FOUNDATION INVESTIGATION AND DESIGN REPORT
ADVANCE FOUNDATION INVESTIGATION
SHEWFELT BRIDGE REPLACEMENT
GOULAIS BAY ROAD, 3 KM WEST OF HIGHWAY 17
DISTRICT OF ALGOMA, ONTARIO
G.W.P. 5290-04-00
SITE 38S-031
GEOCRES NO. 41K-79

Prepared For:

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1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by Lea Consulting Limited (LEA) to carry out a foundation investigation at the site of the proposed replacement of the Shewfelt Bridge located over the Goulais River on Goulais Bay Road between Highway 552 and Pine Shores Road, in the Township of Fenwick, approximately 3 km West of Highway 17. The site falls within the District of Algoma and has MTO Site Number 38S-031.

The existing Shewfelt Bridge is a four-span bridge with a length of 84.7 m, which contains a two-span single lane Bailey bridge (63.1 m) with a timber deck, and two steel girder end spans with a concrete deck, each 10.8 m in length. It is understood that the performance of the existing bridge is affected by the problems of bridge foundation settlements, slope stability, active erosion and riverbank slumping (upstream of the existing bridge).

In response to the RFP for this project (MTO GWP 5290-04-00 dated January 2005), S&P has proposed to carry out a foundation investigation by drilling and sampling a total of 17 boreholes to depths of about 6 m to 30 m (7 boreholes for proposed bridge abutments, piers and approaches, and 10 boreholes for the proposed embankments).

Subsequently, S&P was requested to put down three boreholes (two boreholes for the proposed Shewfelt Bridge and one borehole for slope stability analysis of riverbank) for an Advance Foundation Investigation prior to finalizing the bridge alignment. The purposes of this Advance Foundation Investigation were to provide preliminary assessment of the required pile depth and embankment slope stability to aid in deciding the final bridge alignment by drilling two boreholes (BH1 and BH2), one on each side of the river, and to provide slope assessment of the riverbank near the location of BH3. The borehole locations are shown in Drawing No.1.

The findings of the Advance Foundation Investigation for the proposed Shewfelt Bridge are presented in this report.

2. SITE DESCRIPTION AND GEOLOGY

The Goulais River is located in a deep and wide valley (the Goulais River Valley) north of Sault Ste. Marie. In the general vicinity of the project site, the area is referred to as the Goulais River Beach Ridges, which is described as ancient beach ridges of an alluvial plain. The river meanders on its way toward Lake Superior and numerous oxbow lagoons are evident.

The Goulais River has steep banks, with bank failures having occurred at many river areas. It is evident that the Goulais River is continuing to undercut the banks at turns in the river, resulting in slope failures and re-alignment of the river channel. It is noted that a section of the existing Goulais Bay Road located near the west bank of the river (i.e. near BH3) is at close proximity to a bend in the present river channel and is exposed to the risk of bank failures.

Based on available information, the Goulais River Valley was probably cut by a major pre-glacial river. At the time of the retreat of the last glaciations, a river flowed in the Goulais Valley carrying glacial materials into Glacial Lake Algonquin, resulting in deep glacial deposits. Prior to this, it appears that deep clays were deposited and followed by sands and silts deposited by the river itself.

3. INVESTIGATION PROCEDURES

The fieldwork for the proposed Shewfelt Bridge was performed during the period of February 28, 2006 through March 12, 2006. As agreed with MTO, the fieldwork consisted of drilling and sampling three boreholes (Boreholes 1, 2 and 3), as well as performing field vane tests and Dynamic Cone Penetration Tests (DCPT). The plan location of the boreholes is shown in Drawing No. 1. The following table summarizes the borehole locations and drilling depths.

Table 1 Summary of Borehole Locations and Drilling Depths

Borehole No.	Location	Drilling Depth below Existing Ground Surface (m)	Dynamic Cone Penetration Tests
<u>For Proposed Shewfelt Bridge</u>			
BH1	East side of Goulais River	48.3	• 19.8 m to 24.3 m • 25.9 m to 30.5 m • 48.5 m to 54.9 m
BH2	West side of Goulais River	41.6	• 40.2 m to 41.6 m
<u>For Slope Stability Analysis</u>			
BH3	West side of Goulais River and East side of Goulais Bay Road	31.0	---

A specialist drilling contractor (LANDCORE of Chelmsford, Ontario) carried out the drilling, testing and sampling work under the direction and supervision of a Geotechnical Engineer from S&P. The boreholes were advanced using a track-mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. Drilling started using hollow-stem augers. In Boreholes 1 and 2 wash-boring method (with NW casing) was used to advance the boreholes. Drilling mud was utilized to counter-balance the hydrostatic uplift due to water table. The soil sampler and rods were withdrawn slowly to reduce suction below the groundwater table.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

In cohesive (clayey) deposits, where the consistency of the soil permitted, relatively undisturbed samples (TW) were taken with 50 mm or 70 mm diameter thin-walled (Shelby) tubes which were pushed into the bottom of the borehole by the application of static weight using hydraulic pressure. The undrained shear strength of the soil was also measured in-situ by Field Vane tests. Where consistency permitted, MTO Field Vane was used to conduct the tests but when the soil became stiffer this was changed to small Field Vane.

Plots of undrained shear strength as measured by Field Vane tests are presented in Figures C1 to C3 of Appendix C.

In Dynamic Cone Penetration Test (DCPT), a 51 mm diameter, 60 deg. apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples can be obtained by the DCPT method and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which in many cases affect the SPT values, especially when fine-grained granular soils or cobbles/boulders are encountered.

As summarized in Table 1, Boreholes 1 and 2 were extended to depths of 48.3 m and 41.6 m, respectively, and Borehole 3 was extended to a depth of 31.0 m below ground surface. In addition, Dynamic Cone Penetration tests were performed in Boreholes 1 and 2, extending to depths as shown in Table 1.

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. A piezometer was installed in Borehole 1 and two piezometers were installed in Borehole 3, upon its completion, to enable groundwater level monitoring in the boreholes over a prolonged period of time. Upon their completion, Borehole 2 was grouted using a cement/bentonite mixture as per MTO procedures. Boreholes 1 and 3 will be decommissioned during the detailed investigation phase, in order to obtain more piezometer data.

The borehole locations had been established in the field by surveyors (retained by LEA) prior to our field crew's arrival at the site. Based on the provided bench mark information, the Geodetic ground surface elevations at the borehole locations were surveyed by our fieldwork supervisor.

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content and unit weight determinations, grain size analyses and Atterberg Limits tests, was performed on selected representative samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4. SUBSURFACE CONDITIONS

The sub-surface conditions were explored at three (3) boreholes (see Table 1 in Section 3 above) during the current investigation (Advance Foundation Investigation). The plan locations of the boreholes are shown on Drawing No. 1. Details of sub-surface conditions encountered at each borehole location for the current investigation, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are enclosed in Appendix B.

In general, the sub-surface stratigraphy comprises surficial topsoil and/or fill materials overlying very loose to loose cohesionless sand, sandy silt and silty sand to sandy silt deposits, which are in turn underlain by firm to stiff clayey silt, and followed by a thick deposit of soft to very stiff silty clay. In Boreholes 1 and 2, the silty clay deposit is further underlain by a clayey silt deposit, followed by gravels and cobbles at the bottom of the boreholes. Bedrock was not encountered in the boreholes, within the depths drilled.

The various strata encountered in the boreholes and their geotechnical properties are briefly described in the following subsections of this report. Please note that the following summary is to assist the designers of the project with a preliminary understanding of the anticipated soil conditions across the site. It should also be noted that the soil and groundwater conditions may vary in between and beyond borehole locations.

4.1 TOPSOIL

Topsoil was encountered in Boreholes 1 and 2 with thickness of about 250 mm. It should be noted that the thickness of topsoil may vary in between and beyond the borehole locations.

4.2 FILL

Borehole 3 was drilled at about 1 m east from the east edge of the paved road surface of Goulais Bay Road and encountered 3.1 m thick fill materials. The fill consisted of non-cohesive silty fine sand to sandy silt with gravel, organics and rootlets and the recorded N-values ranged from 6 to 10 blows/0.3 m, indicating loose relative density. The measured moisture contents varied from 9 to 49%.

4.3 SURFICIAL SILTY FINE SAND

Below the topsoil, Boreholes 1 and 2 contacted a surficial silty fine sand material with trace topsoil and rootlets. This non-cohesive layer was found to extend to 0.7 m (Elevation 192.7 m) in Borehole 1 and 0.8 m (Elevation 186.3 m) in Borehole 2.

Moisture contents of samples recovered from this layer ranged from 20% to 47%.

4.4 SAND, SANDY SILT AND SILTY SAND TO SANDY SILT DEPOSITS

Below the topsoil and surficial silty fine sand (at Boreholes 1 and 2) or fill materials (at Borehole 3), sand, sandy silt and silty sand to sandy silt deposits (i.e. non-cohesive) with occasional gravel, rootlets and organics were encountered.

In the boreholes drilled near the proposed Shewfelt Bridge alignment (Boreholes 1 and 2 located on the east and west side of Goulais River, respectively), these deposits were encountered at depths of about 0.7 m to 0.8 m. (Elevations 192.7 m for Borehole 1 to 186.3 m for Borehole 2) and extended to depths of about 9.0 m to 7.6 m (El. 184.3 m to El. 179.5 m), with a thickness ranging from 8.3 m to 6.8 m.

In Borehole 3, which was drilled for the purpose of slope stability analysis, sandy silt with some organics and trace rootlets was encountered at 3.1 m (El. 185.1 m) and extended to 4.4 m (El. 183.7 m) below existing ground surface, with thickness of about 1.3 m.

These non-cohesive deposits consisted of sandy silt, silty sand or sand, or the combination of two or more of these soil types as shown in the Record of Borehole Sheets.

In Boreholes 1 and 2, measured SPT N-values ranged from 2 to 10 blows per 0.3 m penetration, indicating very loose to loose relative density. Measured moisture contents varied from 4 to 26 %.

In Borehole 3, measured SPT N-values were 1 and 2 blows per 0.3 m penetration, indicating very loose relative density. Measured moisture contents of this deposit varied from 21 to 35 %.

4.5 CLAYEY SILT

Underlying the non-cohesive deposits described in the previous sections, all boreholes encountered a layer of grey to reddish grey clayey silt at depths of about 4.4 m to 9.0 m (El. 184.3 m to El. 179.5 m), and extended to 9.2 m to 13.7 m (El. 179.6 m to El. 178.0 m) below ground surface. The thickness of this cohesive deposit varied from 1.6 m to 4.8 m. A Shelby tube sample (50 mm diameter) was taken at about Elevation 181 m in Borehole 1.

Standard Penetration Tests conducted in this clayey silt deposit gave low N-values of zero to 4 blows/0.3 m. Undrained shear strength as measured by Field Vane tests varied from 28 kPa to 64 kPa, indicating a firm to stiff consistency. Measured moisture contents of the samples tested varied from 25 to 41 %.

Unit weight determination, grain size analysis and Atterberg Limits test were performed on one soil sample (Shelby tube sample) in the laboratory and the test results are summarized as follows:

- Natural Moisture Content: 39%
- Unit Weight: 17.6 kN/m^3
- Grain Size:

Silt:	79%
Clay:	21%

The grain size curve for this material is provided on Figure B1.

- Atterberg Limits:

Liquid Limit:	25%
Plastic Limit:	16%
Plasticity Index:	9

These values are generally typical of clayey deposit of low plasticity (i.e. CL). The test results are further summarized in Figure B3.

4.6 SILTY CLAY

At depths ranging from 9.2 m (Boreholes 2 and 3) to 13.7 m (Borehole 1) or below Elevations 179.6 m to 178.0 m, all three boreholes contacted a reddish grey silty clay deposit. In Boreholes 1 and 2, this cohesive deposit was found to extend to depths of about 45.7 m and 39.6 m (El. 147.6 m and 147.5 m) below ground surface, with thickness of about 32.0 m and 30.4 m, respectively. Borehole 3 was terminated in this cohesive deposit at 31.0 m depth (El. 157.2 m). Five Shelby tube samples (50 mm to 70 mm diameter) were recovered at various depths in Boreholes 1 and 2 (refer to Record of Borehole Sheets for details).

Standard Penetration Tests conducted in this silty clay deposit gave low N-values of zero to 4 blows/0.3 m. Undrained shear strength as measured by Field Vane tests varied from 12 kPa to 135 kPa, indicating a soft to very stiff consistency, but in general firm to stiff. The consistency of this cohesive deposit became very stiff below the depths of about 37.2 m (El.156.1 m), 31.6 m (El.155.5 m) and 28.5 m (El.159.6) in Boreholes 1, 2 and 3, respectively. The soft silty clay deposit was found in Borehole 1 at about 31 to 32 m (El.162.3 m to El.161.3 m) below ground surface. Measured moisture contents of the samples tested varied from 34 to 77 %.

Dynamic Cone Penetration Tests (DCPT) were carried out in this deposit in Borehole 1 from 19.8 m to 24.3 m (El. 173.5 m to El. 169.0 m) and from 25.9 m to 30.5 m (El. 167.4 m to El. 162.8 m), with measured blow counts varied from 3 to 16 blows per 0.3 m penetration.

Unit weight determinations, grain size analyses and Atterberg Limits tests were performed on five soil samples (Shelby tube samples) in the laboratory and the test results are summarized as follows:

- Natural Moisture Content: 41 – 66%
- Unit Weight: 15.3 – 18.0 kN/m^3
- Grain Size:

Gravel:	0 – 2%
Sand:	0 – 2%
Silt:	22 – 64%
Clay:	32 – 78%

The grain size curves for this material are presented in an envelope form provided on Figure B2.

- Atterberg Limits:

Liquid Limit:	38 – 79%
Plastic Limit:	18 – 24%
Plasticity Index:	20 – 56

The above test results indicated that the silty clay deposit is of medium to high plasticity (i.e. CI to CH). The test results are further summarized in Figure B3.

Figures C1 to C3 of Appendix C present various types of plots for the variation of measured undrained shear strengths (as measured by Field Vane tests in the silty clay and clayey silt deposits) with elevation in Boreholes 1, 2 and 3. In Figure C4, the effective overburden stresses (P'_o) at Boreholes 1, 2 and 3 locations are superimposed with the plots of undrained shear strength variation of the corresponding boreholes. For normally-consolidated clays in northern Ontario, our experience shows that the undrained shear strengths can be represented by a factor of 0.23 (i.e. $C_u \cong 0.23\ P'_o$), which is also shown in Figure C4. As the measured shear strengths are slightly in excess of 0.23 P_o, the deposits is likely to be slightly over-consolidated, possibly due to ‘aging’ effects. The higher shear strength measurements may also be due to the effects of the inclusion of some granular materials (e.g. sand and/or gravel) in the silty clay deposit.

4.7 LOWER CLAYEY SILT

In Boreholes 1 and 2, the silty clay deposit is underlain by a lower clayey silt deposit at 45.7 m and 39.6 m (El.147.6 m and El. 147.5 m) below ground surface, respectively. This cohesive deposit extended to 47.2 m and 40.5 m (El. 146.1 m and El. 146.6 m), with thickness of about 1.5 m and 0.9 m.

Standard Penetration test performed on this grey cohesive deposit yielded an N-value of 6 blows/0.3 m. Undrained shear strength as measured by Field Vane tests in Borehole 1 varied from 115 kPa to over 200 kPa, while in Borehole 2, the vane could not be advanced in this deposit. These results indicated that this cohesive deposit has very stiff to hard consistency. Measured natural moisture contents varied from 23 to 28%.

4.8 GRAVEL AND COBBLES

Underlying the lower clayey silt deposit, Boreholes 1 and 2 encountered a deposit consisting of gravel and cobbles materials at 47.2 m and 40.5 m (El. 146.1 m and El. 146.6 m) below ground surface, respectively. These two boreholes terminated in this non-cohesive deposit at 48.3 m and 41.6 m (El. 145.0 m and El. 145.5 m).

Measured SPT N-values in this deposit ranged from 44 blows per 0.3 m (Borehole 1) to 50 blows per 0.08 m penetration (Borehole 2), indicating dense to very dense relative density. Measured moisture content in one soil sample was 11%.

Dynamic Cone Penetration Tests (DCPT) were carried out at the bottom of Boreholes 1 and 2. In Borehole 1, DCPT were carried out from 48.5 m to 54.9 m (El. 144.8 m to El. 138.4 m) below ground surface. As can be seen in the DCPT plots in the Record of Borehole Sheets, the DCPT blow counts had high variation with depth (a “zigzag” curve pattern) with test results varied from 19 to over 200 blows per 0.3 m penetration, and this may be due to the presence of cobbles and/or boulders which obstructed the penetration of the cone/rod assembly. This may result in the bending of the rods and the non-vertical penetration of the cone/rod assembly. The DCPT encountered refusal at 54.9 m (El. 138.4 m) below ground surface. From the results, the relative density of the soil below El. 145.0 m can be surmized to be compact to very dense.

In Borehole 2, Dynamic Cone Penetration Tests were carried out from 40.2 m to 41.6 m (El. 146.9 m to El. 145.5 m) and encountered refusal at 41.6 m (El. 145.5 m) below ground surface.

4.9 GROUNDWATER CONDITIONS

Groundwater conditions were observed in the open boreholes during the drilling and upon completion of each borehole. However, because wash-boring method was used in Boreholes 1 and 2, water level conditions observed during drilling in these two boreholes may not be useful.

In Borehole 1, a piezometer was installed at the bottom of the borehole at about 47.8 m depth or El. 145.5 m in the gravel and cobbles deposit. Two piezometers were installed in Borehole 3 at about 7.6 m depth (El. 180.5 m, upper piezometer) in the sandy silt – clayey silt deposits, and at the bottom of the borehole at 30.5 m (El. 157.6 m, lower piezometer) in the silty clay deposit. Groundwater level measurements in the installed piezometers during the time of investigation are summarized in the following table (Table 2).

Table 2 Summary of Groundwater Level Measurements

Date	**Groundwater Level Measurement in Piezometers**			**Remarks**
	Borehole 1	**Borehole 3 Upper Piezometer**	**Borehole 3 Lower Piezometer**	
March 7, 2006	---	2.8 m (El. 185.3 m)	4.5 m (El. 183.6 m)	---
March 8, 2006	---	2.7 m (El. 185.4 m)	3.2 m (El. 184.9 m)	---
March 9, 2006	3.3 m (El. 190.0 m)	2.7 m (El. 185.4 m)	2.8 m (El. 185.3 m)	---
March 10, 2006	4.5 m (El. 188.8 m)	2.7 m (El. 185.4 m)	2.7 m (El. 185.4 m)	---
March 11, 2006	5.8 m (El. 187.5 m)	2.5 m (El. 185.6 m)	2.5 m (El. 185.6 m)	Snow melting noted during measurement.
March 12, 2006	5.6 m (El. 187.7 m)	2.5 m (El. 185.6 m)	2.3 m (El. 185.8 m)	Snow melting noted during measurement. Readings indicate an upward vertical gradient.

From the measured moisture contents of the soil samples, the groundwater level at the time of investigation can be considered to be at a depth of about 5 m (El. 188.3 m) in Borehole 1 and 2 m (El. 185.1m) in Borehole 2. Based on the preliminary contour plan provided to us, the elevation of water surface of the Goulais River is about El. 183.7 m.

The piezometers installed at lower elevations in Boreholes 1 and 3 also indicate an upward vertical hydraulic gradient.

It should be noted that the groundwater table would be subject to seasonal fluctuations and in response to major weather events, as well as the water level in the Goulais River. Seepage and/or perch water conditions could also exist in the top 4 m to 9 m of relatively pervious soil layers.

5. DISCUSSION AND PRELIMINARY RECOMMENDATIONS

The purposes of this Advance Foundation Investigation were to provide preliminary assessment of the required pile depth and approach embankment slope stability to aid in deciding the final bridge alignment by drilling two boreholes (BH1 and BH2), one on each side of the river. It is noted that a section of the existing Goulais Bay Road located near the west bank of the river (i.e. near BH3 and north of the existing Bailey bridge) is at close proximity to a bend in the present river channel and is exposed to the risk of bank failures. Slope stability assessment of this section of the riverbank will also be discussed.

It should be noted that, in this preliminary stage of foundation investigation and based on the results of investigation (drilling and sampling of Boreholes 1, 2 and 3) only preliminary recommendations are provided in the following sub-sections.

Other issues such as recommendations for detailed design of foundations, approach embankments and retaining structures (if any), estimation of embankment settlements and general comments on scour protection will be addressed in the detailed design stage and after the completion of detailed foundation investigation as initially proposed by S&P for this project.

5.1 FOUNDATIONS

The very loose to loose sand to sandy silts are considered unsuitable to support normal shallow spread footing foundations, including the use of spread footing on engineered fill. The relative density of these soils can be improved by surcharging and/or by means of in-situ densification but such operations are considered impractical immediately adjacent to some structures (e.g. residential houses) and the river. As well excessive long term settlements can be expected due to the consolidation of the underlying weak clay deposit. The bridge will, therefore, need to be supported on deep foundations.

The use of drilled and cast-in-place concrete (caisson) foundations to support the structure is considered impractical due to water bearing granular deposits and the lack of a well-defined bearing stratum to support the caissons within the clay. Auger press piles can be extended in cohesionless soils below the groundwater table, but these offer little resistance to lateral loads and will be uneconomical. They are, therefore, not recommended based on reliability and cost.

Expanded base (Franki-type) concrete piles and driven concrete piles are not considered to represent a practical, cost-effective and reliable solution.

The boreholes (BH1 and BH2) show that with the prevailing subsurface conditions the use of a low displacement pile, such as a steel H-pile with a heavy section (e.g. HP 310 x 110), would be better suited than other pile types (e.g. steel tube piles, steel H-piles with lighter sections, Franki Pile or precast concrete piles).

In this preliminary design stage of this project, only preliminary values of pile loading capacity and pile tip elevations for friction piles and end-bearing piles are provided for the purposes as discussed earlier. More borehole information and laboratory testing are required to determine the compressibility and shear strength characteristics of the silty clay deposit at different elevations, in order to provide a better estimation of bearing resistances and corresponding settlements of the steel H-pile with pile tip founded at different elevations.

As can be seen from Figure C4, the undrained shear strength of silty clay as measured by field vane tests generally varies from 40 kPa at El. 179 m to 65 kPa at El. 160 m. The undrained shear strength increased at a higher` rate from 65 kPa at El. 160 m to 110 kPa at about El. 152 m. Between El. 152 m and El.147.5 m, constant undrained shear strength of 110 kPa was assumed.

For friction pile design, the following table summarizes the approximate range of tip elevations that may be utilized for preliminary design purposes.

Support Location	Reference Borehole	Estimated Pile Tip Elevation	Estimated Approximate Pile Length Below Existing Ground Surface	Soil Deposit
East Abutment	BH1	155 to 150 m	38.3 to 43.4 m	Silty Clay
West Abutment	BH2	155 to 150 m	32.1 to 37.1 m	Silty Clay

The following axial resistances are estimated for HP 310 x 110 steel piles driven to tip elevations as shown in the above table.

