

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED REPLACEMENT CULVERT
MCKELLAR CREEK
TOWNSHIP OF WALSH, ONTARIO**

**January 3, 2003
TG02019**

**Prepared For:
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1.0 INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by Philips Engineering Ltd. to conduct a foundation investigation for the proposed replacement culvert at McKellar Creek (48E-46C) in the Township of Walsh. This Foundation Investigation and Design Report summarizes the factual information and provides discussions and recommendations for design in accordance with the Canadian Highway Bridge Design Code (CHBDC).

Authorization to proceed with the work was received from Philips Engineering Ltd. This work was carried as part of their Total Project Management Project for the Ministry of Transportation of Ontario (MTO) under GWP 194-87-00.

At this site, the Ministry has conducted three previous investigations: August '90 and April '91 for an alternative alignment and January '96 for a replacement culvert located in the existing rockfill embankment about 3 metres west of the existing culvert. Borehole logs from the 1996 report have been included as Appendix "D".

The project has been reassessed with this report addressing a replacement culvert spanning the old one in place. The intent is to carry out the construction without disturbing or disrupting the existing creek and its fish habitat. In addition to the culvert replacement the roadway grade will be raised 3 to 4 metres with no change permitted to the location of the toes of the existing embankment slopes. Lightweight fill is to be utilized for the grade raise to minimize the effects of the grade raise. A permanent retaining wall system to keep the grade raise within the existing boundaries is required.

The existing culvert is an approximately 5 m wide timber box culvert, which is in poor condition.

It is proposed to surround the existing culvert with an 8.4 m wide Super.Cor box culvert.

2.0 SITE DESCRIPTION

The site is located at McKellar Creek along Hwy. 17 approximately 34.5 km east of Terrace Bay in the Township of Walsh, District of Thunder Bay. The existing McKellar Creek culvert is a twin cell (4.2 x 1.83 m) timber structure about 29 m in length constructed in 1950. The creek is located in a valley with bedrock outcrops at its crests. The existing rockfill embankment is as much as 7.9 m in height at the culvert location. The slopes of the embankment are typically at 1.25 horizontal (H):1 vertical (V) to 1.5 H:1 V with occasional sections at 1 H:1 V and 2 H:1 V.

The valley floor is underlain by soft organic and clay deposits. The existing timber box culvert is failing as the ceiling and floor struts are buckling under their loads.

Pavement performance records indicate settlements both recently and in the past, mainly in the westbound lane over the culvert. A crescent shaped crack was evident during the field review, and extended from 14+675 to 14+700 from the Rt shoulder into the middle of the eastbound lane.

3.0 INVESTIGATION PROCEDURES

The field investigation for this project was carried out between April 22 and May 8, 2002 at which time thirteen boreholes were advanced. The investigation was carried out utilizing two drill rigs operated by DST, one drill rig operated by Northland Well Drilling, one Cat 225 excavator operated by Valentino Trucking and one HS40C Superhoe operated by Belham Ltd.

Boreholes 2, 4, 5, and 11 were drilled from the existing highway elevation, with the remainder of the boreholes located at the toe of the embankment left and right of centreline. Boreholes 1 to 4 are located in the vicinity of the culvert. For the boreholes drilled through the embankment, (Boreholes 2, 4, 5, and 11) a 200 mm steel casing was first installed through the roadbed and to the base of the rockfill using an Odex drill system. The borehole was then advanced with a CME 750 rig equipped for geotechnical investigations by washboring techniques with "B" size casing. The boreholes at the toe of the embankment were advanced with a CME 45 or CME 750 drill rig using 83 mm inside diameter hollow stem augers. At some locations the hollow stem augers became plugged with sand when the sand could not be prevented from rising up the auger stems to permit sampling. Such boreholes were continued by wash boring techniques with "B" size casing. A dynamic cone penetration test was conducted at the base of Borehole 9, and a bi-cone was used at the base of Boreholes 3 and 9. The boreholes were drilled to depths ranging between 3.0 and 38.6 m below existing ground surface at the time of the field program. Refer to the Record of Borehole drawings for specific depths and comments concerning each borehole.

Samples of the overburden were obtained at 0.75 to 1.5 m intervals of depth using 51 mm outside diameter split-spoon samplers in accordance with the standard penetration test (SPT)

procedure. Relatively undisturbed "Shelby" tube samples were also recovered at selected depths. Field vane shear tests were performed in cohesive soils utilizing the MTO vane to estimate the undrained shear strength of cohesive soil. Slotted PVC standpipes were installed in the boreholes for monitoring of the groundwater level.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, supervised the drilling, sampling and in-situ testing, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis. Laboratory testing on selected samples included: natural moisture content and soil description, plastic and liquid limits, grain size analyses, consolidation tests and direct shear tests (drained and undrained).

The borehole locations were established in the field in relation to the Station numbering system. On completion of the field program the borehole locations and elevations were tied into the survey of the site by Poole Survey Ltd.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The results of the investigation confirm the general overburden stratigraphy noted in the previous reports prepared by MTO. The generalized stratigraphy below the highway embankment at the culvert location based on the conditions at Borehole Locations 1 to 4 consist of asphalt/fill overlying a clay deposit which in turn is underlain with a sand, sand and gravel and/or silt. Beneath this deposit exists bedrock. The following table indicates the top elevation of the different strata beneath the highway at the culvert location.

Stratum	Elevation of top of stratum (m)			
BOREHOLE No.	1	2	3	4
Fill	192.9	197.5	192.0	197.0
Clay	191.4	189.6	189.0	191.6
Sand	182.4	182.3	182.0	183.8
Bedrock	*170.6	171.5	*170.1	163.4

*Elevation at auger or bicone refusal, bedrock not proven with diamond drilling techniques.

The stratigraphy at the remaining boreholes is similar to that above with the strata becoming thinner towards the rock outcroppings to the east and west. Along the toe of the embankment up to 1.4 m. of organics, fill, and/or sand is present above the clay stratum.

Auger refusal occurred in Boreholes 1, 6, 7, 8, 12, and 13. Refusal to "B" casing occurred in Borehole 11. The refusal material was not confirmed by diamond drilling techniques and could be bedrock or boulders.

At Borehole 5 a steel casing was installed through the embankment fill. On reaching a depth of 0.8 m bedrock was encountered. The borehole was terminated with the Odex drilling method at a depth of 3.0 m. below the existing grade.

The plan and location of boreholes and stratigraphical profiles are shown on Drawing No. 1 in the attached appendix. The field and laboratory test results are plotted on the Record of Borehole sheets and in the appendix of this report. A brief description of the different soil types is given below.

4.2 Fill

No soil sampling was conducted in the fill in the boreholes placed through the embankment. The make-up of the embankment is based on visual observation of the drilling. The embankment materials consist of roadbed granulars up to depths of 1.2 m overlying rockfill to depths of 7.9 m below highway grade. In Borehole 11 wood with a cresol odour was encountered at the bottom of the rockfill.

While drilling through the rockfill, very little to no spoil was expelled from the drilling operations. This could imply the rockfill is in a loose condition with voids throughout.

At the toe of the embankment fill was encountered to depths between 0.3 and 3.0 m. and consists of an unsorted mixture of sand, gravel, silt, clay, cobbles and boulders with some organics mixed in.

4.3 Organics

An organic layer is present at surface in the Boreholes 6, 9, and 12 varying in thickness from 0.3 to 0.9 m.

4.4 Clay

Clay exists beneath the above noted layers and extends to a minimum elevation of 182.0 m. Results of grain size distribution tests carried out on select samples are shown on Enclosure 19. The tests indicate 0% gravel, 1% sand, 35 –36% silt, and 63 - 64% clay.

Upon examination of clay samples extracted from thin walled tube samplers, the clay was found to be generally varved with layers of varying degree of plasticity. In addition, occasional to numerous seams of sands and silts were identified, and vary in thickness from approximately 1 to 10 mm.

Within the upper metre of this layer at some locations the clay is desiccated and has field vane strengths greater than 60 kPa. Below this desiccated layer the clay consistency is generally soft to firm with in situ field vane shear strengths varying between 20 and 50 kPa. The field vane tests have been corrected for plasticity (according to Bjerrum, 1972) to estimate the undrained shear strength. In addition, two consolidated undrained direct shear tests were carried out (Borehole 1 at 5.3 m and Borehole 4 at 9.1 m). These tests were consolidated to the in-situ effective overburden pressure to estimate the in-situ undrained shear strength. The estimated undrained shear strength from corrected field vanes tests and undrained direct shear tests have been illustrated graphically on plots of undrained shear strength versus elevation to determine an undrained shear strength profile for use in slope stability analyses. The first undrained shear


Kim, Tae-C (MTO)

From: Gombola, Richard (MTO)
Sent: January 20, 2003 9:23 AM
To: Kim, Tae-C (MTO)
Cc: Krisciunas, Ray (MTO)
Subject: Pile tip question

Tae

Earth-Tech is preparing their design report for us (Cont 526-00-00 Atikokan River Trib Culvert, Hwy 11B) They have a question that Ray K. thought you could respond too. If you need more detailed info about the site, let me know.

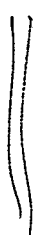
Pile Tips

- TBTE has indicated very very hard granite at approx. 10 m and sloping rock at 45 degrees
- recent experience with very hard rock in Eastern region, they were very hesitant to use:
 - Oslo tips because of penetration problems
 -  auger into rock to provide a "platform" because of slow rate of advancement (0.5 m per day) and cost
- I'm sure the NW structural section has much experience with this condition, would like confirmation on preference / experience

Thanks



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strength plot (Enclosure 30) identifies the design undrained shear strength profile for stability analyses carried out for the culvert replacement works (between Stations 14+680 and 14+750, Boreholes 1 to 4). No clear differentiation was identified for undrained shear strengths below and outside of the embankment and the lower bound limit was established as the design profile. The second and third undrained shear strength plots identify design undrained shear strength profiles for Stations 14+600 to 14+680 and 14+750 to 14+900 (outside of the culvert area).

Plastic and liquid limits conducted on representative samples indicate the clay is medium to high plastic with liquid limits between 39 and 57% and plastic limits between 15 and 20% (Enclosure 18). The natural water content varies between 37 and 51%. Plots of natural moisture content versus elevation for all soil strata have been illustrated on Enclosures 32 to 34.

The clay at this site can be generally defined as being medium sensitive to sensitive with measured sensitivities of 3 to 6 (Enclosures 25 to 27). At Borehole 9 only, a sensitivity of 10 was recorded at elevation 190.5 m, which is defined as extra sensitive.

Two consolidation tests have been carried out on samples taken below the upper desiccated zone of the clay to assess the consolidation characteristics. These are attached as Enclosures 22 and 23. One consolidation test was carried out on a sample from below the existing embankment (Borehole 4 at 9.1 m, Elevation 187.9 m). The second consolidation test was carried out on a sample located beyond the toe of the existing embankment (Borehole 1 at 5.3 m, Elevation 187.6 m).

The consolidation test carried out on the sample from Borehole 4 (below the embankment) indicates a normally consolidated state (overconsolidation ratio = 1) with the current effective overburden pressure including the weight of the embankment (P'_o) approximately equal to the preconsolidation pressure (P'_c) of about 120 kPa. The compression index (C_c) was measured at 0.30 with an initial void ratio (e_o) of 0.97. The coefficient of consolidation was measured between 0.4 and 0.6 m²/yr within the range of expected construction stresses.

The consolidation test carried out on a sample from Borehole 1 (beyond the toe of the existing embankment), also indicates a normally consolidated state (overconsolidation ratio = 1) with the effective overburden pressure (P'_o) approximately equal to the preconsolidation pressure (P'_c) of about 75 kPa. The compression index (C_c) was measured at 0.45 with an initial void ratio (e_o) of 1.34. The coefficient of consolidation was measured between 0.3 and 0.5 m²/yr within the range of expected construction stresses.

Consolidated drained direct shear testing carried out on a sample from Borehole 4 at 9.1 m (Enclosure 24) indicates that the clay has a drained angle of internal friction (ϕ') of 25.5° with a cohesion intercept (c') of 0 kPa.

4.5 Sand

Underlying the above layer is a non-cohesive sand which has a thickness of 0.2 to 11.8 m. Results of a grain size distribution test carried out on a sample from Borehole 3 at 10.7 m are shown on Enclosure 19. The sample comprises 0% gravel, 91% sand, and 9% silt. Results of grain size distribution tests carried out by the Ministry on this stratum from previous investigations indicated that this deposit comprises 0 – 3% gravel, 78% sand, 14 – 19% silt and

3 – 5% clay. A cobble and/or boulder layer was noted at the base of this layer in Boreholes 3, 6, 7, 9, 10, and 11.

A single point drained direct shear test was carried out on a normally consolidated sand sample from Borehole 3 at 10.7 m to estimate the angle of internal friction (ϕ') at 32°.

Standard penetration tests had 'N' values ranging from 2 blows/0.3m to greater than 99 blows/0.3m, indicating a very loose to very dense state. Due to the presence of a high water level samples taken within this layer were susceptible to blow-up conditions and could be disturbed. In these cases the blow counts may not be totally representative. The blow counts greater than 50 blows/0.3 m occur near the base of the stratum.

The exception to the above occurred at Borehole 4 where the non-cohesive deposit extends from Elevation 183.8 to Elevation 163.4 metres (20.4 m in thickness). From Elevation 183.8 to Elevation 176.0 metres silty sand exists which is in a compact condition. A sandy silt then extends from the base of the sand to Elevation 171.8 metres. The silt is in a loose to compact condition. Below the silt, a layer of loose to compact silty sand is present to Elevation 166.2 metres. Underlying the above to Elevation 163.4 metres a dense to very dense sand and gravel was encountered.

4.6 Bedrock

The refusal material was proven by diamond drilling techniques using a 'B' size core barrel at Boreholes 2 and 4. The bedrock was cored 5.0 metres at Borehole 4. At Borehole 2 the

bedrock was cored 4.2 metres. Bedrock surface level was also inferred by observing the Odex drilling at Borehole 5, and the close proximity of bedrock outcrops.

In general, the bedrock consists of syenite and volcanic bedrock. A detailed description of the bedrock core is presented in Appendix 'B'. At the culvert location the bedrock was found to be at Elevations 171.5 and 163.4 metres. Bedrock elevations vary greatly within the valley. Bedrock outcrops exist at each end of the project limits.

In order to classify the intact bedrock with respect to strength, point load tests were conducted on selected core samples. The test results are tabulated below and indicate the strength varies from strong to very strong.

Borehole Number	Elevation (m.)	Strength Index I _s MPa	* Estimated Uniaxial Compressive Strength MPa
1	171.3	8.4	178
	170.4	4.1	86
	169.7	5.7	120
	168.9	6.5	137
	168.6	10.6	225
	167.7	9.1	194
4	162.8	7.8	166
	162.2	7.8	166
	161.8	7.0	148
	160.4	4.1	88
	159.2	8.5	180
	158.9	6.4	134

* Estimate based on published correlations.

The rock quality designation (RQD) is an indirect measure of the number of fractures and the amount of jointing in the rock mass. The RQD is expressed as a percentage of the ratio of

summed core lengths (greater than 100 mm) to the total length cored. The RQD's record for the rock cores are indicated on the Record of Borehole for No. 2 and 4. The RQD varies between 33% and 70%. The RQD index is used to provide a classification for the rock quality according to the following limits.

RQD%	ROCK QUALITY
0 – 25	Very Poor
25 – 50	Poor
50 - 75	Fair
75 - 90	Good
90 -100	Excellent

Using the above limits, the bedrock quality ranges from poor to fair.

4.7 Groundwater

Groundwater levels obtained at the time of the investigation are indicated of the following table.

For design of the culvert replacement the water table was taken at elevation 191.0 m.

**TABLE 1
GROUNDWATER MEASUREMENTS**

BOREHOLE NUMBER	SURFACE ELEVATIONS (m)	WATER ELEVATION (m)
1	192.9	192.0
2	197.5	191.6
3	192.0	191.2
4	197.0	191.5
5	206.2	< 203.2
6	199.7	198.2
7	198.0	191.9
8	198.5	192.3
9	192.6	191.3
10	192.2	191.3
11	200.4	198.1
12	201.5	< 196.5
13	207.5	206.9

It should be noted that groundwater levels are subject to fluctuations with respect to season and precipitation events.

5.0 DISCUSSIONS

DST Consulting Engineers Inc. (DST) has been retained by Philips Engineering Ltd. to conduct a foundation investigation for the proposed replacement culvert at McKellar Creek (48E-46C) in the Township of Walsh. The proposed project consists of the construction of a replacement culvert to surround or replace the existing timber box culvert. The new concrete culvert will be supported on a pile foundation. The concrete culvert will circumvent the existing timber culvert with the walls of the existing culvert possibly left in place. In addition to the culvert replacement, the highway grade will be raised 3 to 4 metres with no change in the position to the existing toe of the slope. Lightweight fill is to be utilized for the grade raise to minimize the effects of the grade raise, and a permanent retaining system will be required to keep the embankment within the existing toe boundaries.

In general, the site conditions are considered poor in relation to the proposed construction. The stability of the existing embankment is such that increased loads and/or over-steepened excavations can induce embankment slope failures. The proposed culvert installation will be challenging with strict adherence to design limits being mandatory. Deviation from the design limits provided in this report can have a significant negative impact on stability. Careful construction planning including the implementation of a monitoring and instrumentation plan, will be mandatory to ensure stability is maintained at a satisfactory level. As such, construction delays are a possibility should contingency plans need to be implemented. In addition, given the relatively steep embankment slopes, staging will be challenging given that it will be restricted by geometric constraints of the existing embankment cross section. Facilitating the proposed 3 to 4 m grade raise while restricting the toe to its current location will require the use of a lightweight fill product. PlastiSpan insulation board has been proposed.

5.1 Embankment Stability

Slope stability analyses were carried out using SLOPE/W software (Version 4.0, by Geo-Slope International Ltd.). The Morgenstern-Price method of analysis was used with a half sine function for distribution of inter-slice shear forces, which satisfies both moment and force equilibrium. The Bishop Simplified method was also used, and found to give similar results (FS within 0.1). For each geometry assessed, the software computes numerous possible slip surfaces with the associated factor of safety to identify the slip surface with the lowest calculated factor of safety. Through this process, the geometry of various sections and loading conditions were altered so as to produce a minimum factor of safety of 1.3, which is normally used for the design of embankment slopes under static conditions. In addition, the Janbu method of analysis (force equilibrium) was used to assess the stability for lateral block slides.

Stability analyses were carried out utilizing both undrained soil strength parameters and drained soil strength parameters, the latter incorporating existing in-situ porewater pressures. The effects of negative porewater pressures from the excavation were conservatively neglected in our analyses. This is due to the unpredictability of estimating the initial negative porewater pressure response and time effects as well as the contractor's schedule.

Embankment loading conditions such as traffic loads and temporary bridge foundation loads have been considered for the geotechnical design of this project. A seismic hazard calculation carried out for this site was provided by the Geological Survey of Canada which indicates ground accelerations are negligible for return periods of up to 475 years. Traffic loads on the embankment have been modelled with a 12 kPa loading placed uniformly over traffic surfaces. Temporary bridge foundation loads were modelled under ultimate limit states and serviceability

limit states with loads provided by Philips Engineering Ltd. Where applicable, the temporary bridge foundation loads were adjusted to model three-dimensional effects. This was accomplished by assuming the width of the slip surface (perpendicular to direction of slope movement) affected by foundation loads would extend beyond the limits of the footing by a distance equal to the depth of any rockfill remaining below the underside of footing. As two-dimensional modelling software was utilized to assess stability, the pressure along the base of footing was adjusted to represent an average uniform pressure over the width of the assumed slip surface. This concept has been illustrated on Enclosure 35.

The design soil properties were derived through in-situ field testing, laboratory testing and correlations where applicable. Design soil properties for both drained and undrained analyses are listed in the following table.

Soil	Unit Weight (kN/m ³)	Undrained Strength Parameters	Drained Strength Parameters
Granular A and B	20	n/a	c' = 0 kPa, ϕ' = 35°
PlastiSpan	0.5	Cu = 10 kPa	n/a
Rockfill	18	n/a	c' = 0 kPa, ϕ' = 45°
Clay – Station 14+600 to 14+680	17	Cu = 16 kPa, el. 199.0 to 195.0 m Cu increases linearly from 16 to 22 kPa from el. 195.0 to 190.3 m Cu increases linearly from 22 to 39 kPa from el. 190.5 to 186.0 m	c' = 0 kPa, ϕ' = 25.5°
Clay – Station 14+680 to 14+750	17	Cu = 24 kPa, el. 191.0 to 187.5 m Cu = 32 kPa, el. 187.5 to 186.0 m Cu increases linearly from 32 to 45 kPa from el. 186.0 to 182.0 m	c' = 0 kPa, ϕ' = 25.5°
Clay – Station 14+750 to 14+900	17	Cu = 20 kPa, el. 199.5 to 189.5 m Cu increases linearly from 20 to 50 kPa from el. 189.5 to 184.0 m	c' = 0 kPa, ϕ' = 25.5°
Sand and Silty Sand	20	n/a	c' = 0 kPa, ϕ' = 32°

A conservatively low shear strength was chosen for the PlastiSpan insulation board so as not to inadvertently model tension through the lightweight fill.

Slope stability analyses were carried out at Station 14+720 to assess the construction staging requirements for the proposed culvert replacement., and at Stations 14+680, 14+720 and 14+750 to design the proposed grade raise for the embankment. At Station 14+720, the embankment is at its highest and the clay foundation soils are thickest. Stations 14+680 and 14+750 were also deemed to be critical design sections even though the overall embankment height was generally lower and the clay foundation soils are thinner, since the undrained shear strength of the clay was found to be lower than at Station 14+720. Stability analyses were carried out through consultation with Philips Engineering Ltd. for the following stages (illustrated on Enclosures 36 to 40).

1. Back analyses of existing geometry and conditions (Station 14+680, 14+720 and 14+750).
2. Stage 1 and 2 construction; excavations for lowering grade on south lane to facilitate temporary bridge on north lane including temporary bridge foundation loads (Station 14+720).
3. Stage 3 construction; excavation for culvert construction on south lane with temporary bridge in use on north lane (including temporary bridge loads and pile driving effects, Station 14+720). In addition to slope stability, the bearing capacity for the temporary bridge foundations was considered (most critical) at this stage.
4. Stage 4 construction; backfilling south lane utilizing lightweight fill to facilitate traffic flow with excavation for culvert replacement on the north lane (including effects of pile driving, Station 14+720).

5. Stage 5 construction; backfilling with rockfill to for both lanes (Station 14+720).
6. Stage 6 construction; raising grade with lightweight fill and pavement structure (Stations 14+680, 14+720 and 14+750).

The geometry, temporary bridge length and foundation location were established through several iterations of stability analyses and consultations with Philips Engineering Ltd. The final results of our stability assessment for each stage are discussed below in the following sections.

5.1.1 Back Analyses

Back analyses of the existing geometry and soil conditions were carried out to validate the design soil properties and to assess the current stability of the embankment. When assessed for slip surfaces that extend into the embankment foundation soils under fully drained, long-term conditions with no excess porewater pressures, minimum factors of safety of 1.38, 1.17, and 1.49 are calculated for Stations 14+680, 14+720 and 14+750 respectively. This indicates that under existing conditions the embankment has sections with factors of safety less than the design factor of safety of 1.3. This may be indicative of the observed crescent shaped crack observed at station 14+675 to 14+700 and could be consistent with the observed historical movements. When these three sections were assessed for foundation slip surfaces with undrained strength parameters, factors of safety of 1.27, 1.17, and 1.28 were computed. This indicates that construction activities that induce excess porewater pressures can reduce the stability of the embankment to unsafe limits and that extreme care must be considered in the design and construction of the proposed works.

Given that the surveyed sections of the rockfill embankment indicate slopes as steep as 1H:1V, the effective angle of internal friction (ϕ') for the rockfill is at least 45°.

5.1.2 Stage 1 and 2 Construction; Excavations for Placement of Temporary Bridge

For Stage 1 and 2 construction, traffic will be initially restricted to the north lane while the south lane is lowered to facilitate placement of the north lane temporary bridge. Two options have been assessed. The first option considers shallow foundations for the bridge while the second option considers a piled bridge foundation. These options have been illustrated on Enclosure

36. The foundation loads on the shallow foundation option have been included in our analyses. A minimum factor of safety of 1.3 is provided for both options.

5.1.3 Stage 3 Construction: Excavation for Culvert on South Lane – Temporary Bridge on North Lane

This stage of construction proved to be the most critical in terms of excavation and placement of the temporary bridge. Two options have been considered and include a temporary bridge supported on either shallow or deep foundations.

5.1.3.1 Option 1, Temporary Bridge with Shallow Foundations:

Based on preliminary assessment and through consultation with Philips Engineering Ltd., it was determined that for a shallow foundation option, a temporary bridge (in the order of 40 m in length) and a maximum excavation depth for placement of the new culvert to el. 189.5 m would be optimal. A shorter bridge would require a piled foundation to facilitate a stable excavation slope (by removing the load on the slope). For a given 39.6 m long bridge (maximum as specified by Philips Engineering Ltd.) with total bearing loads at each end of 2200 kN ULS, the critical design section exists when the excavation partially extents below the temporary bridge. A bin wall will be utilized to contain the soils at the abutments. Preliminary calculations indicate a 5 x 6 m (6 m across the bridge) footing is required extending about 2.6 m below the adjacent bin wall to offset eccentric loads. A uniformly distributed load was modelled in our slope stability analyses. Ideally, placing the footing as low and as far away from the culvert as possible improves the factor of safety for potential slip surfaces. However, positioning the footing closer to the soft clay foundation soils reduces the allowable bearing capacity. Therefore, the final position optimizes the above.

Through iterations of slope stability analyses and bearing capacity assessment for the 39.6 m long bridge, a final configuration which provides a minimum factor of safety against slope instability of 1.3 and provides sufficient bearing capacity was obtained. This involves setting the underside of footing elevation at elevation 192.6 m (about 1.5 m above the clays) and locating the footing partially below the adjacent abutment bin wall to eliminate eccentric loads. As part of this optimization, the following additional requirements must be implemented to achieve a minimum factor of safety of 1.3. These are illustrated on Enclosure 37.

- The grade on the culvert side of the footing must be excavated to elevation 192.6 m for a minimum distance of 5.5 m from the outside of the footing towards the culvert. From this point, the excavation can be cut at a slope of 1.5H:1V through the existing rockfill to elevation 191.0 m (generally upper limit for clay near the culvert). The existing grade to the north side of the temporary bridge foundation is not to be cut below elevation 192.6 m.
- The existing roadway will have to be excavated to elevation 196.3 m within 15 m of the temporary bridge abutments. If the road surface has to be maintained at the original elevation then the roadway can be restored utilizing lightweight fill provided that there is no net increase in loading.
- To permit excavation into the clay (below elevation 191.0 m), the clay must be initially cut at a slope of 1.5H:1V to elevation 189.5 followed by immediate placement of a 2.5 m wide bench consisting of free draining granular fill such as HL4 stone to elevation 191.0 m. The stone should be wrapped with a Class II non-woven geotextile with an FOS of 75 to 150 μm in accordance with OPSS 1860. Excavation below the native clay, or elevation 191.0 must be carried out in maximum 2 m wide sections followed by

placement of the granular bench leaving no more than a 2 m wide section open at any one time.

With the above construction there is a potential for slope instability due to the effects of driving the piles for the culvert foundations. With the presence of silt and sand seams confined within the clay stratum, excess porewater pressures generated during pile driving can be transmitted into the slope. The effects of this phenomenon cannot be accurately assessed due to uncertainties with predicting porewater pressure response and time effects for both unloading (excavation through the embankment) and pile driving. As a worst case scenario, the excavated slope was modelled with predicted excess porewater pressures based on the observed data as reported by J. H. A. Crooks, E. L. Matyes, and H. M. McKay (Excavation Slope Stability Related to Pore-Water Pressure Variation During Piling, Canadian Geotechnical Journal, Vol. 17, 1980). In addition, any negative porewater pressures associated with the excavation were conservatively neglected. The results of this analysis indicate that piles must be installed without generating significant pore-pressures. This can be successfully carried out with a monitoring program and using construction methods that minimize impacts.

To successfully install the piles for the proposed culvert foundation, the following procedure must be followed. Firstly, the piles are to be driven before excavating below elevation 191.0 m. As the initial porewater pressure response within the foundation soils will likely be negative for some period of time after completion of excavation and furthermore, experience shows that pore pressures from driving a line of piles dissipate very quickly, it is unlikely that excess pore pressures will be generated to the degree that will result in instability. Since pore pressures cannot be predicted accurately and therefore to confirm stability during driving operations (minimum factor of safety of 1.3), it is important to monitor and control porewater pressures

(from driving operations) within safe limits. This could involve delays in pile driving to allow for dissipation of porewater pressures to return to acceptable levels. The following monitoring plan must be implemented and followed to confirm stability during pile driving operations.

1. A nest is to be installed with two quick response piezometers at depths of 1.5 and 4.0 m below the base of excavation. Each nest should be located between the toe of excavation and piles (within 1 m of the pile location) at 6 m intervals along the proposed pile arrangement on both the east and west sides of the culvert. The piezometers, are to be installed with care and sealed with a minimum of 600 mm bentonite (top and bottom). The piezometers and data collection system should be calibrated and capable of readings accurate to 1 kPa.
2. Initial readings are to be taken to verify piezometers have stabilized prior to initiating pile driving to establish a base line for the existing in situ pressures.
3. During pile driving operations, all piezometers should be read at least twice daily with the adjacent piezometer nests (adjacent to pile being driven) read every 10 minutes during driving and every 30 minutes after driving until pressures have stabilized, or have started to dissipate.
4. The maximum safe piezometric head for the piezometers installed at the 1.5 m depth is 0.3 m above the base of excavation. The maximum safe piezometric head for the piezometers installed at

4.5 m below the base of excavation is 0.8 m above base of excavation.

5. Should these maximum levels be reached during driving operations, driving should be suspended until such time as the piezometric levels drop to safe limits.

An inherent risk associated with this approach would be costs associated with construction delays and the costs associated with any potential contingency measures required to expedite construction. Delays may be required to allow porewater pressures to dissipate during driving operations and/or implement alternative measures such as installation of pressure relief drains. To assess these effects, a pile / porewater pressure response testing program could be carried out on site prior to construction.

Prior to excavation below elevation 191.0 m, excess porewater pressures generated during pile driving operations must be allowed to fully dissipate.

As soil will remain against the existing timber culvert during pile driving operations, the integrity of the existing culvert must be maintained during construction. Should the timber culvert wall collapse, soils will be spill into the creek. In addition, if temporary excavations are required below the existing culvert, a 3 horizontal to 1 vertical slope extending horizontally from the base of the culvert should be maintained.

Sketches illustrating the proposed excavation limits and temporary bridge foundation on footings have been illustrated on Enclosures 36 to 37.

5.1.3.2 Option 2, Temporary Bridge with Deep Foundations:

Through consultation with Philips Engineering Ltd., a piled foundation (utilizing end bearing piles) for the temporary bridge was to be considered to facilitate a shorter temporary bridge and a deeper excavation for the construction of the proposed culvert. The temporary bridge length would be 36.6 m in length, and the base of excavation would be carried out to an elevation of 188.6 m. It is understood that the temporary bridge pile cap will be set at about elevation 194 m. The following requirements must be implemented to achieve a minimum factor of safety of 1.3. These are illustrated on Enclosure 37.

- PlastiSpan lightweight fill is required within and behind the proposed abutment bin walls.
- The slope towards the culvert from the pile cap must be cut from elevation 193.5 m at no steeper than 2H:1V through the existing rockfill to elevation 191.0 m (generally upper limit for clay near the culvert).
- The piles for the temporary bridge foundation will be installed prior to excavation below elevation 194 m.
- For excavation of the clay below elevation 191.0 m, the clay must be initially cut at a slope of 1.5H:1V to elevation 188.6 followed by immediate placement of a 3.7 m wide bench consisting of free draining granular fill such as HL4 stone to elevation 191.0 m. The stone should be wrapped with a Class II non-woven geotextile with an FOS of 75 to 150 μm in accordance with OPSS 1860. Excavation below the native clay, or elevation 191.0 must be carried out in maximum 2 m wide sections followed by placement of the granular bench leaving no more than a 2 m wide section open at any one time. ime.

Similarly, as with the shallow foundation option discussed in the previous section, a potential for slope instability exists during pile driving operations for the culvert foundation. As discussed, a monitoring approach can be applied to successfully install the proposed culvert foundation piles.

Sketches illustrating the proposed excavation limits and temporary bridge foundation on piles have been illustrated on Enclosures 36 and 37.

5.1.4 Stage 4 Construction; Backfilling South Lane to Facilitate Traffic Flow with Excavation and Culvert Replacement on North Lane

The critical section for slope instability at this stage of construction exists adjacent the newly installed culvert with a potential for slope movement into the excavation on the north lane. Two options have been assessed at this section for the new culvert pile cap located at either elevation 189.5 or 188.6 m.

The first option considered was to reconstruct the south lane to elevation 197.5 m and minimize lightweight fill quantities. Philips Engineering Inc. requested, a second option utilizing lightweight fill and bin wall facing, to bring the south lane to the final design elevation of 200.5 m. Conceptual designs for both options have been illustrated on Enclosure 38. PlastiSpan 24 insulation board (lightweight fill) was utilized to improve stability to design limits. Both of these options provide a minimum factor of safety of 1.3 against slope instability. The internal stability of the gabion or bin wall should be assessed by a structural engineer and/or product supplier. It is common practice to construct and place rockfill at a slope of no steeper than 1.25 horizontal to 1 vertical.

For the above options a gabion or bin wall has been proposed to retain fills around the end of the newly installed culvert near the existing road centreline. The centreline gabion and/or bin wall will be installed to the height of the new culvert. As a minimum, the lightweight PlastiSpan fill must be utilized over the limits of the centreline gabion or bin wall, or to where a rockfill slope can be constructed with a slope no steeper than 1.25H:1V.

When considering pile driving activities for the proposed culvert, there is a possibility that the slope factor of safety may be affected. The reasons for this phenomenon and potential risks and mitigation measures have been discussed in the previous section.

5.1.5 Stage 5 Construction: Backfilling North Lane

Stage 5 construction consists of backfilling the north lane with rockfill. Based on slope stability analyses, this can be successfully accomplished with rockfill slopes of 1.25H:1V to elevation 195.3 m. This applies to the selection of the Option 1 only. For Option 2, proceed to the next section for backfilling of the north lane.

5.1.6 Stage 6 Construction: Raising Grade with Lightweight Fill and Placement of Final Pavement Structure

Slope stability analyses were carried out for both 3 and 4 m high maximum grade raises. As the stability of the existing embankment has a calculated factor of safety between 1.2 to 1.3 for both undrained and drained (long term) conditions, slope stabilization measures are required to facilitate the grade raise. It is understood that the placement of fill materials beyond the toe of the existing embankment to accommodate slope flattening and/or the construction of flanking berms is not permitted for this project.

