several times throughout each test to obtain the unload/reload moduli of the soil. The test was terminated when a strain of 10 to 12% was achieved in any of the three sensors. Following the initial test, the SBPM was advanced at 0.75 m intervals and another test was commenced. This process was continued until the testing reached the top of the hard cohesive glacial till layer.

The SBPM was then extracted from the borehole and the HW casings advanced to the surface of the cohesive till layer. Once the casing was set, the borehole was advanced an additional 2.1 m using an NQ core barrel through the dense/hard soils. The High Pressure Pressuremeter (HPPM) was then inserted to the base of the test pocket and two pressuremeter tests were carried out in the 2.1 m long, pre-bored section. The first test was conducted at the base of the test pocket and the second approximately 1.0 m above the first HPPM test location.

Following completion of the pressuremeter testing, the SBPM was run through a test cycle to calibrate the instrument and the membrane in atmospheric pressure conditions.

Images of the SBPM and the HPPM are provided at the end of this report in Appendix D as Figure 13. Additional details of the pressuremeter and CPT tests are presented in the In Situ Engineering report attached as Appendix E.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg Limits testing. The results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A and to the Borehole Locations and Soil Strata Drawing in Appendix D. An overall description of the stratigraphy based on the conditions encountered in the boreholes is given in the following paragraphs. However, the factual data presented in the borehole logs takes precedence over this general description for interpretation of the site conditions. It should be recognized that soil conditions may vary between and beyond borehole locations.

In general the subsurface materials within the wharf consist of a layer of asphalt cement underlain by a granular fill which overlies a layer of loose silty sand and a layer of firm to stiff silty clay which is underlain by a hard clayey silt till. The sediments encountered in the lake consist of a surficial layer of silty sand to sandy silt underlain by a firm to stiff silty clay, which overlies a clayey silt till, a layer of sand and gravel and a silt to silt till.

5.2 Asphalt Cement

A layer of asphaltic cement was encountered from the ground surface in Boreholes 09-01N, 09-01C and 09-01S. The thickness of the asphalt was measured to be approximately 200mm in each of the borehole locations. Frequent cracks were observed throughout the surface of the pavement structure.

5.3 Granular Fill

Immediately beneath the layer of asphalt within the wharf area, a layer of granular fill was encountered to a depth of approximately 6.1 m below the ground surface (El. 169.9 m). The fill was observed to vary significantly in composition ranging from a silt and sand to cobbles up to 200 mm in diameter as observed in several core samples. The granular fill was brown and moist to wet below the surface of the lake. A layer of white to light grey silty sand to sandy silt layer was encountered at an approximate depth of 3 m below the ground surface.

The SPT N-values observed in the fill layer ranged from 5 to 15 blows for 0.3 m of penetration, corresponding with a loose to compact relative density.

Assumed voids were frequently encountered within the granular fill. The assumption of the presence of voids was based on the rate of advancement of the drill string and split spoon sampler during drilling operations. During advancement of the augers through the fill, the auger head would frequently separate from the drill stem distances of 25 to 75mm. Assumed voids were observed during the SPT samples by rapid settlement of the rods without additional energy, followed by slight resistance at the base of the suspected void.

Grain size analyses conducted on one (1) sample of the granular fill is presented on the individual Record of Borehole Log Sheets (BH09-01N) attached in Appendix A and on Figure 1 in Appendix B.

The result of the laboratory gradation test is summarized as follows:

Gravel %	60
Sand %	38
Silt and Clay %	2

Moisture contents measured in the granular fill ranged from 5 to 19%.

5.4 Silty Sand to Sandy Silt

Beneath the granular fill and at the mudline (current dredge line), a layer of fine grained silty sand to sandy silt with trace of gravel and clay was encountered. The cohesionless sediment is grey, and is composed of frequently alternating layers of sand and silt. The laminations were observed to range from 5 to 25 mm in thickness, generally increasing in thickness with depth. The silty sand was saturated (fully submerged) and possessed a slight sulphuric odour in the samples recovered from within the lake bed.

The silty sand to sandy silt was encountered at a depth of 6.1 m (El. 169.9 m) in Borehole 09-01N, and at depths of 8.8 and 7.9 m below the platform level (El. 167.4 and 168.3 m) in Boreholes 09-02C and 09-03S respectively. The cohesionless layer extended to depths ranging from 9.5 m (El. 166.5 m) in Borehole 09-01N to 11.6 and 10.7 m (El. 164.6 and 165.6 m) in Boreholes 09-02C and 09-03S respectively.

The SPT N-values recorded in the cohesionless sediment layers ranged from the weight of the hammer (0 blows) to 6 blows for 0.3 m of penetration indicating a very loose to loose relative density.

Grain size analyses conducted on four (4) samples of the silty sand to sandy silt are presented on the individual Record of Borehole Log Sheets attached in Appendix A and on Figure 2 in Appendix B.

The result of the laboratory gradation test is summarized as follows:

Gravel %	0 to 7
Sand %	38 to 70
Silt %	20 to 51
Clay %	5 to 18

Moisture contents measured in the samples of the silty sand to sandy silt recovered from within the wharf area and from below the mudline ranged widely from 12 to 47%, though were typically around 22 to 27%.

5.5 Silty Clay

A layer of silty clay was encountered below the above cohesionless sediment in all of the boreholes. The material was observed to be brown and contain trace quantities of fine sand and gravel. The surface of the cohesive material was contacted at a depth of 9.5 m (El. 166.5 m) in Borehole 09-01N and at depths of 11.6 and 10.7 m below the surface of the drill platform (El. 164.6 and 165.6 m) in Boreholes 09-02C and 09-03S respectively. The base of the silty clay was encountered at a depth of 12.7 m (El. 163.3 m) in Borehole 09-01N and at 13.9 and 13.5 m below the top of the deck (El. 162.3 and 162.7 m) in Boreholes 09-02C and 09-03S respectively.

SPT N-values in the silty clay deposit ranged from the weight of the hammer (0 blows) to 6 blows for 0.3 m of penetration, indicating a very soft to firm consistency. In-situ shear vane tests in the silty clay layer ranged from 60 to 115 kPa, though were typically between 72 and 75 kPa indicating a stiff to very stiff consistency. The sensitivity of the clay was found to range from 0.5 to 3.2 indicating a low sensitivity material.

Grain size analyses tests conducted on three (3) samples of the silty clay are presented on the individual Record of Borehole Log Sheets attached in Appendix A and on Figure 3 in Appendix B. Atterberg limits testing carried out on two (2) samples of the silty clay are presented on Figure 7 in Appendix B.

The result of the laboratory gradation test is summarized as follows:

Gravel %	0 to 1
Sand %	16 to 17
Silt %	31 to 32
Clay %	52 to 53
Liquid Limit %	39 to 42
Plastic Limit %	19

The Atterberg Limits indicate that the silty clay corresponds with a CI classification (intermediate plasticity) material based on the Unified Soils Classification System (USCS).

Moisture contents of the recovered samples of the silty clay were found to range from 22 to 31%.

5.6 Clayey Silt Till

Below the layer of silty clay, a layer of clayey silt till was encountered. The till was observed to be grey, moist and contain trace to some sand and trace of gravel. Though not encountered in the sampled boreholes, several cobbles ranging in thickness from approximately 200 to 300 mm were encountered within the cohesive till layer in the two pressuremeter boreholes. A white cemented sand to fractured siltstone was encountered in Borehole 09-02C within the till layer, though was easily penetrated by the split spoon and casing.

The till layer was contacted at depths of 12.7 m (El. 163.3 m) below the ground surface in 09-01N and 13.9 and 13.5 m below the top of the temporary deck (El. 162.3 and 162.7 m) in Boreholes 09-02C and 09-03S respectively. The thickness of the till layer was variable and ranged from 2.4 and 1.5 m (El. 159.9 and 161.3 m) respectively in Boreholes 09-02C and 09-03S. Borehole 09-01N was terminated within the cohesive till layer at a depth of 13.5 m below the surface of the wharf (El. 162.5 m).

SPT N-values of the clayey silt till were found to range from 5 to 81 blows for 0.3 m of penetration indicating a firm to hard consistency. One N-value within the till was measured as 50 blows for 0.075 m of penetration.

Grain size analyses conducted on two (2) samples of the clayey silt till are presented on the individual Record of Borehole Log Sheets attached in Appendix A and on Figure 4 in Appendix B. Atterberg limits testing carried out on one (1) sample of the clayey silt till is presented on Figure 7 in Appendix B.

The result of the laboratory gradation test is summarized as follows:

Gravel %	4
Sand %	33 to 36
Silt %	38 to 40
Clay %	20 to 26
Liquid Limit %	22
Plastic Limit %	13

The Atterberg Limits indicate that the clayey silt till corresponds with a CL classification (low plasticity) material based on the Unified Soils Classification System (USCS).

The natural moisture contents measured in the clayey silt till were found to range from 9 to 15%.

5.7 Sand and Gravel

The cohesive till layer was underlain by a sand and gravel layer with traces of siltstone fragments. The cohesionless layer was wet and dark brown to black in colour. The granular layer was initially contacted at depths of 16.3 and 15 m (El. 159.9 to 161.3 m) in Boreholes 09-02C and 09-03S respectively. The base of the sand and gravel layer was encountered at depths of 17.5 and

17.7 m below the top of the temporary platform (El. 158.7 and 158.6 m) in Boreholes 09-02C and 09-03S respectively.

SPT N-values recorded in the granular layer were observed to range from 13 to 33 blows for 0.3 m of penetration corresponding with a compact to dense relative density.

Grain size analyses conducted on two (2) samples of the sand and gravel are presented on the individual Record of Borehole Log Sheets attached in Appendix A and on Figure 5 in Appendix B.

The result of the laboratory gradation test is summarized as follows:

Gravel %	36 to 54
Sand %	43 to 60
Silt and Clay %	3 to 4

Natural moisture contents measured in the recovered samples of the cohesionless material range from 9 to 14%.

5.8 Silt

Below the sand and gravel, a layer of silt to fine sandy silt was encountered. The material has traces of clay and gravel at depth and is grey, wet and dilatant upon disturbance. The surface of the silt was encountered at depths of 17.5 and 17.7 m (El. 158.7 and 158.6 m) and was found to extend to depths of 18.9 and 18.8 m below the top of the deck (El. 157.3 and 157.4 m) in Boreholes 09-02C and 09-03S respectively.

SPT N-values recorded in the silt layer range from 16 to 27 indicating a compact relative density.

Grain size analyses tests conducted on one (1) sample of the silt is presented on the individual Record of Borehole Log Sheets (09-03S) attached in Appendix A and on Figure 6 in Appendix B.

The result of the laboratory gradation test is summarized as follows:

Gravel %	0
Sand %	26
Silt %	65
Clay %	9

Natural moisture contents of the silt were measured to range from 14 to 23%.

5.9 Sandy Silt Till

Boreholes 09-02N and 09-03S were terminated in a grey to dark grey sandy silt till with some clay and trace gravel. Occasional layers of a fine to medium grained sand were observed to be interbedded within the cohesionless till layer. The surface of the cohesionless till was encountered at depths of 18.9 and 18.8 m (El. 157.3 and 157.4 m) in Boreholes 09-02C and 09-

03S respectively. Both boreholes were terminated within the till layer at respective depths of 19.5 and 18.9 m below the surface of the temporary deck (El. 156.7 and 157.3 m).

A single SPT N-value of 17 blows for 0.3 m of penetration was recorded in the cohesionless till layer, indicating a compact relative density.

The natural moisture content of the till layer was found to range from 12 to 19%.

5.10 Groundwater

The groundwater level was not measured in any of the open boreholes due to the use of drill fluid throughout the drilling process. No piezometers were installed during the current investigation. The groundwater level is expected to reflect the water level measured in the lake (approximately El. 174.4 m \pm 1m). The groundwater level is also expected to fluctuate with the change in the water level of the lake. A standpipe piezometer installed by others (CT Soils, 2008) into the clayey silt till layer indicates that the groundwater level is approximately 1.8 m below the ground surface (wharf level) or at a similar elevation (El. 174.2 m) as the lake level.

6 IN-SITU TEST RESULTS

The results of the in-situ testing program carried out in two of the CPT(u) boreholes and the two pressuremeter holes are summarized at the end of this report in Table 1. The test data provided by In Situ Engineering is also included at the end of this report as Appendix E. Comparisons of the in-situ testing results, relative to the findings of the original investigation by Golder are shown in Tables 2 through 6 located at the end of this report. In all cases, a Poisson's ratio (ν) of 0.2 has been assumed (drained conditions) to convert the shear modulus to an elastic modulus.

Based on the findings of the in-situ testing carried out within the wharf and between the caissons (toe pins) the following material parameters have been inferred.

6.1 Silty Sand to Sandy Silt (El. 169.9 to 164.6 m)

The effective friction angle (ϕ') of the silty sand to sandy silt layer was assessed from the SBPM tests to range from 30 to 32°, while the CPT(u) tests indicate a range of ϕ' from between 28 to 30°.

The Modulus of Elasticity (E) based on the SBPM results range from 3,350 to 6,620 kPa in the lake and from 21,600 to 60,000 kPa beneath the wharf. The equivalent strain moduli inferred from the seismic CPT(u) test beneath the wharf was estimated to be in the order of 30,000 kPa.

It should be noted that the low modulus of elasticity of 3,350 kPa obtained from the first test within the lake bed may be due to disturbance and may not be representative of the in-situ conditions.

6.2 Silty Clay (El. 166.5 to 162.3)

Based on the interpretation of the pressuremeter test results, the angle of internal friction was measured to range from 28 to 30° within the silty clay sediment.

The undrained shear strength of the silty clay layer was measured to vary from 60 to 75 kPa in the tests carried out with the shear vane and from 79.7 to 102.8 kPa in the tests with the SBPM.

The modulus of elasticity of the silty clay was found to range from 6,500 to 6,820 kPa in the SBPM hole advanced in the lake and from 15,450 to 26,400 kPa in the borehole advanced beneath the wharf. The equivalent strain elastic moduli inferred from the seismic CPT(u) tests were respectively observed as 16,450 kPa within the lake and 16,675 kPa beneath the wharf.

6.3 Clayey Silt Till /Sand and Gravel (El. 163.3 to 158.6 m)

The in-situ testing in the clayey silt till / sand and gravel indicated an internal friction angle (ϕ') of 38° for both HPPM tests conducted within the clayey silt till / sand and gravel. The modulus of elasticity (E) was estimated to be 216,000 and 360,000 kPa in the tests carried out within the lake and beneath the wharf respectively.

7 COMPARISON OF FINDINGS

Based on the findings of the supplemental geotechnical investigation and in-situ testing, a comparison of the soil conditions observed at the site has been made. A series of tables comparing the soil conditions and parameters reported by Golder (2004) during the preliminary investigation stage have been assembled and arranged based on the observed stratigraphy and is provided at the end of this report as Tables 2 through 6.

7.1 Silty Sand to Sandy Silt

The silty sand to sandy silt was initially encountered at an elevation of 167.26 m and fully penetrated at an elevation of 165.4 m in the Borehole 102 drilled by Golder Associates within the water in 2004. In the current investigation, the cohesionless lake sediment was encountered at an elevation ranging from 167.4 to 168.3 m in the two boreholes advanced within the lake bed. The bottom of the sand layer was encountered at an elevation ranging from 164.6 to 165.6 m (Boreholes 09-02C and 09-03S respectively). In the borehole drilled through the wharf (09-01N), the silty sand to sandy silt was encountered at an elevation of 169.9 m and the base of the layer was encountered at an elevation of 166.5 m.

Comparison of the grain size analyses results reported by Golder indicate a similar fines content to those noted during the current site investigation. Golder reported a percentage of fines in the upper silty sand to sandy silt sediment ranging from 40 to 45% while Thurber has observed a percentage of fines ranging from 25 to 47%.

The SPT N-values reported by Golder for samples obtained from the cohesionless sediment ranged from 3 to 4 blows for 0.3 m of penetration, while the current investigation indicated N-values ranging from 1 to 6 blows for 0.3 m of penetration.

The internal friction angle was reported as 28° by Golder in 2004 and is considered very similar based on the current SBPM testing in the cohesionless sediment, which has been interpreted to range from 28 to 30° .

The current investigation indicated a modulus value of 3,350 to 6,620 kPa for the lake bed silty sand layer (09-02S). In the borehole advanced through the wharf (09-01C), the elastic modulus of the silty sand layer ranged from 21,600 to 60,000 kPa. No stiffness or modulus values were provided in the Golder report for the silty sand to sandy silt lake sediment hence this comparison cannot be made.

7.2 Silty Clay

The surface of the silty clay sediment was initially contacted in the borehole drilled by Golder in the water (BH 102) at an elevation of 165.4 m and the base of the layer was contacted at an elevation of 163.2 m. The current investigation indicated that for the two boreholes drilled in the water, the surface of the clay layer was encountered at an elevation ranging from 164.6 to 165.6 m. The base of the cohesive deposit was contacted at elevations of 162.3 and 162.7 m in Boreholes 09-02C and 09-03S respectively. Within the wharf area, the surface of the silty clay was contacted at an elevation of 166.5 m and the lower limit of the clay was encountered at an elevation of 163.3 m.

The clay content was not assessed by Golder during their investigation, however the Atterberg limits testing carried out on the silty clay material indicated a range of plastic and liquid limits from 16 to 23% and 33 to 37% respectively. The limits indicated during the current investigation were found to be 19% and from 39 to 42% for the plastic and liquid limits respectively which is similar to the Golder findings.

SPT N-values in the silty clay recorded by Golder indicate an N-value of 5 blows for 0.3 m of penetration. The current investigation indicates N-values ranging from the weight of hammer (0 blows) to 6 blows for 0.3 m of penetration.

The effective internal friction angle of the silty clay was reported as 29° by Golder in 2004 and is considered very similar to the results of the current SBPM and CPT(u) testing in the silty clay. The respective friction angles have been interpreted from the SBPM and the CPT(u) to range from 28 to 29° and from 28 to 30° .

The undrained shear strength (S_u) measured by Golder using a shear vane ranges from 54 to 115 kPa (average of 85 kPa) in 2004 which is similar to the recent in-situ findings of 60 to 75 kPa measured with a shear vane and from 78.2 to 102.8 kPa using the SBPM.

Based on the CIU triaxial tests conducted by Golder, the average undrained elastic modulus of the silty clay was estimated to be 7,500 kPa. In the lake borehole, 09-02S, the modulus of elasticity based on the SBPM tests ranges from 6,500 to 6,820 kPa. In the borehole drilled through the wharf (09-01C), the elastic modulus of the silty clay ranges from 15,450 to 26,400 kPa.

7.3 Clayey Silt Till

The surface of the clayey silt till was encountered at an elevation of 163.2 m by Golder in BH 102 and at an elevation of 162.3 m in BH09-02C drilled in close proximity to BH 102. The base of the clayey silt till was encountered at an elevation of 161.9 m in the Golder investigation and at an elevation of 159.9 m in BH09-02C during the present investigation.

SPT N-values within the hard clayey silt till layer were measured by Golder as 110 blows for 0.3 m of penetration, while the current investigation indicated N-values ranging from 26 blows for 0.3 m to 50 blows for 0.075 m of penetration.

No friction angle or modulus of elasticity was reported for this layer by Golder. The testing in the present investigation indicates an assessed friction angle of 38° and a modulus of elasticity ranging from 216,000 to 360,000 kPa.

7.4 Sand and Gravel

A layer of sand and sand and gravel was encountered below the cohesive till layer at an elevation 161.9 m in the investigation carried out in 2004. The current investigation contacted a sand and gravel layer at an elevation of 159.9 m in BH09-02C. The base of the cohesionless layer was encountered at an elevation of 158.7 m in both the previous and current investigation.

SPT N-values in the sand and gravel layer were previously reported to range from 41 to 112 blows for 0.3 m of penetration and were measured as 21 to 23 blows for 0.3 m of penetration in the borehole (09-02C) drilled in the vicinity during the current investigation.

No artesian groundwater flow originating from this formation was reported by Golder during the previous investigation or by the findings of the current investigation.

7.5 Conclusion

The following conclusions have been drawn from the current study:

- i. The soil stratigraphy encountered at the borehole locations drilled during the current investigation is very similar to the stratigraphy reported in the original investigation by Golder (2004);
- ii. The friction angle and shear strength of the surficial silty sand and the underlying silty clay layer determined from the current study is similar to those reported in the original investigation by Golder (2004);
- iii. The Golder report did not provide any elastic modulus parameters for the surficial silty sand layer. The current investigation indicates a relatively low modulus of elasticity of 3,350 to 6,620 kPa for this layer in the lake bed. The modulus of the silty sand measured below the wharf is higher and ranges from 21,600 to 60,000 kPa;
- iv. For the silty clay layer underlying the surficial silty sand layer, the undrained elastic modulus estimated from Golder's triaxial test is in the order of 7,500 kPa. The current

investigation indicates a modulus ranging from 6,500 to 6,820 kPa for the clay below the lake bed which is similar to the modulus reported by Golder (2004). The modulus measured beneath the wharf was estimated during the current investigation to range from 15,450 to 26,400 kPa.

9 MISCELLANEOUS

Thurber marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber also surveyed the as-drilled locations with a differential GPS (dGPS) and recorded the coordinates. Ground surface elevations were estimated based on measurements from the surface of the lake to the wharf and temporary deck levels.

George Downing Estate Drilling Inc. of Grenville-Sur-la-Rouge, Quebec supplied and operated a truck-mounted CME-75 drill rig to conduct the drilling, sampling and in-situ testing operations.

The shear strength of cohesionless soils was measured using an electric piezocone (CPT) and either a self-boring (SBPM) or pre-bored (HPPM) pressuremeters. All pressuremeter and CPT(u) testing and equipment was provided and operated by In Situ Engineering Ltd. of Snohomish, Washington.

The drilling, sampling and in-situ testing operations in the field were supervised on a full time basis by Mr. David E. Elwood, P.Eng. of Thurber.

Laboratory testing was carried out by Thurber Engineering Ltd. in its MTO-approved Oakville laboratory.

Interpretation of the field data and preparation of the investigation report was completed by Mr. David E. Elwood, P.Eng and Dr. P.K. Chatterji, P. Eng. Overall supervision of the field program was performed by Mr. David E. Elwood, P.Eng. The report was reviewed by Mr. Steven M. Sather, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

David E. Elwood, M.Sc., P.Eng.
Geotechnical Engineer

P.K. Chatterji, Ph.D., P.Eng.
Principal

Steven M. Sather, M.Eng., P.Eng.
Review Principal

Table 1
Summary of In-Situ Test Results

Borehole No.	Material Type	Depth (m)	Shear Vane		Pressuremeter			CPT(u)	
			Undrained Shear Strength s_u (kPa)	Sensitivity	Elastic Modulus (kPa)	Friction Angle ϕ' (°)	Undrained Shear Strength s_u (kPa)	Equivalent Strain Elastic Modulus (kPa)	Friction Angle ϕ' (°)
09-01	Silty Sand to Sandy Silt	7.5	N/A	N/A	60,000	32°	N/A	30,000	28 - 30°
		8.25	N/A	N/A	21,600	30°	N/A		
		8.8	N/A	N/A	36,000	30°	N/A		
	Silty Clay	10.5	75	2.3	26,400	29°	102.8	16,675	N/A
		11.25	72	2.3	16,800	29°	95		
		12	72	2.2	16,450	28°	79.7		
	Clayey Silt Till	12.75	72	1.6	15,450	28°	78.2	N/A	N/A
	Sand & Gravel	15.3	N/A	N/A	360,000	38°	N/A		
09-02	Silty Sand to Sandy Silt	10.5	N/A	N/A	3,350*	30°	N/A	N/A	N/A
		11.25	N/A	N/A	6,620	32°	N/A		
	Silty Clay	12.0	60	3.0	6,500	30°	80.8	16,450	N/A
		12.75	64	3.2	6,820	29°	80.8		
	Clayey Silt Till	14.0	N/A	N/A	216,000	38°	N/A	N/A	N/A

* - Sample likely disturbed during testing.

Table 2
Comparison of Soil Parameters
Leamington Dock
Silty Sand Layer (in Lake Erie)

Parameter	GAL BH102	TEL 09-02S (SBPM) 09-02C (sampled)		
		Depth (m)	Pressuremeter	CPT
SPT N	3/4	8.8–11.6 (167.4–164.6)	4/1	N/A
Silt & Clay (%)	40% to 45%		46%	
ϕ'	28° (As reported by GAL)	10.5	30°	
		11.25	32°	
Stiffness (Modulus)	Not reported in the Preliminary Investigation Report by Golder (2004)	10.5	3,350* kPa	
		11.25	6,620 kPa	

Table 3
Comparison of Soil Parameters
Leamington Dock
Silty Sand Layer (in Wharf)

Parameter	GAL BH102	TEL 09-01C (SBPM) TEL 09-01N (sampled)		TEL 09-01S	
		Depth (m)	Pressuremeter	Depth (m)	CPT
SPT N	3/4	6.1 – 9.5 (169.9 – 166.5)	5/6/2/3	N/A	N/A
Silt & Clay (%)	40% to 45%		25 to 47%		
ϕ'	28° (As reported by GAL)	7.5	32°	7.0 (169.0)	28° - 30°
		8.25	30°		
		8.8	30°		
Stiffness (Modulus)	Not reported in the Preliminary Investigation Report by Golder (2004)	7.5	60,000 kPa	5.6 – 9.2 (170.4 – 166.8)	30,000 kPa (γ_{adj})
		8.25	21,600 kPa		
		8.8	36,000 kPa		

Notes

GAL – Golder Associates Ltd.; TEL – Thurber Engineering Limited.

* - Disturbed sample

γ_{adj} – Adjusted elastic modulus (E) to applicable strain ($\gamma_{shear} < 10^{-2}$).

Table 4
Comparison of Soil Parameters
Leamington Dock
Silty Clay Layer (in Lake Erie)

Parameter	GAL BH102	TEL 09-02S (SBPM) TEL 09-02C (sampled)		TEL 09-02 N	
		Depth (m)	Sampled BH	Depth (m)	CPT
N	5	11.6-13.9 (164.6-162.3)	2/6	N/A	N/A
Clay content	N/A		52%		
Vane Shear Strength (Su)	54 - 115 kPa (Avg 85 kPa)	13.5	60 - 64 kPa		
Pressuremeter Results					
ϕ'	GAL 29° Peto $c' = 3$ kPa; $\phi' = 28^\circ$	12.0	30°	N/A	N/A
		12.75	29°		
Stiffness (Modulus) and SBPM Shear Strength (Su)	7,500 kPa	12.0	6,500 kPa (Su=80.8 kPa)	9.7 – 13.1 (166.5 – 163.1)	16,450 kPa (γ_{adj})
		12.75	6,820 kPa (Su=80.8 kPa)		

Table 5
Comparison of Soil Parameters
Leamington Dock
Silty Clay Layer (in Wharf)

Parameter	TEL 09-01C (SBPM) 09-01N (sampled)		TEL 09-01 S	
	Depth (m)	Sampled BH	Depth (m)	CPT
SPT N	9.5 - 12.7 (166.5-163.3)	4 / HW	N/A	N/A
Silt & Clay (%)		52%		
Vane Shear Strength (Su)	9.6	75 – 72 kPa		
	11.6	72 kPa		
Pressuremeter Results				
ϕ'	10.5	29°	N/A	N/A
	11.25	29°		
	12.0	28°		
	12.75	28°		
Stiffness (Modulus) and SBPM Shear Strength (Su)	10.5	26,400 kPa Su=102.8)	9.7 – 12.1 (166.3 – 163.9)	16,675 kPa (γ_{adj})
	11.25	16,800 kPa (Su=95)		
	12.0	16,450 kPa (Su=79.7)		
	12.75	15,450 kPa (Su=78.2)		

γ_{adj} – Adjusted elastic modulus (E) to applicable strain ($\gamma_{shear} < 10^{-2}$).

Table 6
Comparison of Soil Parameters
Leamington Dock
Clayey Silt Till

TEL BH No. (SBPM/Sampled)	Depth (m) (Elevation)	N	Silt and Clay (%)	Pressuremeter		
				ϕ'	Depth (m)	Modulus
09-01 C/N	12.7-15.3 (163.3-160.7)	51	64	38°	15.3	360,000 kPa
09-02 S/C	13.9-17.5 (162.3-158.7)	50/.075/26/21/23	3	38°	14	216,000 kPa
09-03 -/S	13.5-17.7 (162.7-158.6)	81/5/13/33/31	60 / 4	---	---	---

APPENDIX A

RECORD OF BOREHOLE SHEETS

RECORD OF BOREHOLE No 09-01N

1 OF 2

METRIC

G.W.P. 17-454-92 LOCATION _____ ORIGINATED BY DEE
 HWY Leamington Ferry Dock BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2009.04.14 - 2009.04.14 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
176.0	Geodetic							<div>20406080100</div> <div>○ UNCONFINED + FIELD VANE</div> <div>● QUICK TRIAXIAL × LAB VANE</div>					
0.0	ASPHALT (200mm)						176	<div>4080120160200</div> <div>W P W W L</div> <div>WATER CONTENT (%)</div>					
0.2	SAND, trace to some gravel Grey Loose (FILL)		1	SS	9		175	<div>○</div>					
174.5	SAND and GRAVEL, recycled asphalt, recycled concrete, cobbles Compact Moist (FILL) Frequent probable voids ranging from 25 to 75mm (estimated) based on separation of the top of auger with drive head cap. Voids noted throughout the depth of the granular fill layer. Increasing silt content, some white silty sand to sandy silt layers (possible concrete grout) Probable voids observed during SPT advancement. Spoon would fall up to 25mm under self weight and require energy to advance further. Occasional grinding on probable cobbles during auger advancement. Probable voids observed during advancement of the borehole. Voids range from 25 to 75mm based on separation of top of auger with drive head cap.		2	SS	15		174	<div>○</div>					
1.5			3	SS	13		173	<div>○</div>					
			4	SS	12		172	<div>○</div>					
			5	SS	7		171	<div>○</div>					
			6	SS	5		170	<div>○</div>					
			7	SS	5		169	<div>○</div>					
			8	SS	5		168	<div>○</div>					
169.9	Silty SAND, trace to some gravel, trace clay Loose Brown Wet Some Clay Stratified with alternating layers of sand and silt ranging in thickness from 5 to 25mm. Lamination thickness increasing with depth.		9	SS	6		167	<div>○</div>					
6.1			10	SS	2			<div>○</div>					
			11	SS	3			<div>○</div>					
			12	SS	2			<div>○</div>					
166.5								<div>○</div>					
9.5								<div>○</div>					

60382
(SI+CL)

570205
Approximately
2.4m of blowback
into augers,
switched to NW
casing and wash
boring. SS9 likely
highly disturbed;
stratigraphic
change assumed
based on SPT
sample.

053398

60 38 2
(SI+CL)

5 70 20 5
Approximately
2.4m of blowback
into augers,
switched to NW
casing and wash
boring. SS9 likely
highly disturbed;
stratigraphic
change assumed
based on SPT
sample.

0 53 39 8

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-01N

2 OF 2

METRIC

G.W.P. 17-454-92 LOCATION _____ ORIGINATED BY DEE
 HWY Leamington Ferry Dock BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2009.04.14 - 2009.04.14 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						PLASTIC LIMIT W P NATURAL MOISTURE CONTENT W LIQUID LIMIT W L WATER CONTENT (%)
	Continued From Previous Page							20 40 60 80 100 40 80 120 160 200						
163.3	Silty CLAY , trace sand Soft to Very Soft Brown Moist (CI)		13	SS	4		166	23 +						0 17 31 52
12.7	Clayey SILT , some sand, trace gravel, frequent cobbles (as noted in BH09-1C approximately 2.1m south)						165							
162.5	Hard Grey Moist (TILL)		14	SS	HW		164	22 + 16 +						4 33 38 26
13.5	END OF BOREHOLE AT 13.5m UPON SPLIT SPOON REFUSAL. OPEN BOREHOLE WATER LEVEL MEASURED AT 1.8 m DEPTH UPON COMPLETION OF DRILLING (APPROXIMATE LAKE ELEVATION) LARGE VOIDS OBSERVED BELOW ASPHALT UPON COMPLETION OF BOREHOLE. THE VOLUME OF VOIDS WAS ESTIMATED TO BE APPROX. 5 TO 6 CUBIC METERS. VOIDS BACKFILLED WITH UNCOMPACTED GRANULAR A MATERIAL TO ABOVE GROUND SURFACE TO MINIMIZE SETTLEMENT OF PAVEMENT.		15	SS	51		163							

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

METRIC

[illegible]

ONTMT4S 5492.GPJ 8/5/09

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 09-02C

2 OF 3

METRIC

G.W.P. 17-454-92 LOCATION _____ ORIGINATED BY DEE
HWY Leamington Ferry Dock BOREHOLE TYPE HW / NW Casing COMPILED BY AN
DATUM Geodetic DATE 2009.04.18 - 2009.04.18 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _P W W _L WATER CONTENT (%)							
Continued From Previous Page														
164.6	Silty SAND , trace to some clay, trace gravel Very Loose Grey Slight Organic Odour Wet		2	SS	1		166							
								165						
11.6	Silty CLAY , some sand, trace gravel Soft Brown with Grey Laminations Moist Gravel trapped in split spoon tip (~50mm dia.) (CI) Gravel trapped in split spoon tip (~50mm dia.)		3	SS	2									
								164						
			4	SS	6									
162.3							163	3.0 +						
								3.2 +						
			5	SS	50/									
13.9	Clayey SILT , some sand, trace gravel Hard Grey Moist (TILL) Very Stiff White cemented sand to fractured siltstone, thinly spaced, highly weathered Weak (assumed)				0.075		162							
				6	SS	26		161						
159.9			7	SS	21		160							
16.3	SAND and GRAVEL , trace silt, trace siltstone Compact Dark Brown to Black Wet No siltstone fragments observed													
				8	SS	23		159						
158.7														
17.5	SILT , some sand to sandy, trace clay Compact Grey Wet Dilatant		9	SS	24		158							
				10	SS	16								
157.3														
18.9	Sandy SILT , some clay, trace gravel Compact Dark Grey Moist (TILL)		11	SS	17		157							
156.7														
19.5	Medium to coarse grained sand layer encountered in last 100mm of SPT.													

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
20
15
10
(%) STRAIN AT FAILURE

METRIC

[illegible]

RECORD OF BOREHOLE No 09-03S

1 OF 3

METRIC

G.W.P. 17-454-92 LOCATION _____ ORIGINATED BY DEE
 HWY Leamington Ferry Dock BOREHOLE TYPE HW / NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2009.04.15 - 2009.04.15 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
176.2	Geodetic													
0.0	Platform level above Lake Erie surface						176							
174.4							175							
1.9	Water to a depth of 7.9m from top of platform						174							
							173							
							172							
							171							
							170							
							169							
168.3							168							
7.9	Sandy SILT, some clay, trace gravel Very Loose Grey Wet		1	SS	HW									
	Possible Concrete Grout in Split Spoon Tip		2	SS	HW									
	Trace to Some Clay													

Split spoon
pushed
hydraulically to
8.4m to start SPT

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15 10 5 0
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-03S

2 OF 3

METRIC

G.W.P. 17-454-92 LOCATION _____ ORIGINATED BY DEE
 HWY Leamington Ferry Dock BOREHOLE TYPE HW / NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2009.04.15 - 2009.04.15 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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+³, ×³: Numbers refer to
Sensitivity 20
15 10 5 10
(%) STRAIN AT FAILURE

METRIC

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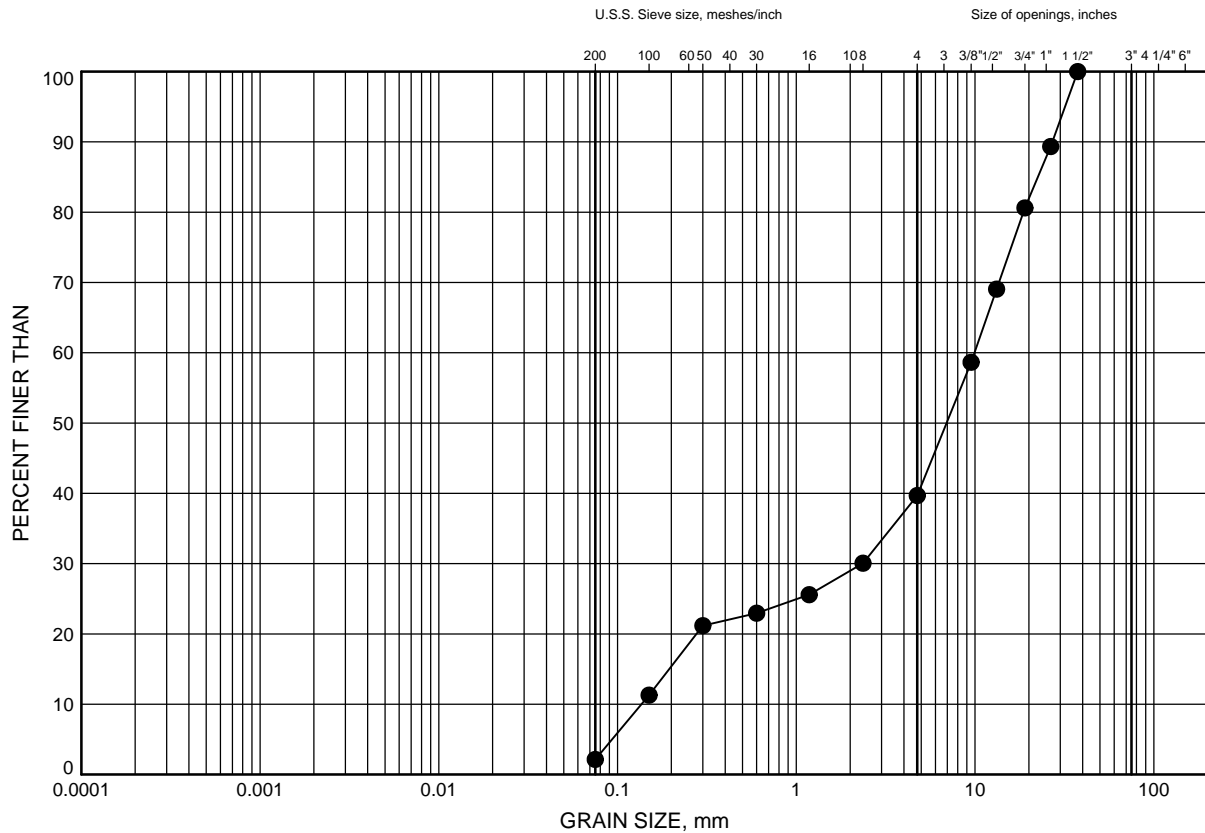
APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

Delcan Corporation
GRAIN SIZE DISTRIBUTION

FIGURE B1

GRANULAR FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01N	3.35	172.65

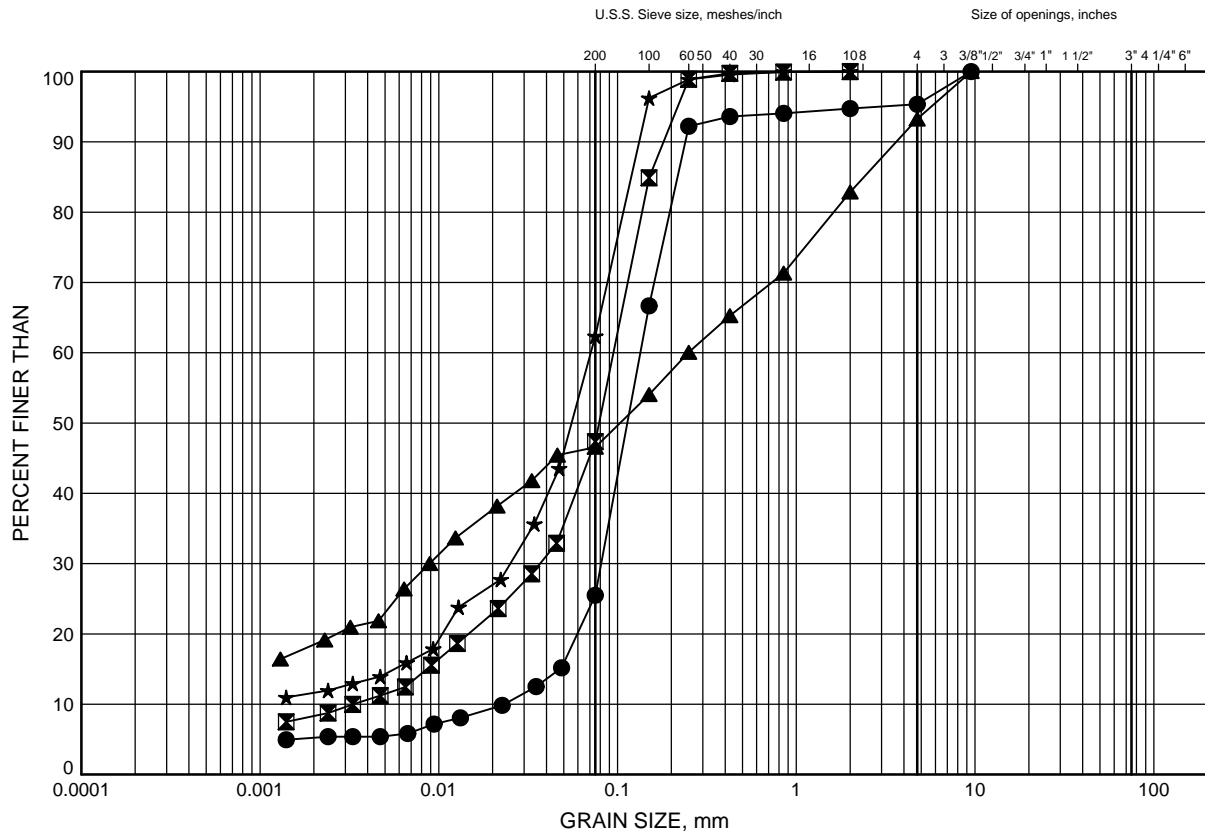


W.P.# 17-454-92
 Prepared By MFA
 Checked By DEE

Delcan Corporation
GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND to SANDY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01N	6.40	169.60
⊠	09-01N	8.69	167.31
▲	09-02C	9.75	166.45
★	09-03S	10.21	166.03

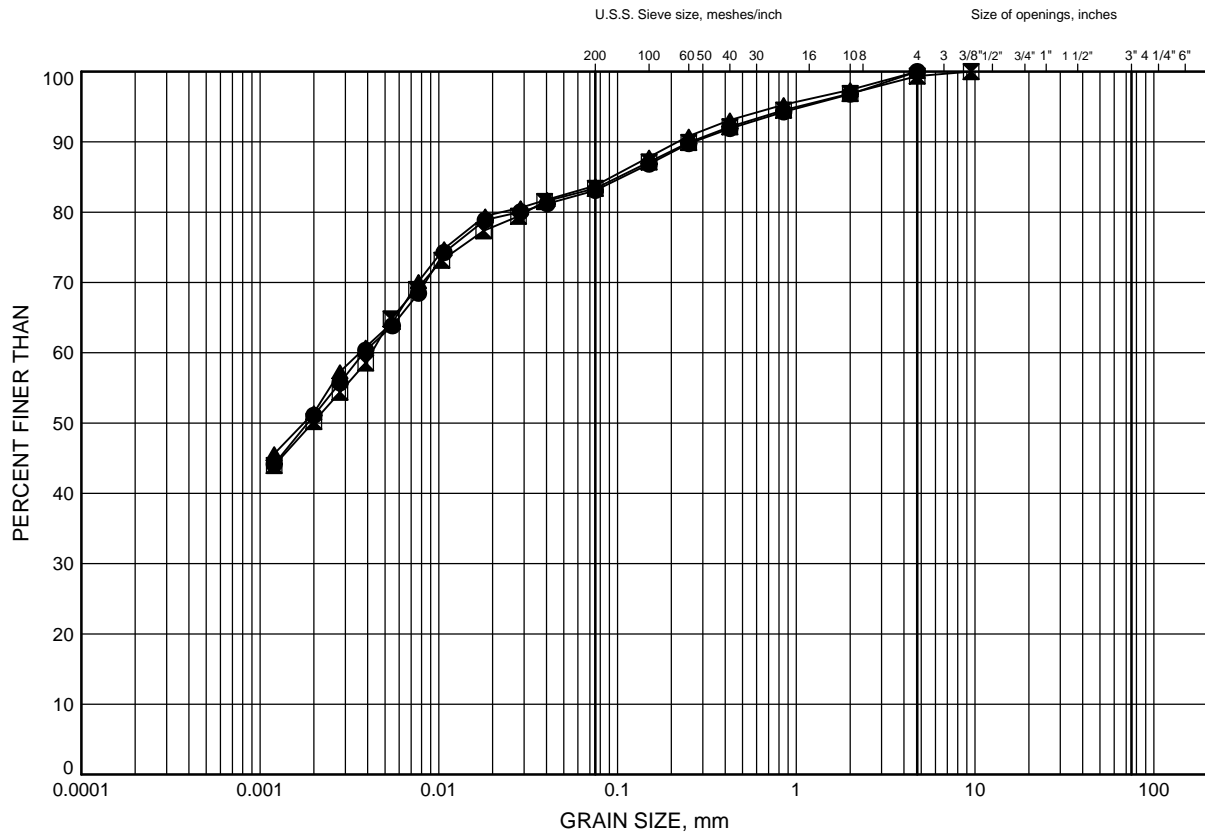


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Prepared By MFA
Checked By DEE

Delcan Corporation
GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01N	10.97	165.03
⊠	09-02C	12.50	163.70
▲	09-03S	12.50	163.75

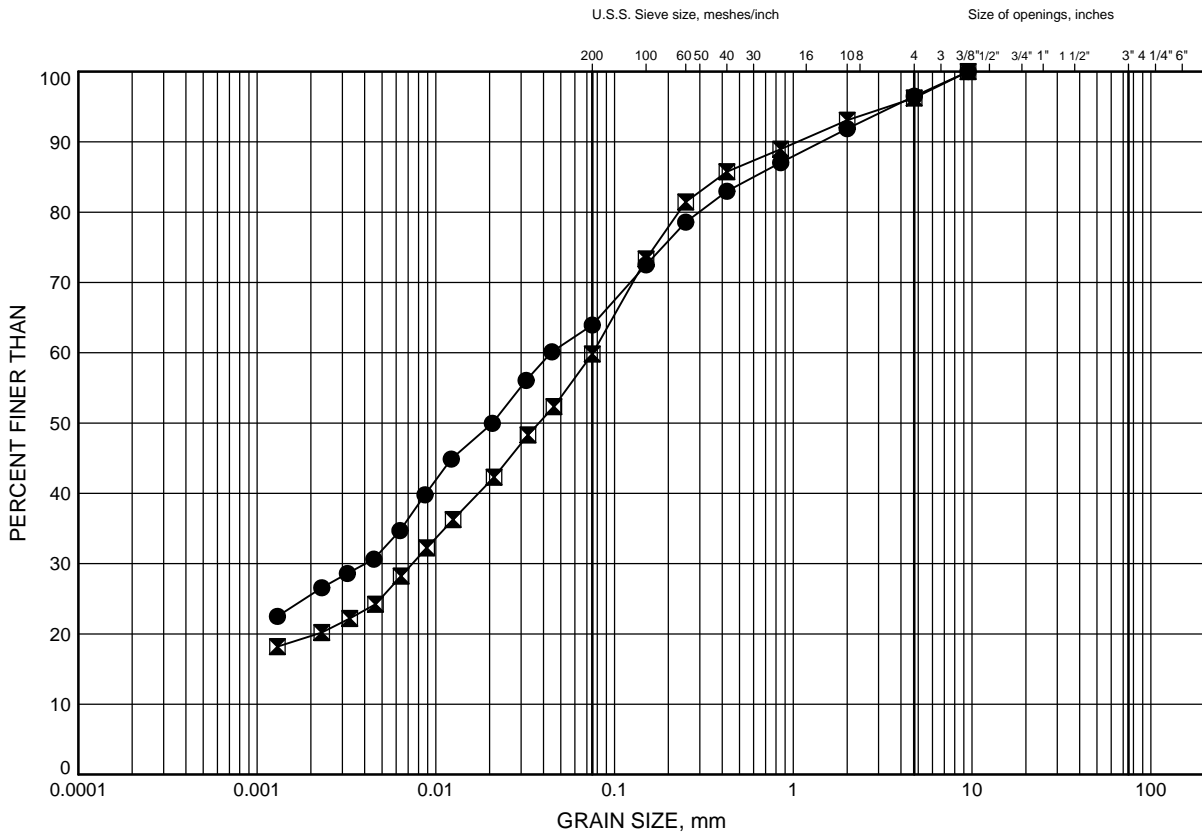


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Delcan Corporation
GRAIN SIZE DISTRIBUTION

FIGURE B4

CLAYEY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01N	13.26	162.74
⊠	09-03S	14.02	162.22

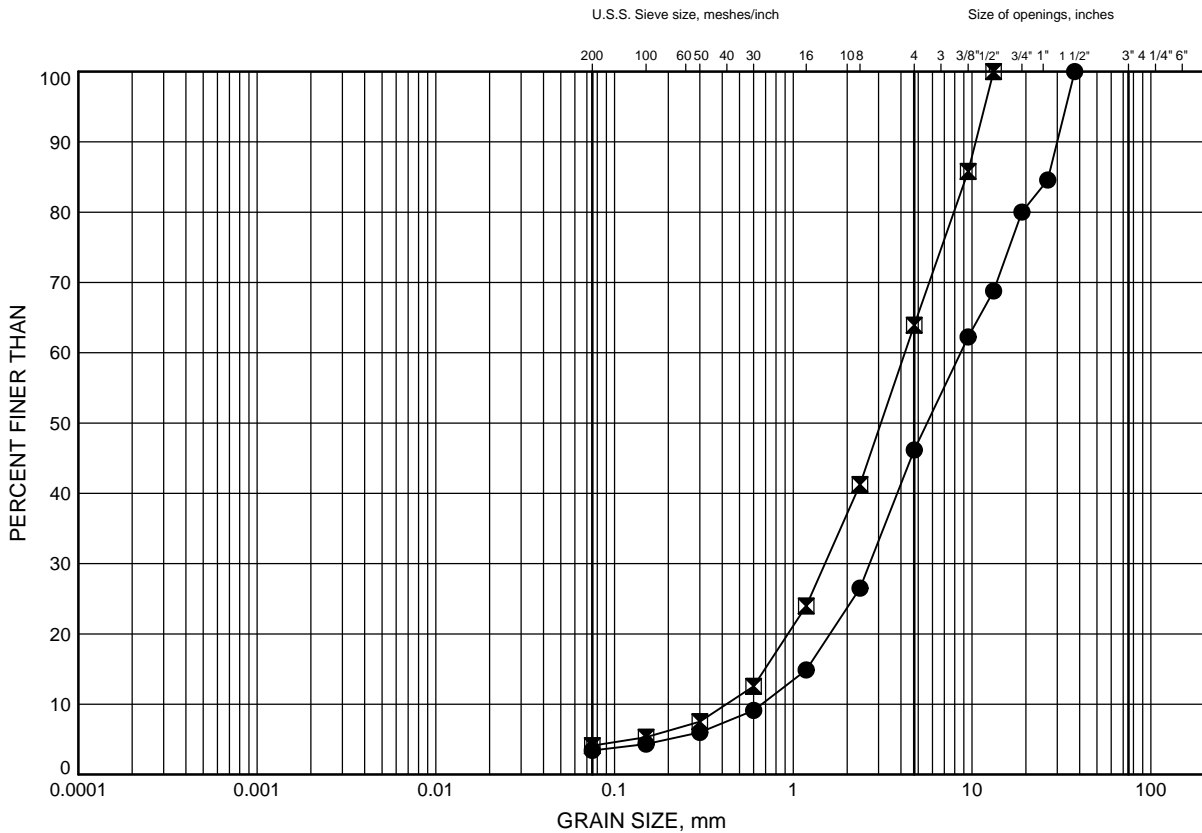


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Delcan Corporation
GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND & GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