Factored Axial Resistance at U.L.S. = 950 kN/pile
Axial Resistance at S.L.S. = 600 kN/pile

It should be noted that, as can be seen in the Record of Borehole Sheets for BH1 and Figures C3 and C4, a soft pocket with low undrained shear strength values (about 12± kPa) was detected at about El. 161 to 162 m in the silty clay deposit. Further investigation is therefore required to assess the lateral extent of this soft deposit as this may have adverse impact to the bearing resistance of the H-piles.

Another option is to drive the H-Pile to practical refusal within the overburden or bedrock at or below Elevation 138.4 m (pile length will be in the order of 50 to 60 m, or more). The following axial resistances are estimated for HP 310 x 110 steel H-piles driven to practical refusal in the overburden.

- Factored Axial Resistance at U.L.S.= 1500 kN/pile
- Axial Resistance at S.L.S. = 1000 kN/pile

These values can be increased if piles are driven to the surface of bedrock. However, we understand that on the west side where the grade is approximately 6 m lower than the east side, a grade raise of about 4.5 m will be required at the abutment locations. The stresses induced by the grade raise will cause settlements of the underlying soils and therefore downdrag effects on the piles will need to be considered if the piles are founded end bearing on bedrock or on boulders.

The following are our recommended pile resistances (HP310 x 110 steel piles) and tip elevations with the present information, driven into the gravel and cobbles stratum underlying the silty clay and the lower clayey silt strata:

- East Abutment (BH1): SLS = 800 kN/pile, ULS = 1200 kN/pile
- West Abutment (BH2): SLS = 700 kN/pile, ULS = 1050 kN/pile

Pile tip depth (below existing grade)/elevation:

- East Abutment (BH1): 48 – 53 m (El.145 – 140 m)
- West Abutment (BH2): 42 – 47 m (El.145 – 140 m)

Pile load test may be necessary to verify the proposed resistances.

As can be seen from the resistance values a modest allowance for downdrag was made at the west abutment locations in case the piles “hang-up” on boulders encountered at about El. 145m at both borehole locations.

Further investigation is required in the final design stage to confirm the cobbles/boulders and bedrock elevations, as well as downdrag evaluation.

5.2 SETTLEMENT AT THE WEST ABUTMENT LOCATION (BH2)

We understand that a grade raise of about 4.5 m is required at the west abutment location. Our preliminary calculations, based on our experience with similar soils, indicated a possible settlement in the order of 400 mm, due to the settlement of the foundation soils (in addition to the settlement of the embankment under self weight). The surficial sand to sandy silt deposits can be expected to settle rather rapidly in comparison with the underlying clay. The consolidation settlement of the underlying thick clayey deposits (33 to 38 m thick) can be expected to take place in excess of 40 years. Consideration should therefore be given to surcharging which can be speeded-up by means of wick drains. This aspect may need to be carefully considered in the detail design stage. The use of light weight fill can also be considered. In any event a minimum of 2 months surcharge be allowed to ensure all the settlement in the surficial sand to sandy silt deposits be effected (settlement of about 120 mm) prior to driving the piles.

5.3 SLOPE STABILITY OF APPROACH EMBANKMENTS

The slope stability analysis was carried out using the topographic information and preliminary contour plan provided by LEA. The state-of-the–art slope stability software, Slope/W Version 5, was used for analysis. Total stress analysis and effective stress analysis were carried out to assess the short-term and long-term slope stability, respectively.

The soil profiles used for the slope stability were based on the boreholes drilled on each side of the river (BH1 and BH2 located on the east and west sides of the river, respectively). The soil parameters adopted in the analysis are summarised below:

Soil Type	**Unit Weight (kN/m^3)**	**Shear Strength Parameters**			
		Undrained		**Drained**	
		Shear Strength (kPa)	**Angle of internal friction (deg)**	**Cohesion (kPa)**	**Effective angle of internal friction (deg)**
Embankment Fill	20	---	33	---	33
Sand, sandy silt, silty sand	20	---	28	---	28
Clayey silt	17.5	45	---	3	30
Silty clay	16.5	40 to 65 and increase linearly with depth (between El.179m to 160m)	---	5	26

Based on the information provided to us, the grade raise of East Approach Embankment will be minimal, while a 4.5 m high embankment will be constructed on the west side of the river (West Approach Embankment).

In order to design a stable embankment on the weak foundation soils and to limit lateral movements, a factor of safety against slope instability in the long term should be a minimum of 1.5.

Based on the results of slope stability analyses, for the new Approach Embankments slope in the direction of the road alignment (i.e., towards the river), the side/front slope of the West Approach Embankment will be stable at 3H:1V. The east and west riverbank slopes under the proposed Shewfelt Bridge alignment will need to be cut to 3H:1V. The results of slope stability analyses are presented in Figures D1 to D20 of Appendix D and further summarized in the following table.

For East Approach

Slope Profile	Computed Factor of Safety	
	Total Stress Analysis (Short Term)	**Effective Stress Analysis (Long Term)**
Existing condition	1.05 (Figure D1)	0.93 (Figure D6)
Existing condition with 12kPa surcharge	1.04 (Figure D2)	0.88 (Figure D7)
2H:1V cut slope of riverbank with surcharge	1.08 (Figure D3)	1.02 (Figure D8)
3H:1V cut slope of riverbank with surcharge	1.33 (Figure D4)	1.5 (Figure D9)
3H:1V cut slope of riverbank with no surcharge	1.35 (Figure D5)	1.52 (Figure D10)

For West Approach Embankment

Slope Profile	Computed Factor of Safety	
	Total Stress Analysis (Short Term)	**Effective Stress Analysis (Long Term)**
Existing condition	1.04 (Figure D11)	1.05 (Figure D16)
Existing condition with 4.5m embankment and 12kPa surcharge. 2H:1V embankment slope	1.01 (Figure D12)	0.95 (Figure D17)
2H:1V cut slope of riverbank with surcharge and 2H:1V embankment slope	1.05 (Figure D13)	1.09 (Figure D18)
3H:1V cut slope of riverbank with surcharge and 2H:1V embankment slope	1.40 (Figure D14)	1.37 (Figure D19)
3H:1V cut slope of riverbank with surcharge and 3H:1V embankment slope	1.51 (Figure D15)	1.58 (Figure D20)

Considering the West Approach Embankment cross-section in the direction perpendicular to the proposed bridge alignment, the side-slope of the new embankment required to provide a stable embankment was determined by analysing the stability of the new embankment at various slope inclinations. The results of slope stability analyses (see Figures D21 and D22 of Appendix D) indicate that the side slopes of 2H:1V are stable.

In summary, 2H:1V embankment side slopes are considered stable, while the recommended forward slope configuration is 3H:1V for both abutments (including the fill to be used to raise the grade on the west side).

It should be noted that the above analyses were carried out based on the assumptions that the riverbank will be protected from erosion, and the embankment will be constructed using current MTO practice.

5.4 SLOPE STABILITY OF RIVERBANK NEAR GOULAIS BAY ROAD

This issue will be discussed as soon as the contour plan of the site is available to us.

5.5 SCOUR PROTECTION

This sub-section provides preliminary comments and discussions regarding scour protection of the riverbank and bridge foundations.

It is evident that slumping of the riverbank occurred in some areas along the Goulais River. Rip rap and gabion mattresses may not be effective at this site due to possible high water flow. With reference to the MTO Drainage Management Manual, rock protection (machine placed rockfill) is considered a good option for scour protection of the river channel. The scour protection may possibly consist of 600 mm size well-graded rock, as per OPSS 1004. The gradation of the rock could be designed once the channel and water flow parameters are available during detail design. The rock should be placed at a stable inclination of 3H:1V or flatter. A granular filter or a suitable geotextile separator below the scour protection may also be required. In this instance rock need not be well graded. We will be able to provide more comments on these when additional boreholes are drilled and grain-size analyses as well as other details are available.

The rockfill should be placed to at least 1 to 2 m above the high water level. Scour protection is required to protect the proposed bridge foundations, as well as the riverbank (3H:1V cut slopes) under the proposed bridge alignment. It should be considered to extend the scour protection a sufficient distance along the river channel banks to the north and south of the proposed bridge. Due to the extreme erodibility of the natural materials (very loose to loose non-cohesive granular materials), rock protection may be required to extend across the bottom of the river and up to the other side of channel side slope, if non-cohesive materials extend across the base of the channel or a toe trench may also be feasible.

For a better design of scour protection, if required, it is recommended that further study be carried out to define the following parameters for erosion control measures:

- flow rate
- water depth
- type of transported sediments
- detailed cross section survey
- stream pattern and alignment
- channel gradient
- effects of the constriction of river flow due to the construction of the bridge piers
- effects due to extreme ice conditions
- effects of flooding

Channel and bridge scour protection and erosion control should be designed by experienced Hydraulic Engineer.

5.6 FROST PROTECTION

Design frost protection depth for the general area is about 2.1 m. Therefore, a permanent soil cover of about 2.1 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE

The sub-soil information and recommendations contained in this report should be used solely for the purpose of preliminary geotechnical assessment of this project and should not be used for final/detailed design. Additional foundation investigation should be carried out to fulfill the detailed design requirements.

The Limitations of Report, as quoted in Appendix F, are an integral part of this report.

SHAHEEN & PEAKER LIMITED

K.R. Peaker, PhD, P.Eng

Z.S. Ozden, P.Eng.

Drawings

METRIC

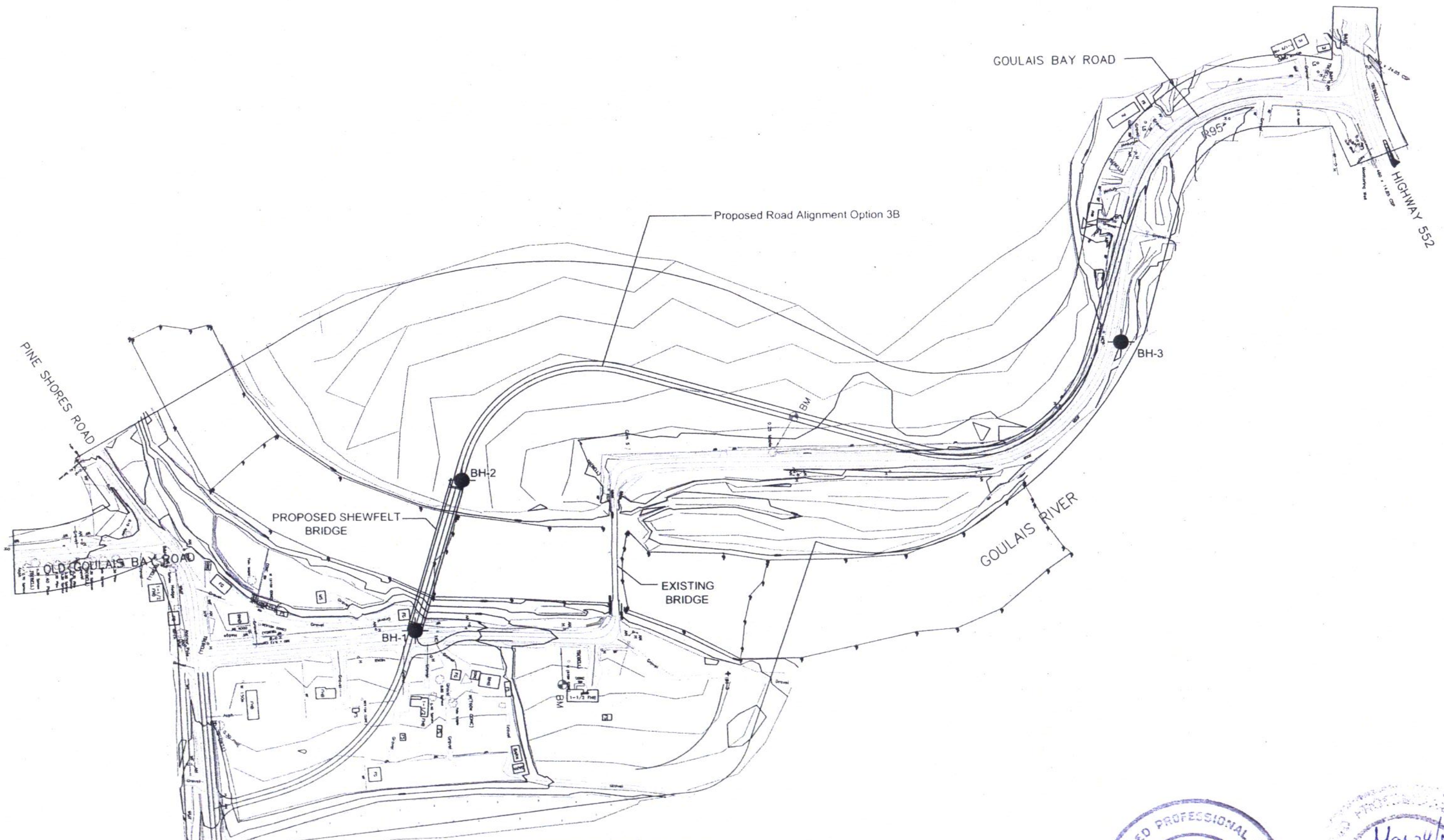
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

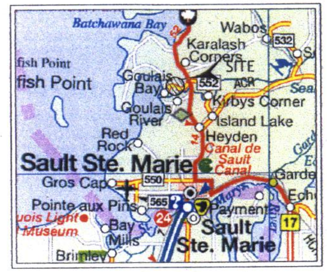
CONT No.

GWP: 5290-04-00

ADVANCE FOUNDATION INVESTIGATION
SHEWFELT BRIDGE REPLACEMENT
BOREHOLE LOCATION PLAN

SHAHEEN & PEAKER LIMITED





KEY PLAN
N.T.S

LEGEND

● Approx. Borehole Location (THIS INVESTIGATION)

BH No.	ELEV.	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	193.3	5 175 914.4	275 624.7
2	187.1	5 175 950.6	275 523.1
3	188.1	5 176 401.5	275 449.8

=NOTE=

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

1. THIS DRAWING TO BE READ IN CONJUCTION WITH ASSOCIATED FOUNDATION INVESTIGATION REPORT.
2. DRAWING NOT TO SCALE. DO NOT SCALE DRAWING.
3. ALL BOREHOLE LOCATIONS ARE APPROXIMATE.
4. FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.
5. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION. THE PROPOSED STRUCTURE LOCATION IS SHOWN FOR ILLUSTRATION PURPOSES ONLY AND MAY NOT BE CONSISTENT WITH THE FINAL DESIGN CONFIGURATION.
6. SURVEY BASE PLAN PROVIDED BY LEA CONSULTING LIMITED.

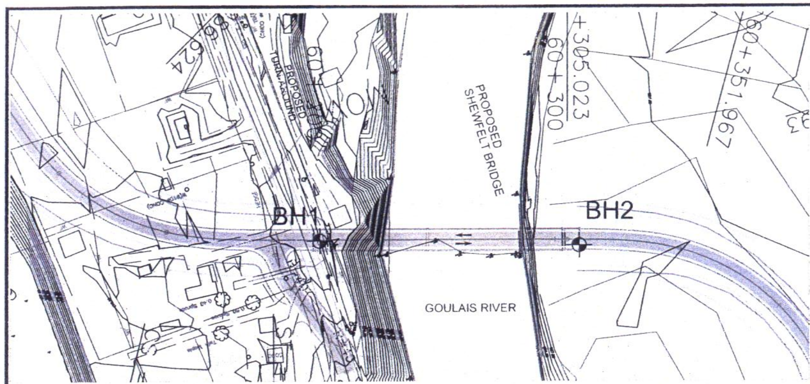
REGISTERED PROFESSIONAL ENGINEER
May 24/06
Z. S. OZDEN
ZSO
PROVINCE OF ONTARIO

May 24/06
K. R. PEAKER

REV.	DATE	BY	DESCRIPTION

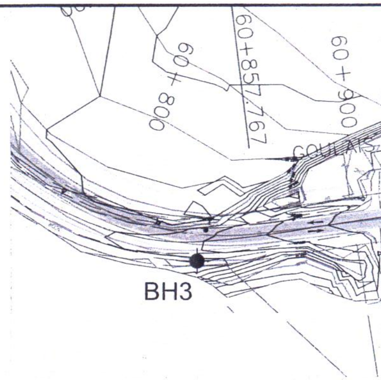
Geocres No. 41K-79

HWY No. GOULAIS BAY ROAD			DIST ALGOMA
SUBM'D ZO	CHECKED ZO	DATE May 2006	SITE 38S-031
DRAWN JZ	CHECKED	APPROVED ZO	DWG 1



PLAN

SCALES 25m 0 25 50m



PLAN

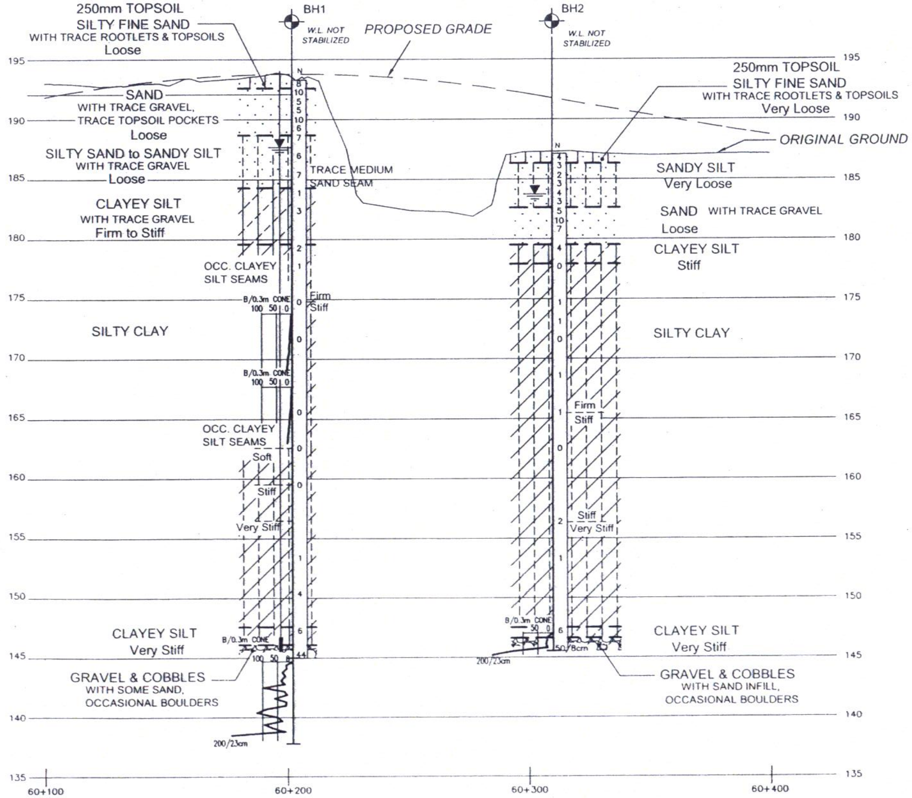
SCALES 25m 0 25 50m

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

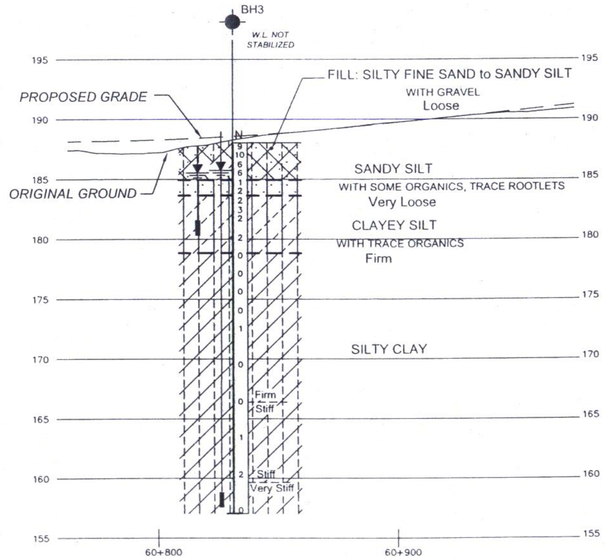
NOTES:

- FOR DETAILED SUBSURFACE CONDITIONS AND DYNAMIC CONE PENETRATION TESTS REFER TO RECORD OF BOREHOLE SHEETS.
- FOR BOREHOLES LOCATIONS REFER TO DRAWING 1.



℄ PROFILE

SCALES VERT 5m 0 5 10m HOR 25m 0 25 50m



℄ PROFILE

SCALES VERT 5m 0 5 10m HOR 25m 0 25 50m

REGISTERED PROFESSIONAL ENGINEER
May 24/06
Z. S. OZDEN
PROVINCE OF ONTARIO

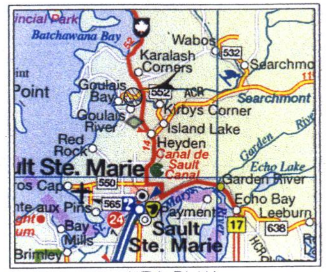
LICENSED PROFESSIONAL ENGINEER
May 24/06
K. R. PEAKER
PROVINCE OF ONTARIO

CONT No.

GWP: 5290-04-00

ADVANCE FOUNDATION INVESTIGATION
SHEWFEL BRIDGE REPLACEMENT
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- Bore Hole & Cone
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation
- Water Level in Piezometer
- Piezometer

BH No.	ELEV.	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	193.3	5 175 914.4	275 624.7
2	187.1	5 175 950.6	275 523.1
3	188.1	5 176 401.5	275 449.8

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No. 41K-79

HWY No. GOULAIS BAY ROAD			DIST ALGOMA
SUBM'D ZO	CHECKED ZO	DATE May, 2006	SITE 38S-031
DRAWN JZ	CHECKED	APPROVED ZO	DWG 2

Appendix A

Record of Borehole Sheets
(Advance Foundation Investigation)

RECORD OF BOREHOLE No BH1

METRIC

GWP 5290-04-00	LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7		ORIGINATED BY G.I.
DIST Algoma	HWY 17	BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT	COMPILED BY J.Z.
DATUM Geodetic		DATE 2/28/2006	CHECKED BY KSH

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.3	Ground Surface					
0.0	250mm**TOPSOIL** **SILTY FINE SAND** with trace rootlets & topsoil dark brown, loose, moist (frozen)	1	SS	8		
192.7 / 0.7	**SAND** with trace gravel, trace topsoil pockets brown, loose, moist	2	SS	10		
		3	SS	5		
		4	SS	5		
		5	SS	10		
		6	SS	6		
188.8 / 4.6	**SILTY SAND to SANDY SILT** with trace gravel brown loose, wet	7	SS	7		
		8	SS	6		
	trace medium sand seam	9	SS	7		
184.3 / 9.0	**CLAYEY SILT** with trace gravel grey, firm to stiff	10	SS	1		
		11	SS	3		
		12	TW	PH	17.6	50mm diameter Shelby tube sample (0) 79 21
179.6 / 13.7	**SILTY CLAY** reddish grey, firm	13	SS	2		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (193 to 179); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

Field vane sensitivity values plotted: 4, 4, 4, 3, 4, 4, 4, 3.