Due to the above noted restrictions, the use of mechanically stabilized earth slopes and/or bin walls were considered to facilitate a higher embankment (utilizing granular fill), which can produce steeper slopes. The mechanically stabilized earth slopes and/or bin walls can provide satisfactory stability within the embankment fills. However, due to the available strength of the foundation soils, these systems prove to heavy when utilizing granular fill.

The use of a piled system to both improve stability of embankment and facilitate construction of the grade raise was considered. However, driving piles through the significant amounts of rockfill present is not considered to be practical.

Therefore for the proposed grade raise the utilization of lightweight fills with or without retaining systems to support the lightweight fills was assessed for the proposed construction.

Lightweight fills considered include the following:

- Elastizell Concrete Fill
- Slag Fill
- PlastiSpan (Insulation Board)

Elastizell concrete fill is the second lightest of the fill materials considered with a unit weight of about 6.6 kN/m^3 and a compressive and shear strengths of 800 kPa and 198 kPa respectively. For added frost resistance, the manufacturer has recommended the use of Class IV Elastizell EF. This material requires soil cover for protection from weathering. The amount of Elastizell fill required to provide a stable embankment configuration to accommodate the 3 to 4 m raise in grade is approximately 1.8 m of Elastizell for every 1 m of grade raise (including pavement structure). This will result in excavation depth of up to 4 m for a 3 m grade raise and 5 m for a 4 m grade raise. This would require almost full removal of the existing embankment, which is not considered practical.

Steel furnace slag fill consisting of 90 mm size structural coarse has a unit weight of about 11.5 kN/m^3 . The amount of slag fill required to provide a stable embankment to accommodate the 3 to 4 raise in grade is about 2.1 m of slag fill for every 1 m of grade raise (including pavement

structure). This would result in excavation below the existing embankment, which is not considered practical.

Through discussions with the client, PlastiSpan 24, plastic foam for road embankments, has been assessed favourably for use as lightweight fill for the proposed construction. This product has a design unit weight of 0.5 kN/m^3 where adequate drainage is assured (such as this case where the foam will be underlain by free draining rockfill). The amount of PlastiSpan insulation board required to provide a stable embankment to accommodate the 3 to 4 m raise in grade is 1.15 m of insulation board for every 1 m of grade raise. This will result in excavation depth of up to 1.95 m for a 3 m grade raise and 2.1 m for a 4 m grade raise (adjusted for 1.5 m thick pavement structure above PlastiSpan). It is anticipated that PlastiSpan will be required from Station 14+600 to 14+850. It is understood that the PlastiSpan insulation board and pavement structure will be supported laterally by structural means likely consisting of a back to back tied steel or concrete wall. Proposed design sections, including an alternate configuration (Option 2) provided by the client have been illustrated on Enclosures 39 and 40. The internal stability of the gabion or bin walls should be assessed by a structural engineer and/or product supplier.

5.2 Embankment Settlement

Given the PlastiSpan lightweight fill treatment proposed above, no net increase in load is expected. As such, settlements induced from the proposed grade raise are not expected. It should be noted that settlements from the original construction are expected to occur over a period of 20 to 50 years from the original construction. However, due to the presence of occasional to numerous sand and silt seams within the clay, shorter consolidation times are expected. As such, future settlements from the original construction are not expected.

5.3 Temporary Bridge Foundations

Shallow and deep foundations may be considered. Shallow bridge foundations may be designed on the following bearing resistances and following the recommendations provided in the previous sections (where applicable):

Temporary Bridge Foundations at Elevation 192.6 m

Factored Bearing Resistance at ULS	160 kPa
Bearing Resistance at SLS	110 kPa (25 mm settlement in 6 months)
	125 kPa (50 mm settlement in 6 months)

The above design values are for vertical concentric loads in compression. Settlements have been estimated for a maximum footing width of 6 m. The rockfill surface should be chinked in accordance with OPSS 206.07.08 and covered with a levelling coarse of 150 mm of Granular "B", Type 1 fill compacted to 95% of standard Proctor maximum dry density. A woven geotextile should be placed as a separator below the Granular "B", Type 1 fill as settlement can be caused by the finer materials infiltrating voids in the rockfill.

Given the large size of footing required, it may prove more practical to utilize a piled foundation for the temporary bridge. However, difficult driving through existing rockfill can be expected. Alternative installation methods can be considered such as a partially shored excavation through rockfill, or installation of casing. If piles for the temporary bridge are considered they should be installed prior to excavating below elevation 193.0 m to ensure slope stability is not jeopardized from driving operations. Should this not be the case, the pile installation methodology must be reviewed. Section 5.5 provides recommendation for driven steel piles.

5.4 Culvert Foundation

The use of footings for the proposed culvert is not practical. This is due to the relatively large loads for the proposed culvert foundation (500 to 1000 kN/m on each side of culvert) in relation to the low strength of the underlying clays which provide a bearing capacity in the order of 70 kPa ULS. To provide a footing to meet this bearing capacity, construction would be required within the creek. A piled foundation (consisting of end bearing piles) will be the most practical foundation type.

5.5 Driven Steel Piles

The geotechnical capacity of steel piles driven to refusal into bedrock under both ULS and SLS is in excess of 2000 kN for HP 310 x 79 steel H-piles. This is greater than the structural capacity of the pile itself. Therefore piles driven to refusal into bedrock can be designed on their full structural capacity in accordance with the Canadian Highway Bridge Design Code (CHBDC), Sections 6 and 10.

The new culvert will be of larger size than the existing timber culvert, and the new culvert loads will be supported on end bearing piles. As such, there will be a net unloading of the foundation soils within the area of influence of the new culvert. The proposed raise in grade will result in a further reduction of foundation loads through the use of lightweight fill. Therefore, settlement resulting in drag down forces on the piles is not expected.

The bedrock at this location was proven at Boreholes 2 and 4 between elevations 171.5 m and 163.4 m. Auger refusal occurred in Boreholes 1 and 3 at elevations 170.6 m and 170.1 m. As such, the founding elevation of the piles will likely vary between elevation 163.4 and 171.5 m. However, the depth to competent bedrock may vary significantly and unexpectedly over short distances.

For piles supporting the proposed culvert, the assessed horizontal passive resistance for a single HP 310 x 79 pile can be taken as 120 kN at ULS and 35 kN at SLS. These assessed values assume a pile inclination less than 15° and a horizontal deformation of 10 mm at ground surface. Where piles are driven in groups, the passive resistance per pile may reduce. As a general rule, for piles spaced normal to the direction of loading, no reduction is required for a spacing greater than 2.5 times the pile diameter. For pile spacing in the direction of loading, no reduction is required for a pile spacing of 8 times the pile diameter or greater. Reduction factors of 0.25, 0.4, and 0.7 should be applied to pile spacing (in the direction of loading) of 3, 4, and 6 times the pile diameter, respectively. Alternatively, the use of battered piles may be considered to resist horizontal loads.

The driving equipment should have the capacity to exert a driving energy of at least 55 kJ. The steel section should be adequate for sustained heavy driving.

Refusal is tentatively defined as 20 blows for 25 mm for 75 consecutive millimetres subject to the discretion of the geotechnical engineer. Approval of satisfactory refusal should also involve consideration of the pile tip elevation with respect to the borehole data and other adjacent piles.

Inspection and quality control should be in accordance with CHBDC 6.4.6.

Driving records should be kept for each pile. Information to be recorded should include but not necessarily limited to: pile dimensions, hammer type, rated energy, ram weight, cap block weight and type, anvil weight, number of blows for each 0.3 m of penetration and final set. All pile driving equipment must be in good working order.

The base of the rockfill at the borehole locations is at elevations 189.6 m and 191.6 m therefore, the rockfill should not hinder the driving of the pile for the proposed culvert foundations. Because of the presence of boulders in the sand stratum, and high stresses induced during driving on the bedrock or boulders, the pile tips should be reinforced to prevent damage. It is recommended the piles be equipped with a driving shoe such as Oslo Points as per OPSD 3304.000 or equivalent. The behaviour of the piles should be monitored during driving for any signs of pile damage. To ensure that rock points are set into the bedrock, it is recommended that the driving energy at refusal be gradually increased to the maximum specified energy in accordance with OPSS 903.07.02.05.

The elevation of the tops of driven piles should be measured immediately after driving. If uplift occurs in any piles during the driving of adjacent piles, the displaced piles should be re-driven to at least their previous final elevation and final set.

Pile spacing should be in accordance with CHBDC 6.8.9.2.

Pile splices should be in accordance with CHBDC 6.8.10.2

5.6 Frost Protection

In accordance with the Ministry of Transportation Structural Manual for District 19, the design frost protection depth is 2.2 m.

Rockfill and rock protection shall count for half of their thickness in determining the depth of cover provided.

As the native soils are highly frost susceptible, culvert pile caps should have a minimum of 2.2 m of earth cover to protect against frost penetration. If less soil cover is provided, then equivalent synthetic insulation should be provided for frost protection. A minimum of 150 mm thick Styrofoam SM insulation (or equivalent) placed below the pile cap and extending a minimum of 2.44 m beyond the footprint should be provided. The placement of Styrofoam should consider buoyancy forces (if placed below the water table) and protection from the environment. Alternative frost protection measures may be considered.

5.7 Lateral Earth Pressure

It is understood that the culvert will require a granular cover. Earth pressure should be computed as per Section 6.9.2 of the Canadian Highway Bridge Design Code (CHBDC). Granular "A" or "B", Type 1 backfill should be in accordance with Ontario Provincial Specifications (OPSS 1010). The following parameters are recommended when calculating earth pressures.

	Granular "A"	Granular "B", Type 1	Rockfill
Angle of Internal Friction	$\phi' = 35^\circ$	$\phi' = 30^\circ$	$\phi' = 40^\circ$
Unit Weight (kN/m ³)	$\gamma = 23$	$\gamma = 21$	$\gamma = 20$

5.8 Scour Protection

The pile caps should be provided with sufficient scour protection to ensure the piles are not exposed. Scour protection should be in accordance with Section 1.10 of the CHBDC.

5.9 Construction Considerations

In general, the proposed culvert installation will be challenging with strict adherence to design limits being mandatory. Deviation for the design limits can have a significant impact on stability.

The following items must be considered:

- Given the relatively steep embankment slopes, staging will be challenging given that it is restricted by geometric constraints of the existing embankment cross section.
- All excavations should be carried out in accordance with the Occupational Health and Safety Act of Ontario. Surcharge loads such as stockpiles of fill and/or excavated

materials must not be placed on the embankment as this could significantly lower the design factor of safety for embankment stability.

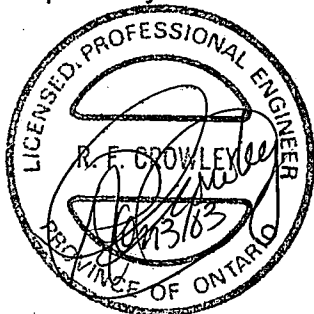
- If the design incorporates excavations to the creek base, then dewatering of the excavation will be required which will be difficult. Dewatering may also be hindered due to the presence of sand seams within the clay soils. A detailed dewatering system will have to be designed to accommodate a dry excavation. The use of sheet piles may be required to facilitate dewatering. It should be noted that a potential for base heave exists for excavations below the clay surface and a system of relief wells may be required. Careful monitoring and inspection of excavations for potential heaving conditions should be carried out.
- Driving of piles may induce excess porewater pressures near the toe of the excavation slope. This has the potential to destabilize the slope and must be monitored for during construction. Delays during pile driving operations may be required to allow excess porewater pressures to dissipate.
- Should piles be considered for the temporary bridge foundation, difficult driving conditions can be expected where piles are to be driven through some thickness of existing rock fill. Alternatively, partial excavation and/or drilling methods through the rockfill may be required.
- The clay foundation will be highly sensitive to construction traffic, and as such should not be disturbed. Exposed clay surfaces should be covered without delay.
- The excavation slopes should be visually monitored especially during operation of heavy equipment that may induce vibrations. Should slope distress be suspected, alternate construction methods may be required.

6.0 LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:



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Sr. Project Engineer



Gordon Maki, P. Eng.
Sr. Project Engineer

Reviewed by:



Mike Fabius, P.Eng.
Principal

APPENDIX 'A'
LIMITATIONS OF REPORT

APPENDIX 'A'

LIMITATIONS OF REPORT

The conclusions and recommendations presented in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. 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, excavation, 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 details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given 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, e.g. 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 conclusion as to how the subsurface conditions may affect their work.

APPENDIX 'B'

ROCK CORE DESCRIPTIONS

PRECISION AGE AGGREGATE TESTING

Rock Core Description

Project No.: 194-87-00

Site No.:

Client No.:

Location: 14 + 710 - Walsh Township

Bore Hole No.: 2

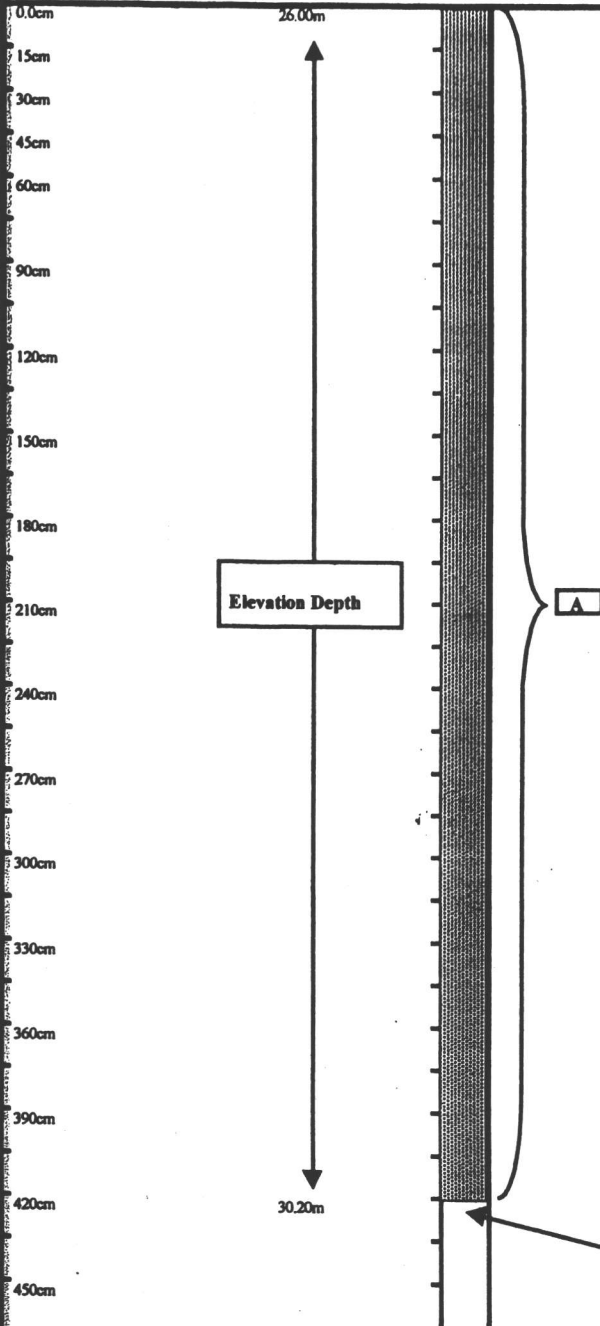
Depth: 26.0m to 30.2m

Field No.:

Lab No.: 02-WW-3001

Depth:

Logged by: B Werbowetski



A - Approximately 26.00m to 30.20m

Rock Type: Syenite

Physical Characteristic's

- Hypabyssal Igneous Rock - massive structure - aphanitic (fine grained) and equigranular.
- Mesocratic - composed of almost equal amounts of light and dark constituents.
- Colour: Pale red and homogeneous throughout the length of core.
- Hardness: High strength - generally cannot be scratched.
- Minerals: Feldspar plus one or more ferromagnesian minerals.

Fractures:

- From 26.00m to 28.35m orientation of fractures appear N Vertical to Dipping - faces appear relatively smooth - a thin veneer of soft dark patches or crusts appear on the face of several fractures. Spacing of fractures close (5 to 30cm).
- From 28.35m to 30.20m the description of fractures is basically the same - the only exception is the orientation which is predominately flat.

Note: Hardness or strength based on MTO LS 609

% Core Recovery:

% Rock Quality Designation:

Discontinuities (J)

Symbol	Spacing	Orientation
SS - Well-sorted Sand	VW - Very Wide - > 1m	0 - Flat (0° dip)
CS - Coarse Sand	W - Wide - 1-10cm	1 - Dipping (1° to 10°)
SA - Slightly Altered Clay Sand	MD - Moderate - 1-10cm	2 - Dipping (10° to 30°)
SL - Sandy Silty Silt/Clay	CL - Close - 1-10cm	3 - Dipping (30° to 60°)
MC - Medium-grained Clay	VC - Very Close - < 1cm	4 - Dipping (60° to 90°)
SC - Silty Clay		

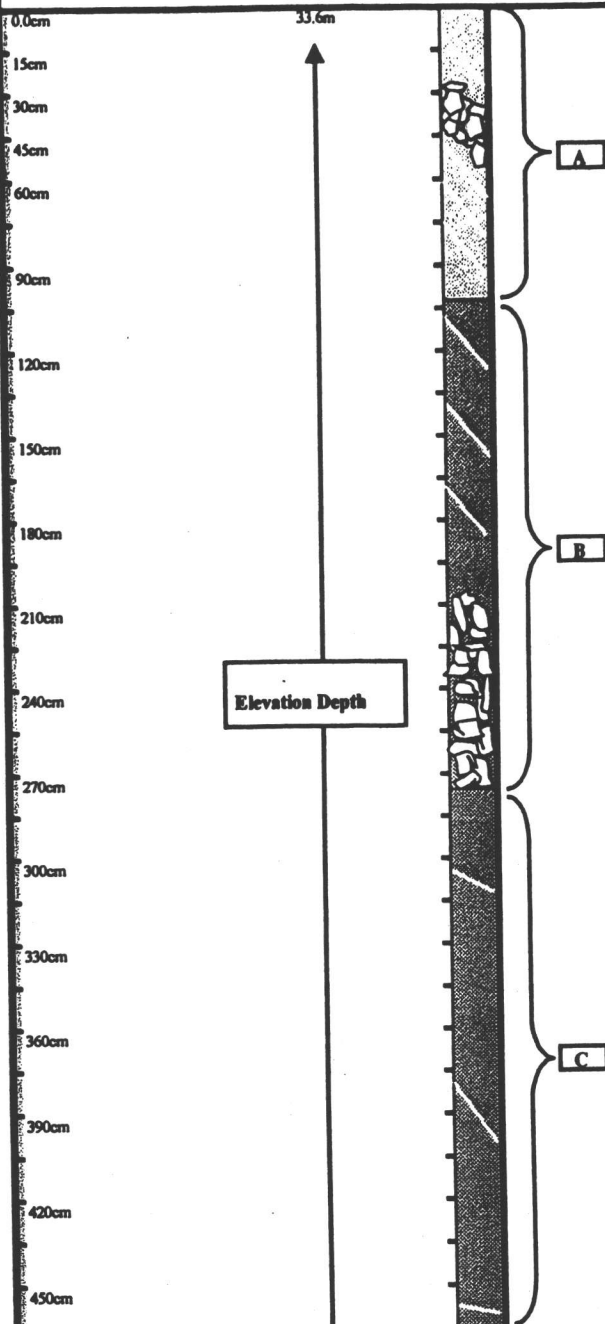
PRECISION AGE AGGREGATE TESTING **Rock Core Description**

Project No.: 194-87-00
Site No.:
Clasts No.:

Bore Hole No.: 4
Depth: 33.6m to 38.6m
Field No.:

Lab No.: 02-WW-3002
Depth:
Logged by: B Werbowetski

Location: 14 + 740 Walsh Township



A - Approx. 33.60m to 34.60m

Rock Type: Volcanic

Physical Characteristic's

- Extrusive Igneous Rock (e.g. of a Flow Breccia) - Porphyritic like texture with phenocrysts set in an aphanitic equigranular matrix. This section of core also includes Breccia like fragments (< than 3cm) intermittently dispersed.
- Felsic or Leucocratic
- Colour: Pale brown.
- Hardness: Medium strength - can be scratched with moderate ease and scraped with some difficulty - brittle (edges and corners can be plucked).
- Thin white bands (<1mm in thickness) occur intermittently - orientation of these bands are N Vertical and are spaced approximately 7 to 10 cm apart.

B - Approx. 34.60m to 36.35m

Rock Type: Volcanic

Physical Characteristic's

- Extrusive Igneous Rock (e.g. of a Flow Breccia) - Porphyritic like texture with phenocrysts set in an aphanitic equigranular matrix. This section of core also includes Breccia like fragments (< than 3cm) intermittently dispersed.
- Felsic or Leucocratic
- Colour: Pale red.
- Hardness: Medium strength - can be scratched with moderate ease and scraped with some difficulty - brittle (edges and corners can be plucked).
- Thin white bands (<1mm in thickness) are spaced approximately 20cm apart or more- orientation of bands appear N Vertical.

% Core Recovery:

% Rock Quality Designation:

Discontinuity (J)

Fracture

SS - Sandstone

CL - Claystone

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

Fracture

SL - Shale

CL - Claystone

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

Fracture

SL - Shale

CL - Claystone

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

SL - Shale

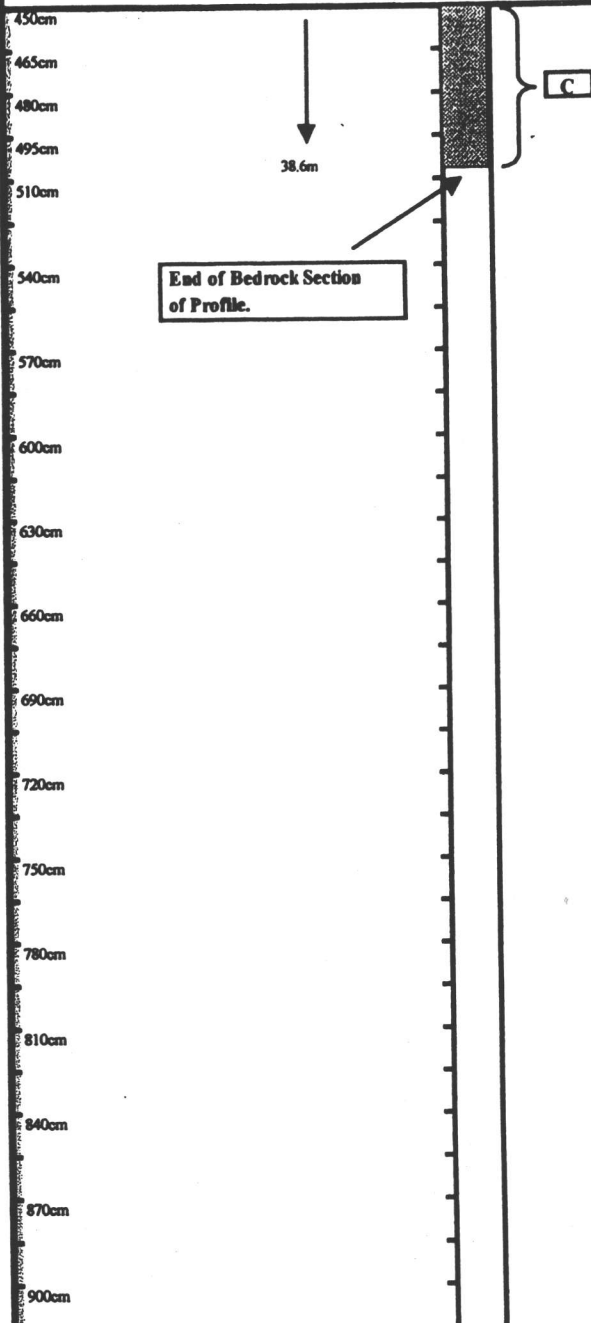
PRECISION AGE AGGREGATE TESTING
Rock Core Description

Project No.: 194-87-00
Site No.:
Client No.:

Bore Hole No.: 4
Depth: 33.6m to 38.6m
Field No.:

Lab No.: 02-3002
Depth:
Logged by: B Werbowetski

Location: 14 + 740 Walsh Township



C - Approx. 36.70m to 38.60m

Rock Type: Volcanic

Physical Characteristic's

- Extrusive Igneous Rock (e.g. of a Flow Breccia) - Porphyritic like texture with phenocrysts set in an aphanitic equigranular matrix. This section of core also includes Breccia like fragments (< than 5cm) intermittently dispersed.
- Felsic or Leucocratic
- Colour: Pale red.
- Hardness: Medium strength (can be scratched with some difficulty) to high strength (difficult to scratch).
- Thin white bands (<1mm in thickness) occur less intermittently - orientation of these bands are N Vertical and are spaced approximately 20cm apart or more.

A to C (Approx. 33.6m to 38.6m)

Fractures: A number of fractures occur along the thin white bands - orientation ranged from N Vertical to Flat with a thin veneer of crust on the face that feels slightly soapy - face relatively smooth and flat.

A thin veneer of soft dark patches or crusts appear on the face of several other fractures including evidence of oxidation - orientation varies from N Vertical to Flat - face relatively smooth to flat. Spacing of fractures vary from moderate to close.

Note: Hardness or strength based on MTO LS 609

% Core Recovery:

% Rock Quality Designation:

Discard/Issues (#)

Fractures

FR - Fractures (100)

CR - Cracks

SA - Single Mineral Core Frag

MA - Multiple Mineral Core

MC - Multiple Mineral Core

SC - Single Mineral Core

Structure

FW - Very Weak

W - Weak

MD - Moderate

CL - Clear

SC - Very Strong

Orientation

1 - Flat (0-90 deg)

2 - Dip (90-180 deg)

3 - Vertical (180-270 deg)

A P P E N D I X 'C'

SLOPE STABILITY

Back Analysis of Cross Section at Station 14+680

Drained Analysis

Description:

File Name: dn-Station-14+680-back-1-c.slp

Last Saved Date: 04/07/02

Last Saved Time: 8:40:40 AM

Analysis Method: Morgenstern-Price

Direction of Slip Movement: Right to Left

Slip Surface Option: Grid and Radius

P.W.P. Option: Piezometric Lines / Ru

Soil: 1

Description: Traffic Load

Soil Model: No Strength

Unit Weight: 120

Piezometric Line #: 0

Soil: 2

Description: Granular

Soil Model: Mohr-Coulomb

Unit Weight: 20

Cohesion: 0

Phi: 32

Piezometric Line #: 0

Soil: 3

Description: Rock Fill

Soil Model: Mohr-Coulomb

Unit Weight: 18

Cohesion: 0

Phi: 45

Piezometric Line #: 0

Soil: 4

Description: Clay 1

Soil Model: Mohr-Coulomb

Unit Weight: 18

Cohesion: 0

Phi: 25.5

Piezometric Line #: 1

Soil: 5

Description: Sand and Silt

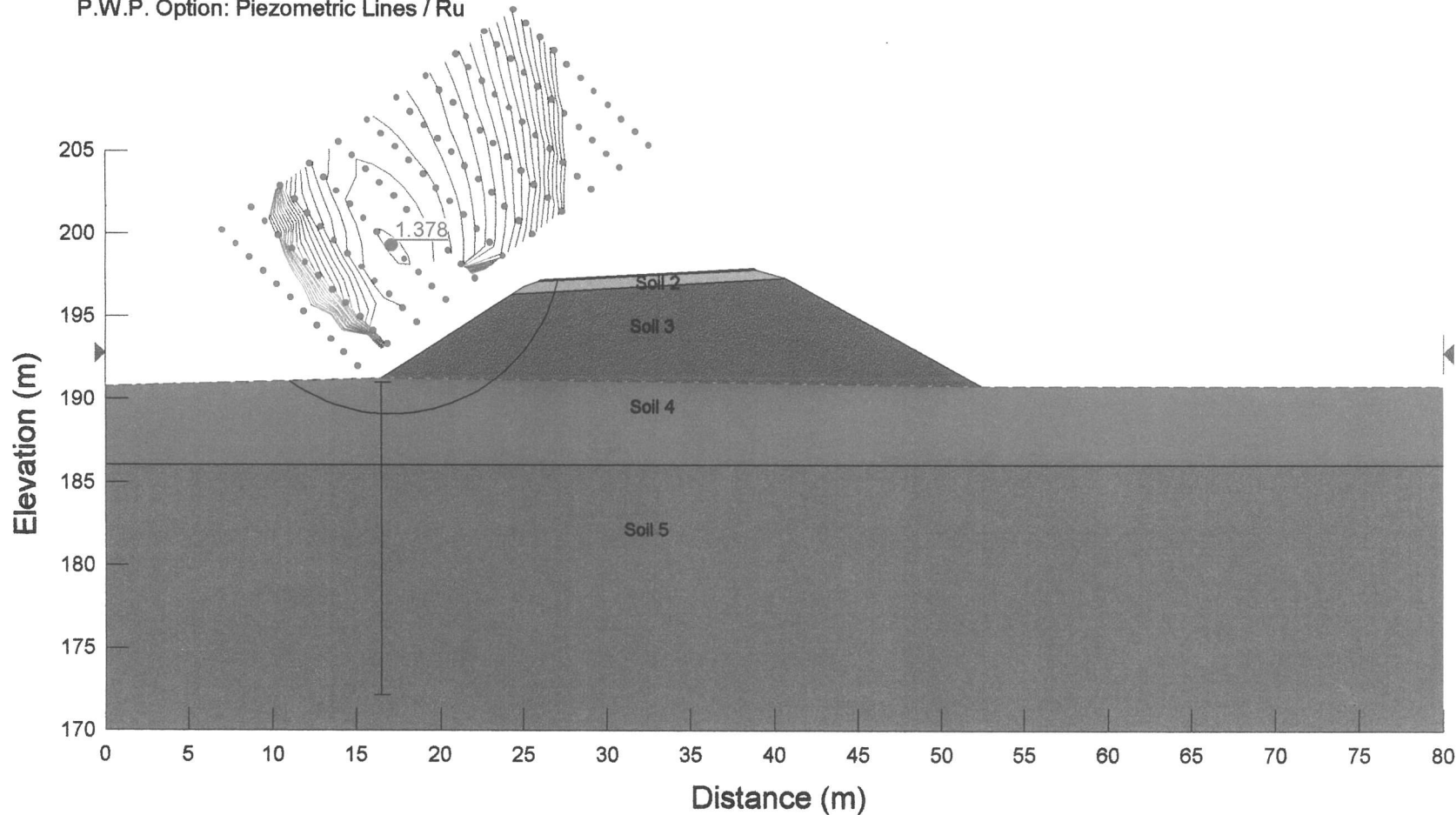
Soil Model: Mohr-Coulomb

Unit Weight: 20

Cohesion: 0

Phi: 32

Piezometric Line #: 1



Back Analysis of Cross Section at Station 14+680

Undrained Analysis

File Name: un-Station-14+680-back-1-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 1:57:54 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

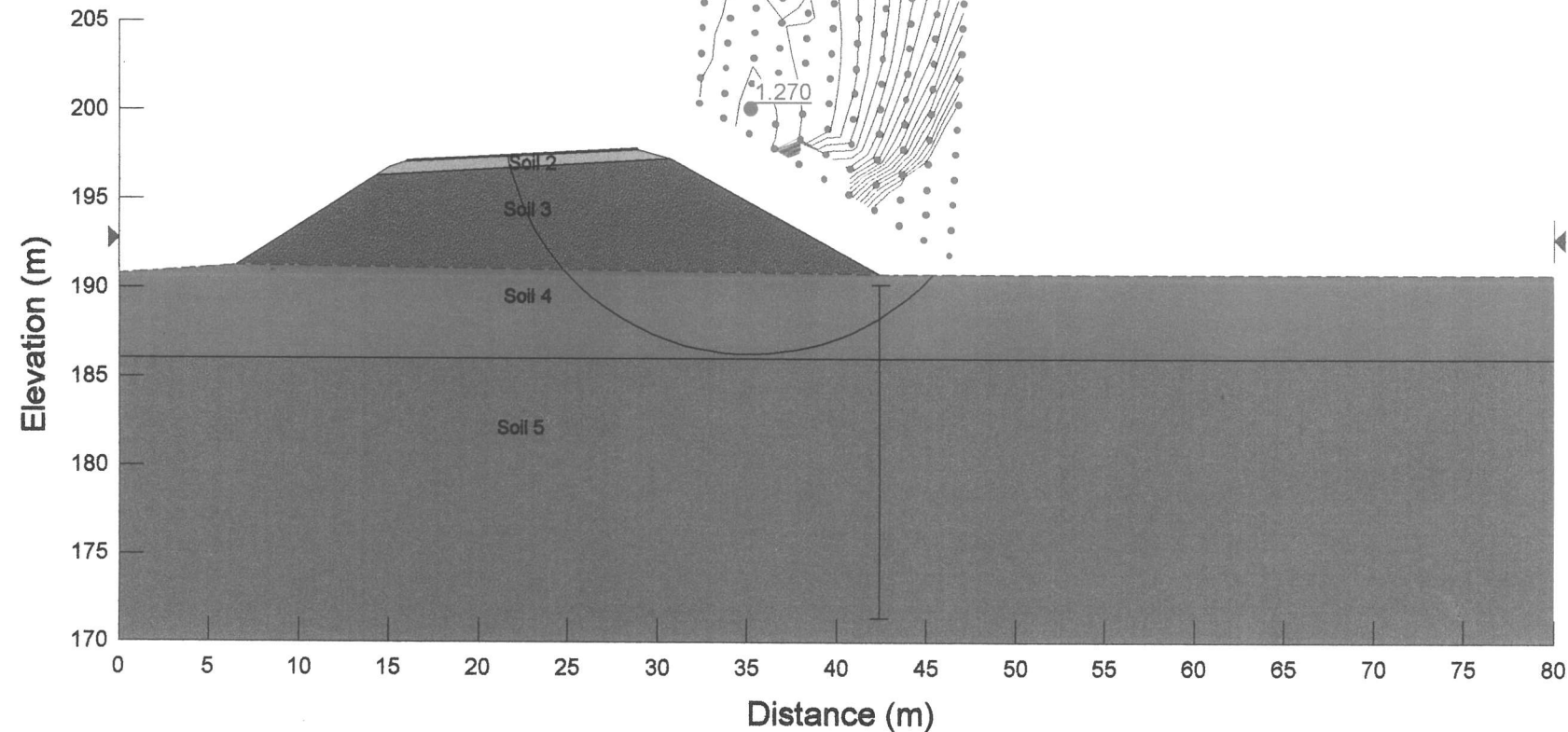
Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Granular
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 0

Soil: 3
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 4
Description: Clay 1
Soil Model: $S=f(\text{depth})$
Unit Weight: 18
C-Top of Layer: 23
Rate of Increase: 1.6
C - Maximum: 39
Piezometric Line #: 1

Soil: 5
Description: Sand and Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1



Back Analysis of Cross Section at Station 14+720 Drained Analysis

Description: McKellar Creek Station 14+720
File Name: dn-Station-14+720-back-1-c.slp
Last Saved Date: 04/07/02
Last Saved Time: 12:10:58 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 3
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