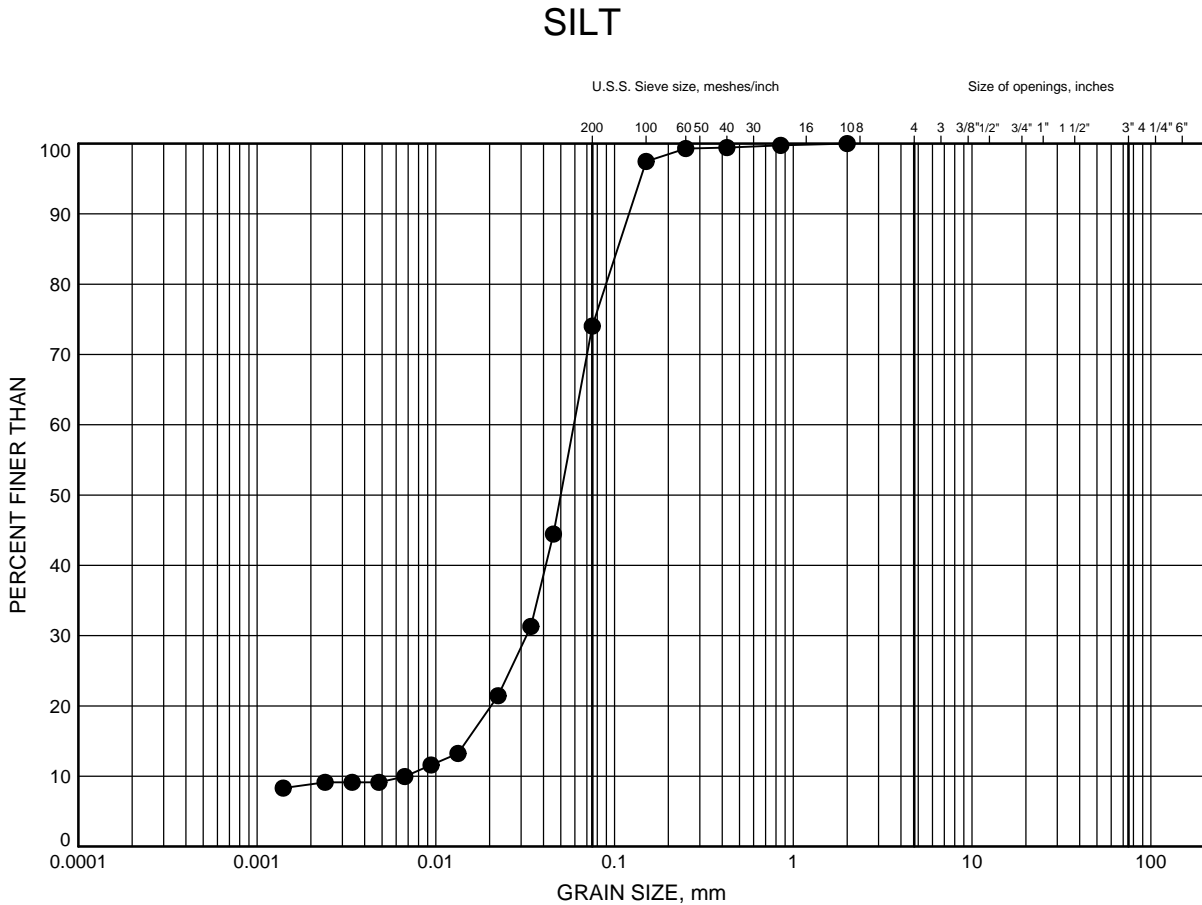
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-02C	16.31	159.89
⊠	09-03S	16.31	159.94



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GRAIN SIZE DISTRIBUTION

FIGURE B6



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-03S	17.83	158.41

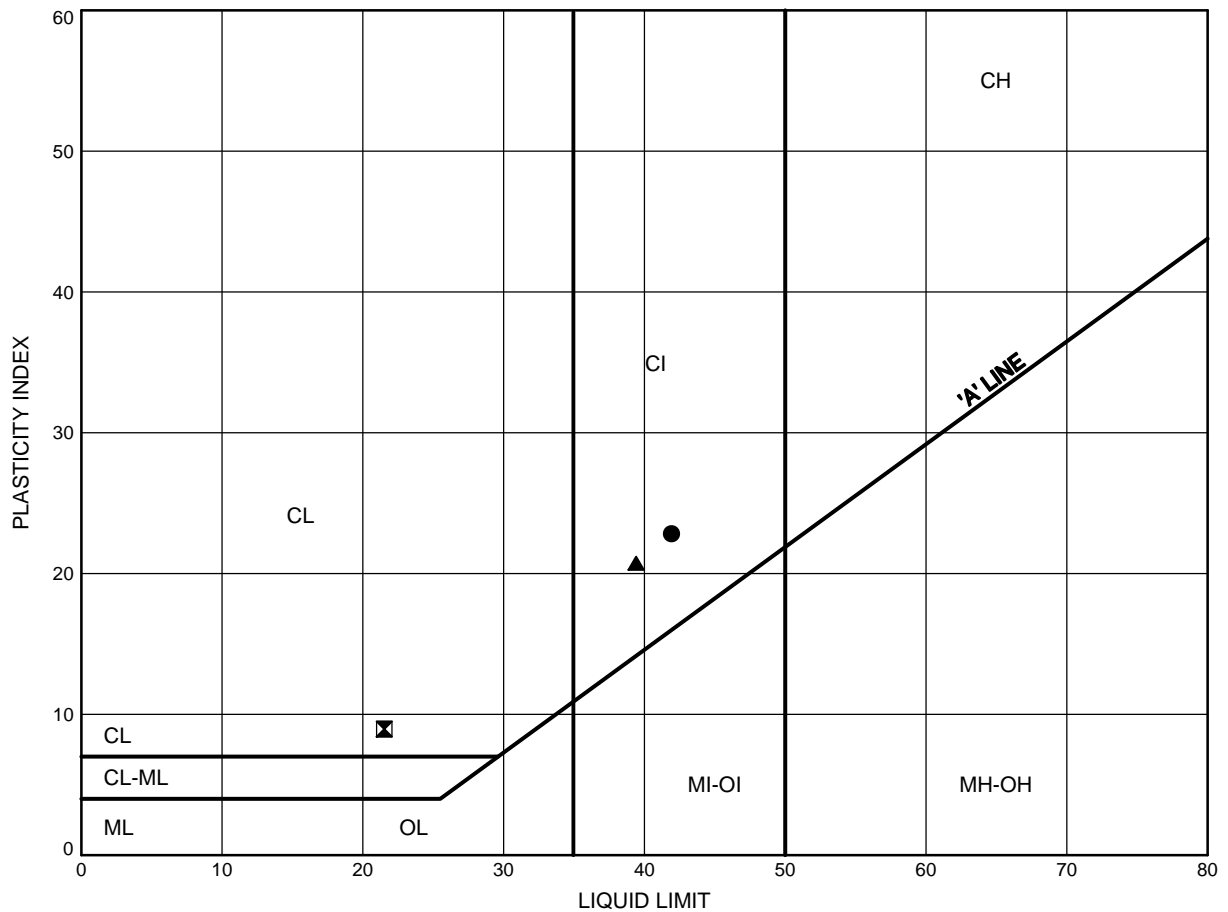


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Checked By DEE

Delcan Corporation

ATTERBERG LIMITS TEST RESULTS

FIGURE B7



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-01N	10.97	165.03
⊠	09-01N	13.26	162.74
▲	09-03S	12.50	163.75

Date May 2009
 Project 17-454-92



Prep'd MFA
 Chkd. DEE

APPENDIX C

BOREHOLE LOCATIONS AND SOIL STRATA DRAWING

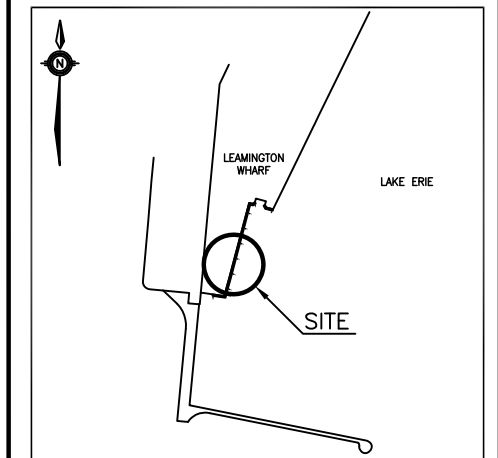
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No







LEAMINGTON DOCK
SOIL TESTING
LEAMINGTON, ONTARIO
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET |



KEYPLAN

LEGEND

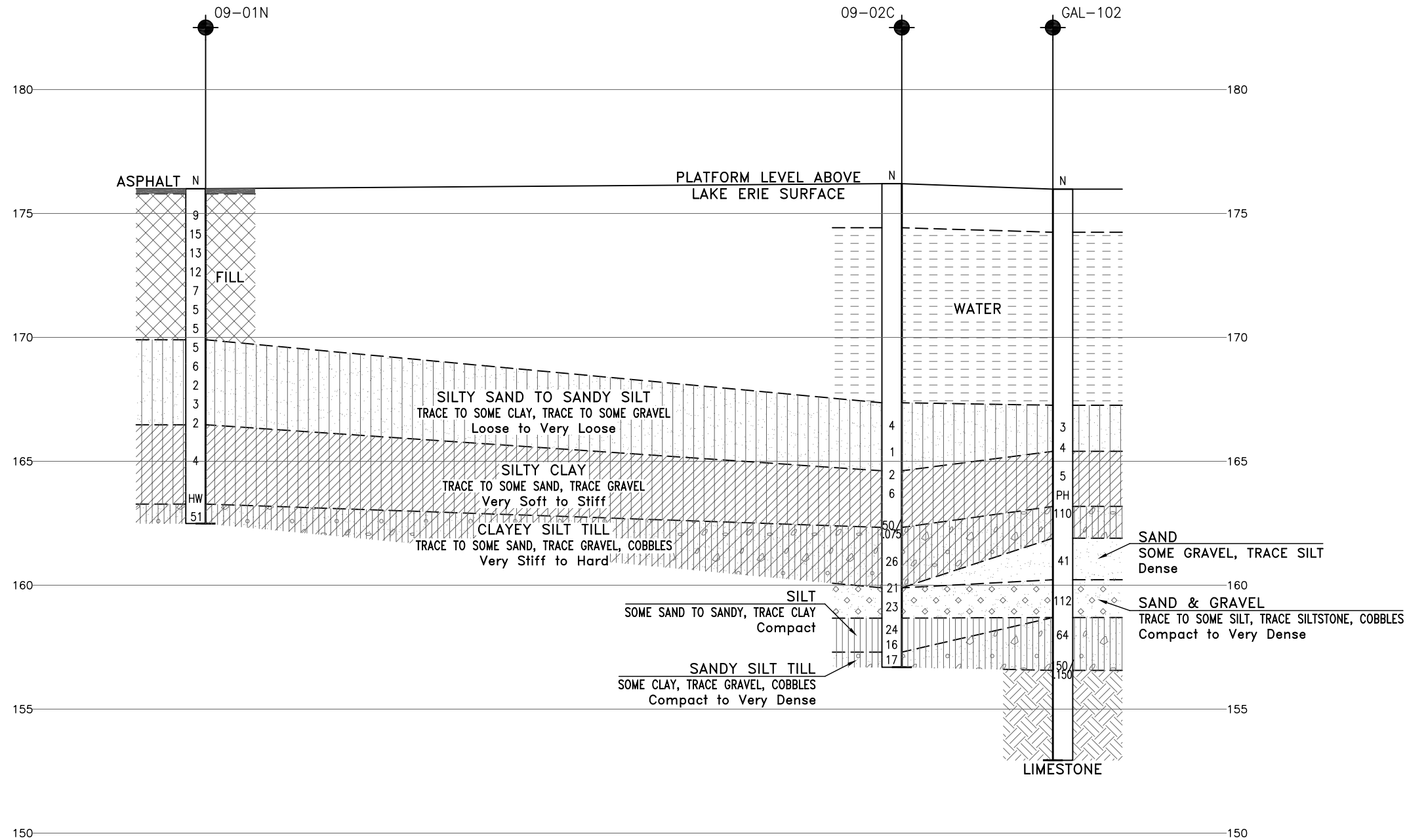
	Thurber Borehole (2009)
	CT Soils Borehole (2008)
	GAL Borehole (2004)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

[illegible]

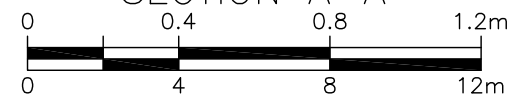
-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 40J2-111



SECTION A-A



H 1:20

V 1:200

[illegible]

APPENDIX D

SITE PHOTOS



Figure 1 – Drill rig and temporary platform over Lake Erie (09-03)



Figure 2 – Temporary mats for the drill rig to bridge voids under the asphalt



Figure 3 – MTO 'B' vane following completion of sampling boreholes



Figure 4 – CPT(u) prior to testing at (09-03N)



Figure 5 – Shear wave beam under drill rig outriggers prior to testing

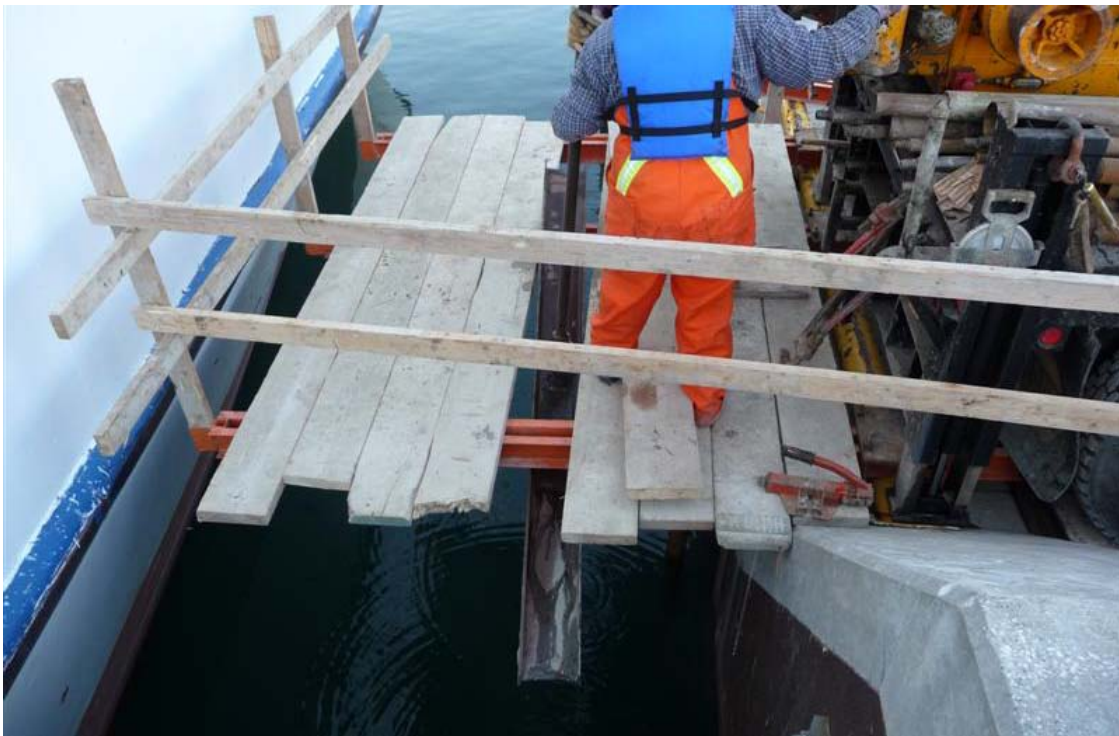


Figure 6 – Lowering of shear wave beam to mudline (09-02N)

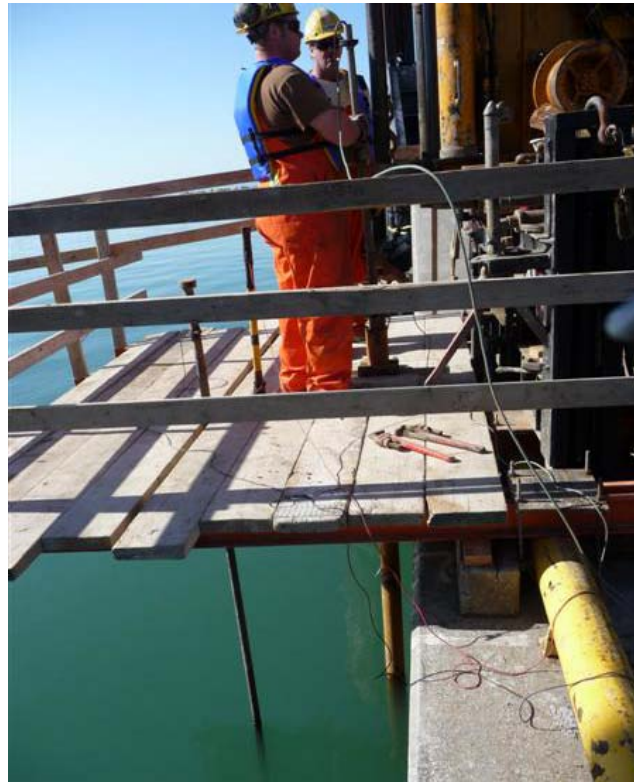


Figure 7 – CPT(u) and shear wave rods cantilevered over the dock

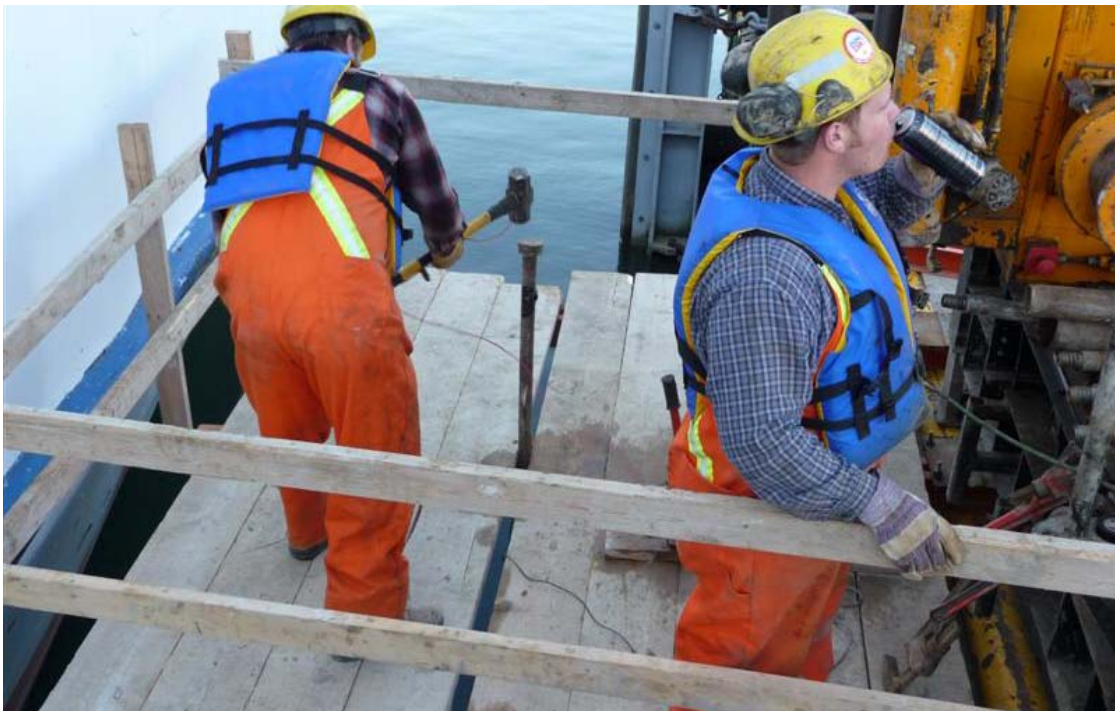


Figure 8 – Shear wave impact within temporary dock structure



Figure 9 – Self-Bore Pressuremeter and jetting tool (unassembled)

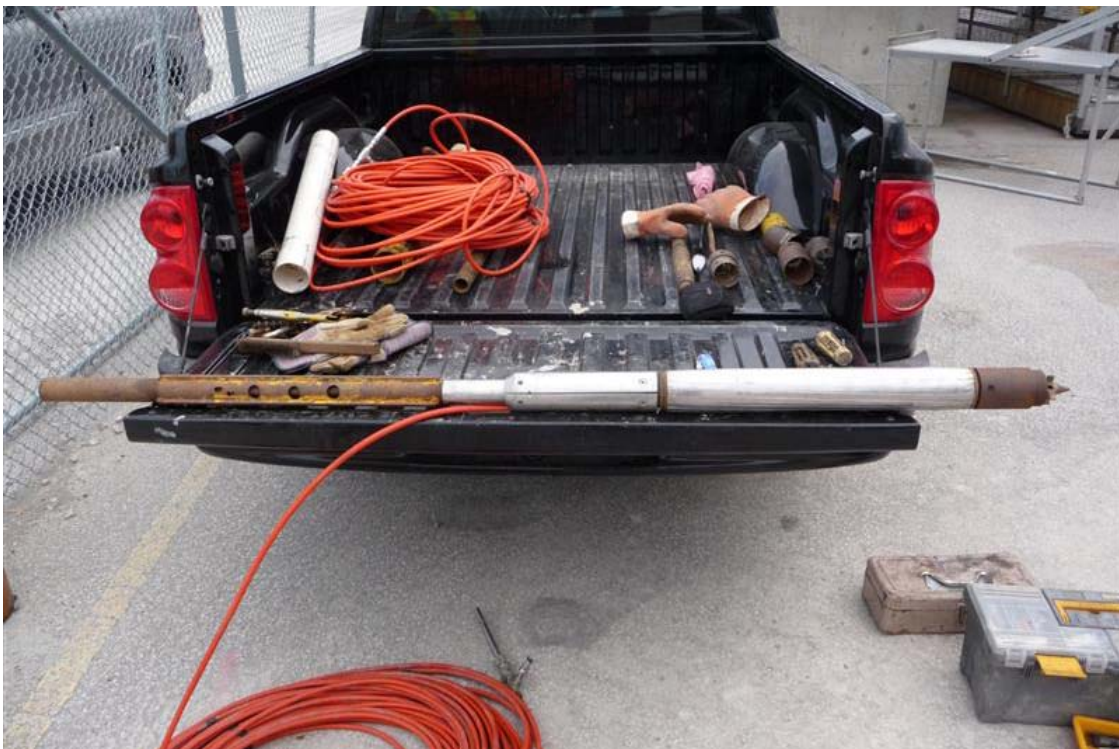


Figure 10 – Self-Bore Pressuremeter and jetting tool (assembled) without cutting shoe



Figure 11 – Downhole insertion of the SBMP (09-02S)



Figure 12 – Damage to SBPM shoe from gravel obstruction (09-01C)



Figure 13 – SBPM and High Pressure PM



Figure 14 – Insertion of HPPM in (09-02S)

APPENDIX E

IN SITU ENGINEERING REPORT

Report of In Situ Pressuremeter and Cone Penetration Testing

Conducted on:

Leamington Dock Temporary Works
Leamington, ON

Submitted to:

Thurber Engineering, Ltd.
Oakville, ON

In Situ Engineering Project Number 835

April 2009

Testing conducted and report prepared by:

In Situ Engineering

6232 195th Avenue SE

Snohomish, WA 98290 360-568-2807

keith@insituengineering.com

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APPENDIX I – PRESSUREMETER DATA AND ANALYSIS

APPENDIX II – CONE PENETROMETER DATA AND ANALYSIS

1.0 INTRODUCTION

This report presents the results of in situ geotechnical testing conducted over a 5 day period from April 16th to April 20th 2009 at the Leamington Dock in Leamington, ON. Testing was conducted by In Situ Engineering, Snohomish, WA under contract to Thurber Engineering, Ltd of Oakville, ON. Three Cone Penetration Tests and two Pressuremeter Borings were completed as part of the investigation: CPT09-1S, CPT09-2N, CPT09-3N, PM09-1C and PM09-2S. The deployment of instruments and drilling was performed by Downing Drilling of Grenville-Sur-La-Rouge, QC under contract to Thurber. Testing was performed both through the dock surface as well as on an overwater platform.

The purpose of this study was to evaluate the *in-situ* material properties of the soft soils below and adjacent to the Leamington Dock as part of an independent investigation. Minimal testing was also performed in dense material to be used for comparative purposes.

Fieldwork at the Leamington Dock was performed under the supervision of an onsite engineer from Thurber Engineering. Operation of the test instrumentation was performed by a geotechnical engineer from In Situ Engineering. The cone penetrometer (CPT) and pressuremeter (PMT) instrumentation was provided by In Situ Engineering. CPT09-1S and PM09-1C were performed through the dock surface and subsequent rock fill. CPT09-2N, CPT09-3N and PM09-2S were conducted with the use of an overwater platform attached to the dock in a cantilever fashion allowing access to soils over the side of the dock. See Figure 1.

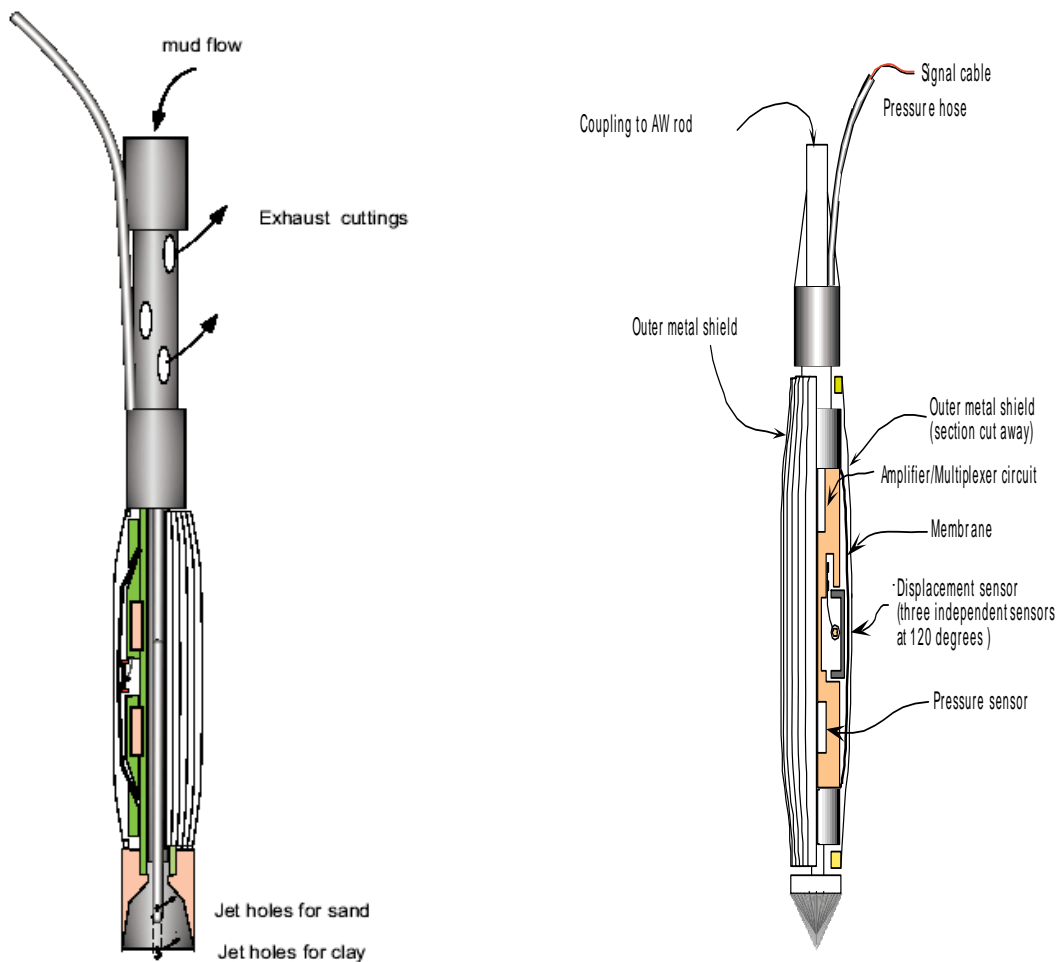


Fig.1. Picture of Cone Penetrometer Shear Wave Testing using the overwater platform.
The platform was used for both CPT and PMT.

2.0 PRESSUREMETER TESTING

The pressuremeters used for this study were a pre-bored mono-cell pressuremeter and a self-boring mono-cell pressuremeter. Both pressuremeters have three electronic displacement sensors, spaced 120 degrees apart that are located at the center of the pressuremeter. A flexible membrane is placed over the sensors and clamped at each end. The membrane is covered by a protective sheet of stainless steel strips that expand with the instrument. The unit is pressurized using compressed nitrogen which expands the membrane and deforms the adjacent material. The electronic signals from the displacement sensors and the pressure sensor are transmitted by cable to the surface. During the test, the average expansion versus pressure is displayed on a computer screen. The pressuremeter is expanded by regulating the flow of compressed nitrogen to the PMT unit.

Fig.2. Below presents the essential details of both the self-boring and pre-bored pressuremeters.



**Fig.2. Schematic Details of the Pressuremeter
Self-boring (left), Pre-bored (right)**

2.1 PRESSUREMETER HOLE FORMATION

A CME-75 mud rotary rig was used to advance both the self-boring and pre-bored pressuremeter instruments. At the location of PM09-1C, HW casing was advanced through rock fill below the surface to a depth of 6m below the deck of the dock. At the PM09-2S location, casing was set to the mudline. The self-boring pressuremeter was then lowered to the mudline and a test pocket was jetted with a polymer mixture and down-pressure from the drill rig to a depth of 1.5m below mudline. When the pressuremeter test was completed at this depth, it was advanced another 0.75m. This process was continued until the glacial till layer was reached, allowing four test intervals in PM09-2S and six tests in PM09-1C. A dense layer, or possible drop stone encountered in PM09-1C caused one test pocket to be stopped shy of 0.75m. The instrument was brought to the surface and cleaned before advancing the pressuremeter another 1.5m into the soil. Intervals of 0.75m were then used for the remainder of the self-boring testing in PM09-1C.

The pre-bored instrument was used in the denser sand, gravel and till below the soft soils. Two tests were performed in each borehole, attempting one test in sand and gravel and one in till. The test pocket was created using an NQ core barrel drilled to a length of 2m. The instrument was then advanced using down pressure from the drill rig to the bottom of the test pocket. After a test was performed, the instrument was lifted 1m and a second test was performed.

Test LD15 in test hole PM09-1C had the membrane rupture and could not be completed. Test LD05 appears to be highly disturbed and the modulus determined from this test appears to be not representative of till.

In all, 15 pressuremeter tests were attempted. The borehole name, test depths and soil description are presented in Table 1A and 1B.

Table 1A Pressuremeter Test Depth and Material Description: PM09-1C

Hole	Test	Date	Test Depth* (m)	Soil Description
PM09-1C	LD07	4/19/2009	7.5	sand
PM09-1C	LD08	4/19/2009	8.25	silty sand
PM09-1C	LD09	4/19/2009	8.8	sand to silty sand
PM09-1C	LD10	4/20/2009	10.5	sandy silt, clay
PM09-1C	LD11	4/20/2009	11.25	silty clay
PM09-1C	LD12	4/20/2009	12	silty clay
PM09-1C	LD13	4/20/2009	12.75	silty clay
PM09-1C	LD14	4/20/2009	15.3	till
PM09-1C	LD15	4/20/2009	14.3	till

***Test depths are measured from the deck of the dock surface**

Table 1B Pressuremeter Test Depth and Material Description: PM09-2S

Hole	Test	Date	Test Depth* (m)	Soil Description
PM09-2S	LD01	4/19/2009	10.5	silty clay, clayey silt
PM09-2S	LD02	4/19/2009	11.25	clayey silt
PM09-2S	LD03	4/19/2009	12	sandy silt, clay
PM09-2S	LD04	4/19/2009	12.75	sandy silt, clay
PM09-2S	LD05	4/19/2009	14.7	till
PM09-2S	LD06	4/19/2009	14	till

***Test depths are measured from the deck of the overwater platform**

2.2 PRESSUREMETER TEST PROCEDURE

The membrane was expanded by controlling the flow of compressed nitrogen into the pressuremeter, increasing the pressure in small steps until the membrane starts to expand against the borehole wall. Once the average strain of the wall was greater than about 1.5% the pressure is reduced to no more than 40% of the maximum past pressure, then increased again.

The resulting unload-reload loop can be used to evaluate the elastic behavior of the material. In materials which behave in a plastic manner, the loops will exhibit a hysteretic behavior. That is, the unloading path will follow the “mirror” image of the reloading path. In linear materials such as sands the loops will be very tight exhibiting little hysteretic behavior.

The pressure is then advanced in steps until the strain is increased a further 3% before completing a second unload-reload cycle. In many tests the procedure is repeated until a third unload-reload loop is completed. In some cases, a fourth loop is completed in order to achieve the goal of obtaining parallel loops. If the disturbance is small, the slope of the loops will tend to be parallel.

In Figure 3, Test LD07, from hole PM09-1C, is a typical example of a test in sand. The membrane begins to expand, pushing a small amount of slough until the membrane is pushing against the solid borehole wall. The steps above are then carried out in accordance with the operator’s judgment.

After the strain exceeds about 12%, the pressure is reduced to zero. The exact strain at which the pressure is reduced is again a judgment controlled by the operator based on the behavior of the three arms and pressures experienced.

In cohesive materials, the final unloading curve can be used to measure the shear stress as the direction of the strains reverse. This strength can be used to give a conservative indication of the shear strength.

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 7.5m	File C:\DATA\SE-835\LD07.P

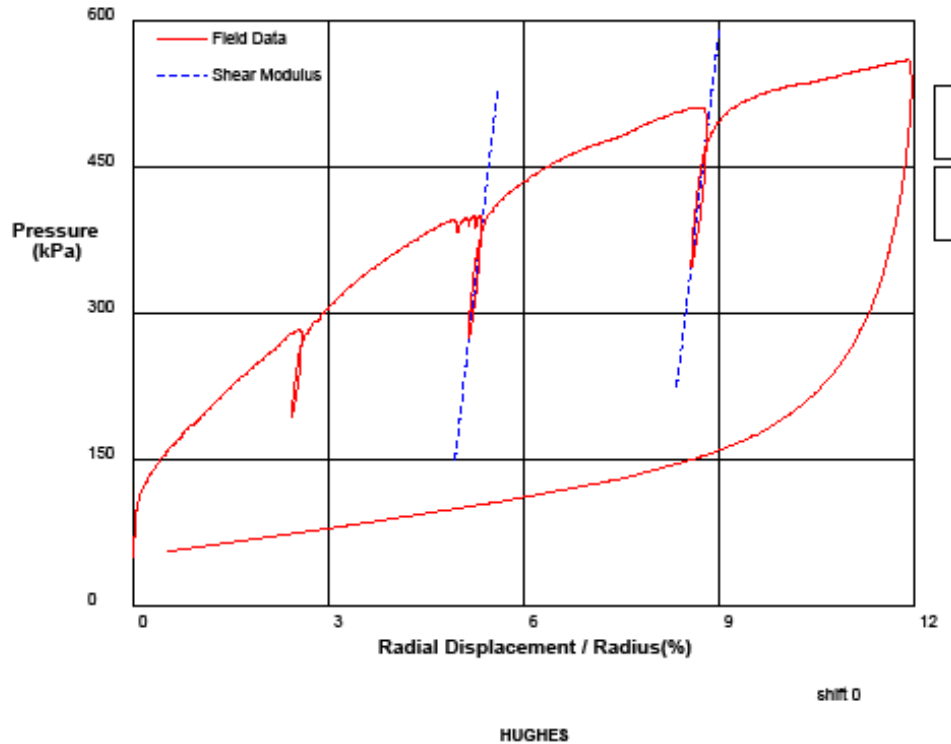


Fig.3. Test LD07 in borehole PM09-1C at 7.5m below deck

2.3 STANDARD METHOD OF ANALYSIS OF THE SHEAR MODULUS

If the material surrounding the pressuremeter is assumed to extend to infinity, and assumed to behave as an idealized linear elastic, homogeneous material, which does not fail under shear or tension, then the displacement on the boundary of the pressuremeter, u_a , for a given pressure, P , is given by:

$$u_a = P(a) (1+\mu) / E \quad 1)$$

where “E” is the Young’s Modulus, “a” the radius of the pressuremeter cavity, and “μ” the Poisson’s ratio. As the shear modulus, “G”, and the Young’s modulus, “E”, are related by the following relationship:

$$E=2(G)(1+\mu) \quad 2)$$

Equation 1 reduces to:

$$u_a = 0.5P(a) / G \quad 3)$$

Hence, the shear modulus G is given by:

$$G = 0.5 * \Delta \text{ Pressure} / \Delta(\text{radial displacement/radius}) \quad 4)$$

The modulus for the average slope of the initial part of the pressuremeter curve (A-B in Fig.4) expressed as a Young's modulus (assuming a Poisson's ratio of 0.33) is the same as the "pressuremeter modulus" defined in the American Society for Testing and Materials (ASTM) D4719, Section 9.5. However, the modulus determined from the unload-reload loops, which is often higher than the initial loading modulus, is probably more representative of the modulus for the *in-situ* material. This data is summarized in Tables 2A and 2B.

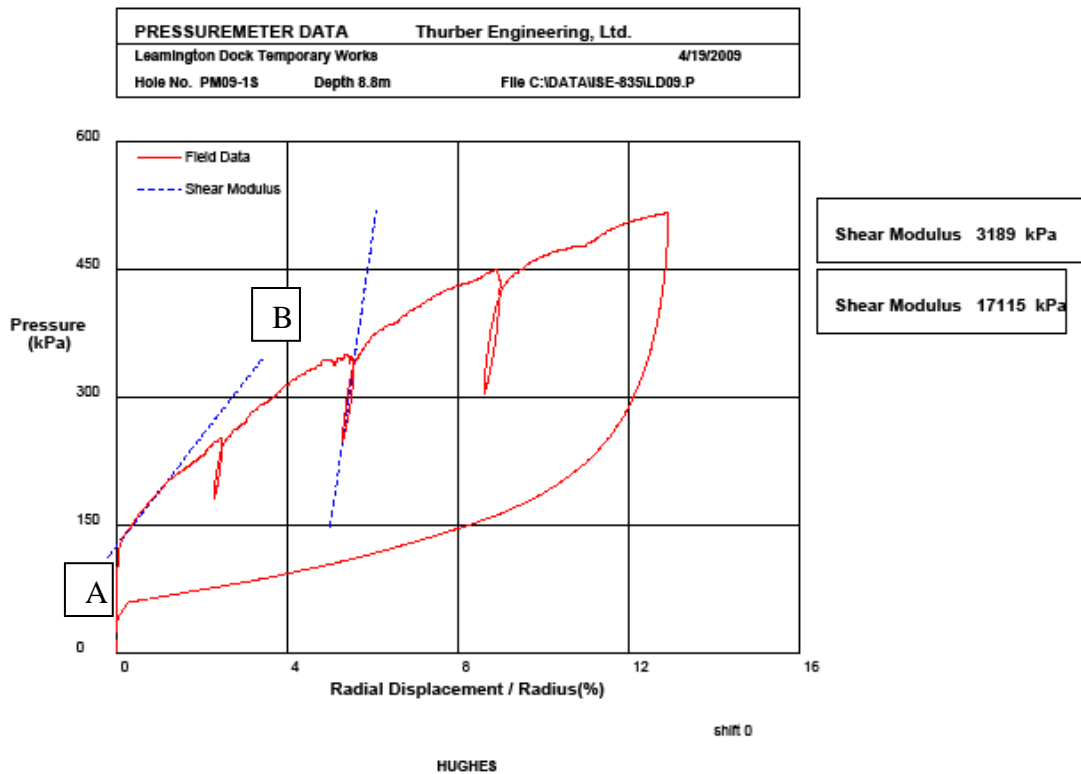


Fig.4. Shear Modulus Determination for Test LD09

2.4 DETERMINATION OF THE LIMIT PRESSURE

From a visual inspection of the typical pressuremeter curve, LD07, as shown in Fig. 3, the pressure tends to a limit. For Test LD07, this limit pressure is in the range of 550 psi. However, to make this limit pressure a quantitative measurement, the limit pressure is defined as that pressure which occurs when the volume of the pressuremeter has doubled. However, few pressuremeter tests ever actually expand this far before reaching the limit of the strain sensing system. The pressuremeters used in this investigation will only expand to about 20% before the displacement limit is reached.

If the material being tested is assumed to behave as an elastic cohesive material, then the equation governing the pressure-displacement curve is given by:

$$P = P_L + (c) \log_e (u_a/a) \quad 5)$$

$$P_L = P_o + c + (c) \log_e [G/c] \quad 6)$$

where “ P_L ” is the theoretical limit pressure at infinite expansion, “ c ” is the undrained cohesive strength, “ P_o ” is the total *in-situ* lateral stress, and “ G ” is the shear modulus. For typical values of G and c , the ratio of G/c lies between 50 and 100. Hence, the limit pressure is approximately 5 times the shear strength (assuming P_o is small relative to c) of the soil.

From Equation 5, a plot of pressure P against the log of u_a/a will be a straight line, provided the shear strength remains constant with strain. The slope of this line will provide a measure of the undrained shear strength, c . The Limit Pressure, as defined by the ASTM code D4719, Section 10.6, is the pressure at which the cavity has doubled in size. This doubling in size occurs when u_a/a is equal to 41%. (The origin of the strain used in the log/normal plots is the assumed origin at the *in-situ* stress state). If any disturbance is present, the above method of determining the cohesive strength usually provides an overly optimistic value. In Fig. 5, Test LD07 is plotted in the above manner. Limit pressures and shear strengths calculated by this method are presented in Tables 2A and 2B.

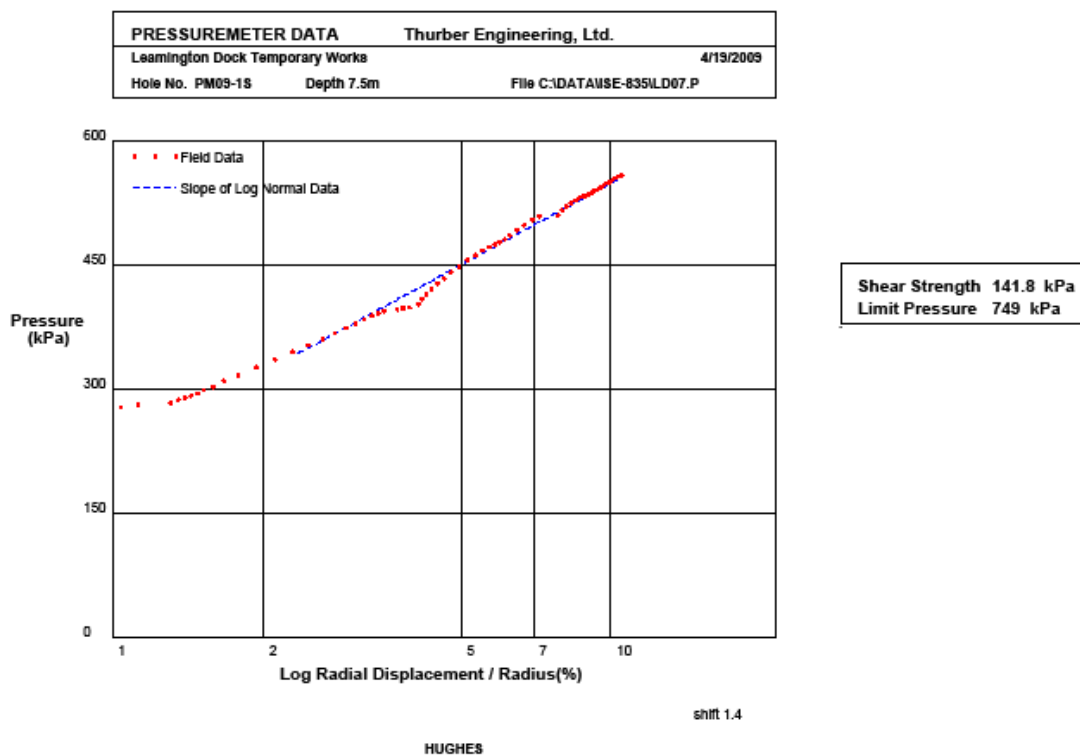


Fig.5. Limit Pressure determination for Test LD07

Table 2A Limit Pressure, Shear Modulus and Shear Strength (log method): PM09-1C

Hole	Test	Depth* (m)	Limit Pressure (kPa)	Unload-Reload Shear Modulus (kPa)	Shear Strength (log method) (kPa)	Comments
PM09-1C	LD07	7.5	749	29,047	141.8	
PM09-1C	LD08	8.25	545	10,357	98.4	
PM09-1C	LD09	8.8	684	17,115	131.8	
PM09-1C	LD10	10.5	646	12,857	102.8	
PM09-1C	LD11	11.25	613	8125	95	
PM09-1C	LD12	12	595	6857	79.7	
PM09-1C	LD13	12.75	603	6450	78.2	
PM09-1C	LD14	15.3	>1,500	272,982	>450	
PM09-1C	LD15	14.3	na	na	na	Ruptured membrane

***Test depths are measured from the deck of the dock surface**

Table 2B Pressure, Shear Modulus and Shear Strength (log method): PM09-2S

Hole	Test	Depth* (m)	Limit Pressure (kPa)	Unload-Reload Shear Modulus (kPa)	Shear Strength (log method) (kPa)	Comments
PM09-2S	LD01	10.5	253	1389	31.3	disturbance
PM09-2S	LD02	11.25	407	3047	77.6	
PM09-2S	LD03	12	439	2508	80.8	
PM09-2S	LD04	12.75	464	2693	80.8	
PM09-2S	LD05	14.7	na	na	na	disturbance
PM09-2S	LD06	14	>1,500	123,666	>450	

***Test depths are measured from the deck of the overwater platform**

2.5 DETERMINATION OF STRENGTH PROPERTIES

The PMT data can be used directly to determine the *in-situ* shear strength of cohesive material. To do so, a material model and failure mechanism must be assumed. If it is assumed that the material behaves in an ideal manner, in that the clay remains at constant volume throughout the test, i.e. it does not consolidate, the pressuremeter curve can be interpreted by simple analytical means. The slope of the plot of pressure against the log of the strain can be used to give a direct measure of the shear strength, as discussed in Section 7. Unfortunately, real materials do not quite behave in this manner, and the shear strength determined by this method may not be accurate, particularly in disturbed material, materials which degrade or partial tests in an enlarged hole. However, it does form a basis of rating all materials.

A more realistic method of determining the shear strength in clays is to compare the field PMT data with an ideal model pressuremeter curve based on an assumed set of material parameters. If, for instance, the material is assumed to be cohesive and fails at a constant shear strength and at constant volume, then the material parameters required for this model are the shear strength, lateral stress, and shear modulus. Adjustments can be made to those three parameters until a mathematical curve can be fitted to the field data. Judgment is required to adjust these three parameters to determine the best fit to the data, particularly if there is disturbance present.

It is important to recognize that while the set of parameters may match the field data, no one parameter is necessarily more accurately defined than any other. The better the definition of the field data – that is, the less disturbance in the test data – the more accurately the data can be analyzed.

For Test LD11, the shear strength determined by this method is shown in Fig. 6.

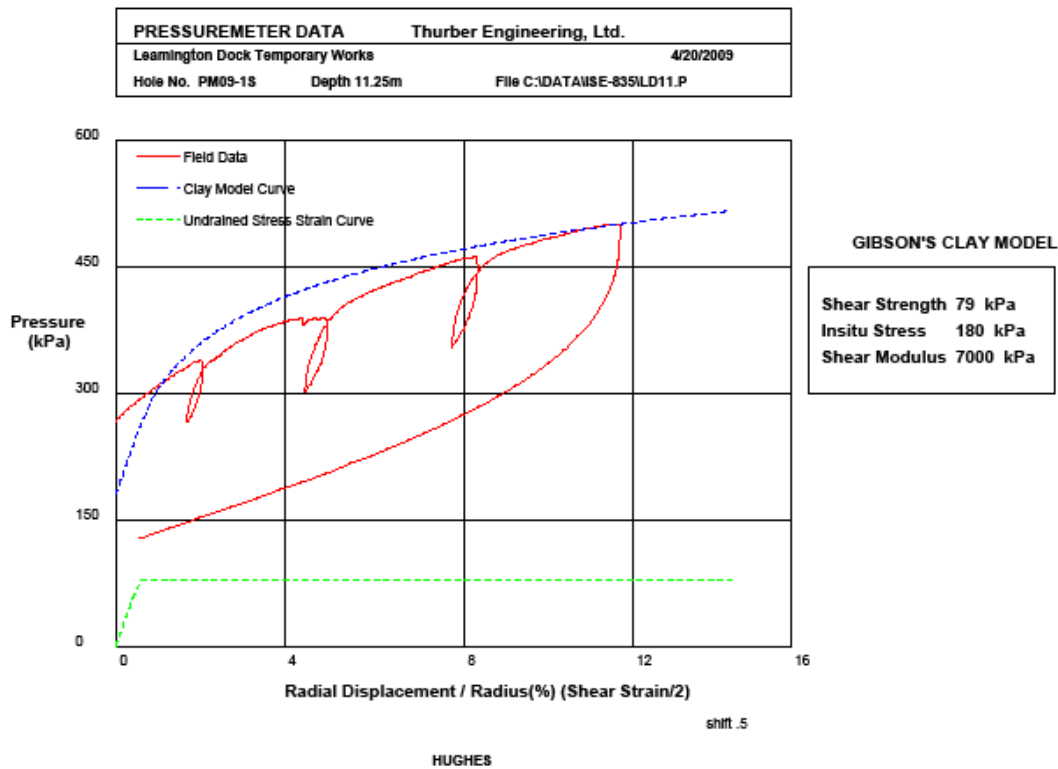


Fig.6. Simple clay model analysis for test LD11

The shear strength determined by the above modeling method gives a shear strength of 79 kPa compared with 95 kPa from the log method.

Analysis of the material strength of clays during unloading is as follows. During the final complete unloading the material surrounding the pressuremeter will fail inwards as the pressure reduces – that is, in an analogous manner to the material surrounding a tunnel or shaft which can collapse inwards if there is not sufficient support. Unlike the tunnel or shaft problem, the stress path followed in the final unloading of the pressuremeter is a little more complex. Fig. 7 shows the ideal stress path followed in an undrained cohesive material during loading and unloading. The effective stress path goes from A to B on loading until it intercepts the failure stress state. The effective stress state remains at B even though the total radial stress increases. On unloading, the effective stress path reverses from B to A, and then crosses over to intersect the failure condition in which the circumferential stress is the major principle stress (the tunnel and shaft problem). The total stress path goes from A' to B' to C'. As it goes from B' to C' the pore pressure rises. On unloading from C' the total stress path follows from C' to D' as the circumferential stress becomes the major principal stress. The ideal pressuremeter curve is shown on the right in Fig. 7. In crossing from C' to D' the shear stress change in ideal materials is twice the loading shear stress. The stress-strain curves for an ideal linear material and a non-linear degrading material are sketched in Fig. 8.

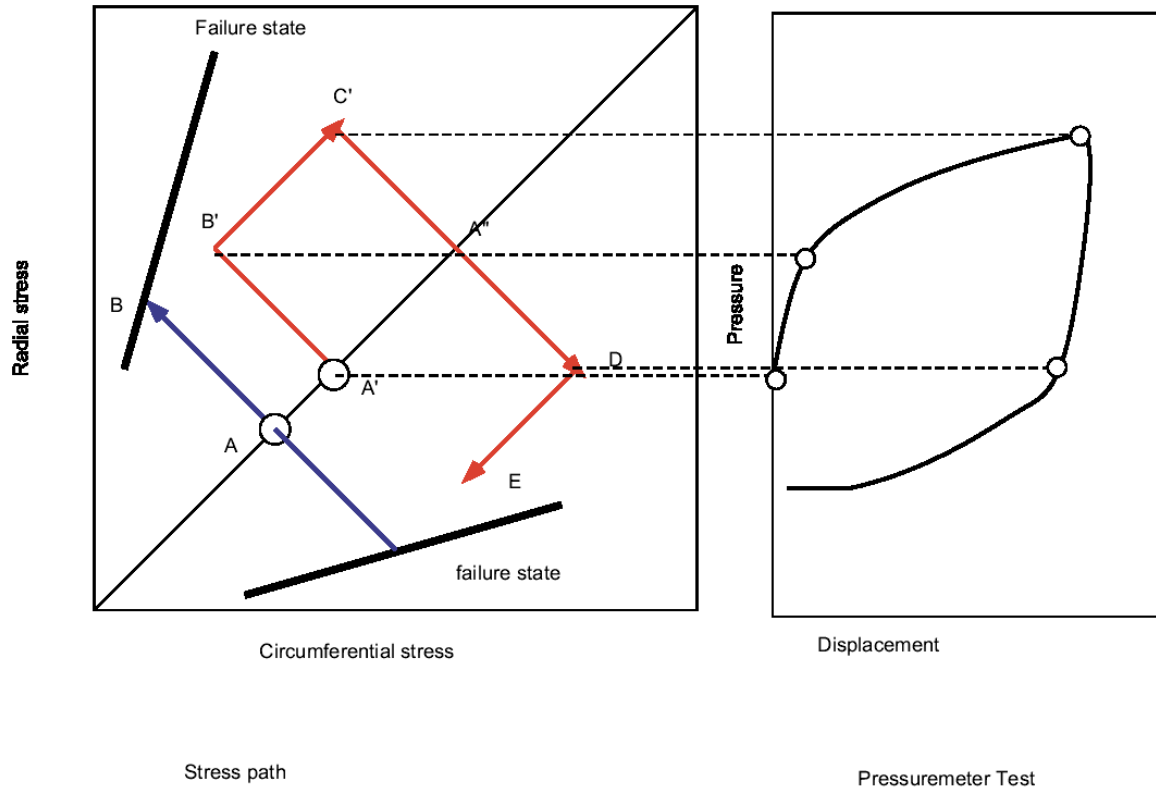


Fig.7. The ideal stress path followed during loading and unloading in a pressuremeter test

Therefore the final unloading curve can be analyzed to determine the shear strength by assuming that the loading shear strength is one half of the unloading shear strength. As a simplification the unloading curve can be analyzed assuming that the material behaves in a linear manner. For convenience with the existing computer programs, the final unloading curve is inverted with the maximum strain and pressure placed at the origin as shown in Fig. 9.