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH1

METRIC

GWP 5290-04-00	LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7	ORIGINATED BY G.I.
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT	COMPILED BY J.Z.
DATUM Geodetic	DATE 2/28/2006	CHECKED BY KSH

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT

SILTY CLAY
reddish grey, firm to stiff
occ. clayey silt seams
firm
------
stiff

SAMPLES NUMBER	TYPE	"N" VALUES	WATER CONTENT	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14	SS	1	~40%		
15	SS	0			
16	SS	0	~65%		N-value not reliable
17	TW	PH	Wp ~19%, w ~40%, WL ~46%	16.6	70mm diameter Shelby tube sample (0) 34 66
18	SS	0	~47%		N-value not reliable

Elevation scale: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH1

METRIC

GWP 5290-04-00	LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7	ORIGINATED BY G.I.
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT	COMPILED BY J.Z.
DATUM Geodetic	DATE 2/28/2006	CHECKED BY KSH

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT
SAMPLES: NUMBER | TYPE | "N" VALUES
GROUND WATER CONDITIONS | ELEVATION SCALE
DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100)
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)
UNIT WEIGHT γ kN/m3
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Description	Sample No.	Type	"N" Values	Remarks
SILTY CLAY reddish grey, soft to very stiff; occ. clayey silt seams	19	SS	0	
soft				
stiff	20	SS	0	
very stiff	21	TW	PH	50mm diameter Shelby tube sample. γ = 15.4; (0) 25 75
	22	SS	1	Augering stopped on March 2, 2006 at 39.6m (El.153.7) and wash boring started on March 8, 2006 using NW casing.
	23	SS	4	

Elevation scale: 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149

Field vane sensitivity values: 2, 2, 3, 3, 4, 4, 3, 3, 3, 2

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

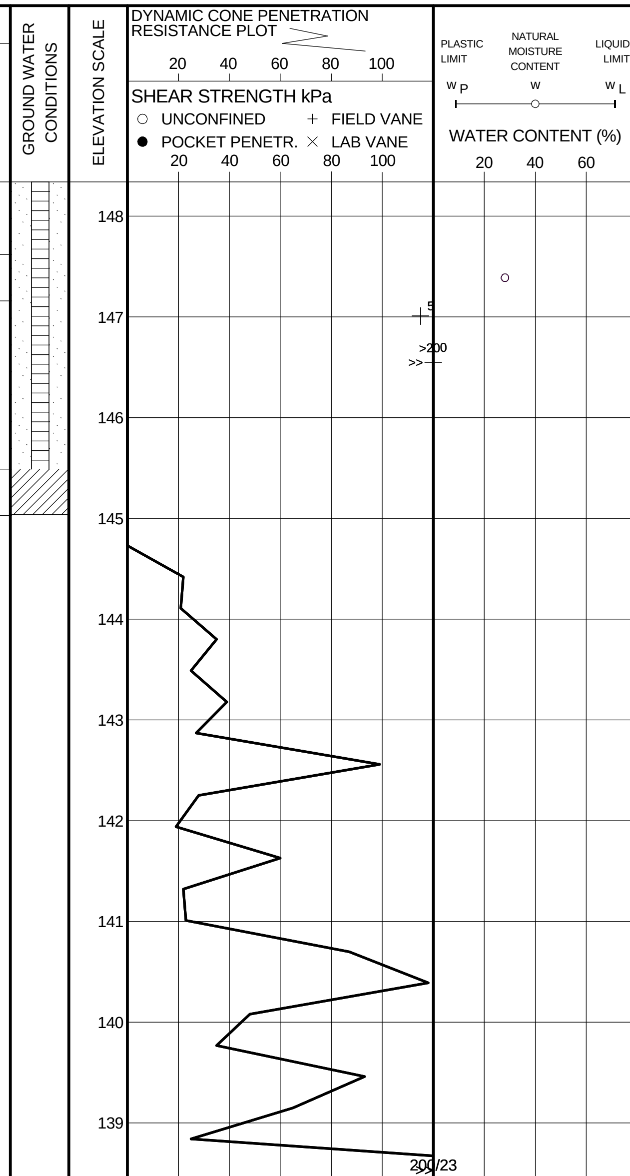
20 / 15 ◆ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH1

METRIC

GWP	5290-04-00	LOCATION	Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7	ORIGINATED BY	G.I.
DIST	Algoma HWY 17	BOREHOLE TYPE	Hollow Stem Augers+Wash Boring+DCPT	COMPILED BY	J.Z.
DATUM	Geodetic	DATE	2/28/2006	CHECKED BY	KSH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
147.6 / 45.7	**CLAYEY SILT** grey, very stiff	24	SS	6	
146.1 / 47.2	**GRAVEL & COBBLES** with some sand, occasional boulders reddish brown, wet	25	SS	44	
145.0 / 48.3	End of Borehole. Piezometer installed to 47.8m (El. 145.5m). Water level on: Mar. 9, 2006=3.3m (El. 190.0m) Mar. 10, 2006=4.5m (El. 188.8m) Mar. 11, 2006=5.8m (El. 187.5m) Mar. 12, 2006=5.6m (El. 187.7m) below ground surface.				
138.4 / 54.9	End of DCPT. Dynamic Cone Penetration Test (DCPT) perfromed from: 19.8m (El. 173.5m) to 24.3m (El. 169.0m), 25.9m (El. 167.4m) to 30.5m (El. 162.8m), and 48.5m (El. 144.8m) to 54.9m (El. 138.4m).				



UNIT WEIGHT γ kN/m³ — GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Ministry of Transportation
Ontario

SPT1156A

RECORD OF BOREHOLE No BH2

METRIC

GWP 5290-04-00 LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 950.6; E 275 523.1 ORIGINATED BY G.I.

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT COMPILED BY J.Z.

DATUM Geodetic DATE 3/10/2006 CHECKED BY KSH

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.1	Ground Surface							
0.0	250mm **TOPSOIL** **SILTY FINE SAND** with trace topsoil & rootlets brown, very loose, moist	1	SS	4		187		
186.3 / 0.8	**SANDY SILT** brown, very loose, moist	2	SS	3		186		
		3	SS	2		185		
		4	SS	3		184		
		5	SS	4	▼			
		6	SS	3		183		
182.6 / 4.5	**SAND** with trace gravel brown, loose, wet	7	SS	5		182		
		8	SS	10		181		
		9	SS	7		180		
179.5 / 7.6	**CLAYEY SILT** grey, stiff	10	SS	4		179		
178.0 / 9.2	**SILTY CLAY** reddish grey, firm	11	SS	0		178, 177		
		12	TW	PH		176	18.0	50 mm diameter Shelby tube sample. 2 2 64 32
		13	SS	1		175, 174		
		14	SS	1		173		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH2

METRIC

GWP 5290-04-00	LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 950.6; E 275 523.1		ORIGINATED BY G.I.
DIST Algoma	HWY 17	BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT	COMPILED BY J.Z.
DATUM Geodetic		DATE 3/10/2006	CHECKED BY KSH

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT

SAMPLES: NUMBER | TYPE | "N" VALUES

GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

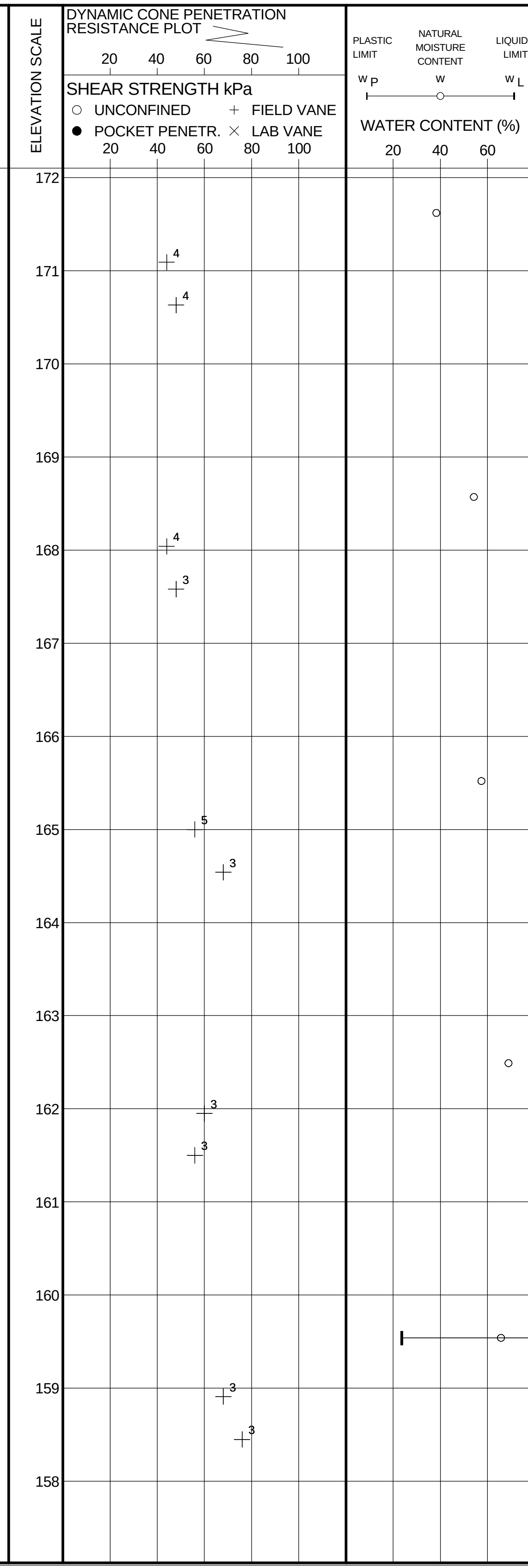
UNIT WEIGHT γ kN/m3

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

SILTY CLAY
reddish grey, firm to stiff

firm
----
stiff

NUMBER	TYPE	"N" VALUES
15	SS	0
16	SS	1
17	SS	1
18	SS	0
19	TW	PH



Unit weight (Sample 19): 15.3

Remarks:
Wash boring (NW casing) started at 15.2m (El. 171.9m)

50 mm diameter Shelby tube sample.
(0) 22 78

Continued Next Page

+3, ×3 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH2

METRIC

GWP 5290-04-00 LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 950.6; E 275 523.1 ORIGINATED BY G.I.

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT COMPILED BY J.Z.

DATUM Geodetic DATE 3/10/2006 CHECKED BY KSH

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY reddish grey, stiff to very stiff (stiff ------------ very stiff)	20	SS	2		
		21	SS	1		
		22	TW	PH	16.7	50 mm diameter Shelby tube sample. (0) 25 75
147.5 / 39.6	**CLAYEY SILT** grey, very stiff	23	SS	6		could not advance the vane.
146.6 / 40.5	**GRAVEL and COBBLES** with sand infill, occasional boulders reddish brown, wet	24	SS	50/8		
145.5 / 41.6	End of borehole.					

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146

Field vane sensitivity values: 4, 3, 3, 4, 2, 3; 135

DCPT: 200/23

End of borehole.

Dynamic Cone Penetration Test carried out from 40.2m (El. 146.9m) to 41.6m (El. 145.5m).

* Water level in open borehole: 3.4m (not stabilized).

+ 3, × 3 : Numbers refer to Sensitivity

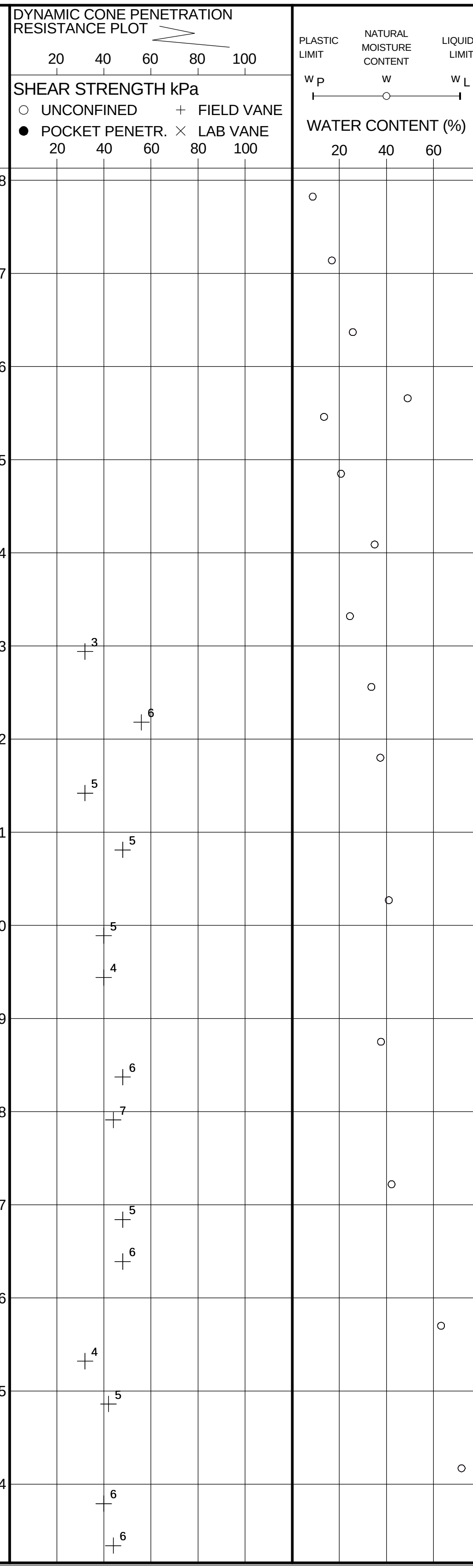
20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH3

METRIC

GWP 5290-04-00	LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 176 401.5; E 275 449.8		ORIGINATED BY G.I.
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY J.Z.
DATUM Geodetic	DATE 3/4/2006		CHECKED BY KSH

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
188.1	Ground Surface				
0.0	**FILL : SILTY FINE SAND to SANDY SILT** with gravel brown, loose, moist	1	SS	9	frozen
		2	SS	10	
	with sandy silt organics, peat, trace rootlets	3	SS	6	
	with fine to medium sand trace coarse sand trace gravel	4	SS	6	
185.1 / 3.1	**SANDY SILT** with some organics, trace rootlets brown, very loose, wet	5	SS	1	
		6	SS	2	
183.7 / 4.4	**CLAYEY SILT** with trace organics grey to reddish grey, firm	7	SS	2	
		8	SS	3	
	stiff	9	SS	2	
		10	SS	2	
179.0 / 9.2	**SILTY CLAY** reddish grey, firm	11	SS	0	
		12	SS	0	
		13	SS	0	
		14	SS	0	



UNIT WEIGHT γ kN/m³ — GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH3

METRIC

GWP 5290-04-00	LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 176 401.5; E 275 449.8	ORIGINATED BY G.I.	
DIST Algoma HWY 17	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY J.Z.	
DATUM Geodetic	DATE 3/4/2006	CHECKED BY KSH	

SOIL PROFILE: ELEV DEPTH / DESCRIPTION / STRAT PLOT
SAMPLES: NUMBER / TYPE / "N" VALUES
GROUND WATER CONDITIONS; ELEVATION SCALE
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m3
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

SILTY CLAY
reddish grey, firm to very stiff
firm
------
stiff

trace seams of clayey silt

stiff
----
very stiff

NUMBER	TYPE	"N" VALUES
15	SS	1
16	SS	0
17	SS	0
18	SS	1
19	SS	2

Elevation scale: 173, 172, 171, 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159

Field vane sensitivity labels: 5, 4, 4, 5, 4, 4, 3, 4, 3, 3

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH3

METRIC

GWP 5290-04-00 LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 176 401.5; E 275 449.8 ORIGINATED BY G.I.

DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY J.Z.

DATUM Geodetic DATE 3/4/2006 CHECKED BY KSH

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY reddish grey, very stiff						158				
			20	SS	0				○		
157.2 31.0	End of borehole.							>>5 135			
	Upper piezometer installed at 7.6m (El.180.5m). Water level on: Mar. 7, 2006 = 2.8m (El. 185.3m) Mar. 8, 2006 = 2.7m (El. 185.4m) Mar. 9, 2006 = 2.7m (El. 185.4m) Mar. 10, 2006 = 2.7m (El. 185.4m) Mar. 11, 2006 = 2.5m (El. 185.6m) Mar. 12, 2006 = 2.5m (El. 185.6m) below ground surface. Lower piezometer installed at 30.5m (El. 157.6m) Mar. 7, 2006 = 4.5m (El. 183.6m) Mar. 8, 2006 = 3.2m (El. 184.9m) Mar. 9, 2006 = 2.8m (El. 185.3m) Mar. 10, 2006 = 2.7m (El. 185.4m) Mar. 11, 2006 = 2.5m (El. 185.6m) Mar. 12, 2006 = 2.3m (El. 185.8m) below ground surface.							>>4 135			

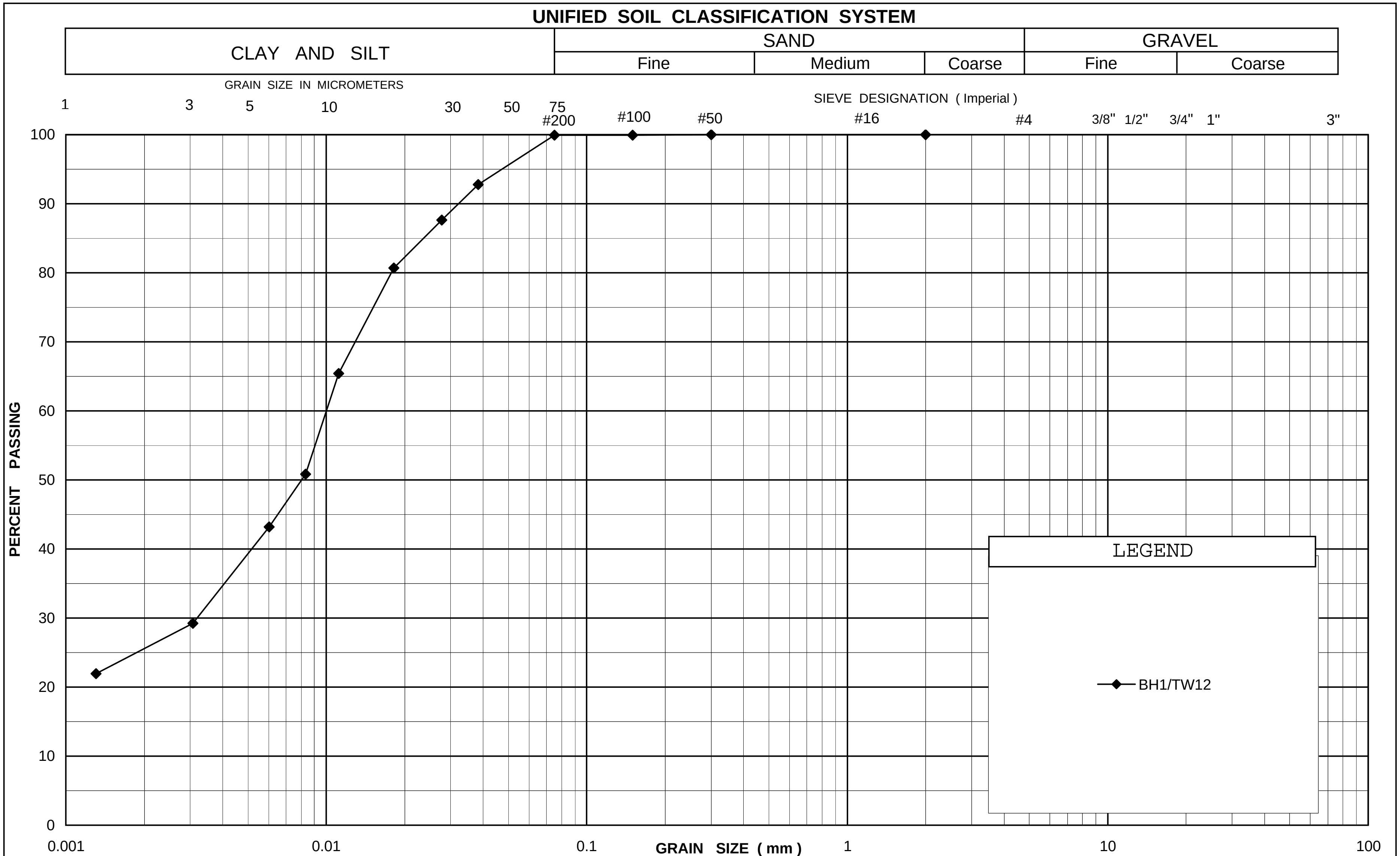
Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT scale 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE, scale 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60.