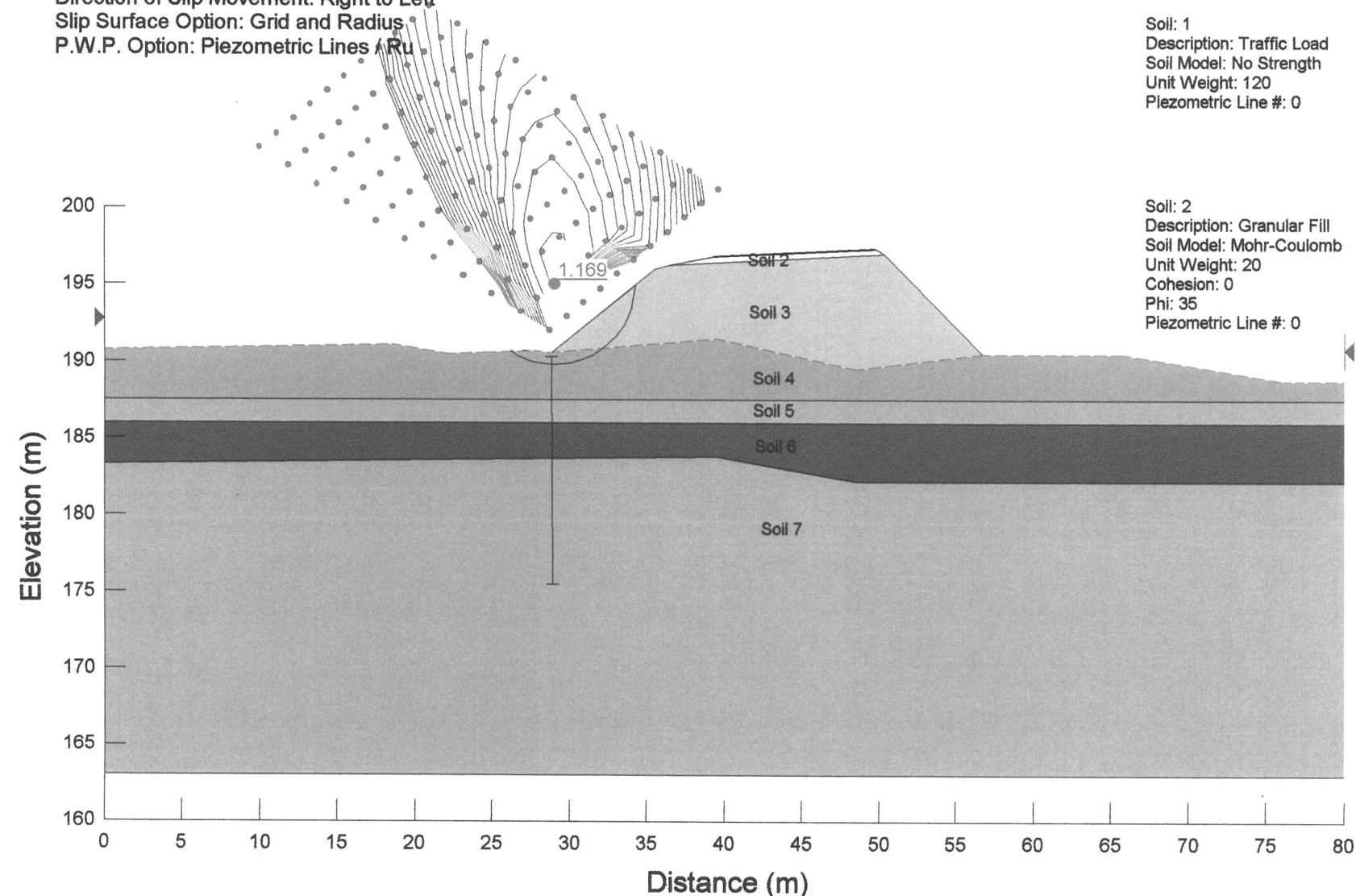
Soil: 2
Description: Granular Fill
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Piezometric Line #: 1

Soil: 5
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

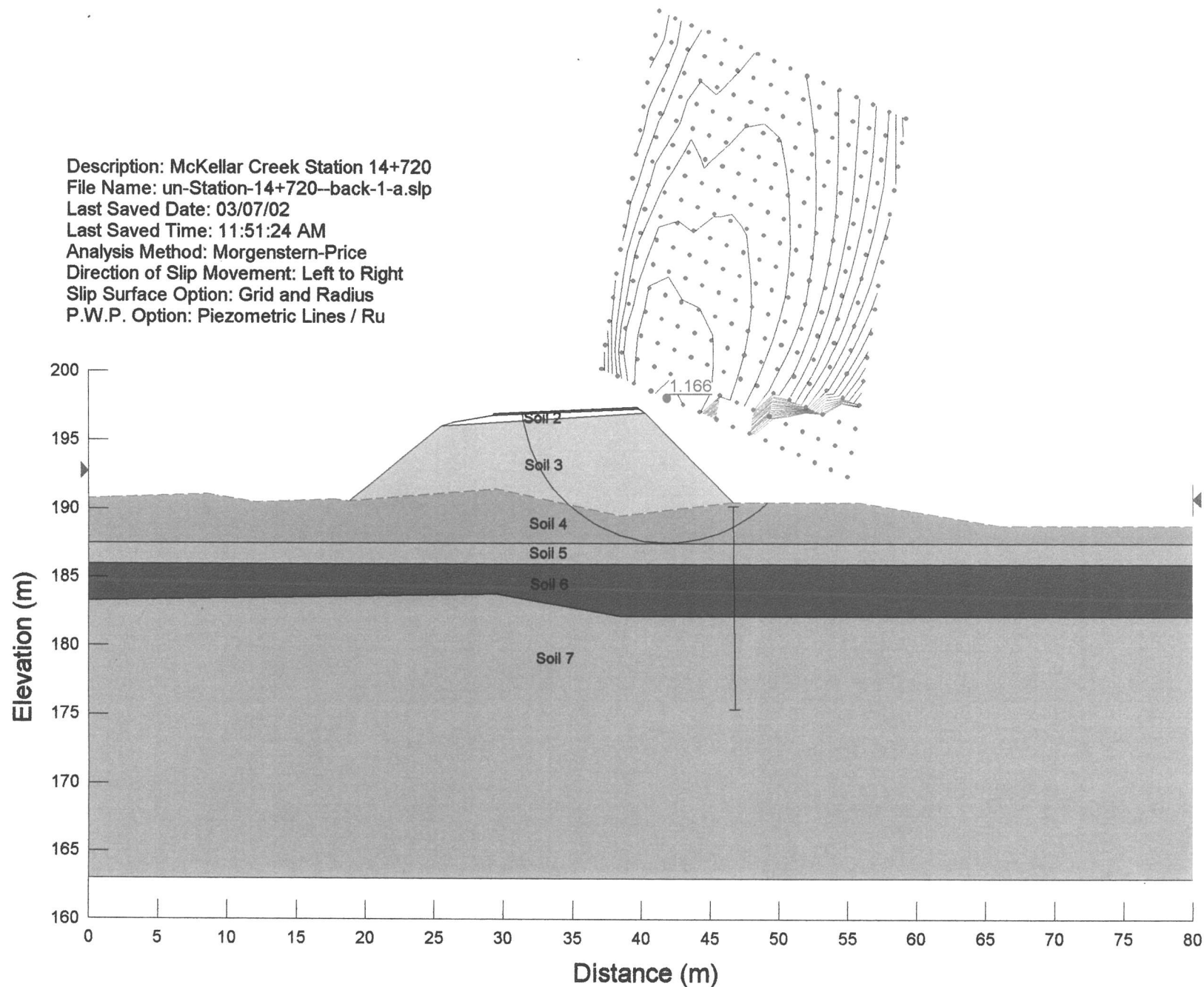
Soil: 6
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 7
Description: Sand & Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1



Back Analysis of Cross Section at Station 14+720 Undrained Analysis

Description: McKellar Creek Station 14+720
File Name: un-Station-14+720-back-1-a.slp
Last Saved Date: 03/07/02
Last Saved Time: 11:51:24 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Granular Fill
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 3
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 4
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 18
Cohesion: 24
Piezometric Line #: 1

Soil: 5
Description: Clay 2
Soil Model: Undrained (Phi=0)
Unit Weight: 18
Cohesion: 32
Piezometric Line #: 1

Soil: 6
Description: Clay 3
Soil Model: S=f(depth)
Unit Weight: 18
C-Top of Layer: 32
Rate of Increase: 3.25
C - Maximum: 45
Piezometric Line #: 1

Soil: 7
Description: Sand & Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Back Analysis of Cross Section at Station 14+750 Drained Analysis

Description: McKellar Creek Station 14+750
File Name: dn-Station-14+750-back-1-a.slp
Last Saved Date: 03/07/02
Last Saved Time: 12:33:20 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

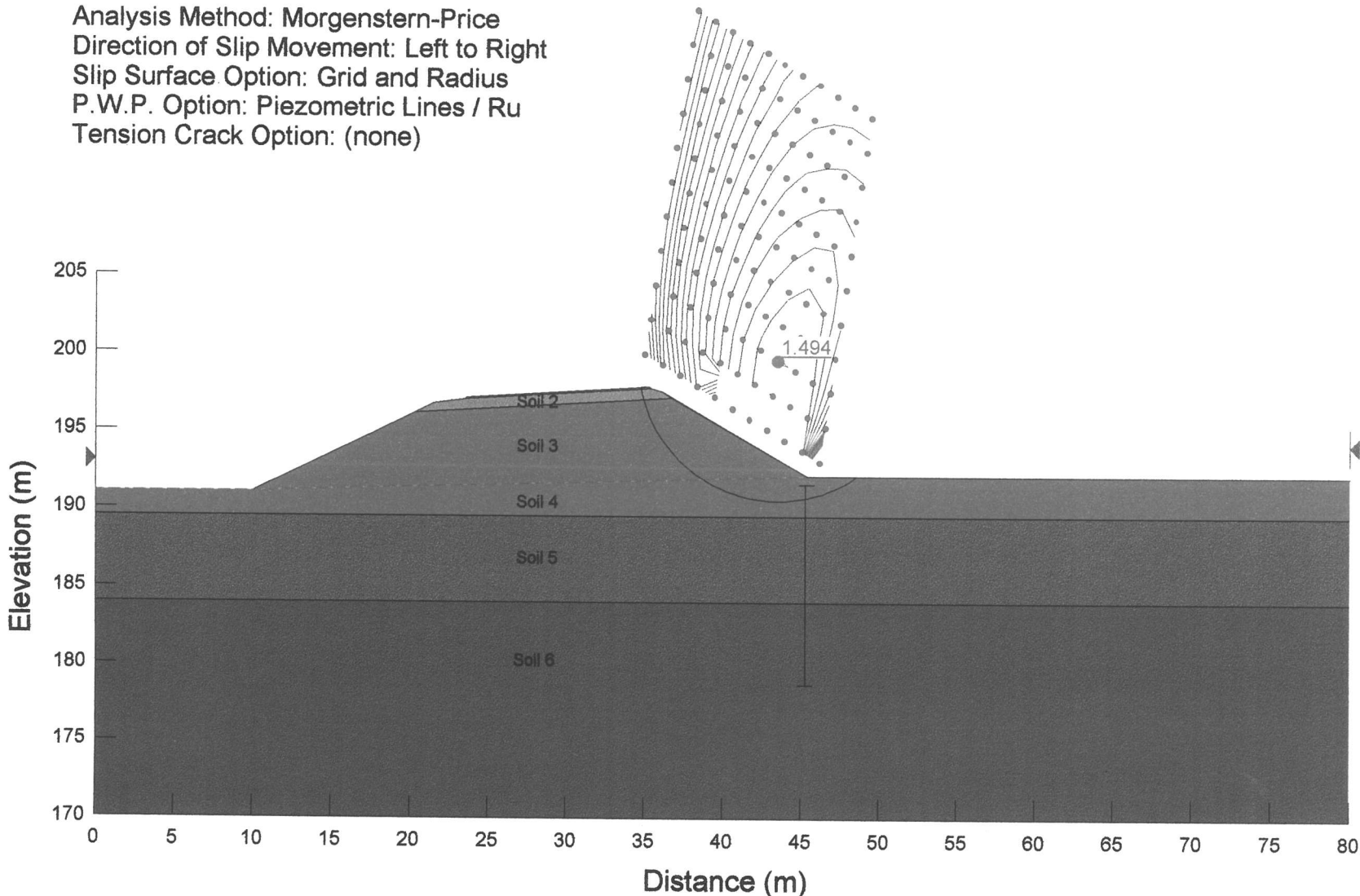
Soil: 2
Description: Granular Fill
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 3
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 4
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 5
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 6
Description: Sand and Silts
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1



Back Analysis of Cross Section at Station 14+750 Undrained Analysis

Description: McKellar Creek Station 14+750
File Name: un-Station-14+750-back-1-a.slp
Last Saved Date: 03/07/02
Last Saved Time: 12:32:50 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

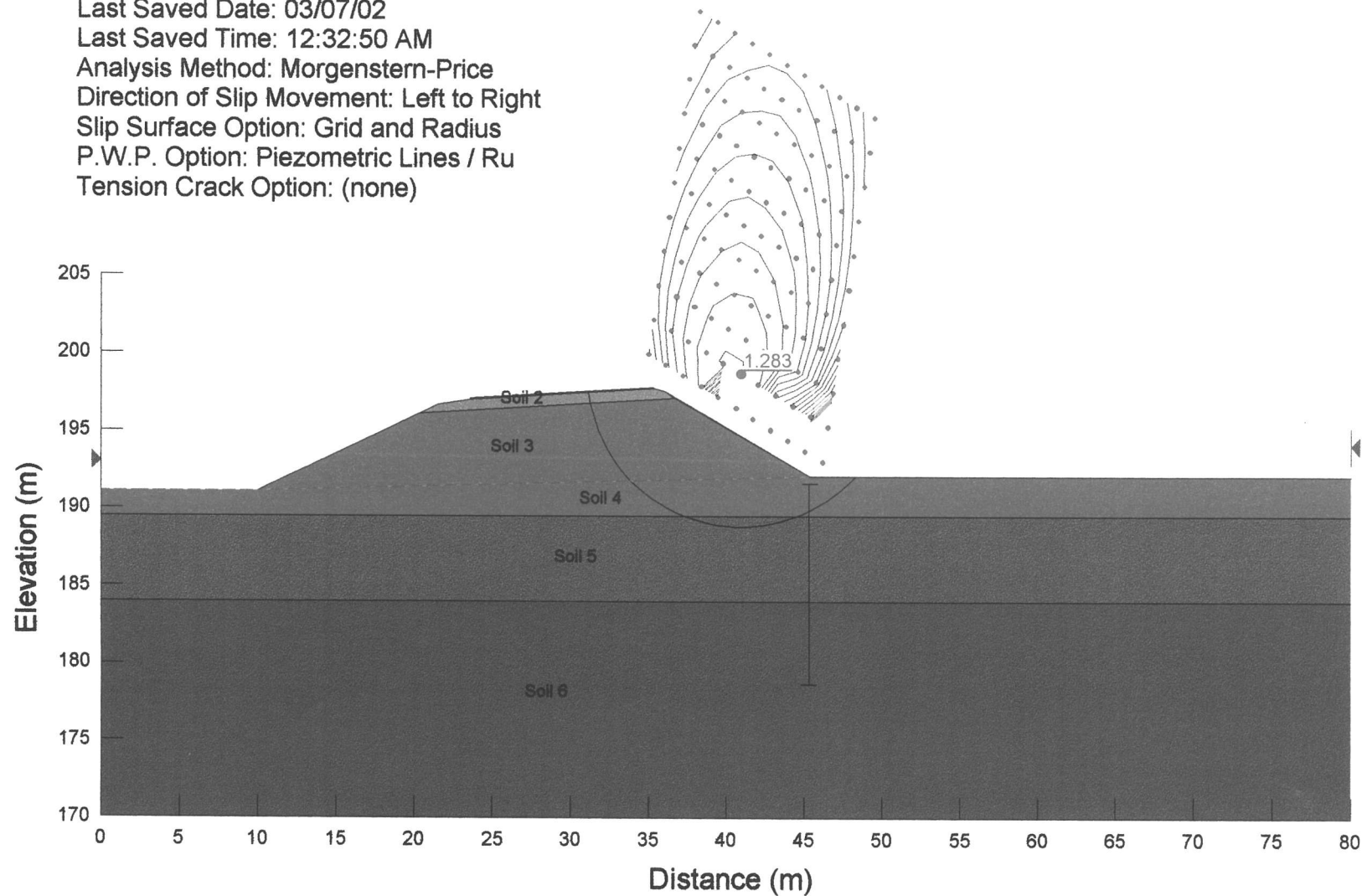
Soil: 2
Description: Granular Fill
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 3
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 4
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 18
Cohesion: 20
Piezometric Line #: 1

Soil: 5
Description: Clay 2
Soil Model: $S=f(\text{depth})$
Unit Weight: 18
C-Top of Layer: 20
Rate of Increase: 5.6
C - Maximum: 50
Piezometric Line #: 1

Soil: 6
Description: Sand and Silts
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1



Proposed Conceptual Design of Stage 2 Construction Drained Analysis -Option 1

Description: McKellar Creek Station 14+720
File Name: dn-mcklr-final-width-2-b.slp
Last Saved Date: 04/07/02
Last Saved Time: 12:43:35 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:

- 1) 5 x 6 m bearing area for bridge
- 2) 39.6 m long bridge
- 4) Elevation at bottom of bridge footing is 192.6 m
- 5) Temporary Roadway at elevation 195.5 m
- 6) Bench is at elevation 192.6 m
- 7) 6.2 m Roadway

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

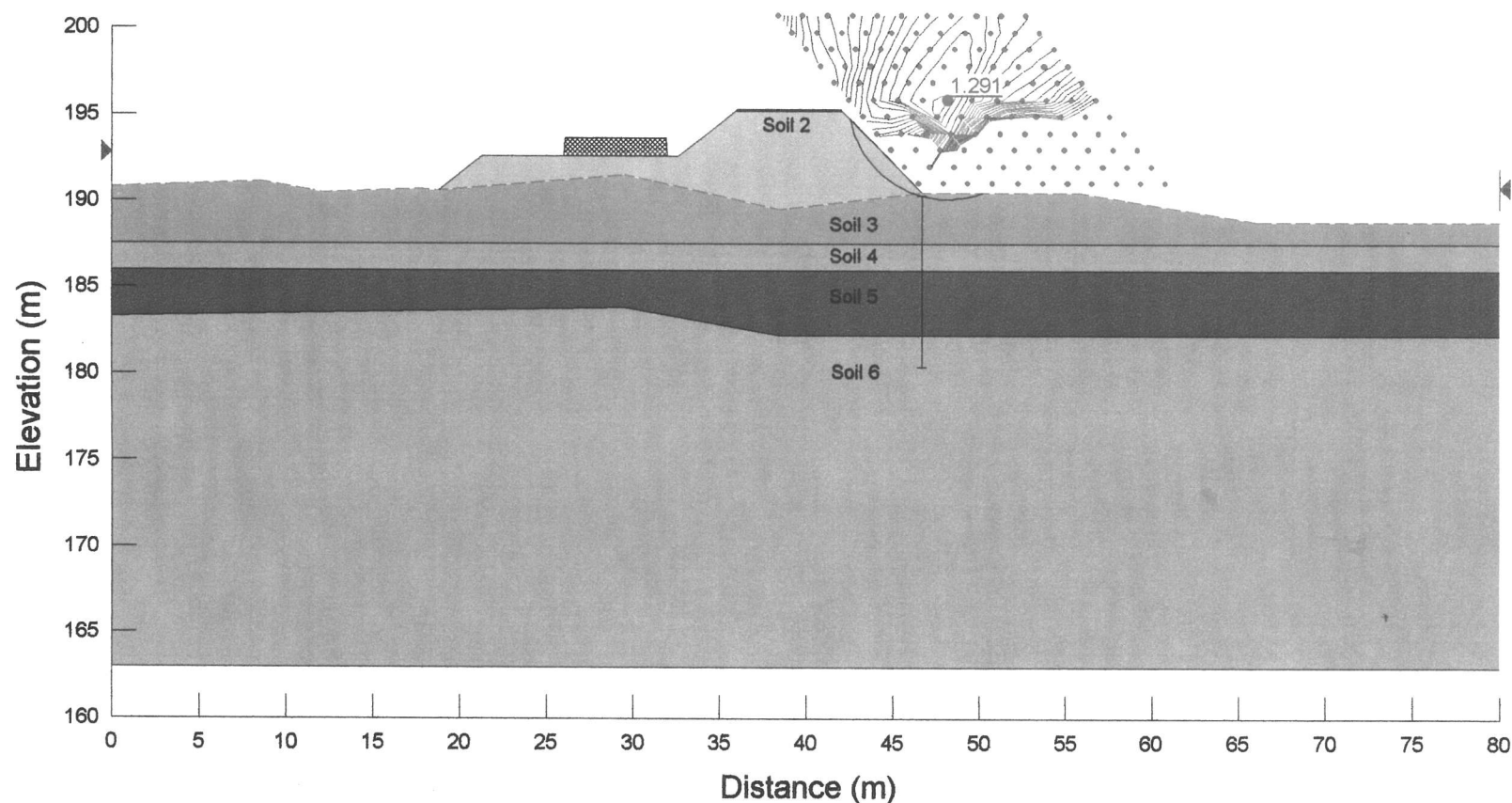
Soil: 2
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 3
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 4
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 5
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 6
Description: Sand & Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

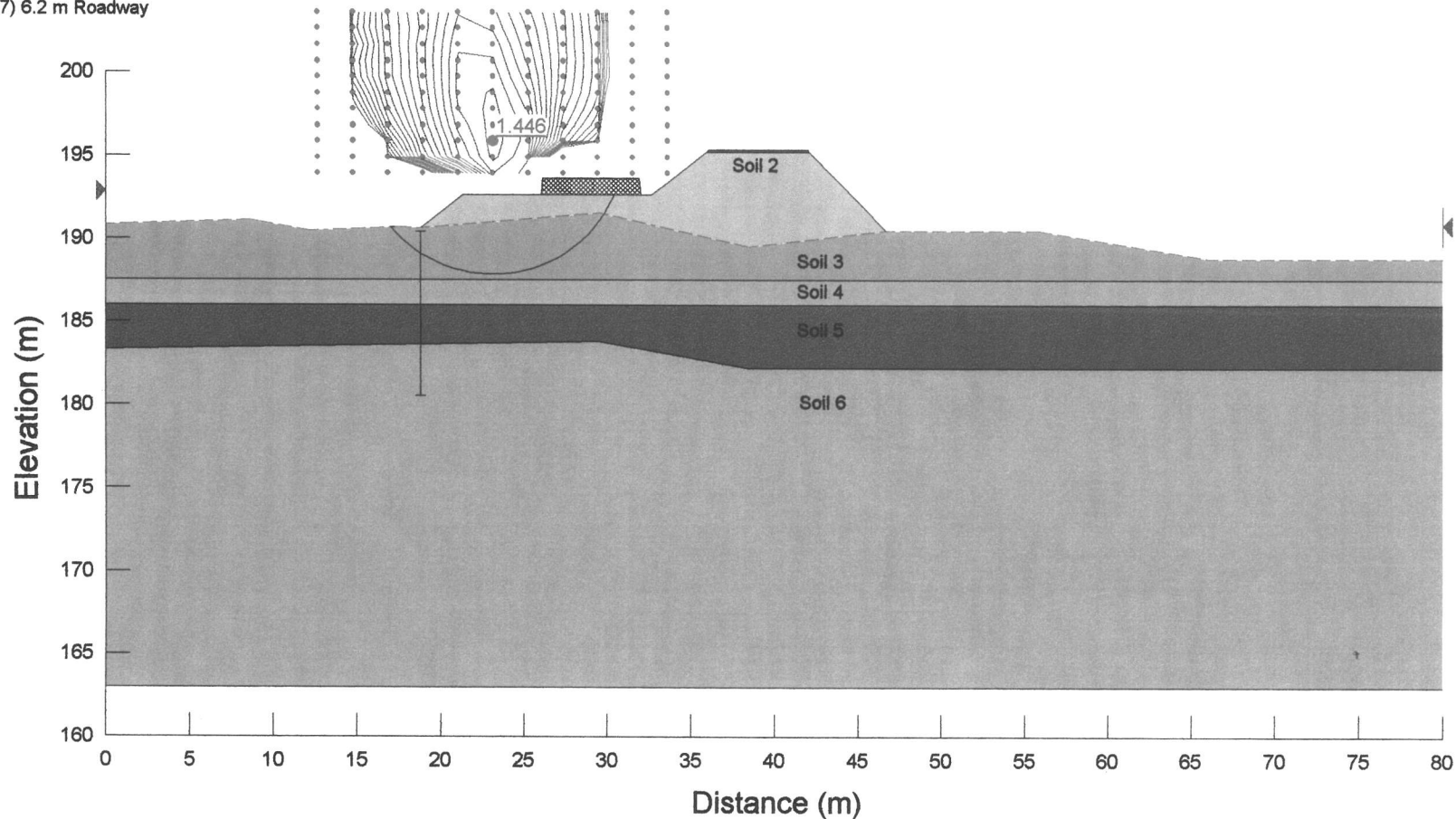


Proposed Conceptual Design of Stage 2 Construction Undrained Analysis-Option 1

Description: McKellar Creek Station 14+720
File Name: un-mcklr-final-width-2-a.slp
Last Saved Date: 04/07/02
Last Saved Time: 12:44:44 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:

- 1) 5 x 6 m bearing area for bridge
- 2) 39.6 m long bridge
- 4) Elevation at bottom of bridge footing is 192.6 m
- 5) Roadway at elevation 195.5 m
- 6) Bench is at elevation 192.6 m
- 7) 6.2 m Roadway



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 3
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 24
Piezometric Line #: 1

Soil: 4
Description: Clay 2
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 32
Piezometric Line #: 1

Soil: 5
Description: Clay 3
Soil Model: S=f(depth)
Unit Weight: 17
C-Top of Layer: 32
Rate of Increase: 3.25
C - Maximum: 45
Piezometric Line #: 1

Soil: 6
Description: Sand & Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Proposed Conceptual Design of Stage 3 Construction 8 m Culvert Drained Analysis - Option 1

Description: McKellar Longitudinal
File Name: dn-mcklr-long-eccfoot-final-1-a.slp
Last Saved Date: 04/07/02
Last Saved Time: 2:29:50 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

- Conditions:
- 1) 5 x 6 m footing for bridge el. 192.6m
 - 2) 72 kPa (concentric load) footing below bin wall
 - 3) 39.6 m long bridge
 - 4) 0.9 m of granular material removed from roadway.
 - 5) The elevation of the bench is 192.6 m

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 3
Description: Clear Stone
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 2
Description: Engineered Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 16
Phi: 45
Piezometric Line #: 0

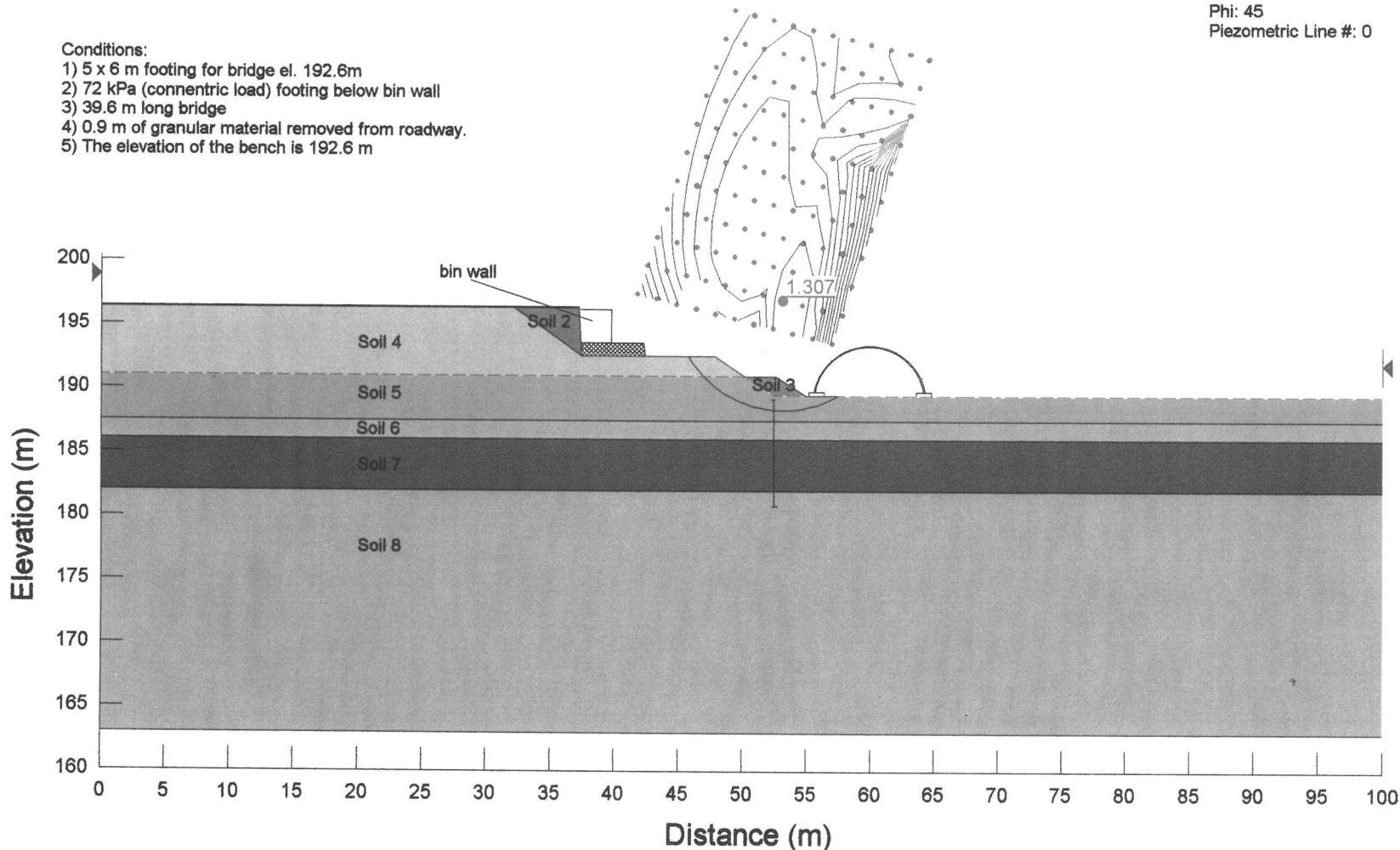
Soil: 4
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 5
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 6
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 7
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 8
Description: Silt & Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1
Pore-Air Pressure: 0



Proposed Conceptual Design of Stage 3 Construction 8 m Culvert Undrained Analysis -Option 1

Description: McKellar Longitudinal
File Name: un-mcklr-long-eccfoot-final-1-a.slp
Last Saved Date: 04/07/02
Last Saved Time: 2:48:46 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:
1) 5 x 6 m footing for bridge el. 192.6m
2) 72 kPa (concentric load) footing below bin wall
3) 39.6 m long bridge
4) 0.9 m of granular material removed from roadway.
5) The elevation of the bench is 192.6 m

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 3
Description: Clear Stone
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 2
Description: Engineered Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 16
Phi: 45
Piezometric Line #: 0

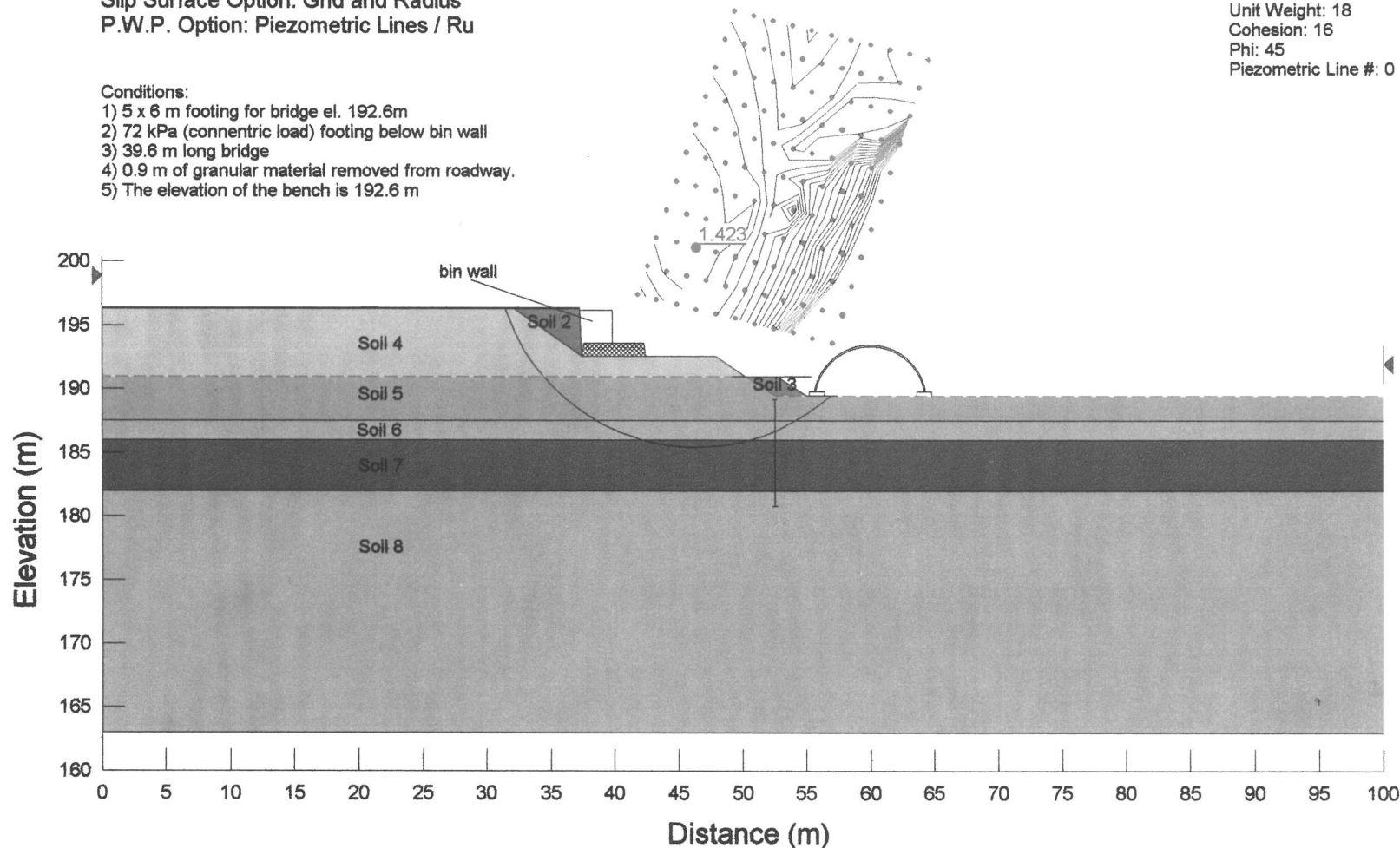
Soil: 4
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 5
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 24
Piezometric Line #: 1