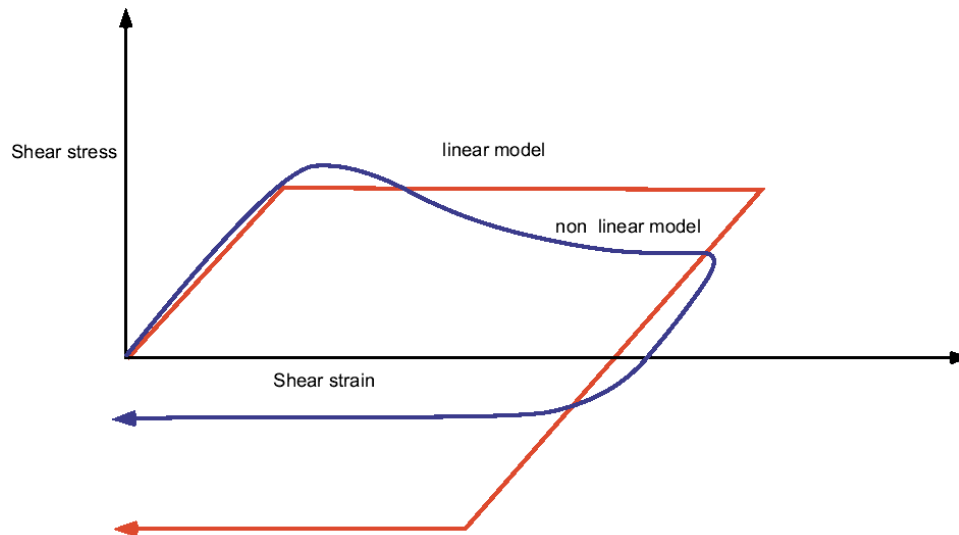


Fig.8. The ideal stress strain curves followed in loading and unloading

To use the available computer programs the final unloading curve is inverted, with the peak stress placed at the origin as shown in Fig. 9. The shear strength determined from the unloading curve is 79 kPa. Hence the loading shear strength, derived from the unloading curve, is 39.5 kPa. In view of the likely degradation of shear stress during the reversing of the shear direction, on unloading, the shear strength derived from the unloading curve is likely to be a conservative estimate.

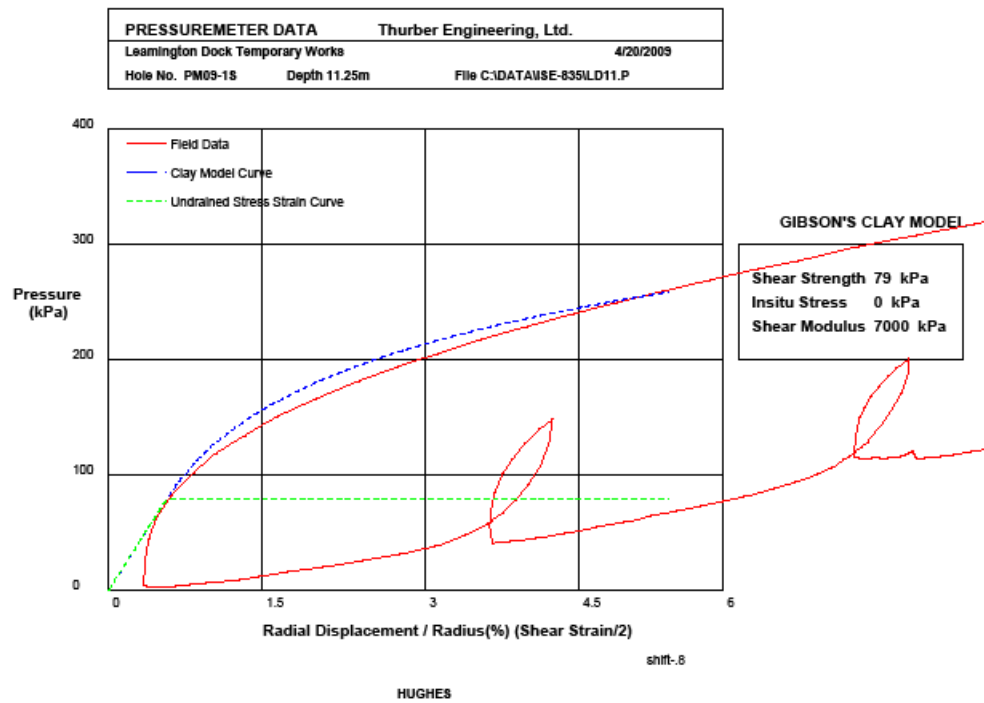


Fig.9. Final unloading curve shear strength analysis.

A similar modeling method can be used with frictional materials. If the material has an assumed friction angle, lateral stress and modulus then the parameters can be adjusted to match the field data. Fig.10 is an example of this method applied to test LD07. The simple model used assumes that there is no cohesion. The parameters given in Tables 3A and 3B summarize the shear strength and friction angles determined by the modeling approach, assuming a simple friction model with no cohesion and a cohesive model with no friction.

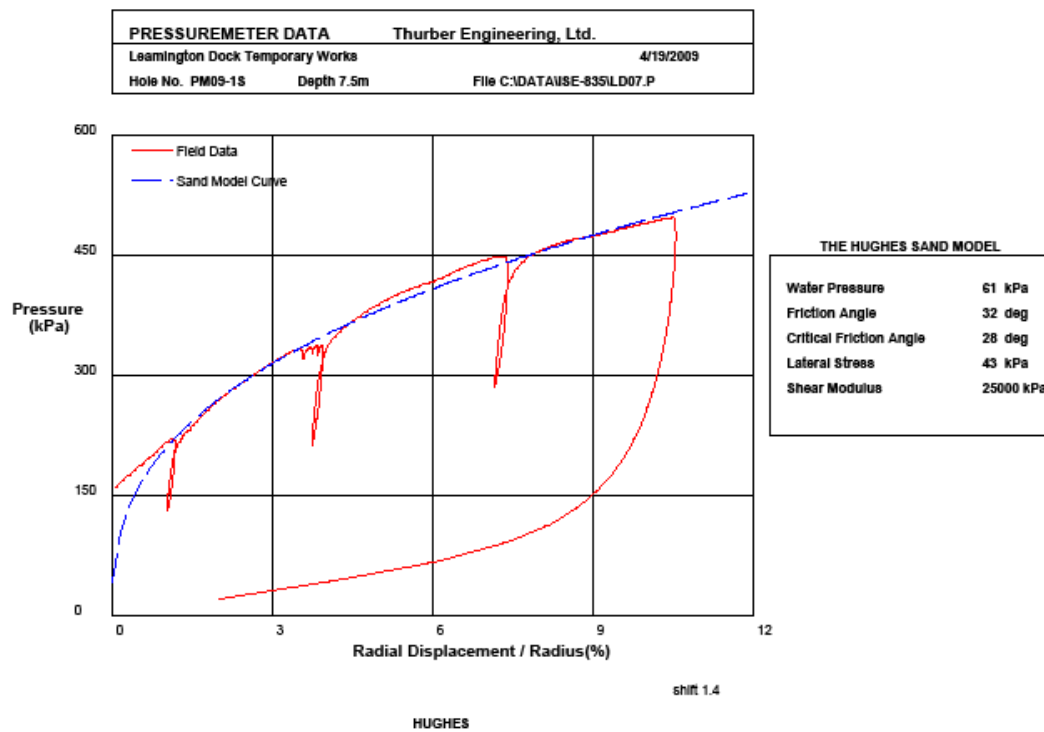


Fig.10. Frictional Model for Test LD07

Table 3A Material Properties from Model Analysis: PM09-1C

Hole	Test	Depth* (m)	Shear Strength (model) (kPa)	Shear Strength (unloading model) (kPa)	Friction Angle (model)
PM09-1C	LD07	7.5	-	-	32
PM09-1C	LD08	8.25	-	-	30
PM09-1C	LD09	8.8	-	-	30
PM09-1C	LD10	10.5	80	$82/2 = 41$	29
PM09-1C	LD11	11.25	79	$79/2 = 39.5$	29
PM09-1C	LD12	12	73	$75/2 = 37.5$	28
PM09-1C	LD13	12.75	74	$75/2 = 37.5$	28
PM09-1C	LD14	15.3	-	-	38
PM09-1C	LD15	14.3	na	na	na

***Test depths are measured from the deck of the dock surface**

Table 3B Material Properties from Model Analysis: PM09-2S

Hole	Test	Depth (m)	Shear Strength (model) (kPa)	Shear Strength (unloading model) (kPa)	Friction Angle (model)
PM09-2S	LD01	10.5	-	-	30
PM09-2S	LD02	11.25	-	-	32
PM09-2S	LD03	12	-	-	30
PM09-2S	LD04	12.75	-	-	29
PM09-2S	LD05	14.7	na	na	na
PM09-2S	LD06	14	-	-	38

***Test depths are measured from the deck of the overwater platform**

3.0 CONE PENETROMETER TESTING

Three Cone Penetrometer Tests (CPTs) were conducted to depths of approximately 12-13m below the deck or dock surface, with dissipation testing and shear wave data collection at various intervals in each of the nine locations. The cone was pushed by Downing Drilling using a CME-75 drill rig.

CPT09-1S was advanced through the dock surface. CPT09-2N and CPT09-3N were performed using the overwater platform to test the soil adjacent to the dock structure. Dissipation testing was performed at selected intervals; every 0.5m in CPT09-01N and every meter in CPT09-2N and CPT09-3N until cohesive soil was reached. One dissipation test was performed in cohesive soil at each of the three locations. At each location shear wave data was also collected at 0.5m intervals.

At the location of CPT09-1S, rock fill was drilled through and N casing was advanced. BQ casing was then set inside the N casing in order to provide lateral support for the cone rods. N casing was lowered to the mudline for CPT09-2N and CPT09-3N. BQ casing was then lowered down inside the N casing to provide additional lateral strength before advancing the cone.

A friction reducer was used to reduce the sidewall friction of the rods to optimize the ease of penetration and withdrawal. The CPT was advanced until refusal in all of the test locations.

3.1 INSTRUMENTATION AND EQUIPMENT

The drill rig rotary head was used to advance standard one meter CPT rods with a 10cm² cross sectional area. The axial load capacity of the rods is 20 tons. The rods were stored in a rack in the bed of In Situ Engineering's pickup truck and were pre-strung with a 75 meter data cable through 20 meters of testing rods. The cable sent electronic signal data from the instrument to a junction box at the surface, which in turn communicated with an attached laptop computer.

The CPT instrument which was used is a 10 ton digital subtraction type piezocone manufactured by the Vertek Company of Randolph, Vermont. The instrument measures tip resistance (0-100,000 kPa), sleeve friction (0 – 1,000 kPa), pore pressure (0 – 3500 kPa), inclinometer (0 -15 degrees) and shear waveforms (0 – 800 ms).

Readings were taken at 50 mm intervals of the tip pressure, friction and pore pressure as the rods were advanced using a roller depth counter which recorded the depth as rods were advanced. The data was immediately stored on the hard drive of the computer at every reading interval.

3.2 DISSIPATION TESTING

The dissipation testing was performed in all of the test locations to determine drainage conditions, distribution of hydraulic head and consolidation properties. Numerous tests were performed in all the explorations. The dissipation testing consists of two types of tests: 1) Dissipation tests which are when elevated pore pressures are encountered and the test consists of recording the pore pressure decay over time. These tests are helpful in fine grained soils to assess hydraulic flow characteristics. 2) Equalization tests conducted in a more frictional material to determine the hydrostatic head at a point. These tests could consist of either a rise in pore pressure or a drop in pore pressure with the test usually continuing until either equilibrium or near equilibrium condition is achieved. The pressure readings in both types of tests are recorded at times which increase on a logarithmic scale. The pore pressure was measured through the use of a porous stone and pore pressure transducer located immediately behind the tip.

3.3 SHEAR WAVE TESTING

Shear (S) wave data was collected at each of the 3 locations at .5m intervals. Shear wave data can determine the velocity at which a shear wave will travel through the given soil. It can be useful in determining stiffness properties and seismic effects, such as liquefaction of granular materials. Generally in terrestrial applications such as CPT09-1S, data is collected by placing a steel beam adjacent to the cone being advanced into the soil. At the desired intervals a sledge hammer is used to strike the beam sending a shear wave into the soil. The cone penetrometer detects the vibration through use of a geophone and reports the information to the computer at the surface. A second strike on the opposite end of the beam ideally produces a reversal of the shear waveform. Comparison of the two waveforms helps identify when the shear wave arrives at the instrument and helps eliminate spurious readings such as compression (P) waves. A crossing point between the two waves can then be used as a reference point to determine the velocity of the wave is traveling through the soil.

At the location of CPT09-1S a beam was placed under the hydraulic jacks of the drill rig. Because CPT09-2N and CPT09-3N were performed on an overwater platform a different method had to be used. The beam was lowered to the mudline and attached to AW rod with a swivel connection. Enough AW rod was added to be able to access the rod at the surface of the platform. The strikes with the sledge hammer were made to the rod and would be alternated back and forth to retrieve the two waves. Due to the limited work space on the overwater platform, the AW rod could not be “laid over” sufficiently to achieve a proper horizontal component. Thus

the results for the overwater work have a huge P wave component that masks and complicates the S wave arrival interpretation.

Both the overwater and terrestrial shear wave testing should be considered of poorer quality than that generally achieved onshore without the influence of fill, caissons, wave noise, and spatial limitations of striking the shear beam.

3.4 CPT DATA PRESENTATION AND ANALYSIS

Plots of the CPT results are presented in APPENDIX II. These plots include columns for uncorrected tip resistance (Q_c), Friction Ratio (F_s/Q_c), Pore Pressure, an interpreted soil behaviour type, and an equivalent Standard Penetration Test (SPT) N value. The interpreted soil behaviour type and equivalent N value are based upon correlations outlined in the Robertson and Campanella reference cited in Section 5.0. The plot is standard output from a software program provided by Vertek, the instrument manufacturer.

An analysis of the friction angle was performed by In Situ Engineering at selected intervals of the sands. The analysis was performed using a Microsoft Excel spreadsheet and correlations provided by the Lunne et. al. reference cited in Section 5.0. Copies of the respective spreadsheet pages are presented in Appendix II.

Table 4 Strength Properties from CPT Correlations

		friction angle	
Test	Depth	method 1	method 2
	(m)	(degrees)	(degrees)
CPT-01	5.5	35	37
CPT-01	7	28	30
CPT-02	11		
CPT-03	9	44	44
CPT-03	9.75	44	44
CPT-03	10.25	46	46
CPT-03	12		

Method 1 refers to a bearing capacity correlation and Method 2 refers to a Q_c verses effective stress correlation.

Shear wave analysis was performed on the wave forms to determine the average velocity for the sand and silty clay layers and determination of a low strain bulk modulus. Because of the poor quality of the wave forms, initial arrival times could not be predicted accurately, however, by examining relative differences between similar points on the waveform, some estimate of the velocity can be achieved. Plots of the waveforms with the points of interest are presented in Appendix II. The table below summarizes the results of the analysis.

Table 5 Shear Wave Velocities and Bulk Modulus from CPT

Test	Depth	Velocity	Modulus	Material
	meters	meters/sec	kPa	
CPT09-1S	5.6 – 9.2	169	62500	Sand
CPT09-1S	9.7 – 12.1	129	34763	Silty Clay
CPT09-2N	9.7 – 13.1	128	34280	Silty Clay
CPT09-3N	8.2 – 10.6	206	92792	Sand
CPT09-3N	11.1 – 13.2	243	122606	Silty Clay

4.0 CONCLUSIONS

In clay with no frictional component which has been tested in an undrained manner, the values for strength obtained from model analysis of the initial curve and the unloading curve should be similar. In our analysis the values are very different and we believe that this is due to a frictional component in the clay and also that some drainage took place during testing which is not reflected in the unloading curve as it occurred at a much faster rate.

The modulus values obtained from CPT shear wave correlations are much higher than that derived from the pressuremeter data. This is probably in large part due to the fact that it is a very low strain measurement system as opposed to the pressuremeter.

Frictional angle correlations of the CPT are much more reliable in materials with higher overburden stress and highly susceptible to minor changes in particle size.

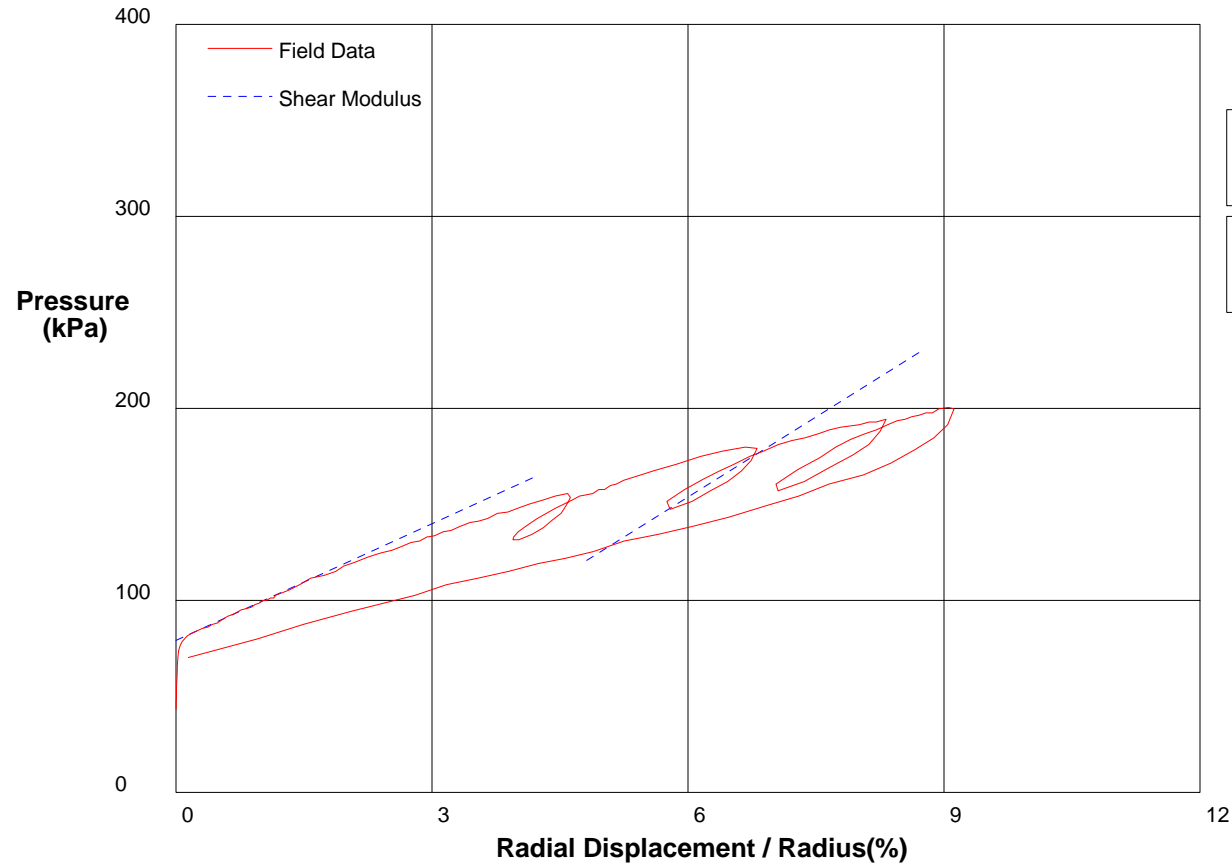
5.0 REFERENCES

- Mair, R.J. and Wood, D.M. 1987. Pressuremeter testing: methods and interpretation. CIRIA Ground Engineering Report. Butterworths, London.
- Riaud, J. and Miran, J., 1992. The Cone Penetrometer Test. FHWA Report #FHWA-SA-91-043.
- Lunne, T. et. al., 1997. Cone Penetrometer Testing in Geotechnical Practice. Spon Press, London.
- Robertson, P.K., and Campanella, R. G., 1983, 1989. Guidelines for Geotechnical Design using the Cone Penetrometer Test and CPT with Pore Pressure Measurement. Hogentogler and Company, Columbia, MD

APPENDIX I- PRESSUREMETER DATA AND ANALYSIS

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 10.5m	File C:\DATA\ISE-835\LD01.P



Shear Modulus 1013 kPa

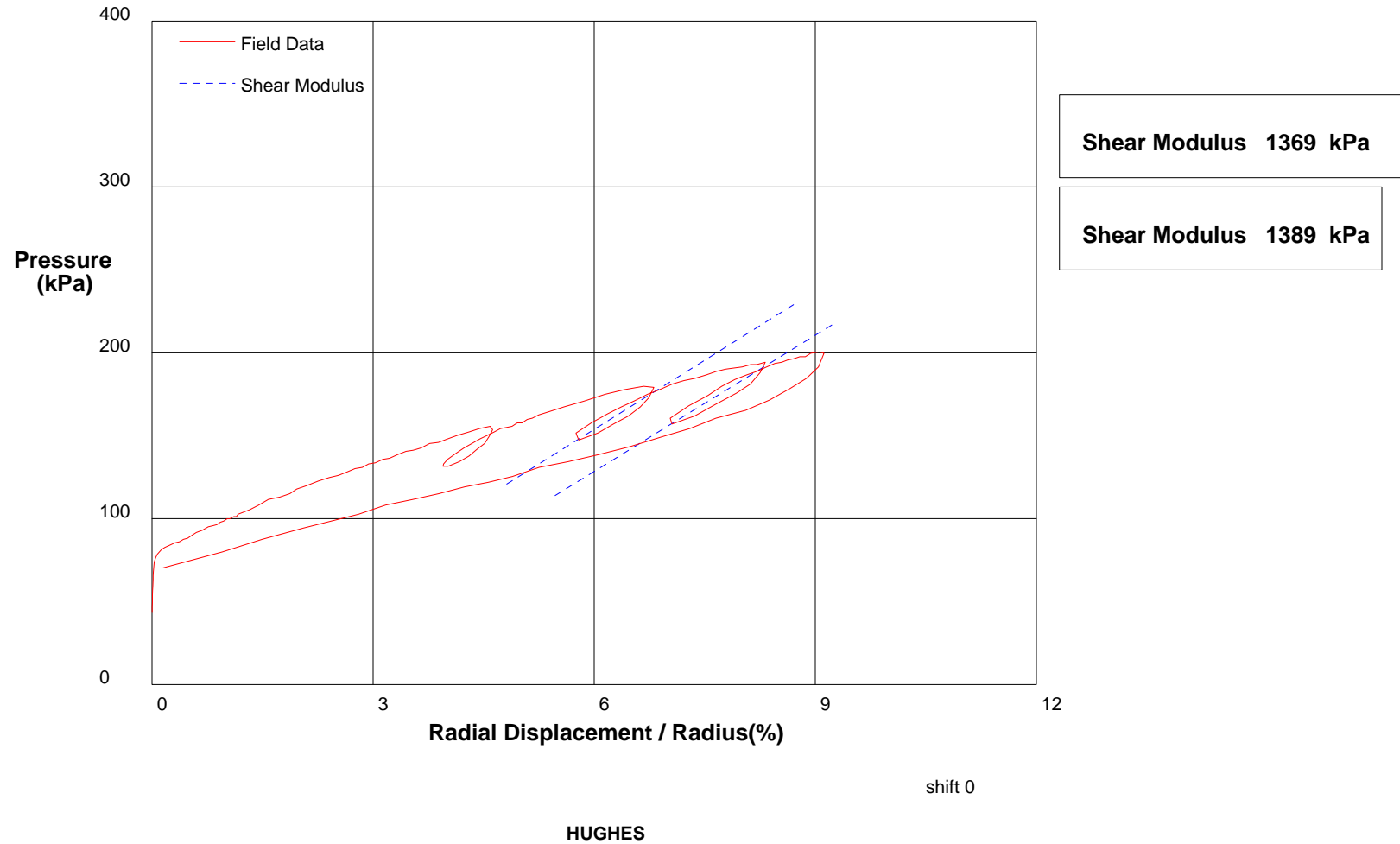
Shear Modulus 1389 kPa

shift 0

HUGHES

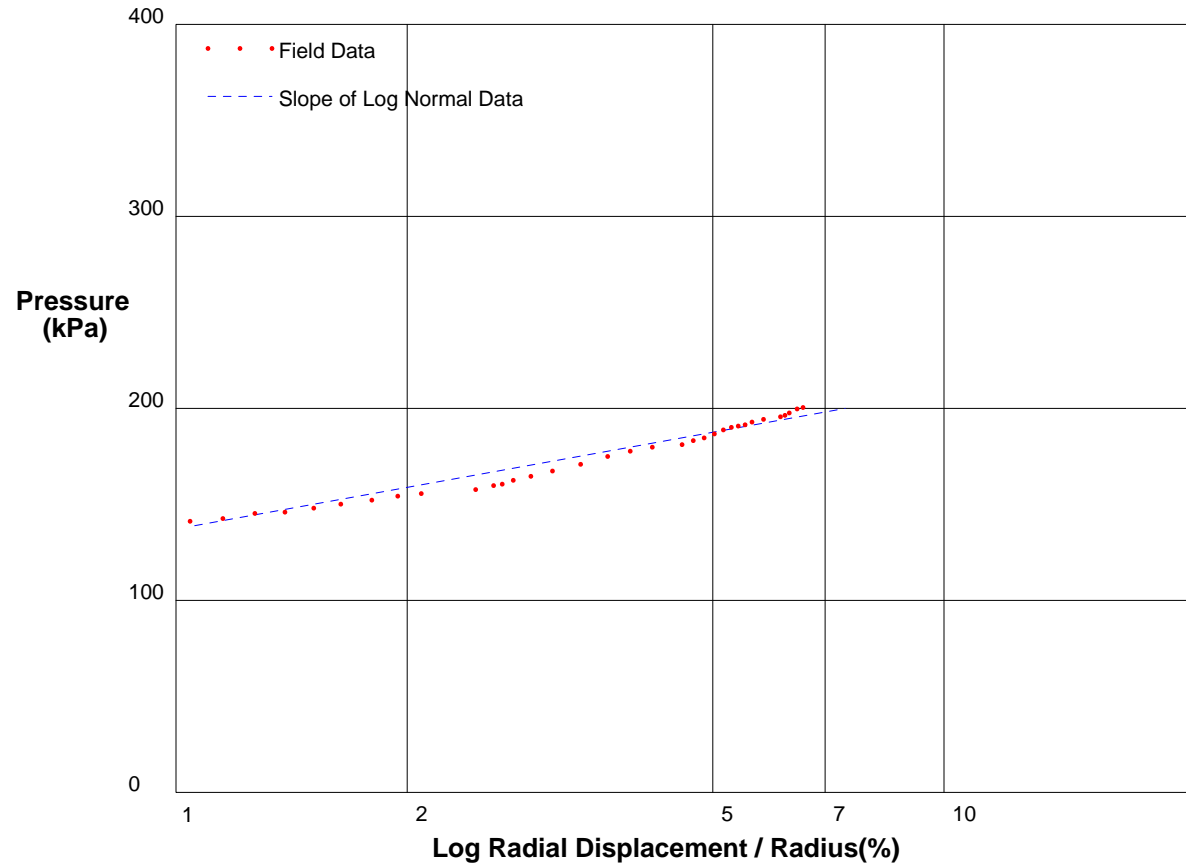
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 10.5m	File C:\DATA\SE-835\LD01.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 10.5m	File C:\DATA\ISE-835\LD01.P



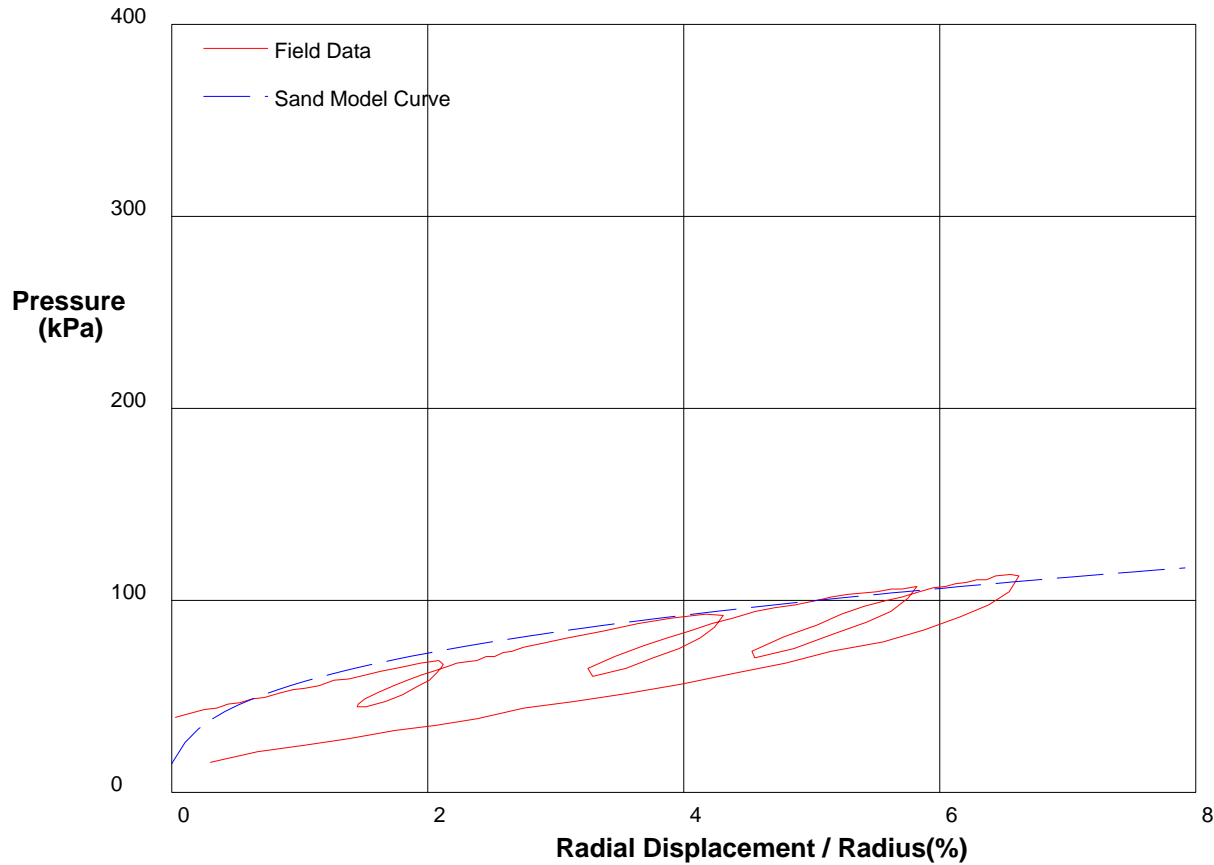
Shear Strength 31.3 kPa
Limit Pressure 253 kPa

shift 2.5

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 10.5m	File C:\DATA\ISE-835\LD01.P



THE HUGHES SAND MODEL

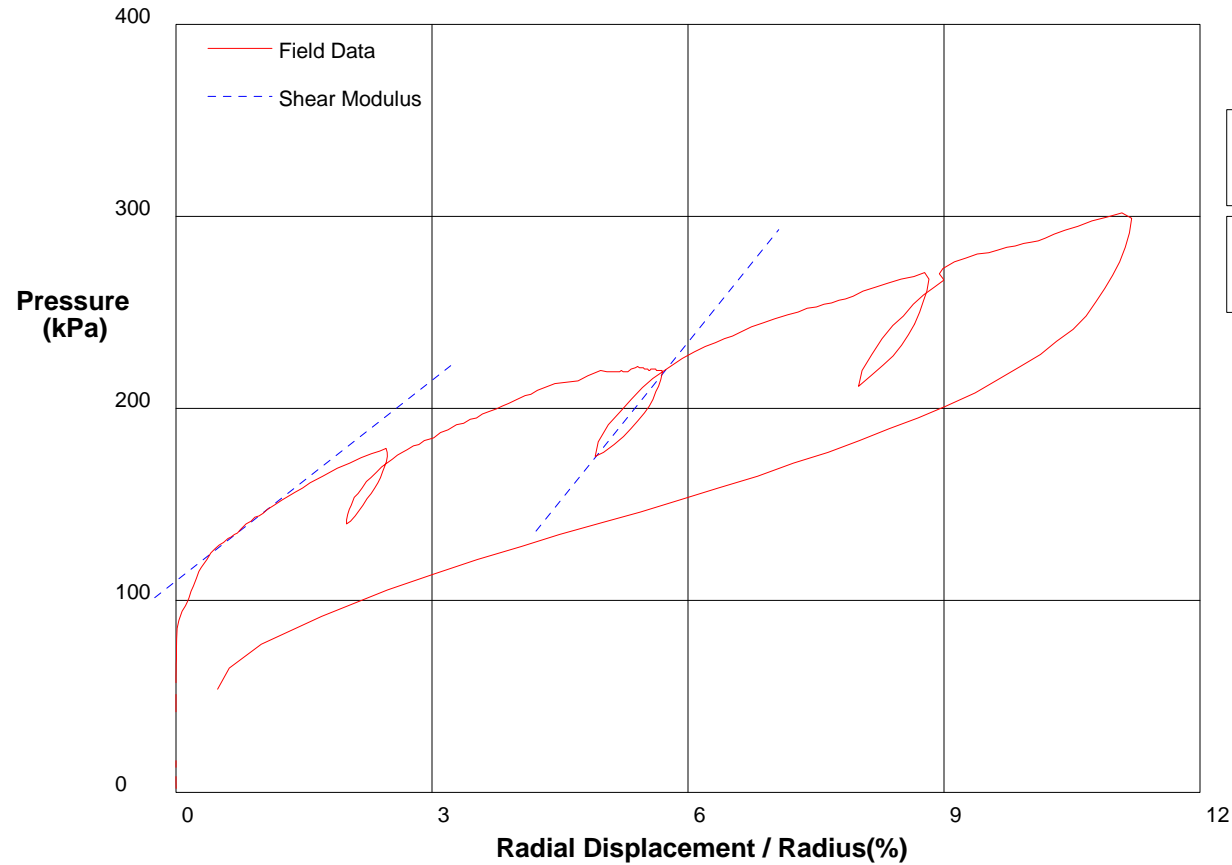
Water Pressure	87 kPa
Friction Angle	30 deg
Critical Friction Angle	28 deg
Lateral Stress	15 kPa
Shear Modulus	5500 kPa

shift 2.5

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 11.25m	File C:\DATA\ISE-835\LD02.P



Shear Modulus 1741 kPa

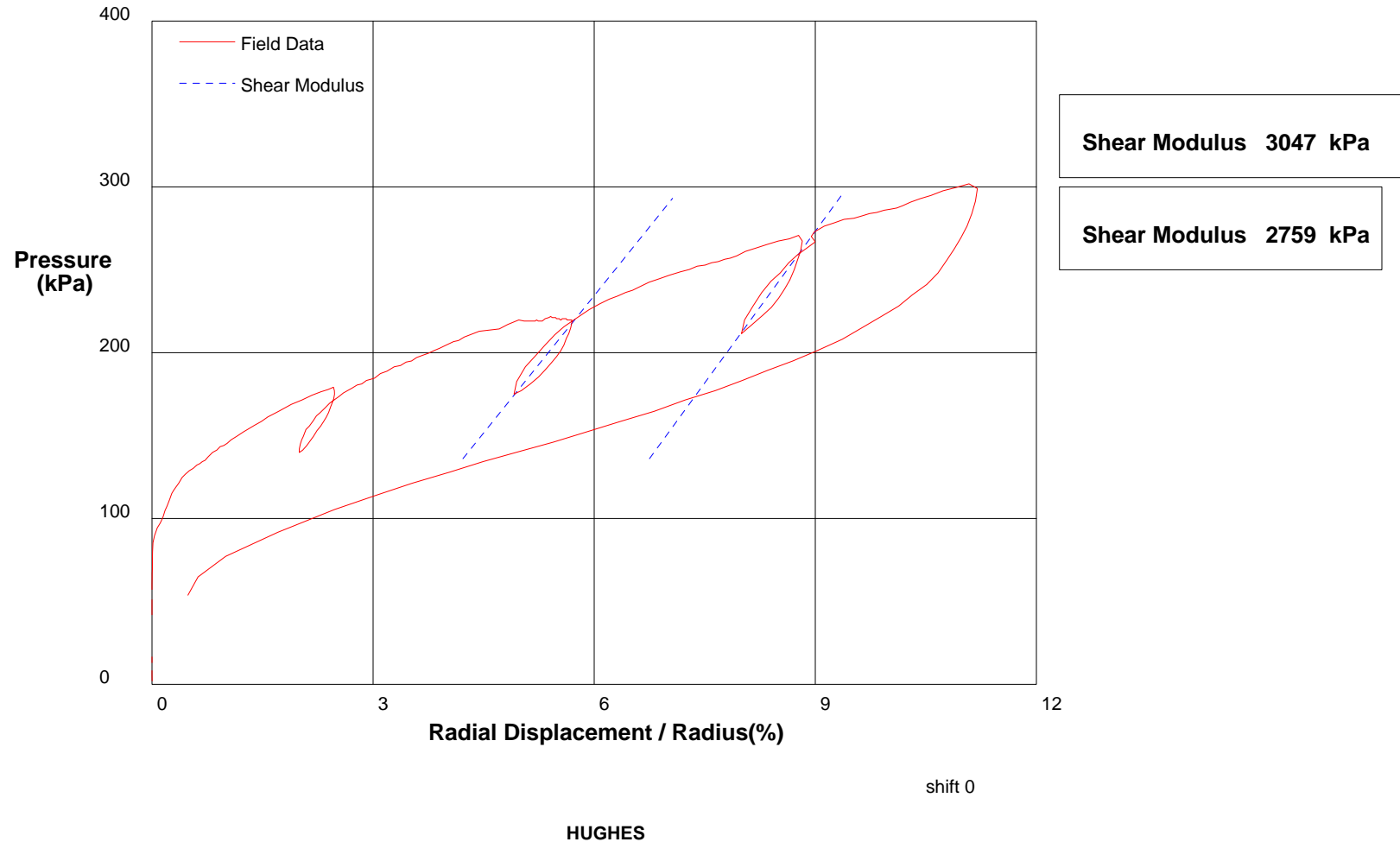
Shear Modulus 2759 kPa

shift 0

HUGHES

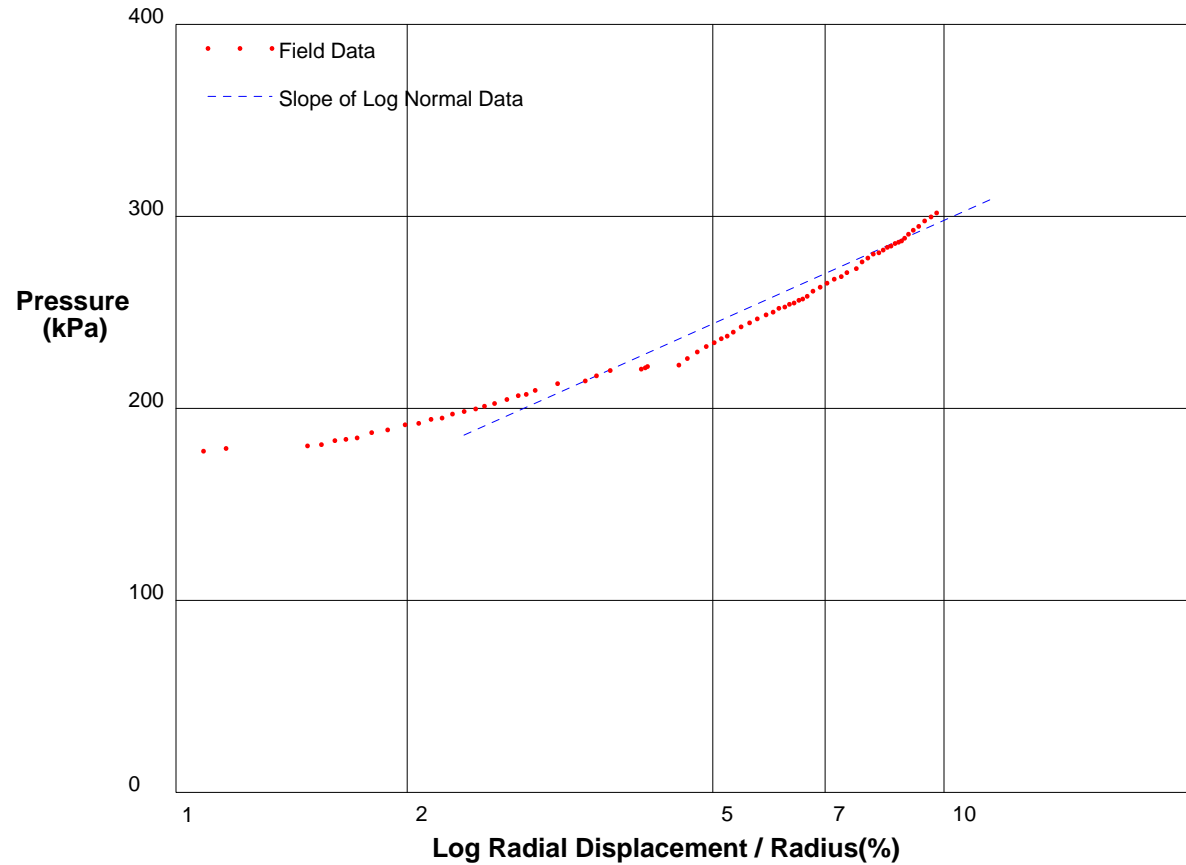
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 11.25m	File C:\DATA\ISE-835\LD02.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 11.25m	File C:\DATA\ISE-835\LD02.P



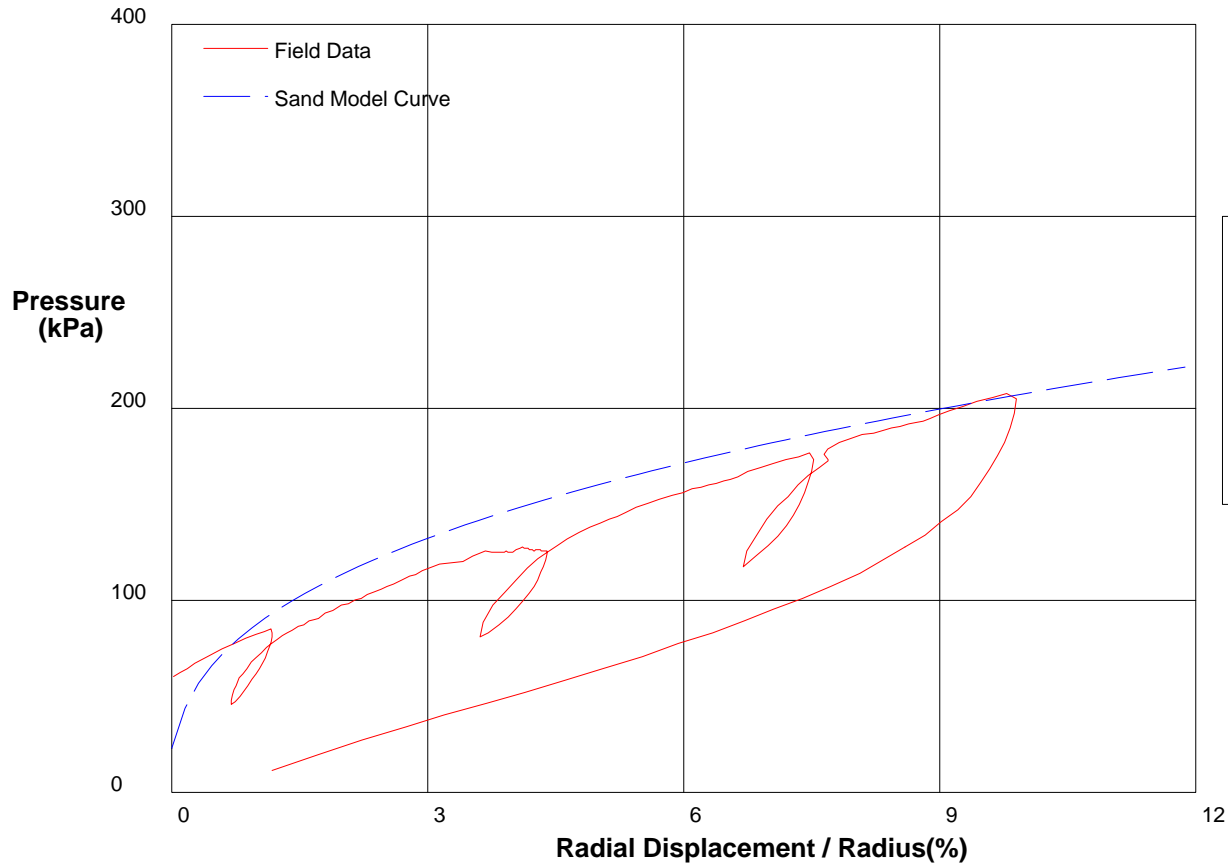
Shear Strength 77.6 kPa
Limit Pressure 407 kPa

shift 1.3

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 11.25m	File C:\DATA\ISE-835\LD02.P



THE HUGHES SAND MODEL

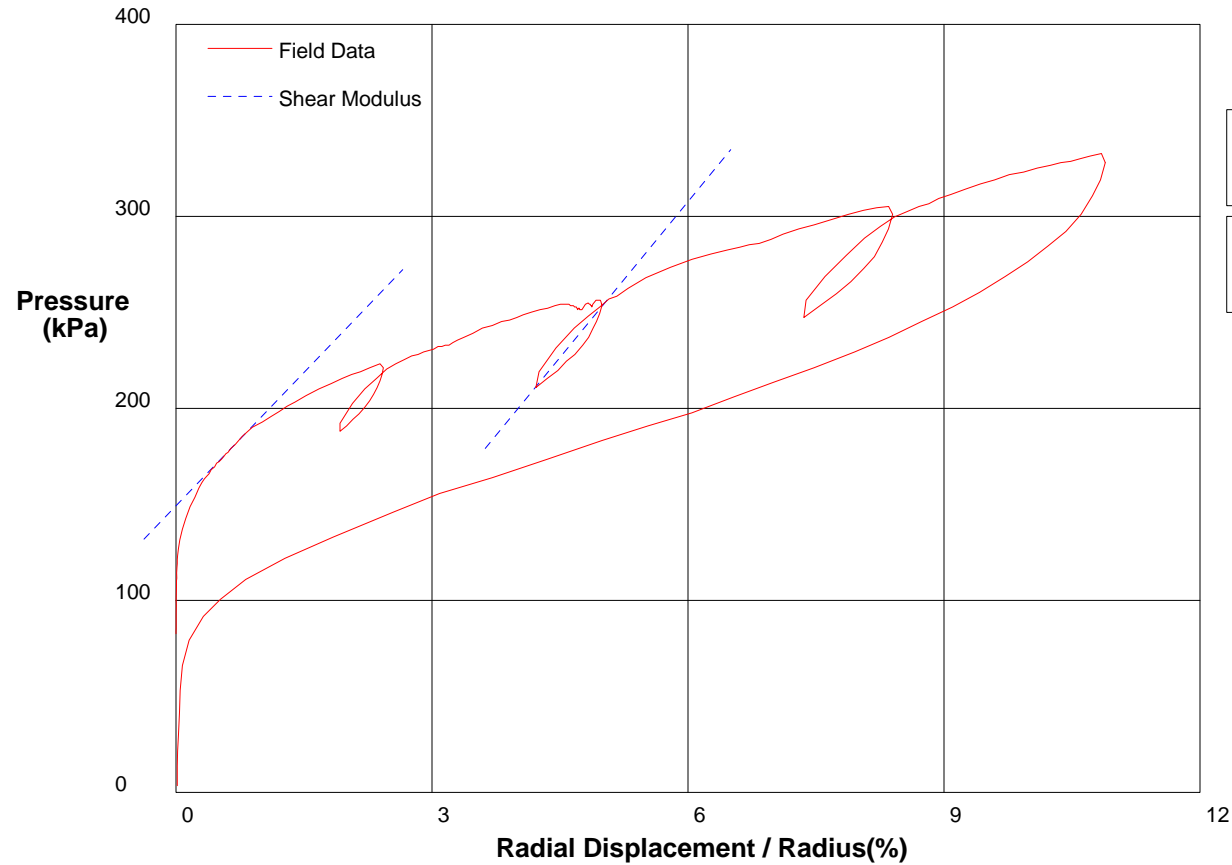
Water Pressure	94 kPa
Friction Angle	32 deg
Critical Friction Angle	28 deg
Lateral Stress	23 kPa
Shear Modulus	7000 kPa

shift 1.3

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12m	File C:\DATA\SE-835\LD03.P



Shear Modulus 2313 kPa

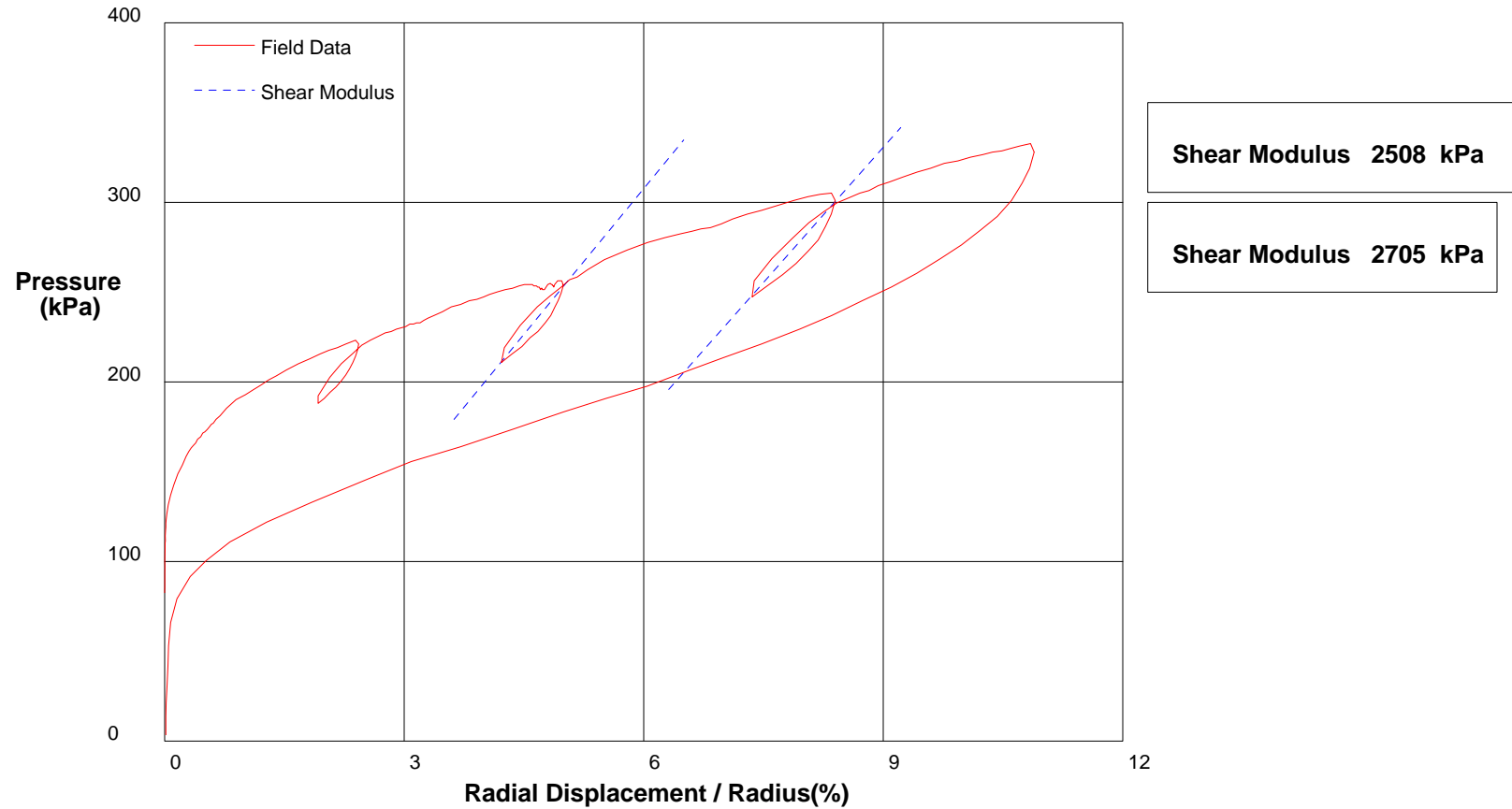
Shear Modulus 2705 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12m	File C:\DATA\SE-835\LD03.P

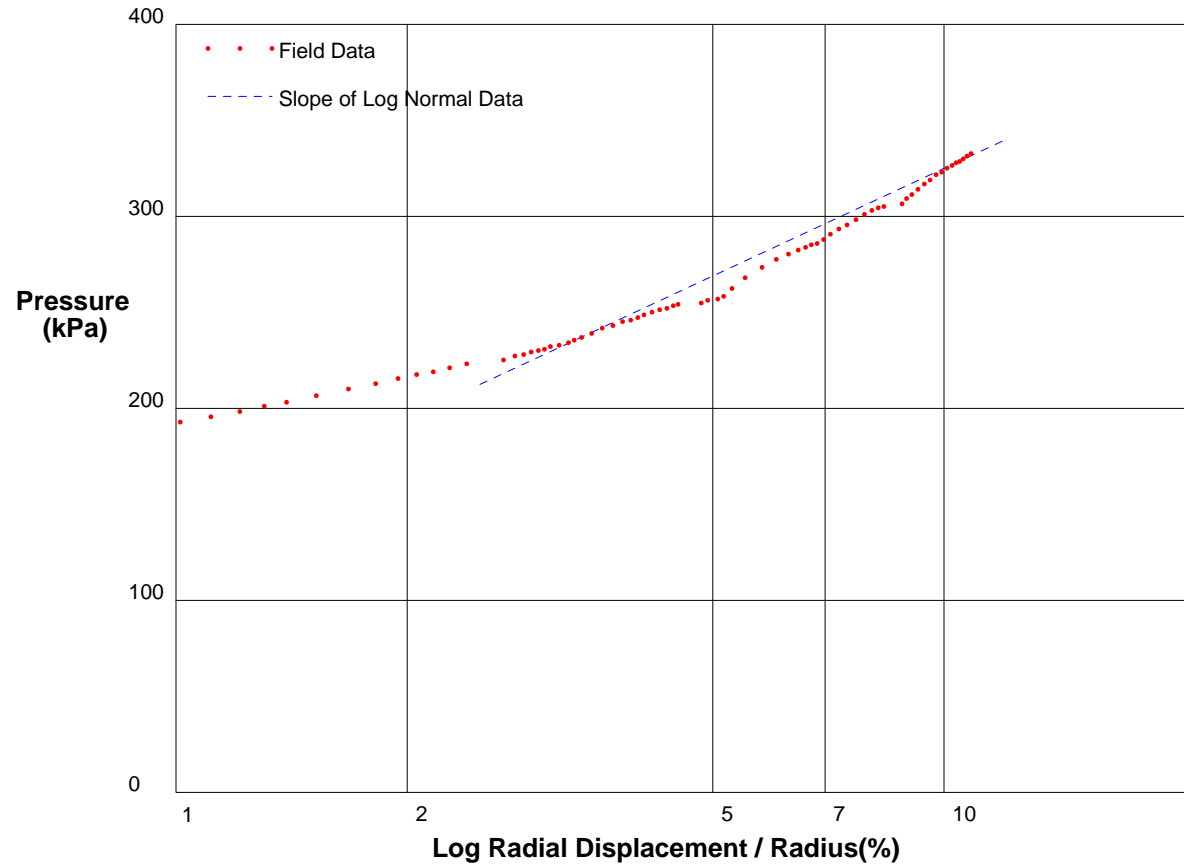


shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12m	File C:\DATA\ISE-835\LD03.P