+3, ×3 : Numbers refer to Sensitivity

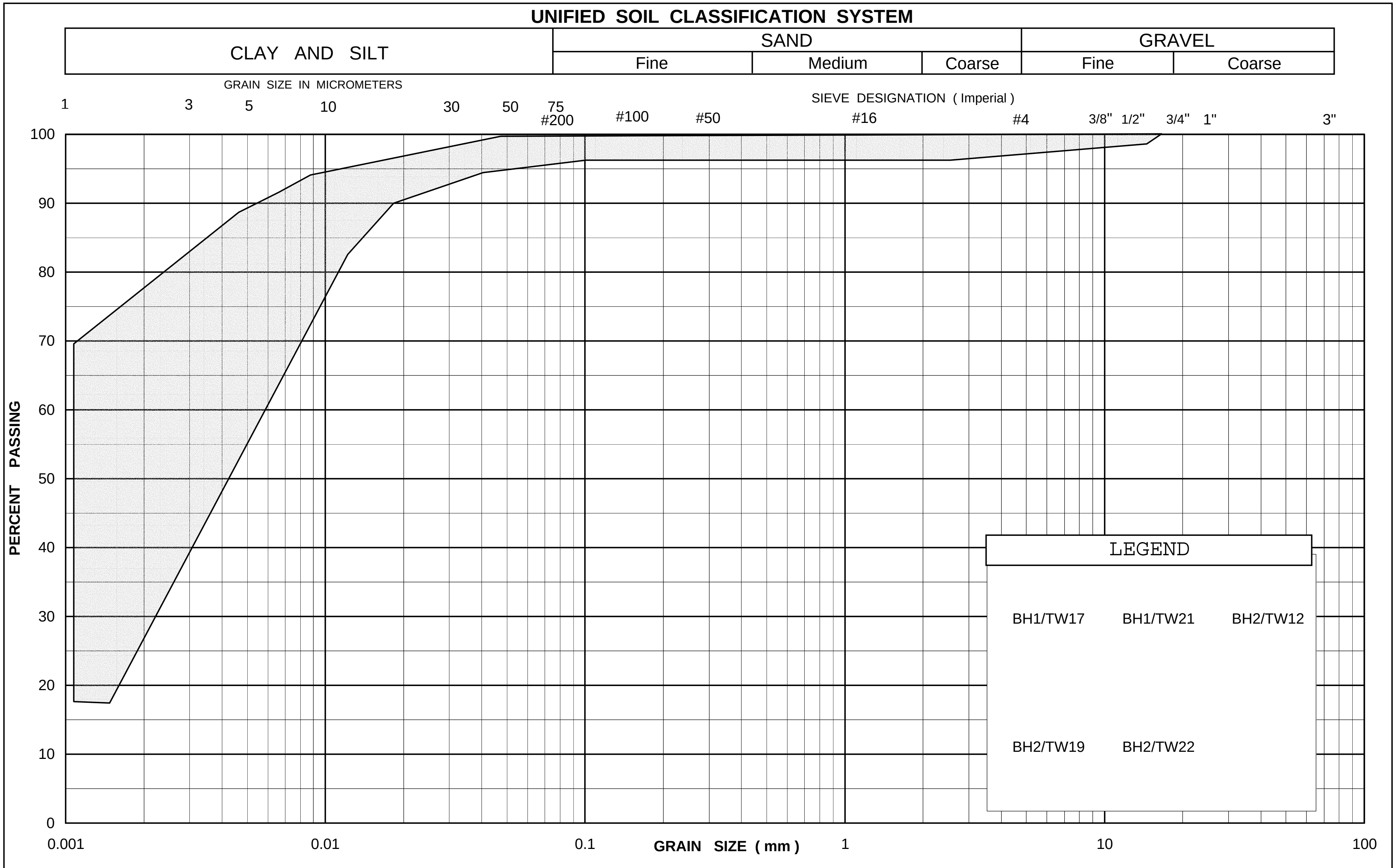
20 / 15 5 / 10 (%) STRAIN AT FAILURE

Appendix B

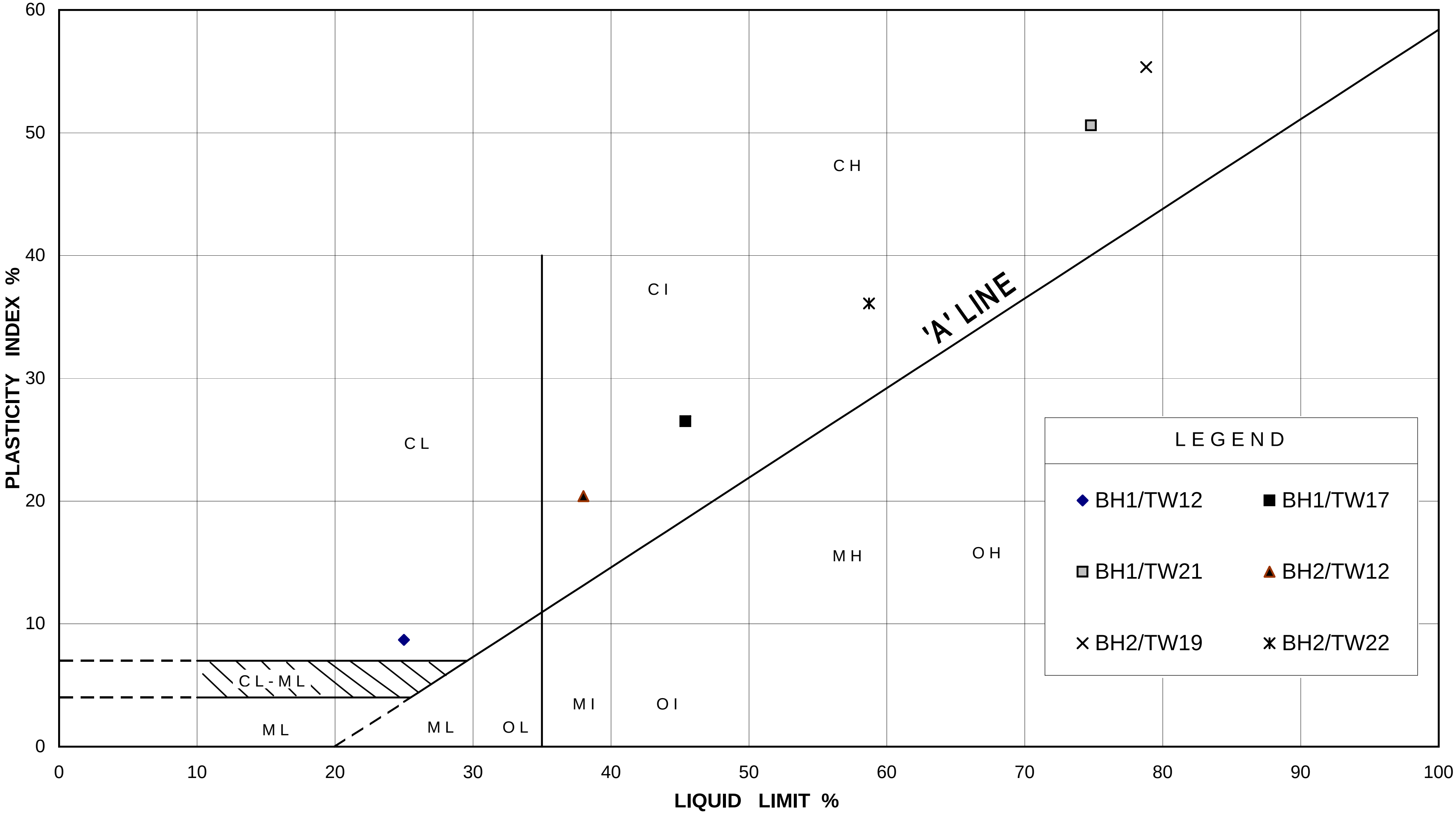
Laboratory Test Results



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION CLAYEY SILT	FIGURE No. B1 REF. No. SPT 1156A DATE APRIL, 2006



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION SILTY CLAY	FIGURE No. B2 REF. No. SPT 1156A DATE APRIL, 2006



SHAHEEN & PEAKER LIMITED	PLASTICITY CHART	FIG No B3 G.W.P. 5290-04-00 REF No SPT 1156A

Appendix C

Plots of Measured Undrained Shear Strength Results with Depth

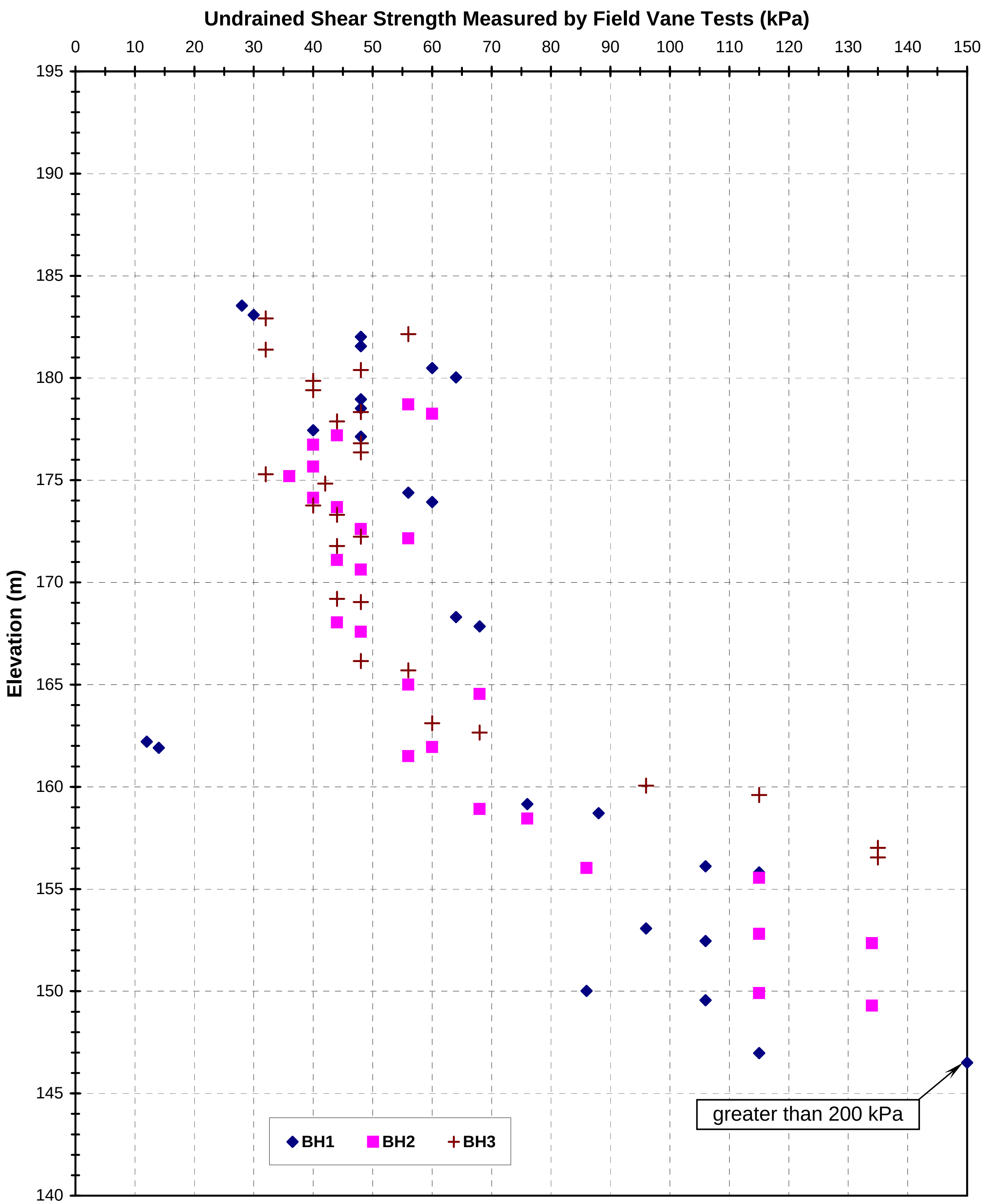


Figure C1 Plot of Undrained Shear Strength with Elevation for Boreholes 1, 2 and 3

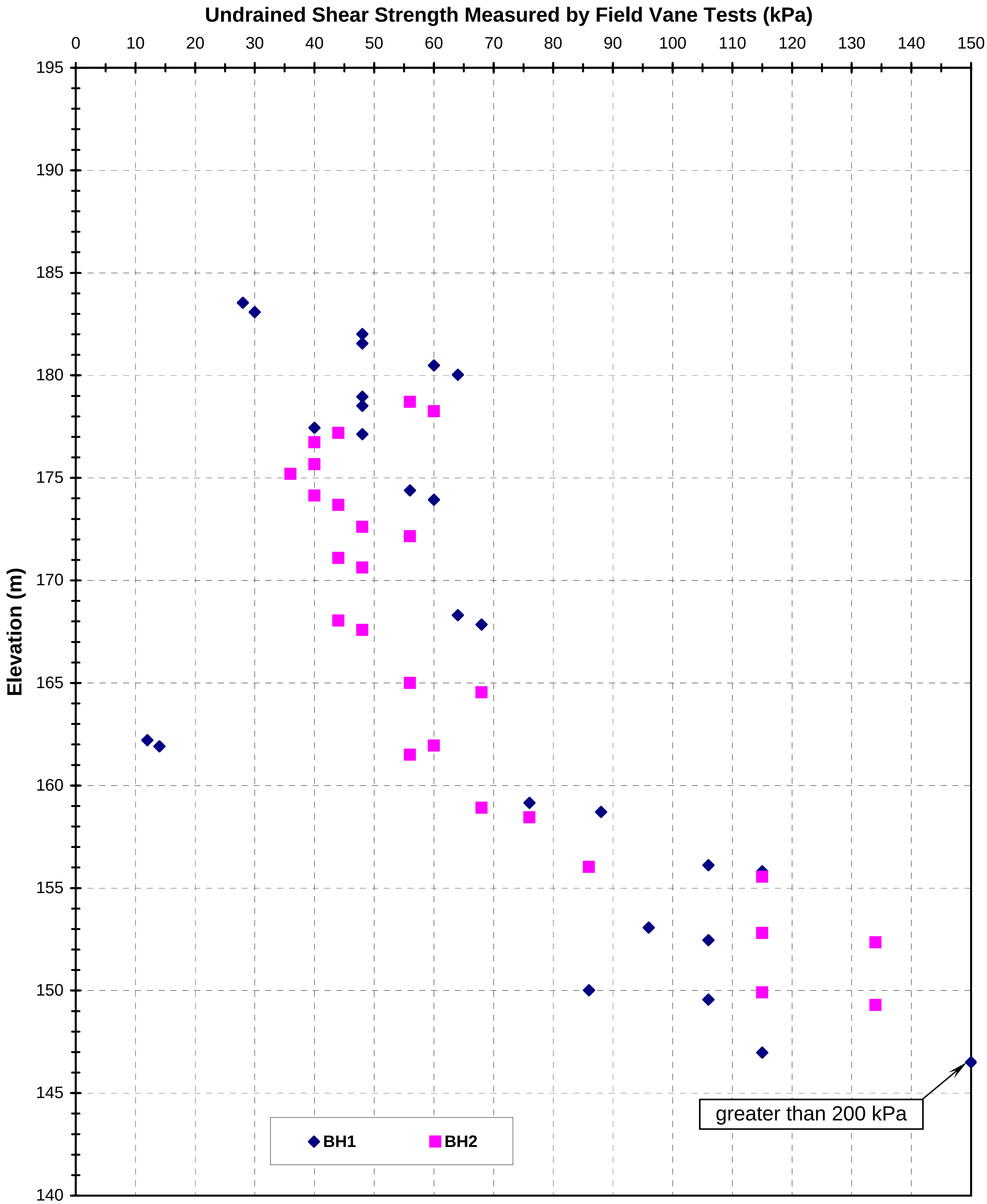


Figure C2 Plot of Undrained Shear Strength with Elevation for Boreholes 1 and 2

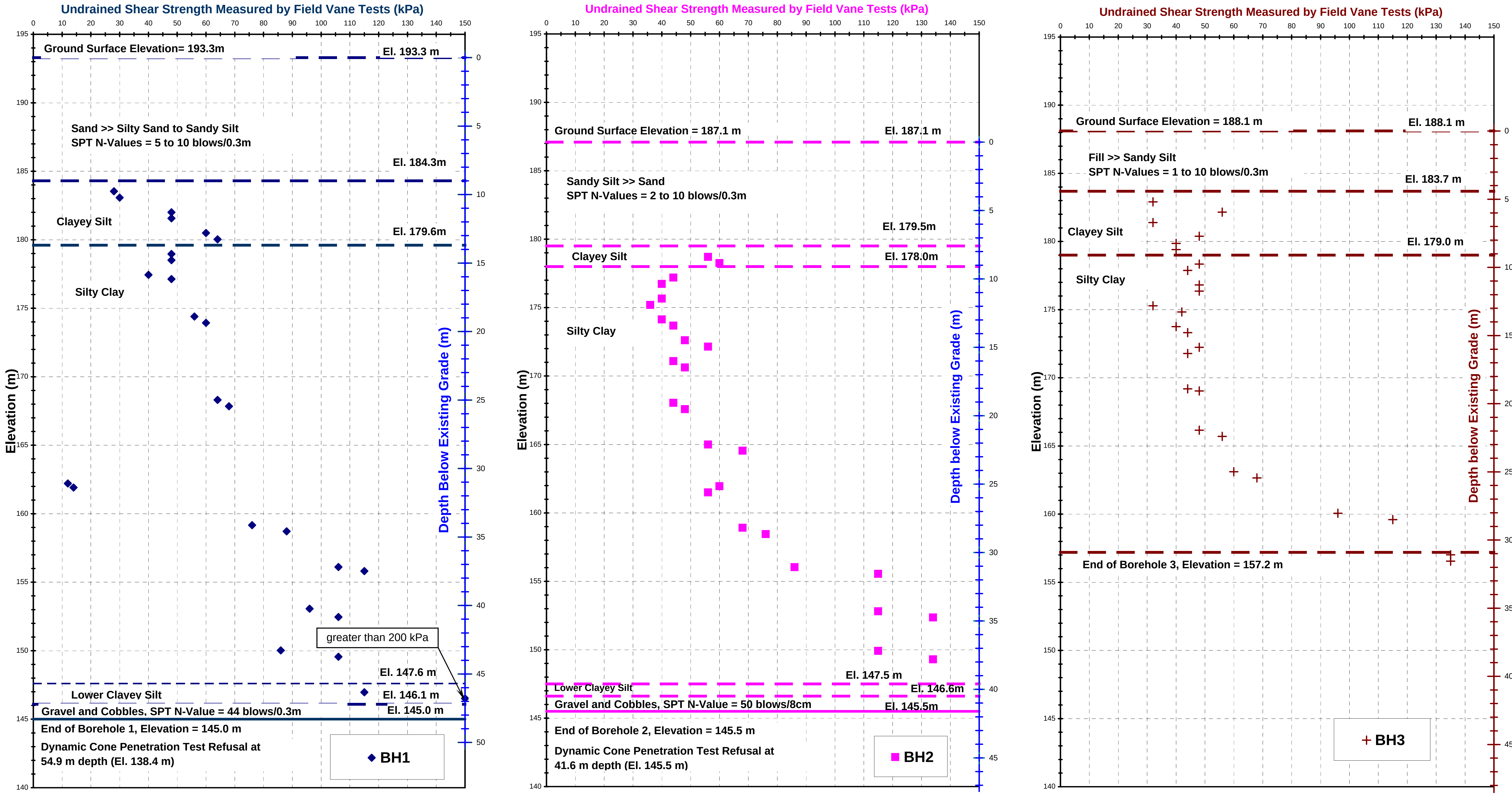
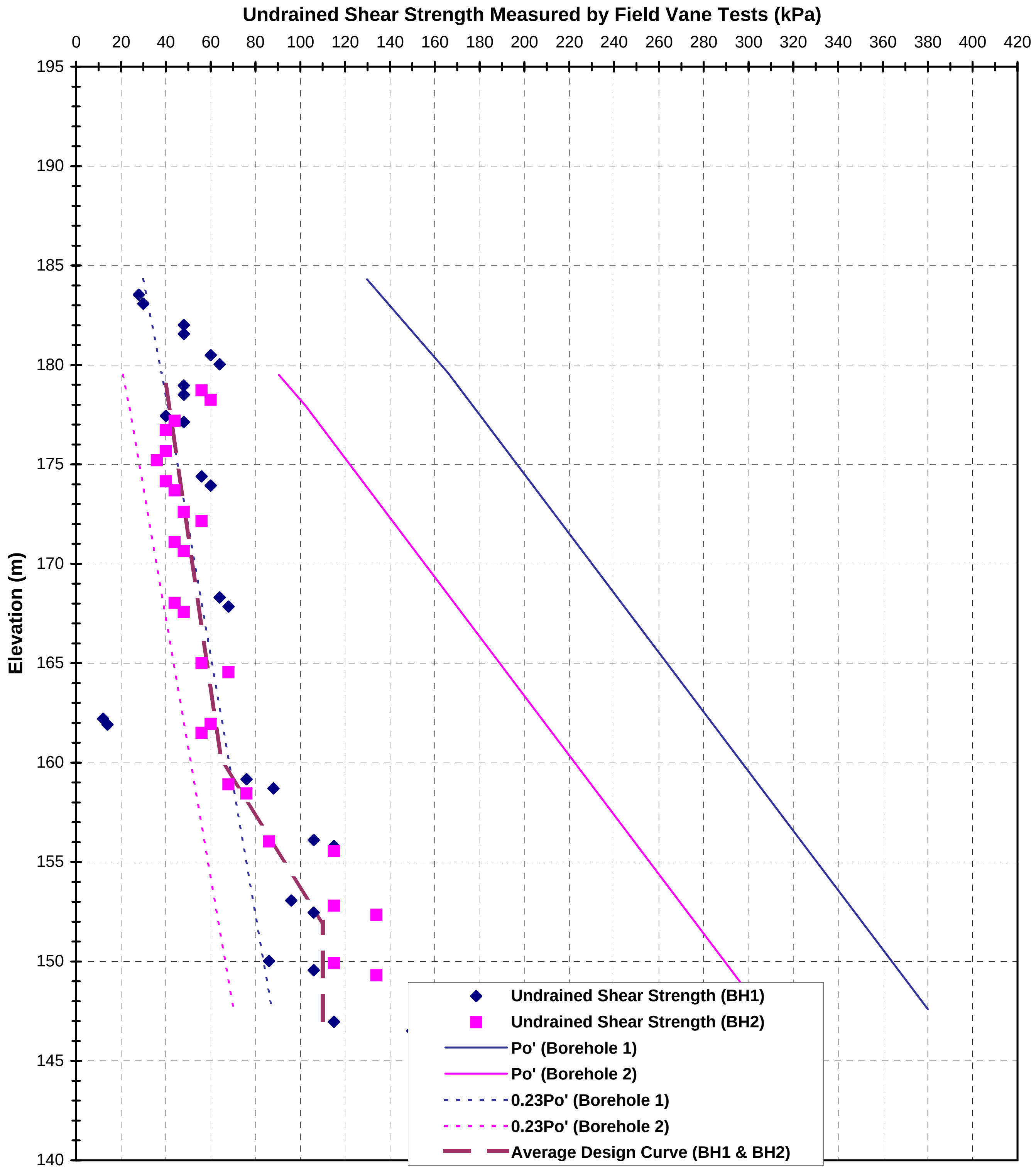


Figure C3 Plots of Undrained Shear Strength Measured in Each Borehole (Boreholes 1, 2 and 3)



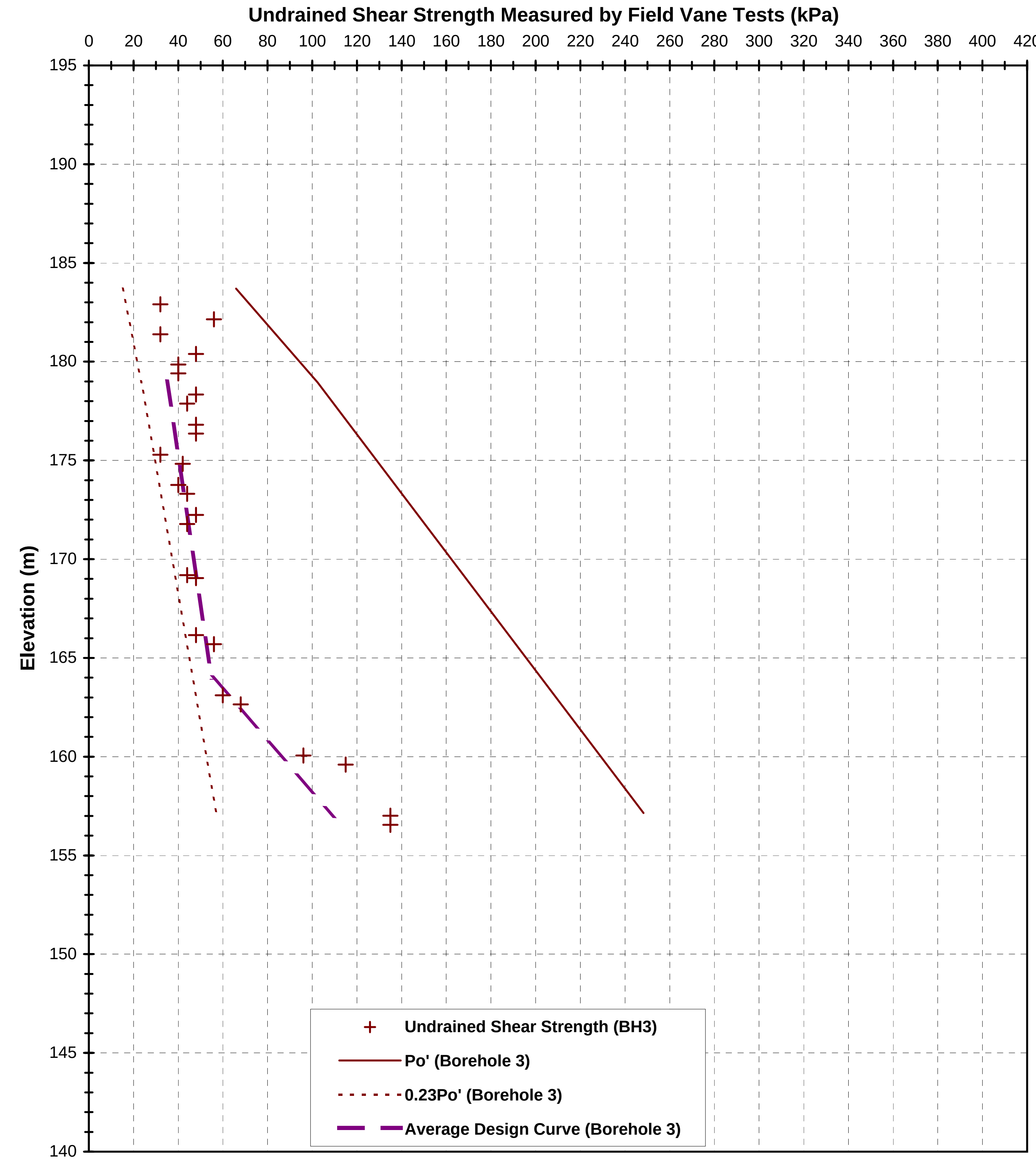


Figure C4 Plots of Stress Ratio and Average Design Curve with Elevation for Boreholes 1, 2 and 3