Soil: 6
Description: Clay 2
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 32
Piezometric Line #: 1

Soil: 7
Description: Clay 3
Soil Model: S=f(depth)
Unit Weight: 17
C-Top of Layer: 32
Rate of Increase: 3.25
C - Maximum: 45
Piezometric Line #: 1

Soil: 8
Description: Silt & Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1



Proposed Conceptual Design of Stage 3 8 m Culvert Drained Analysis -Option 1

Description: McKellar Longitudinal
File Name: dn-mcklr-long-eccfoot-final-2-pwp-2-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 3:17:00 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:

- 1) 5 x 6 m footing for bridge el. 192.6m
- 2) 72 kPa (concentric load) footing below bin wall
- 3) 39.6 m long bridge
- 4) 0.9 m of granular material removed from roadway.
- 5) The elevation of the bench is 192.6 m

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 3
Description: Clear Stone
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 6
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 3

Soil: 2
Description: Engineered Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 16
Phi: 45
Piezometric Line #: 0

Soil: 4
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 7
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 4

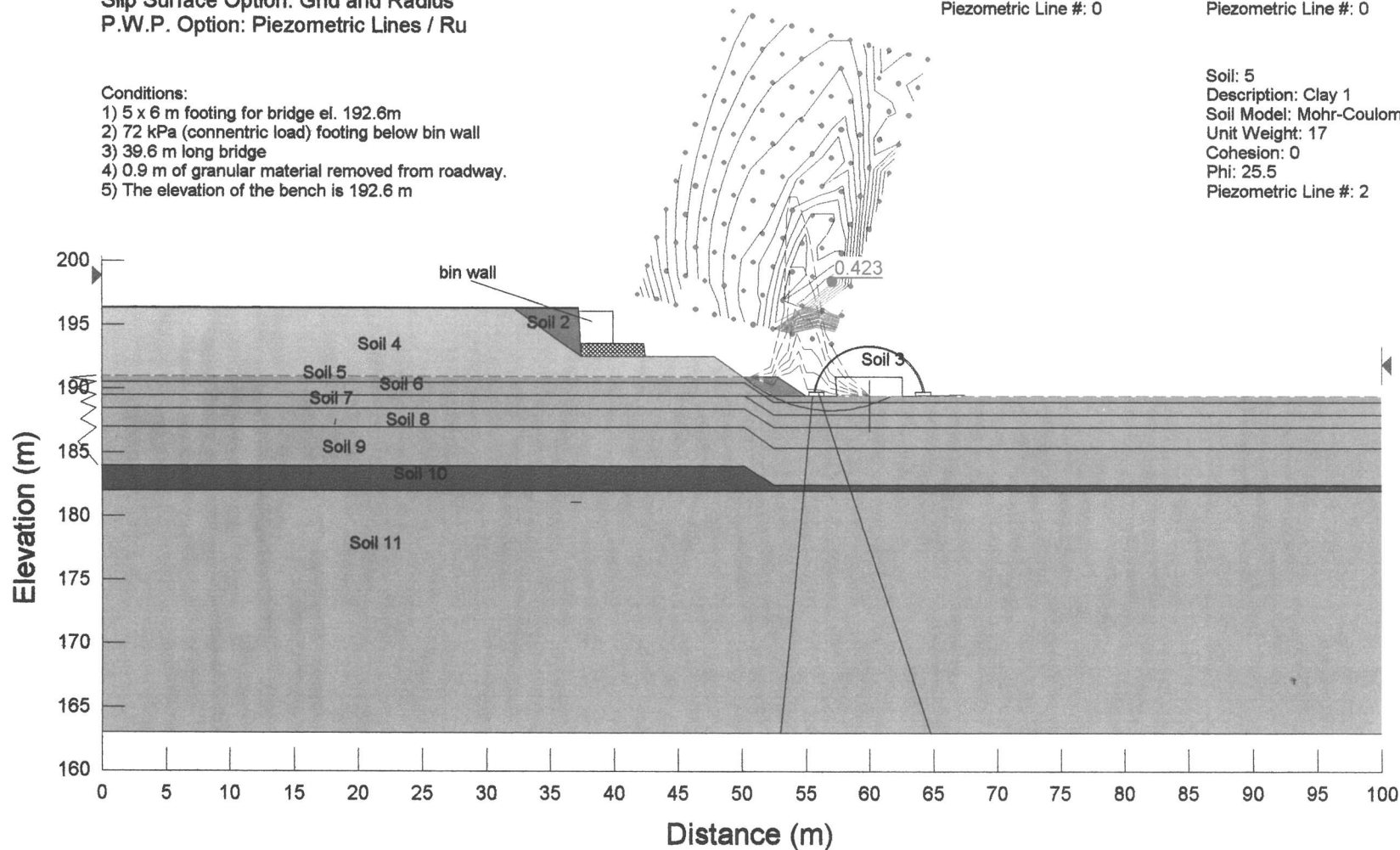
Soil: 5
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 2

Soil: 8
Description: Clay 4
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 5
Pore-Air Pressure: 0

Soil: 9
Description: Clay 5
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 6

Soil: 10
Description: Clay 6
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 7

Soil: 11
Description: Silt & Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

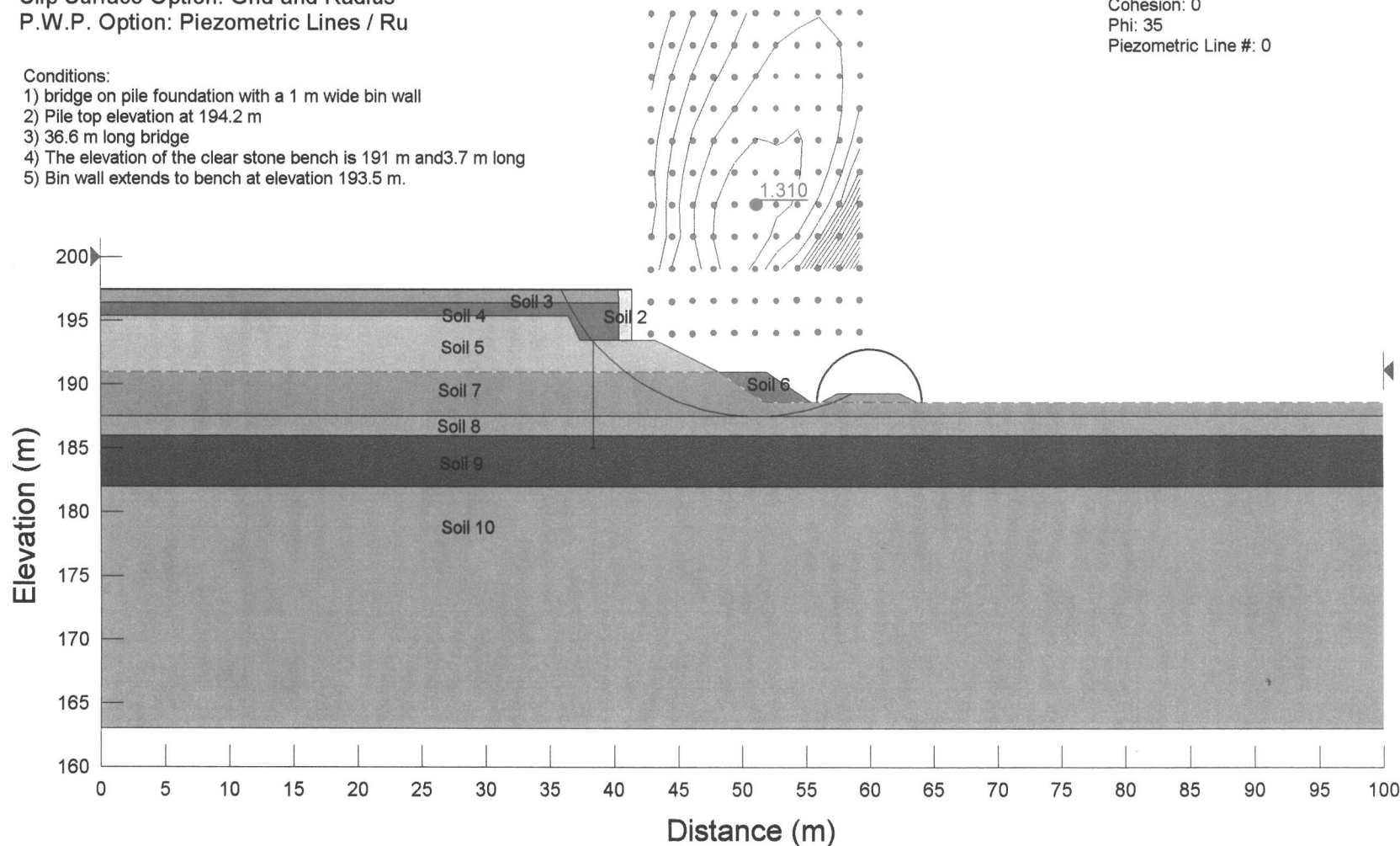


Proposed Conceptual Design of Stage 3 Construction Drained Analysis - Option 2

Description: McKellar Longnitudinal
File Name: dn-mcklr-short-confoot-revision-piles-1-x.slp
Last Saved Date: 08/01/03
Last Saved Time: 4:47:57 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:

- 1) bridge on pile foundation with a 1 m wide bin wall
- 2) Pile top elevation at 194.2 m
- 3) 36.6 m long bridge
- 4) The elevation of the clear stone bench is 191 m and 3.7 m long
- 5) Bin wall extends to bench at elevation 193.5 m.



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Cohesion: 0
Piezometric Line #: 0

Soil: 2
Description: Bin Wall
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 25
Phi: 35
Piezometric Line #: 0

Soil: 3
Description: Pavement Structure
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Plasti-Span
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 13
Piezometric Line #: 0

Soil: 5
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 6
Description: Clear Stone
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 7
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 8
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 9
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

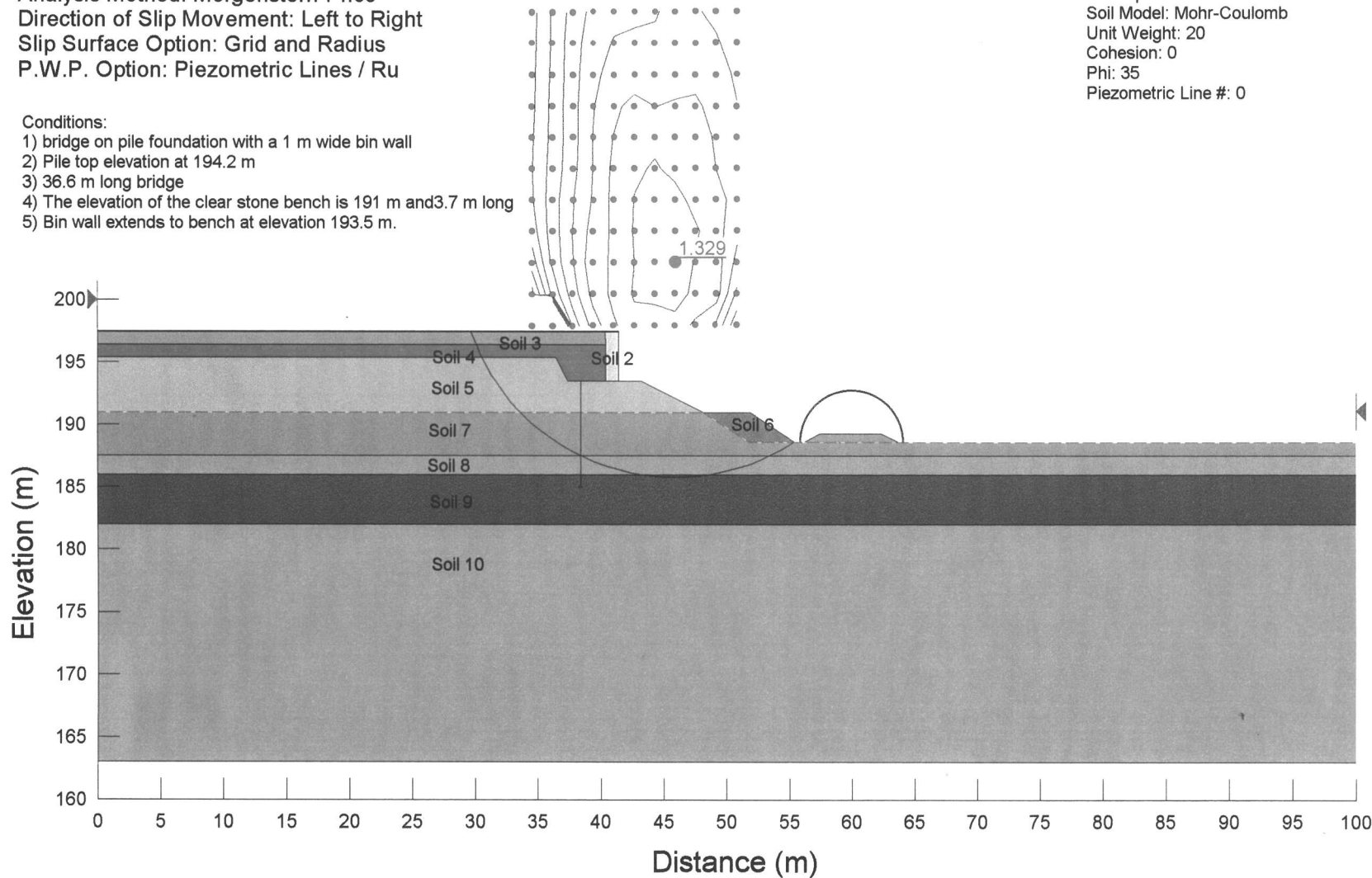
Soil: 10
Description: Silt & Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Proposed Conceptual Design of Stage 3 Construction Undrained Analysis - Option 2

Description: McKellar Longitudinal
File Name: un-mcklr-short-confoot-revision-piles-1-x.slp
Last Saved Date: 08/01/03
Last Saved Time: 4:52:20 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:

- 1) bridge on pile foundation with a 1 m wide bin wall
- 2) Pile top elevation at 194.2 m
- 3) 36.6 m long bridge
- 4) The elevation of the clear stone bench is 191 m and 3.7 m long
- 5) Bin wall extends to bench at elevation 193.5 m.



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Bin Wall
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 25
Phi: 35
Piezometric Line #: 0

Soil: 3
Description: Pavement Structure
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Plasti-Span
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 13
Piezometric Line #: 0

Soil: 5
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 6
Description: Clear Stone
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 7
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 24
Piezometric Line #: 1

Soil: 8
Description: Clay 2
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 32
Piezometric Line #: 1

Soil: 9
Description: Clay 3
Soil Model: S=f(depth)
Unit Weight: 17
C-Top of Layer: 32
Rate of Increase: 3.25
C - Maximum: 45
Piezometric Line #: 1

Soil: 10
Description: Silt & Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Proposed Conceptual Design of Stage 4 Construction Drained Analysis-Option 1

Description: McKellar Creek Station 14+720
File Name: dn-mcklr-final-14+720-Stage4-1-a-.slp
Last Saved Date: 05/07/02
Last Saved Time: 11:56:45 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:
1) Gabion Basket filled with rock.
2) Soil 2 represents the extents of the styrofoam fill required

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 3
Description: Light Weight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

Soil: 2
Description: Granular
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

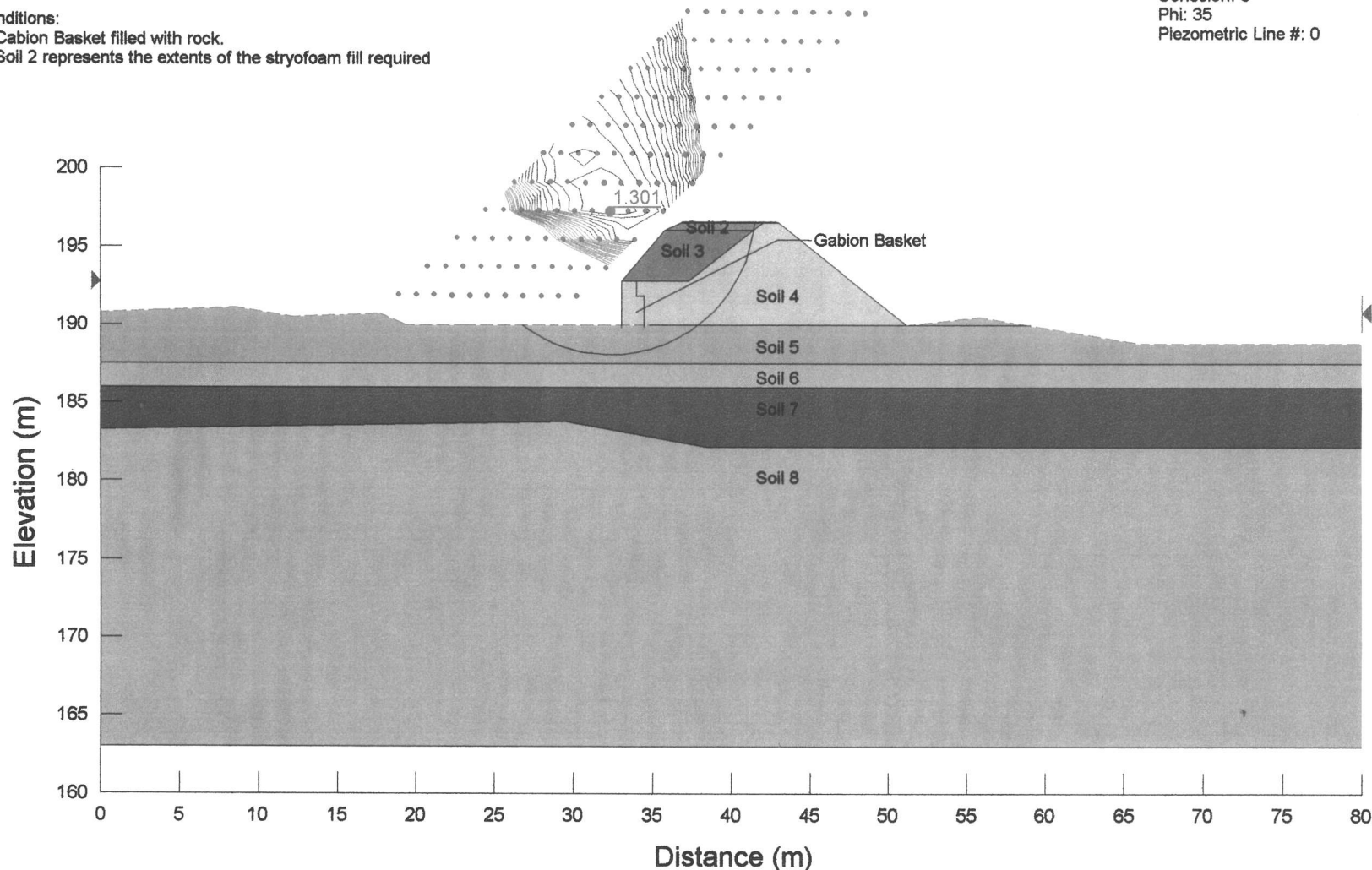
Soil: 4
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 5
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 6
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 7
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 8
Description: Sand & Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1



Proposed Conceptual Design Stage 4 Construction Undrained Analysis - Option 1

Description: McKellar Creek Station 14+720
File Name: un-mcklr-final-14+720-Stage4-1-b-.slp
Last Saved Date: 05/07/02
Last Saved Time: 4:47:38 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:
1) Gabion Basket filled with rock.
2) Soil 2 represents the extents of the lightweight fill required

Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Granular
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 0

Soil: 3
Description: Light Weight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

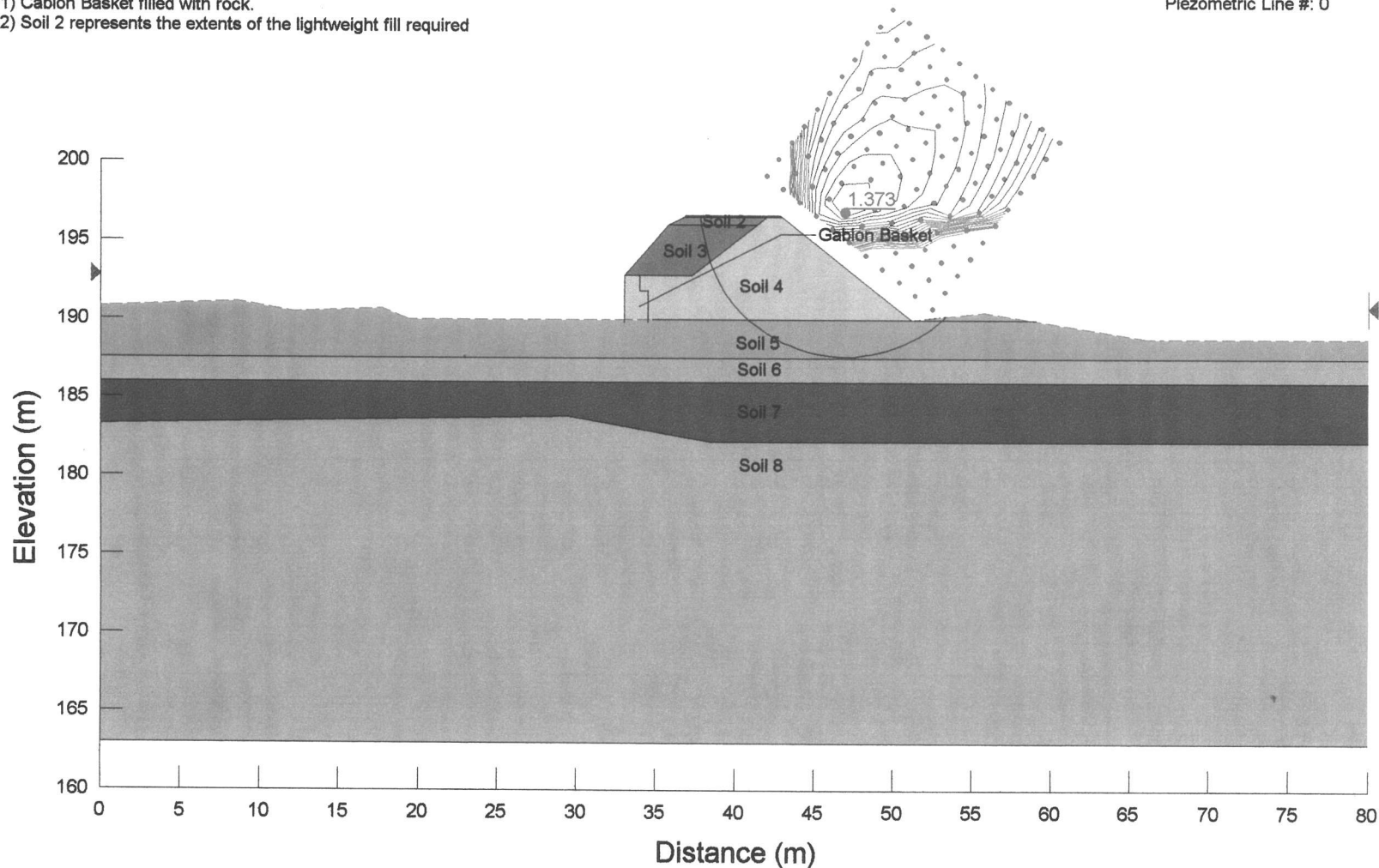
Soil: 4
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 5
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 24
Piezometric Line #: 1

Soil: 6
Description: Clay 2
Soil Model: Undrained (Phi=0)
Unit Weight: 17
Cohesion: 32
Piezometric Line #: 1

Soil: 7
Description: Clay 3
Soil Model: S=f(depth)
Unit Weight: 17
C-Top of Layer: 32
Rate of Increase: 3.25
C - Maximum: 45
Piezometric Line #: 1

Soil: 8
Description: Sand & Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

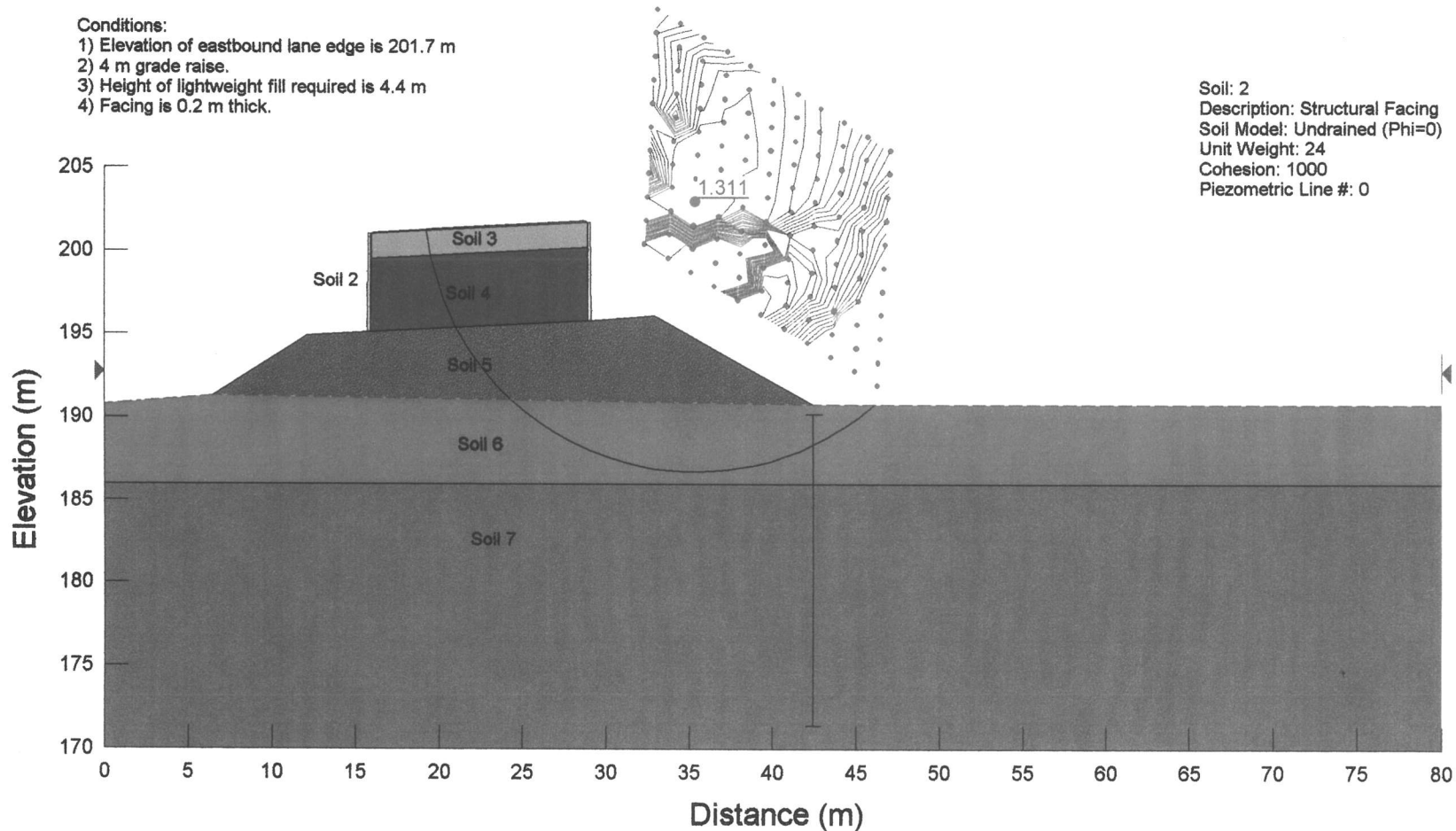


Proposed Conceptual Design of Stage 6 at Station 14+680 Undrained Analysis -Option 1

Description: Grade Raise for Station 14+680
File Name: un-Station-14+680-raise-1-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 1:11:27 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:

- 1) Elevation of eastbound lane edge is 201.7 m
- 2) 4 m grade raise.
- 3) Height of lightweight fill required is 4.4 m
- 4) Facing is 0.2 m thick.



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Structural Facing
Soil Model: Undrained (Phi=0)
Unit Weight: 24
Cohesion: 1000
Piezometric Line #: 0

Soil: 3
Description: Granular
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Lightweight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

Soil: 5
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 6
Description: Clay 1
Soil Model: S=f(depth)
Unit Weight: 18
C-Top of Layer: 23
Rate of Increase: 1.6
C - Maximum: 39
Piezometric Line #: 1

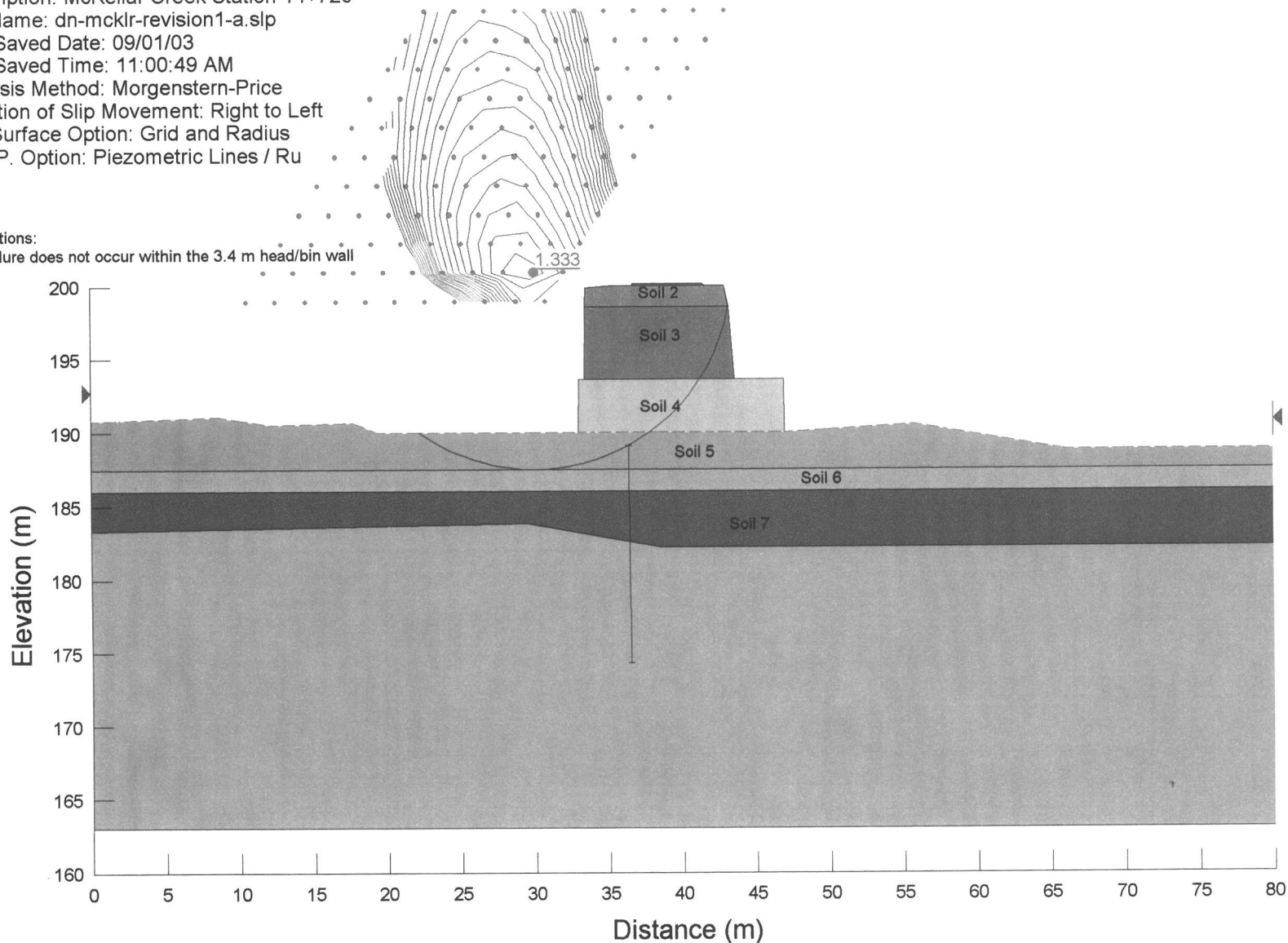
Soil: 7
Description: Sand and Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Proposed Conceptual Design of Stage 4 Construction Drained Analysis - Option 2

Description: McKellar Creek Station 14+720
File Name: dn-mcklr-revision1-a.slp
Last Saved Date: 09/01/03
Last Saved Time: 11:00:49 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Right to Left
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

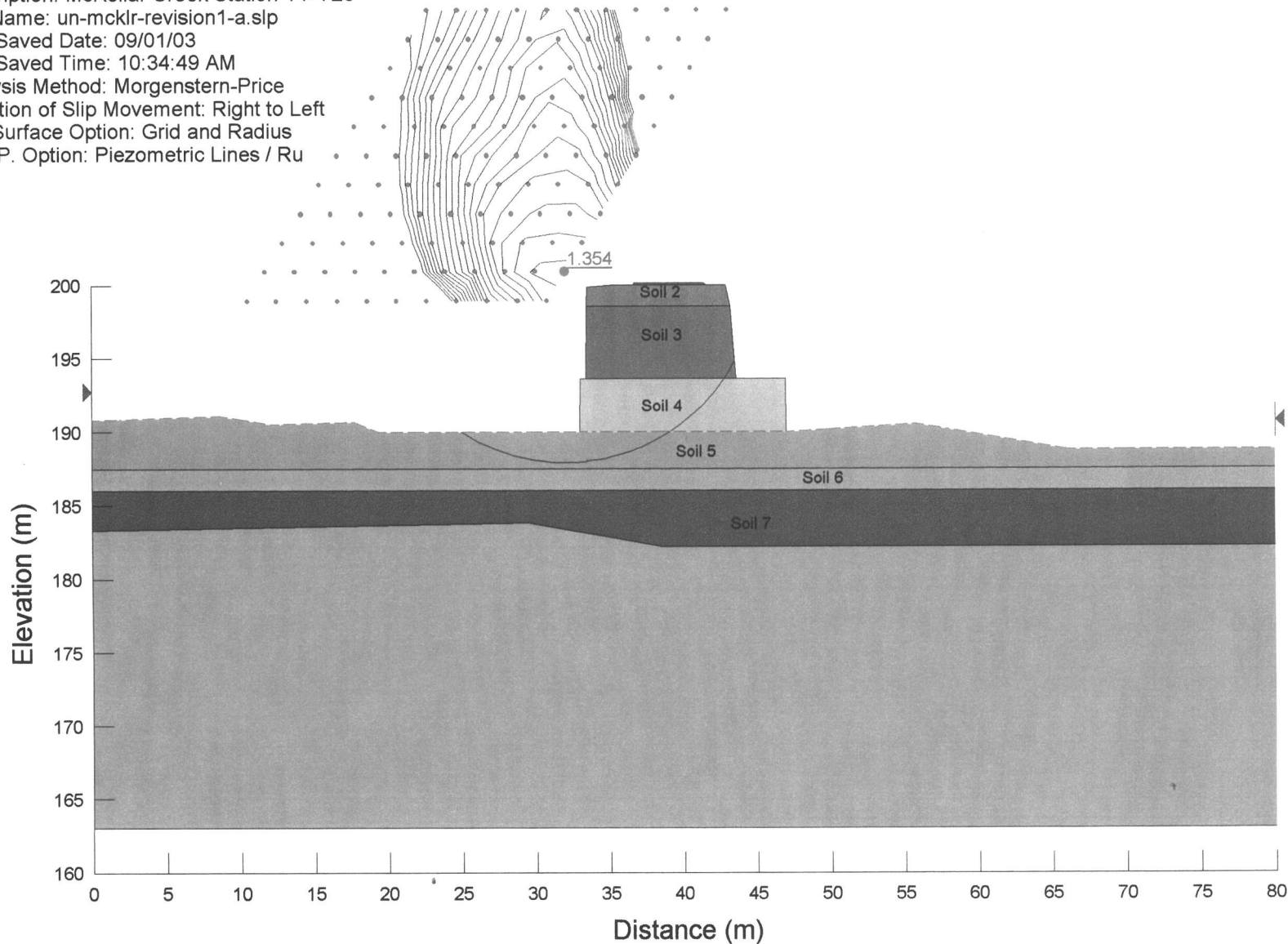
Conditions:

- 1) Failure does not occur within the 3.4 m head/bin wall



Proposed Conceptual Design of Stage 4 Construction Undrained Analysis - Option 2

Description: McKellar Creek Station 14+720
 File Name: un-mcklr-revision1-a.slp
 Last Saved Date: 09/01/03
 Last Saved Time: 10:34:49 AM
 Analysis Method: Morgenstern-Price
 Direction of Slip Movement: Right to Left
 Slip Surface Option: Grid and Radius
 P.W.P. Option: Piezometric Lines / Ru



Soil: 1
 Description: Traffic Load
 Soil Model: No Strength
 Unit Weight: 120
 Piezometric Line #: 0

Soil: 2
 Description: Granular
 Soil Model: Mohr-Coulomb
 Unit Weight: 20
 Cohesion: 0
 Phi: 32
 Piezometric Line #: 0

Soil: 3
 Description: Light Weight Fill
 Soil Model: Undrained (Phi=0)
 Unit Weight: 0.5
 Cohesion: 10
 Piezometric Line #: 0

Soil: 4
 Description: Rock Fill
 Soil Model: Mohr-Coulomb
 Unit Weight: 20
 Cohesion: 0
 Phi: 35
 Piezometric Line #: 0

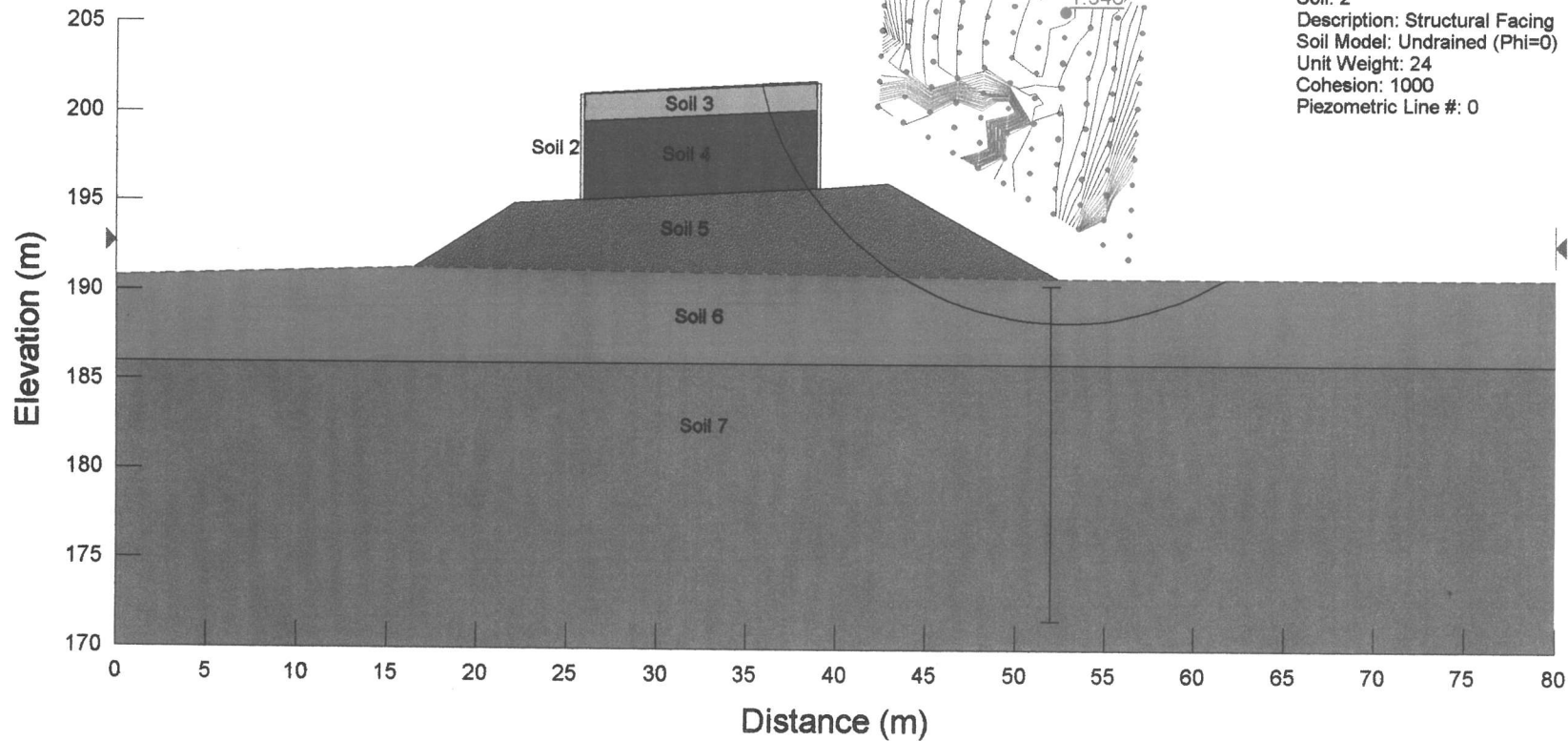
Soil: 5
 Description: Clay 1
 Soil Model: Undrained (Phi=0)
 Unit Weight: 17
 Cohesion: 24
 Piezometric Line #: 1

Soil: 6
 Description: Clay 2
 Soil Model: Undrained (Phi=0)
 Unit Weight: 17
 Cohesion: 32
 Piezometric Line #: 1

Soil: 7
 Description: Clay 3
 Soil Model: S=f(depth)
 Unit Weight: 17
 C-Top of Layer: 32
 Rate of Increase: 3.25
 C - Maximum: 45
 Piezometric Line #: 1

Proposed Conceptual Design of Stage 6 at Station 14+680 Drained Analysis-Option 1

Description: Grade Raise for Station 14+680
File Name: dn-Station-14+680-raise-1-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 2:01:33 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Structural Facing
Soil Model: Undrained (Phi=0)
Unit Weight: 24
Cohesion: 1000
Piezometric Line #: 0

Soil: 3
Description: Granular
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Lightweight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

Soil: 5
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

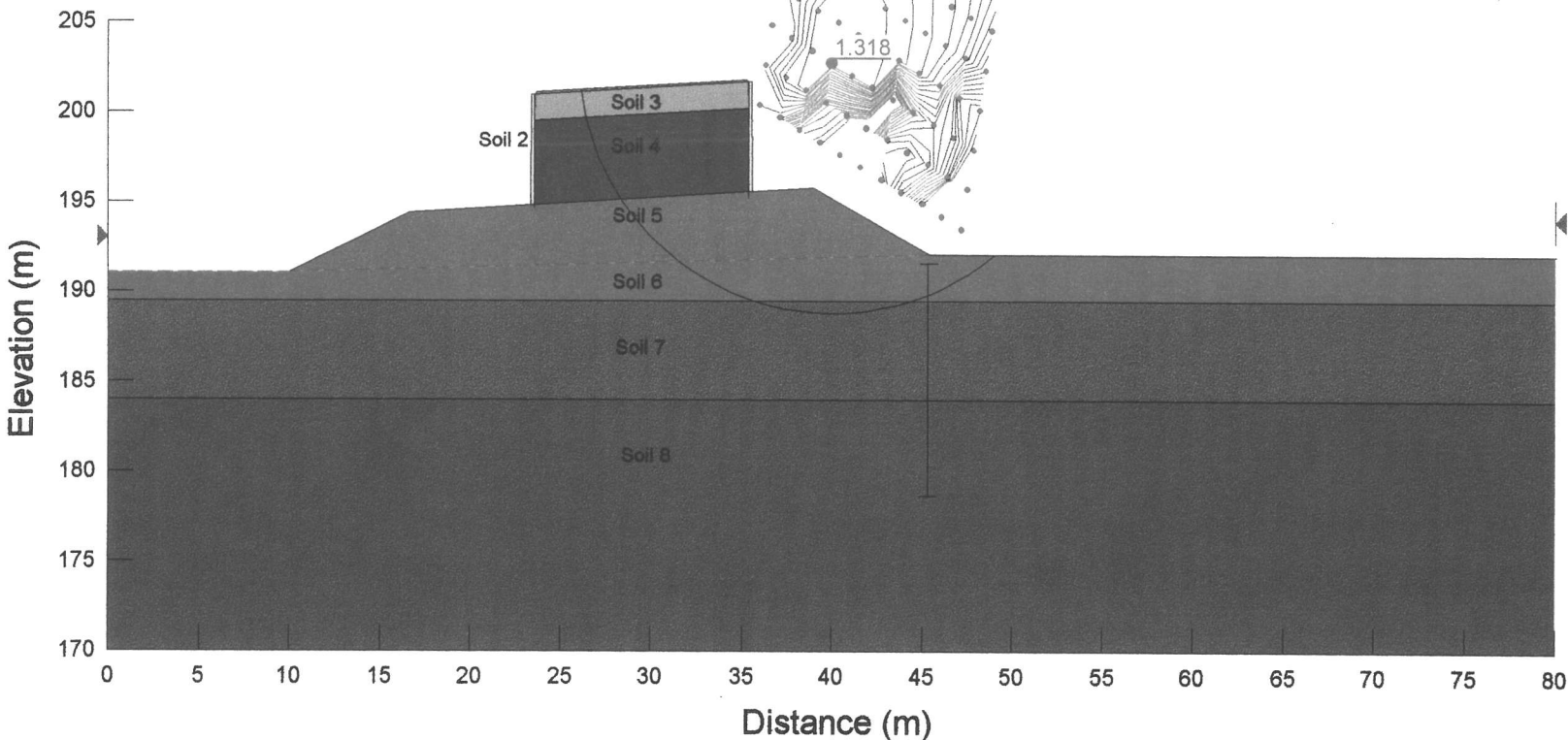
Soil: 6
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 7
Description: Sand and Silt
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Proposed Conceptual Design of Stage 6 at Station 14+750 Undrained Analysis -Option 1

Description: McKellar Creek Station 14+750
File Name: un-Station-14+750-raise-1-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 1:38:01 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:
1) Elevation of eastbound lane is 201.7 m
2) 4 m grade raise.
3) 4.6 m of lightweight fill is required.
4) The structural facing is 0.2 m thick.



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Structural Facing
Soil Model: Undrained (Phi=0)
Unit Weight: 24
Cohesion: 1000
Piezometric Line #: 0

Soil: 3
Description: Granular Fill
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Lightweight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

Soil: 5
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 6
Description: Clay 1
Soil Model: Undrained (Phi=0)
Unit Weight: 18
Cohesion: 20
Piezometric Line #: 1

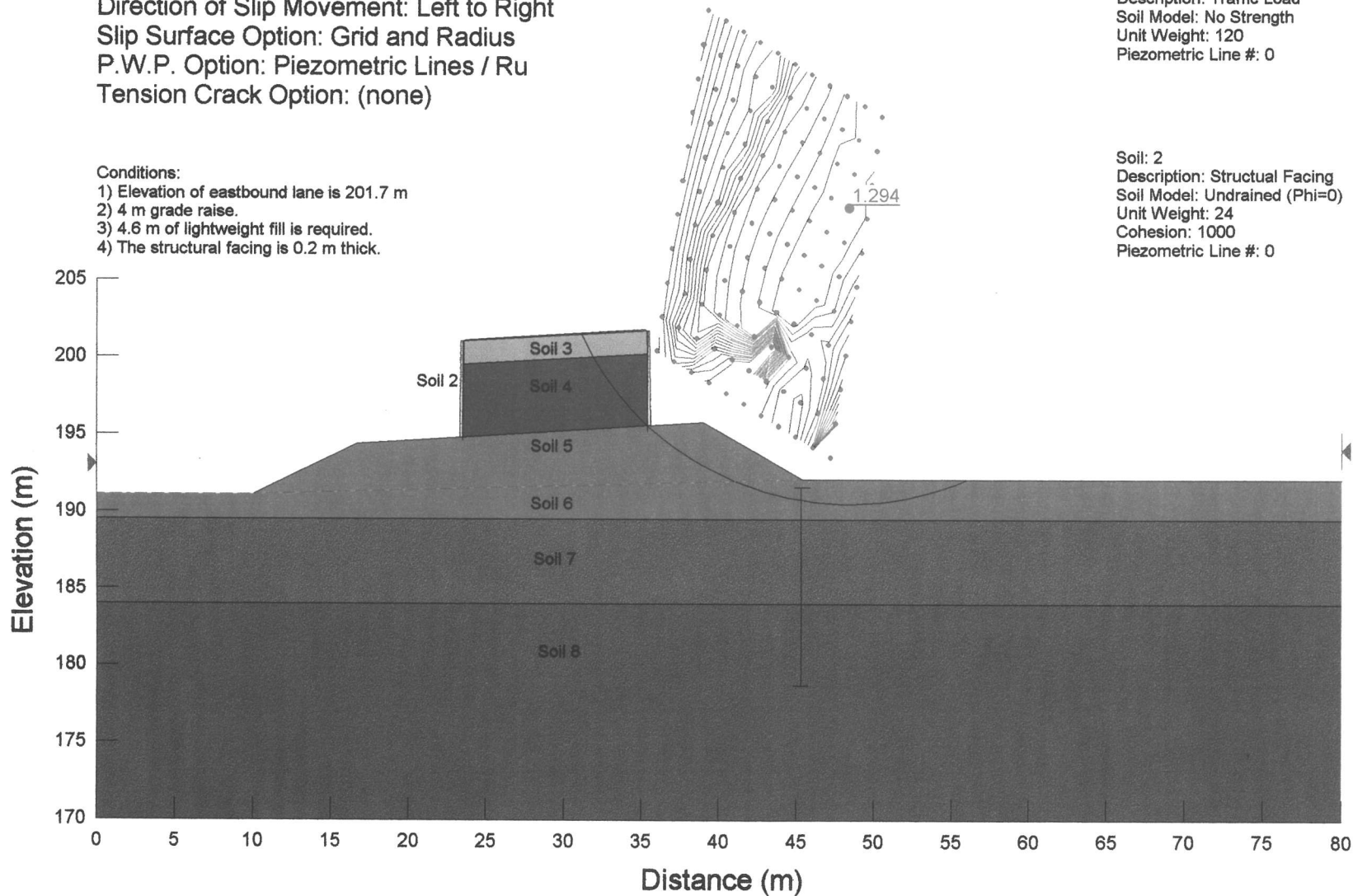
Soil: 7
Description: Clay 2
Soil Model: S=f(depth)
Unit Weight: 18
C-Top of Layer: 20
Rate of Increase: 5.6
C - Maximum: 50
Piezometric Line #: 1

Soil: 8
Description: Sand and Silts
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Proposed Conceptual Design of Stage 6 at Station 14+750 Drained Analysis -Option 1

Description: McKellar Creek Station 14+750
File Name: dn-Station-14+750-raise-1-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 2:13:20 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)

Conditions:
1) Elevation of eastbound lane is 201.7 m
2) 4 m grade raise.
3) 4.6 m of lightweight fill is required.
4) The structural facing is 0.2 m thick.



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Structural Facing
Soil Model: Undrained (Phi=0)
Unit Weight: 24
Cohesion: 1000
Piezometric Line #: 0

Soil: 3
Description: Granular Fill
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 4
Description: Lightweight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

Soil: 5
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 6
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

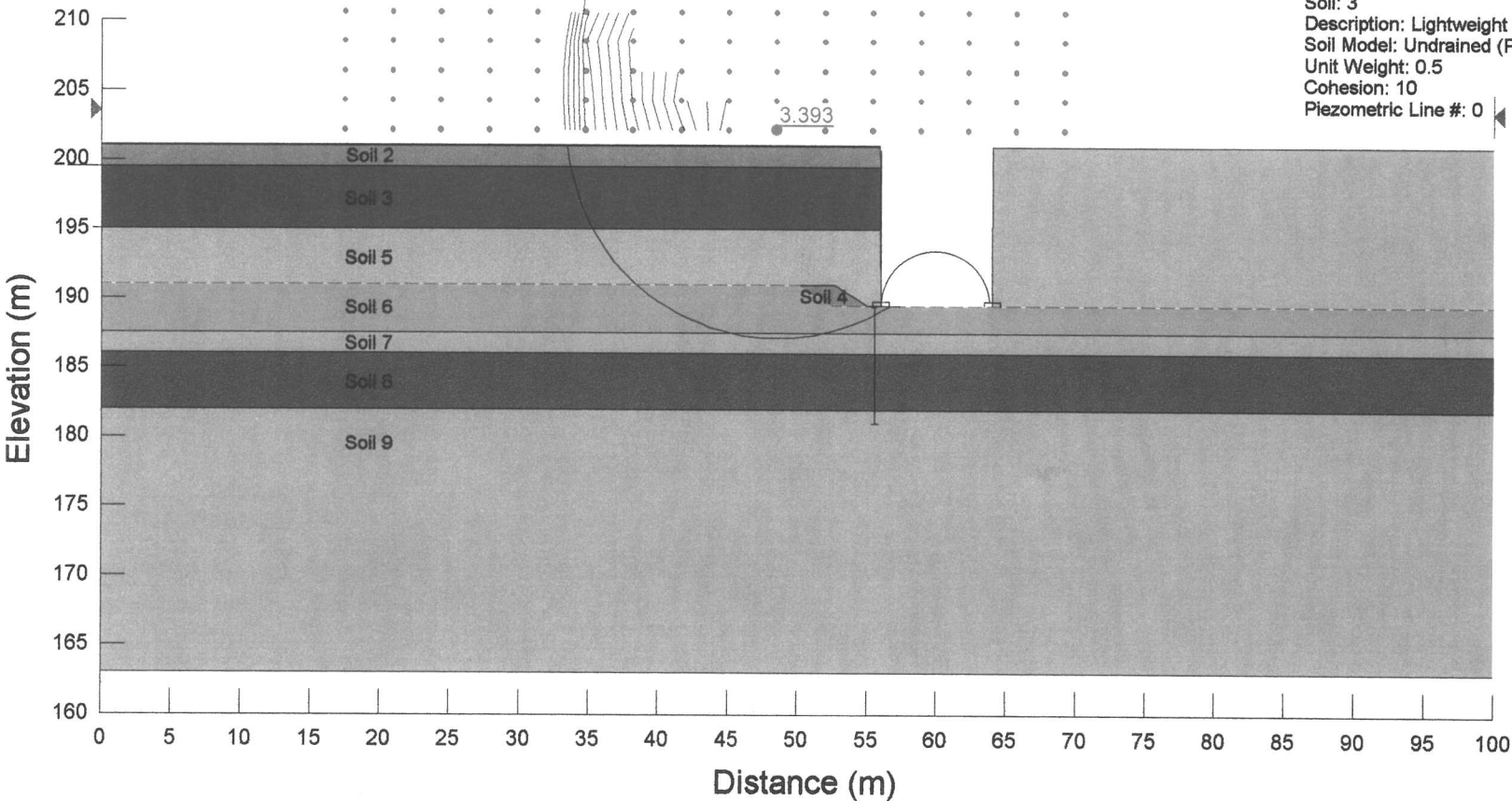
Soil: 7
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 8
Description: Sand and Silts
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

Failure Beneath Pile Cap at Elevation 189.5 m Long Term Dained Analysis-Option 1

Description: McKellar Longitudinal
File Name: dn-mcklr-long-eccfoot-8mculvert-1-a.slp
Last Saved Date: 05/07/02
Last Saved Time: 3:28:05 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru

Conditions:
1) Elevation at the bottom of the pile cap is 189.5 m
2) The culvert is 8 m wide
3) The grade raise is 4 m above original grade
4) At least 4.5 m of lightweight fill is used.



Soil: 1
Description: Traffic Load
Soil Model: No Strength
Unit Weight: 120
Piezometric Line #: 0

Soil: 2
Description: Granular
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 3
Description: Lightweight Fill
Soil Model: Undrained (Phi=0)
Unit Weight: 0.5
Cohesion: 10
Piezometric Line #: 0

Soil: 4
Description: Rock Fill
Soil Model: Mohr-Coulomb
Unit Weight: 18
Cohesion: 0
Phi: 45
Piezometric Line #: 0

Soil: 5
Description: Clear Stone
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 35
Piezometric Line #: 0

Soil: 6
Description: Clay 1
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 7
Description: Clay 2
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1

Soil: 8
Description: Clay 3
Soil Model: Mohr-Coulomb
Unit Weight: 17
Cohesion: 0
Phi: 25.5
Piezometric Line #: 1
Pore-Air Pressure: 0

Soil: 9
Description: Silt & Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 32
Piezometric Line #: 1

A P P E N D I X 'D'

**BOREHOLE LOGS - FOUNDATION INVESTIGATION &
DESIGN REPORT- MTO 1996**

RECORD OF BOREHOLE No 95-1

1 OF 1

METRIC

W.P. 185-87-01(C) LOCATION Sta. 14+700.8 o/s 19 m RT. of Centreline Hwy. 17 ORIGINATED BY M.M.
DIST 61 HWY 17 BOREHOLE TYPE H.S. Auger, Rock Coring COMPILED BY M.M.
DATUM Geodetic DATE 1995 02 28 CHECKED BY J.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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							WATER CONTENT (%)		
								20 40 60 80 100		20 40 60 80 100		25 50 75					
191.1	Ground Surface																
0.0	Organics						190						17.4	0 0 25 75			
	Silty Clay to Clay Soft to Firm		1	SS	5												
			2	TW	PH		188										
			3	SS	1												
			4	SS	1												
			5	TW	PH		186						17.2	0 0 24 76			
			6	TW	PH												
184.0							184										
7.1	Silty Sand Trace Clay Very Loose to Very Dense		7	SS	3												
			8	SS	3		182										
			9	SS	17		180										
			10	SS	10		178										
			11	SS	13		176										
			12	SS	8		174							0 78 19 3			
			13	SS	7		172										
			14	SS	6												
171.0			15	SS	120												
20.1	Bedrock Schist		16	RC	REC 100%		170							ROD 80%			
169.5			17	RC	REC 100%	100%								ROD 100%			
21.6	End of Borehole 'N' values within the Silty Sand deposit may be questionable due to blow up and disturbance during sampling.																

RECORD OF BOREHOLE No 95-2

1 OF 1

METRIC

W.P. 185-87-01(G) LOCATION Sta. 14+702.3 e/s 24.8m Lt. of Centreline Hwy. 17 ORIGINATED BY M.M.
 DIST 81 HWY 17 BOREHOLE TYPE H.S. Auger, Rock Coring COMPILED BY M.M.
 DATUM Geodetic DATE 1995 03 01 CHECKED BY T.K.

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	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
190.8	Ground Surface												
0.0	Organics												
	Silty Clay to Clay Soft to Stiff	1	SS	2									
		2	TW	PM									
		3	SS	2									
		4	TW	PM									
183.7													
7.1	Silty Sand Trace Clay Very Loose to Dense	5	SS	3									
		6	SS	36									
		7	SS	5									
		8	SS	13									
		9	SS	7									
		10	SS	8									
		11	SS	5									
		12	SS	12									
		13	SS	23									
	Rock Fragments	14	SS	42									
168.8													
22.0	Bedrock Meta-Volcanic	15	RC	REC 100%									
167.6													
23.2	End of Borehole "N" values within the Silty Sand deposit may be question- able due to blow up and disturbance during sampling.												

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

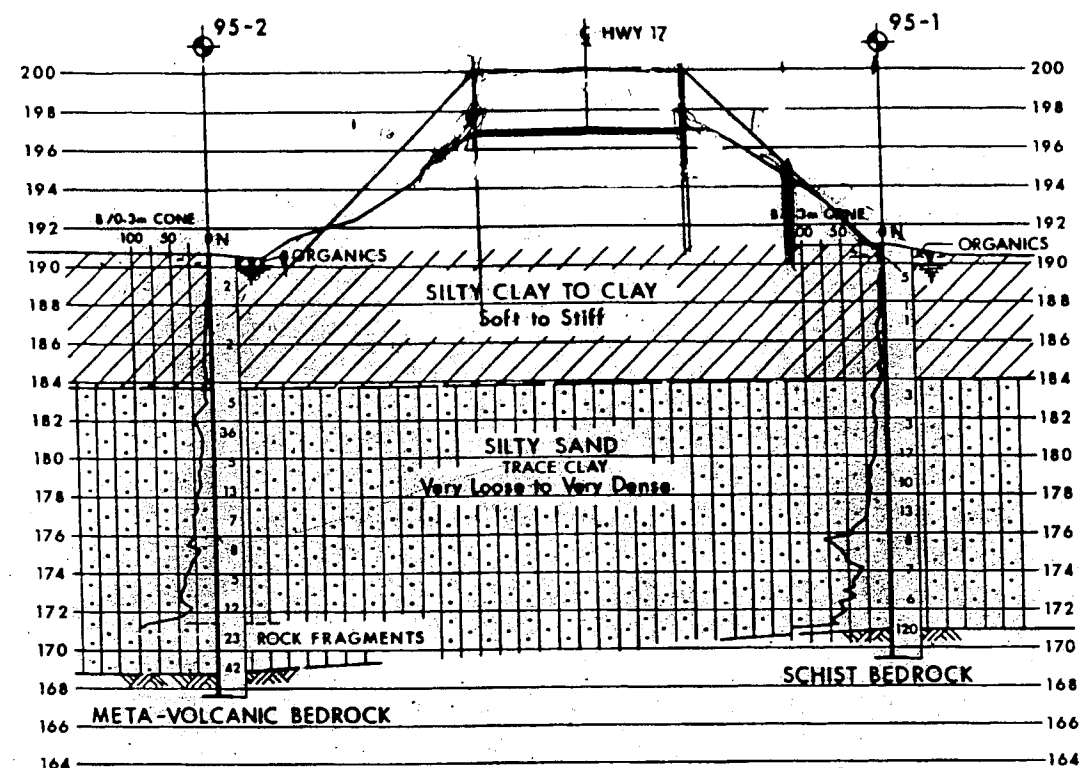
CONT No
WP No 195-87-01(C)