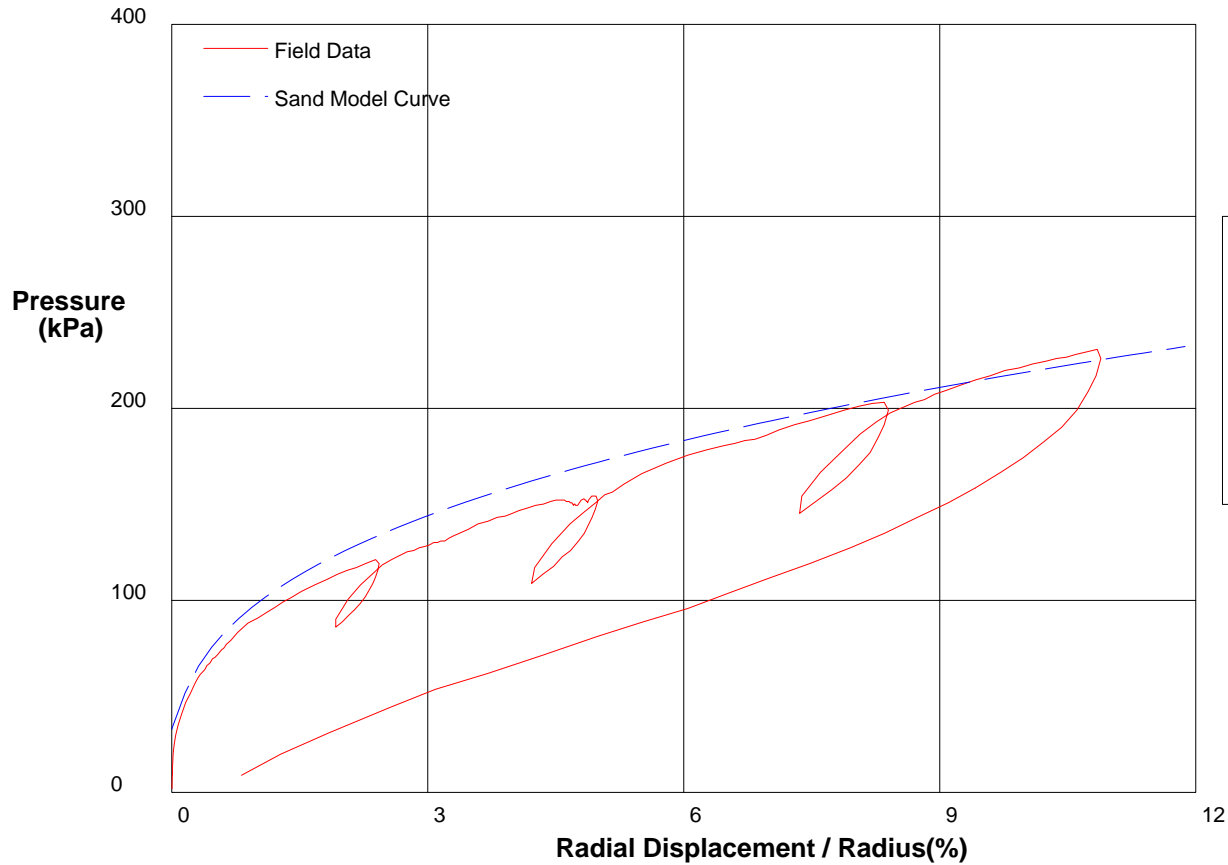
Shear Strength 80.8 kPa
Limit Pressure 439 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12m	File C:\DATA\ISE-835\LD03.P



THE HUGHES SAND MODEL

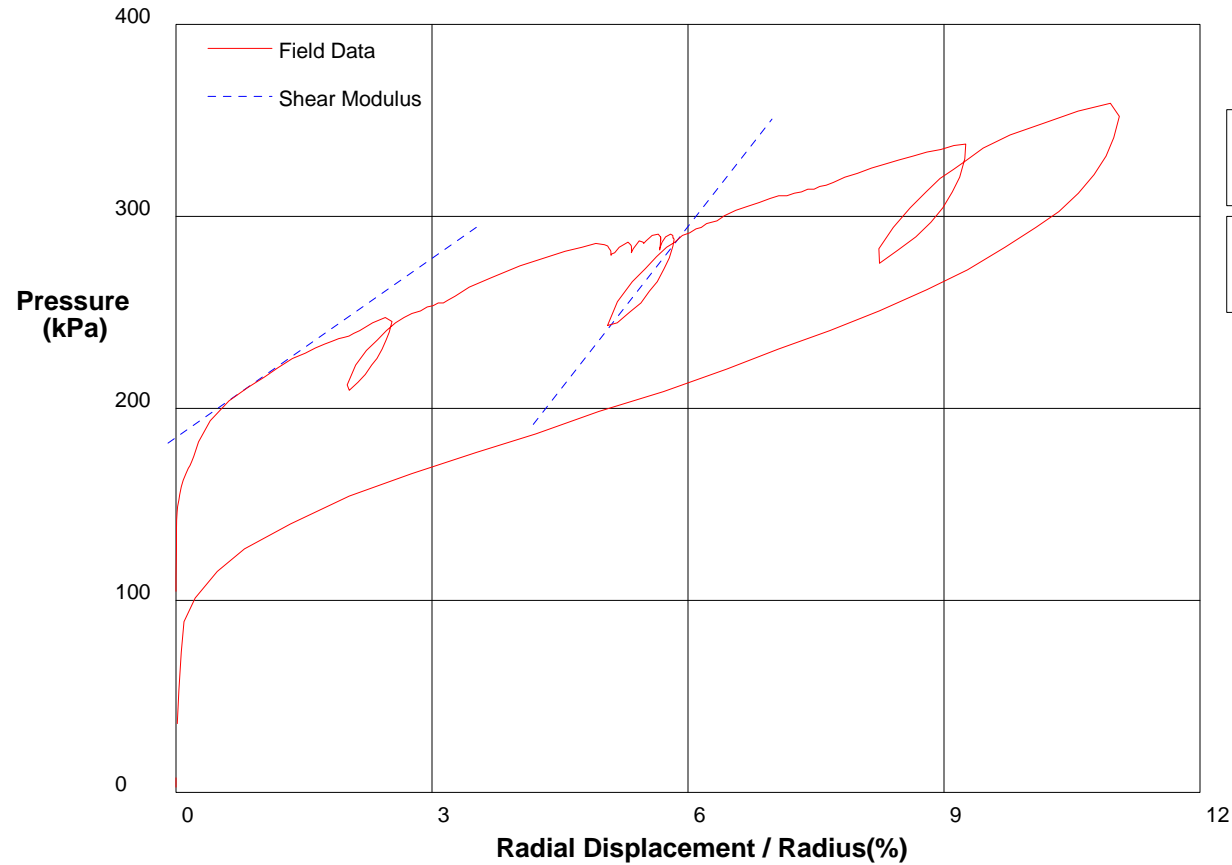
Water Pressure	102 kPa
Friction Angle	30 deg
Critical Friction Angle	28 deg
Lateral Stress	33 kPa
Shear Modulus	6000 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12.75m	File C:\DATA\SE-835\LD04.P



Shear Modulus 1554 kPa

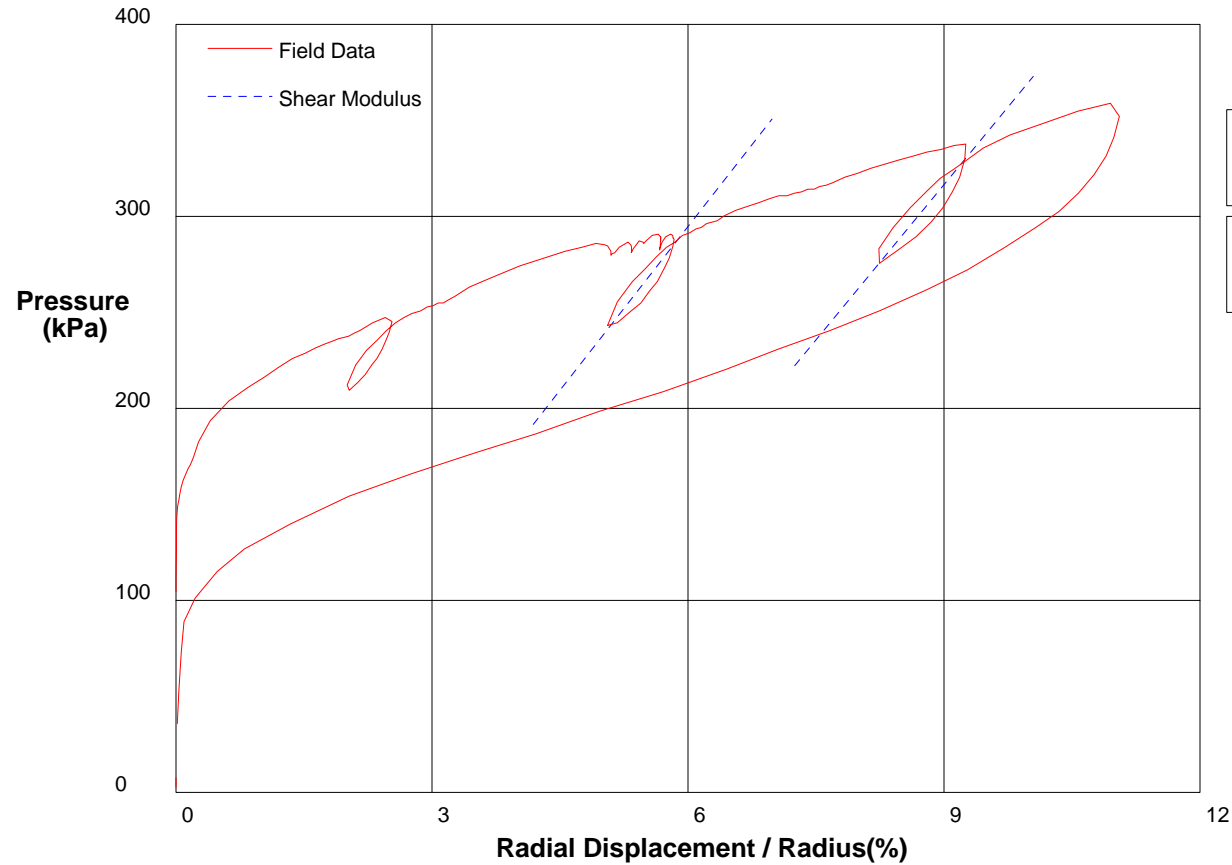
Shear Modulus 2842 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12.75m	File C:\DATA\SE-835\LD04.P



Shear Modulus 2693 kPa

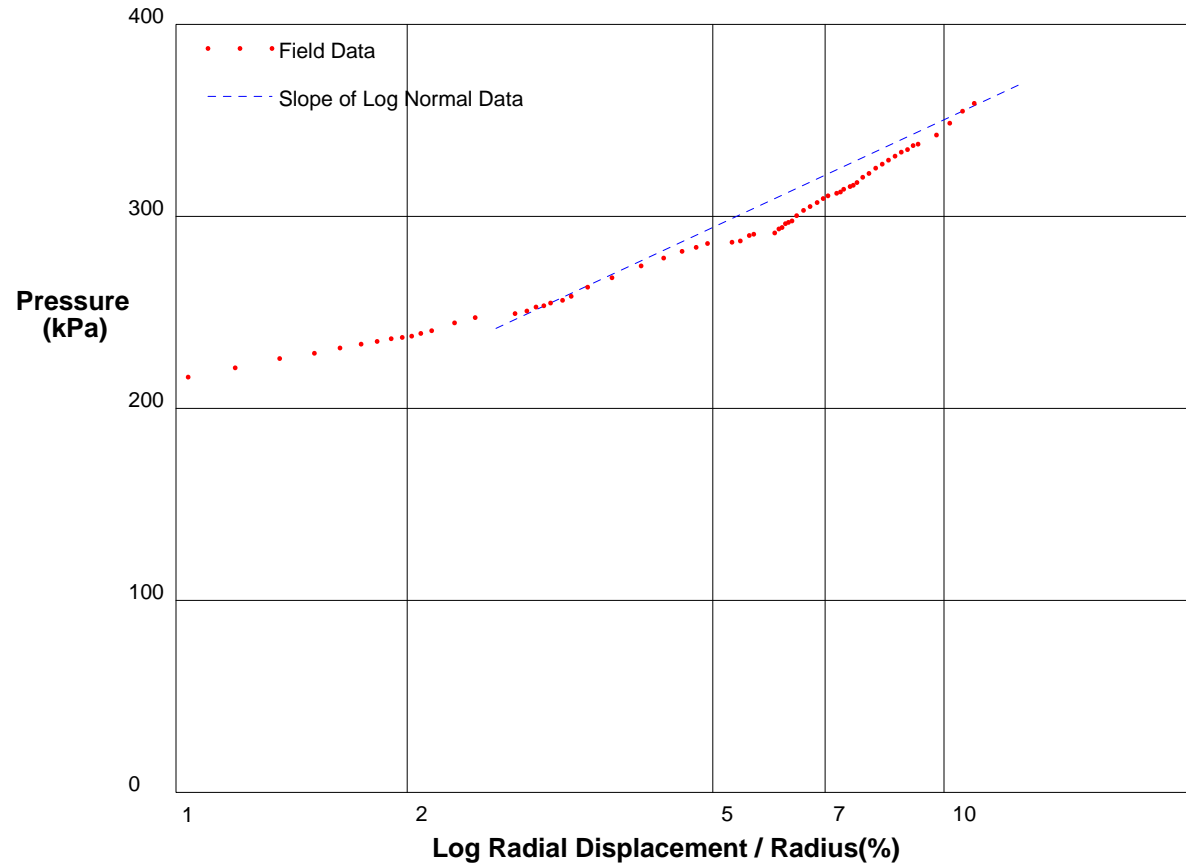
Shear Modulus 2842 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12.75m	File C:\DATA\ISE-835\LD04.P



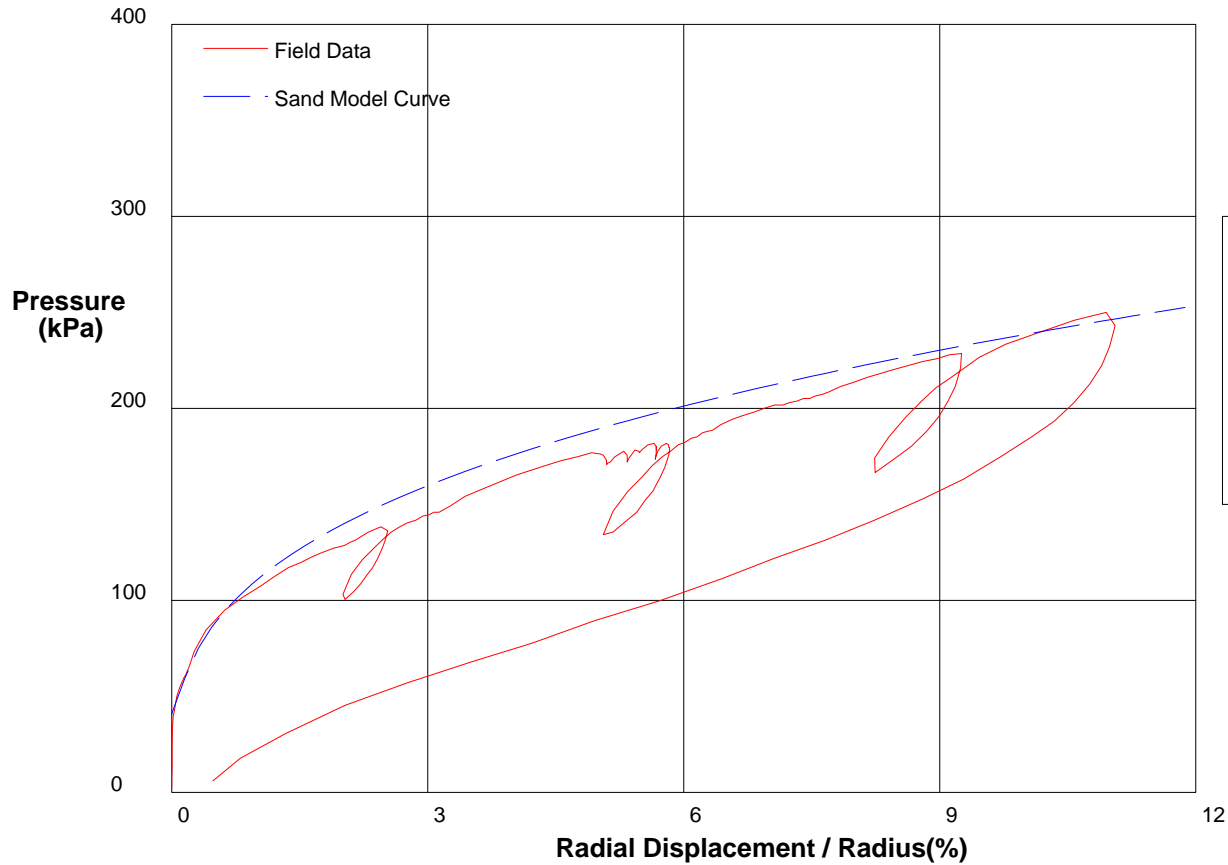
Shear Strength 80.8 kPa
Limit Pressure 464 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 12.75m	File C:\DATA\ISE-835\LD04.P



THE HUGHES SAND MODEL

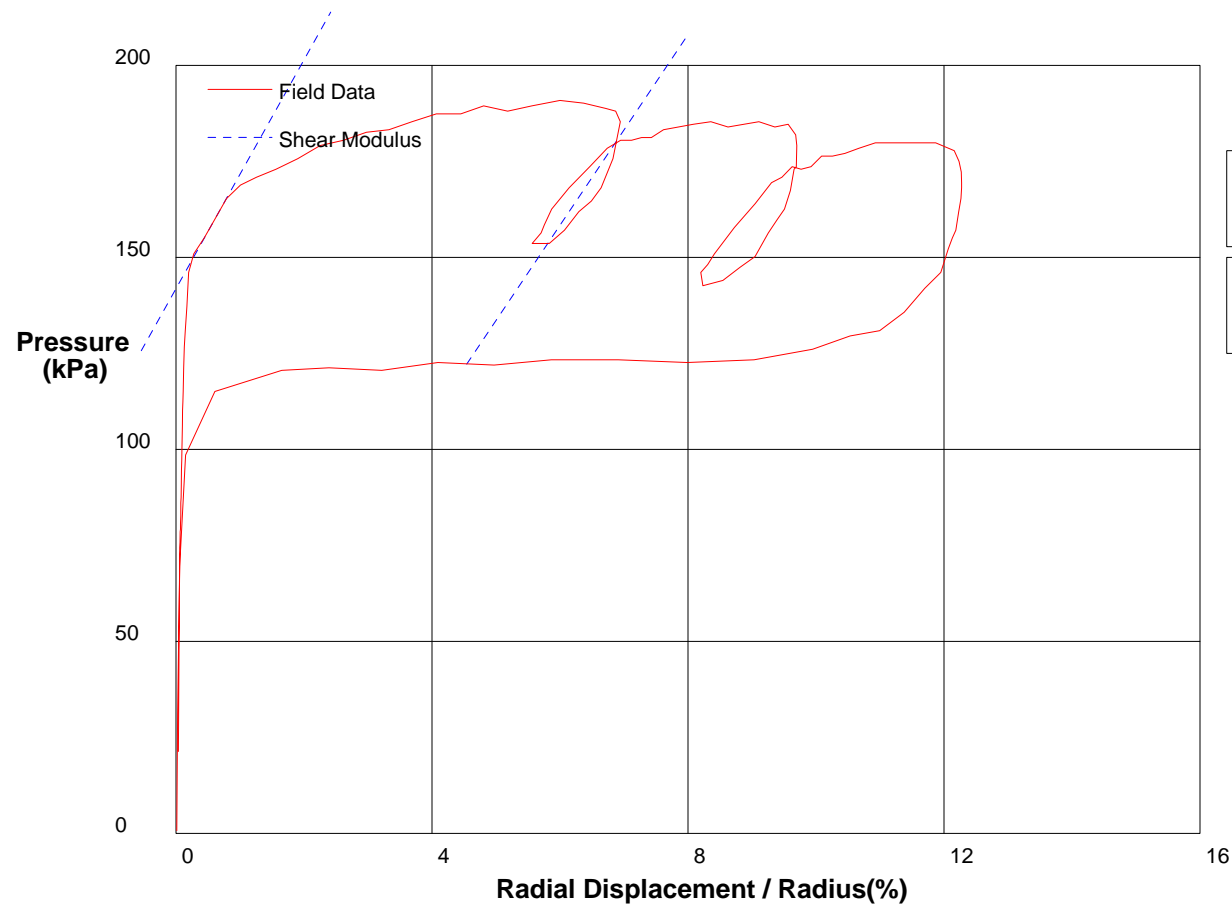
Water Pressure	109 kPa
Friction Angle	29 deg
Critical Friction Angle	28 deg
Lateral Stress	41 kPa
Shear Modulus	6000 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 14.7m	File C:\DATA\SE-835\LD05.P



Shear Modulus 1490 kPa

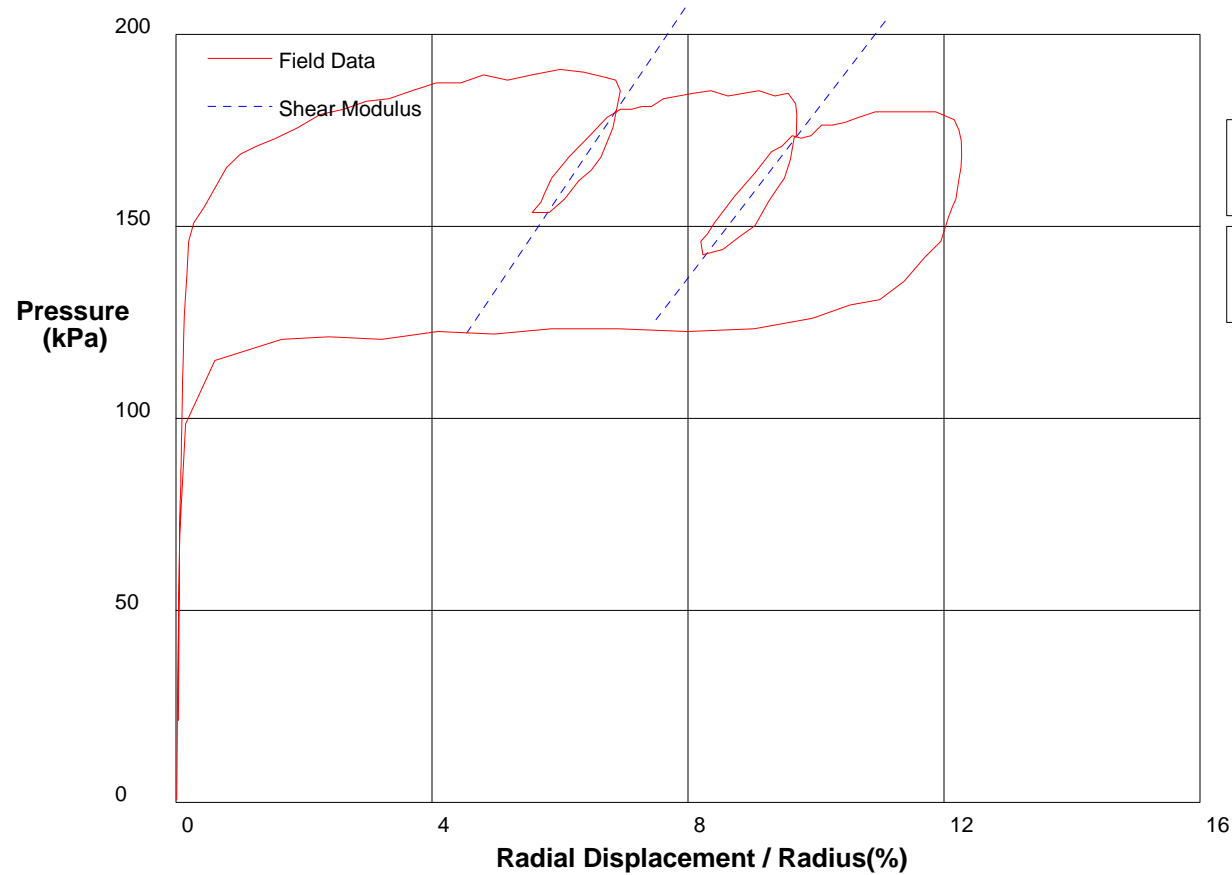
Shear Modulus 1239 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 14.7m	File C:\DATA\SE-835\LD05.P



Shear Modulus 1085 kPa

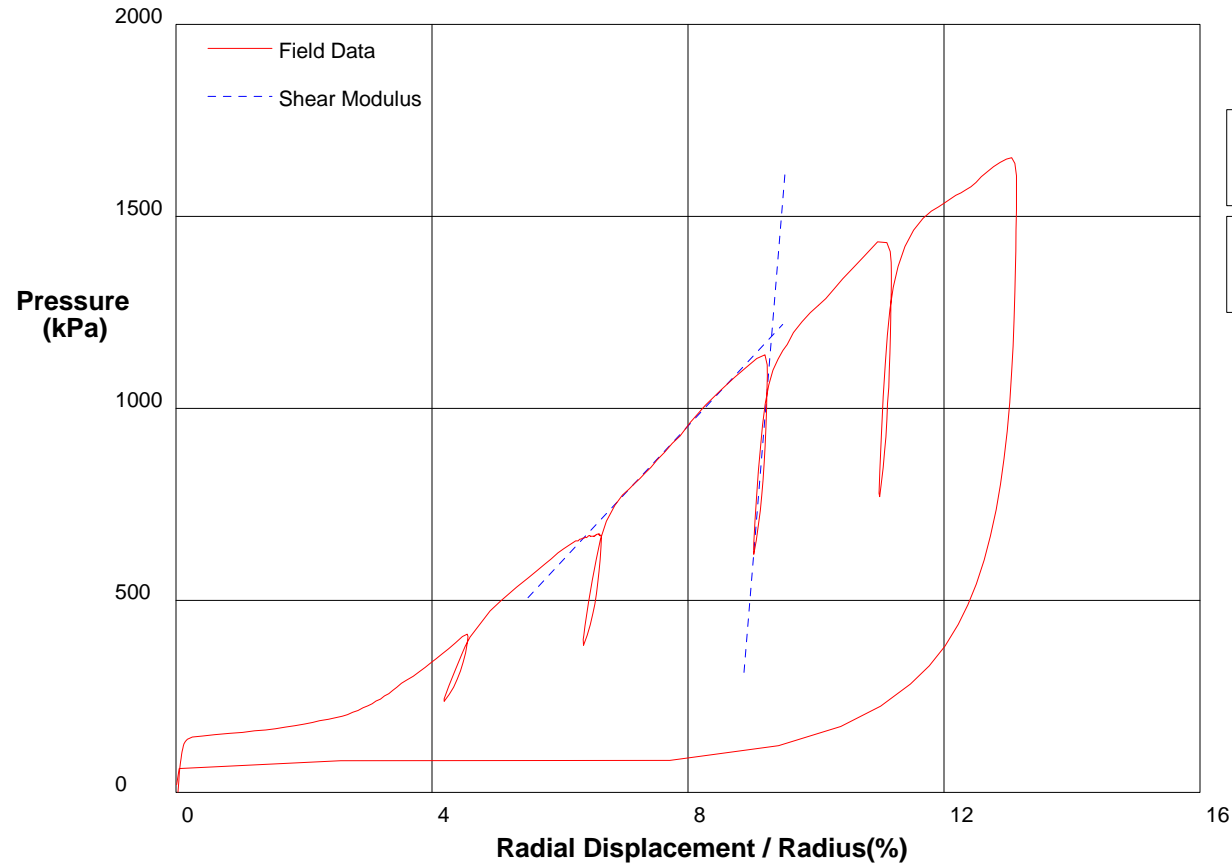
Shear Modulus 1239 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 14m	File C:\DATA\SE-835\LD06.P



Shear Modulus 8944 kPa

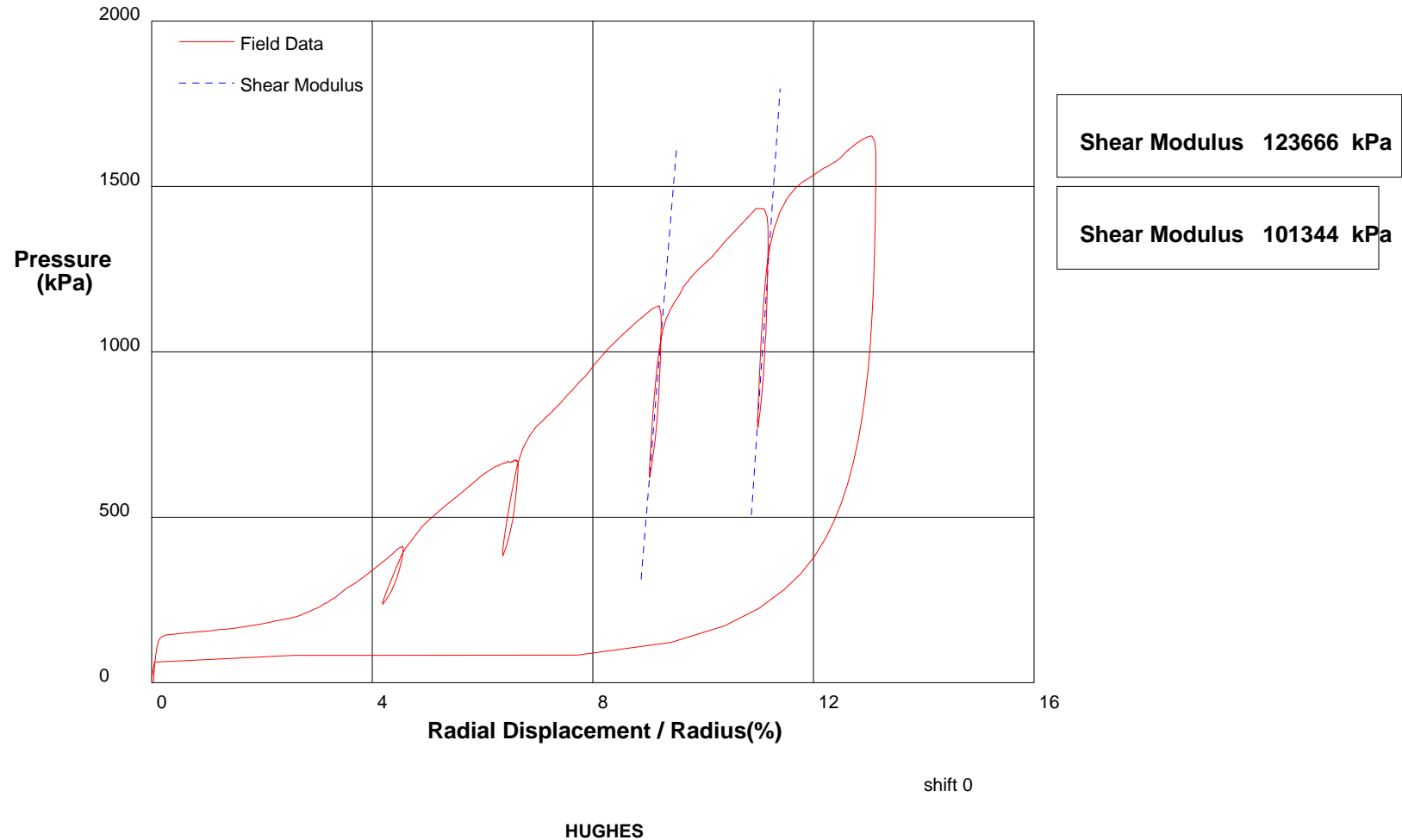
Shear Modulus 101344 kPa

shift 0

HUGHES

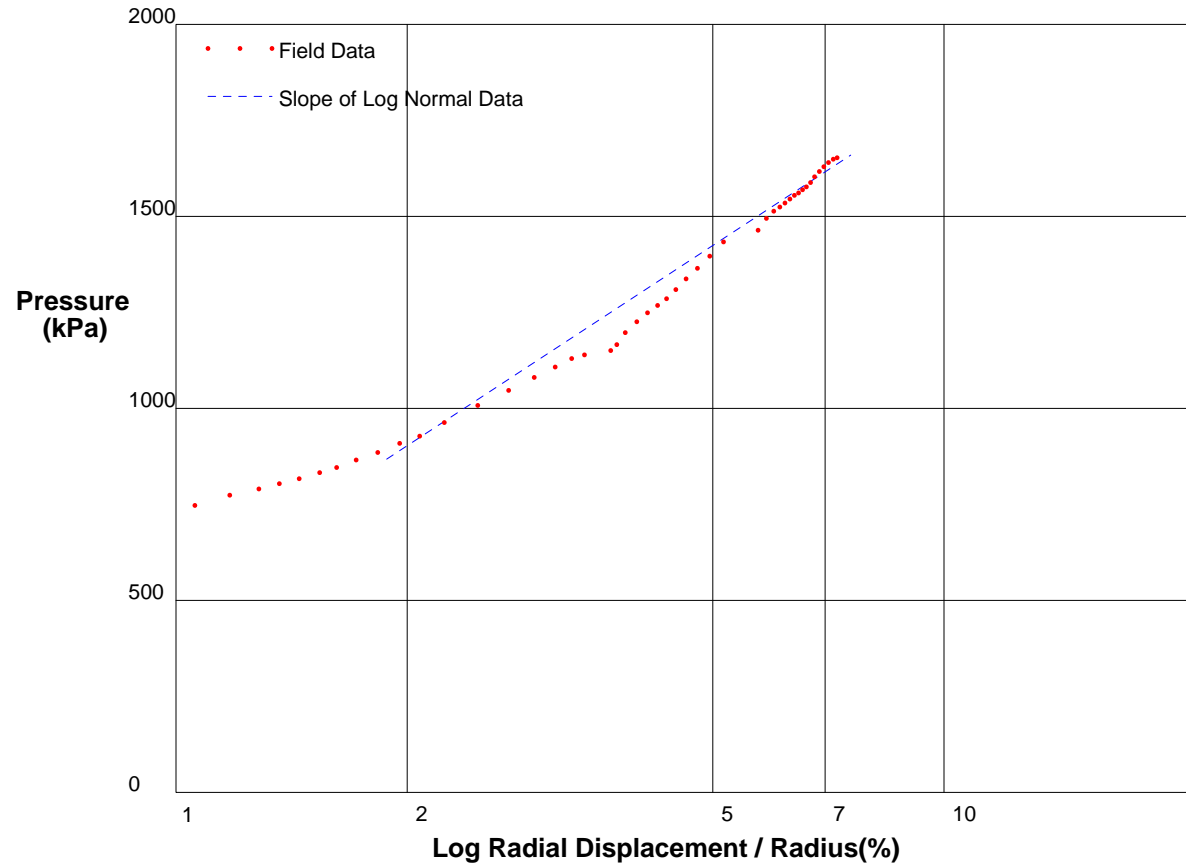
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 14m	File C:\DATA\SE-835\LD06.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 14m	File C:\DATA\ISE-835\LD06.P



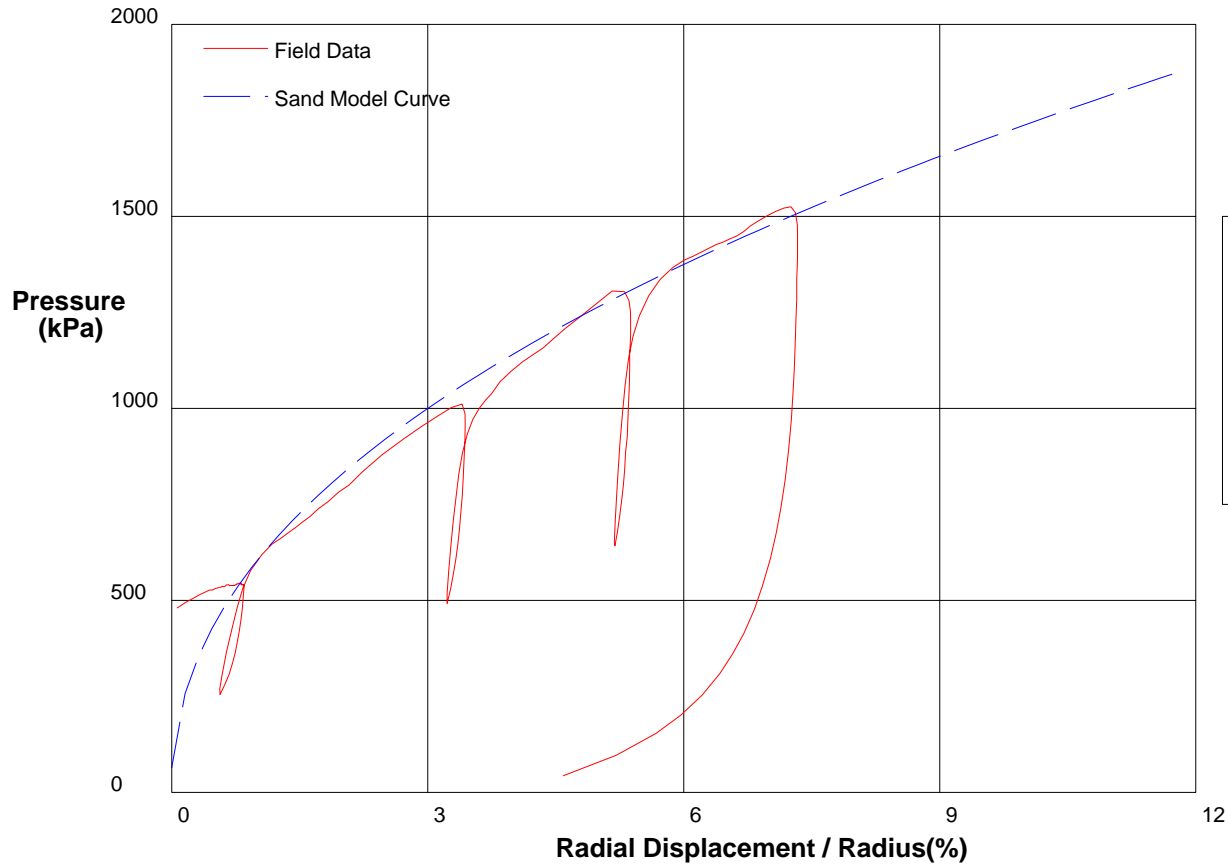
Shear Strength 569 kPa
Limit Pressure 2621 kPa

shift 5.8

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-2S	Depth 14m	File C:\DATA\ISE-835\LD06.P



THE HUGHES SAND MODEL

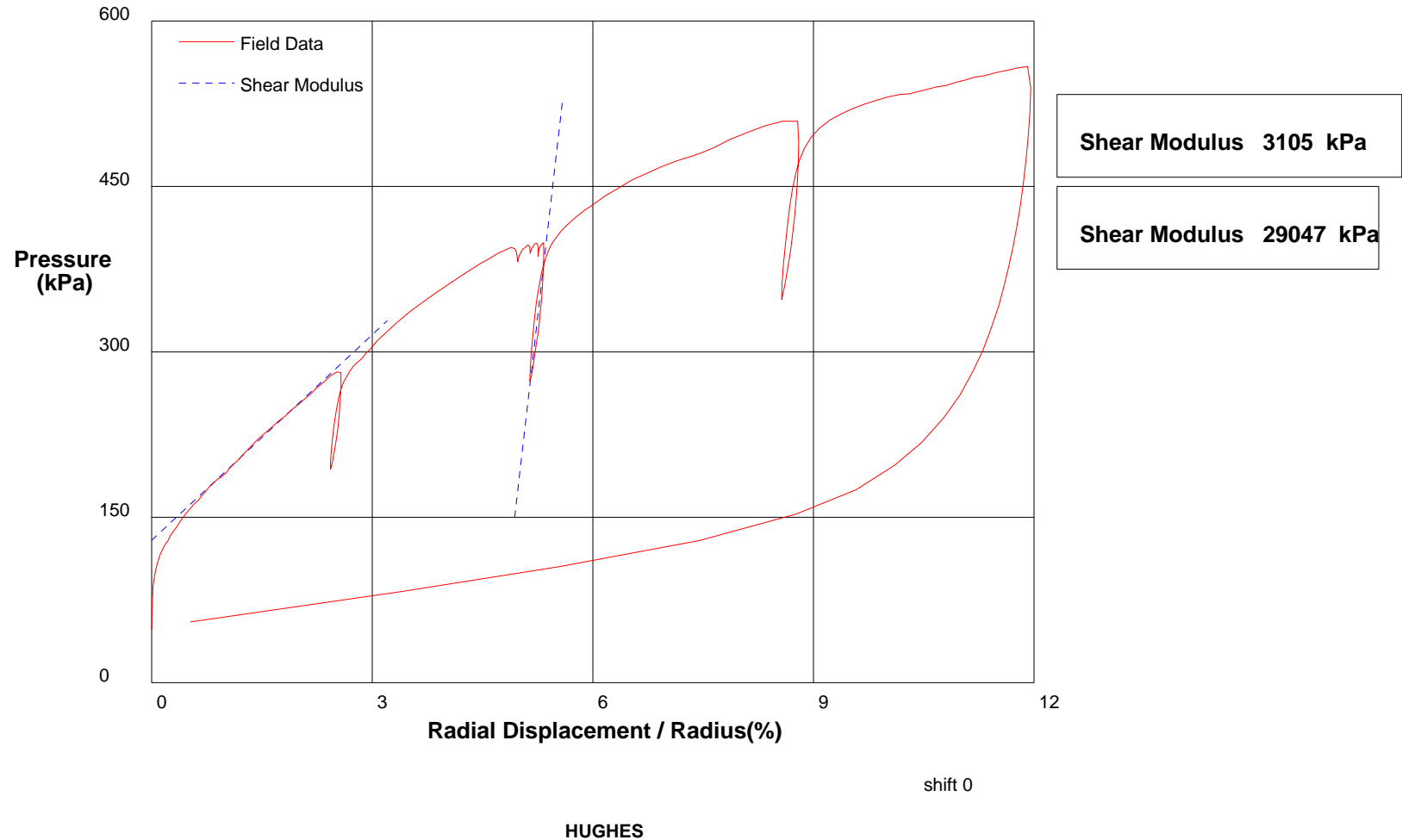
Water Pressure	128 kPa
Friction Angle	38 deg
Critical Friction Angle	28 deg
Lateral Stress	65 kPa
Shear Modulus	90000 kPa

shift 5.8

HUGHES

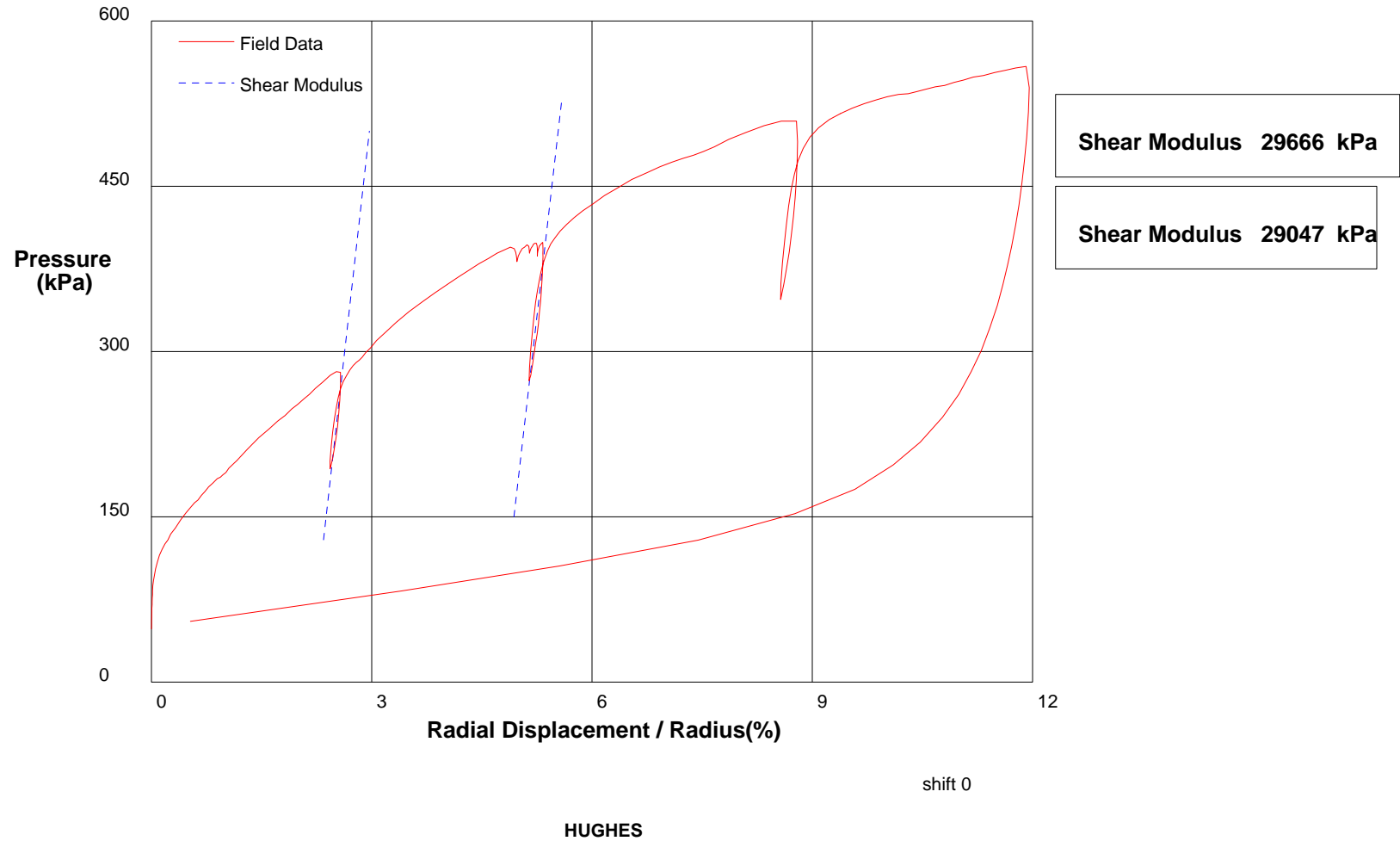
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 7.5m	File C:\DATA\SE-835\LD07.P



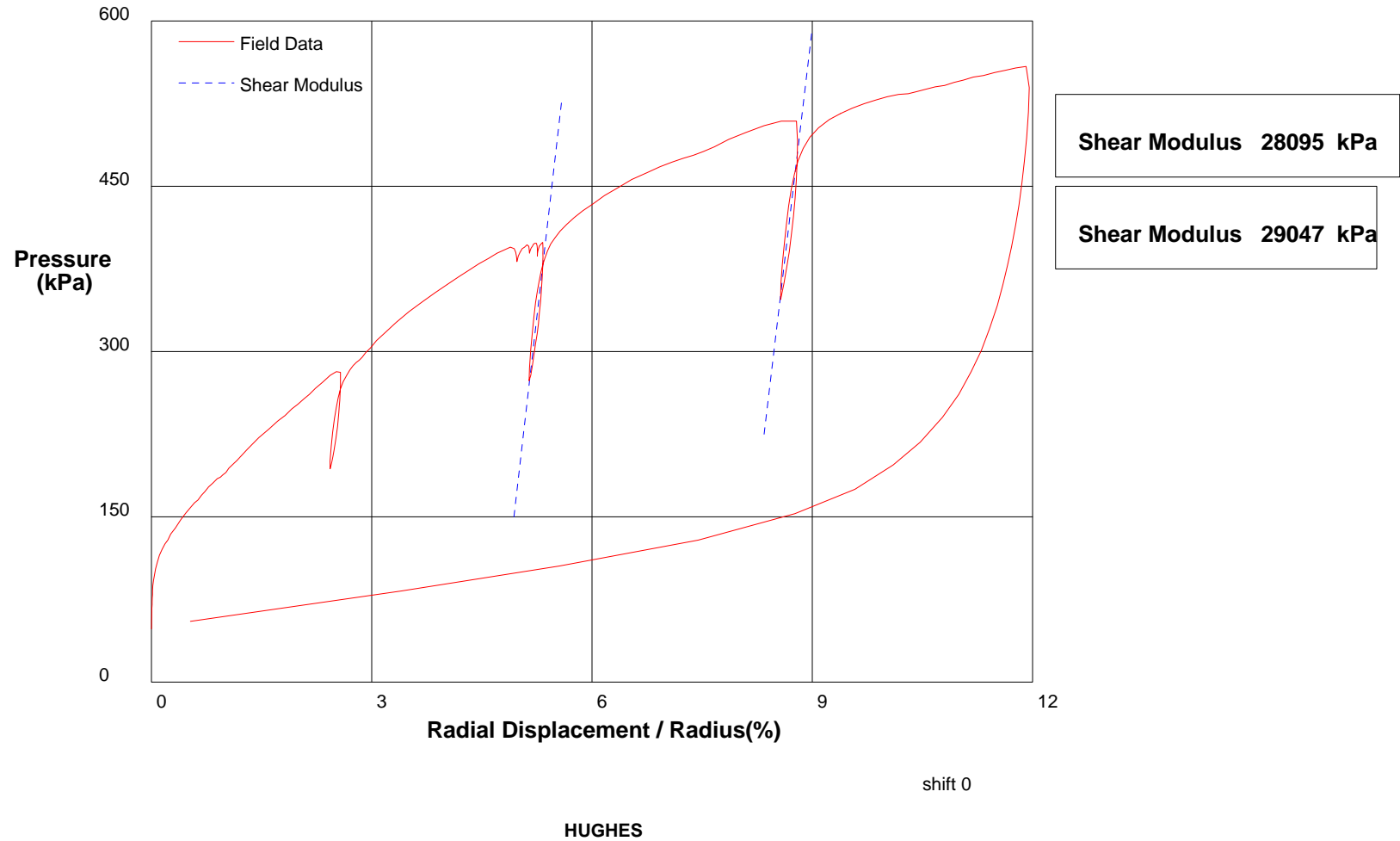
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 7.5m	File C:\DATA\SE-835\LD07.P



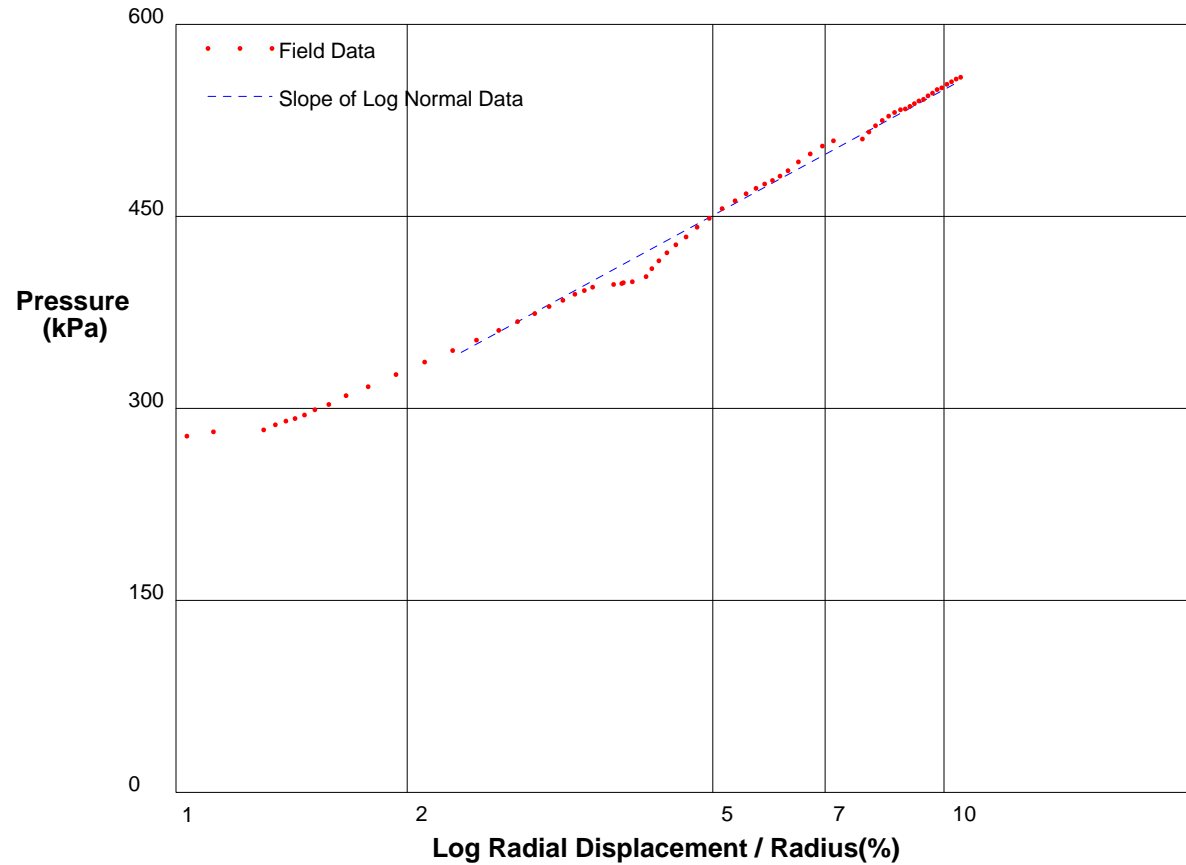
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 7.5m	File C:\DATA\SE-835\LD07.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 7.5m	File C:\DATA\ISE-835\LD07.P