Appendix D

Results of Slope Stability Analyses for Bridge Approach Embankments (BH1 and BH2)

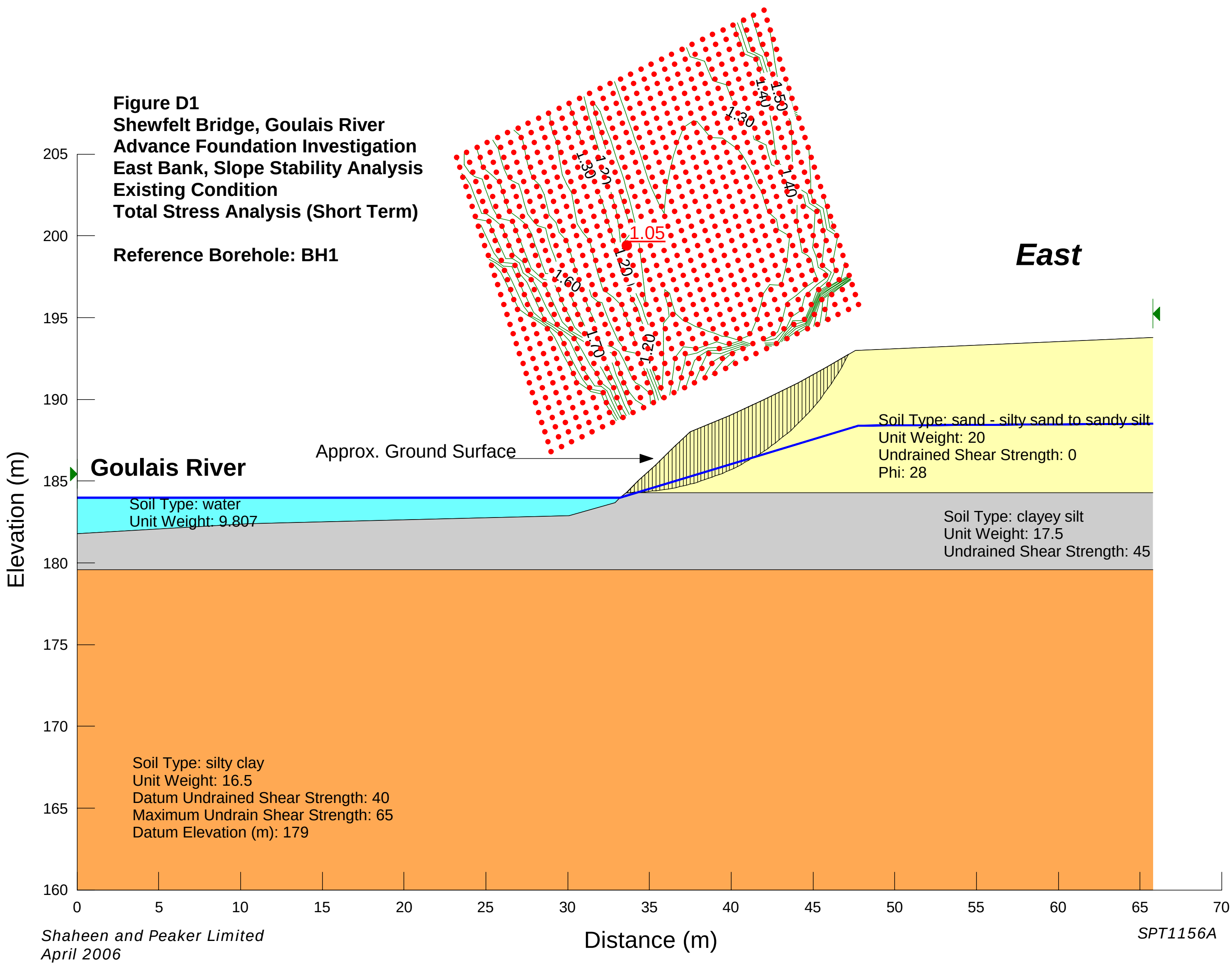


Figure D1
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Existing Condition
Total Stress Analysis (Short Term)
Reference Borehole: BH1
1.05
East
Goulais River
Approx. Ground Surface
Soil Type: water
Unit Weight: 9.807
Soil Type: sand - silty sand to sandy silt
Unit Weight: 20
Undrained Shear Strength: 0
Phi: 28
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrain Shear Strength: 65
Datum Elevation (m): 179
Elevation (m)
Distance (m)
Shaheen and Peaker Limited
April 2006
SPT1156A

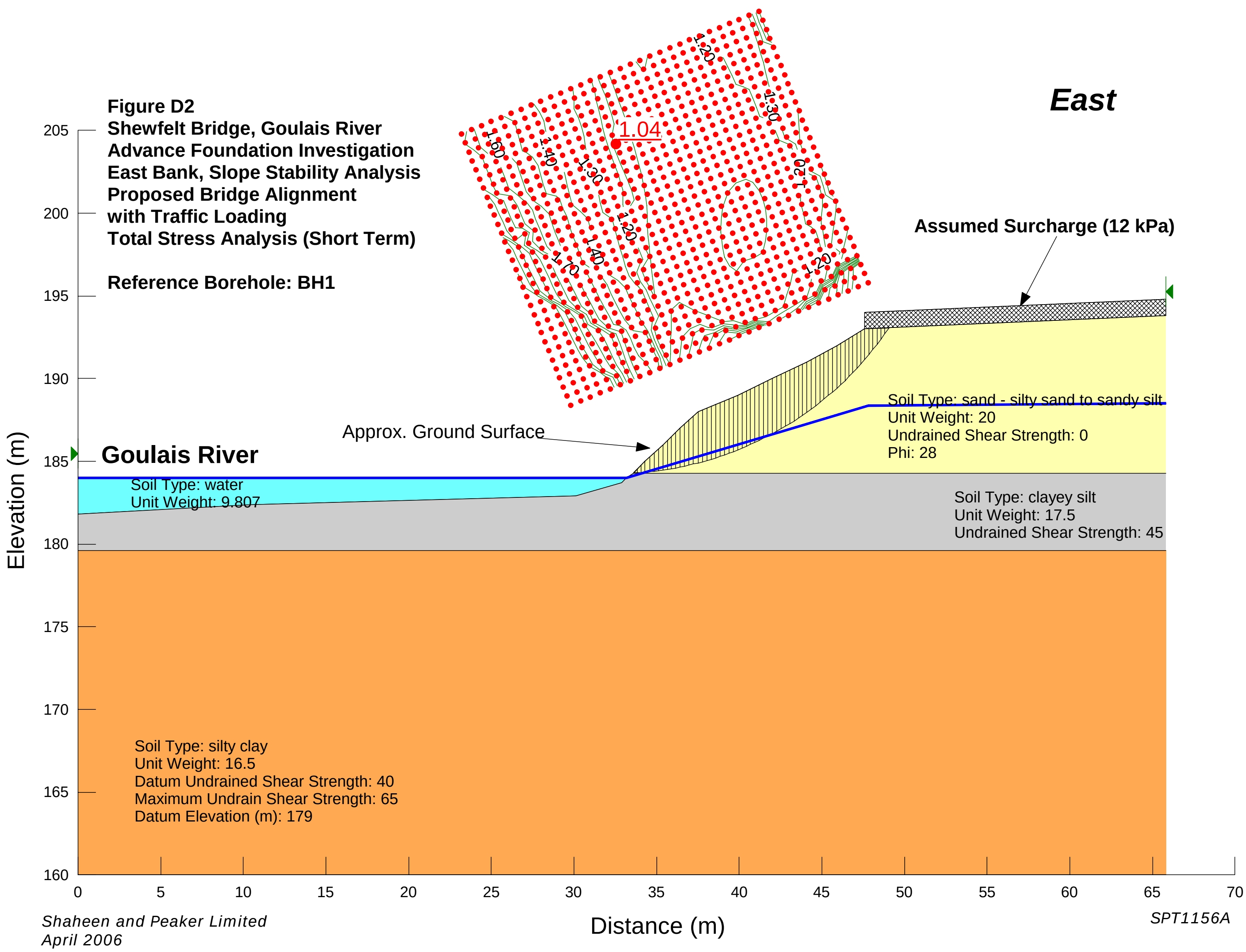


Figure D2
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Traffic Loading
Total Stress Analysis (Short Term)
Reference Borehole: BH1
East
1.04
1.20
1.30
1.40
1.60
1.70
Assumed Surcharge (12 kPa)
Approx. Ground Surface
Goulais River
Soil Type: water
Unit Weight: 9.807
Soil Type: sand - silty sand to sandy silt
Unit Weight: 20
Undrained Shear Strength: 0
Phi: 28
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrain Shear Strength: 65
Datum Elevation (m): 179
Elevation (m)
Distance (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Shaheen and Peaker Limited
April 2006
SPT1156A

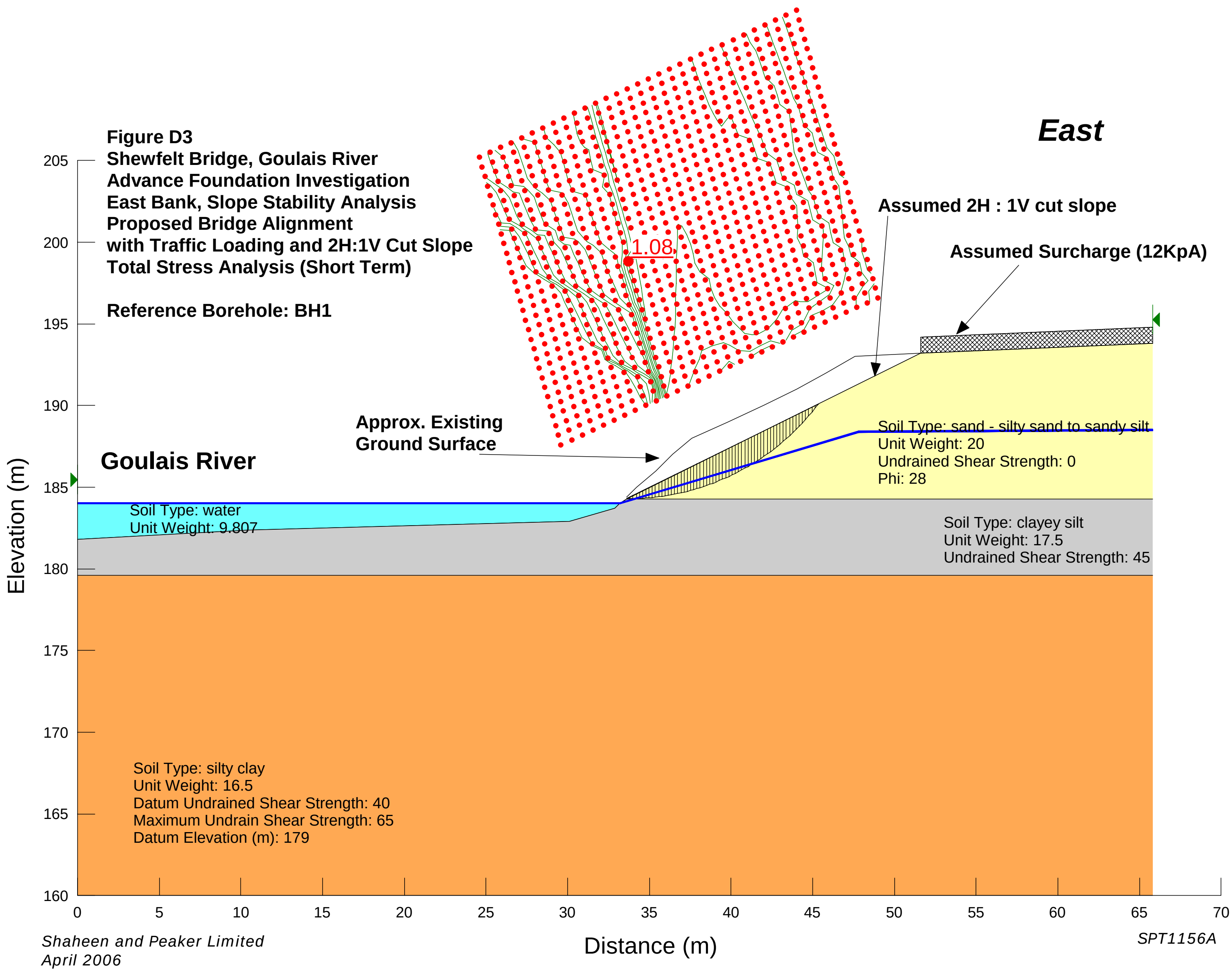


Figure D3
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Traffic Loading and 2H:1V Cut Slope
Total Stress Analysis (Short Term)
Reference Borehole: BH1
East
1.08
Assumed 2H : 1V cut slope
Assumed Surcharge (12KpA)
Approx. Existing
Ground Surface
Goulais River
Soil Type: sand - silty sand to sandy silt
Unit Weight: 20
Undrained Shear Strength: 0
Phi: 28
Soil Type: water
Unit Weight: 9.807
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrain Shear Strength: 65
Datum Elevation (m): 179
Elevation (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Distance (m)
Shaheen and Peaker Limited
April 2006
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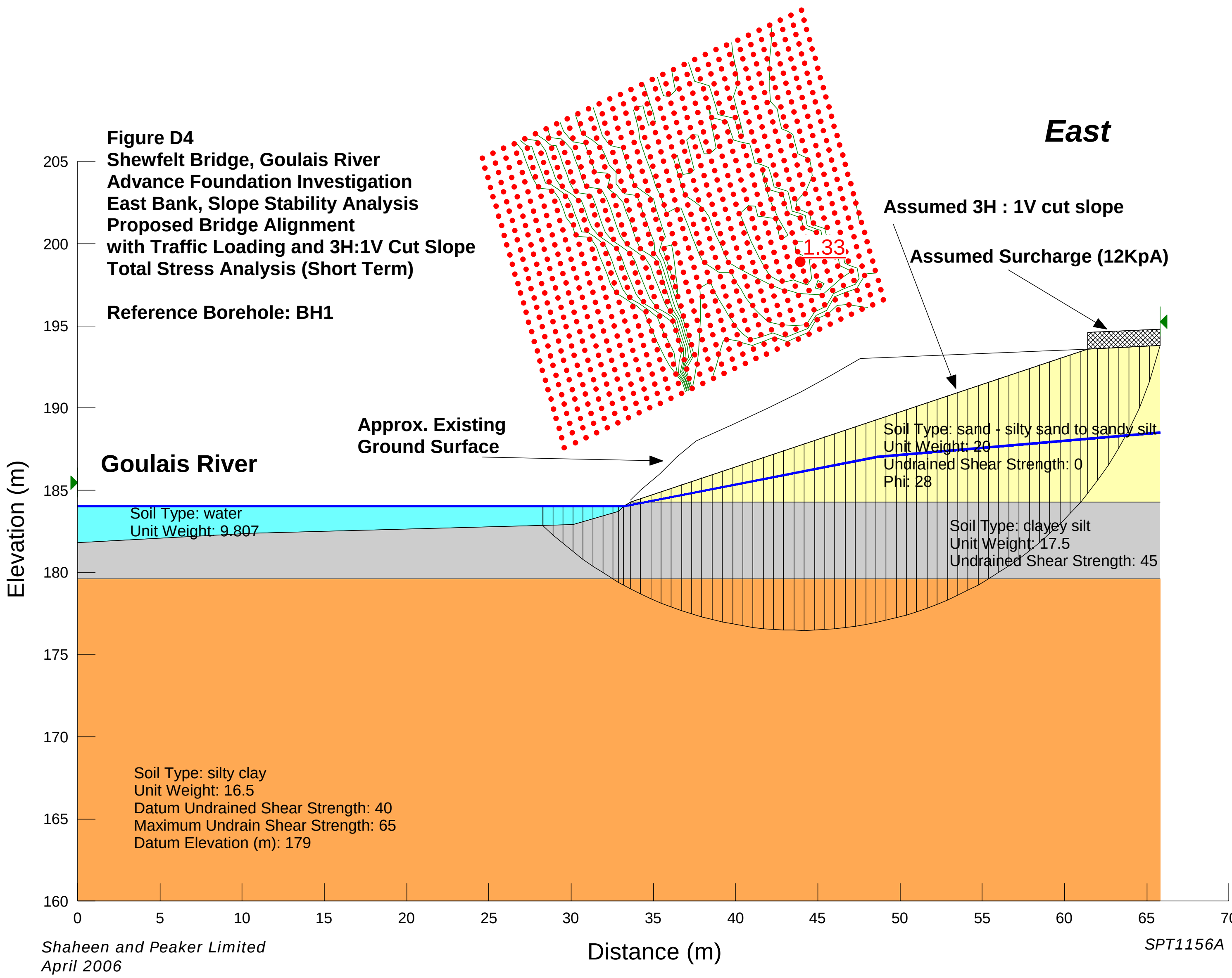

Figure D4
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Traffic Loading and 3H:1V Cut Slope
Total Stress Analysis (Short Term)
Reference Borehole: BH1
East
Assumed 3H : 1V cut slope
1.33
Assumed Surcharge (12KpA)
Approx. Existing
Ground Surface
Goulais River
Soil Type: sand - silty sand to sandy silt
Unit Weight: 20
Undrained Shear Strength: 0
Phi: 28
Soil Type: water
Unit Weight: 9.807
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrain Shear Strength: 65
Datum Elevation (m): 179
Elevation (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Distance (m)
Shaheen and Peaker Limited
April 2006
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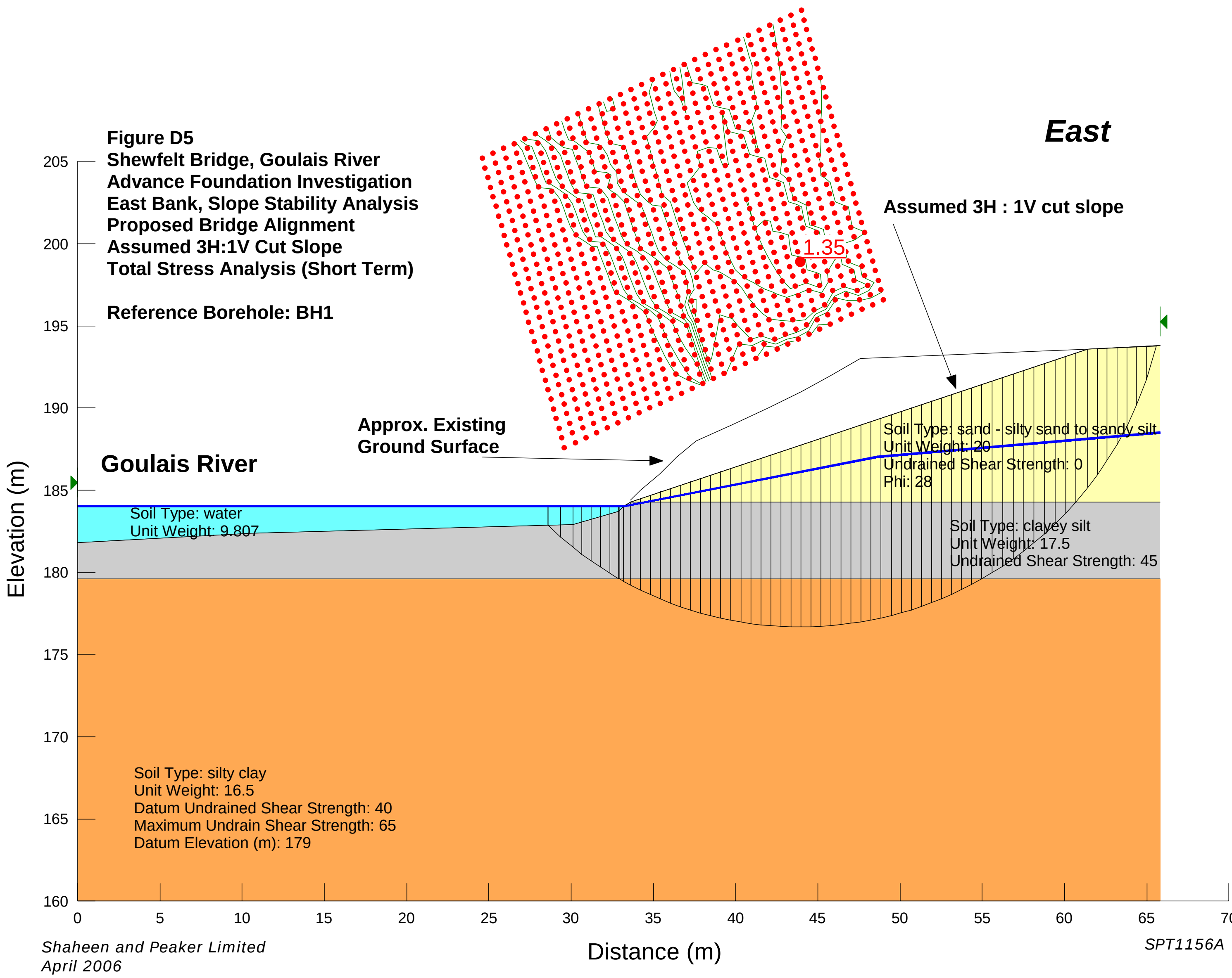
Figure D5
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Proposed Bridge Alignment
Assumed 3H:1V Cut Slope
Total Stress Analysis (Short Term)
Reference Borehole: BH1
East
Assumed 3H : 1V cut slope
1.35
Approx. Existing
Ground Surface
Goulais River
Soil Type: water
Unit Weight: 9.807
Soil Type: sand - silty sand to sandy silt
Unit Weight: 20
Undrained Shear Strength: 0
Phi: 28
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrain Shear Strength: 65
Datum Elevation (m): 179
Elevation (m)
Distance (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Shaheen and Peaker Limited
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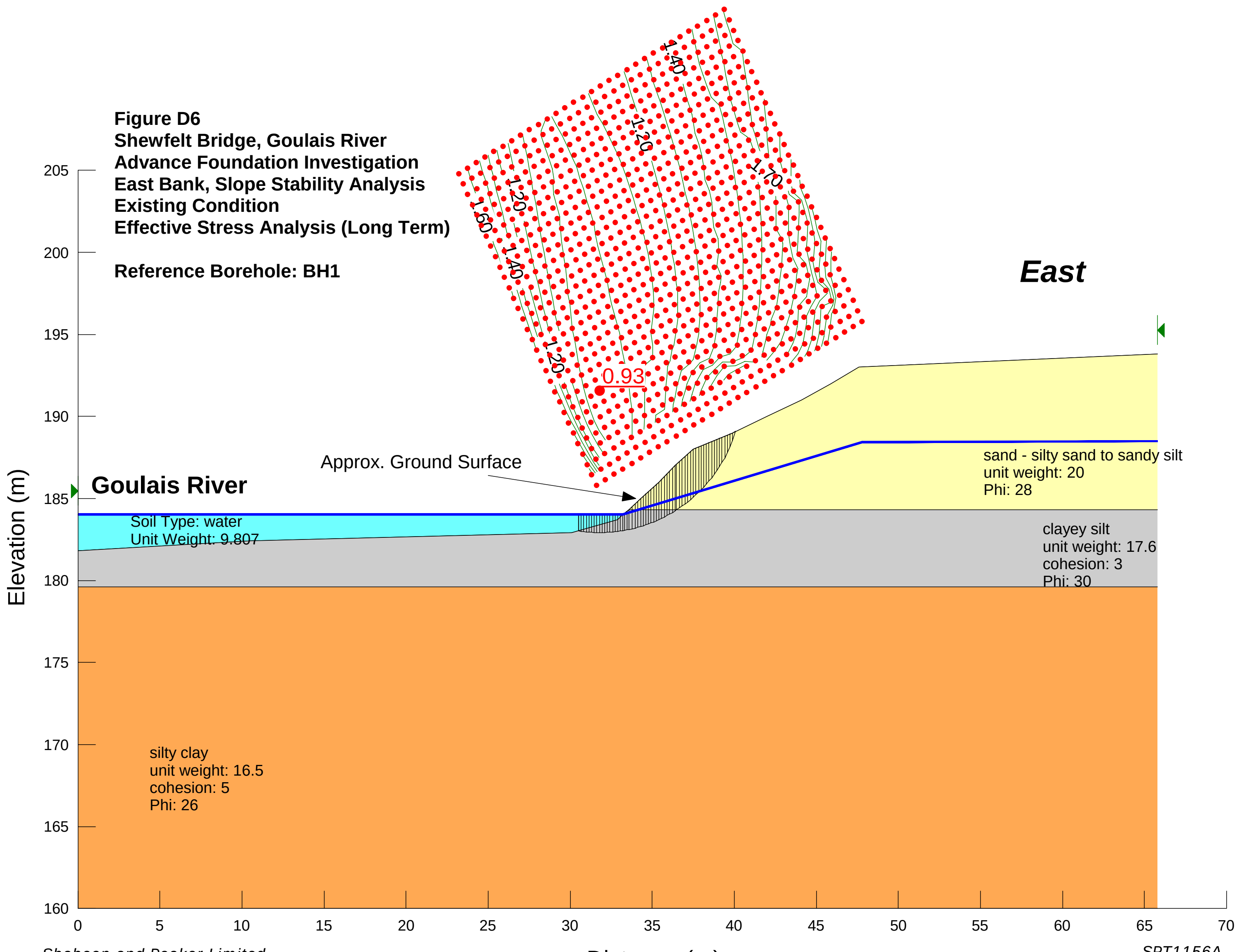