McKELLAR CREEK

BORE HOLE LOCATIONS & SOIL STRATA

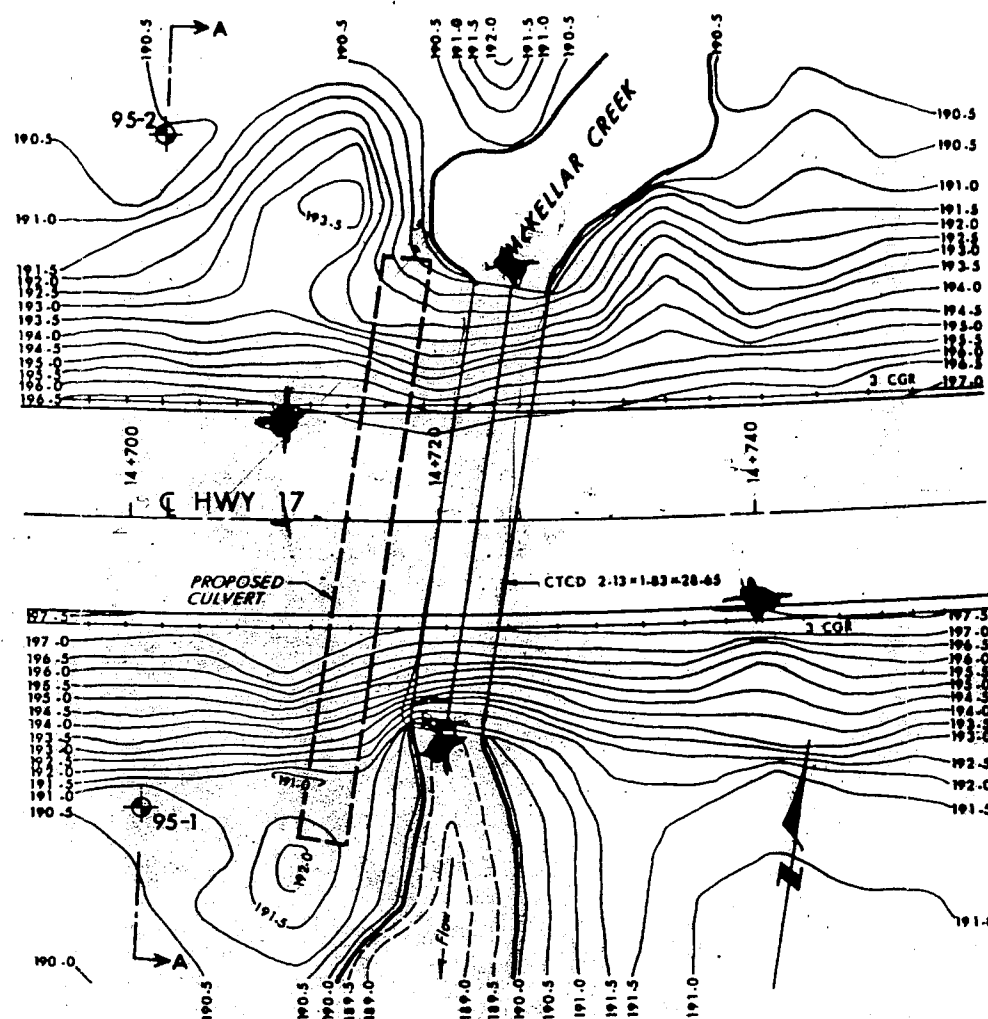


SHEET



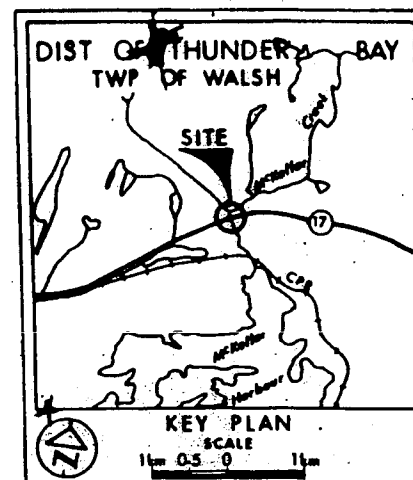
SECTION A-A

SCALE
5m 0 5m Hor
4m 0 4m Vert



PLAN

SCALE
5m 0 5m



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation 1995 02 and 03

No	ELEVATION	STATION	OFFSET
95-1	191.1	14+700.8	19.0m RT
95-2	190.8	14+702.3	24.8m LT

NOTE

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

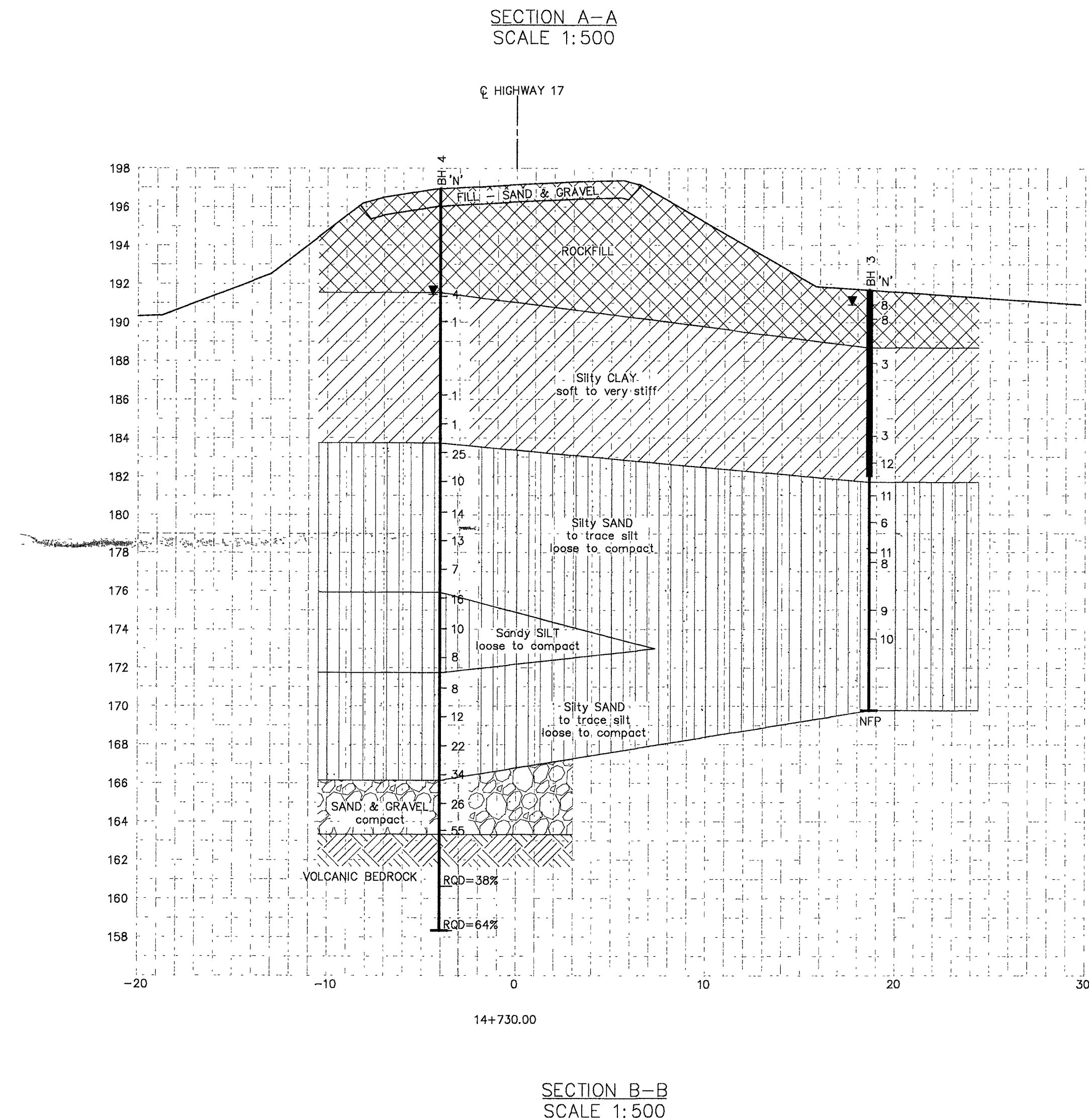
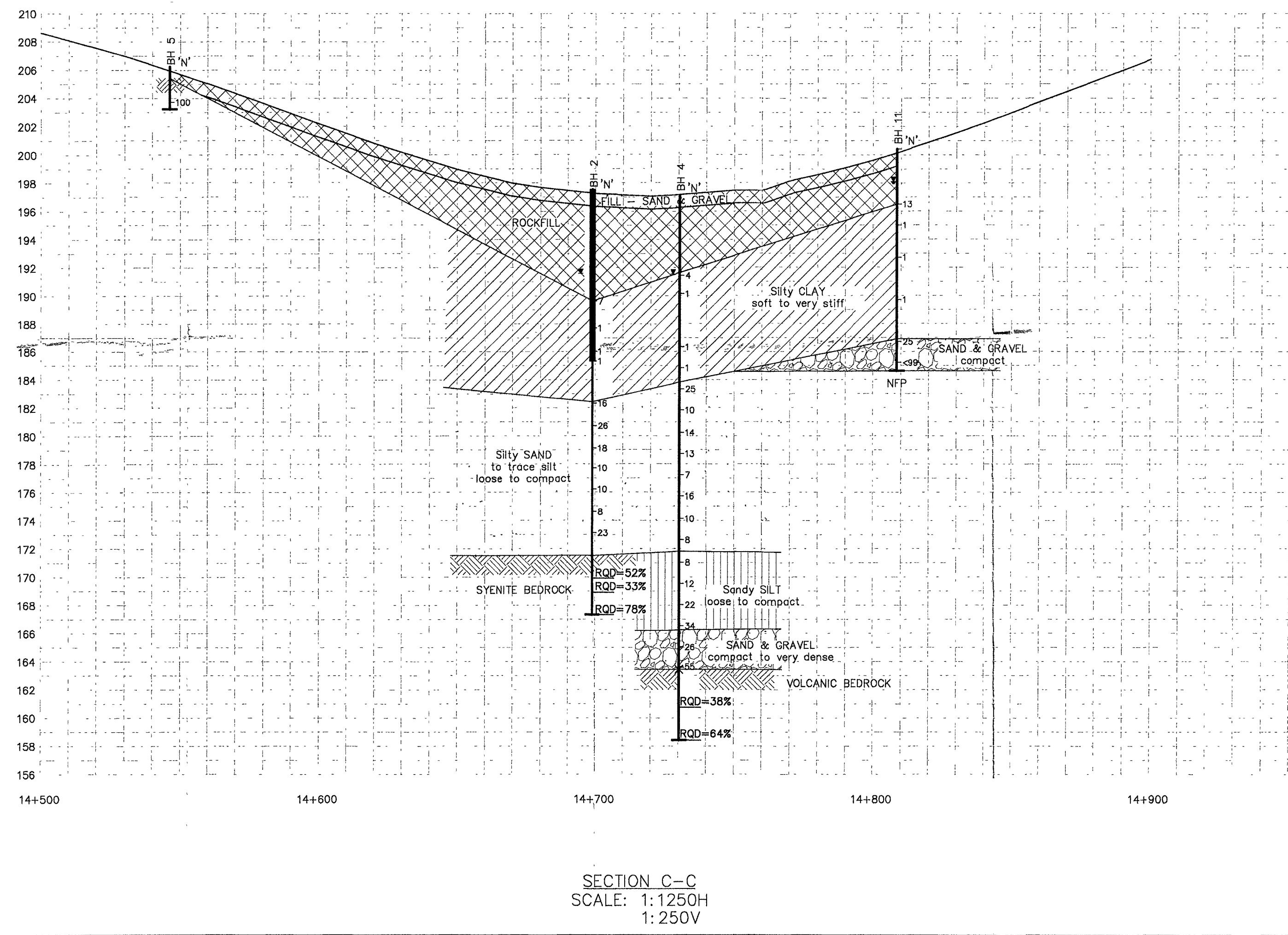
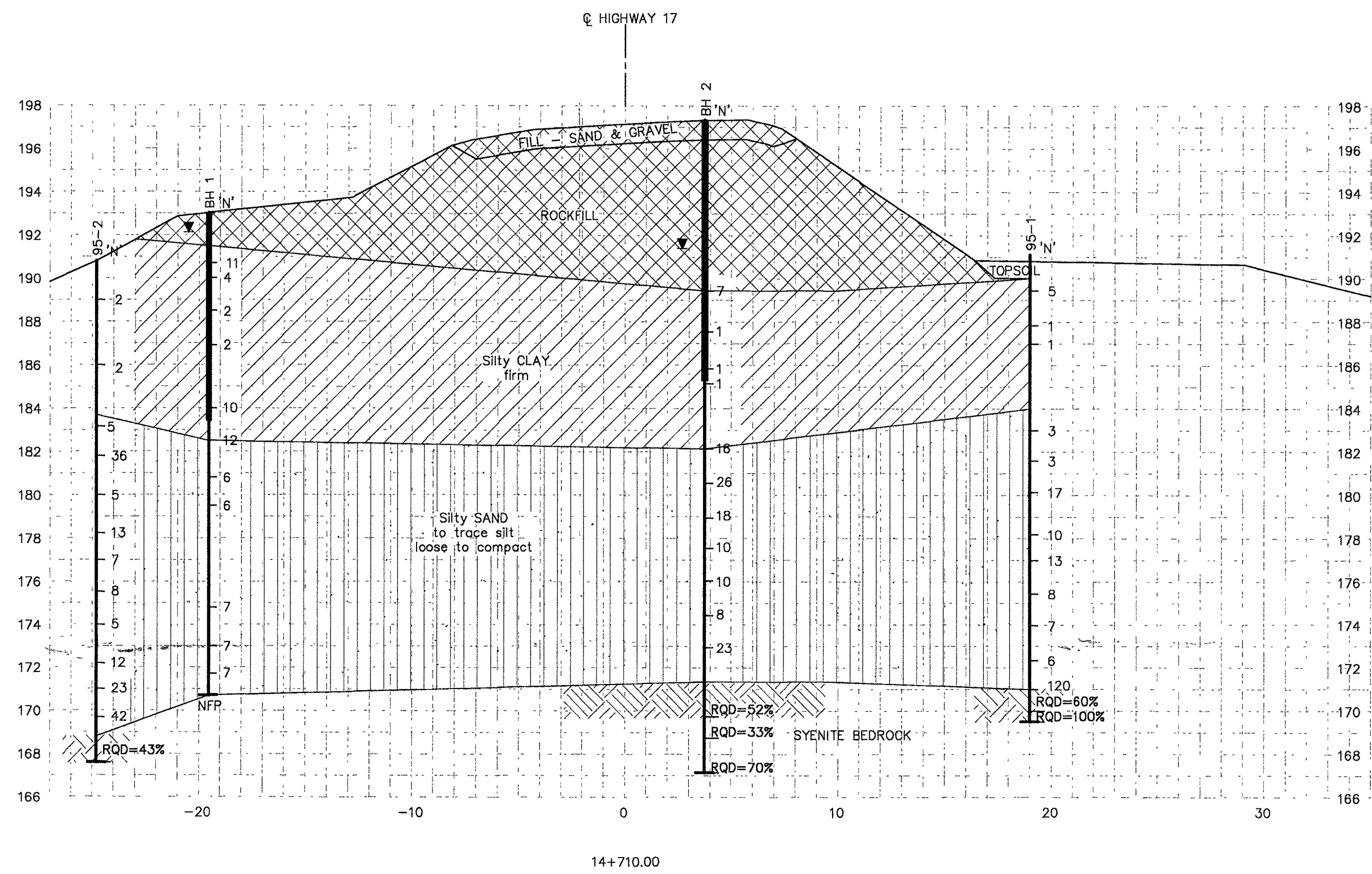
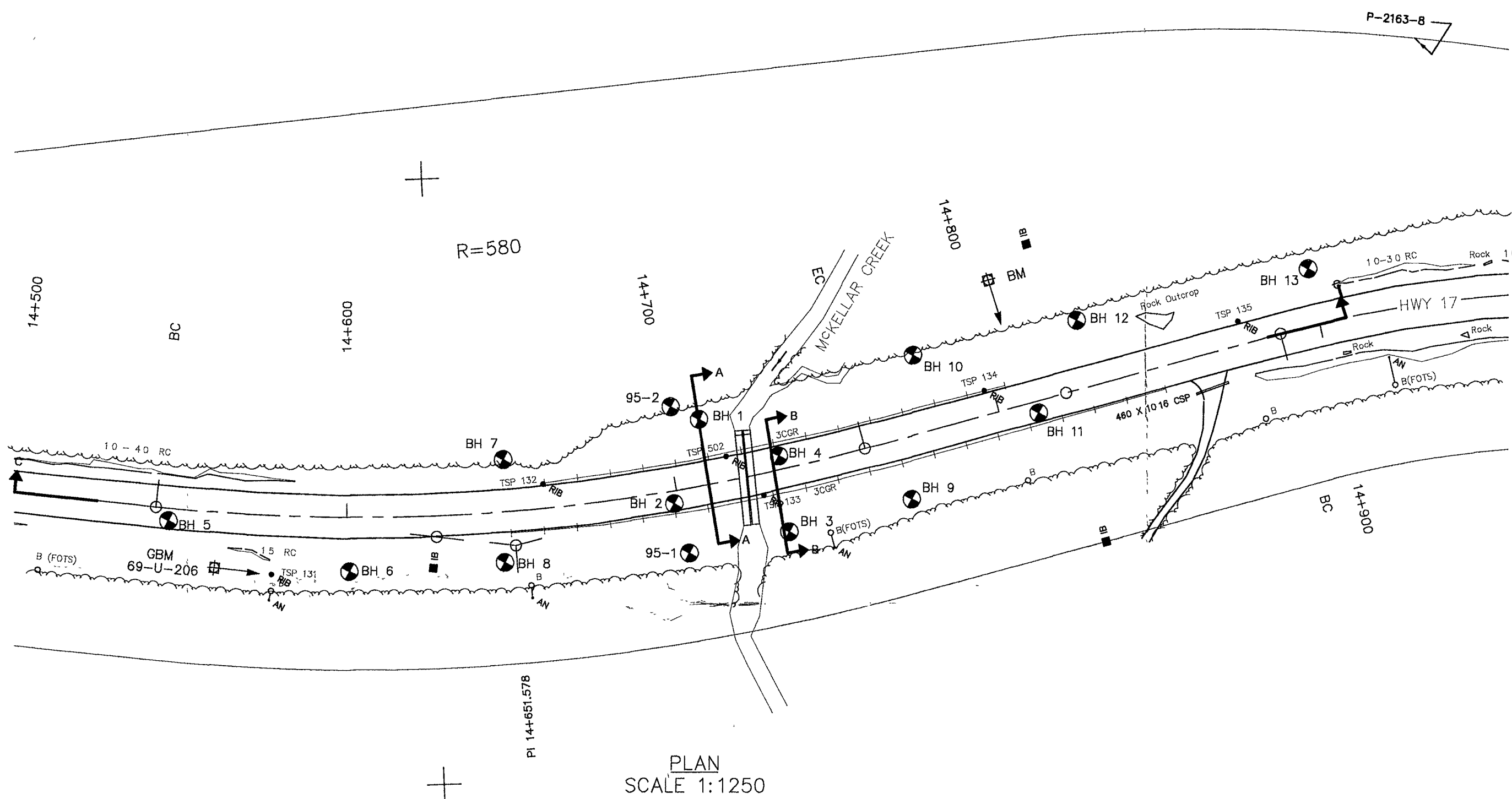
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically included in accordance with the conditions of Section GC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

DATE	BY	DESCRIPTION

Geocore No 420-21
HWY No 17

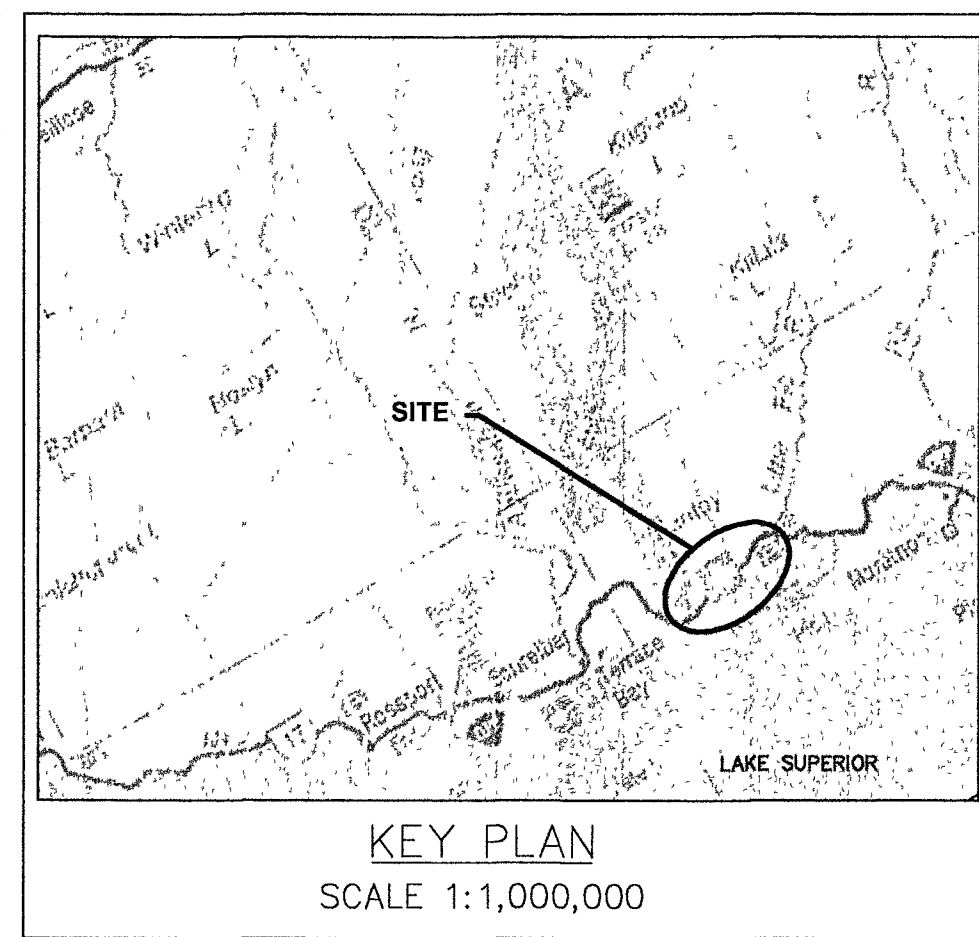


DRAWING



CONT No
WP No 194-87-00
McKELLAR CREEK
14+500 TO 14+900
BOREHOLE LOCATIONS & SOIL STRATA

DST
 CONSULTING ENGINEERS
 DST Consulting Engineers Inc.
 605 Hewitson Street
 Thunder Bay, ON P7B 5V5
 Ph: (807) 623-2929
 Fx: (807) 623-1792
 Email: thunderbay@dstgroup.com



- LEGEND**
- Borehole
 - 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
 - ▼ Water level at time of investigation.
 - NFP No Further Progress
 - █ Standpipe
 - ▨ Fill
 - ▧ Silt
 - ▩ Clay
 - ░ Sand
 - ▤ Sand & Gravel

No.	Elevation	Station	Offset
1	192.9	14+710.6	19.5 Lt
2	197.5	14+698.7	3.7 Rt
3	192.0	14+730.7	18.6 Rt
4	197.0	14+732.3	4.1 Lt
5	206.2	14+546.5	3.6 Rt
6	199.7	14+600.9	16.0 Rt
7	198.0	14+648.3	15.4 Lt
8	198.5	14+646.6	15.0 Rt
9	192.6	14+767.8	18.3 Rt
10	192.2	14+779.3	22.9 Lt
11	200.4	14+811.3	3.5 Rt
12	201.5	14+829.5	20.0 Lt
13	207.5	14+900.3	16.7 Lt
95-1	191.1	14+700.8	19.0 Rt
95-2	190.8	14+702.3	24.8 Lt

—NOTE—
 The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

REVISION	DATE	BY	DESCRIPTION

HWY No 17
 SUBM'D RC/CHECKED GMD/DATE JULY 2002
 DRAWN TG/CHECKED RC/DATE JULY 2002
 DIST 61
 SITE 48E-4bc
 DWG 1

ENCLOSURES

RECORD OF BOREHOLE No 1

2 OF 2

METRIC

W.P. 194-87-00 LOCATION 14+710.6 o/s 19.5m Rt - Walsh Township ORIGINATED BY PR
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 29.04.02 CHECKED BY SS

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					
20.0	SAND - some to trace silt, trace gravel, loose		15	SS	7								
			16	SS	7								
170.6						172							
22.3	End of Borehole @ 22.3m. Auger Refusal.					171							

RECORD OF BOREHOLE No 2										1 OF 2		METRIC		
W.P. 194-87-00		LOCATION 14+898.7 o/s 3.7m Rt - Walsh Township				ORIGINATED BY AF								
DIST 61 HWY 17		BOREHOLE TYPE Wash Boreing				COMPILED BY TG								
DATUM Geodetic		DATE 29.04.02				CHECKED BY SS								
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
197.5														
0.0	FILL - SAND & GRAVEL													Steel casing installed to 7.9m - no sampling. Water level @ 5.95m on June 19, 2002.
	- ROCKFILL													
189.6														Standpipe installed to 12.2m.
7.9	CLAY - Silty, brown, firm		1	SS	7									
	- grey		2	TW										
			3	TW										
			4	SS	1									
			5	TW										
	- layered		6	SS	1									
			7	SS	1									
	- thin silty sand seams, stiff to very stiff		8	TW										
182.3														
15.2	SAND - Silty, grey, compact		9	SS	16									
			10	SS	26									
	- layered		11	SS	18									
177.5														

ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

Continued Next Page

✕, ✱ 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

2 OF 2

METRIC

W.P. 194-87-00 LOCATION 14+898.7 o/s 3.7m Rt - Walsh Township ORIGINATED BY AF
DIST 61 HWY 17 BOREHOLE TYPE Wash Borehole COMPILED BY TG
DATUM Geodetic DATE 29.04.02 CHECKED BY SS

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					
						20 40 60 80 100	20 40 60 80 100						
20.0	SAND - Silty, grey, loose to compact		12	SS	10								
			13	SS	10								
			14	SS	8								
			15	SS	23								
171.5	----- - occasional cobble												
26.0	BEDROCK SYENITE - pale red, high strength		1	RC									RC1 RQD = 52% REC = 85%
			2	RC									RC2 RQD = 33% REC = 92%
			3	RC									RC3 RQD = 70% REC = 100%
167.3	End of Borehole @ 30.2m.												
30.2													

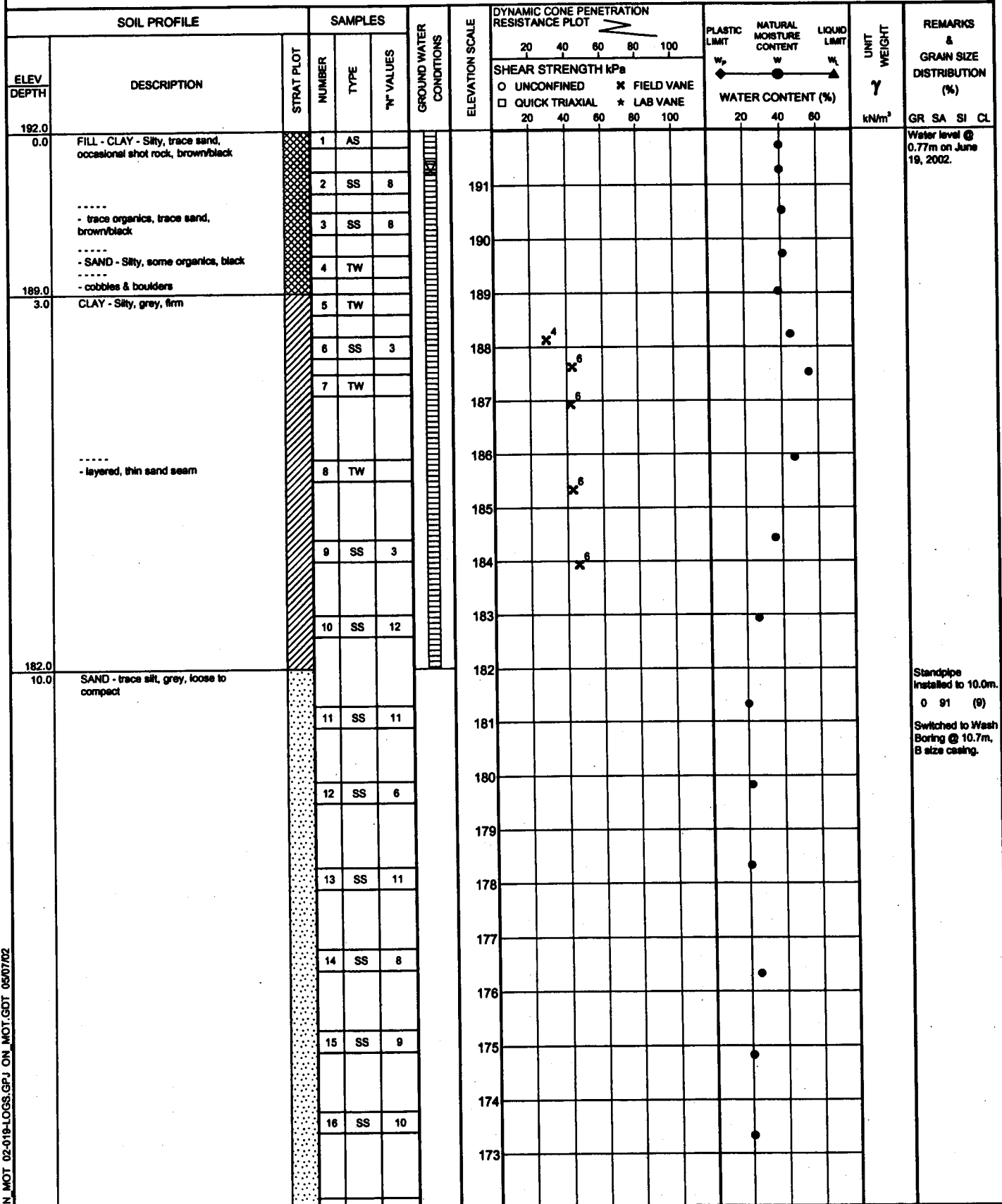
ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

RECORD OF BOREHOLE No 3

1 OF 2

METRIC

W.P. 194-87-00 LOCATION 14+730.7 o/s 18.8m Rt - Walsh Township ORIGINATED BY PR
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 03.05.02 CHECKED BY SS



ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

Continued Next Page

×³, *³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 3										2 OF 2		METRIC	
W.P. 194-87-00		LOCATION 14+730.7 o/s 18.6m Rt - Welsh Township				ORIGINATED BY PR							
DIST 61 HWY 17		BOREHOLE TYPE Hollow Stem Auger				COMPILED BY TG							
DATUM Geodetic		DATE 03.05.02				CHECKED BY SS							
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					
			17	WS									
	----- - cobbles & boulders		18	WS	101								
170.1						171							
21.9	End of Borehole @ 21.9m. Refusal to bi-cons.												Bloomed 21.3m to 21.9m.

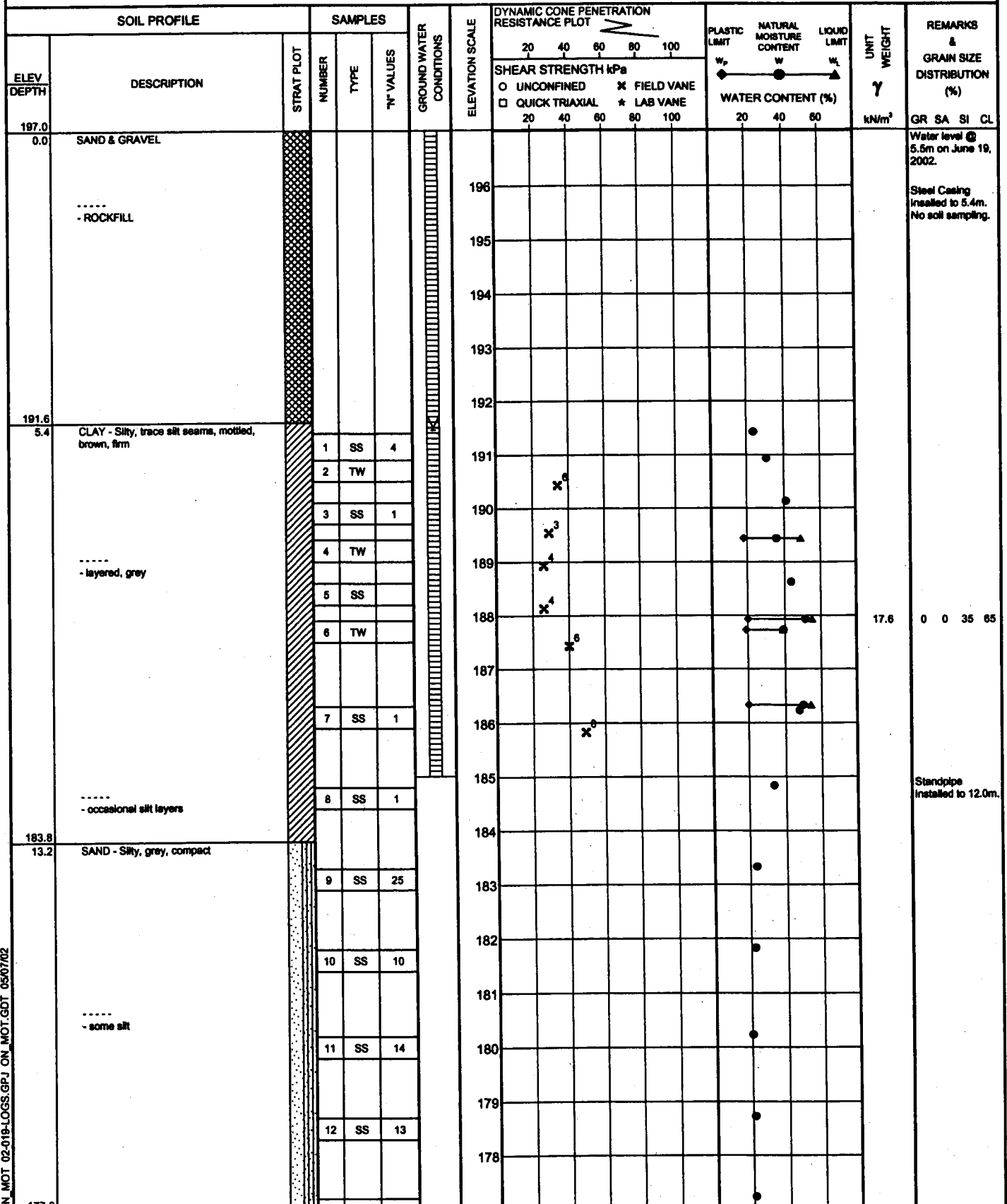
ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

RECORD OF BOREHOLE No 4

1 OF 2

METRIC

W.P. 194-87-00 LOCATION 14+732.3 o/s 4.1m Lt - Walsh Township ORIGINATED BY AF
DIST 61 HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY TG
DATUM Geodetic DATE 30.04.02 CHECKED BY SS



RECORD OF BOREHOLE No 4

2 OF 2

METRIC

W.P. 194-87-00 LOCATION 14+732.3 o/s 4.1m Lt - Walsh Township ORIGINATED BY AF
DIST 61 HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY TG
DATUM Geodetic DATE 30.04.02 CHECKED BY SS

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
20.0	SAND - some silt, brown, loose		13	SS	7									
176.0														
21.0	SILT - Sandy, grey, loose to compact		14	SS	16									
			15	SS	10									
			16	SS	8									
171.8														
25.2	SAND - Silty, grey, loose to compact		17	SS	8									
			18	SS	12									
			19	SS	22									
166.2			20	SS	34									
30.8	SAND & GRAVEL - compact to very dense													
			21	SS	26									
163.4	- Silty, grey		22	SS	55									
33.6	BEDROCK - Volcanic, pale brown, medium strength													
			1	RC										
			2	RC										
158.4														
38.6	End of Borehole @ 38.6m.													

ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

✕³, ✕⁴: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 5										1 OF 1		METRIC		
W.P. 194-87-00		LOCATION 14+546.5 o/s 3.8m Rt - Walsh Township				ORIGINATED BY AF								
DIST 61 HWY 17		BOREHOLE TYPE Hollow Stem Auger				COMPILED BY TG								
DATUM Geodetic		DATE				CHECKED BY SS								
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 20 40 60 80 100 ○ UNCONFINED × FIELD VANE □ QUICK TRIAXIAL * LAB VANE						
206.2 0.0	ASPHALT													Dry on June 19, 2002.
205.4 0.8	- FILL - SAND & GRAVEL													
203.2 3.0	BEDROCK													
203.2 3.0	End of Borehole @ 3.0m.													

ON MOT 02-018-LOGS.GPJ ON MOT.GDT 05/07/02

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 194-87-00 LOCATION 14+800.9 o/s 16.1m Rt - Walsh Township ORIGINATED BY PR
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 08.05.02 CHECKED BY SS

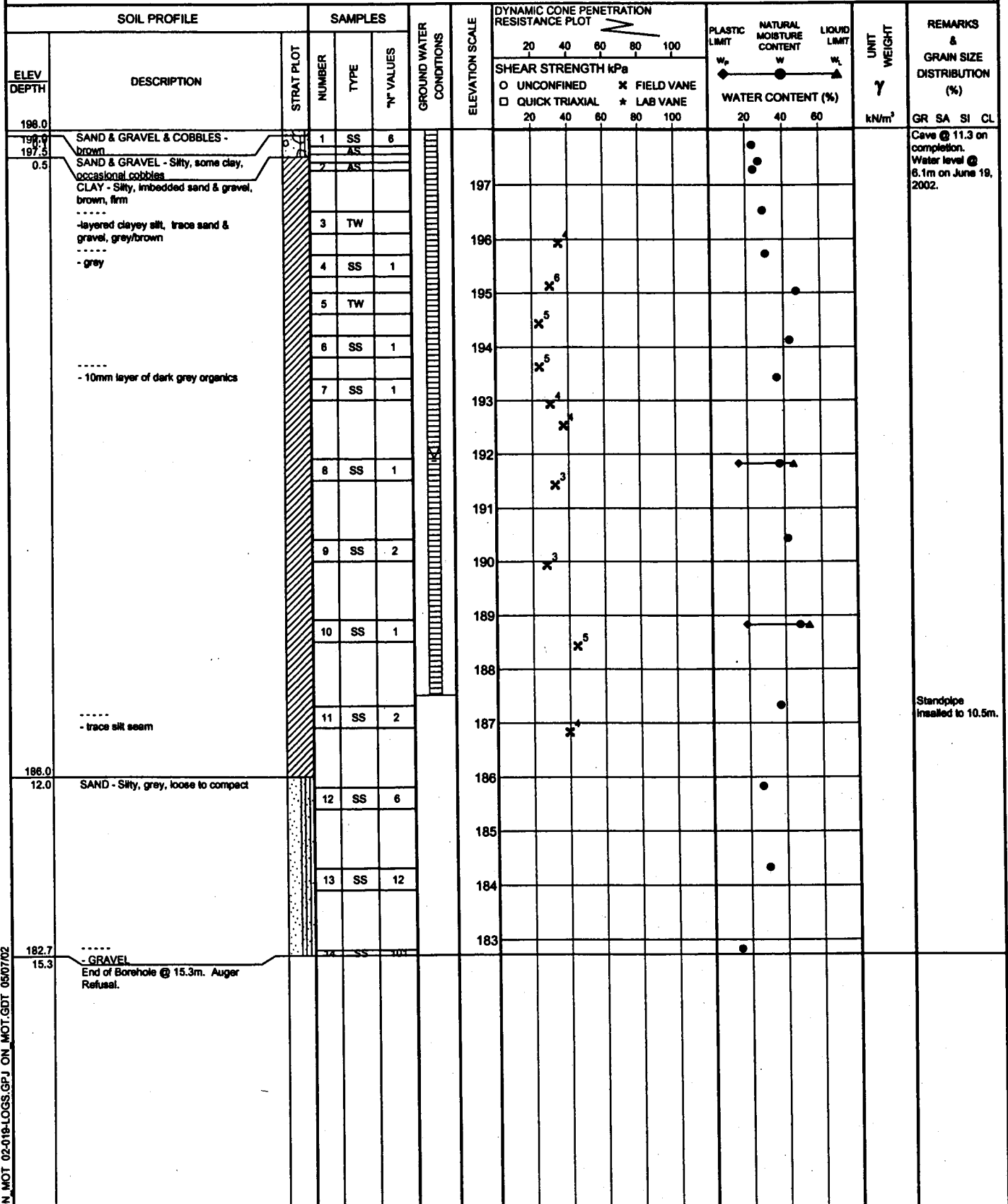
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	T _N VALUES								
199.7	TOPSOIL		1	AS									Water level @ 1.5m on May 14, 2002.
199.9	ORGANICS - black		2	AS									
198.3	SAND - some gravel, trace silt, occasional cobble, brown, very loose		3	SS	3								
196.5	CLAY - Silty, grey/brown, firm		4	SS	2								
			5	TW									
			6	SS	3								
195.9	SAND - some silt, brown, loose		7	SS	12								
193.6	- occasional cobbles		8	SS	7								
6.1	End of Borehole @ 6.1m. Auger Refusal.												Standpipe installed to 5.0m.