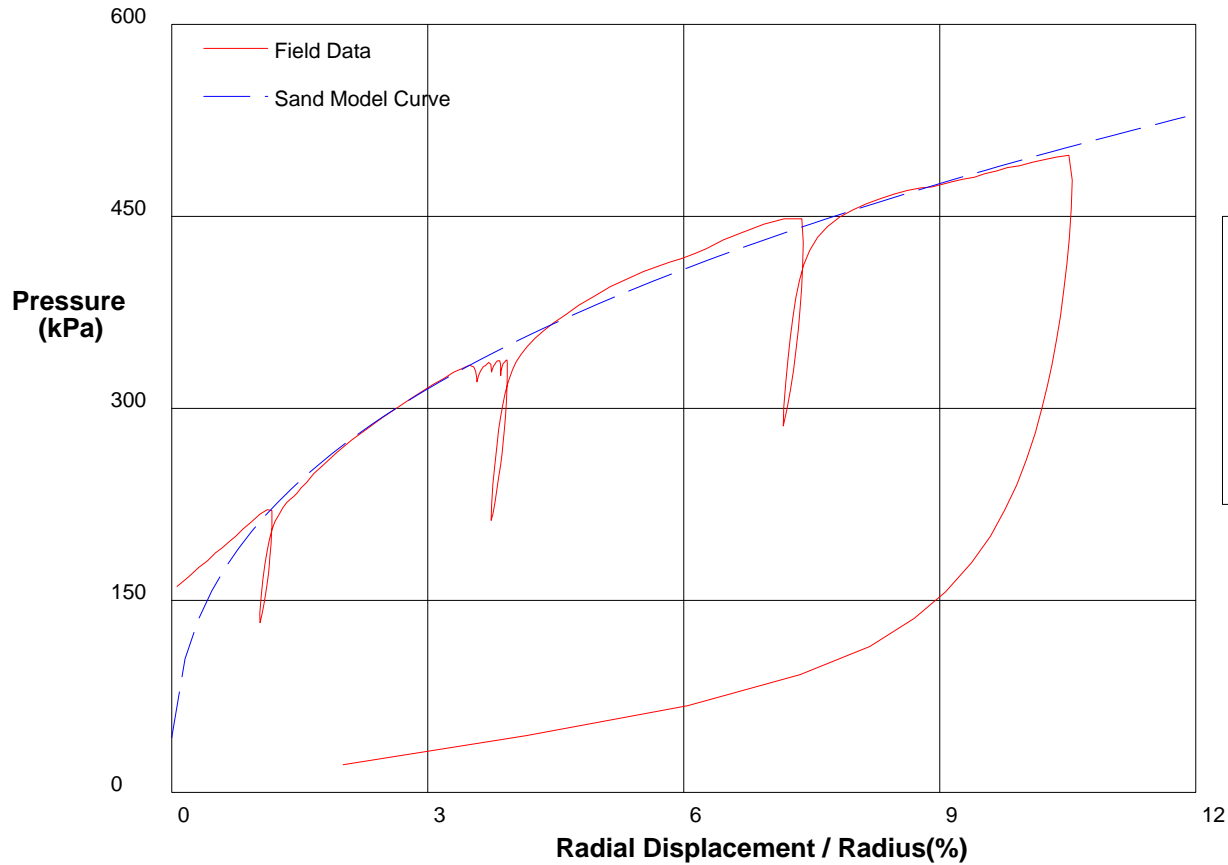
Shear Strength 141.8 kPa
Limit Pressure 749 kPa

shift 1.4

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 7.5m	File C:\DATA\ISE-835\LD07.P



THE HUGHES SAND MODEL

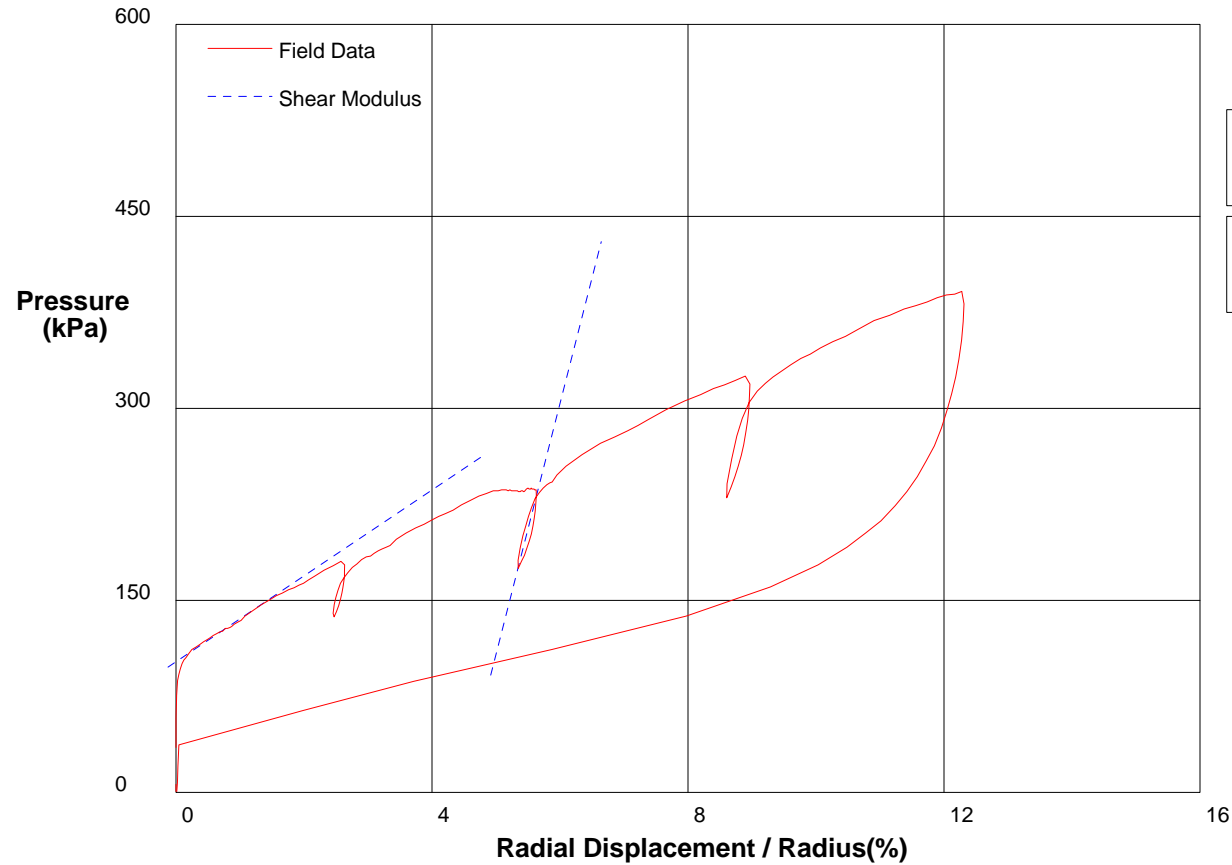
Water Pressure	61 kPa
Friction Angle	32 deg
Critical Friction Angle	28 deg
Lateral Stress	43 kPa
Shear Modulus	25000 kPa

shift 1.4

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.25m	File C:\DATA\ISE-835\LD08.P



Shear Modulus 1677 kPa

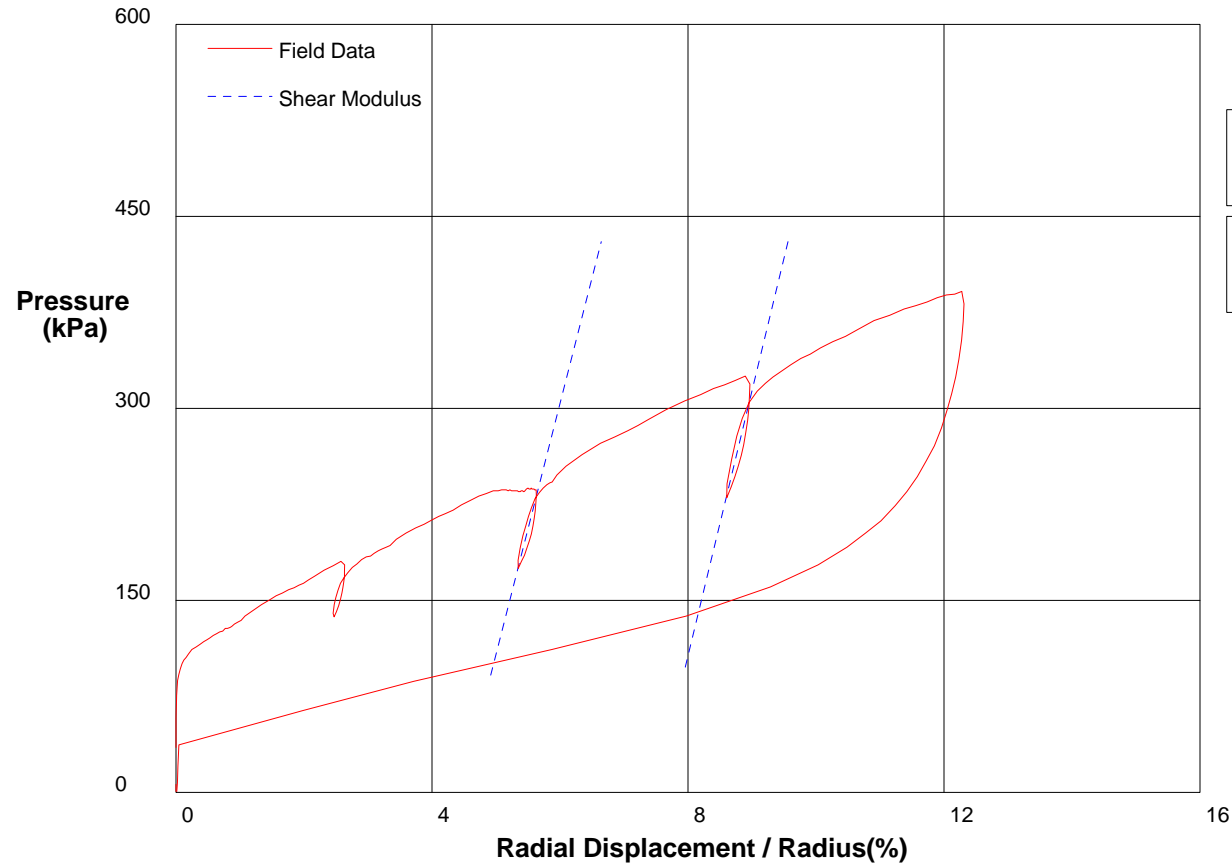
Shear Modulus 9789 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.25m	File C:\DATA\ISE-835\LD08.P



Shear Modulus 10357 kPa

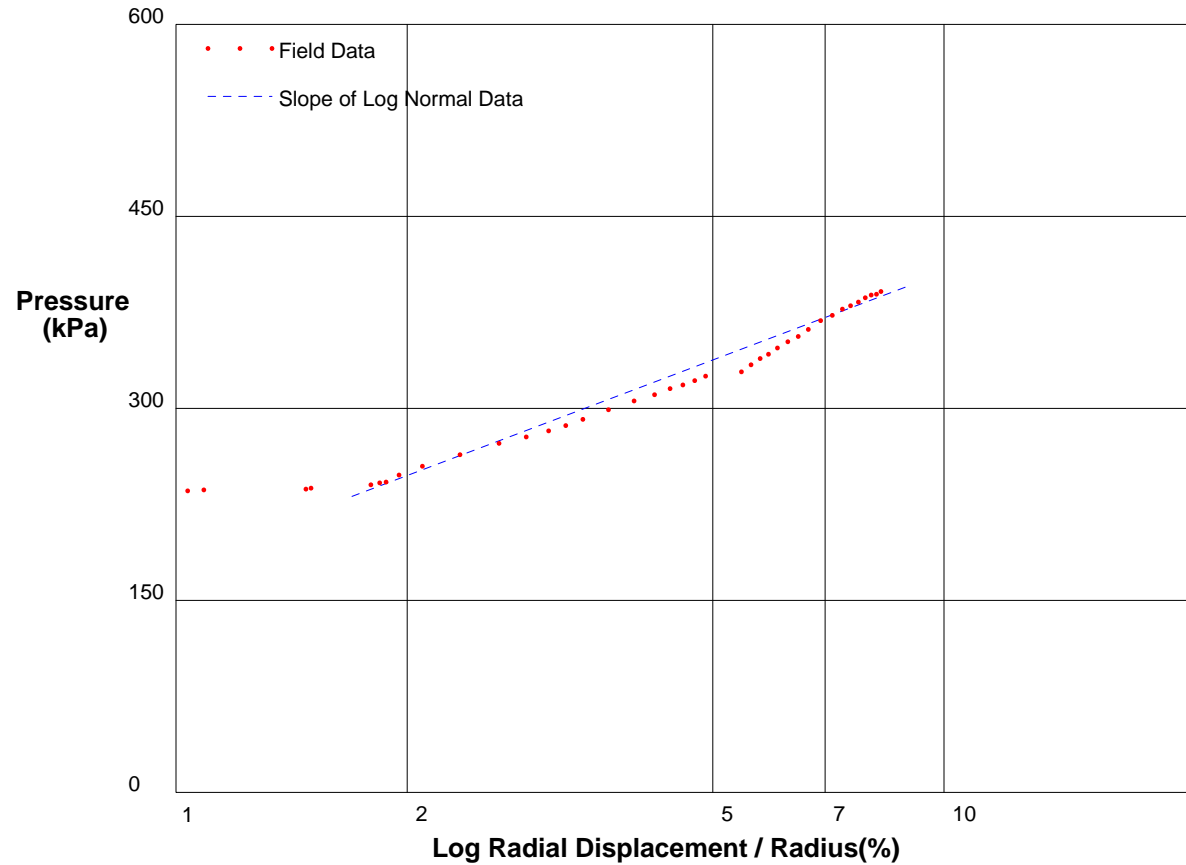
Shear Modulus 9789 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.25m	File C:\DATA\ISE-835\LD08.P



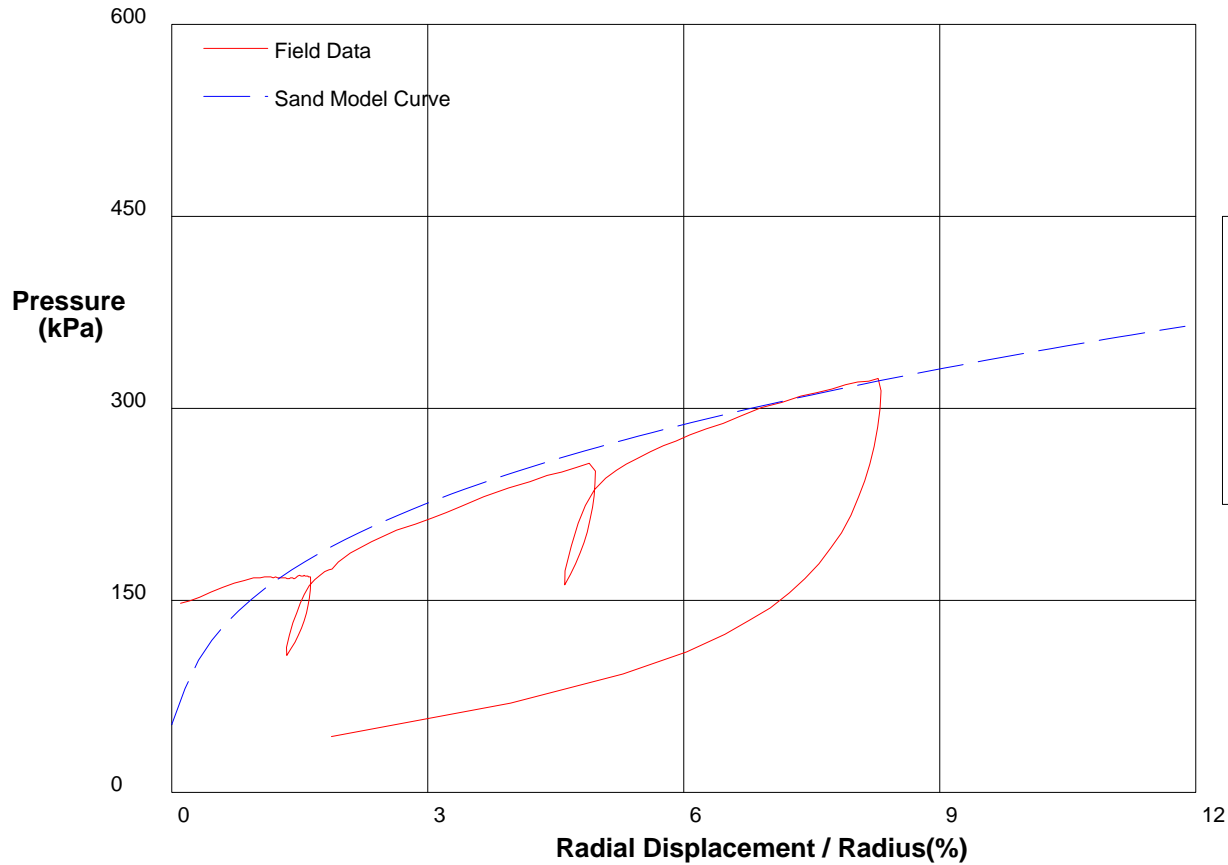
Shear Strength 98.4 kPa
Limit Pressure 545 kPa

shift 4

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.25m	File C:\DATA\ISE-835\LD08.P



THE HUGHES SAND MODEL

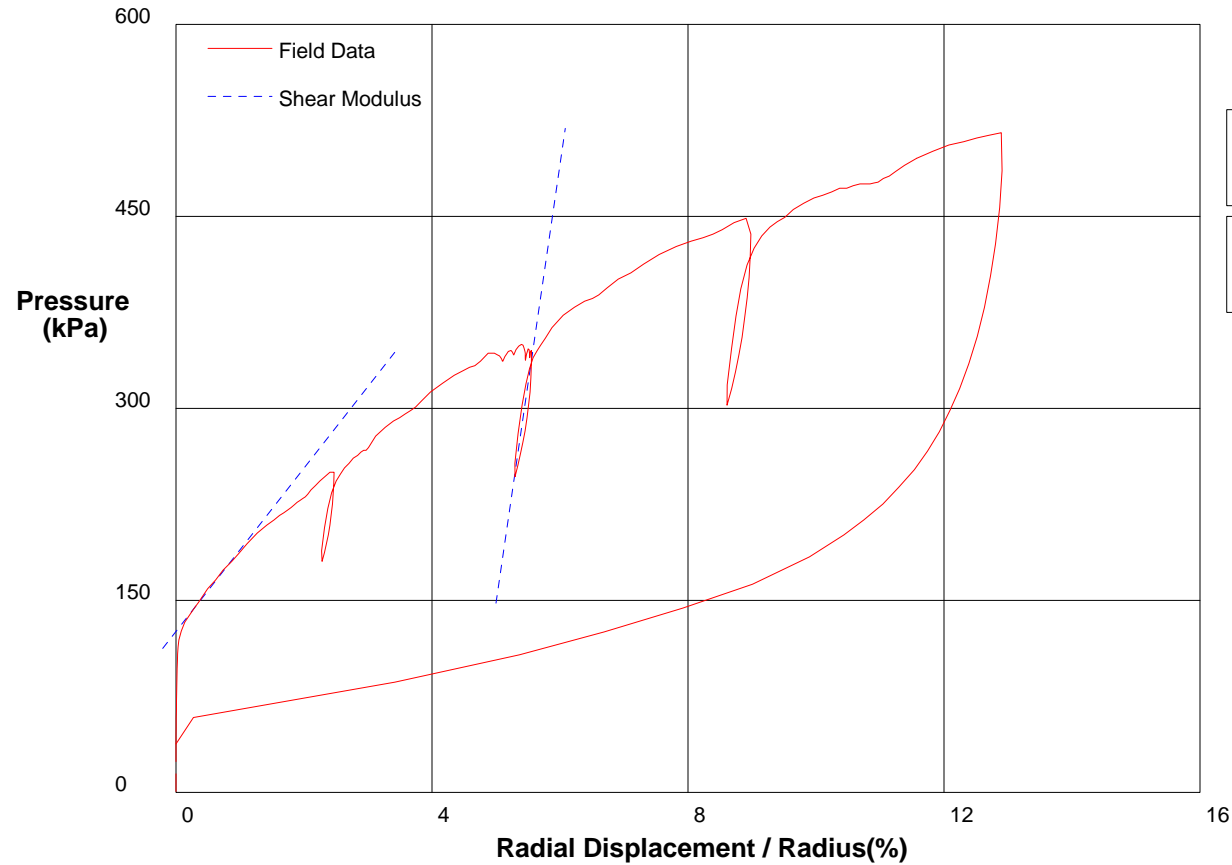
Water Pressure	68 kPa
Friction Angle	30 deg
Critical Friction Angle	28 deg
Lateral Stress	53 kPa
Shear Modulus	9000 kPa

shift 4

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.8m	File C:\DATA\SE-835\LD09.P



Shear Modulus 3189 kPa

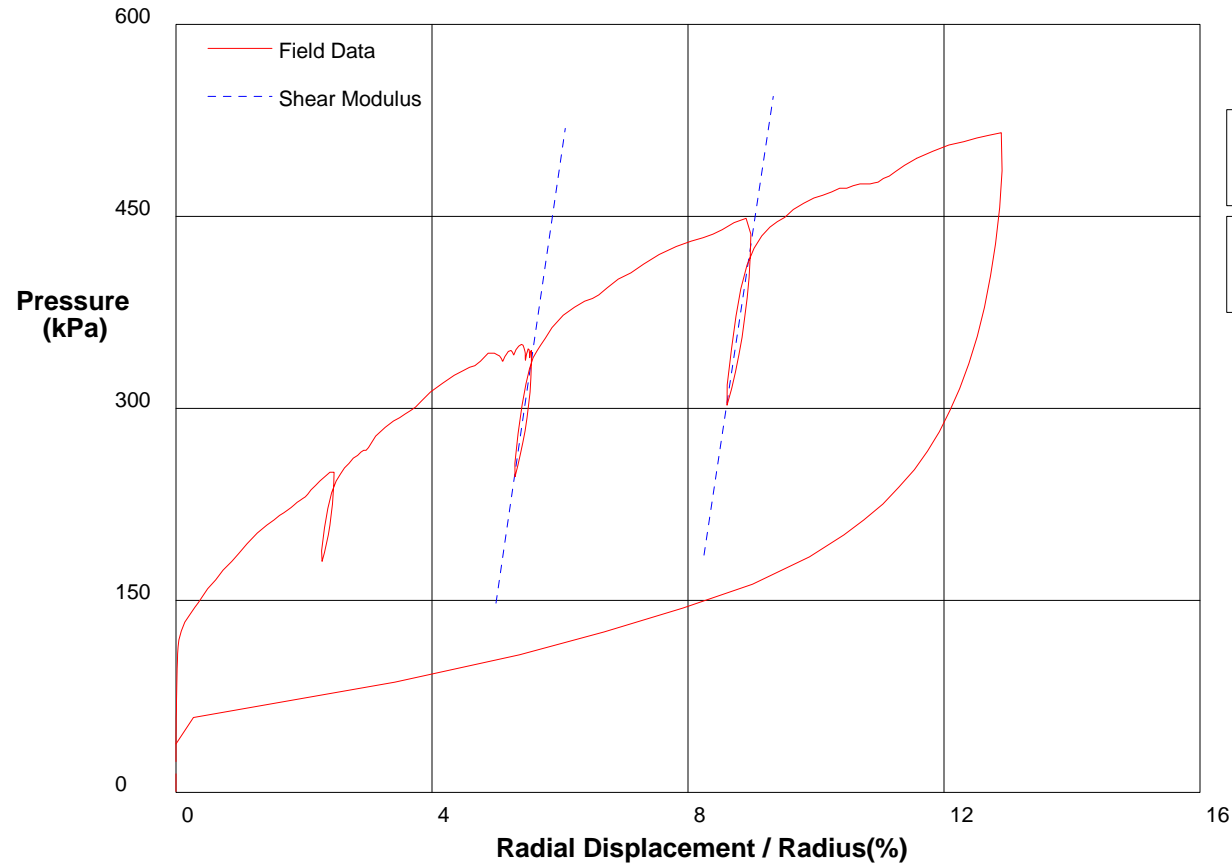
Shear Modulus 17115 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.8m	File C:\DATA\SE-835\LD09.P



Shear Modulus 16538 kPa

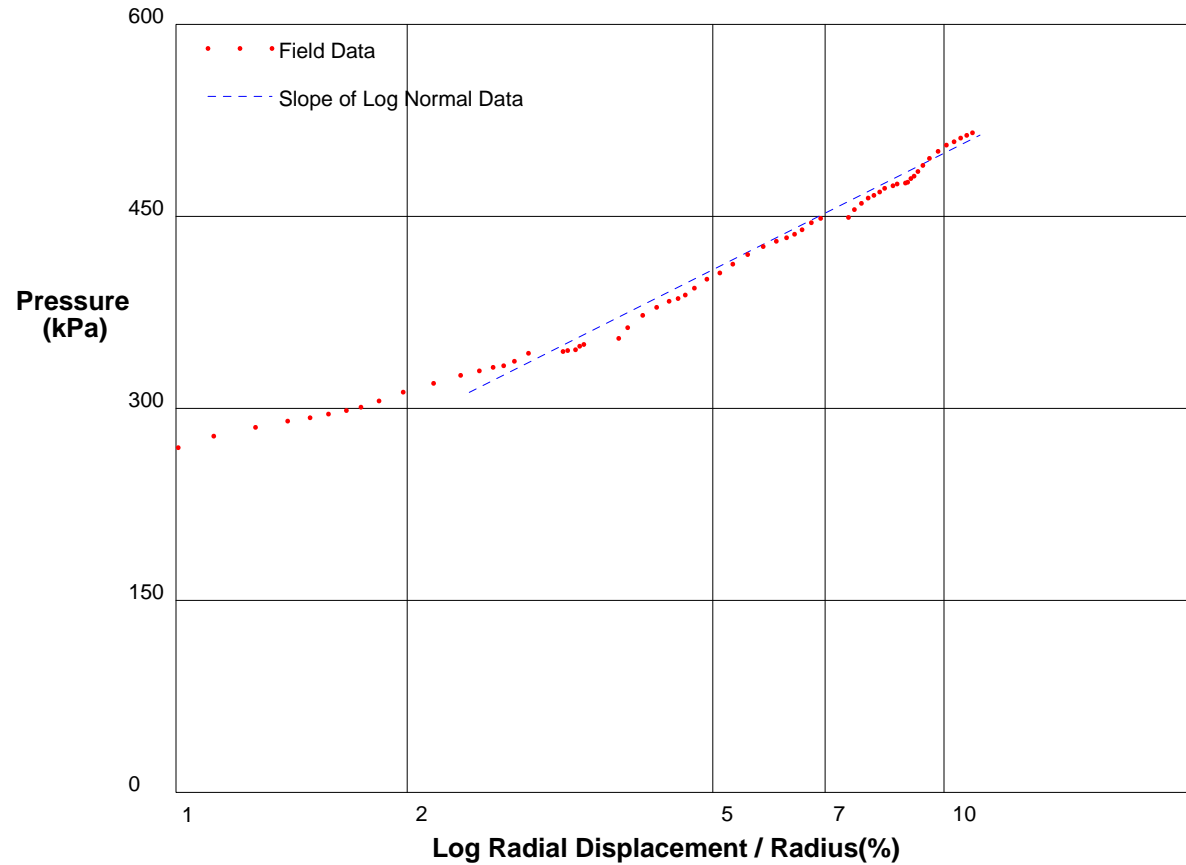
Shear Modulus 17115 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.8m	File C:\DATA\ISE-835\LD09.P



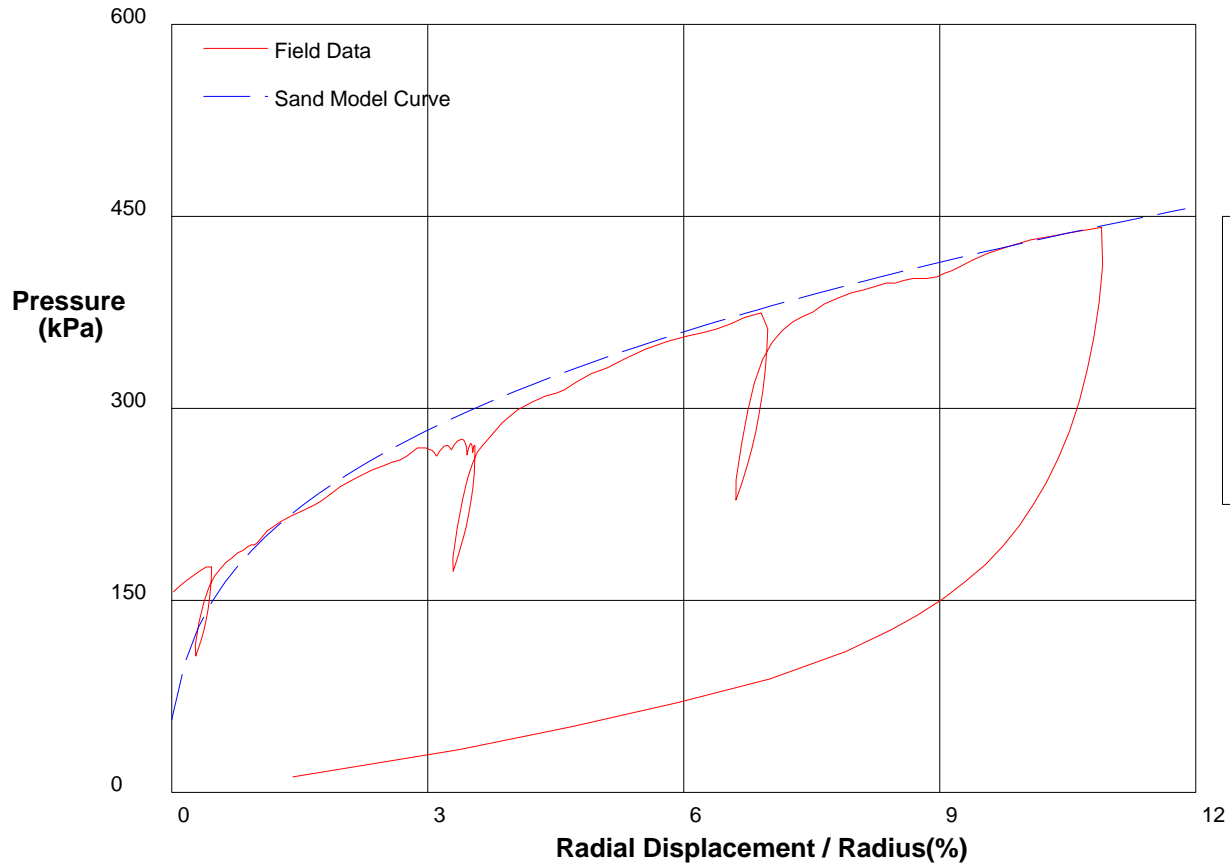
Shear Strength 131.3 kPa
Limit Pressure 684 kPa

shift 2

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/19/2009
Hole No. PM09-1S	Depth 8.8m	File C:\DATA\ISE-835\LD09.P



THE HUGHES SAND MODEL

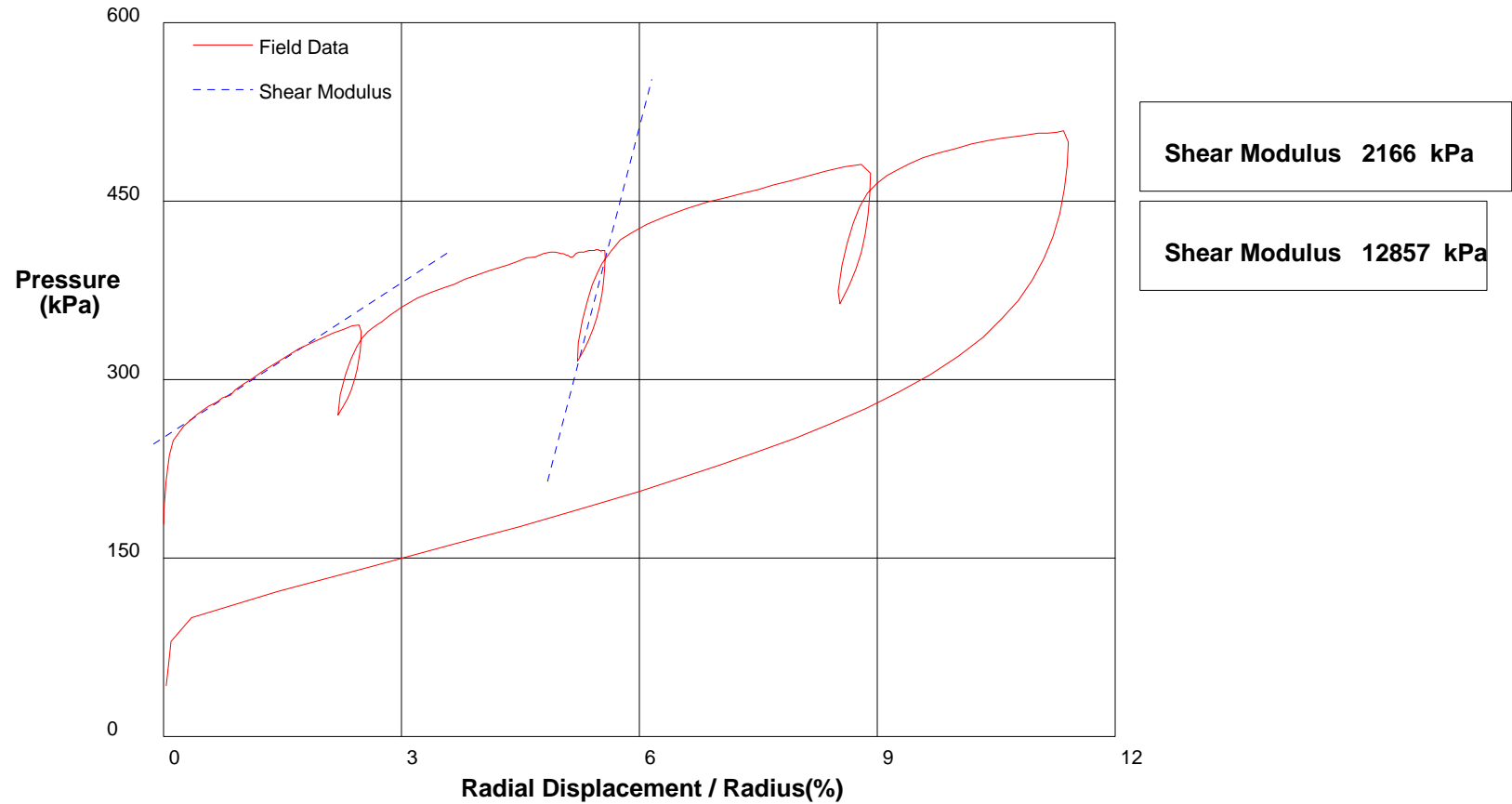
Water Pressure	74 kPa
Friction Angle	30 deg
Critical Friction Angle	28 deg
Lateral Stress	57 kPa
Shear Modulus	15000 kPa

shift 2

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 10.5m	File C:\DATA\SE-835\LD10.P



Shear Modulus 2166 kPa

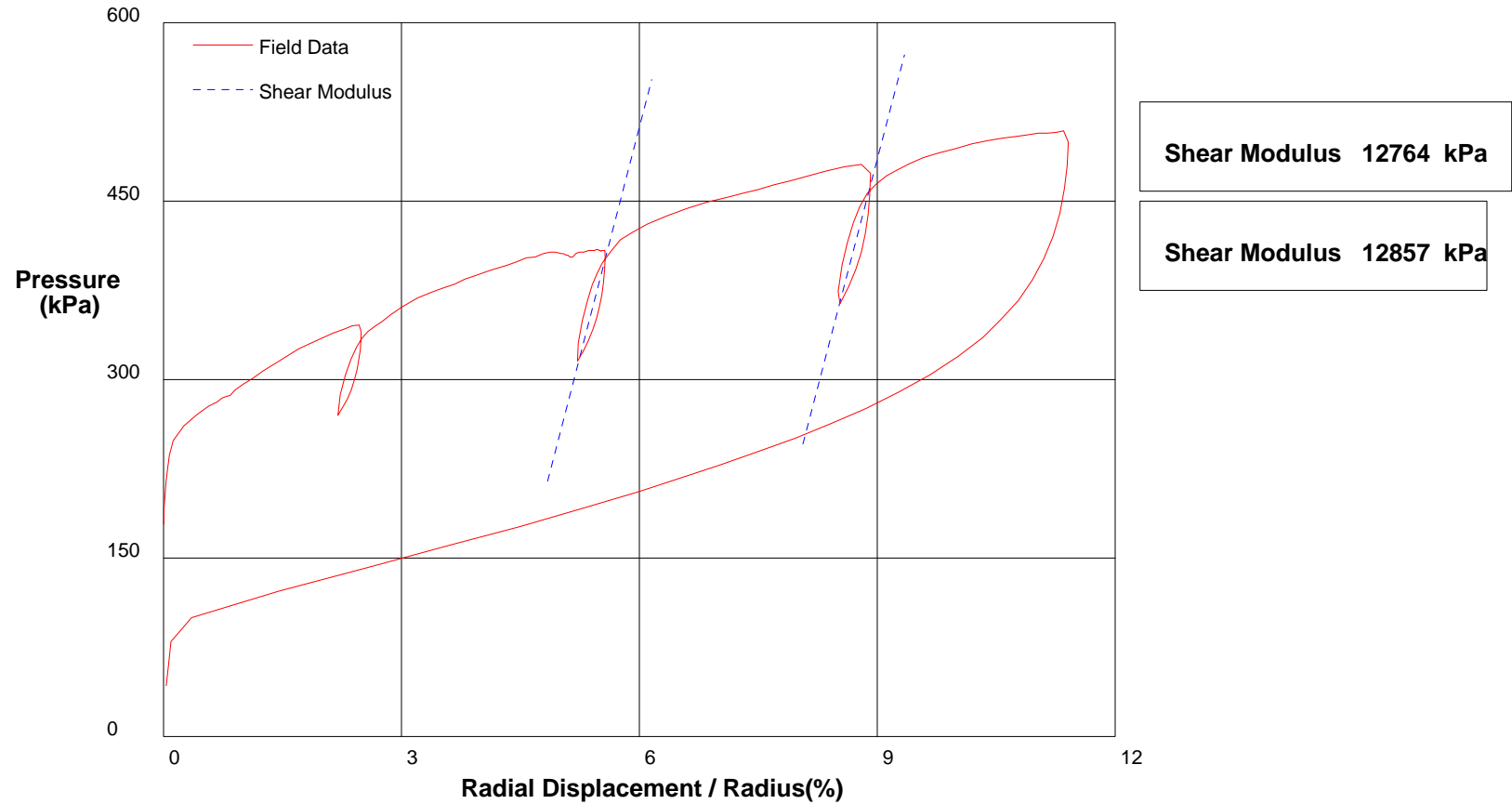
Shear Modulus 12857 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 10.5m	File C:\DATA\SE-835\LD10.P

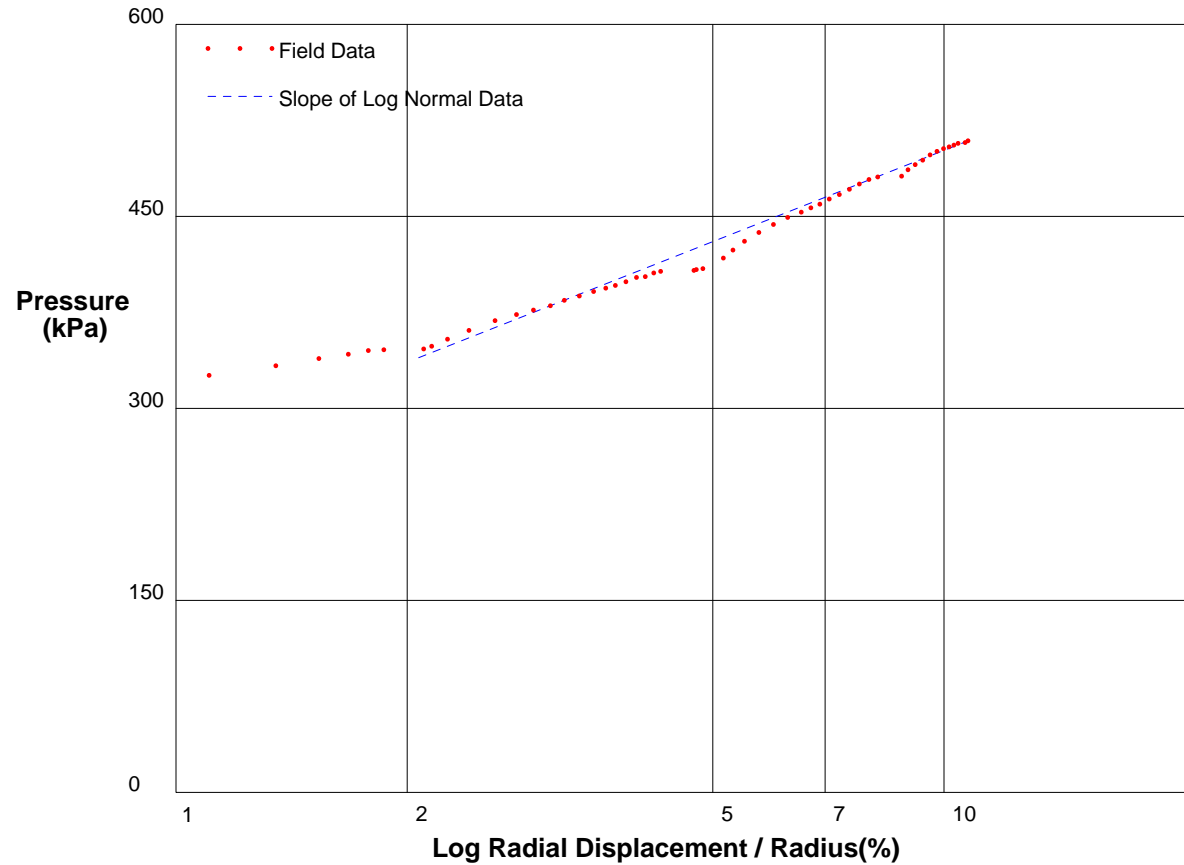


shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 10.5m	File C:\DATA\ISE-835\LD10.P



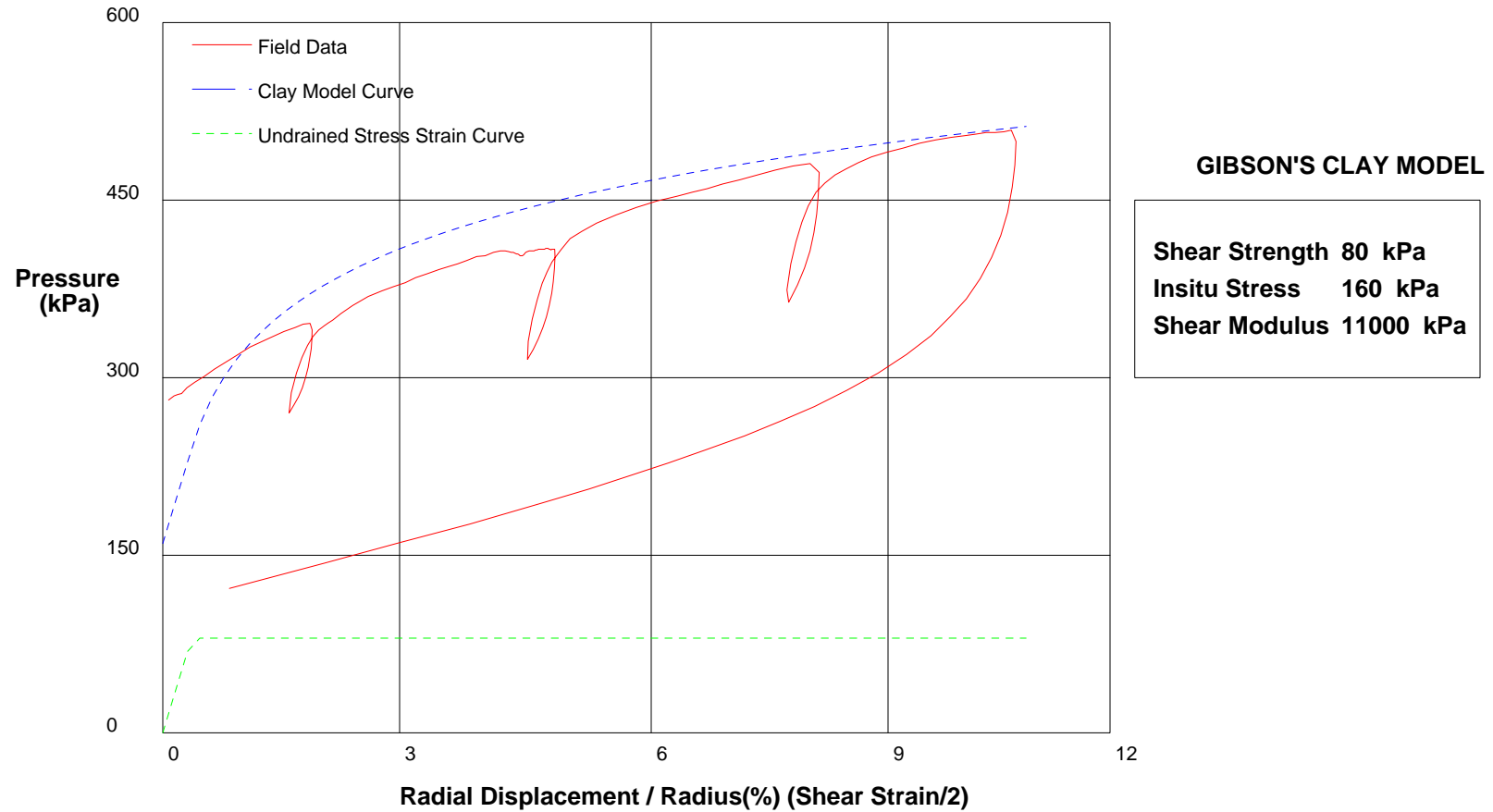
Shear Strength 102.8 kPa
Limit Pressure 646 kPa

shift .6

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 10.5m	File C:\DATA\ISE-835\LD10.P

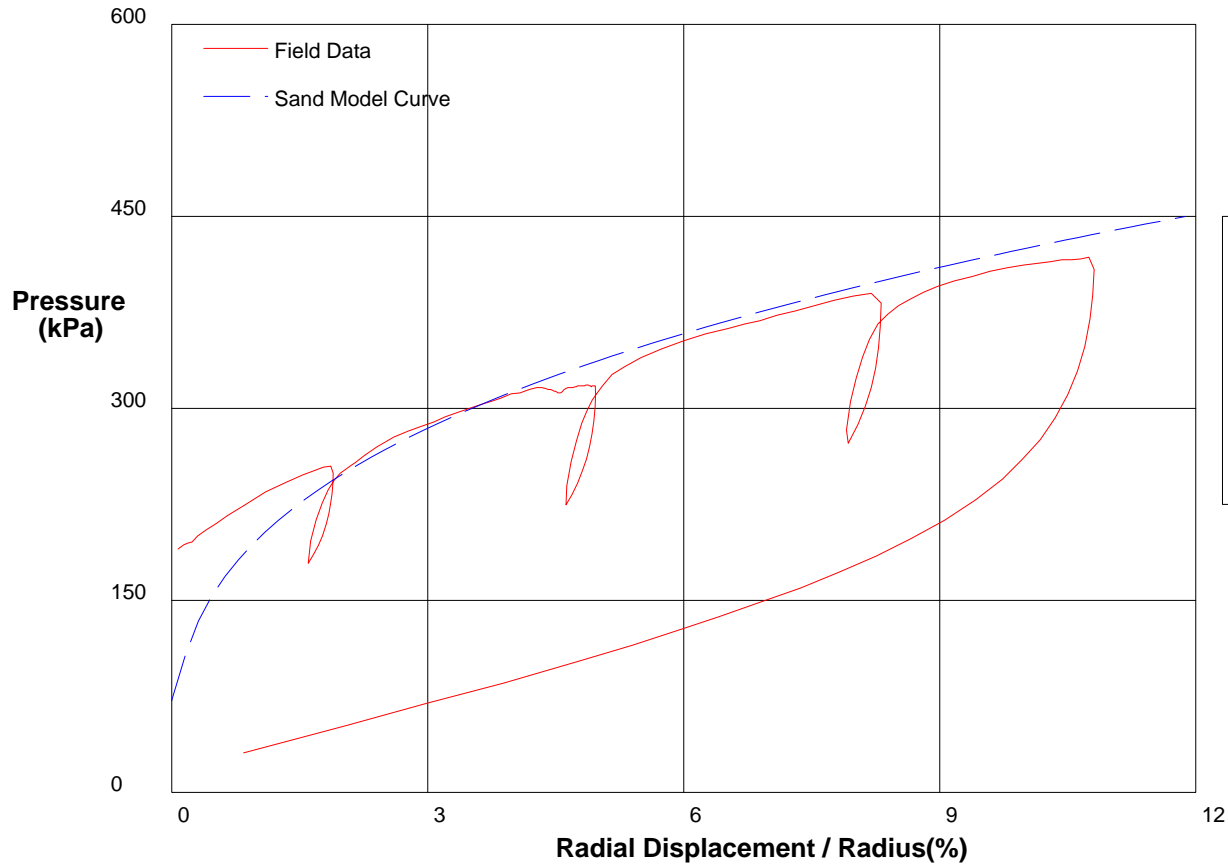


shift .6

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 10.5m	File C:\DATA\ISE-835\LD10.P



THE HUGHES SAND MODEL

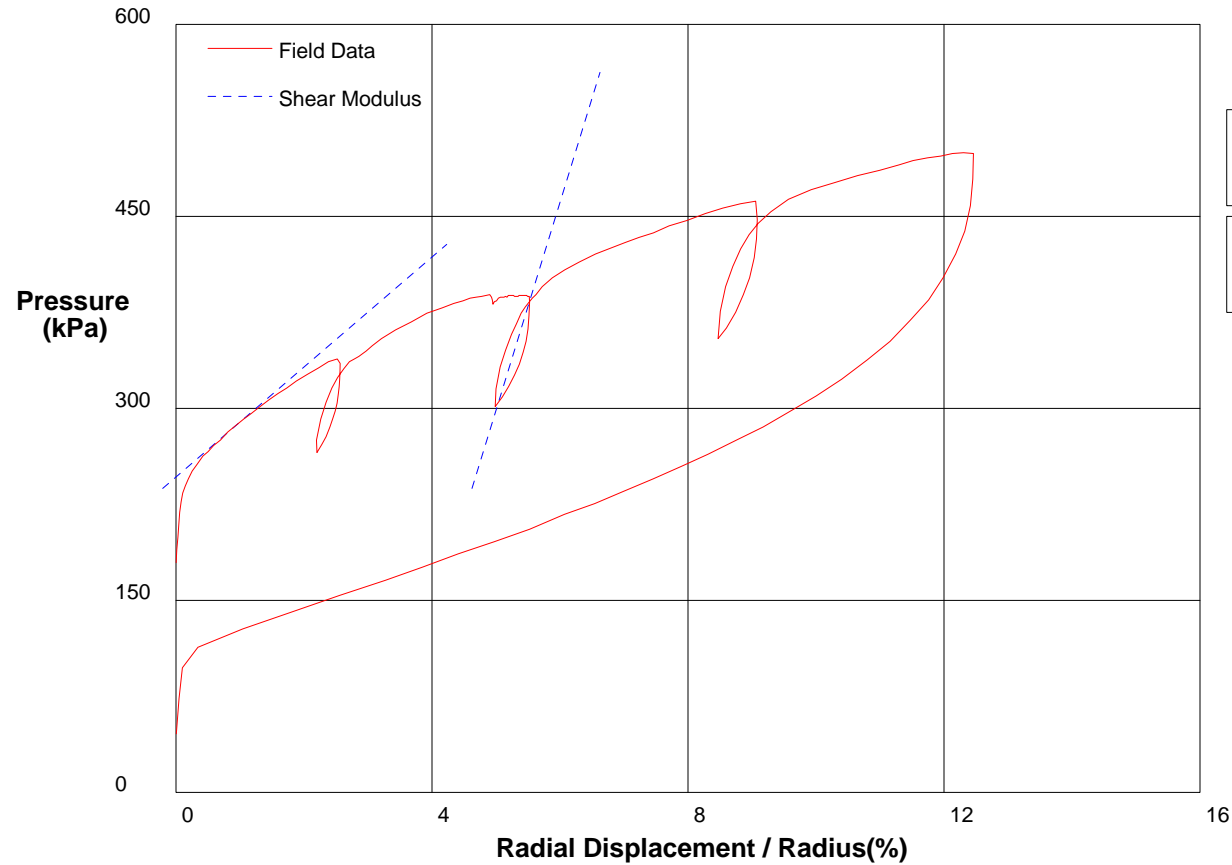
Water Pressure	91 kPa
Friction Angle	29 deg
Critical Friction Angle	28 deg
Lateral Stress	72 kPa
Shear Modulus	11000 kPa

shift .6

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 11.25m	File C:\DATA\SE-835\LD11.P