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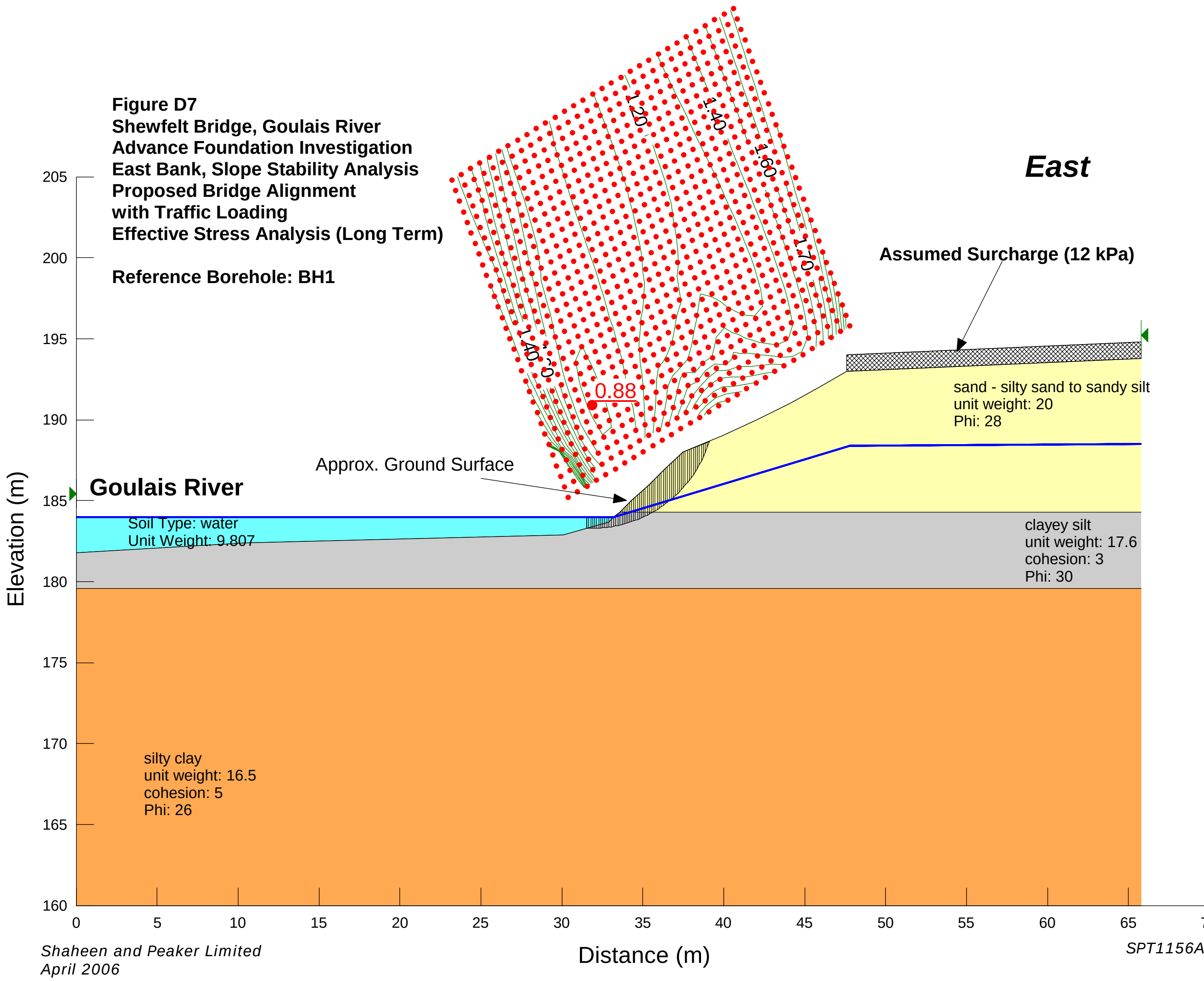

Figure D7
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Traffic Loading
Effective Stress Analysis (Long Term)
Reference Borehole: BH1
East
Assumed Surcharge (12 kPa)
1.20
1.40
1.60
1.70
1.40
0.88
sand - silty sand to sandy silt
unit weight: 20
Phi: 28
Approx. Ground Surface
Goulais River
Soil Type: water
Unit Weight: 9.807
clayey silt
unit weight: 17.6
cohesion: 3
Phi: 30
silty clay
unit weight: 16.5
cohesion: 5
Phi: 26
Elevation (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Distance (m)
Shaheen and Peaker Limited
April 2006
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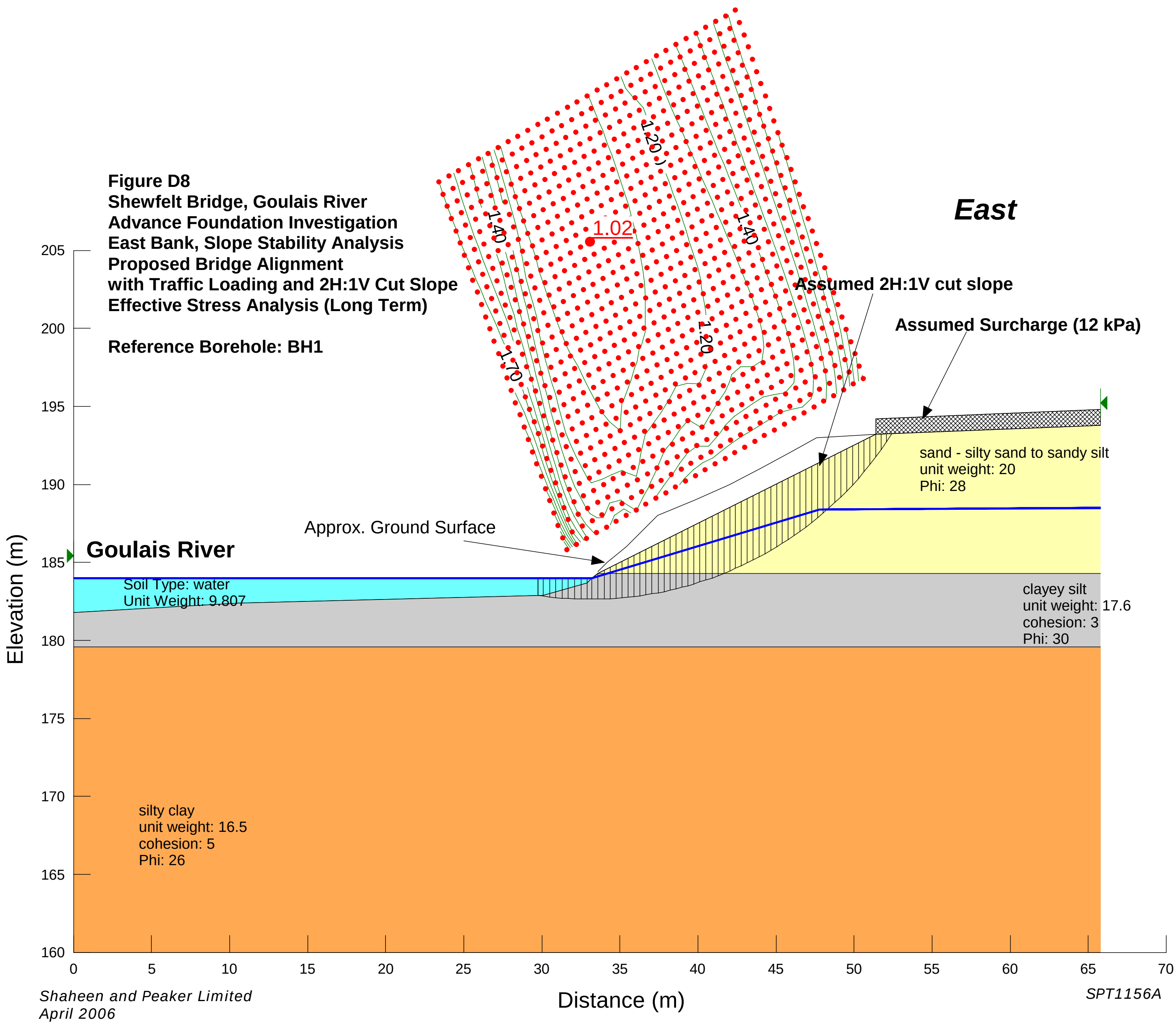

Figure D8
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
East Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Traffic Loading and 2H:1V Cut Slope
Effective Stress Analysis (Long Term)
Reference Borehole: BH1
East
1.02
1.20
1.40
1.70
Assumed 2H:1V cut slope
Assumed Surcharge (12 kPa)
Approx. Ground Surface
Goulais River
Soil Type: water
Unit Weight: 9.807
sand - silty sand to sandy silt
unit weight: 20
Phi: 28
clayey silt
unit weight: 17.6
cohesion: 3
Phi: 30
silty clay
unit weight: 16.5
cohesion: 5
Phi: 26
Elevation (m)
Distance (m)
160
165
170
175
180
185
190
195
200
205
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Shaheen and Peaker Limited
April 2006
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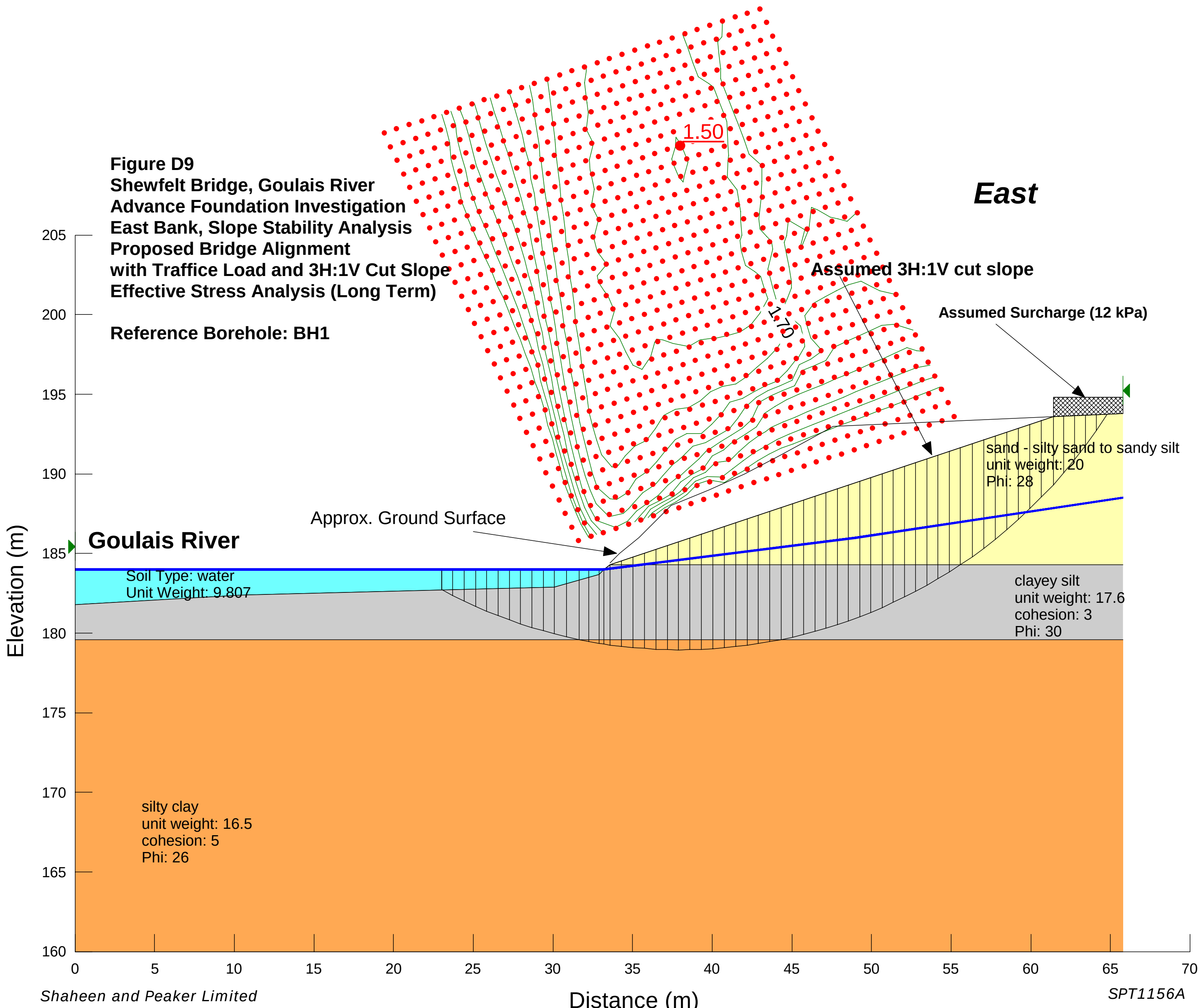

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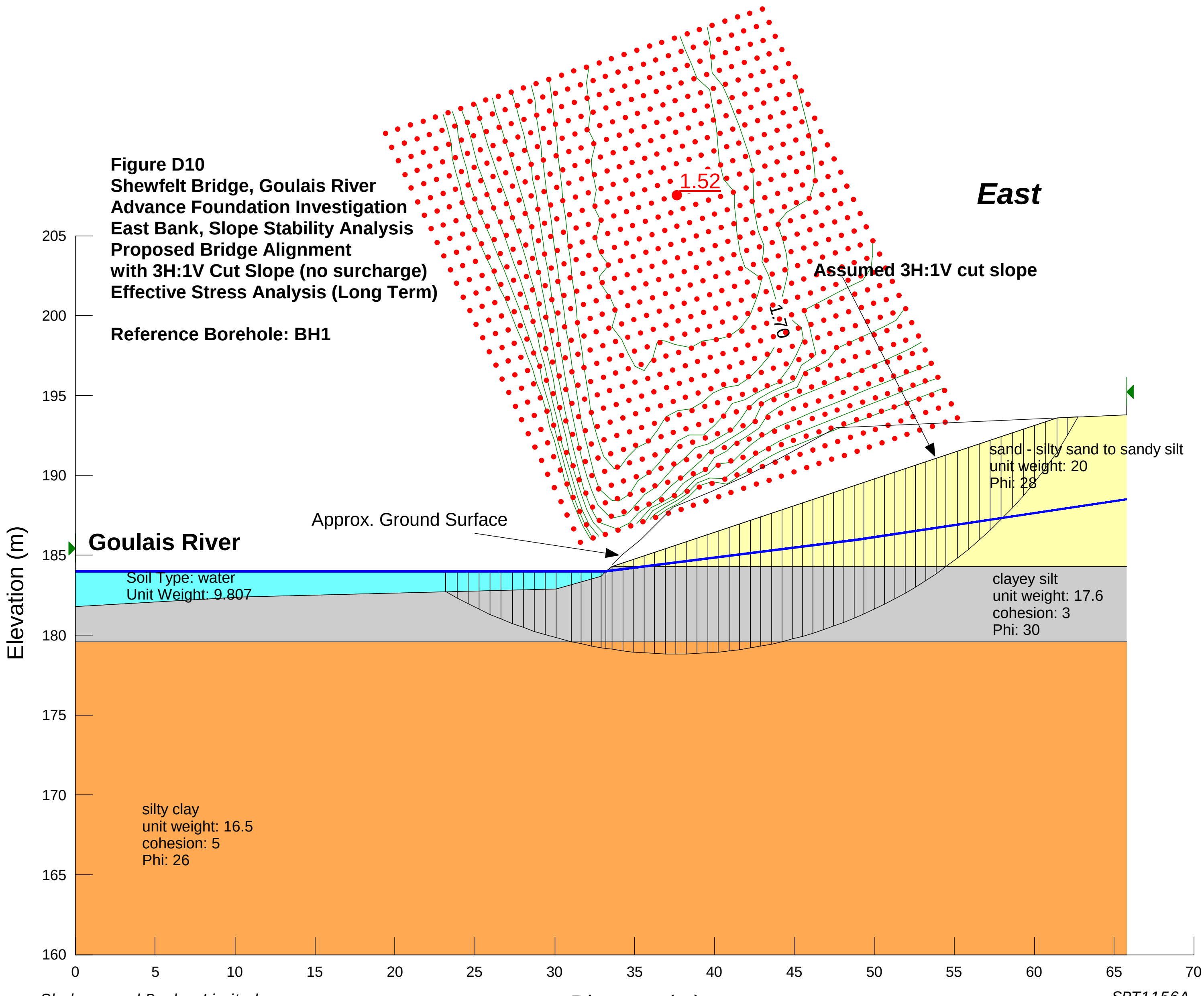

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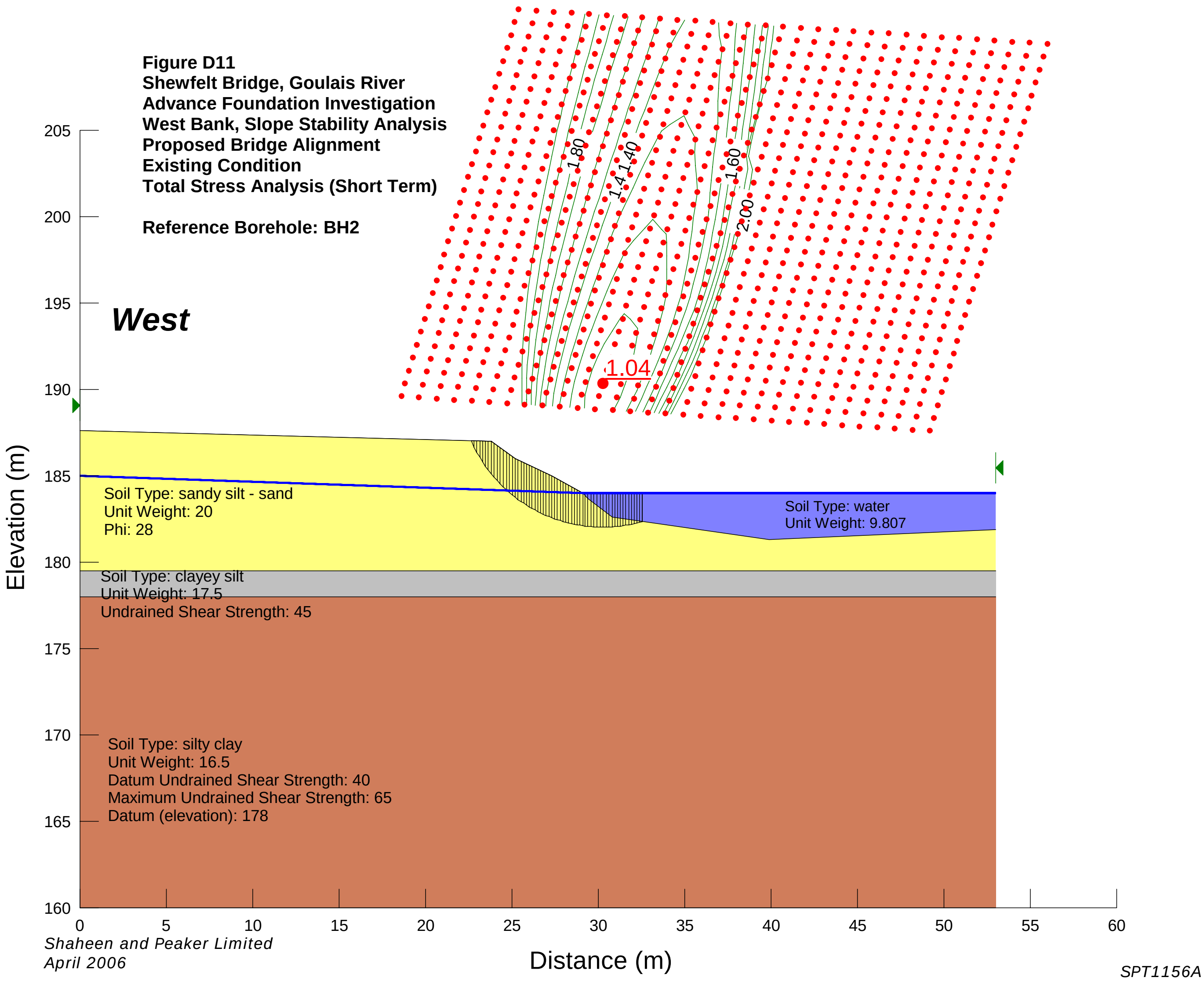