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 194-87-00 LOCATION 14+848.3 o/s 15.4m Lt - Walsh Township ORIGINATED BY AF
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 24.04.02 CHECKED BY SS



1 OF 1

METRIC

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)	
								UNCONFINED ○							FIELD VANE ×
198.5	TOPSOIL		1	AS									Water level @ 6.2m on June 16, 2002.		
198.4	FILL - SAND - some gravel, trace silt, brown		2	SS	9										
197.5	SILT - brown		3	SS	2										
1.0	CLAY - Silty, brown/gray, stiff		4	SS	2										
	- firm		5	TW											
	- layered		6	SS	3										
			7	TW											
			8	TW											
	- occasional silt & sand layers		9	SS	2										
190.1	SAND - some silt, grey, loose		10	SS	6										
8.4			11	SS	2										
			12	SS	7										
185.6			13	SS	101										
12.9	End of Borehole @ 12.9m. Auger Refusal.												Standpipe installed to 7.5m		

ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

✕³, ★³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No 10

1 OF 1

METRIC

W.P. 194-87-00 LOCATION 14+779.3 o/s 22.9m Lt - Walsh Township ORIGINATED BY PR
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 07.05.02 CHECKED BY SS

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 (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								20 40 60 80 100							20 40 60	
								20 40 60 80 100							20 40 60	
192.2																
0.0	FILL - CLAY - Silty, occasional shot rock & cobbles, some topsoil, brown		1	AS			192							Water level @ 0.9m on June 19, 2002.		
191.5																
0.7	CLAY - Silty, brown/gray, stiff to very stiff		2	SS	12		191							Standpipe Installed to 4.6m.		
			3	SS	3		190									
	----- - 8m		4	TW			189									
			5	SS	3		188									
			6	SS	3		187									
187.8	SAND - trace silt, grey, loose to compact		7	SS	4		186									
4.4			8	SS	7		185									
	----- - trace gravel, trace silt		9	SS	36		184									
			10	SS	100		183									
	----- - occasional cobble, very dense															
182.6	End of Borehole @ 9.6m.															
9.6																

N MOT 02-019-LOGS.GPJ ON MOT.GDT 06/07/02

x³, *³: Numbers refer to
Sensitivity

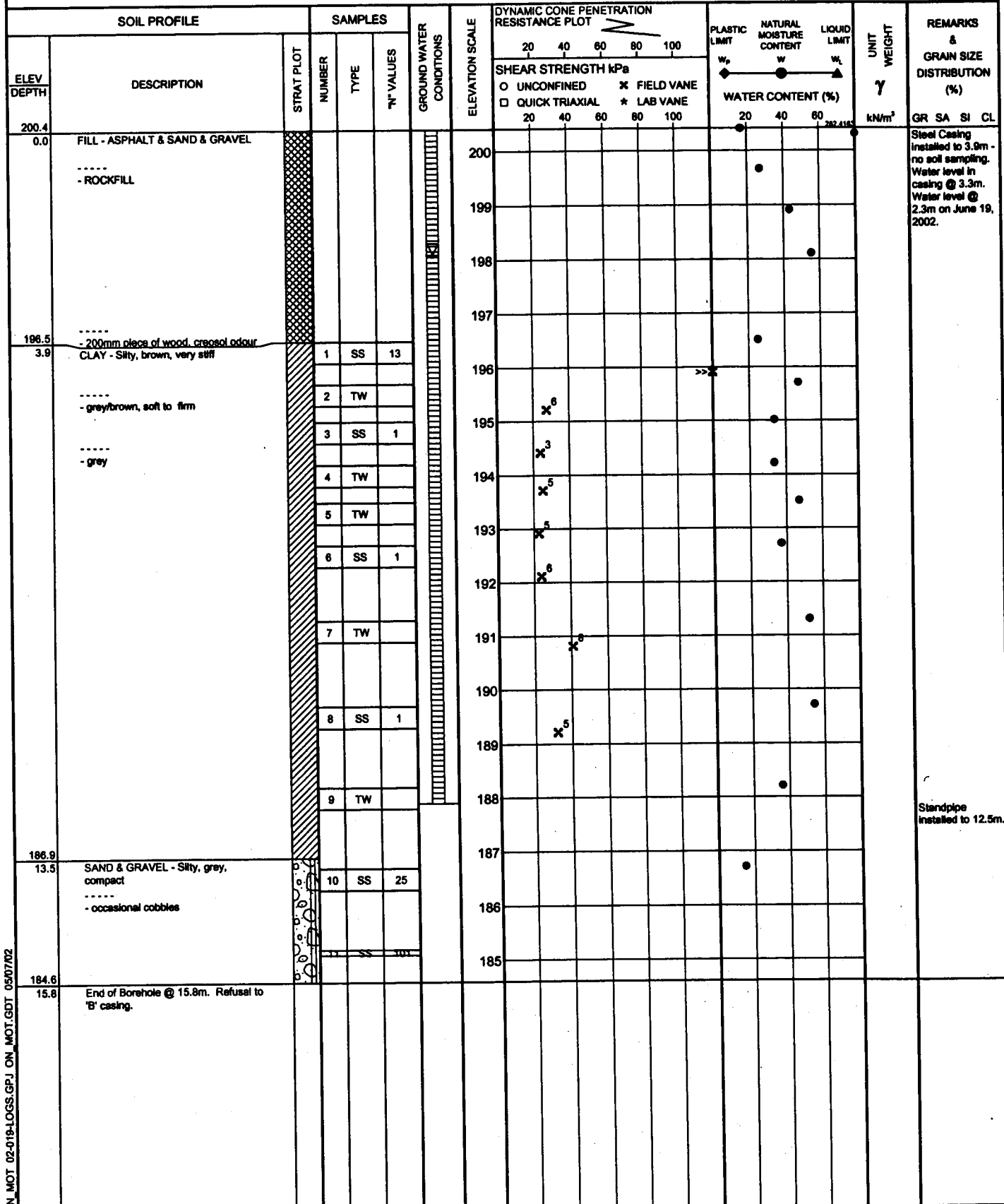
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 11

1 OF 1

METRIC

W.P. 194-87-00 LOCATION 14+811.3 o/s 3.5m Rt - Walsh Township ORIGINATED BY AF
DIST 61 HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY TG
DATUM Geodetic DATE 25.04.02 CHECKED BY SS



RECORD OF BOREHOLE No 12

1 OF 1

METRIC

W.P. 194-87-00 LOCATION 14+829.5 o/s 20.0m Lt - Walsh Township ORIGINATED BY AF
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 25.04.02 CHECKED BY SS

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	WATER CONTENT (%)					
201.5														
200.8	PEAT & ORGANICS - trace sand, trace gravel, trace cobbles, brown		1	SS										Cave @ 5.2m. Water level @ 4.1m on completion. Dry on June 19, 2002.
0.3	SAND & GRAVEL & COBBLES - Silty, brown, loose		2	SS	5									
200.6	CLAY - Silty, brown, soft to firm		3	TW										
0.9			4	SS	1									
	- grey/brown		5	TW										
	- grey		6	SS	1									
			7	SS	2									
	- occasional silt seam		8	AS										
196.2													Standpipe installed to 5m.	
196.8	SAND - Silty, some gravel, brown													
5.6	End of Borehole @ 5.6m. Auger Refusal.													

RECORD OF BOREHOLE No 13

1 OF 1

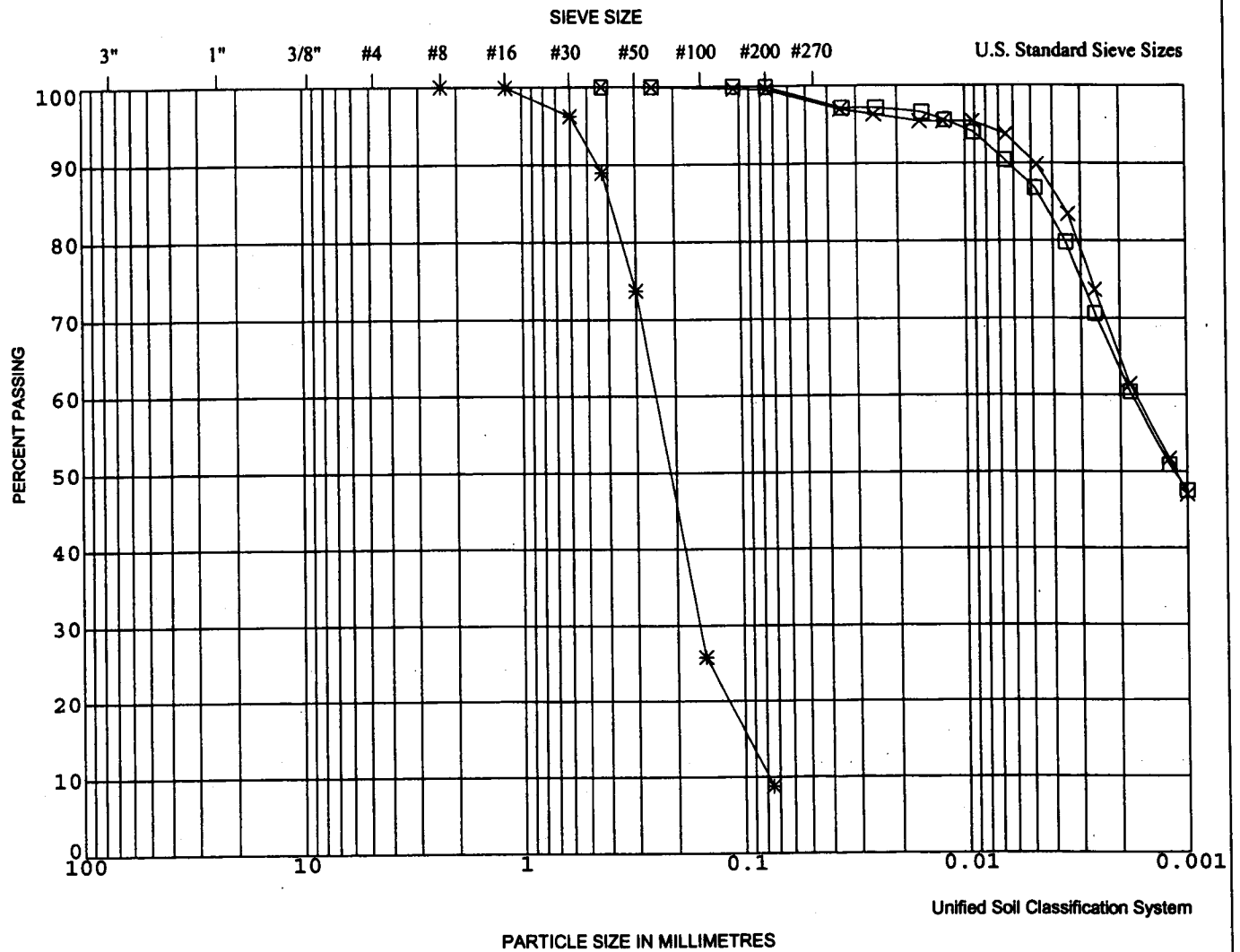
METRIC

W.P. 194-87-00 LOCATION 14+900.3 o/s 16.7m Lt - Walsh Township ORIGINATED BY PR
DIST 61 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY TG
DATUM Geodetic DATE 08.05.02 CHECKED BY SS

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	WATER CONTENT (%)					
207.5														
0.0	FILL - SAND - Silty, with cobbles & boulders, trace clay, brown		1	AS										Water level @ 0.6m on June 19, 2002. 1.2m of rock fill removed to level site. Standpipe installed to 2.0m.
206.2			2	AS										
1.3	CLAY - Silty, brown		3	SS	3									
205.7	-----													
1.8	- grey													
204.8	SAND - Silty, trace gravel, grey, compact		4	SS	30									
2.7	End of Borehole @ 2.7m. Auger Refusal.													

ON MOT 02-019-LOGS.GPJ ON MOT.GDT 05/07/02

GRAINSIZE ANALYSIS



LEGEND:

- BOREHOLE 1 DEPTH 5.30
- * BOREHOLE 3 DEPTH 10.70
- × BOREHOLE 4 DEPTH 9.10

July 2002

Reference No.



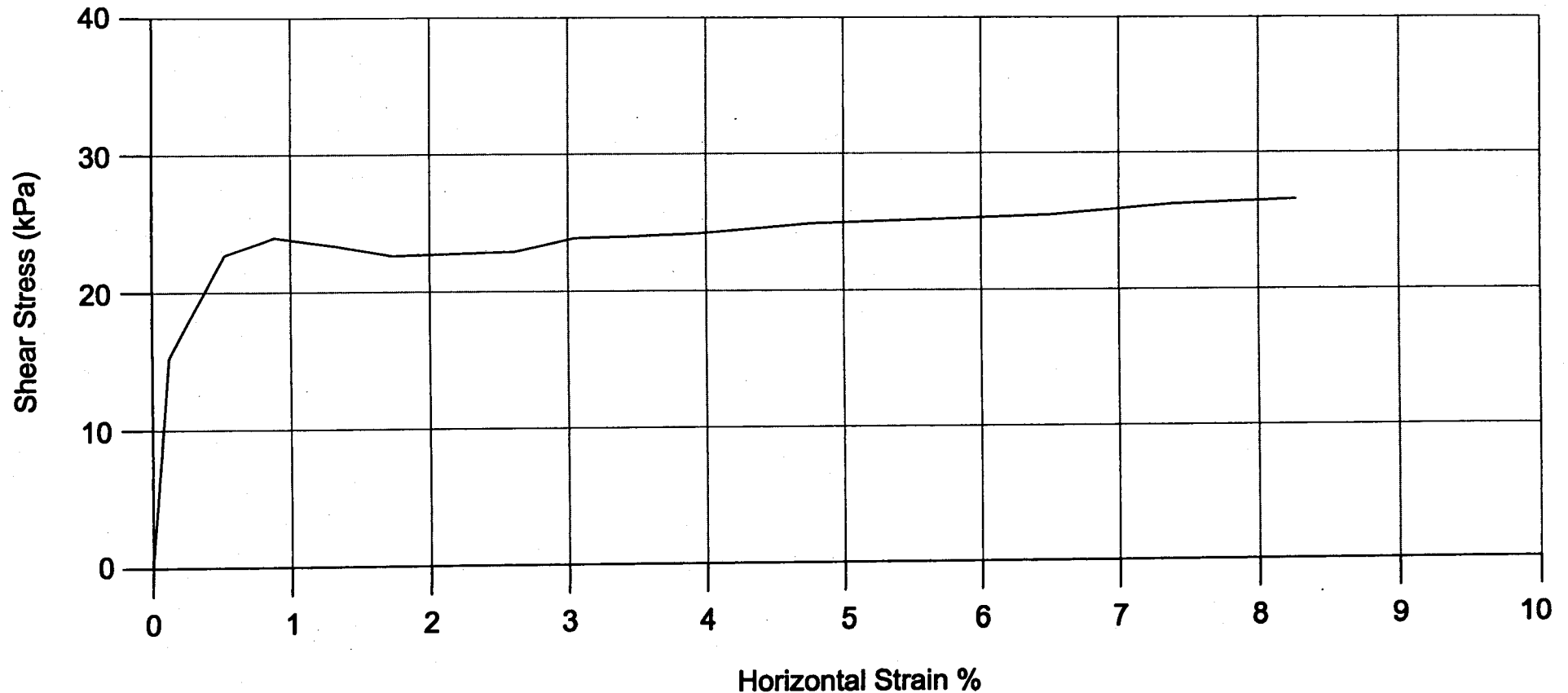
McKellar Creek - Highway 17

ENCLOSURE 19

CONSOLIDATED UNDRAINED DIRECT SHEAR TEST RESULTS

Clay - Borehole 1 @ 5.3m

Normal Stress = 70kPa



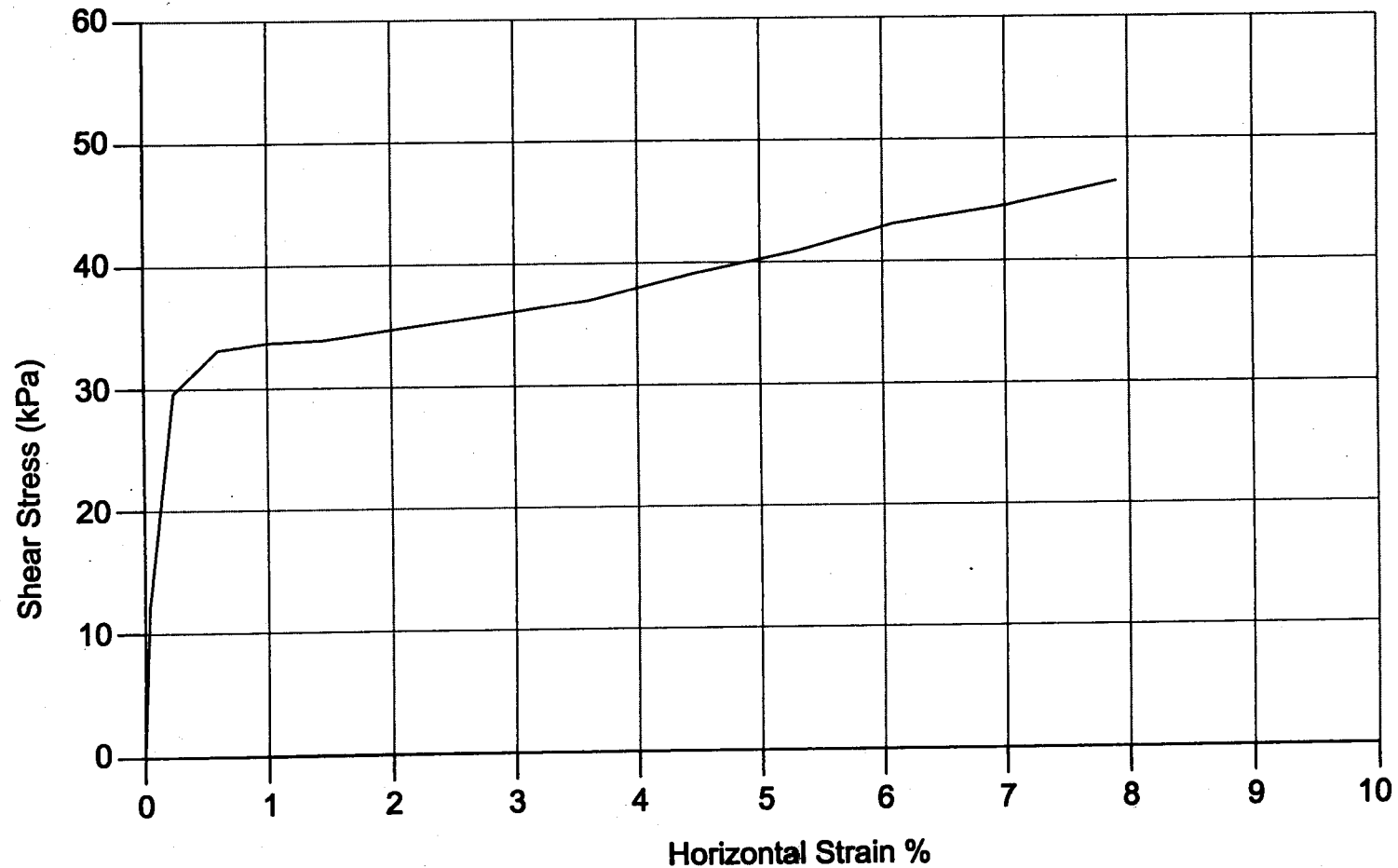
Reference No.: TG-02-019

Project: McKELLAR CREEK CULVERT

Date: JULY 2002

CONSOLIDATED UNDRAINED DIRECT SHEAR TEST RESULTS

Clay - Borehole 4 @ 9.1m
Normal Stress = 70kPa

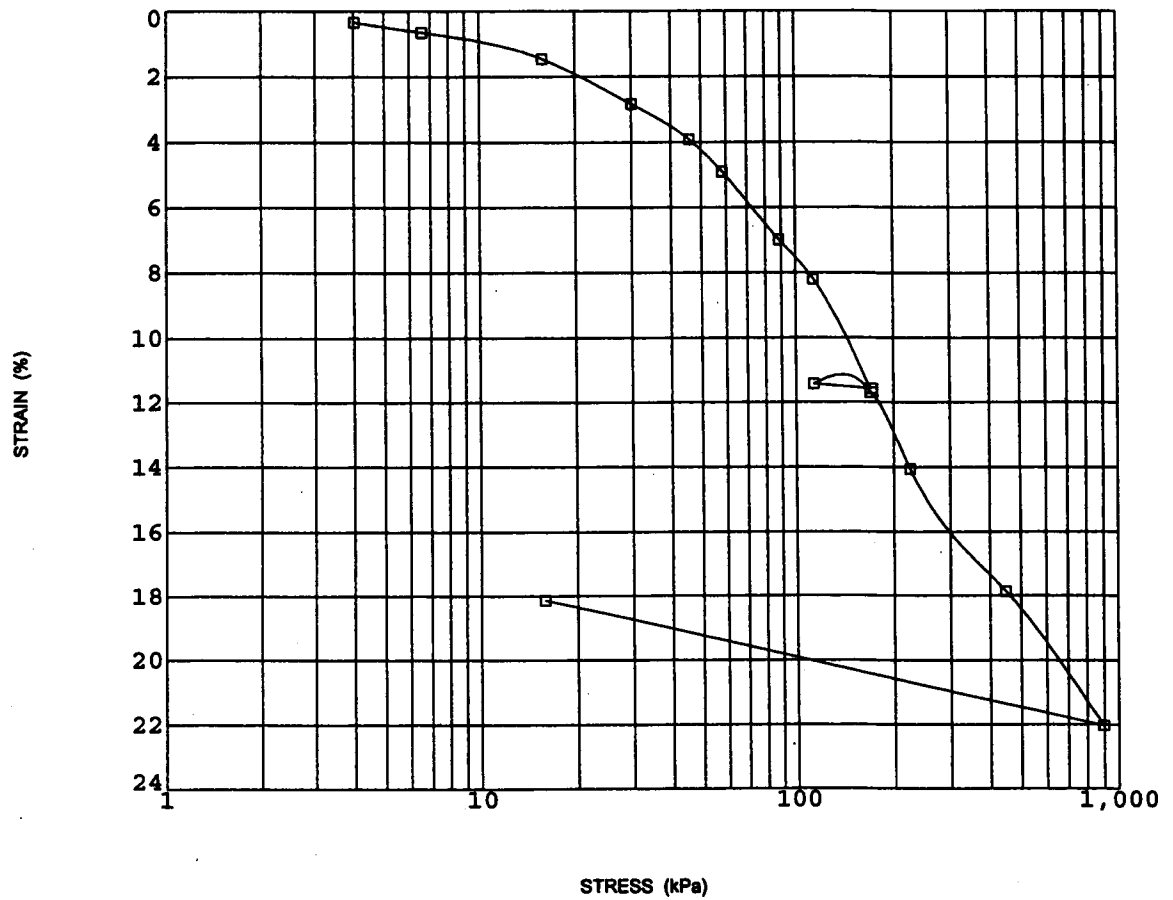


Reference No.: TG-02-019

Project: McKELLAR CREEK CULVERT

Date: JULY 2002

CONSOLIDATION TEST



LEGEND: □ BOREHOLE 4 DEPTH 9.10

Reference No.:

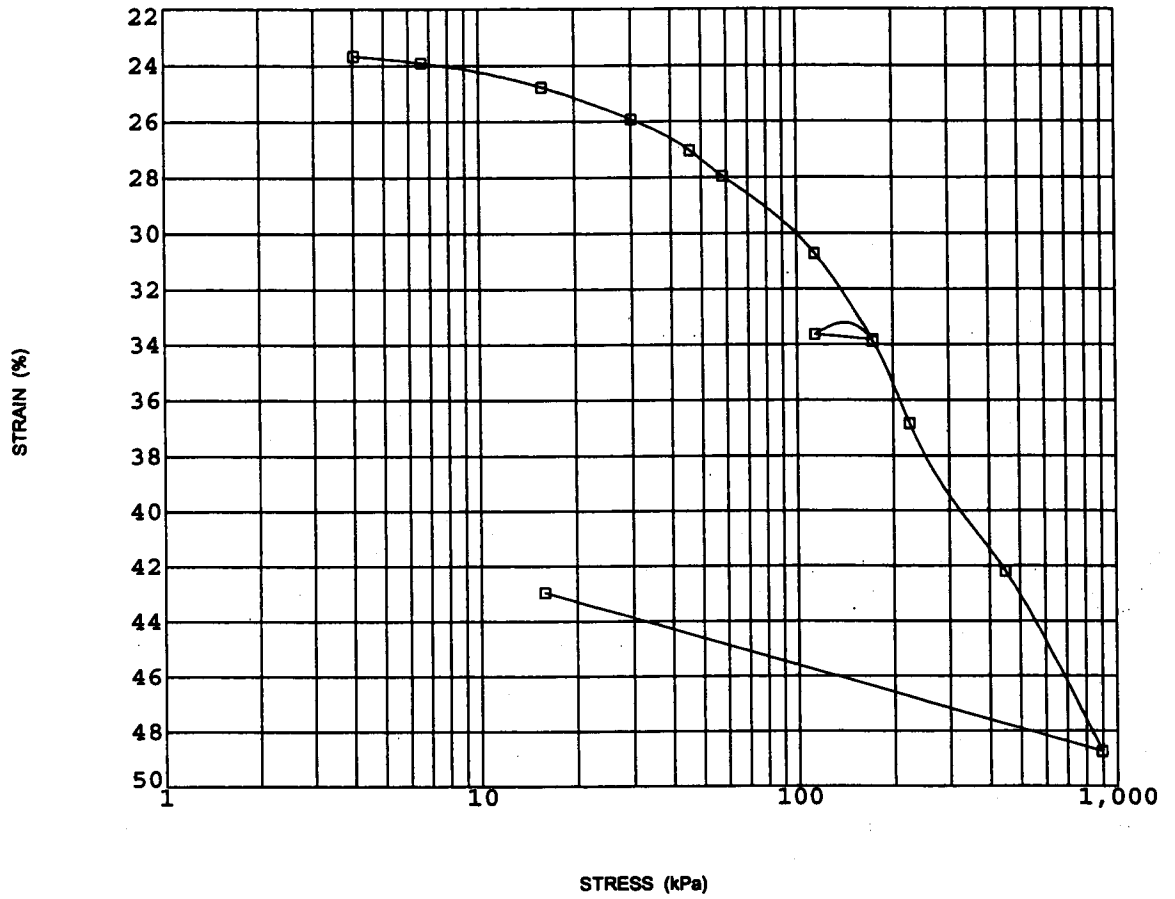
July 2002



McKellar Creek - Highway 17

ENCLOSURE 22

CONSOLIDATION TEST



LEGEND: □ BOREHOLE 1 DEPTH 5.30

Reference No.:

July 2002

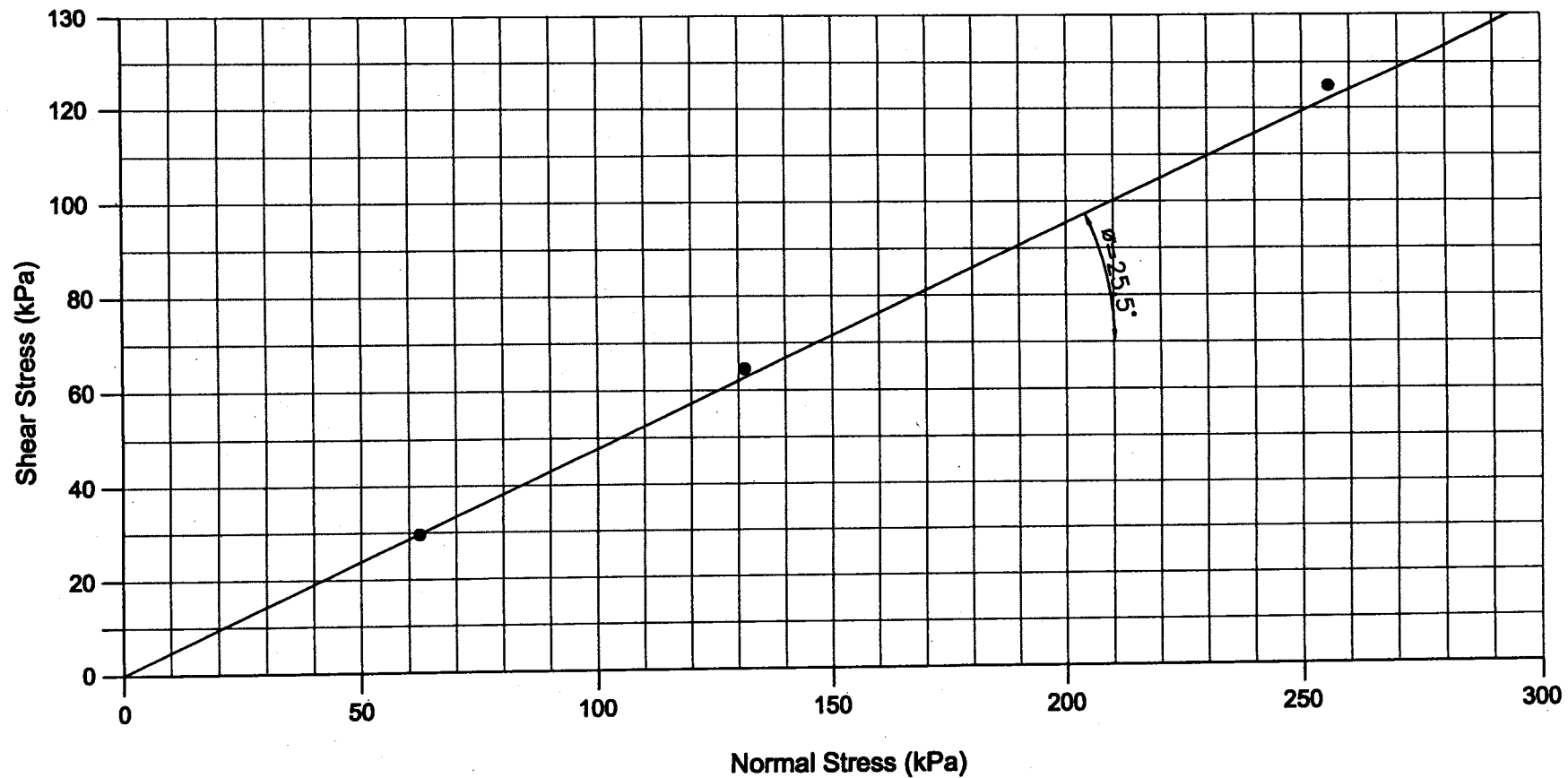


McKellar Creek - Highway 17

ENCLOSURE 23

CONSOLIDATED DRAINED DIRECT SHEAR TEST RESULTS

Clay - Borehole 4 @ 9.1m

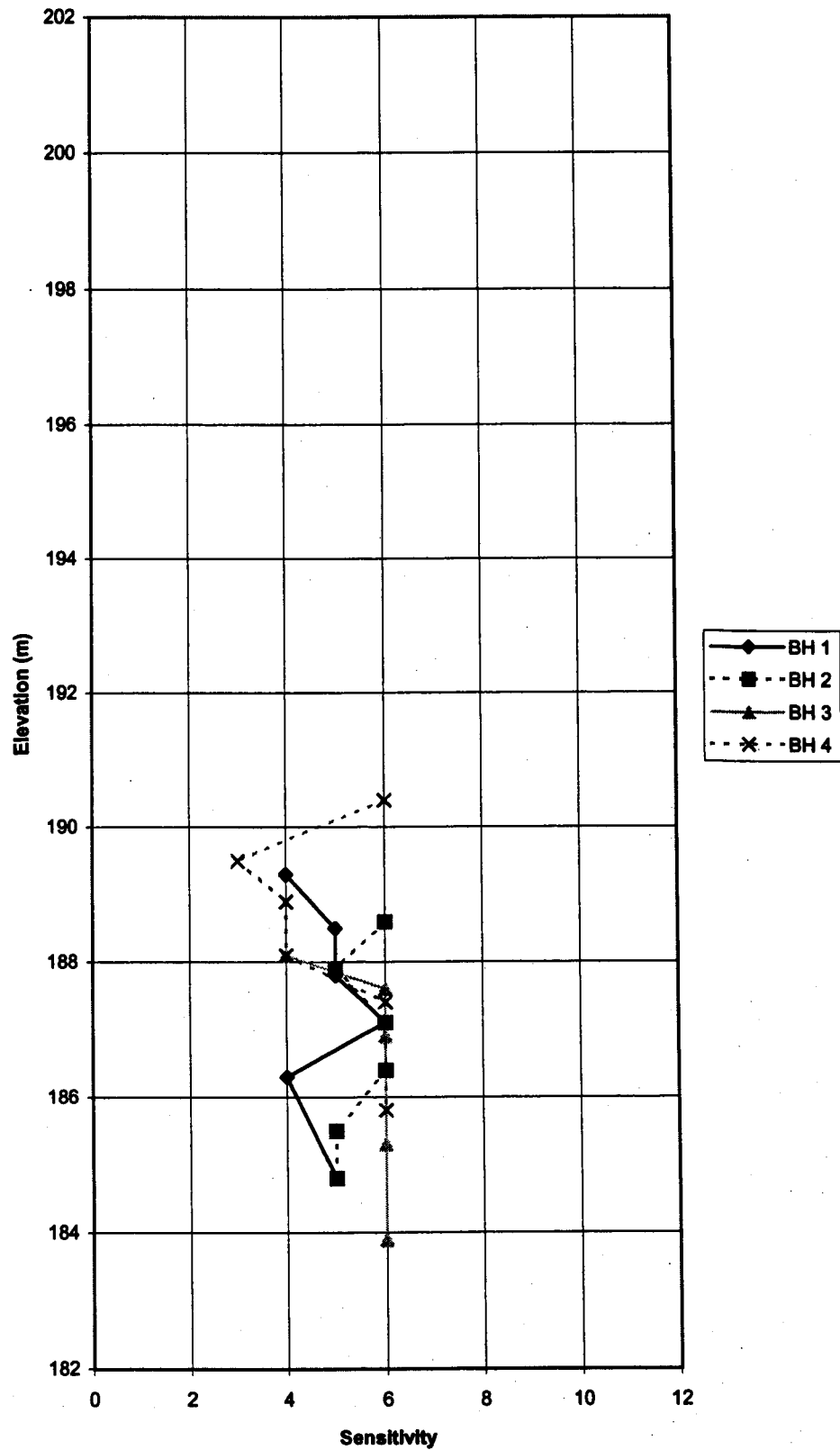


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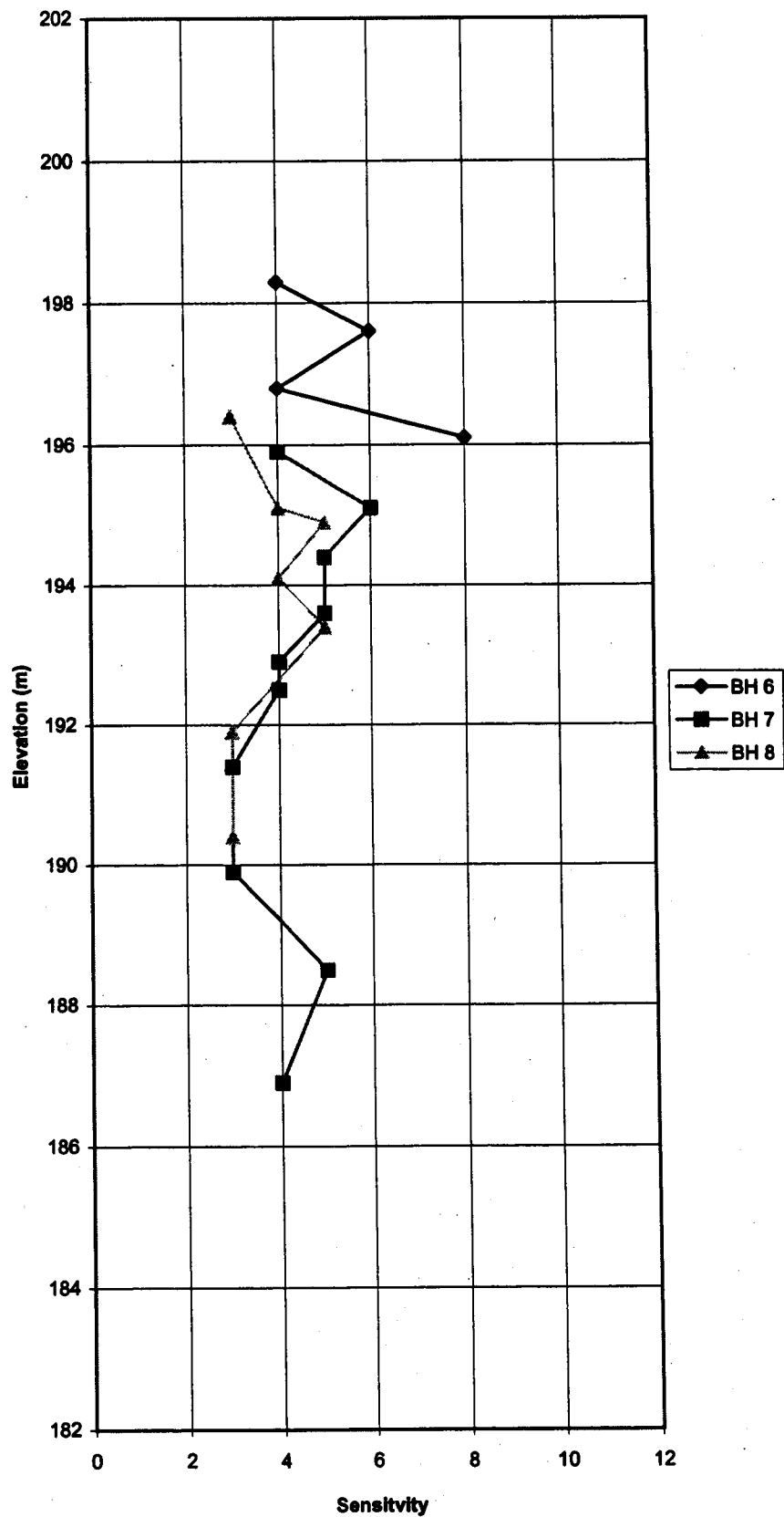
Project: McKELLAR CREEK CULVERT