Shear Modulus 2147 kPa

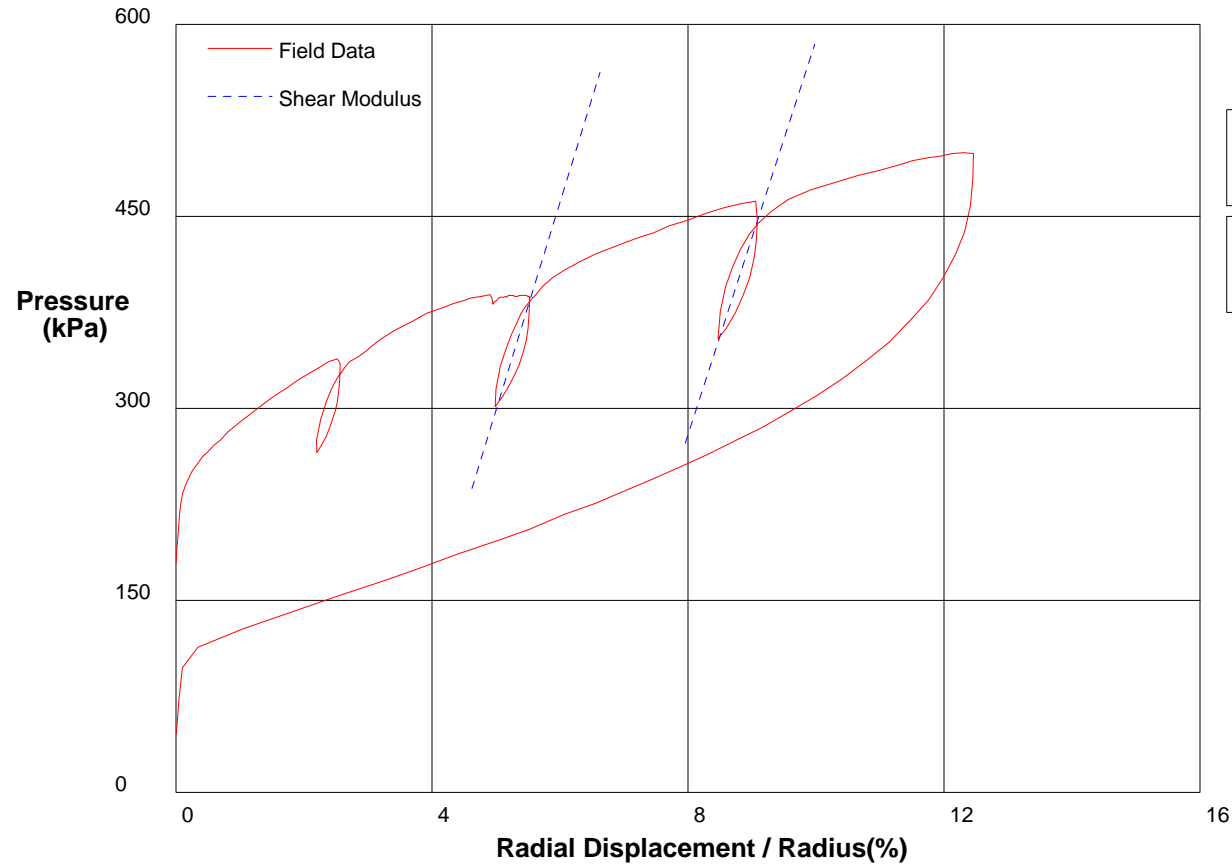
Shear Modulus 8125 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 11.25m	File C:\DATA\SE-835\LD11.P



Shear Modulus 7706 kPa

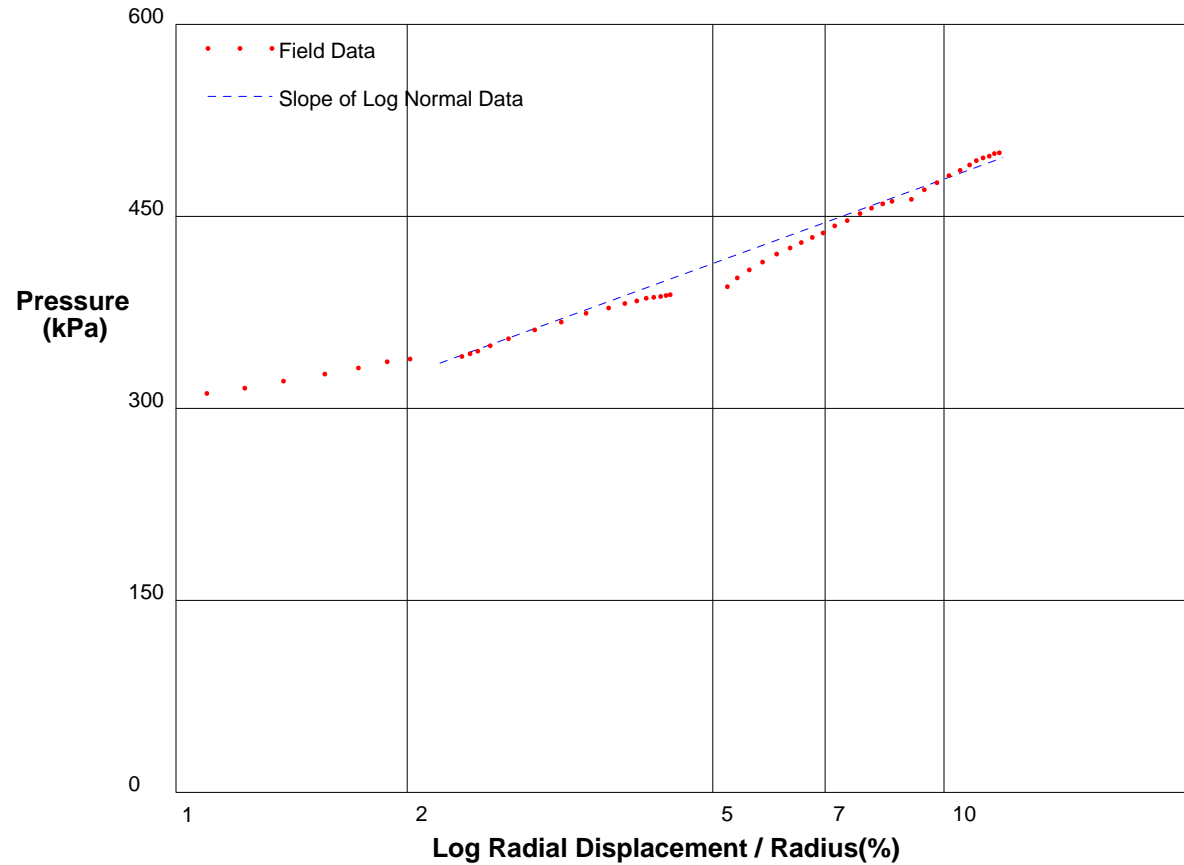
Shear Modulus 8125 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 11.25m	File C:\DATA\ISE-835\LD11.P



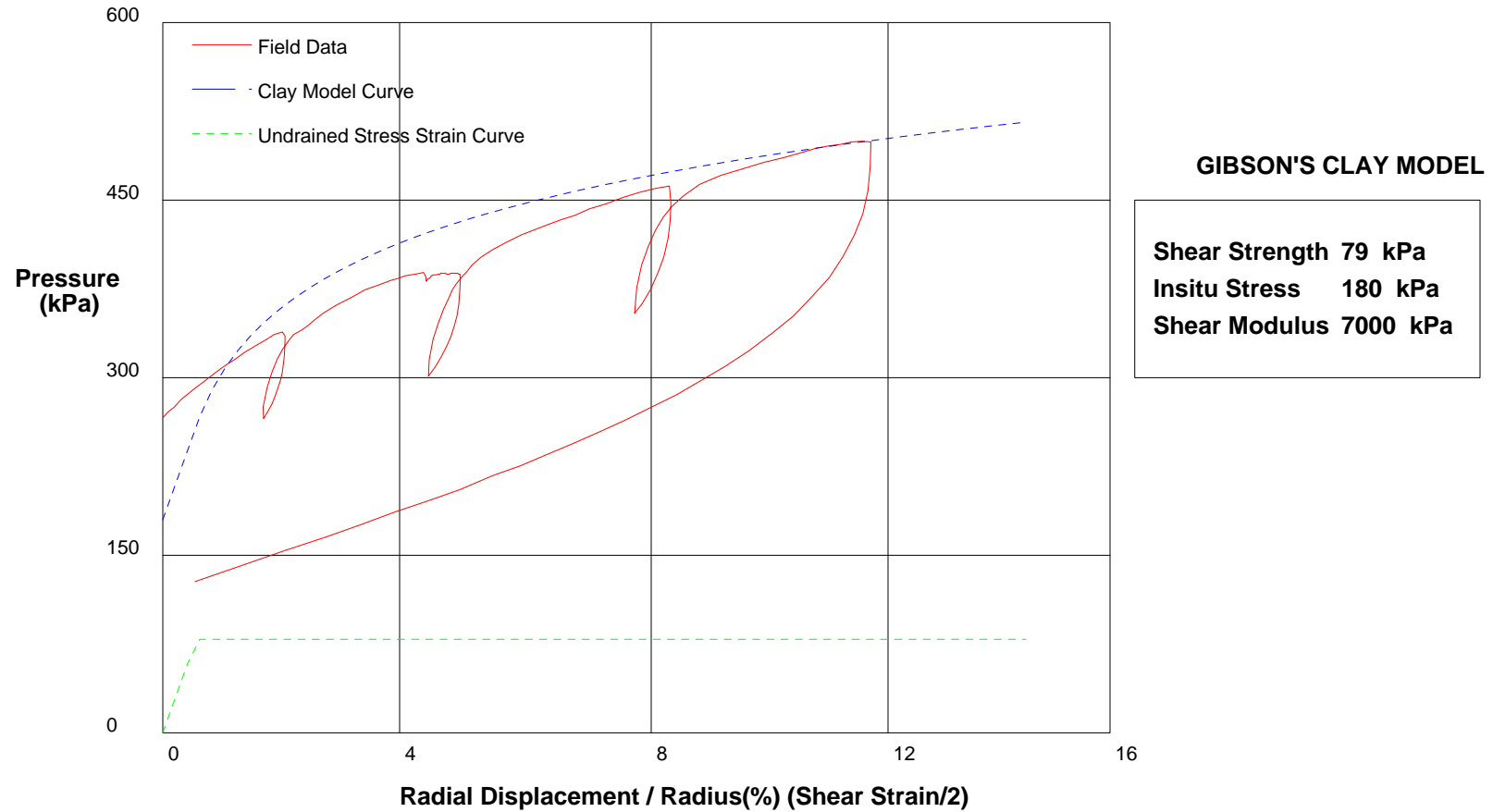
Shear Strength 95 kPa
Limit Pressure 613 kPa

shift .5

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 11.25m	File C:\DATA\ISE-835\LD11.P

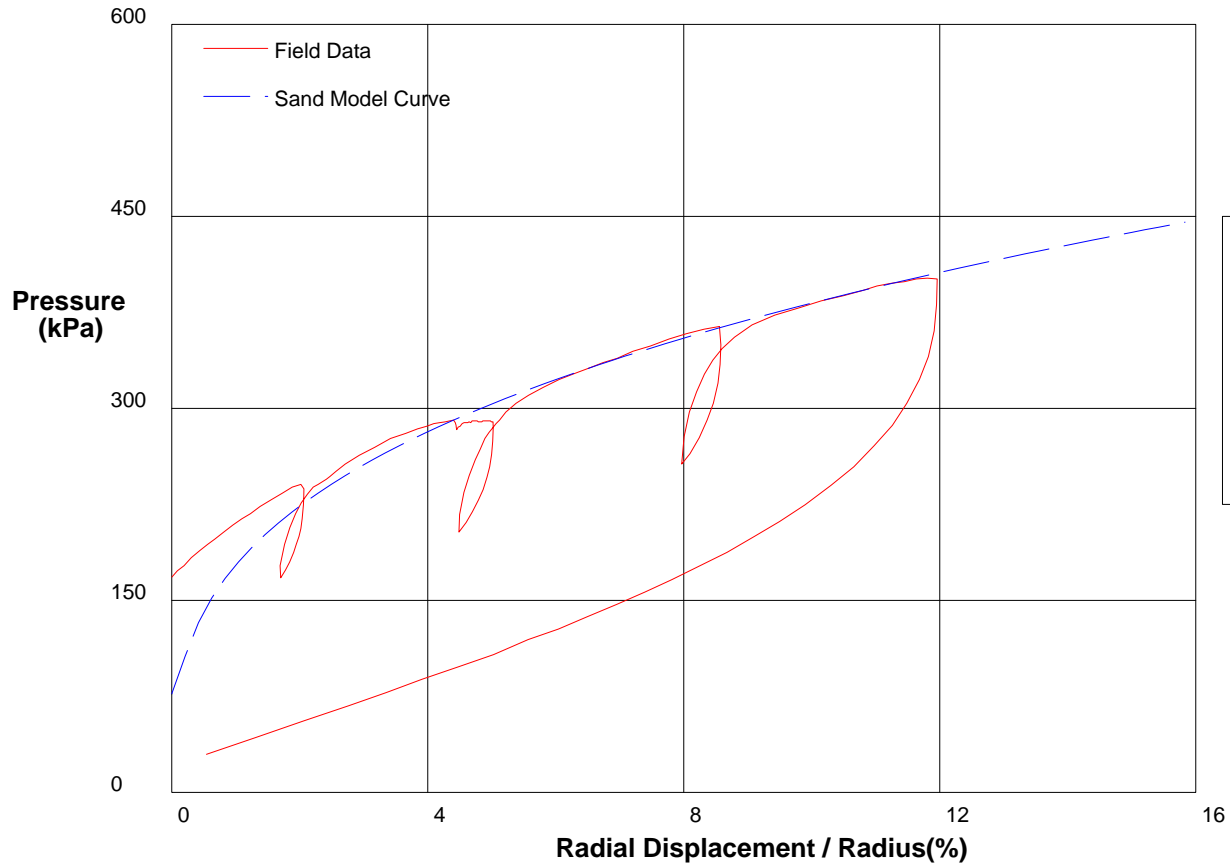


shift .5

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 11.25m	File C:\DATA\ISE-835\LD11.P



THE HUGHES SAND MODEL

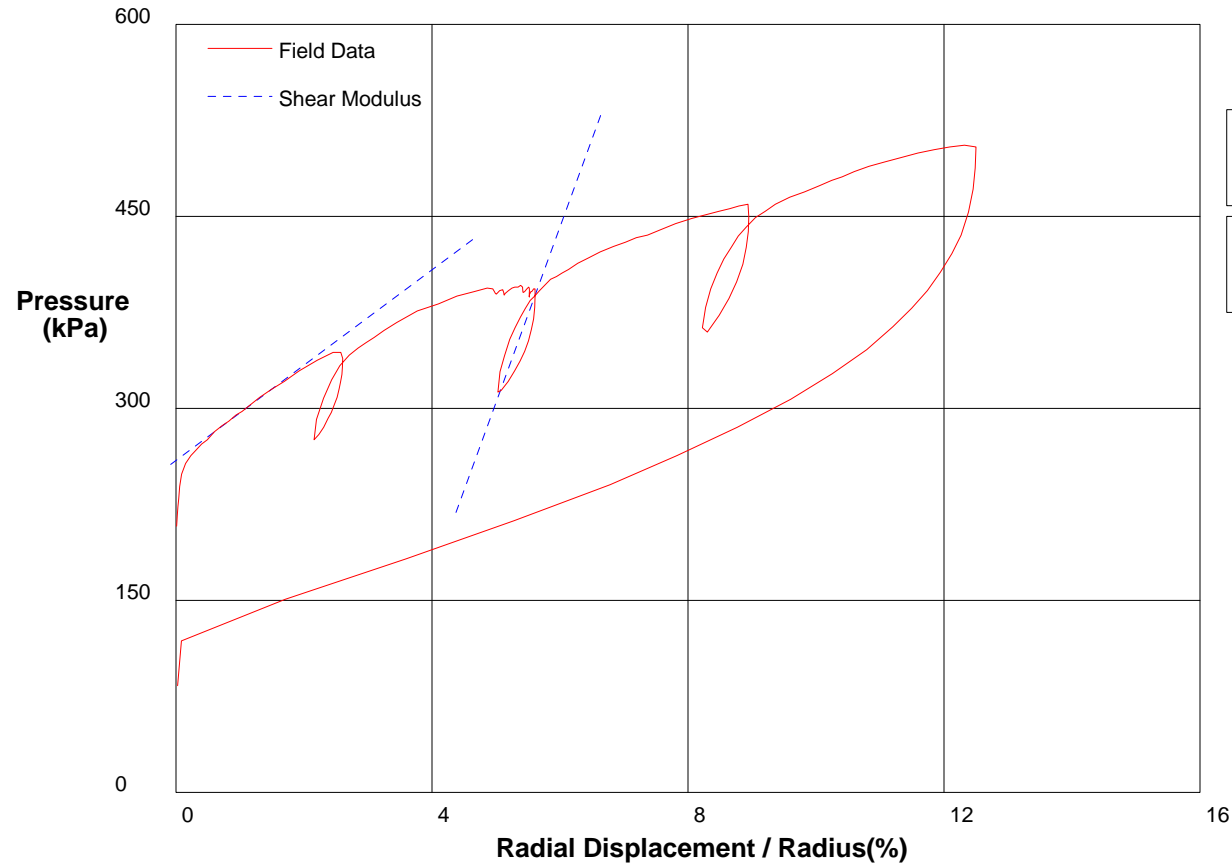
Water Pressure	98 kPa
Friction Angle	29 deg
Critical Friction Angle	28 deg
Lateral Stress	77 kPa
Shear Modulus	7000 kPa

shift .5

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12m	File C:\DATA\SE-835\LD12.P



Shear Modulus 1861 kPa

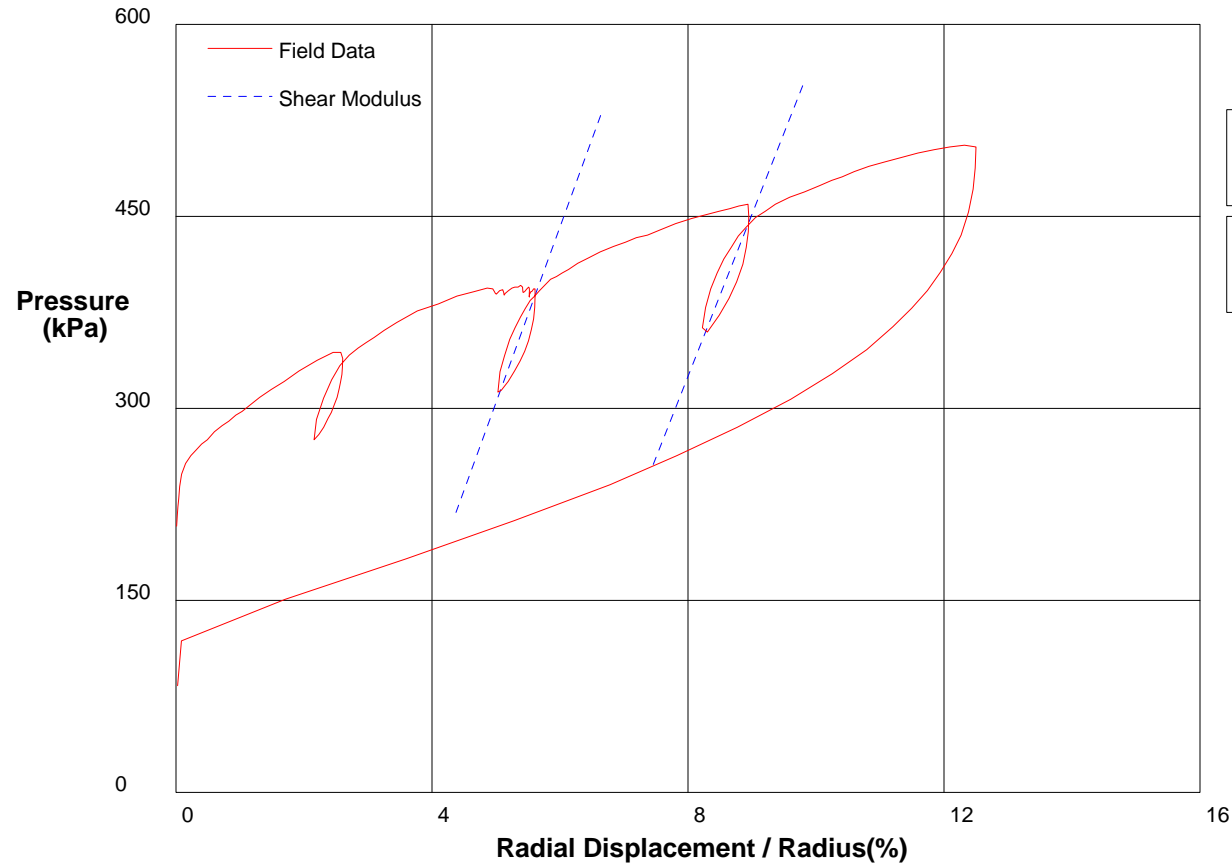
Shear Modulus 6857 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12m	File C:\DATA\SE-835\LD12.P



Shear Modulus 6339 kPa

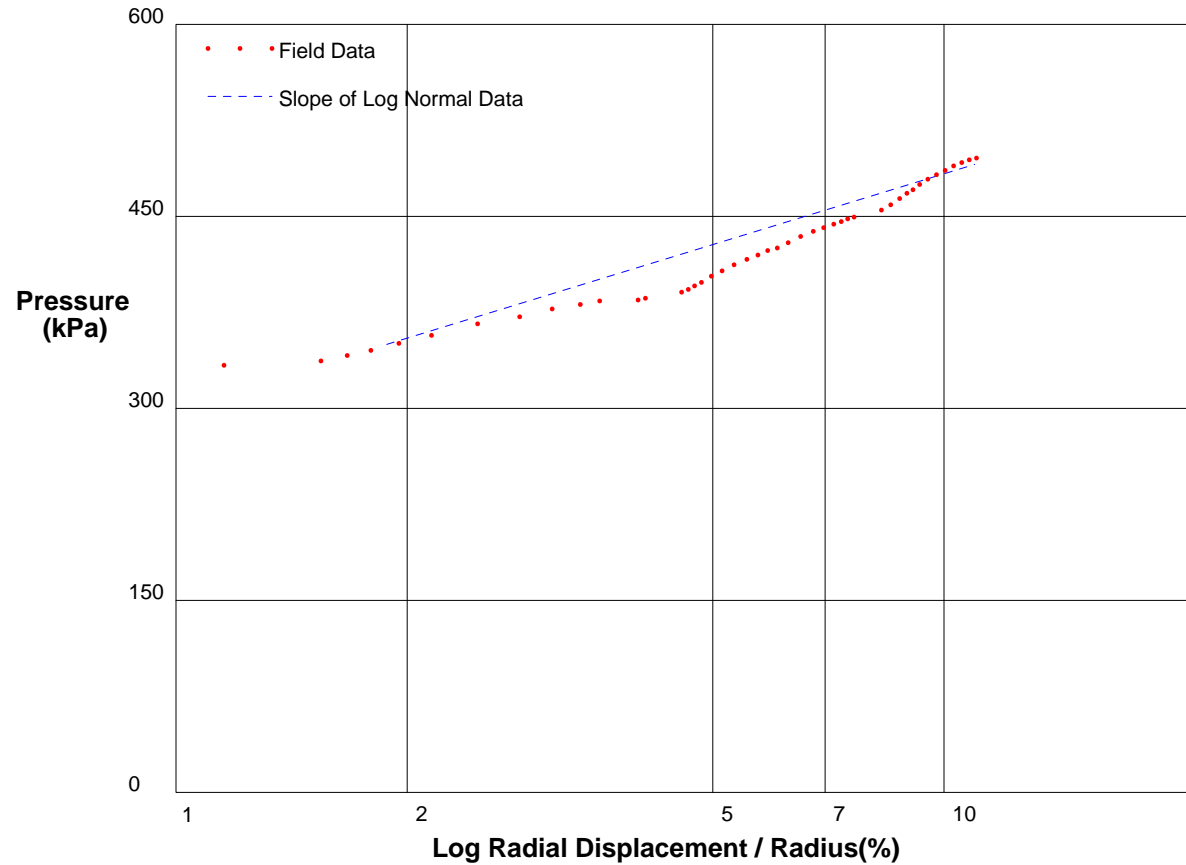
Shear Modulus 6857 kPa

shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12m	File C:\DATA\ISE-835\LD12.P



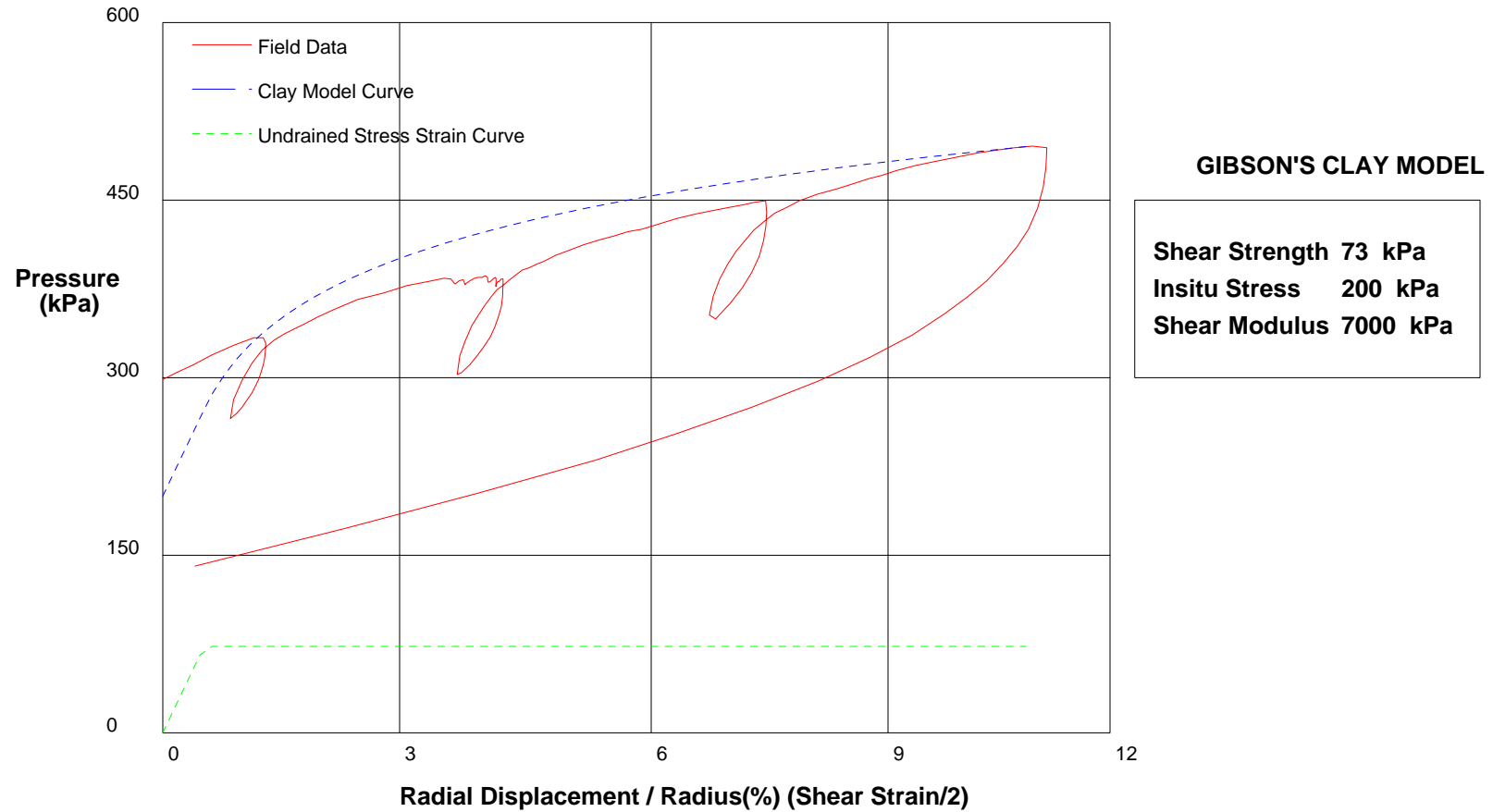
Shear Strength 79.7 kPa
Limit Pressure 595 kPa

shift 1.3

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12m	File C:\DATA\SE-835\LD12.P

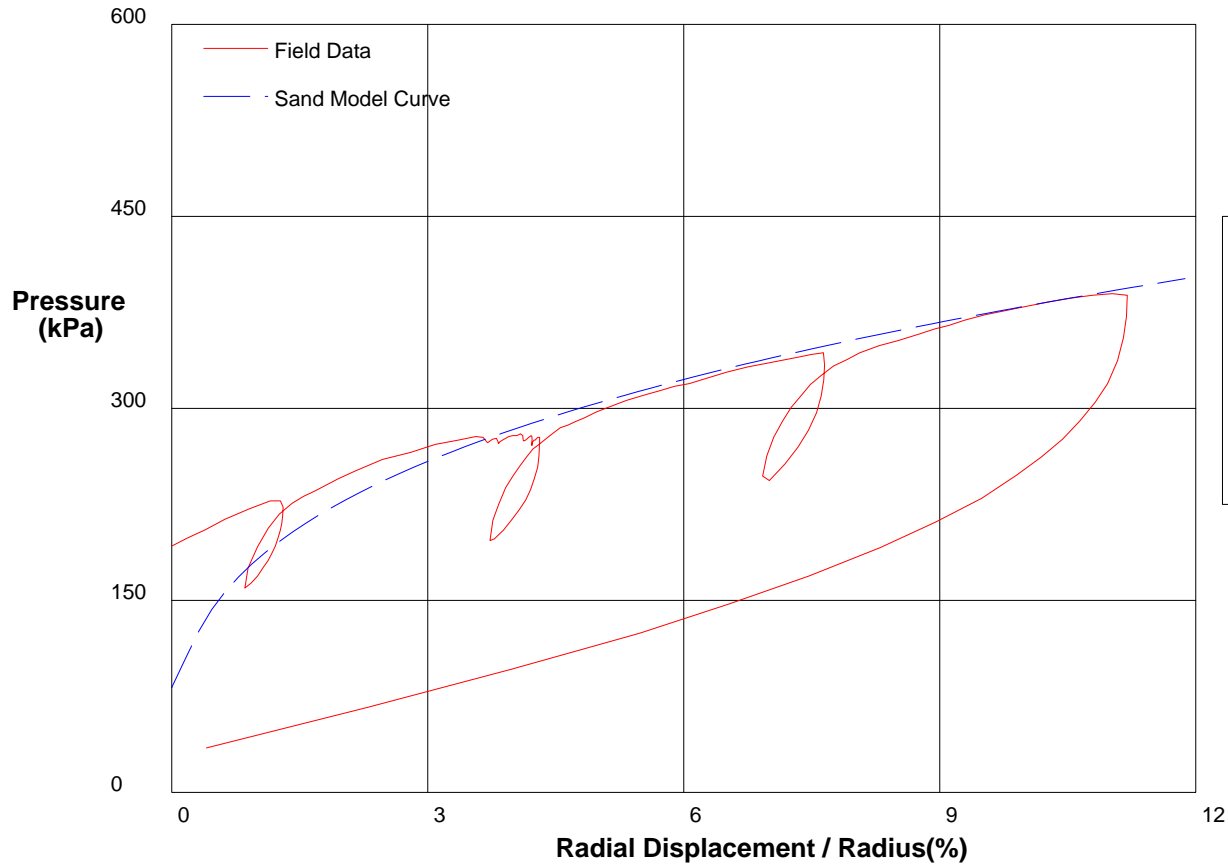


shift 1.3

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12m	File C:\DATA\ISE-835\LD12.P



THE HUGHES SAND MODEL

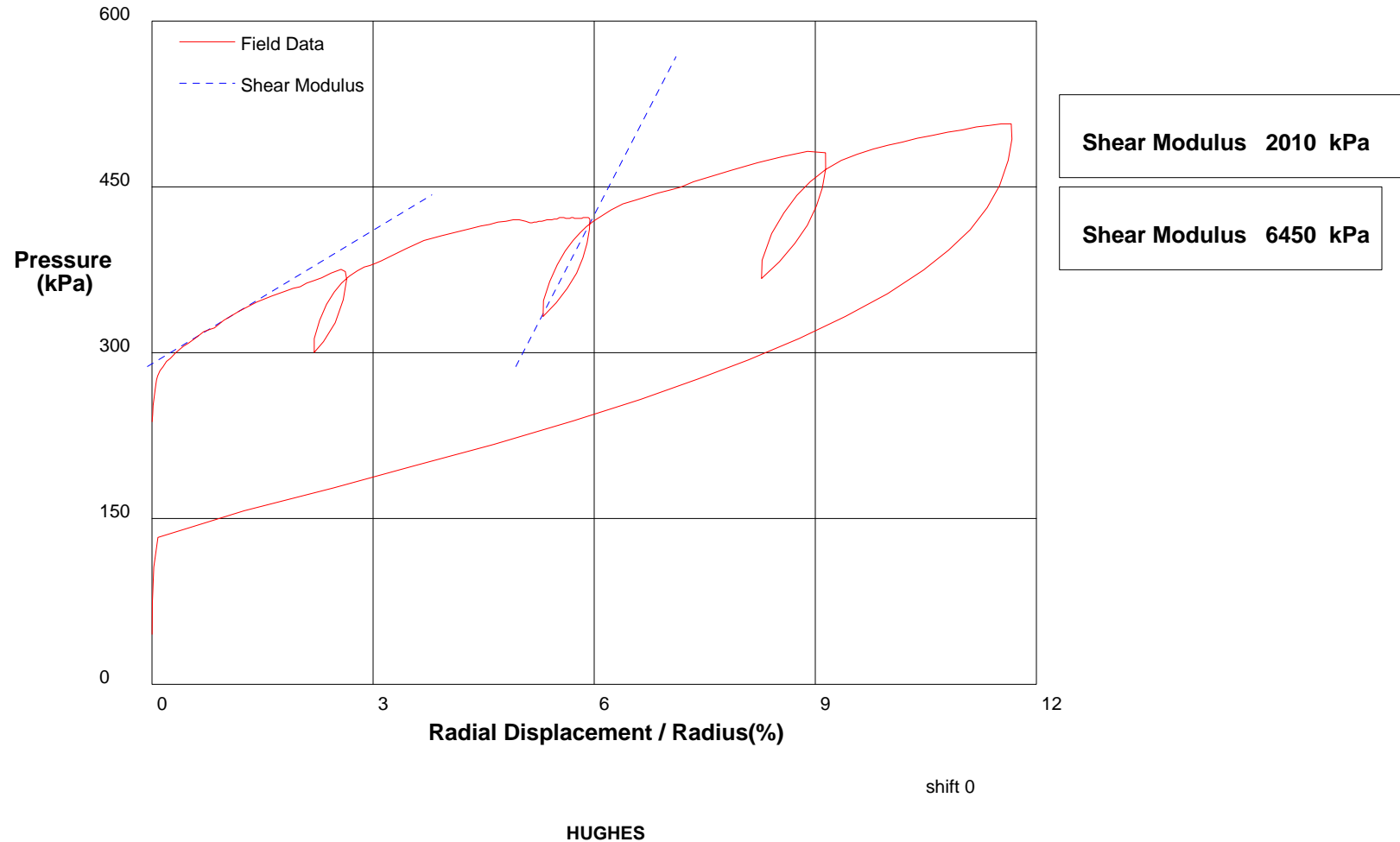
Water Pressure	106 kPa
Friction Angle	28 deg
Critical Friction Angle	28 deg
Lateral Stress	82 kPa
Shear Modulus	7000 kPa

shift 1.3

HUGHES

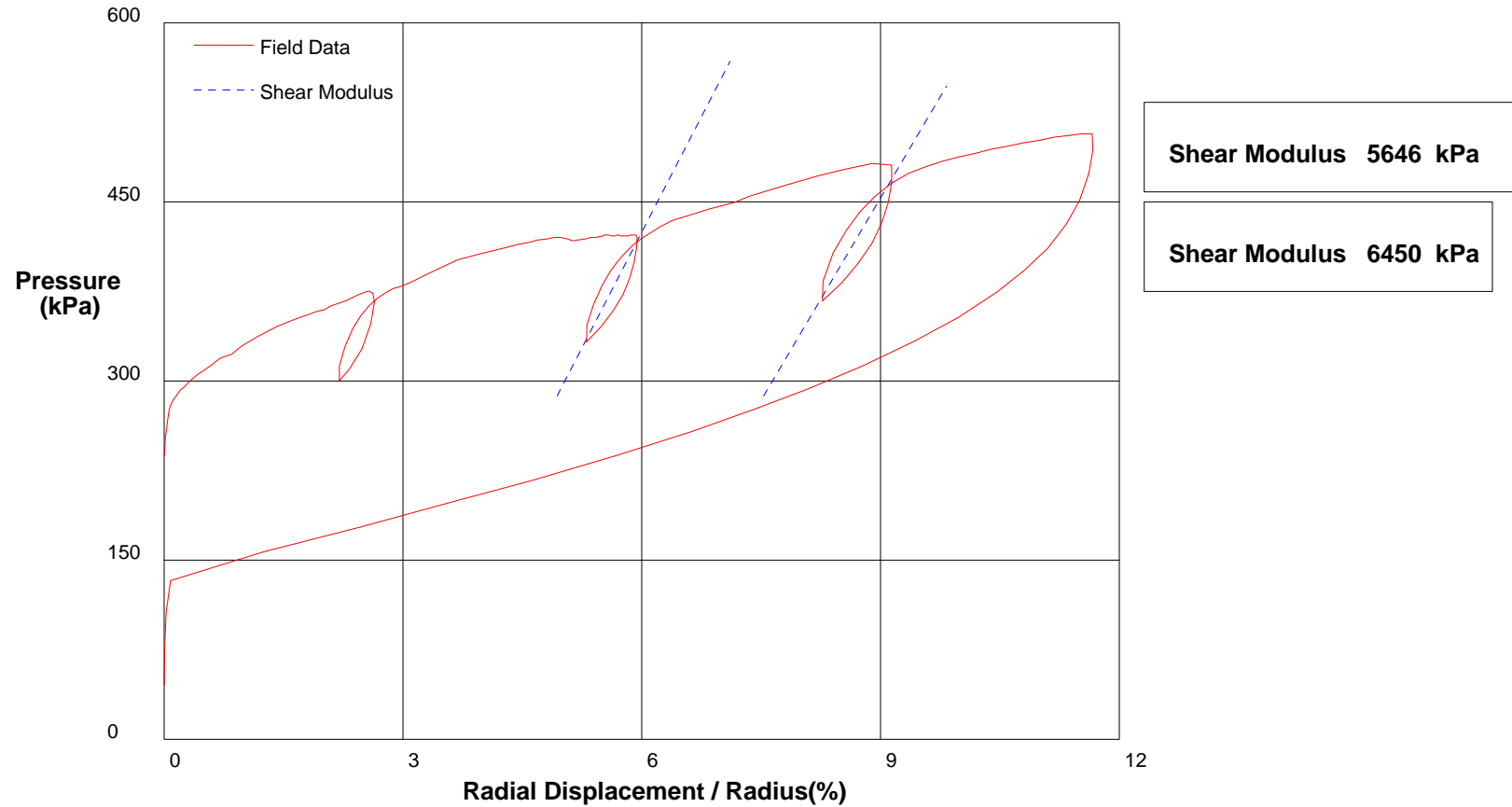
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12.75m	File C:\DATA\SE-835\LD13.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12.75m	File C:\DATA\SE-835\LD13.P

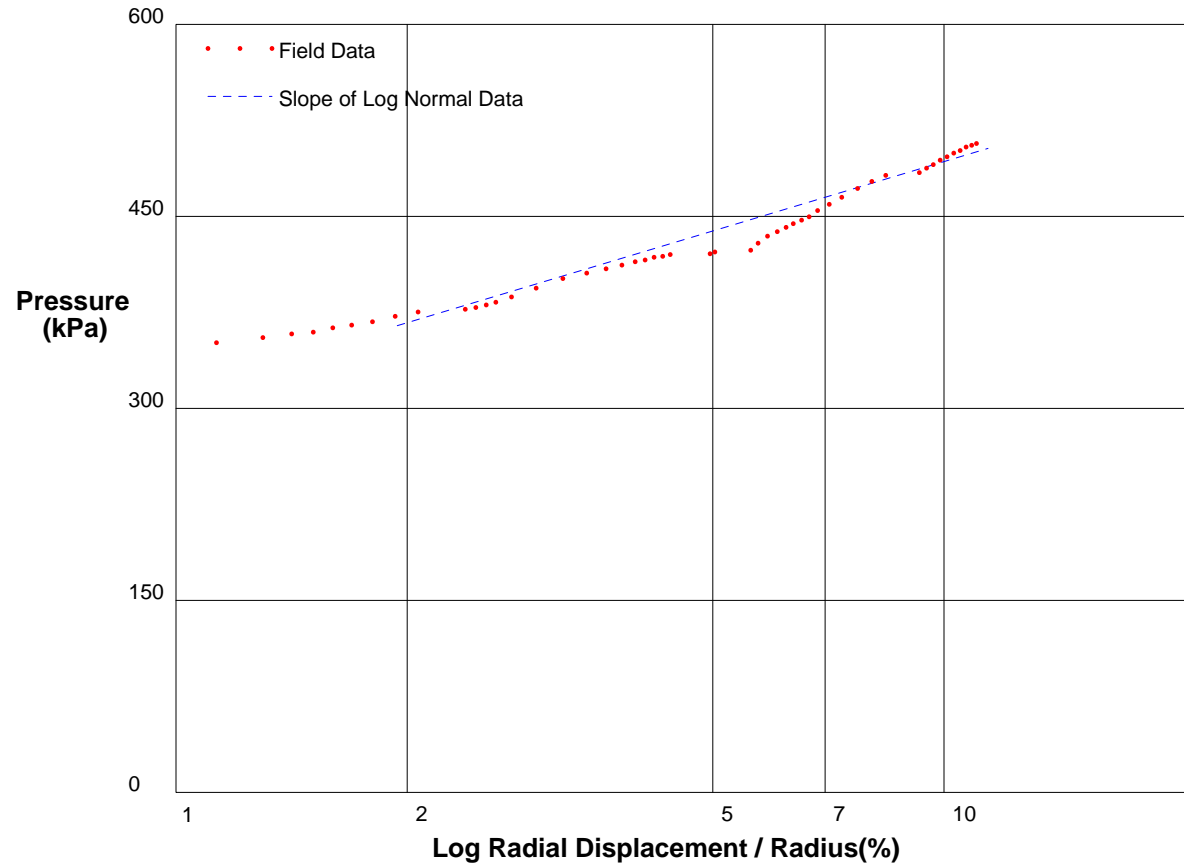


shift 0

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12.75m	File C:\DATA\ISE-835\LD13.P



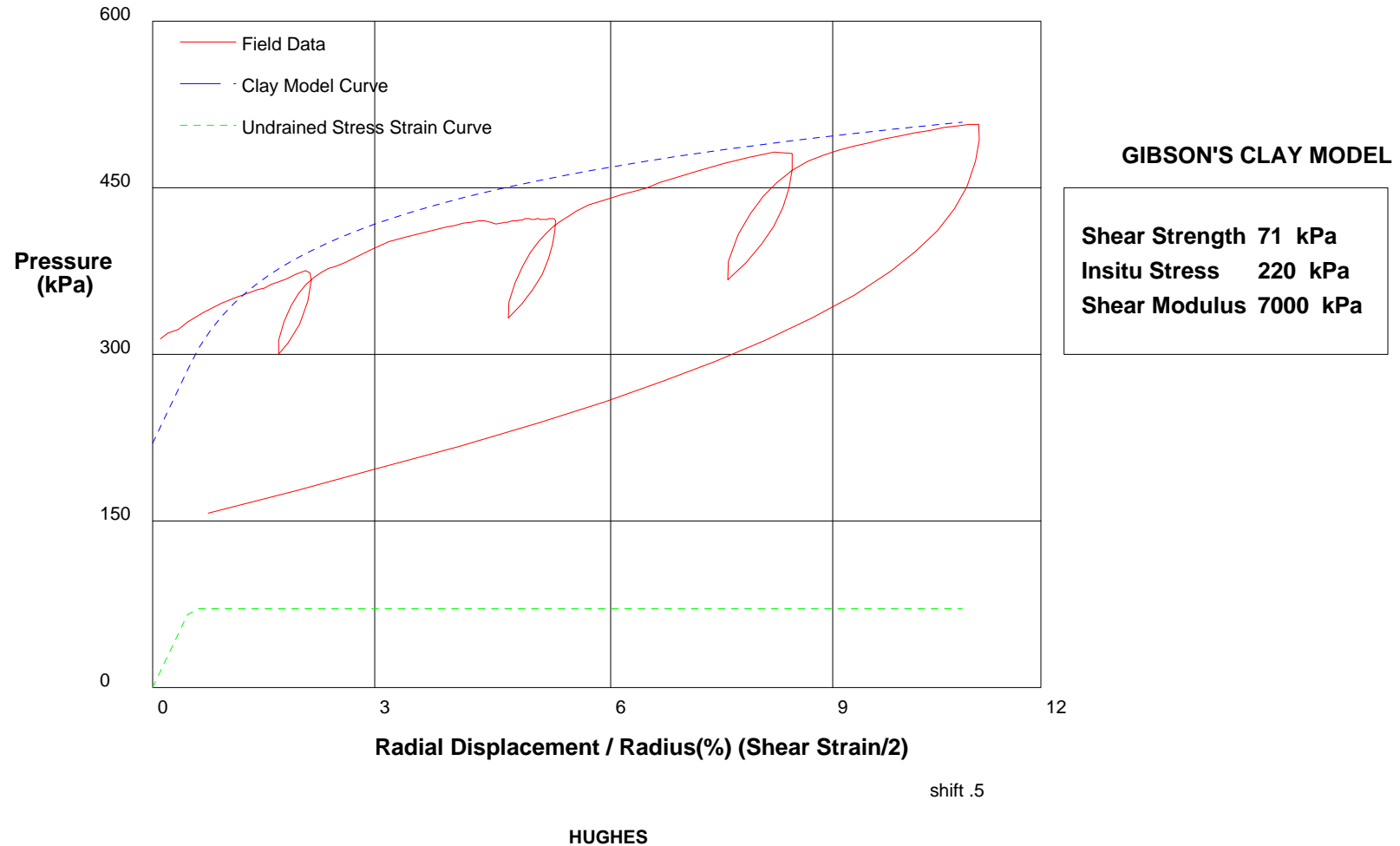
Shear Strength 78.2 kPa
Limit Pressure 603 kPa

shift .5

HUGHES

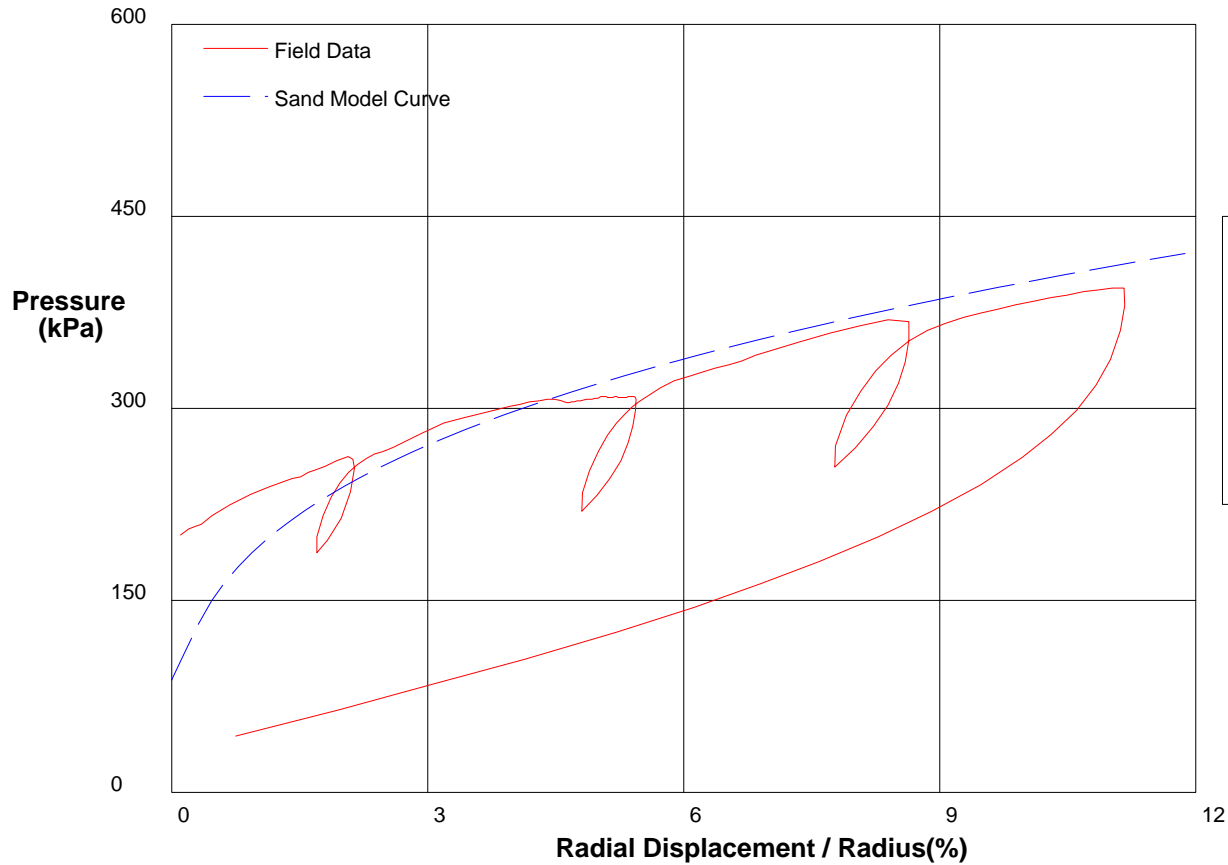
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12.75m	File C:\DATA\SE-835\LD13.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 12.75m	File C:\DATA\ISE-835\LD13.P