Figure D11
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
Existing Condition
Total Stress Analysis (Short Term)
Reference Borehole: BH2
West
1.80
1.40
1.60
2.00
1.04
Soil Type: sandy silt - sand
Unit Weight: 20
Phi: 28
Soil Type: water
Unit Weight: 9.807
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrained Shear Strength: 65
Datum (elevation): 178
Elevation (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
Distance (m)
Shaheen and Peaker Limited
April 2006
SPT1156A

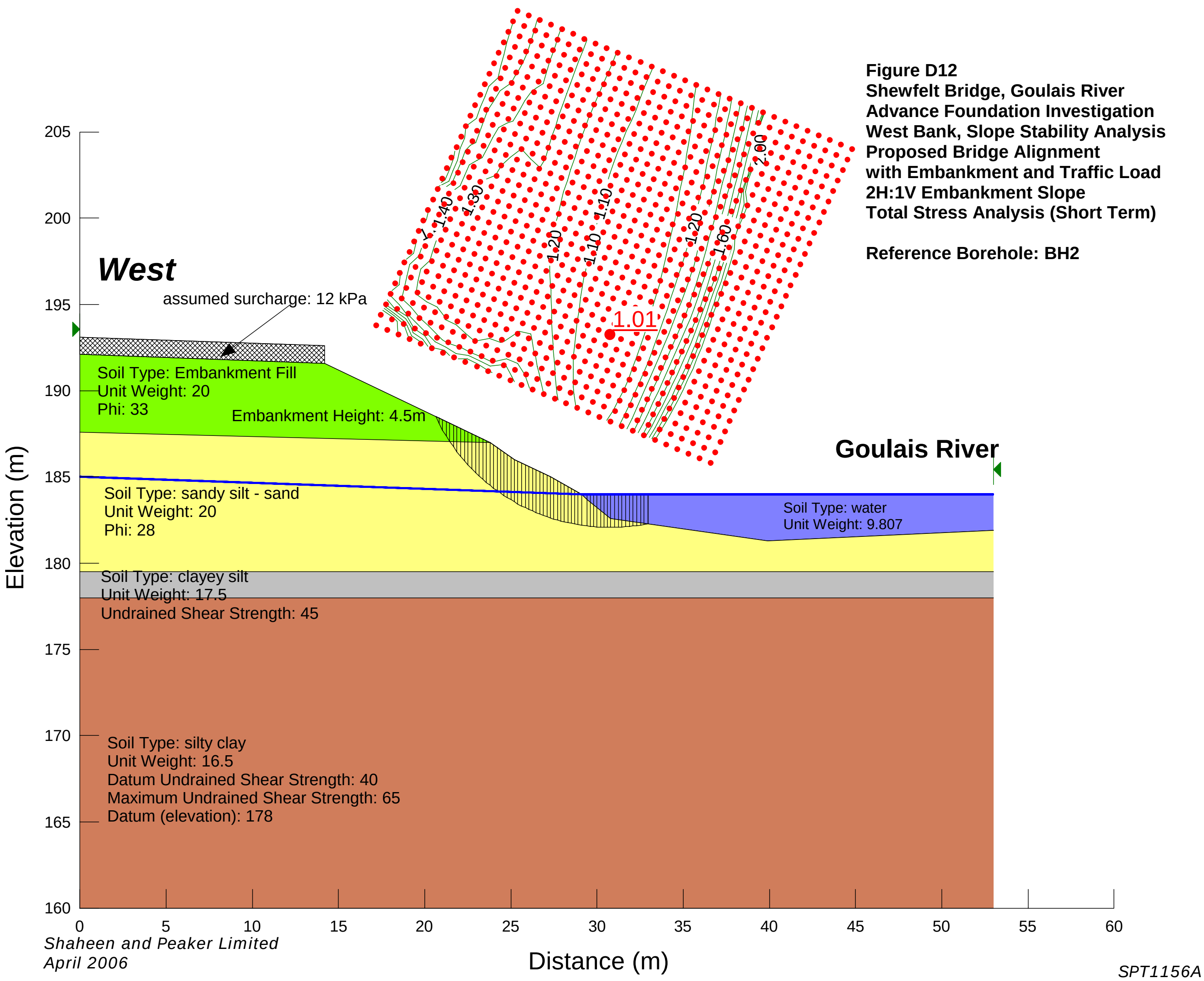


Figure D12
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
2H:1V Embankment Slope
Total Stress Analysis (Short Term)
Reference Borehole: BH2
West
Goulais River
assumed surcharge: 12 kPa
Soil Type: Embankment Fill
Unit Weight: 20
Phi: 33
Embankment Height: 4.5m
Soil Type: sandy silt - sand
Unit Weight: 20
Phi: 28
Soil Type: water
Unit Weight: 9.807
Soil Type: clayey silt
Unit Weight: 17.5
Undrained Shear Strength: 45
Soil Type: silty clay
Unit Weight: 16.5
Datum Undrained Shear Strength: 40
Maximum Undrained Shear Strength: 65
Datum (elevation): 178
1.01
1.40
1.30
1.20
1.10
1.10
1.20
1.60
2.00
Elevation (m)
Distance (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
Shaheen and Peaker Limited
April 2006
SPT1156A

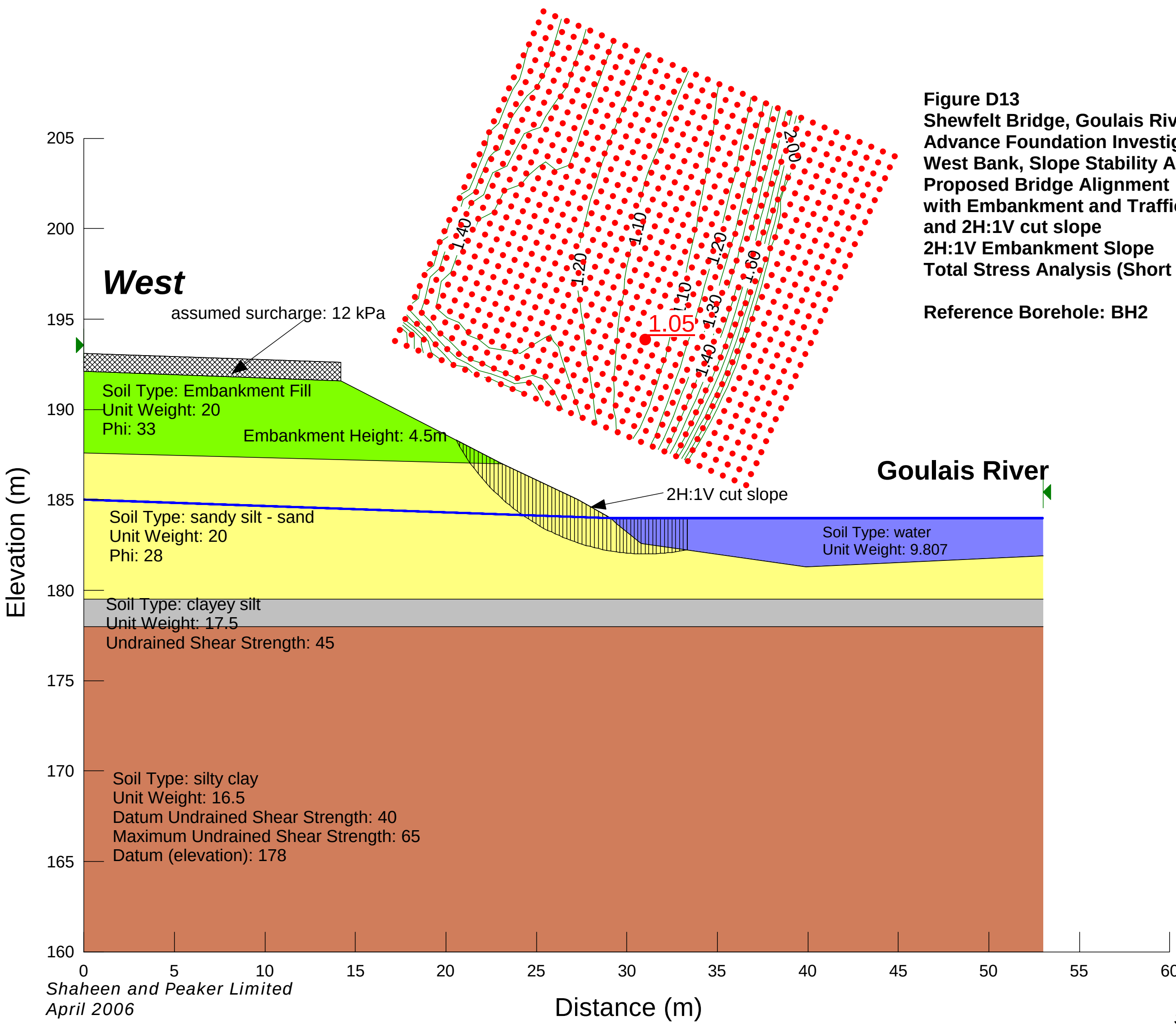


Figure D13
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
and 2H:1V cut slope
2H:1V Embankment Slope
Total Stress Analysis (Short Term)

Reference Borehole: BH2

Shaheen and Peaker Limited
April 2006

SPT1156A

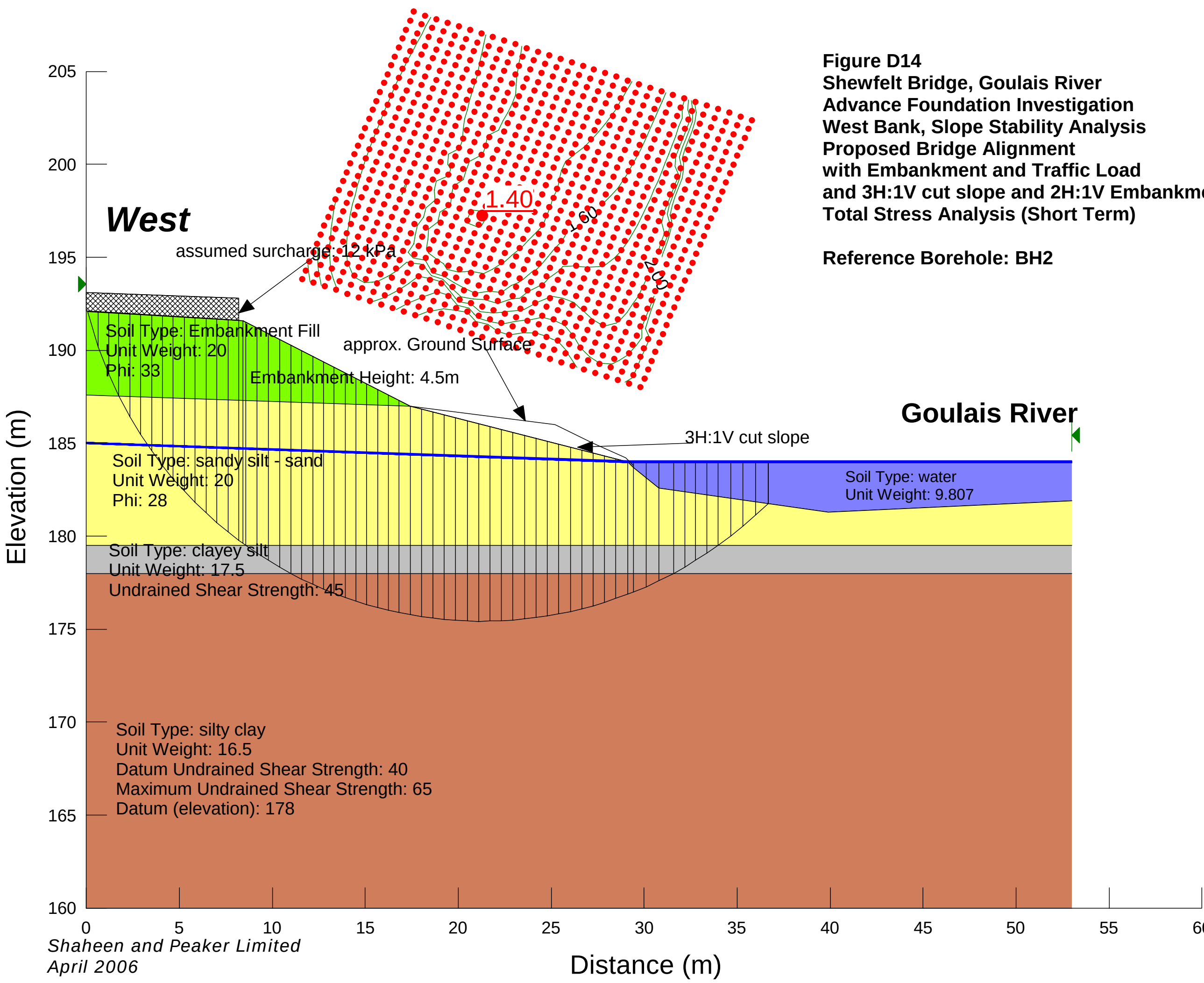


Figure D14
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
and 3H:1V cut slope and 2H:1V Embankment Slope
Total Stress Analysis (Short Term)

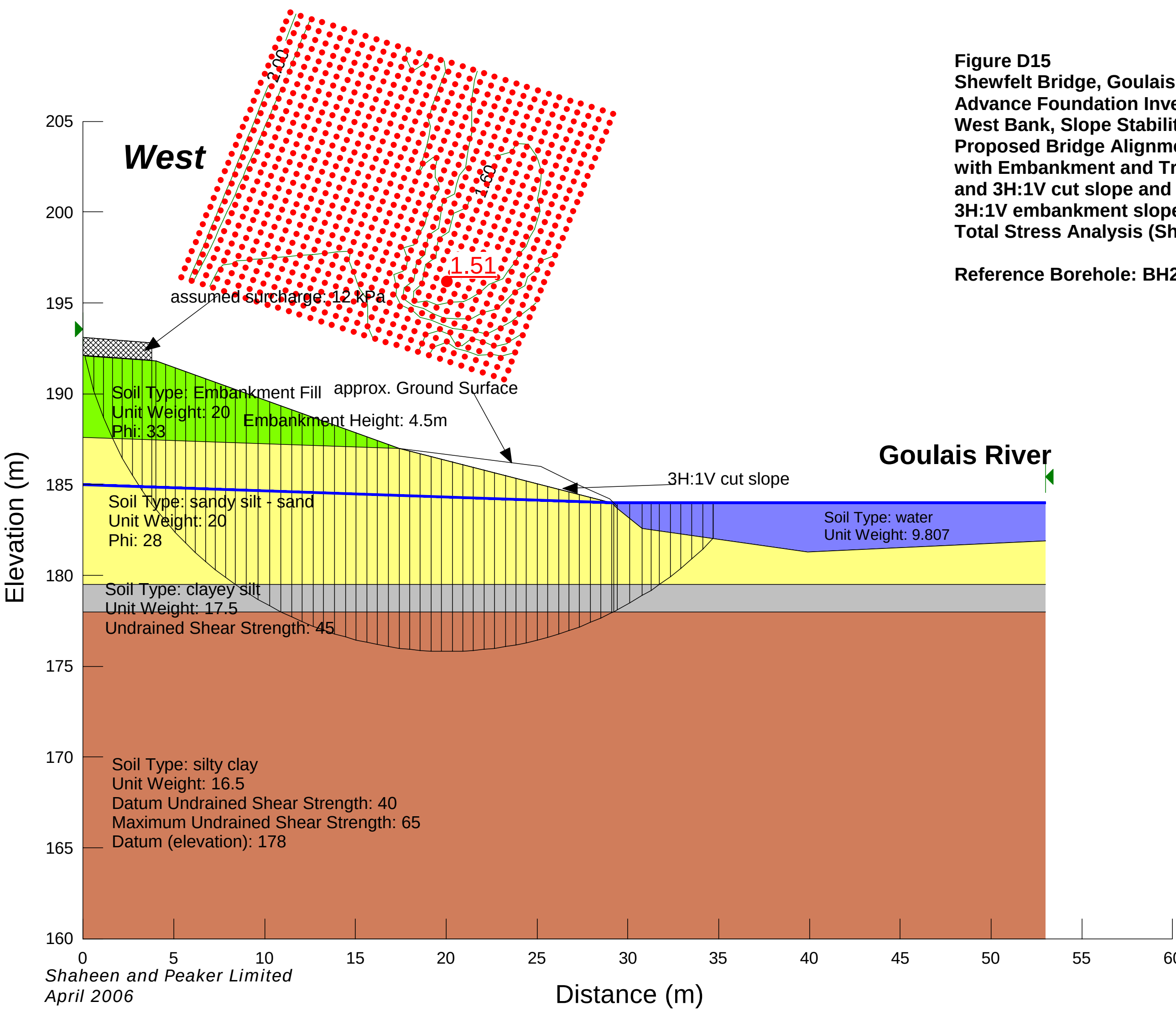
Reference Borehole: BH2

Shaheen and Peaker Limited
April 2006

SPT1156A

Figure D15
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
and 3H:1V cut slope and
3H:1V embankment slope
Total Stress Analysis (Short Term)

Reference Borehole: BH2



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April 2006

SPT1156A

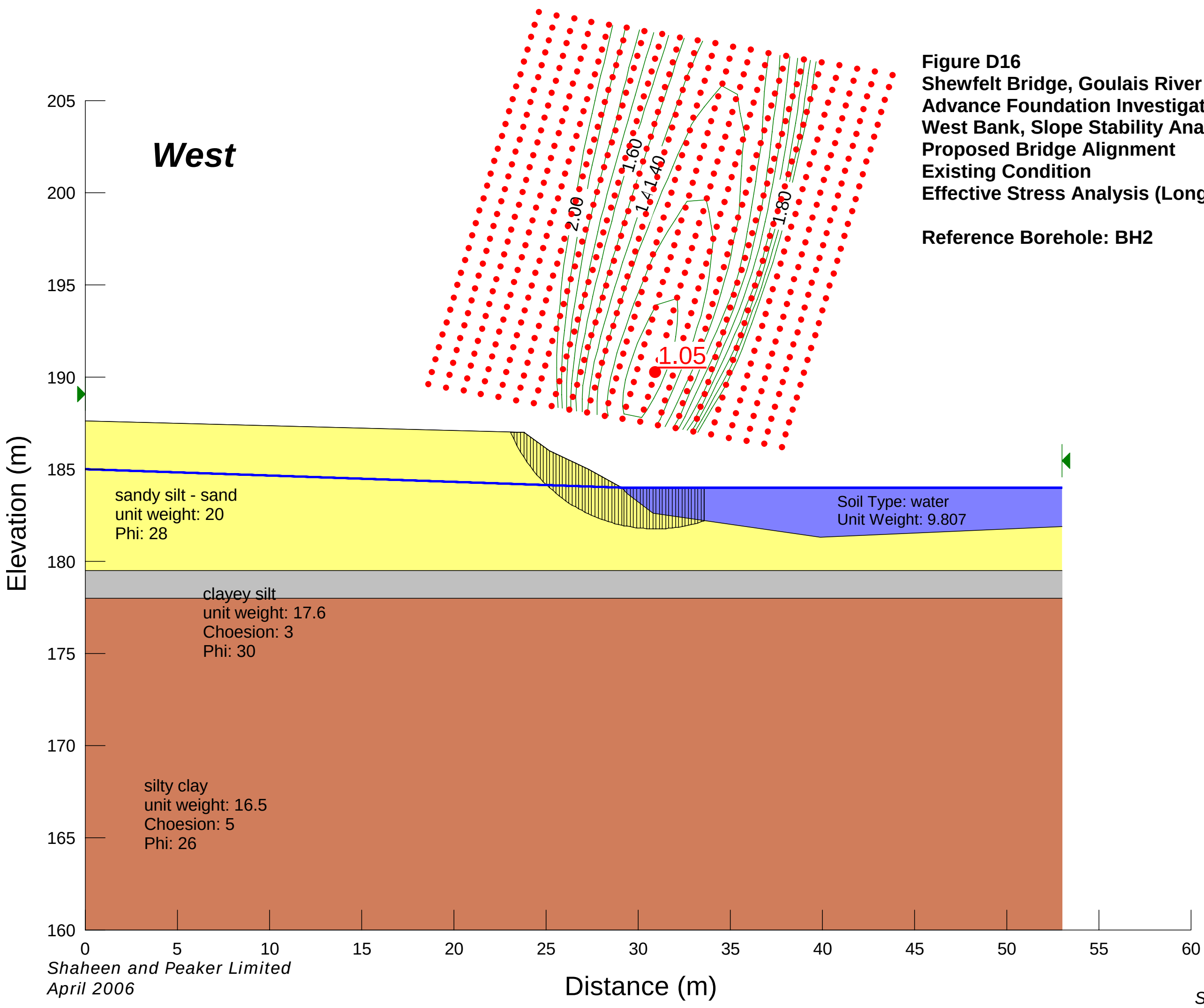


Figure D16
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
Existing Condition
Effective Stress Analysis (Long Term)

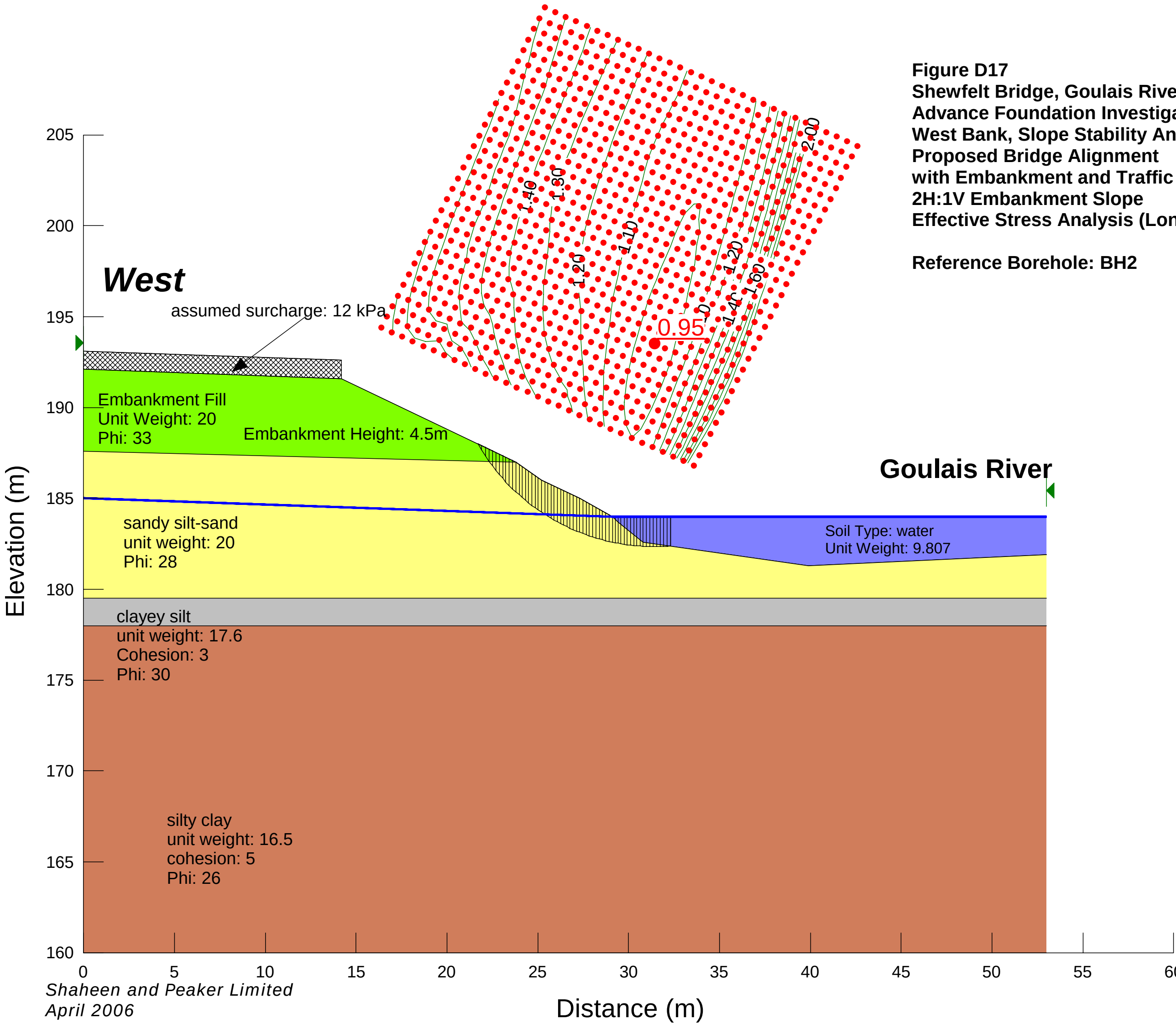
Reference Borehole: BH2

Shaheen and Peaker Limited
April 2006

SPT1156A

Figure D17
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
2H:1V Embankment Slope
Effective Stress Analysis (Long Term)

Reference Borehole: BH2



Shaheen and Peaker Limited
April 2006

SPT1156A

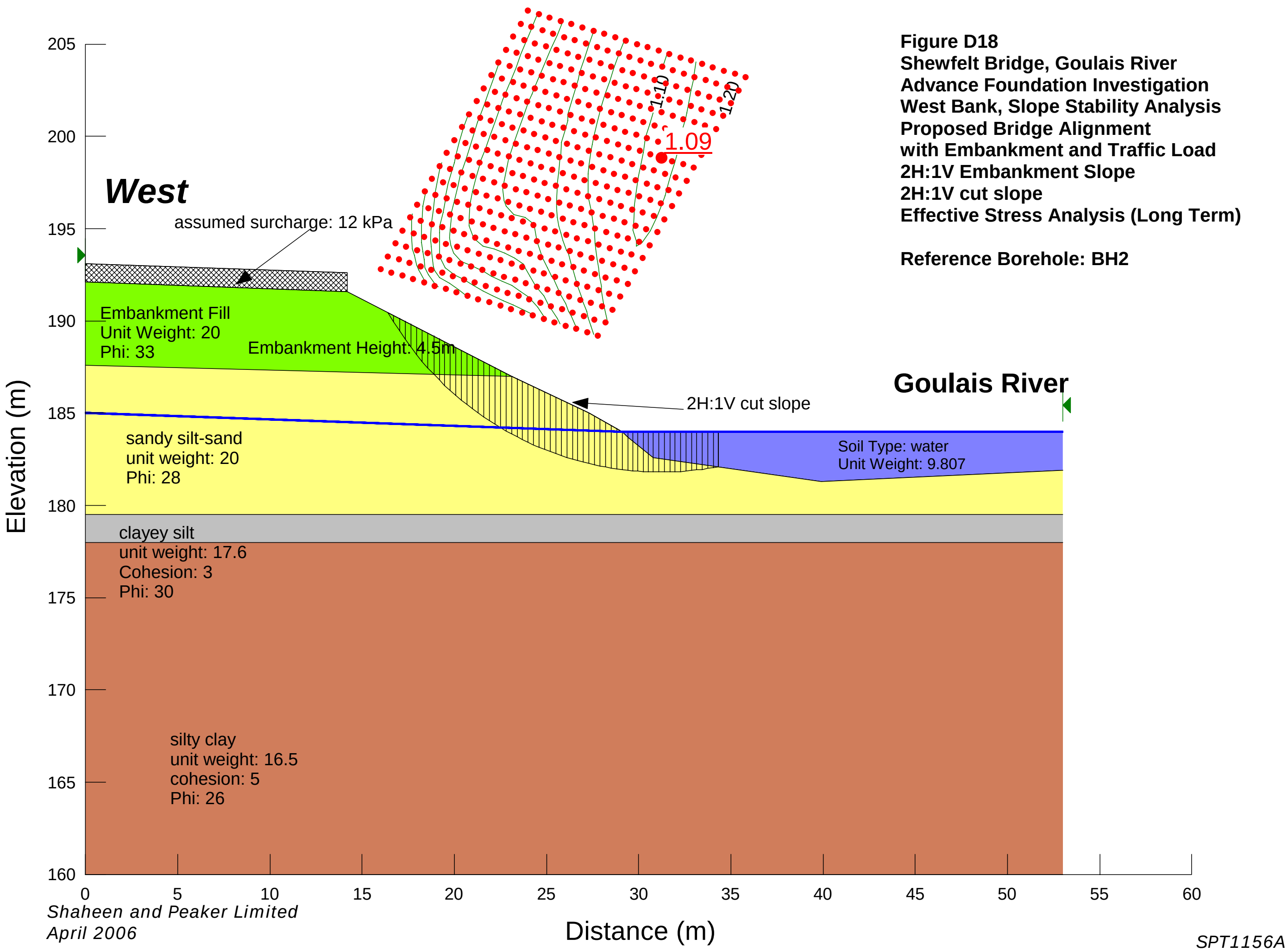


Figure D18
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
2H:1V Embankment Slope
2H:1V cut slope
Effective Stress Analysis (Long Term)
Reference Borehole: BH2
West
assumed surcharge: 12 kPa
1.09
1.10
1.20
Embankment Fill
Unit Weight: 20
Phi: 33
Embankment Height: 4.5m
2H:1V cut slope
Goulais River
sandy silt-sand
unit weight: 20
Phi: 28
Soil Type: water
Unit Weight: 9.807
clayey silt
unit weight: 17.6
Cohesion: 3
Phi: 30
silty clay
unit weight: 16.5
cohesion: 5
Phi: 26
Elevation (m)
205
200
195
190
185
180
175
170
165
160
0
5
10
15
20
25
30
35
40
45
50
55
60
Distance (m)
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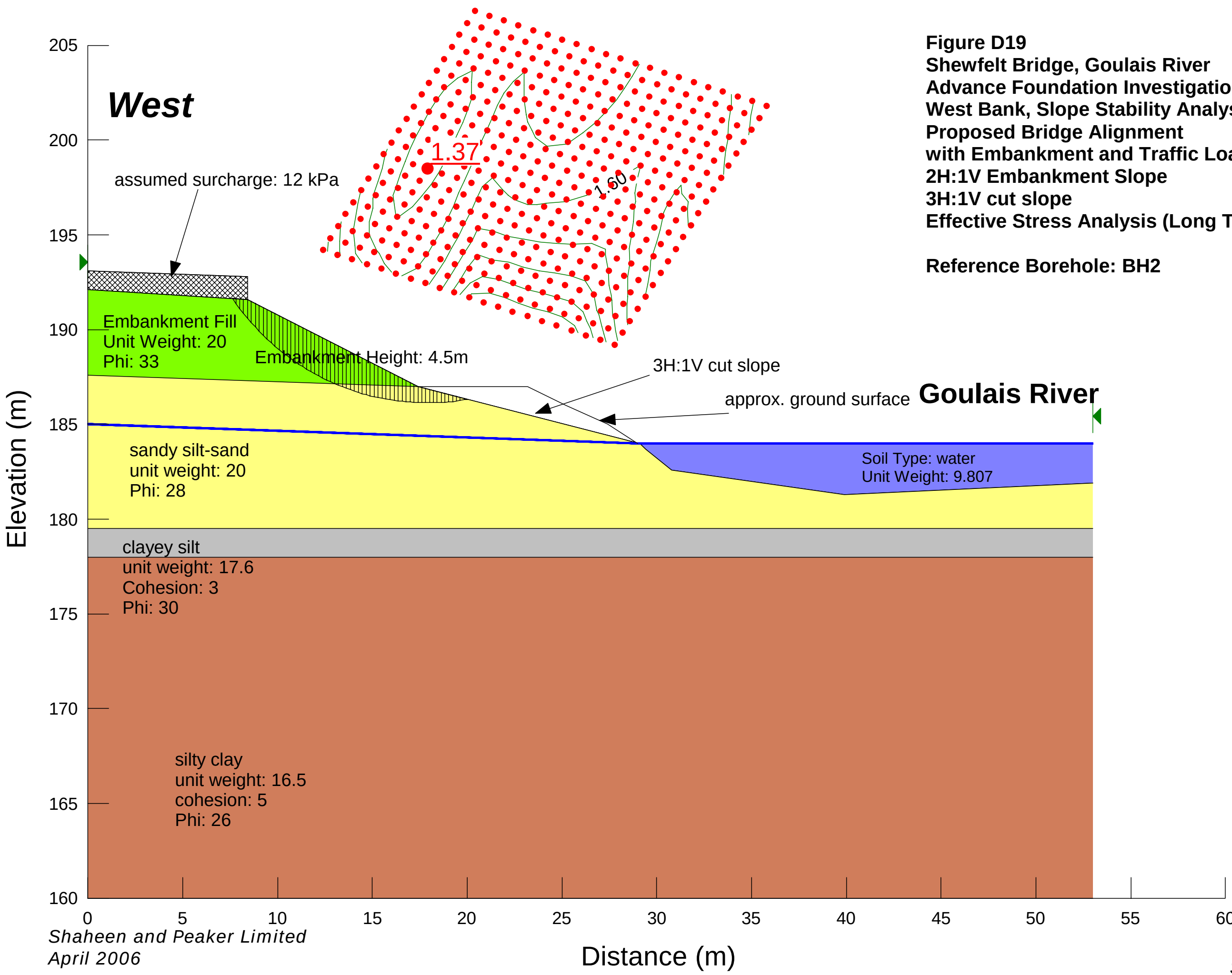
Figure D19
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
2H:1V Embankment Slope
3H:1V cut slope
Effective Stress Analysis (Long Term)

Reference Borehole: BH2



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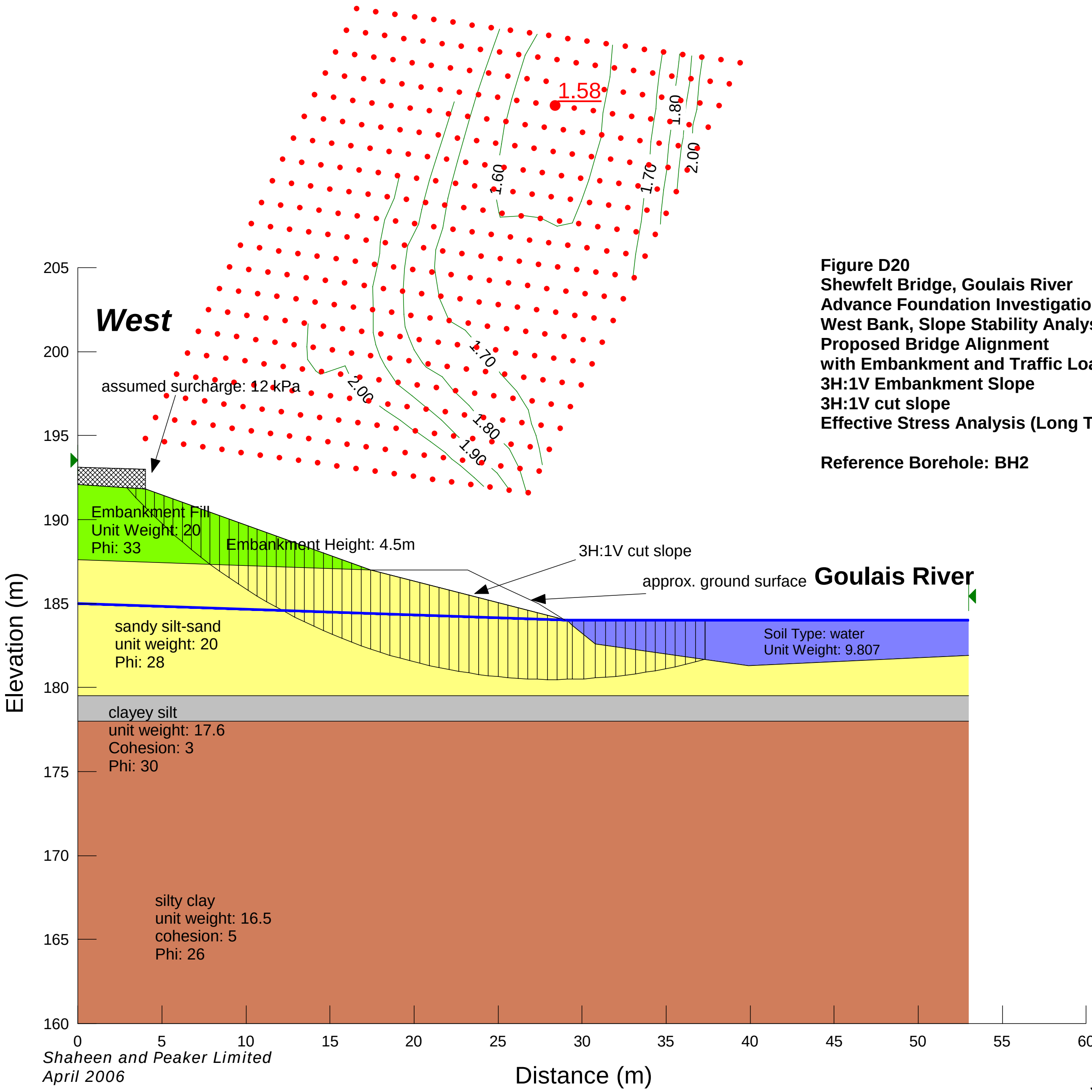

Figure D20
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
West Bank, Slope Stability Analysis
Proposed Bridge Alignment
with Embankment and Traffic Load
3H:1V Embankment Slope
3H:1V cut slope
Effective Stress Analysis (Long Term)

Reference Borehole: BH2

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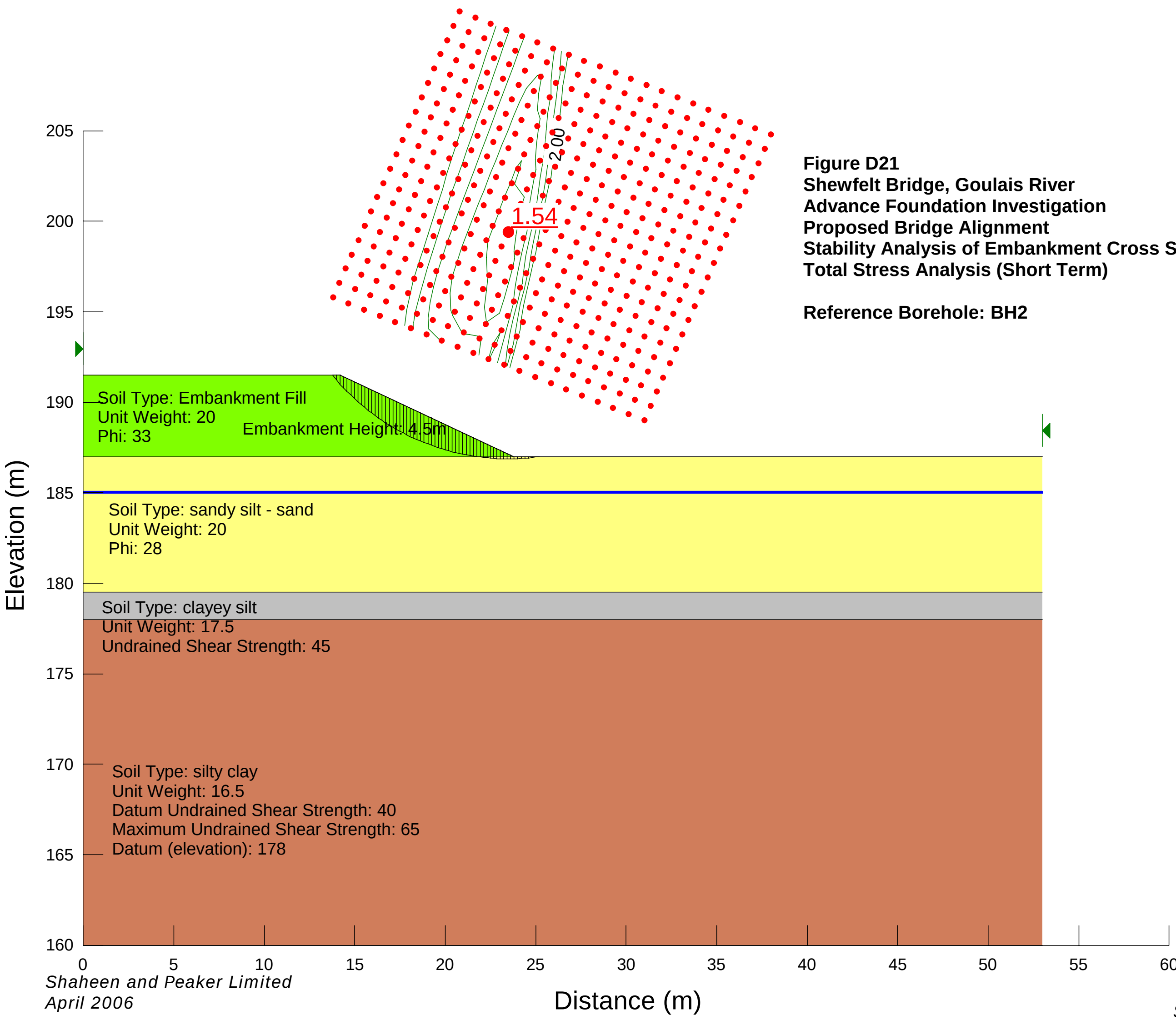

Figure D21
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
Proposed Bridge Alignment
Stability Analysis of Embankment Cross Section
Total Stress Analysis (Short Term)

Reference Borehole: BH2

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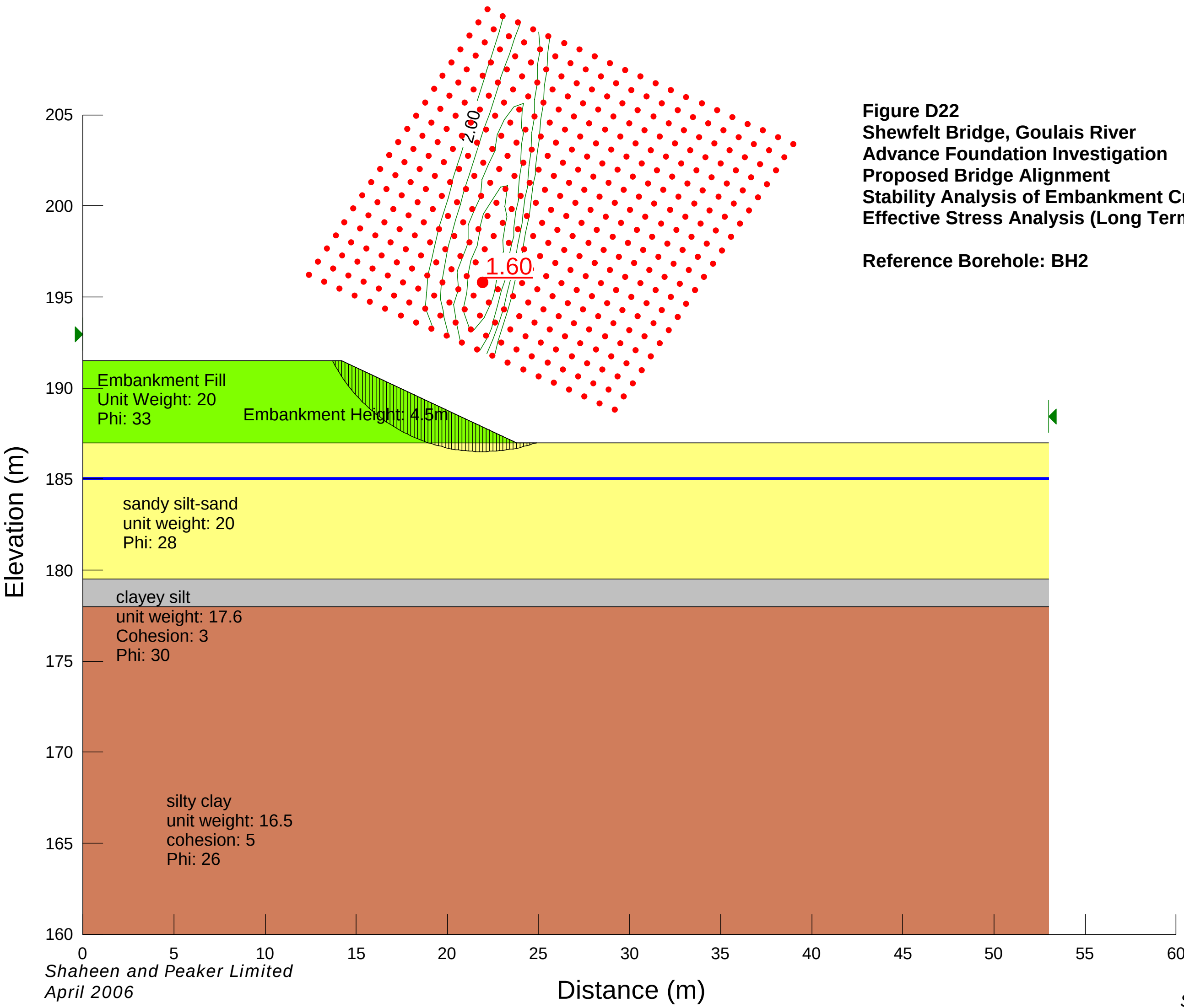
Figure D22
Shewfelt Bridge, Goulais River
Advance Foundation Investigation
Proposed Bridge Alignment
Stability Analysis of Embankment Cross Section
Effective Stress Analysis (Long Term)

Reference Borehole: BH2



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Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
i_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
i_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
i	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L – W_L)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$	i	1	HYDAULIC GRADIENT
i_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

Appendix F

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.