THE HUGHES SAND MODEL

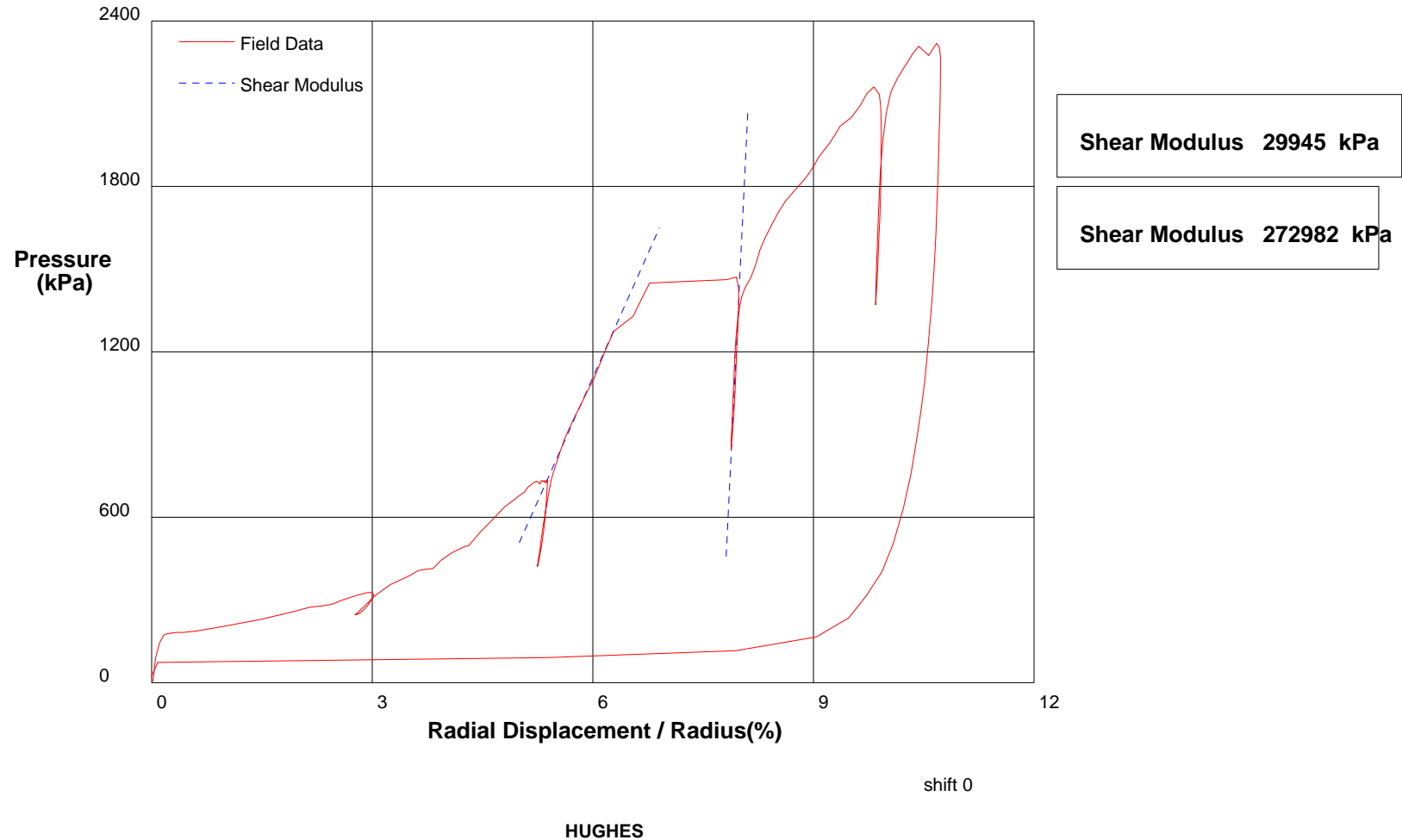
Water Pressure	113 kPa
Friction Angle	28 deg
Critical Friction Angle	28 deg
Lateral Stress	88 kPa
Shear Modulus	7000 kPa

shift .5

HUGHES

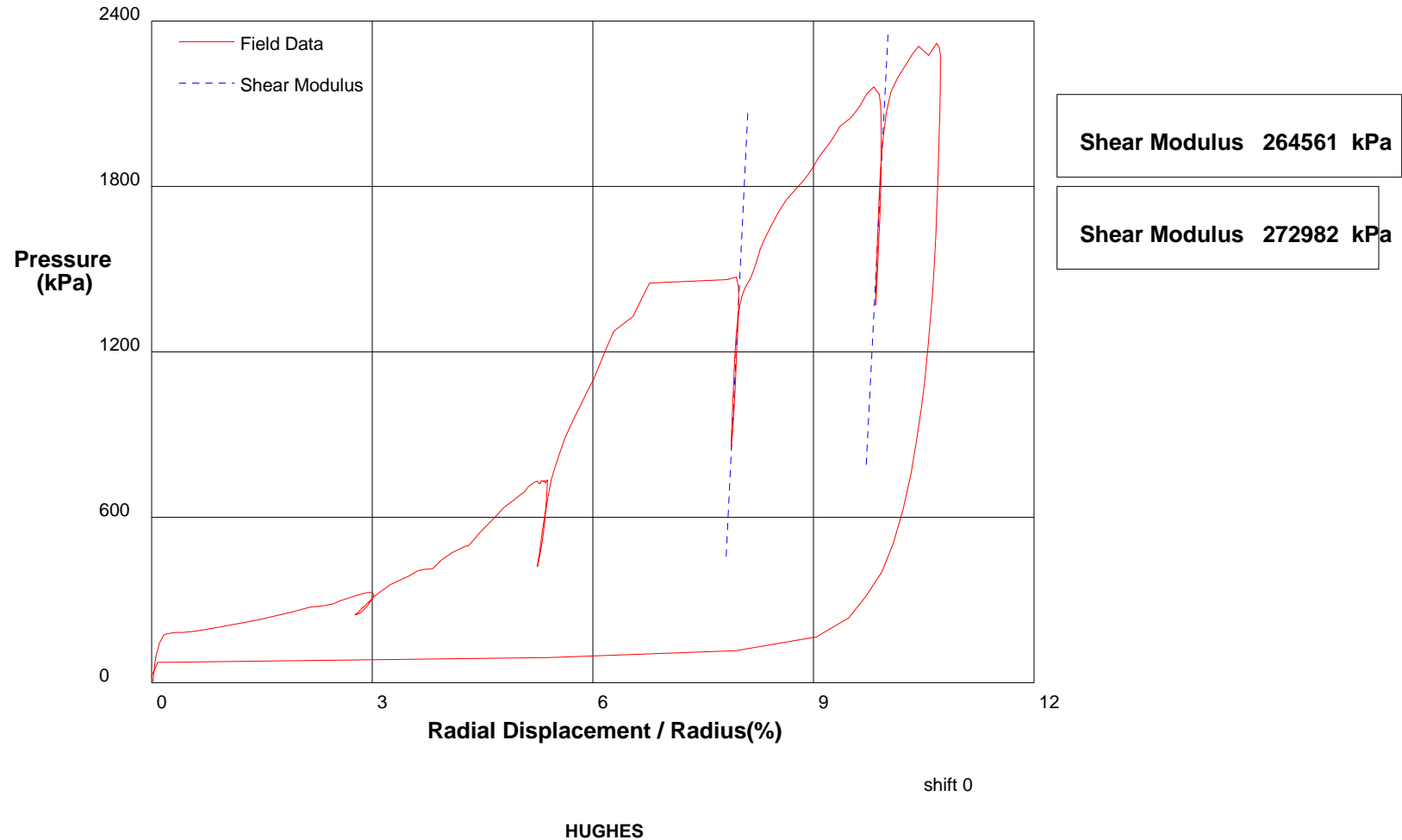
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 15.3m	File C:\DATA\SE-835\LD14.P



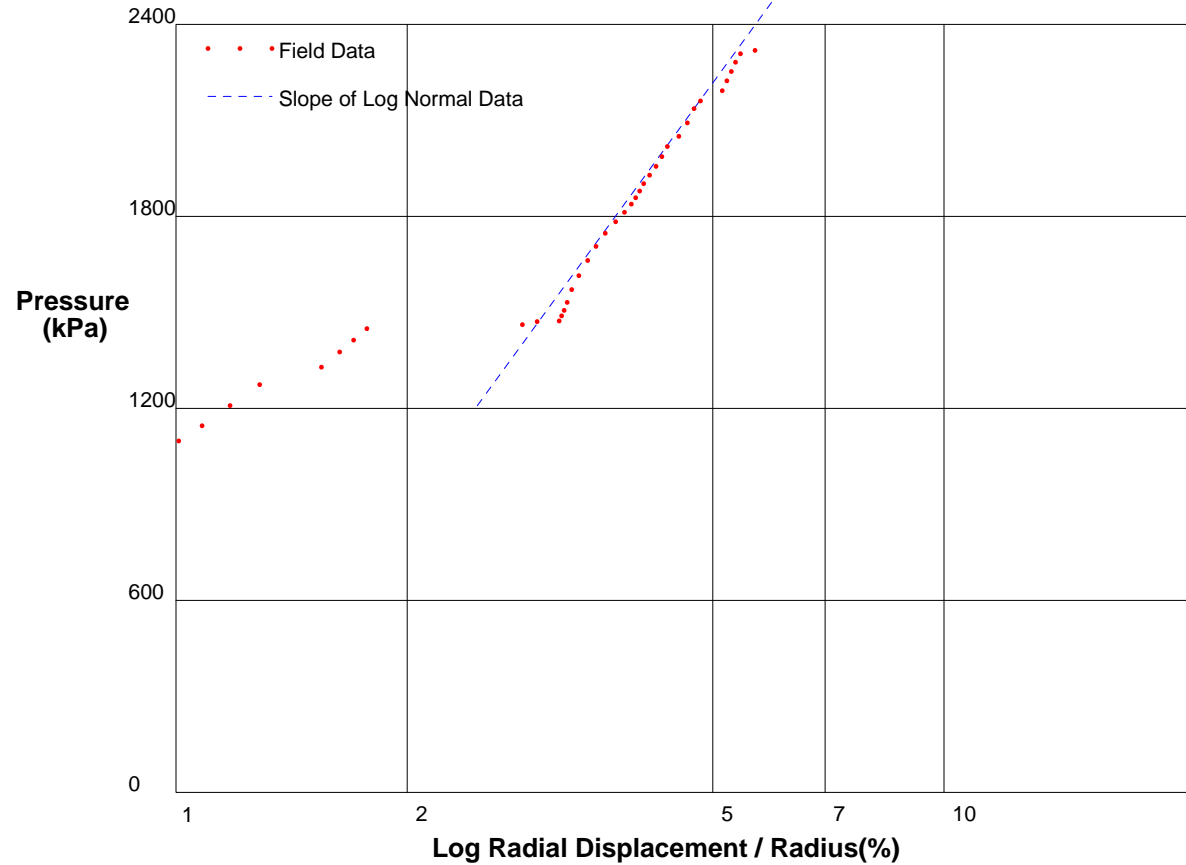
Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 15.3m	File C:\DATA\SE-835\LD14.P



Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 15.3m	File C:\DATA\ISE-835\LD14.P



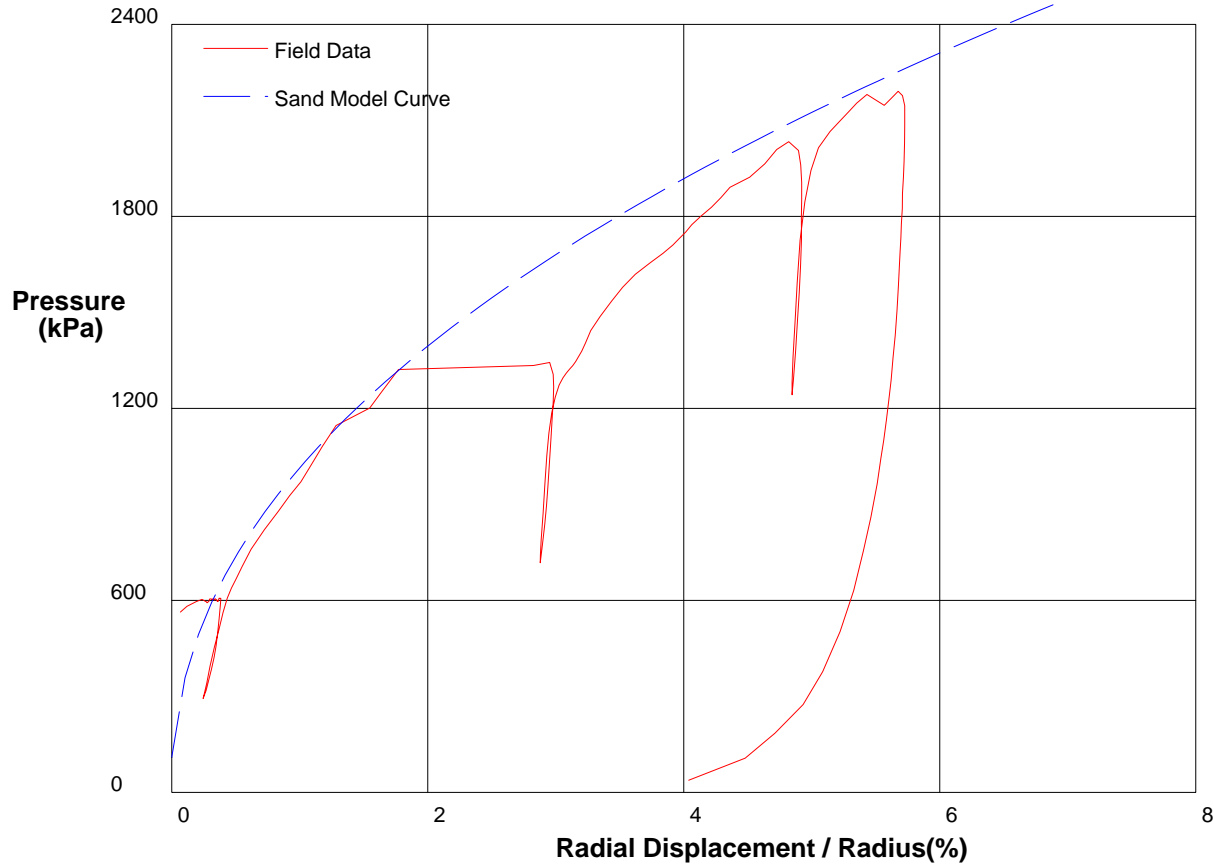
Shear Strength 1428.3 kPa
Limit Pressure 5222 kPa

shift 5

HUGHES

Appendix I - Pressuremeter Data and Analysis

PRESSUREMETER DATA		Thurber Engineering, Ltd.
Leamington Dock Temporary Works		4/20/2009
Hole No. PM09-1S	Depth 15.3m	File C:\DATA\ISE-835\LD14.P



THE HUGHES SAND MODEL

Water Pressure	128 kPa
Friction Angle	38 deg
Critical Friction Angle	28 deg
Lateral Stress	110 kPa
Shear Modulus	150000 kPa

shift 5

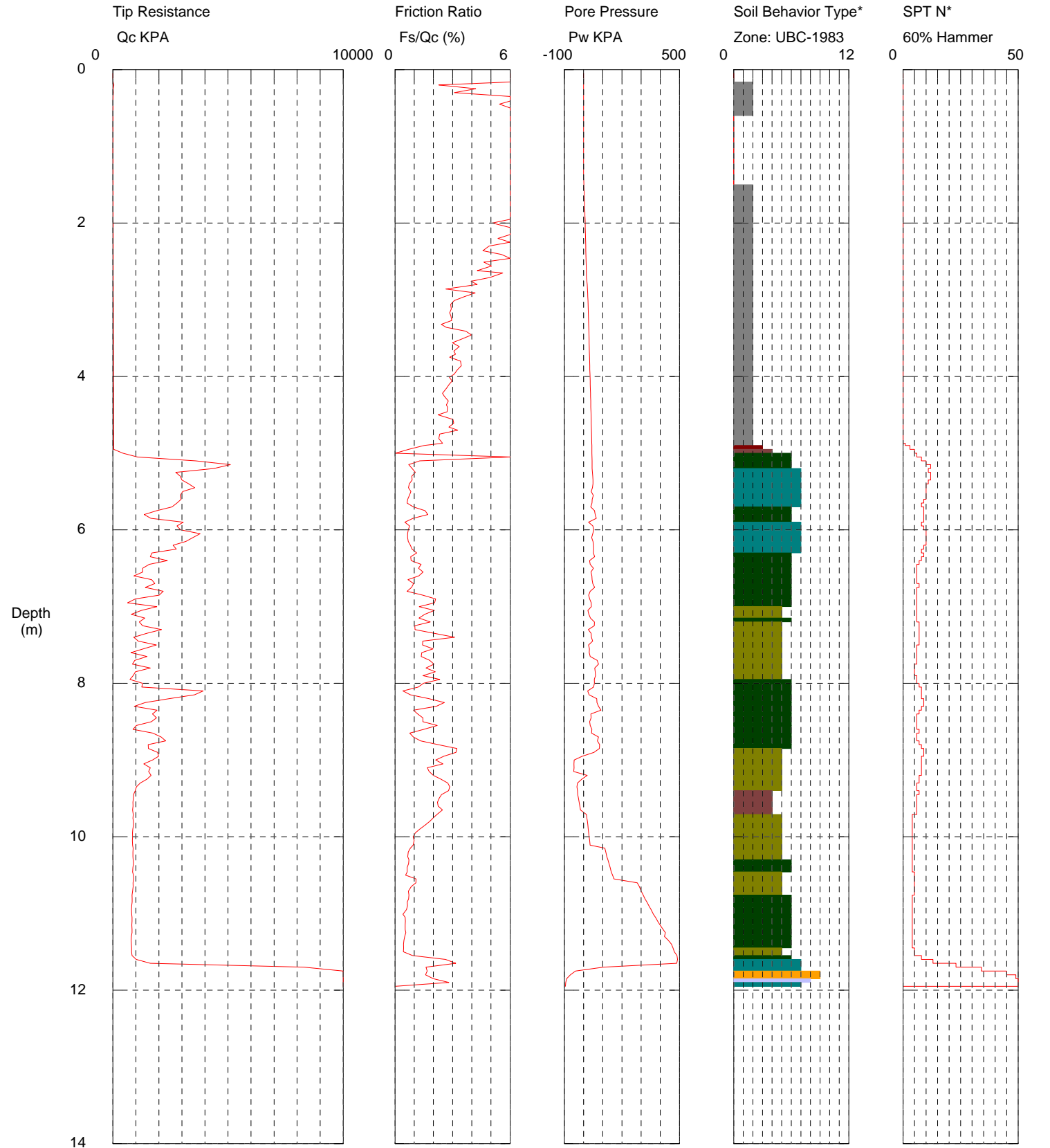
HUGHES

APPENDIX II - CONE PENETROMETER DATA AND ANALYSIS

Thurber Engineering, Ltd

Operator: Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92



Maximum Depth = 11.95 meters

Depth Increment = 0.070 meters

- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

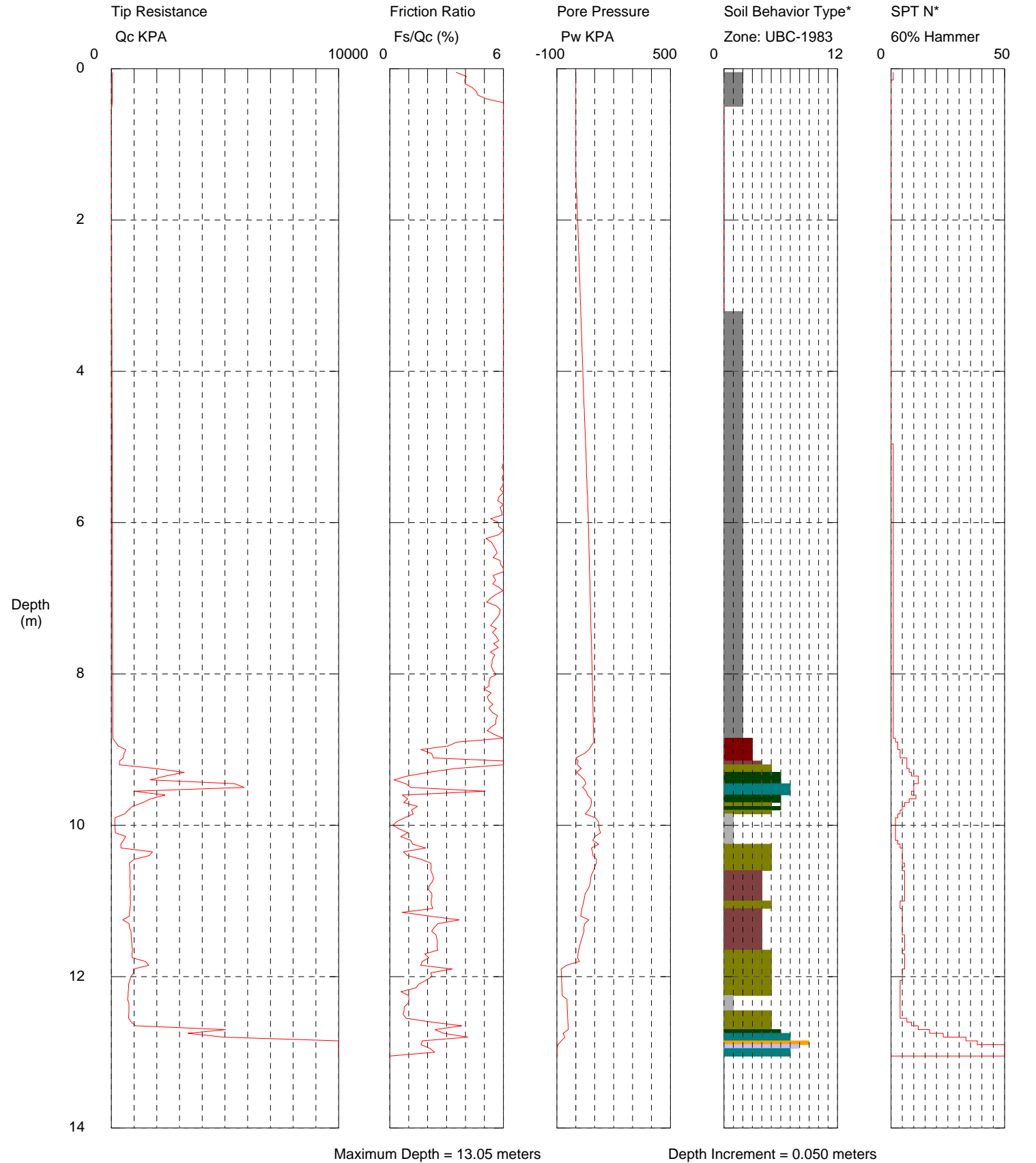
Test depth began at dock surface. The location was pre-drilled through fill behind Situ Engineering.

*Soil behavior type and SPT based on data from UBC-1983

Thurber Engineering, Ltd

Operator: Dafni
Sounding: CPT09-2S
Cone Used: DSG1065

CPT Date/Time: 4/18/2009 5:04:11 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92



- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

Test depth began at dock surface. The test was performed over the side of the dock. Mudline was approximately 9m below the deck.

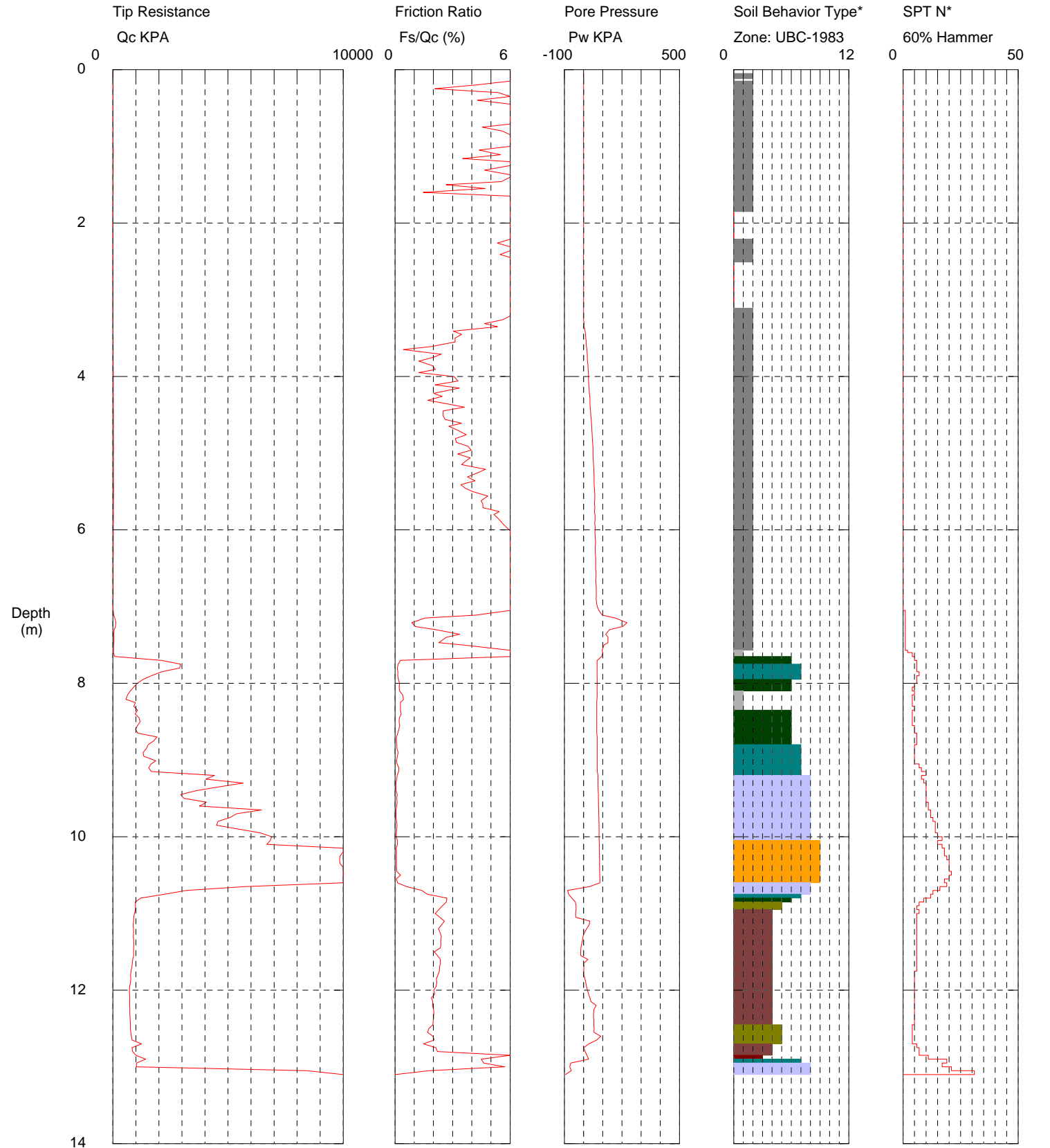
*Soil behavior type and SPT based on data from UBC-1983

In Situ Engineering

Thurber Engineering, Ltd

Operator: Dafni
Sounding: CPT09-3S
Cone Used: DSG1065

CPT Date/Time: 4/17/2009 6:59:39 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92



Maximum Depth = 13.10 meters

Depth Increment = 0.070 meters

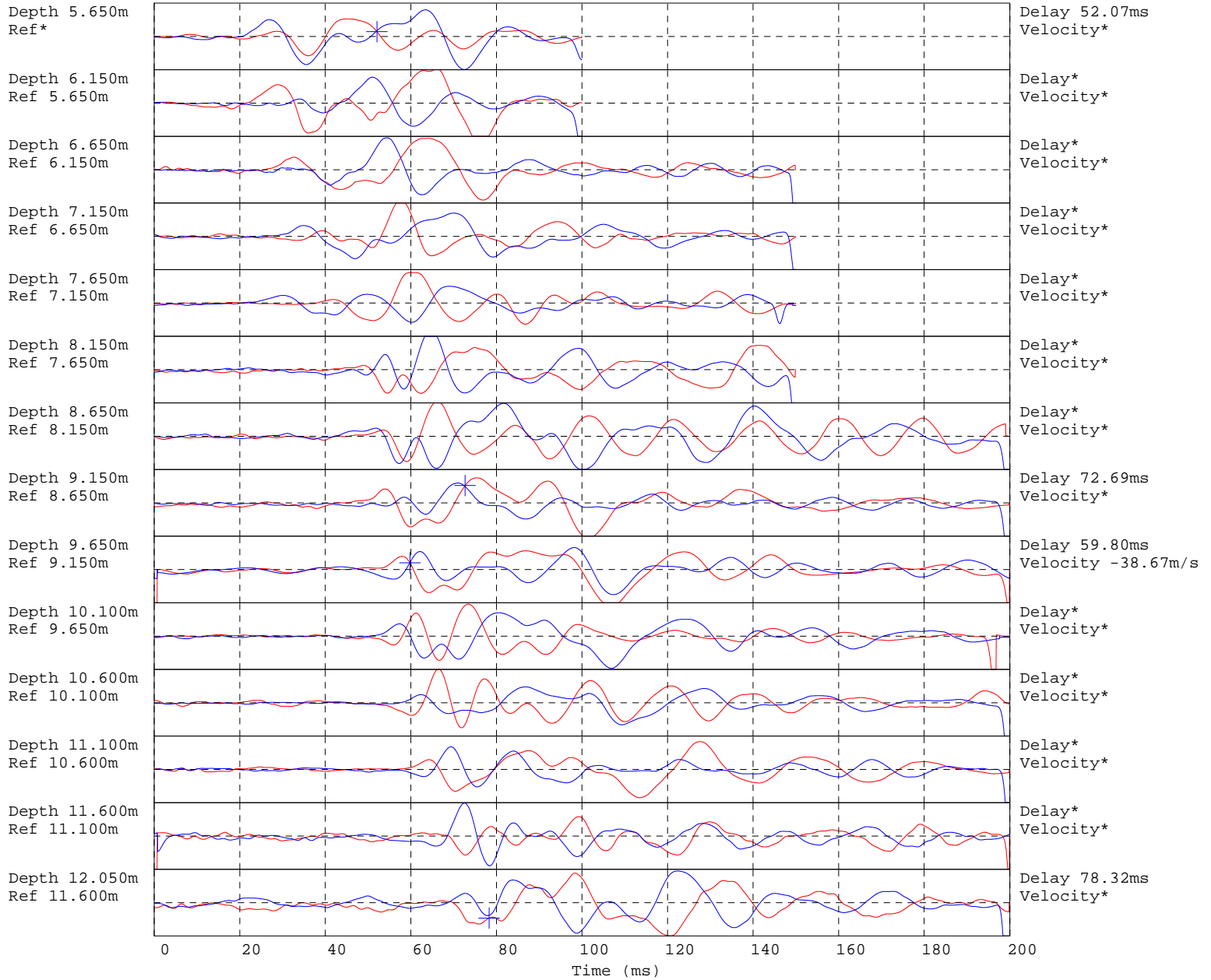
- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

Test depth began at dock surface. The test was performed over the side of the dock. Mudline was encountered at 7.7m depth.

*Soil behavior type and SPT based on data from UBC-1983

In Situ Engineering

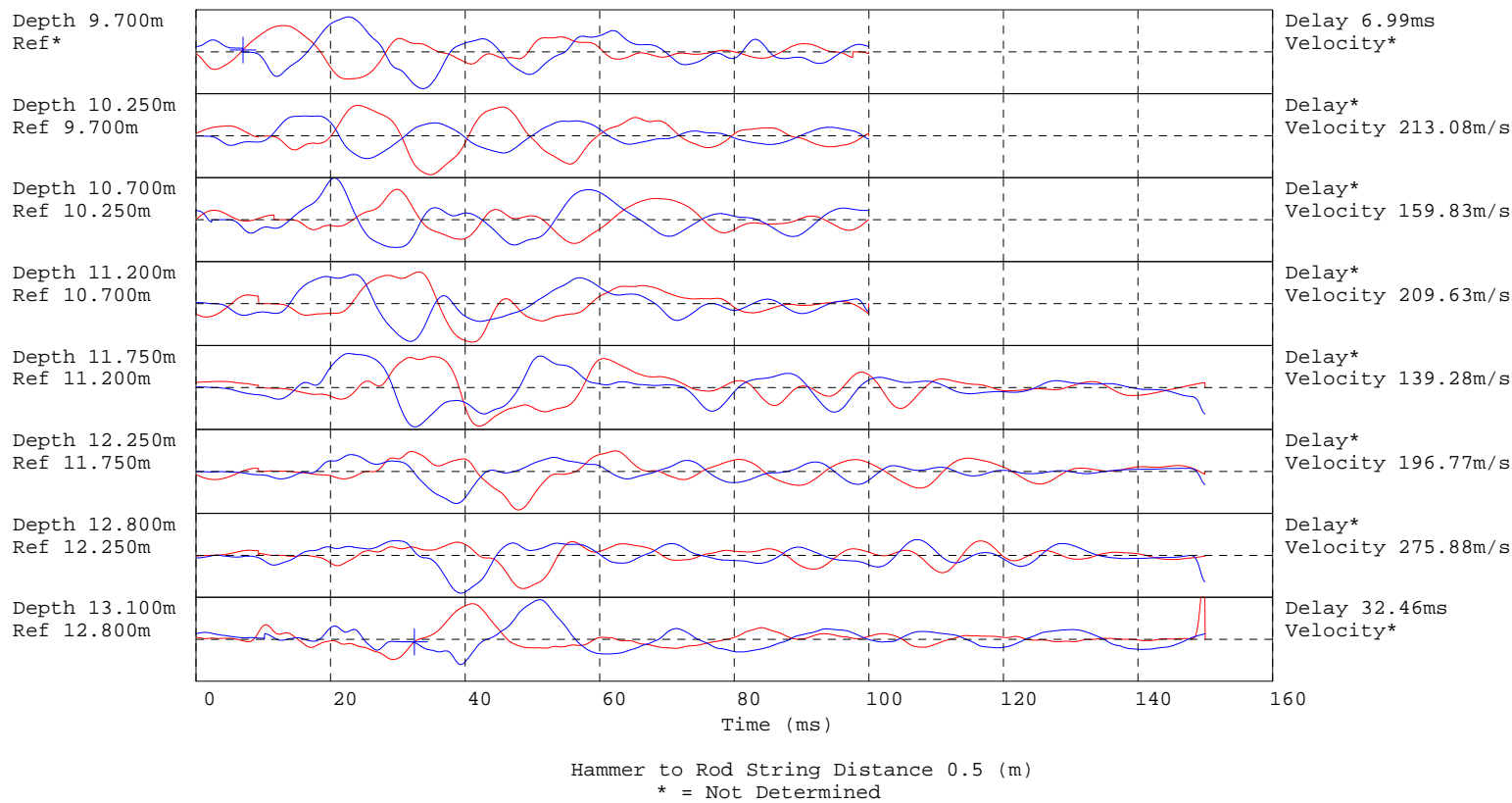
Thurber Engineering, Ltd.
Shear Wave Velocity Analysis - CPT09-1S



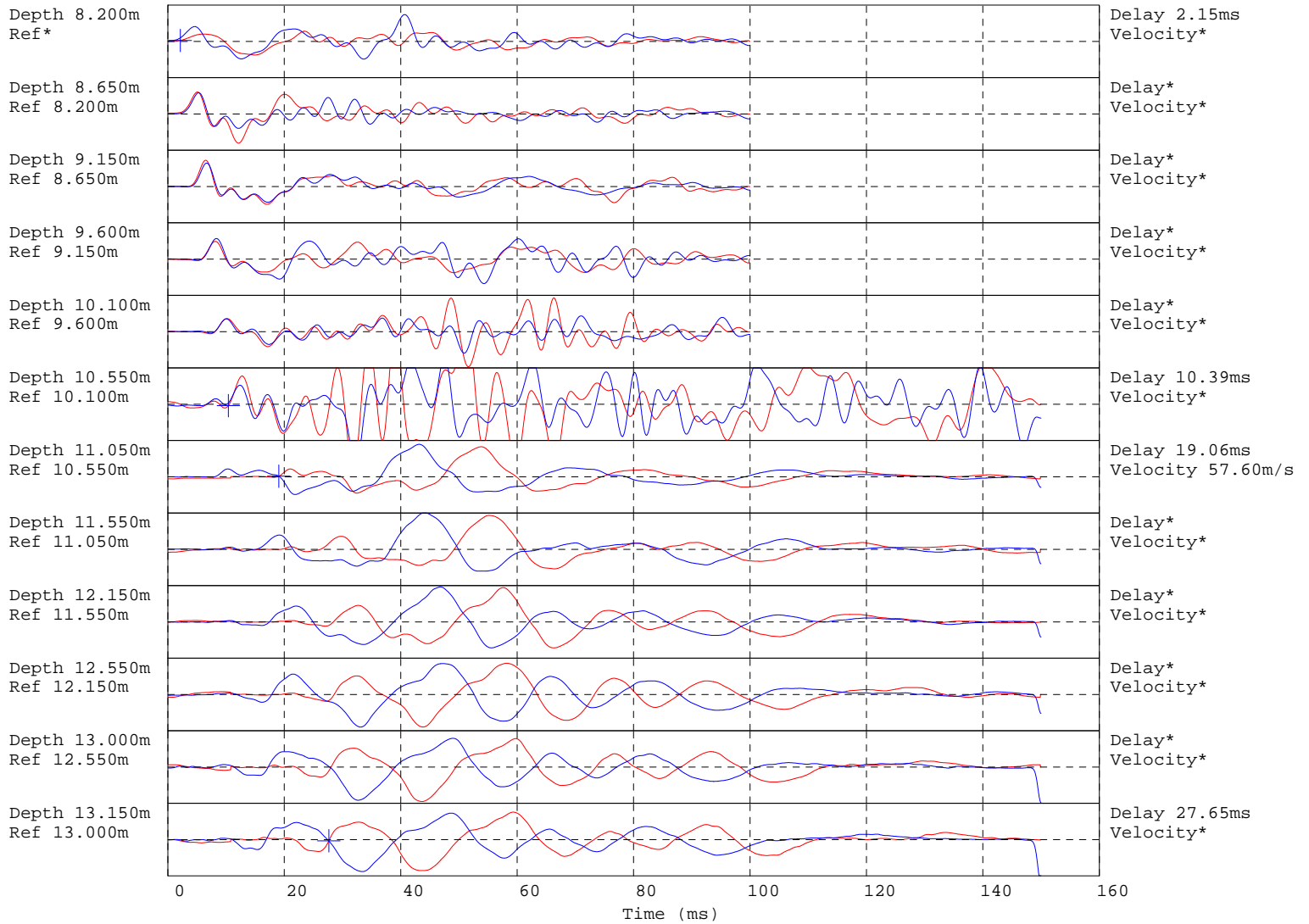
Hammer to Rod String Distance 0.73 (m)

* = Not Determined

Thurber Engineering, Ltd.
Shear Wave Velocity Analysis - CPT09-2S



Thurber Engineering Ltd.
Shear Wave Velocity Analysis - CPT09-03S



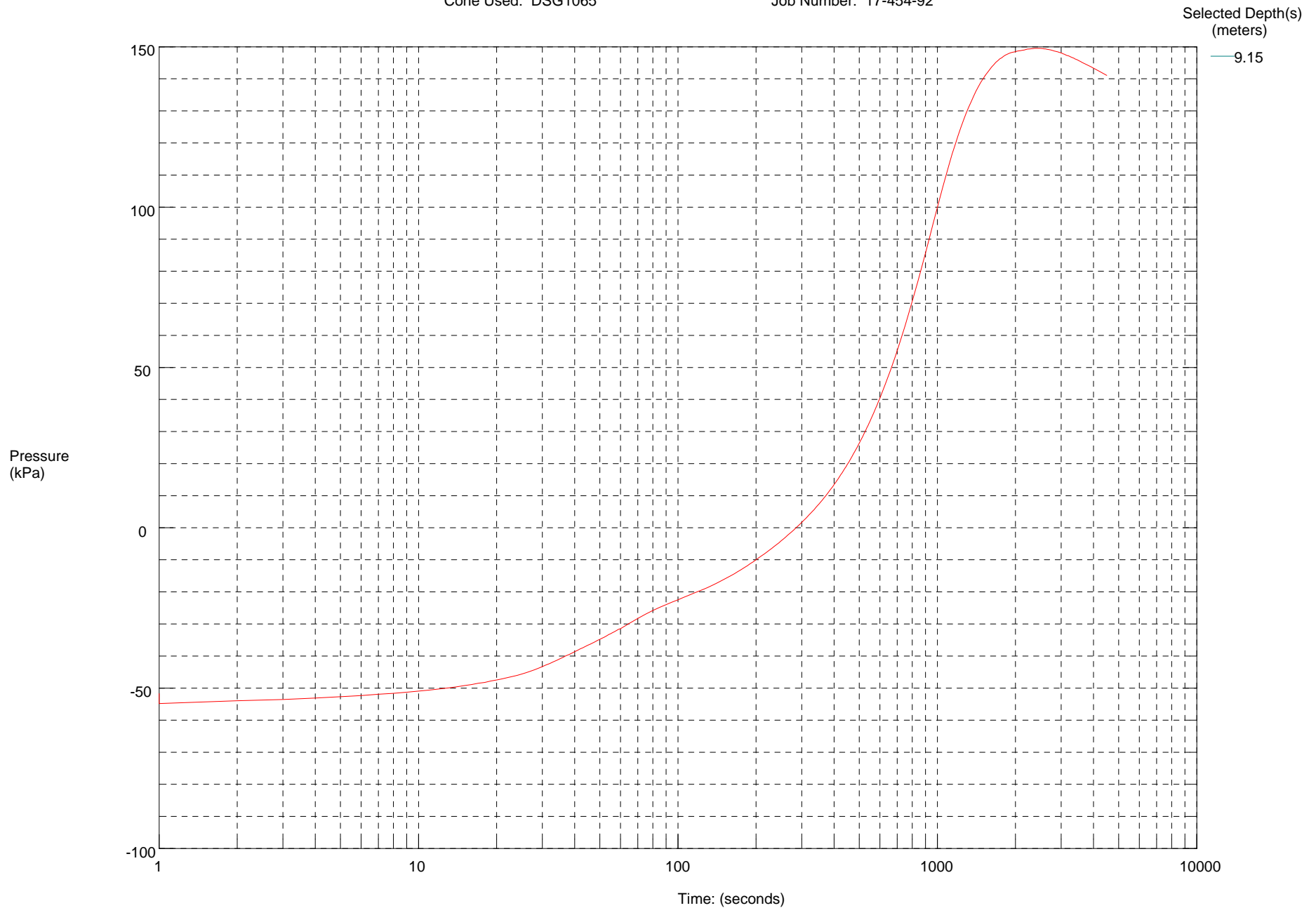
Hammer to Rod String Distance 0.5 (m)

* = Not Determined

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

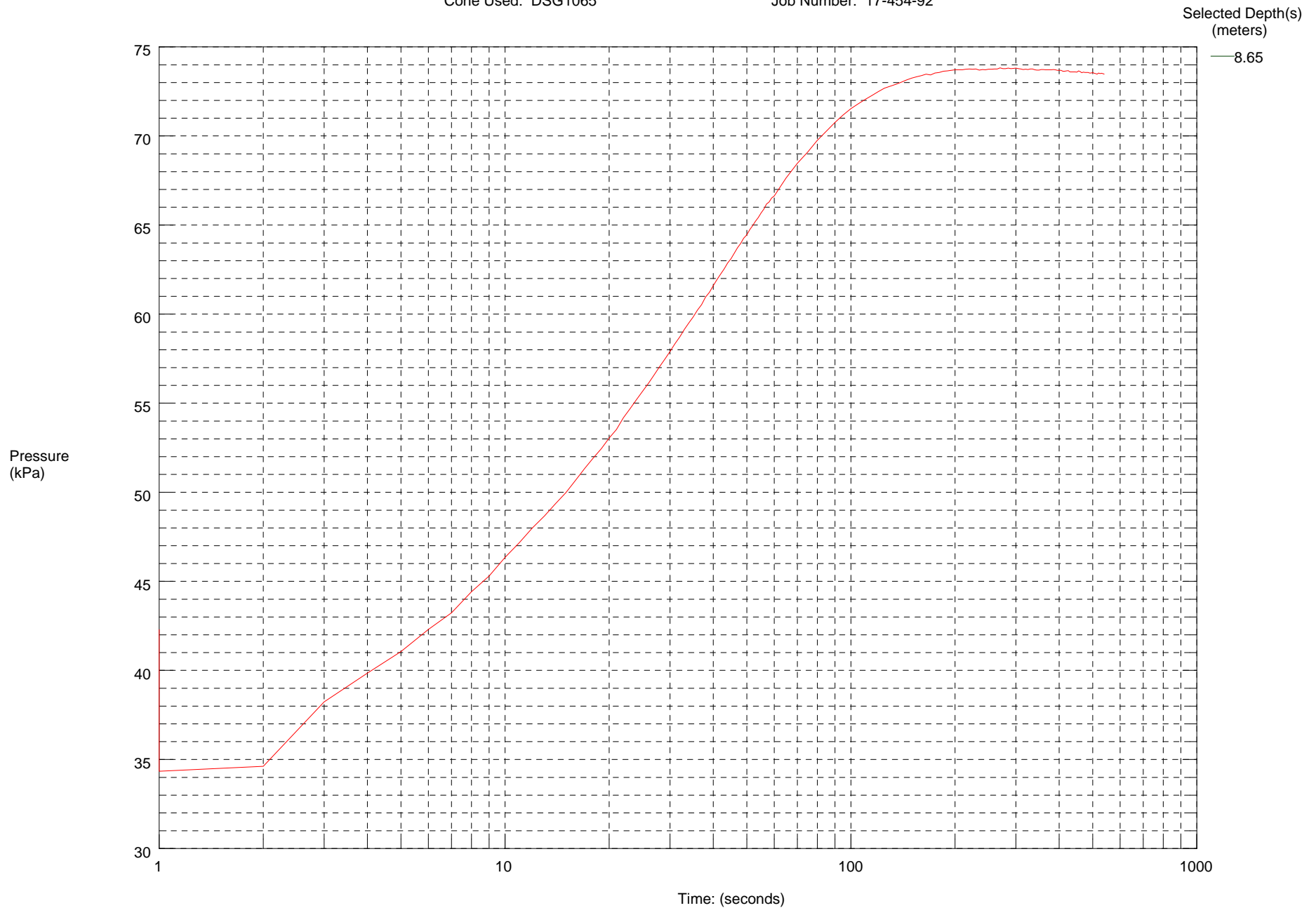


Maximum Pressure = 149.55 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

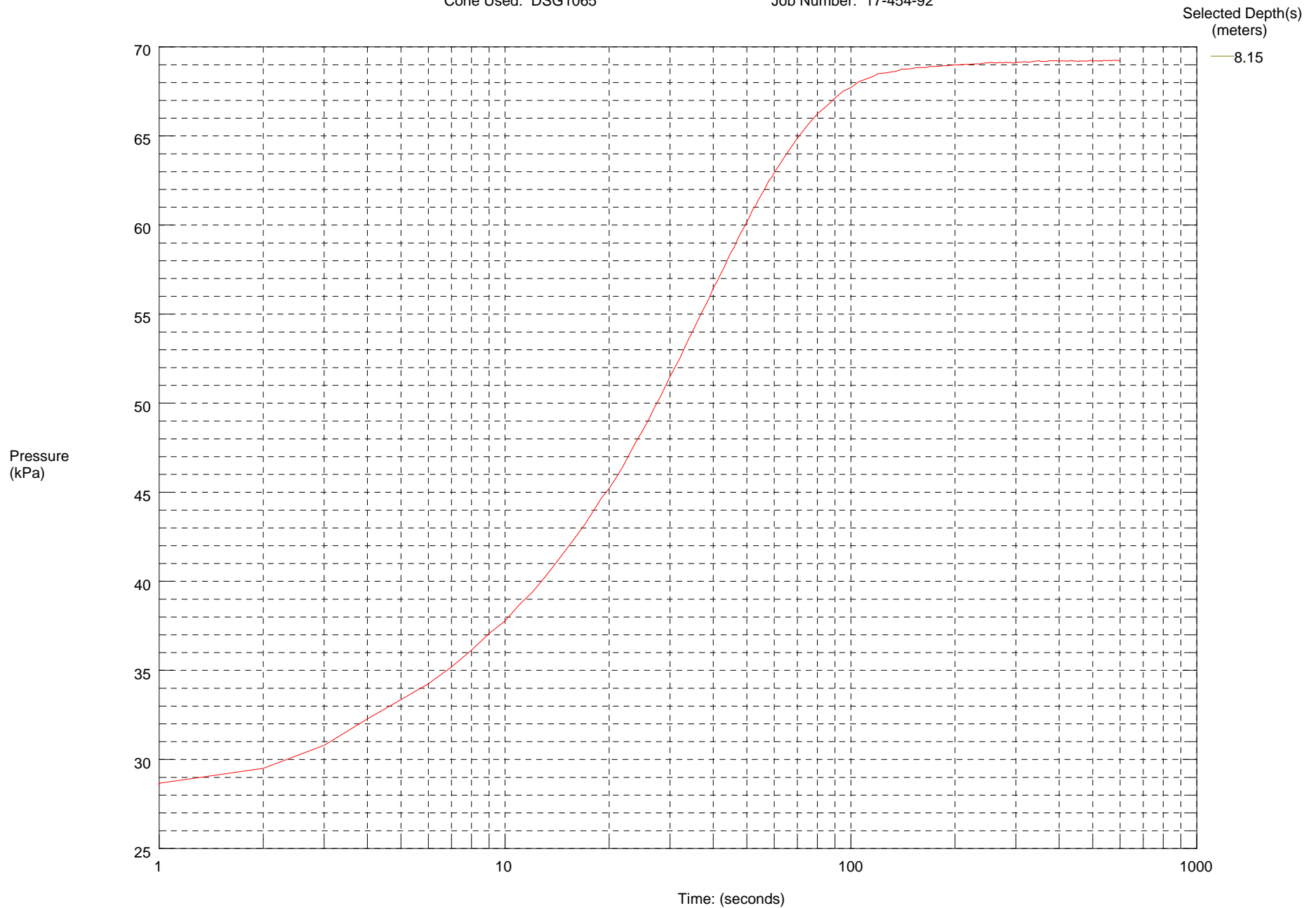


Maximum Pressure = 73.823 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

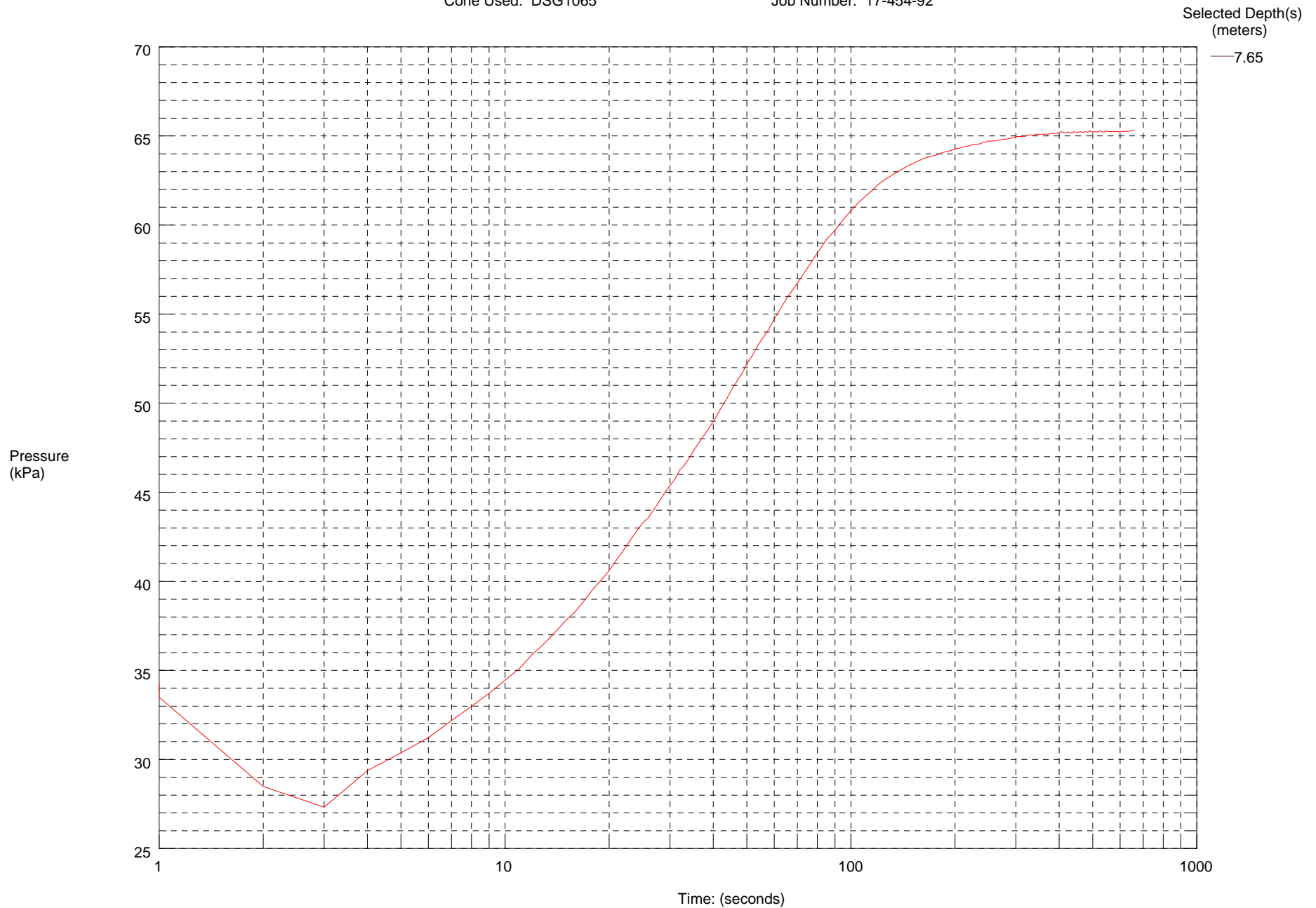


Maximum Pressure = 69.257 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

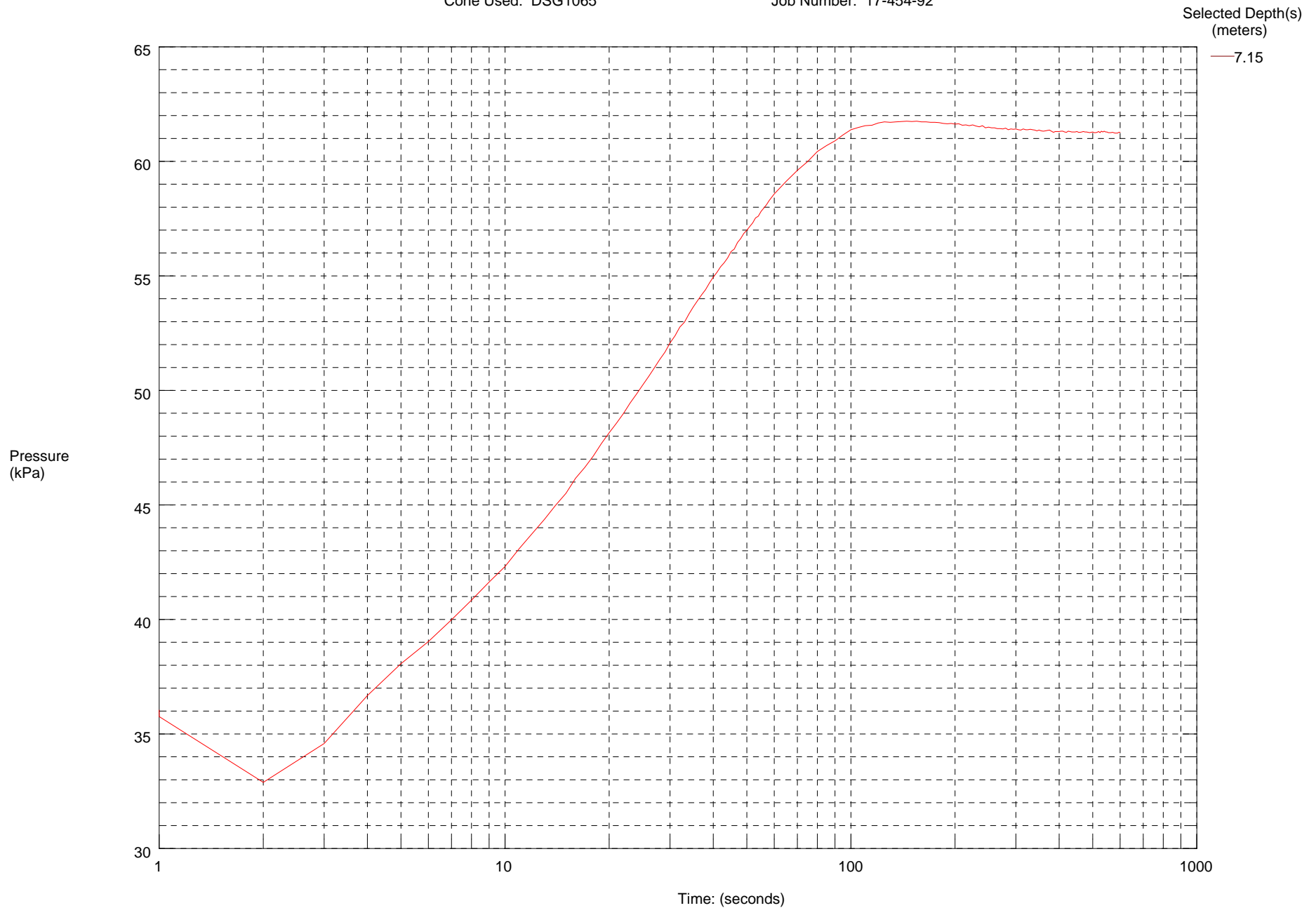


Maximum Pressure = 65.303 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

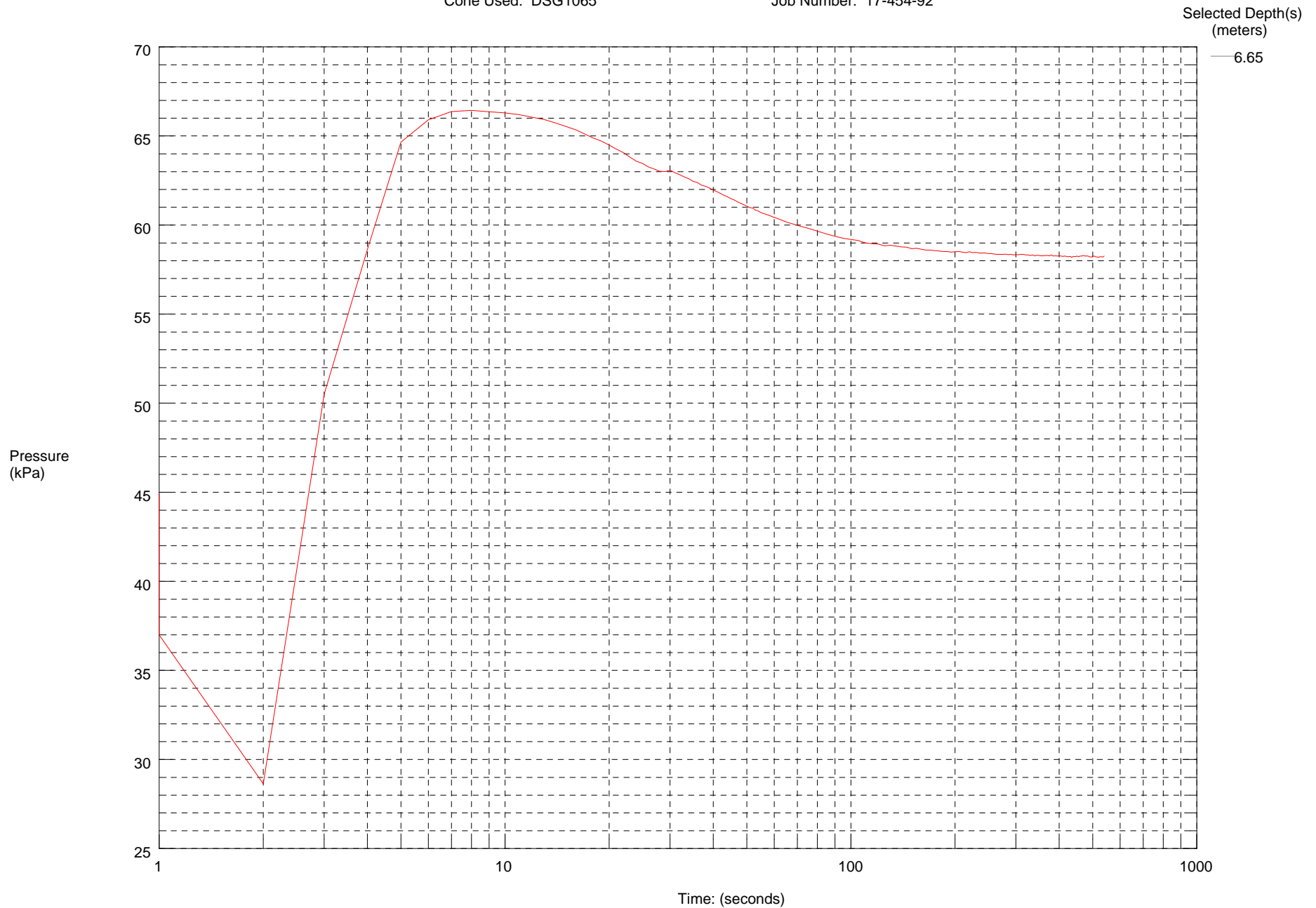


Maximum Pressure = 61.753 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

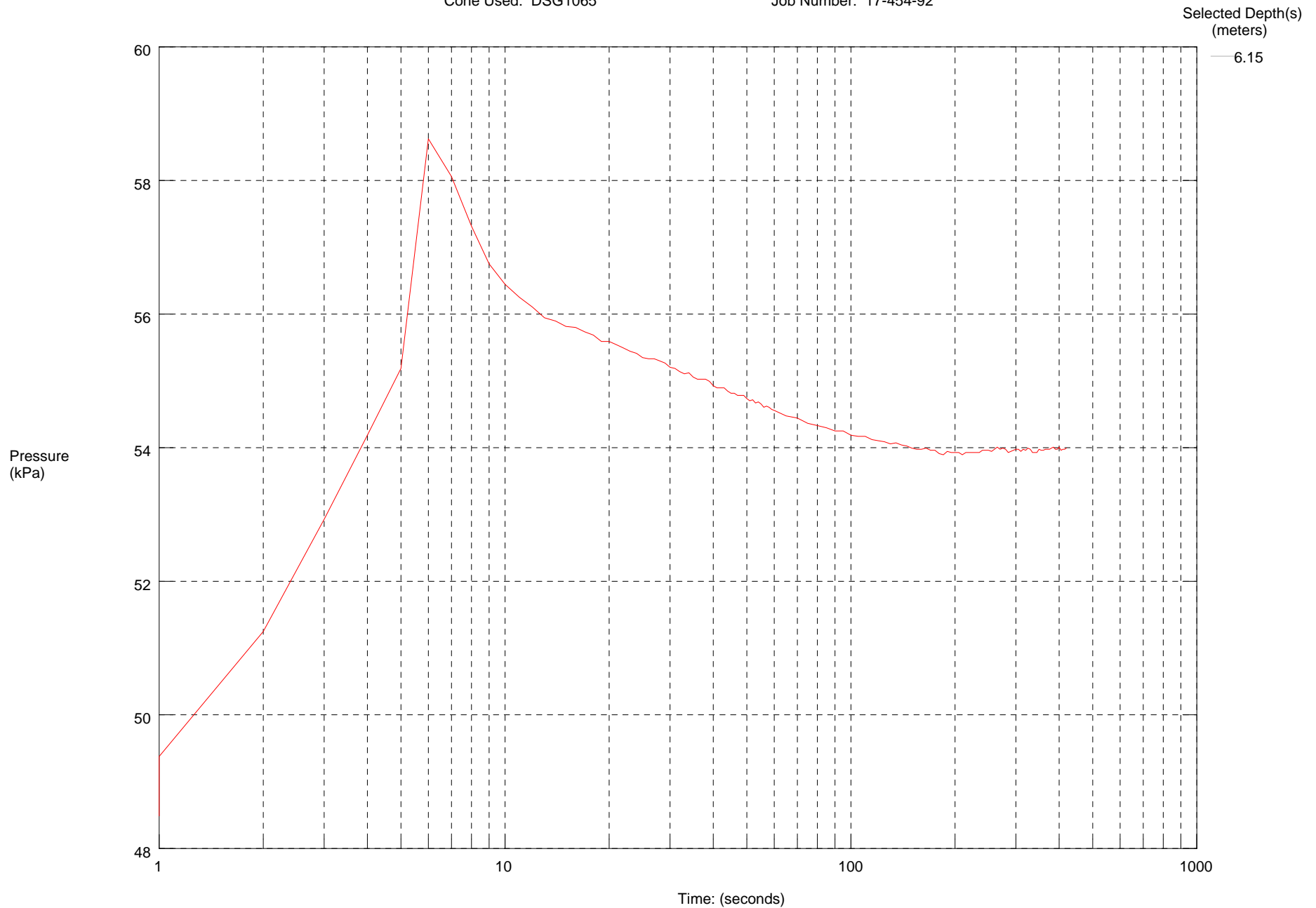


Maximum Pressure = 66.433 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

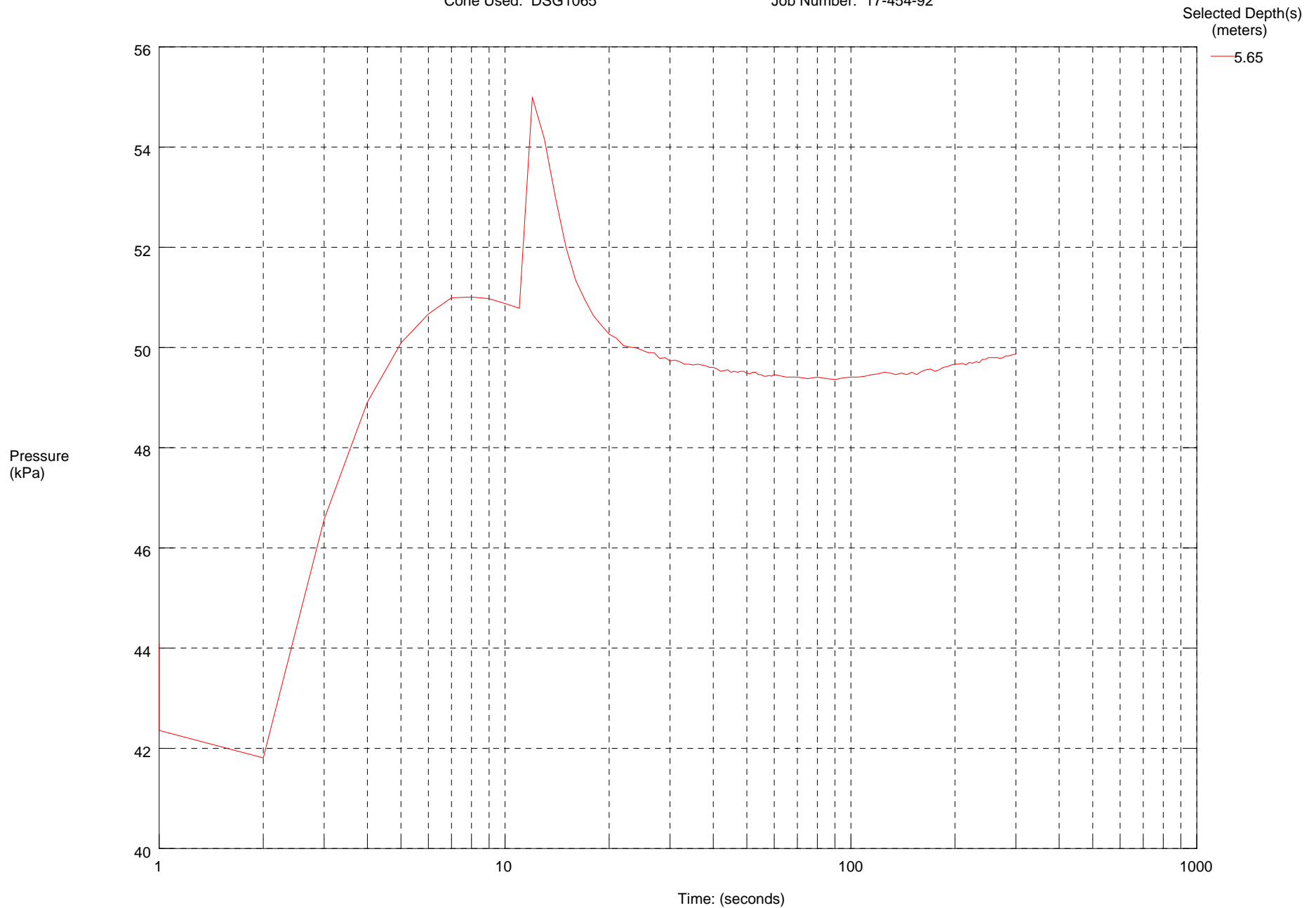


Maximum Pressure = 58.623 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-1S
Cone Used: DSG1065

CPT Date/Time: 4/16/2009 8:03:30 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

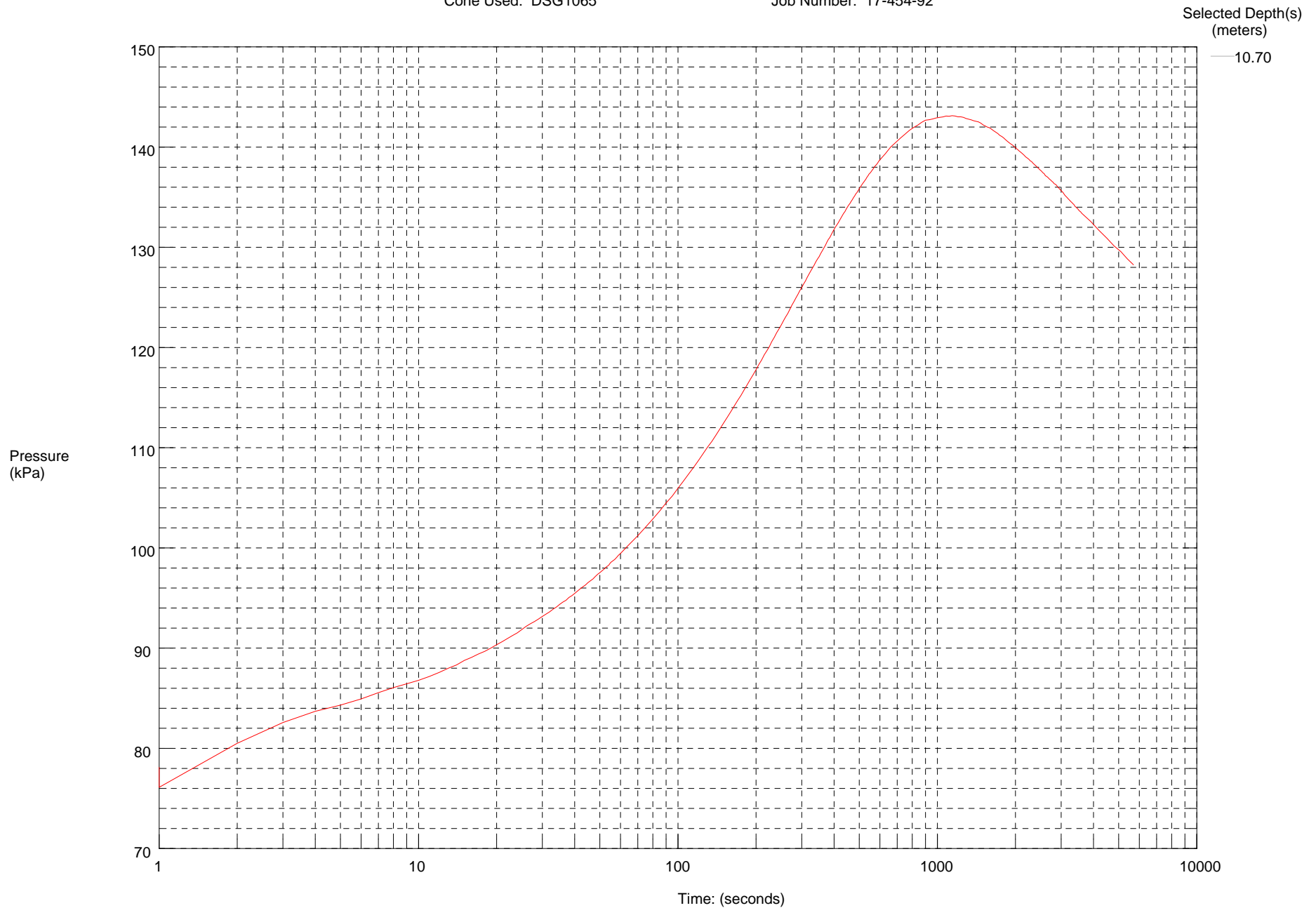


Maximum Pressure = 54.992 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-2S
Cone Used: DSG1065

CPT Date/Time: 4/18/2009 5:04:11 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

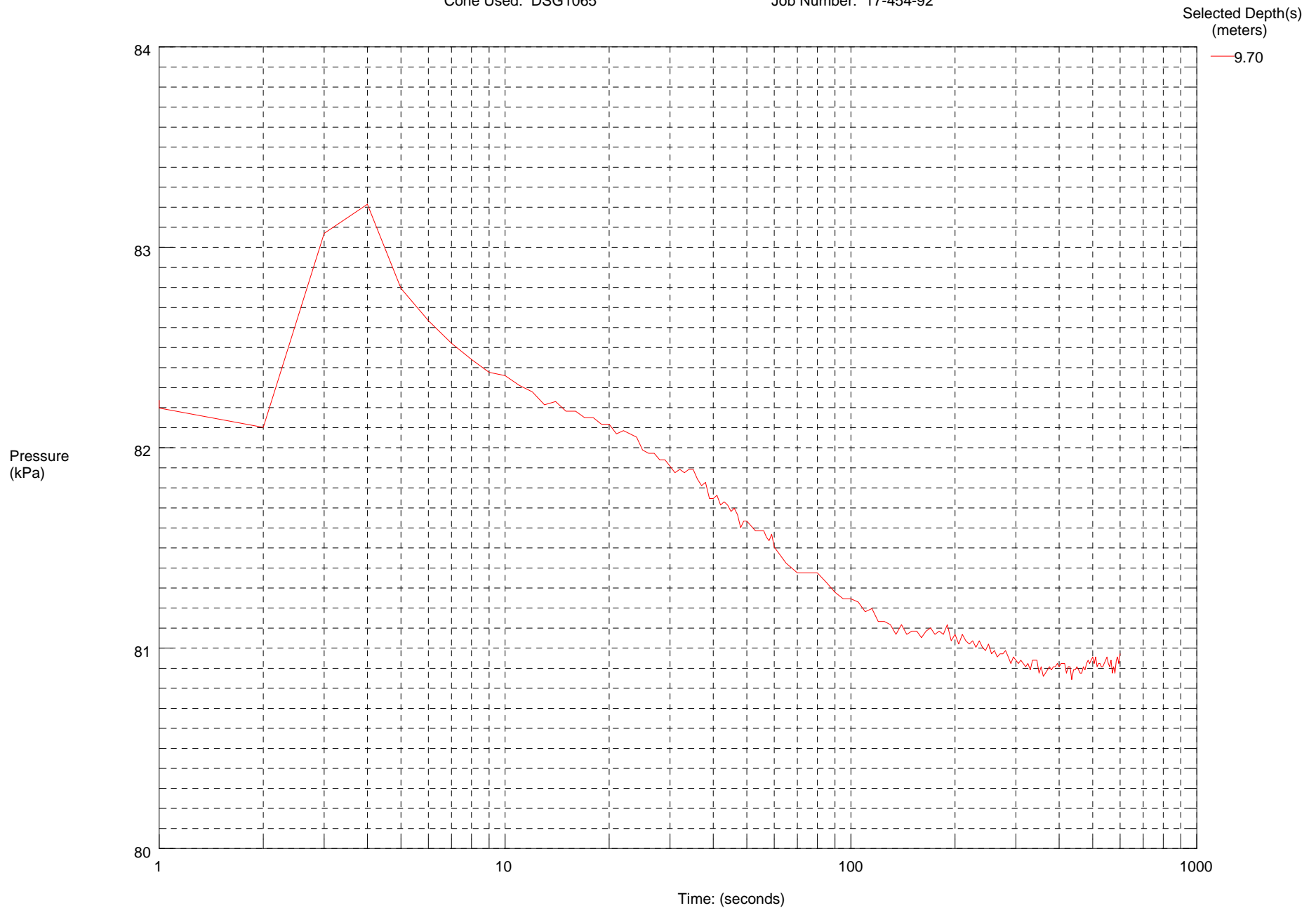


Maximum Pressure = 143.128 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-2S
Cone Used: DSG1065

CPT Date/Time: 4/18/2009 5:04:11 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

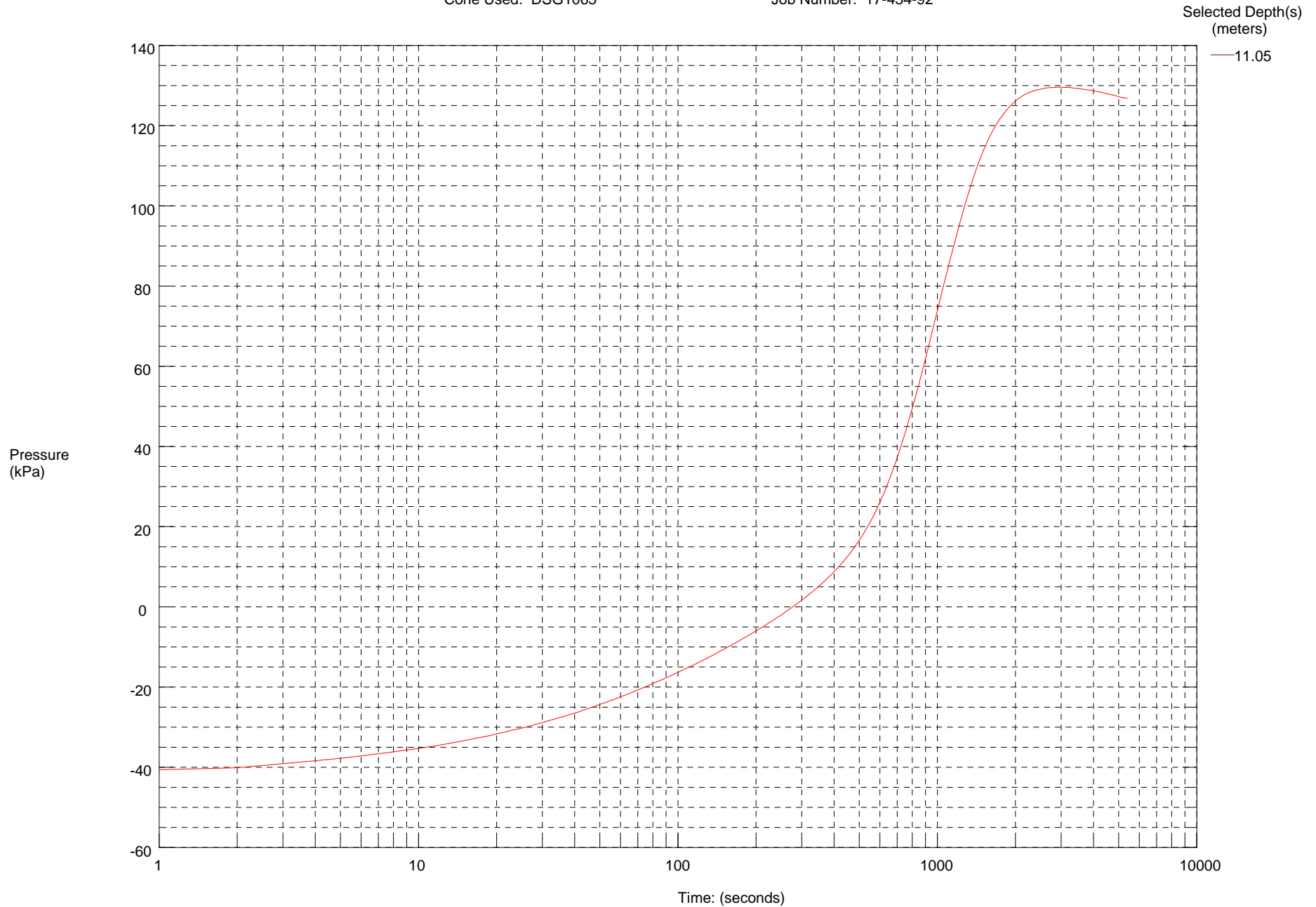


Maximum Pressure = 83.214 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-3S
Cone Used: DSG1065

CPT Date/Time: 4/17/2009 6:59:39 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

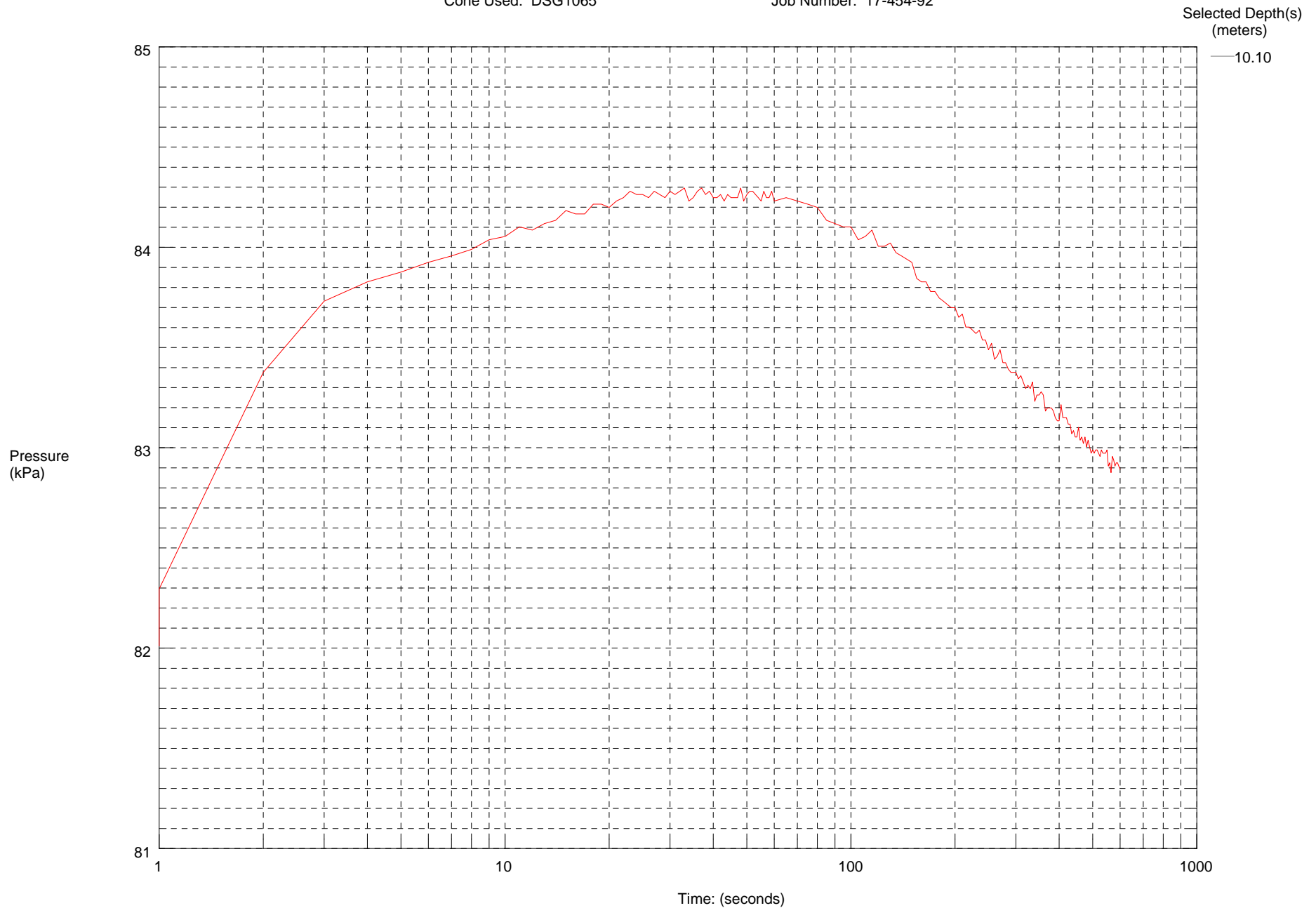


Maximum Pressure = 129.606 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-3S
Cone Used: DSG1065

CPT Date/Time: 4/17/2009 6:59:39 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

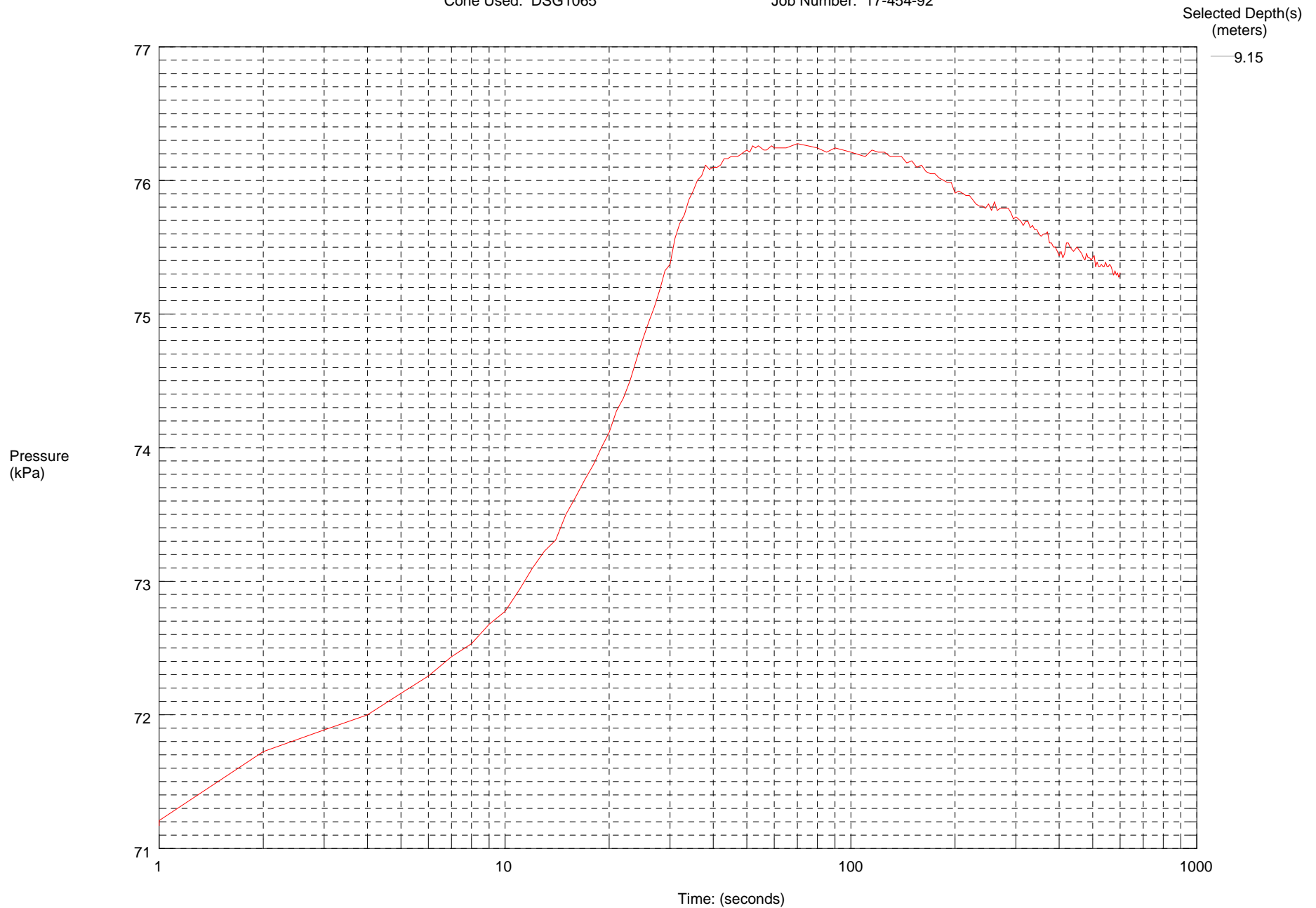


Maximum Pressure = 84.296 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-3S
Cone Used: DSG1065

CPT Date/Time: 4/17/2009 6:59:39 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92

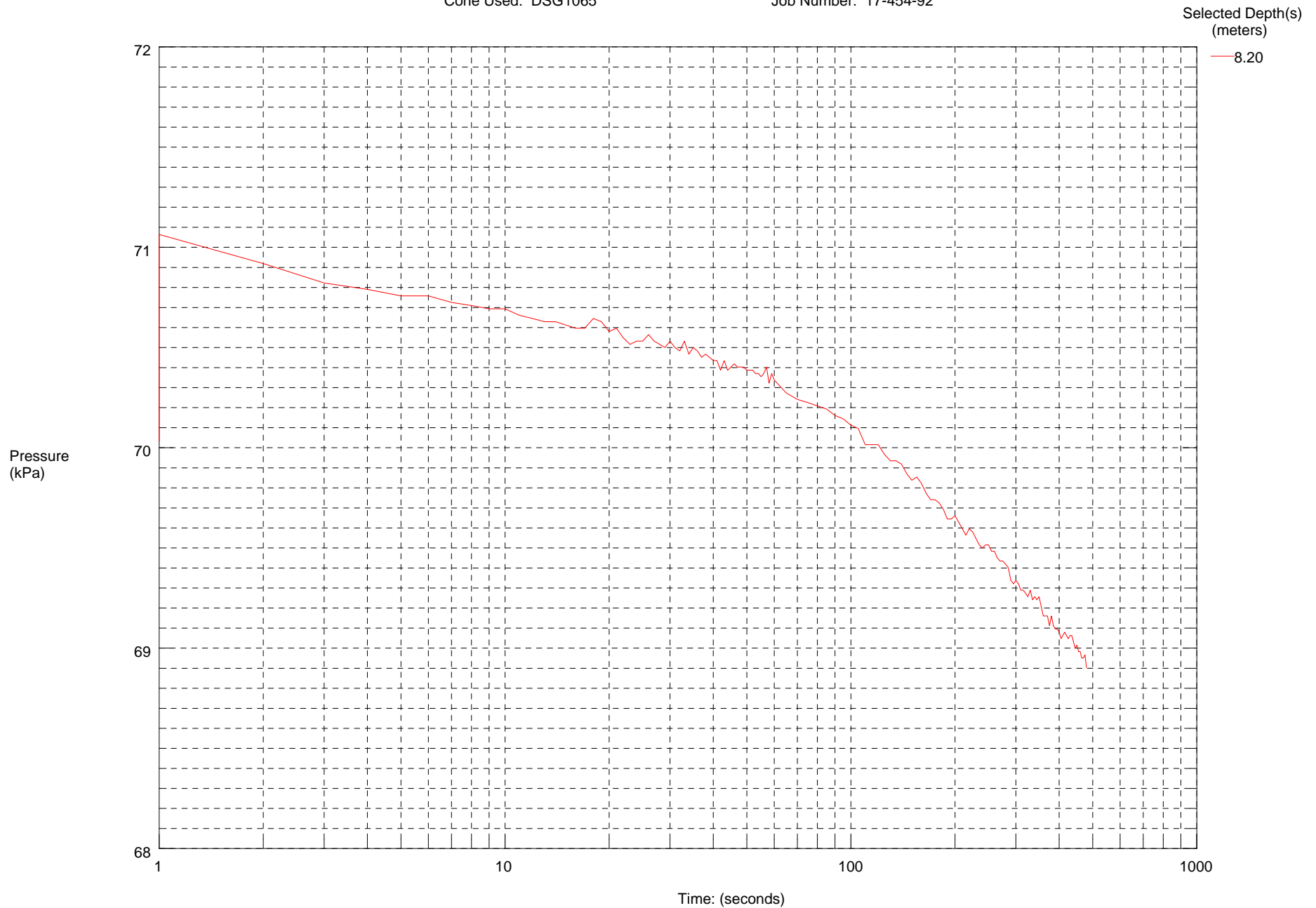


Maximum Pressure = 76.276 kPa

Thurber Engineering, Ltd

Operator Dafni
Sounding: CPT09-3S
Cone Used: DSG1065

CPT Date/Time: 4/17/2009 6:59:39 AM
Location: Leamington Dock Temporary Works
Job Number: 17-454-92



Maximum Pressure = 71.064 kPa

Project:	Leamington Dock, Ontario, Canada				
Date:	4/24/2009		Engineer:	K. Brown	
Purpose:	Estimate Friction Angle from CPT Data				
Reference:	Cone Penetration Testing, Lunne, et. al. 1997				
Stratigraphy:	From CPT log (In Situ Engineering) and boring log (Thurber Engineering)				
Unit weights:	From Peto MacCallum data provided by Thurber Engineering				
Analysis:	friction angle at		5.5 m		
Data:	water table =	1.75		Stratigraphy (m)	density
Test	CPT-01		0	5 sand and gravel fill	22.9
Depth	5.5 m		5	9 silty sand	21
Material	silty sand		9	11.5 silty clay	21
Qt =	3000 kPa				
$\sigma' = \sigma - u$	5.0	22.9	114.5	total stress of fill	
	0.5	21.0	10.5	total stress of sand	
	3.8	9.8	36.8	water pressure	
			88.2	effective stress kPa	
Nq = Qt/ σ'	34.0				

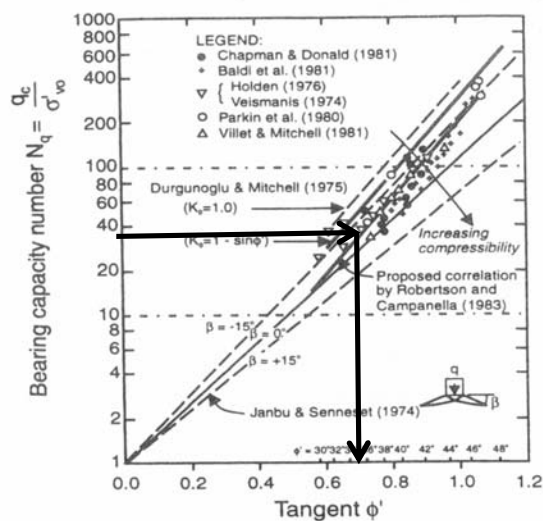


Figure 5.55 Relationship between bearing capacity number and friction angle from large calibration chamber tests (after Robertson and Campanella, 1983b).

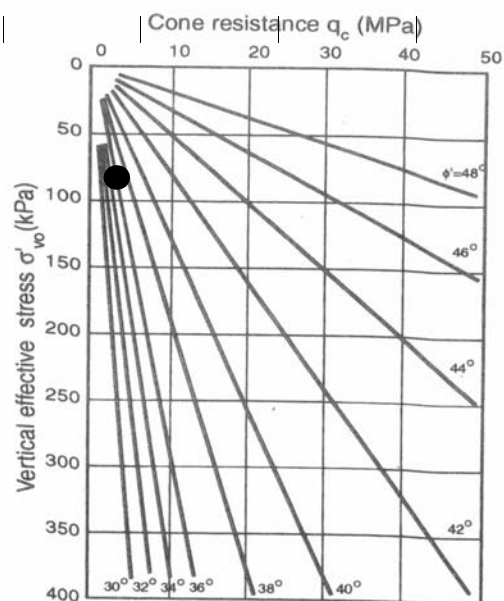


Figure 5.56 σ'_{v0} , q_c , ϕ' relationships (after Robertson and Campanella, 1983b).

Bearing Capacity analysis:		35 degrees			
Qc/ σ' analysis:		37 degrees			

Project:	Leamington Dock, Ontario, Canada							
Date:	4/24/2009		Engineer:	K. Brown				
Purpose:	Estimate Friction Angle from CPT Data							
Reference:	Cone Penetration Testing, Lunne, et. al. 1997							
Stratigraphy:	From CPT log (In Situ Engineering) and boring log (Thurber Engineering)							
Unit weights:	From Peto MacCallum data provided by Thurber Engineering							
Analysis:	friction angle at		7 m					
Data:	water table =	1.75			Stratigraphy (m)		density	
Test	CPT-01		0	5	sand and gravel fill		22.9	
Depth	7 m		5	9	silty sand		21	
Material	silty sand		9	11.5	silty clay		21	
Qt =	1200 kPa							
$\sigma' = \sigma - u$	5.0	22.9	114.5	total stress of fill				
	2.0	21.0	42.0	total stress of sand				
	5.3	9.8	51.5	water pressure				
			105.0	effective stress		kPa		
Nq = Qt/ σ'	11.4							

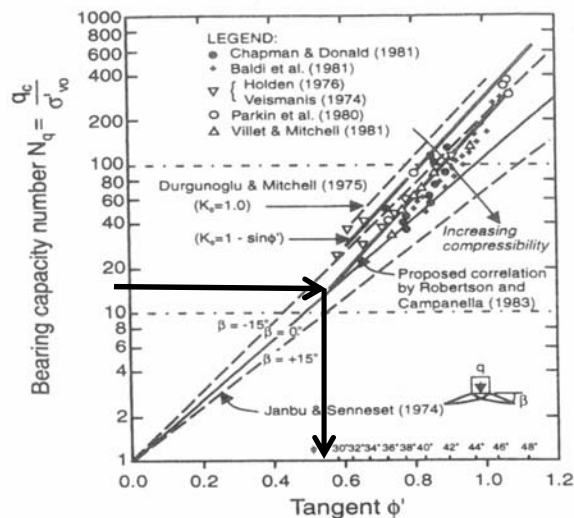


Figure 5.55 Relationship between bearing capacity number and friction angle from large calibration chamber tests (after Robertson and Campanella, 1983b).

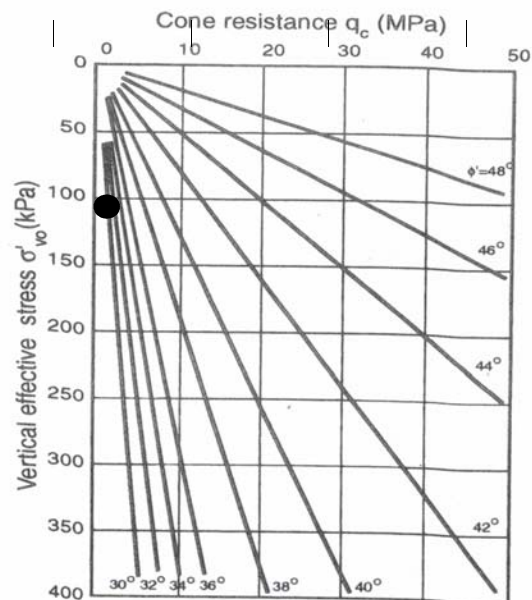


Figure 5.56 σ'_{v0} , q_c , ϕ' relationships (after Robertson and Campanella, 1983b).

Bearing Capacity analysis:	28 degrees				
Qc/ σ' analysis:	30 degrees				

Project:	Leamington Dock, Ontario, Canada					
Date:	4/24/2009		Engineer:	K. Brown		
Purpose:	Estimate Friction Angle from CPT Data					
Reference:	Cone Penetration Testing, Lunne, et. al. 1997					
Stratigraphy:	From CPT log (In Situ Engineering) and boring log (Thurber Engineering)					
Unit weights:	From Peto MacCallum data provided by Thurber Engineering					
Analysis:	friction angle at		9 m			
Data:	water table =	1.75			Stratigraphy (m)	density
Test	CPT-03S		0	7.5	water	9.81
Depth	9 m		7.5	10.75	silty sand	21
Material	silty sand		10.75	13	silty clay	21
Qt =	1700 kPa					
$\sigma' = \sigma - u$	7.5	9.8	73.6	total stress of water		
	1.0	21.0	21.0	total stress of sand		
	9.0	9.8	88.3	water pressure		
			6.3	effective stress		kPa
$Nq = Qt/\sigma'$	270.5					

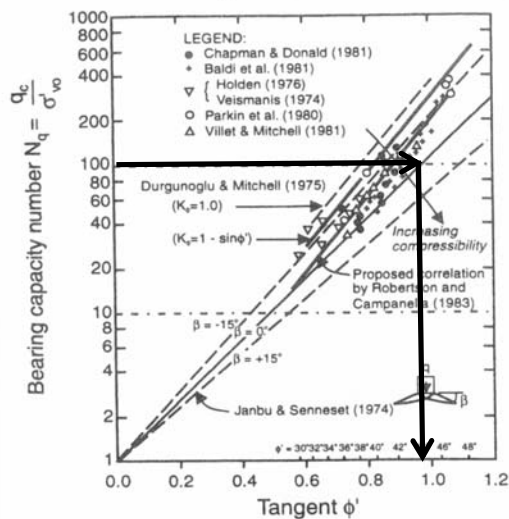


Figure 5.55 Relationship between bearing capacity number and friction angle from large calibration chamber tests (after Robertson and Campanella, 1983b).

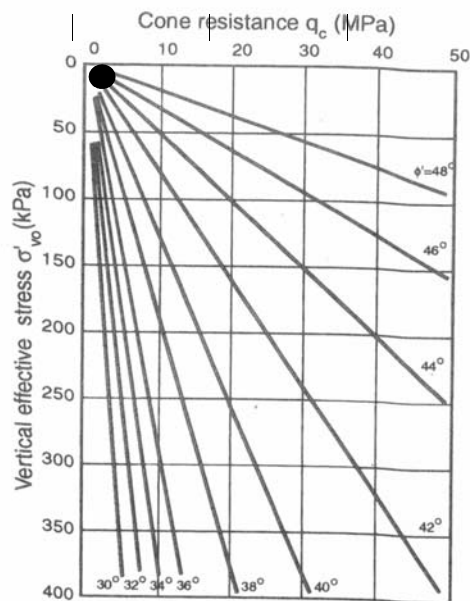


Figure 5.56 σ'_c, q_c, ϕ' relationships (after Robertson and Campanella, 1983b).

Bearing Capacity analysis:		44	degrees			
Qc/ σ' analysis:		44?	degrees			

Project:	Leamington Dock, Ontario, Canada						
Date:	4/24/2009		Engineer:	K. Brown			
Purpose:	Estimate Friction Angle from CPT Data						
Reference:	Cone Penetration Testing, Lunne, et. al. 1997						
Stratigraphy:	From CPT log (In Situ Engineering) and boring log (Thurber Engineering)						
Unit weights:	From Peto MacCallum data provided by Thurber Engineering						
Analysis:	friction angle at		9.75 m				
Data:	water table :	1.75			Stratigraphy (m)		density
Test	CPT-03S		0	7.5	water		9.81
Depth	9.75 m		7.5	10.75	silty sand		21
Material	silty sand		10.75	13	silty clay		21
Qt =	5000 kPa						
$\sigma' = \sigma - u$	7.5	9.8	73.6	total stress of water			
	2.3	21.0	47.3	total stress of sand			
	9.8	9.8	95.6	water pressure			
			25.2	effective stress		kPa	
$N_q = Q_t/\sigma'$	198.6						

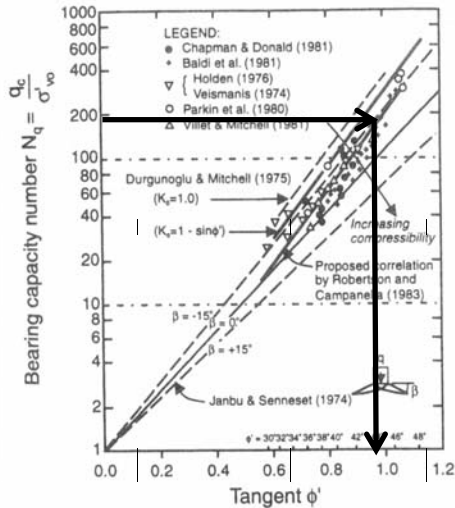


Figure 5.55 Relationship between bearing capacity number and friction angle from large calibration chamber tests (after Robertson and Campanella, 1983b).

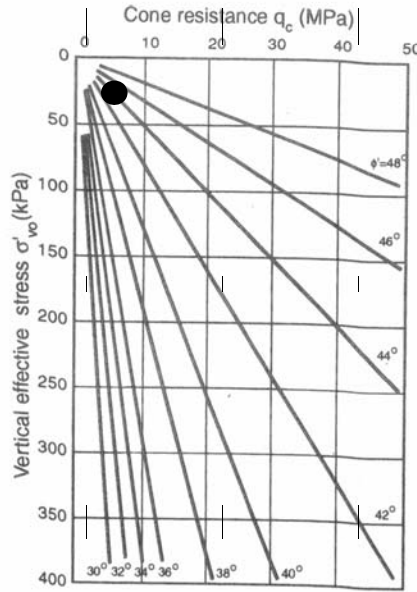


Figure 5.56 σ'_{vo} , q_c , ϕ' relationships (after Robertson and Campanella, 1983b).

Bearing Capacity analysis:		44	degrees		
Qc/ σ' analysis:		44	degrees		

Purpose:	Estimate Friction Angle from CPT Data				
Reference:	Cone Penetration Testing, Lunne, et. al. 1997				
Stratigraph	From CPT log (In Situ Engineering) and boring log (Thurber Engineering)				
Unit weigh	From Peto MacCallum data provided by Thurber Engineering				
Analysis:	friction angle at 10.25 m				
Data:	water table	1.75		Stratigraphy (m)	density
Test	CPT-03S		0	7.5 water	9.81
Depth	10.25 m		7.5	10.75 silty sand	21
Material	silty sand		10.75	13 silty clay	21
Qt =	10000	kPa			
$\sigma' = \sigma - u$	7.5	9.8	73.6	total stress of water	
	2.8	21.0	57.8	total stress of sand	
	10.3	9.8	100.6	water pressure	
			30.8	effective stress	
				kPa	
Nq = Qt/ σ'	325.0				

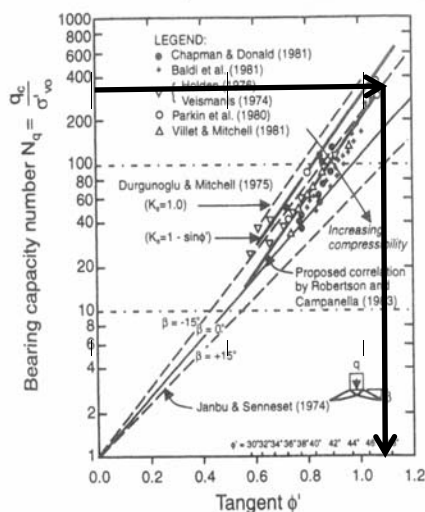


Figure 5.55 Relationship between bearing capacity number and friction angle from large calibration chamber tests (after Robertson and Campanella, 1983b).

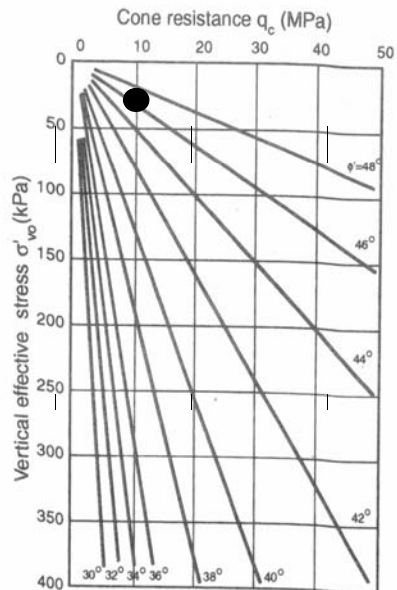


Figure 5.56 σ'_{v0} , q_c , ϕ' relationships (after Robertson and Campanella, 1983b).

Bearing Capacity analysis:	46 degrees
Qc/ σ' analysis:	46 degrees

Summary of Analysis					
		friction angle			
Test	Depth	method 1	method 2		
	(m)	(degrees)	(degrees)		
CPT09-1S	5.5	35	37		
CPT09-1S	7	28	30		
CPT09-2N	11				
CPT09-3N	9	44	44		
CPT09-3N	9.75	44	44		
CPT09-3N	10.25	46	46		
CPT09-3N	12				
Method 1 is bearing capacity correlation					
Method 2 is Q_c vs effective stress correlation					
Friction angle analysis was not done on CPT-02 : judged to be too inconsistent in gradation and may contain gravels.					
Note: low confining stresses make the friction angle interpretations suspect.					

Shear wave analysis summary.										
Test	offset	top	Arrival	bottom	Arrival	Velocity	Material	Density	Modulus	Notes
	(m)	(m)	(ms)	(m)	(ms)	(m/s)		kN/m3	kPa	
CPT09-1S	0.73	5.65	52.07	9.15	72.69	169	Sand	21.5	62500	
CPT09-1S	0.73	9.65	59.8	12.05	78.32	129	Silty Clay	20.4	34763	
CPT09-2N	0.5	0.7	6.99	4.1	32.46	128	Silty Clay	20.4	34280	
CPT09-3N	0.5	0.5	2.15	2.35	10.39	206	Sand	21.5	92792	
CPT09-3N	0.5	3.35	19.06	5.45	27.65	243	Silty Clay	20.4	122606	
	Notes:		9m subtracted from depth on CPT-09-2N to correct for depth to mudline.							
			7.7m subtracted from depth on CPT-09-3N to correct for depth to mudline.							