Date: JUNE 2002

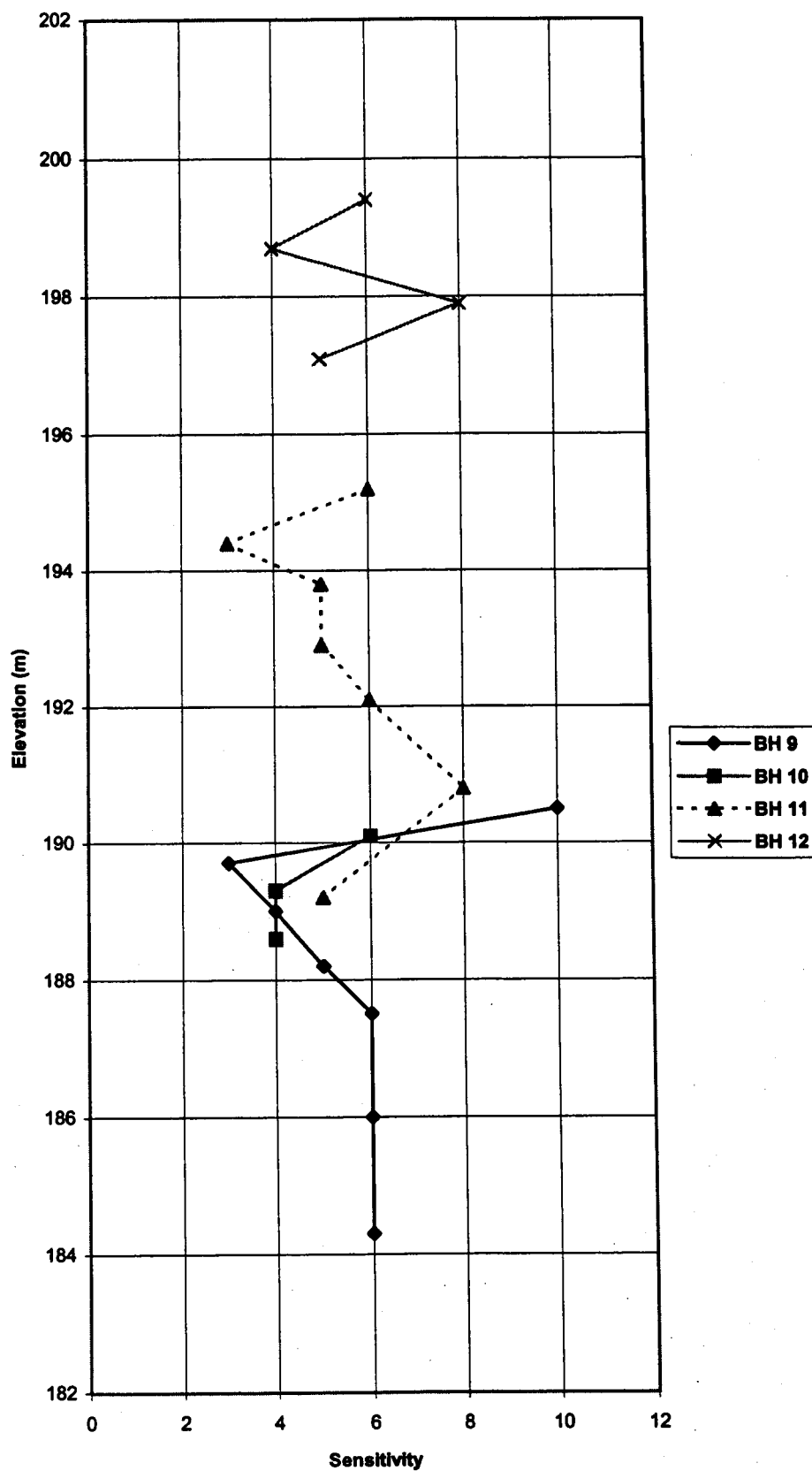
McKellar Creek Station 14+680 to 14+750
Clay Sensitivity vs. Elevation



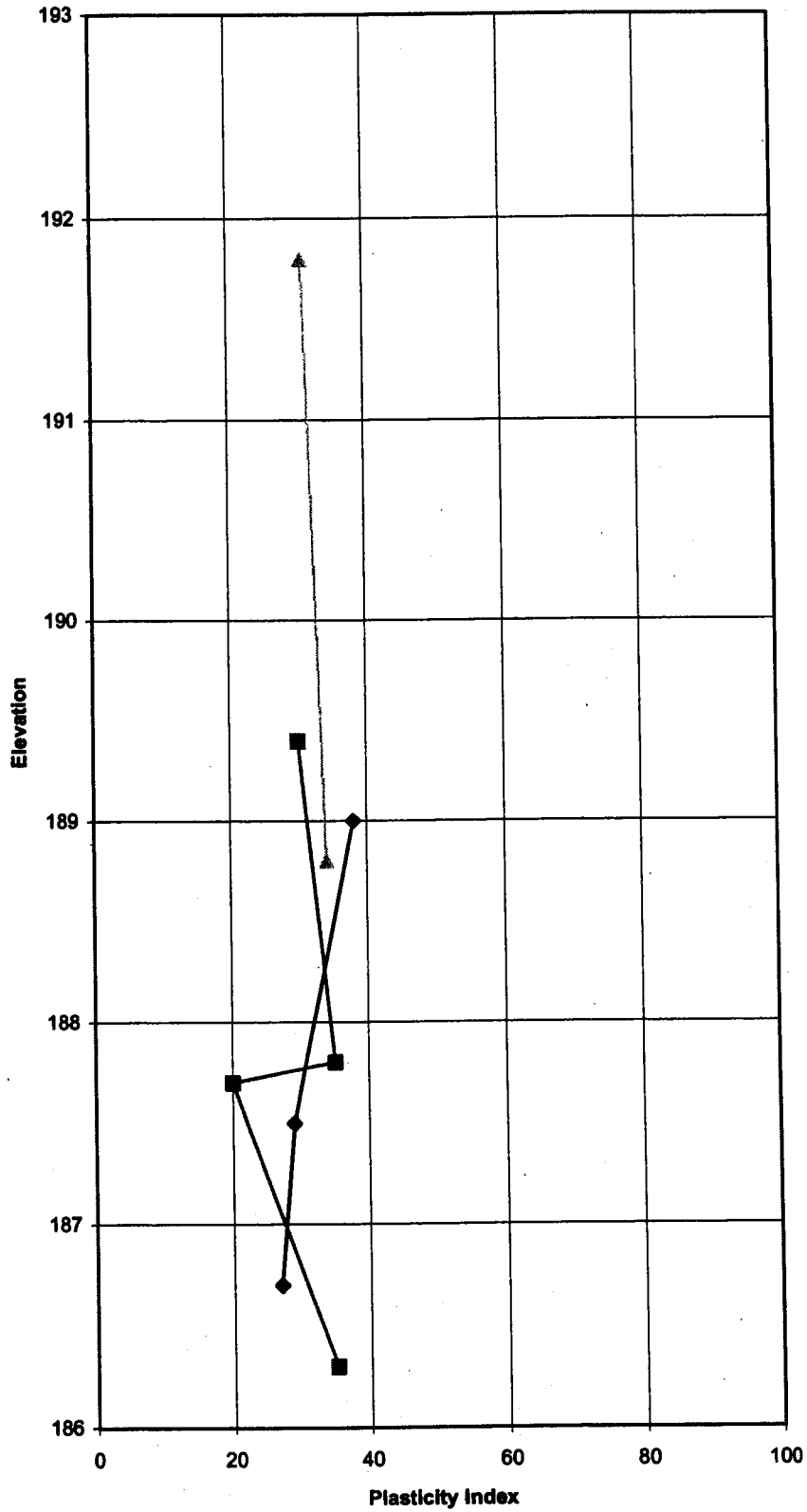
McKellar Creek Station 14+600 to 14+680
Clay Sensitivity vs. Elevation



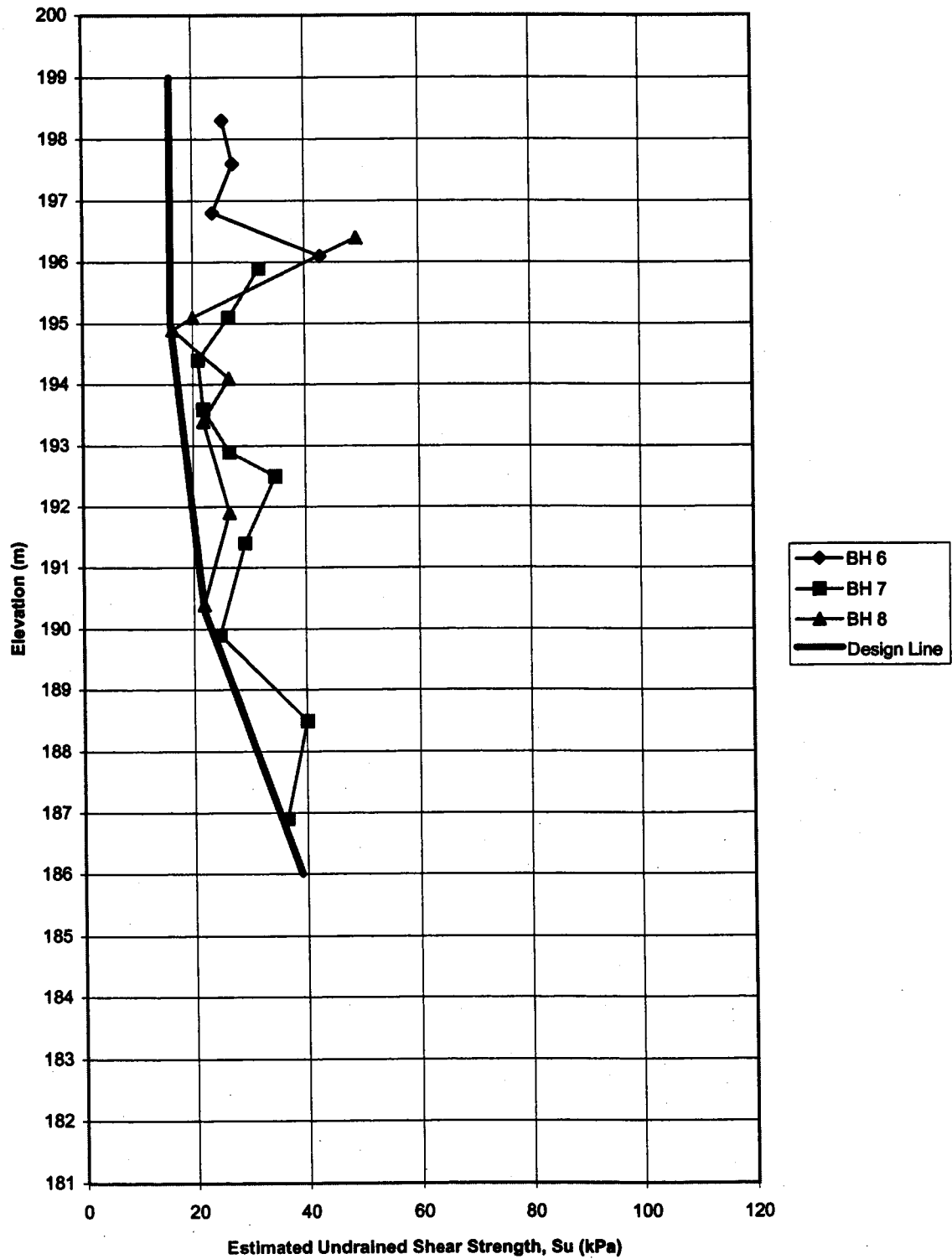
McKellar Creek Station 14+750 to 14+900
Clay Sensitivity vs. Elevation



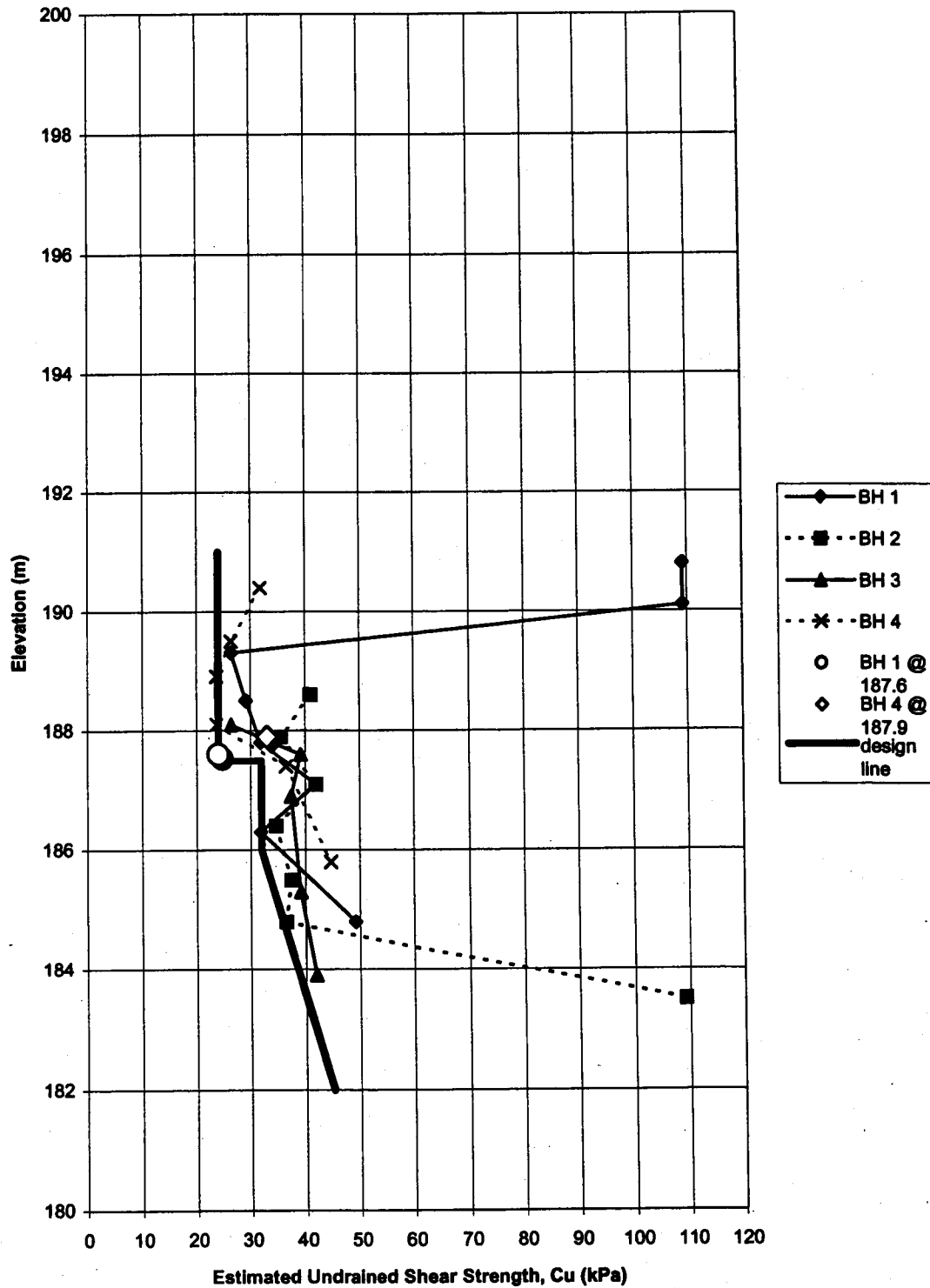
McKellar Creek
Plasticity Index vs. Elevation



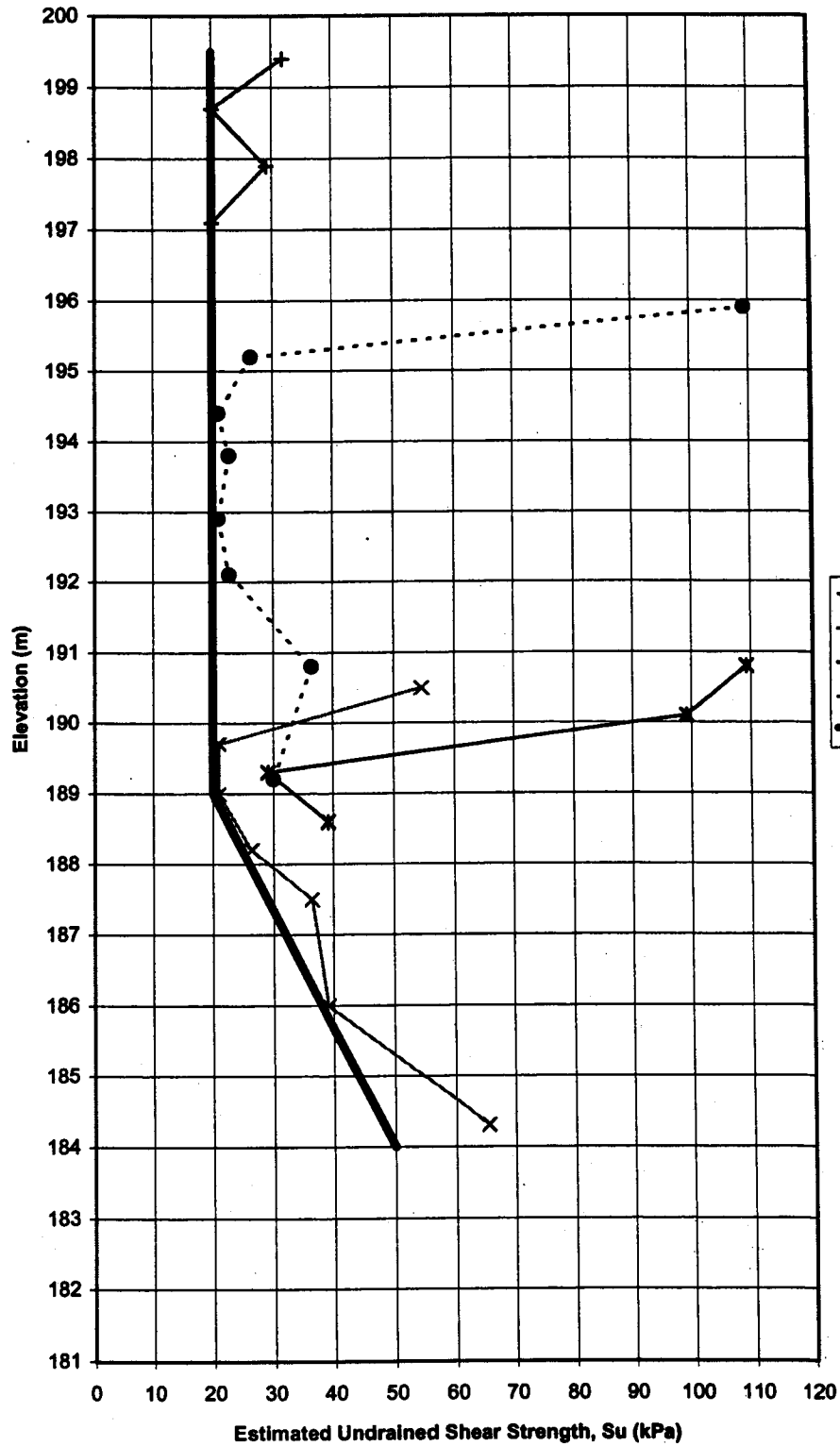
McKellar Creek Culvert Station 14+600 to 14+680
Estimated Undrained Shear Strength vs. Elevation
Based on Corrected Field Vane Testing



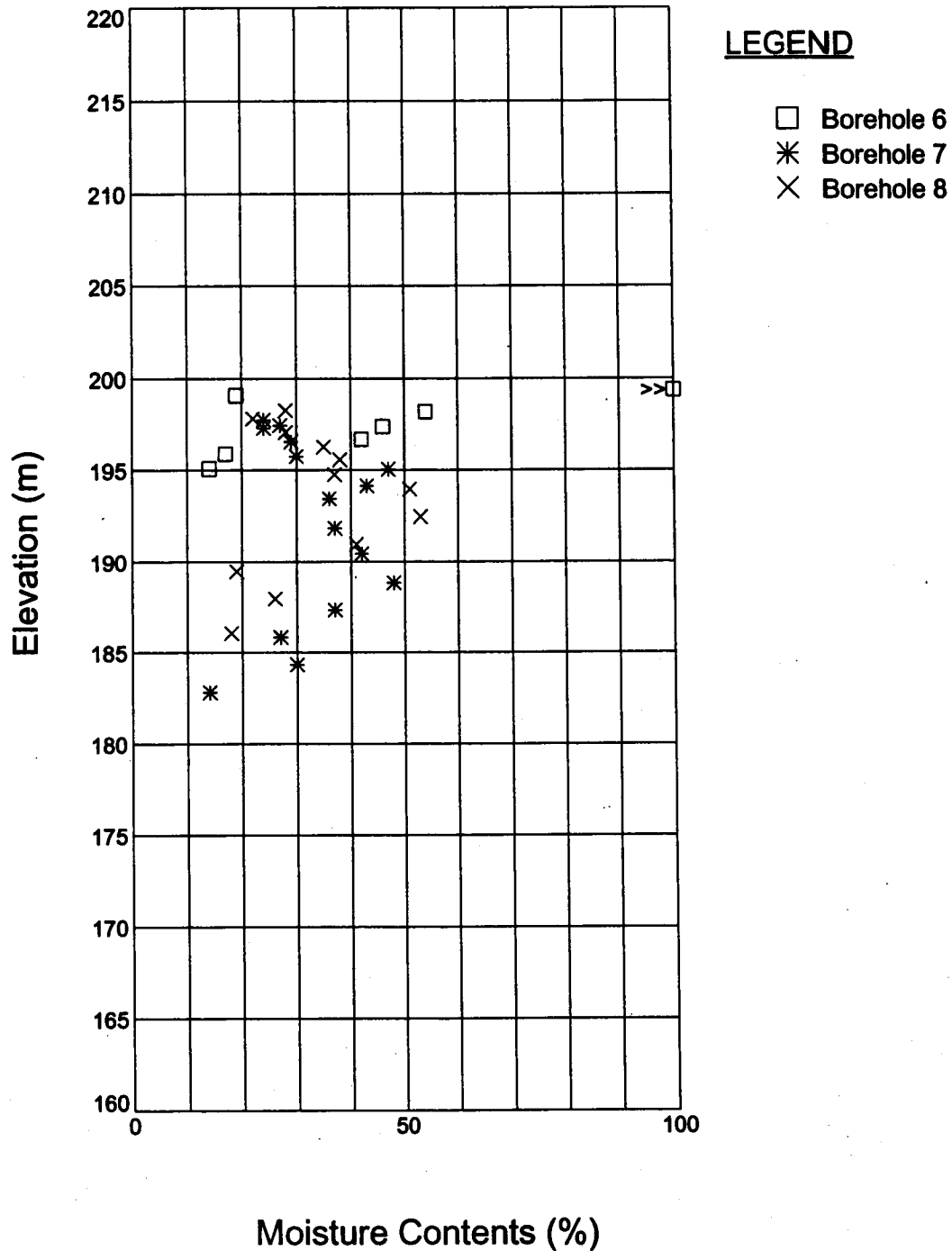
McKellar Creek Culvert Station 14+680 to 14+750
Estimated Undrained Shear Strength vs. Elevation
Based on Corrected Field Vane Testing and Undrained Direct Shear Testing



McKellar Creek Culvert Station 14+750 to 14+900
 Estimated Undrained Shear Strength vs. Elevation
 Based on Corrected Field Vane Testing

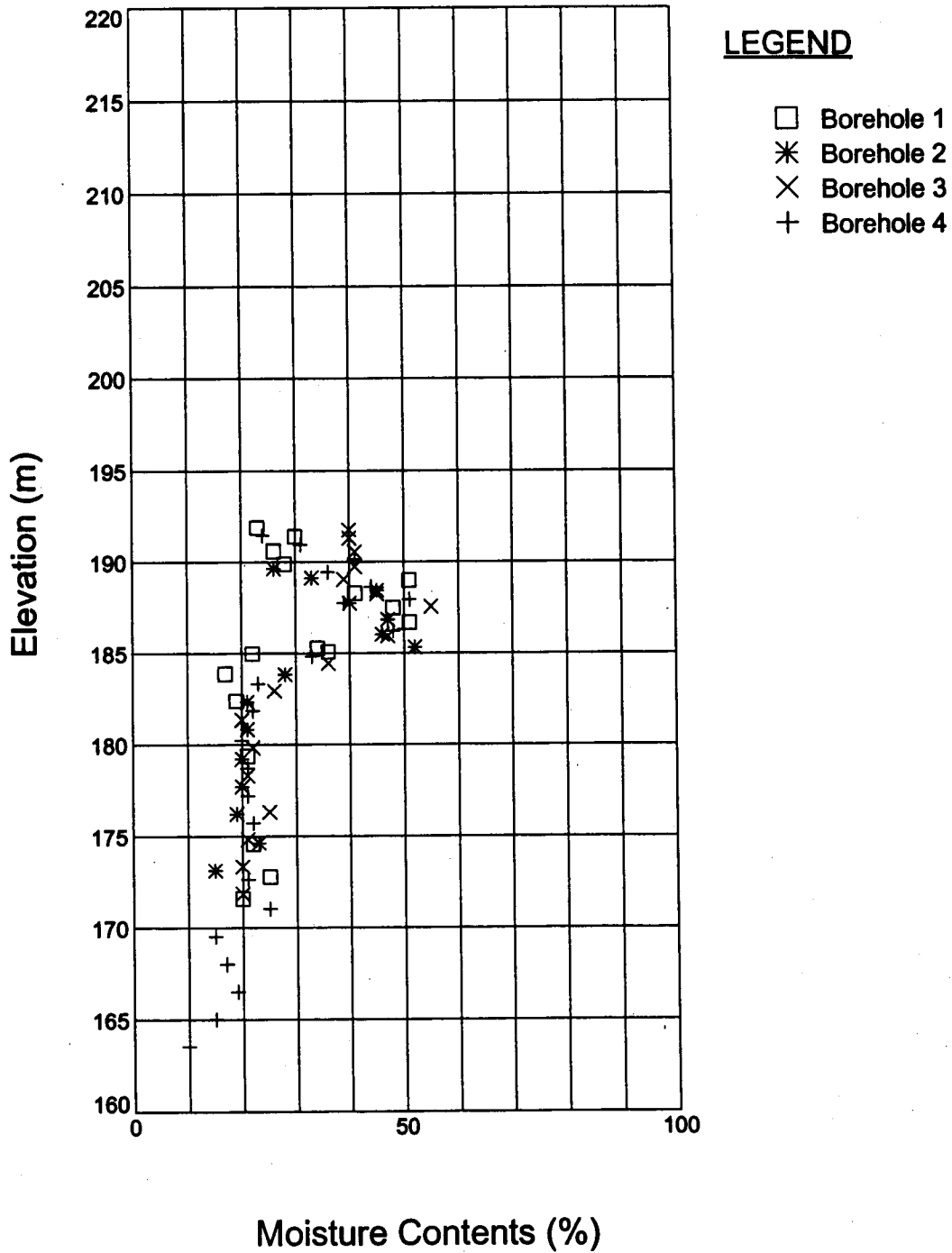


SUMMARY OF MOISTURE CONTENTS



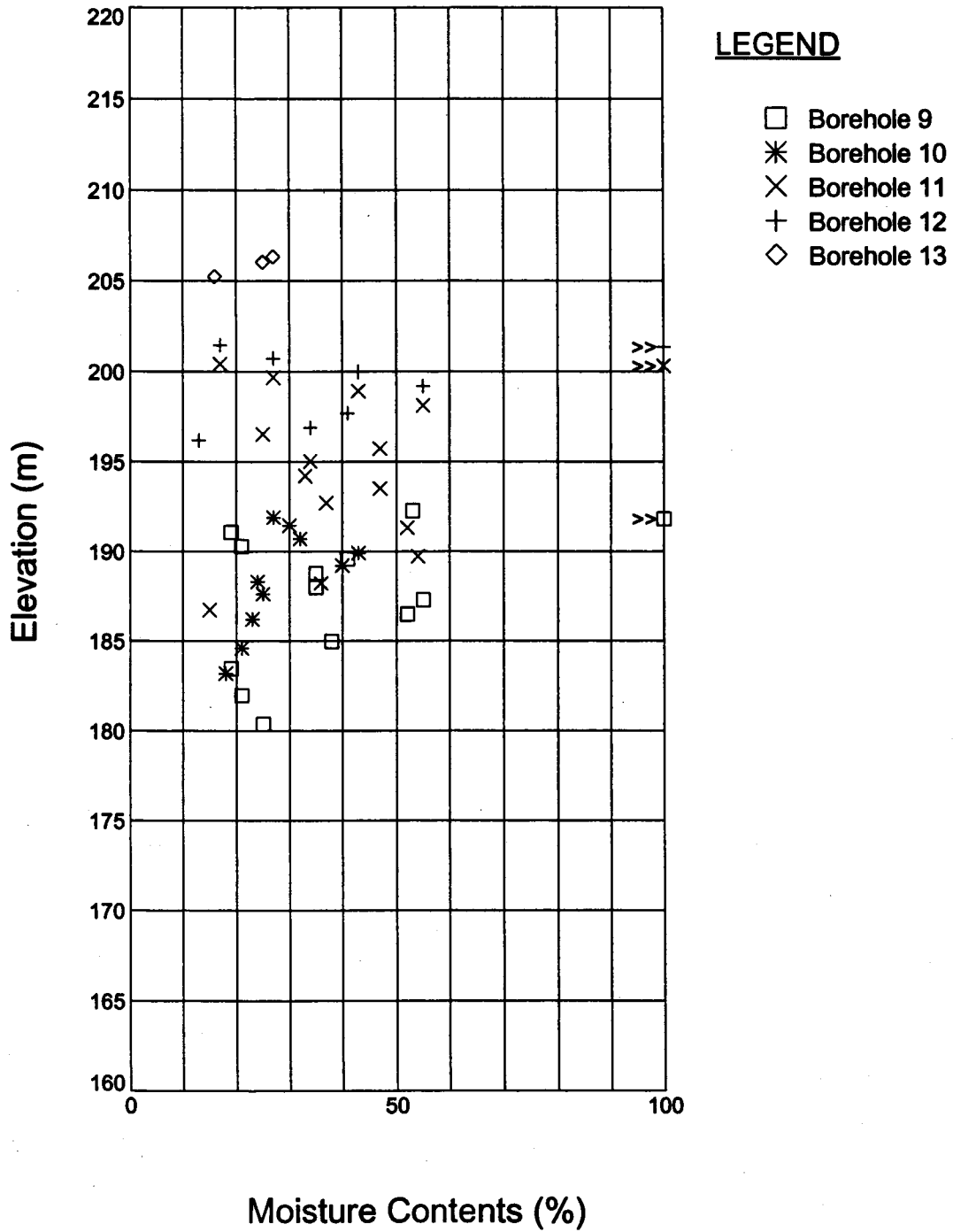
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SUMMARY OF MOISTURE CONTENTS

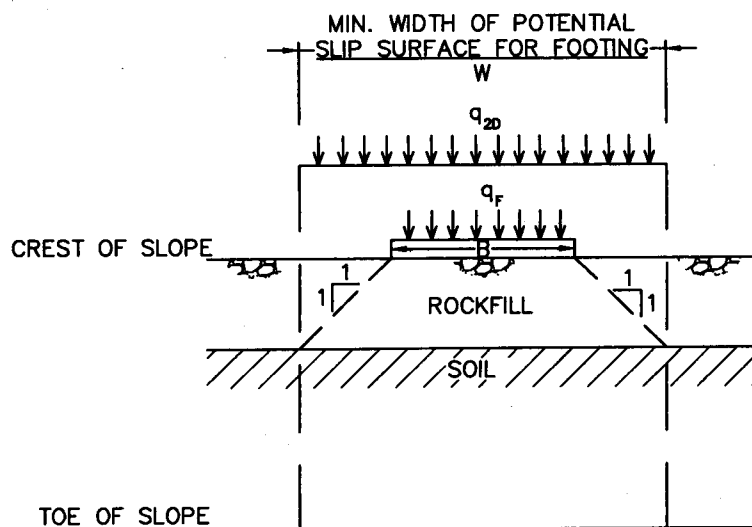


Reference No.

SUMMARY OF MOISTURE CONTENTS

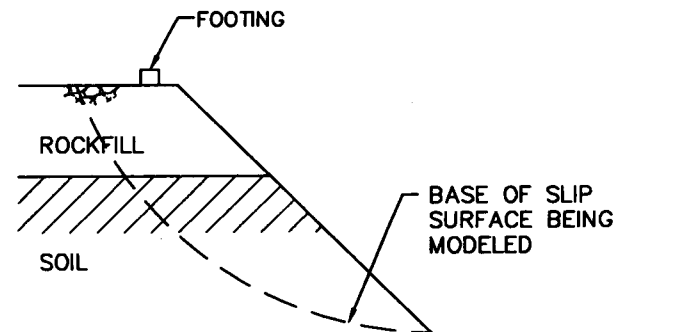


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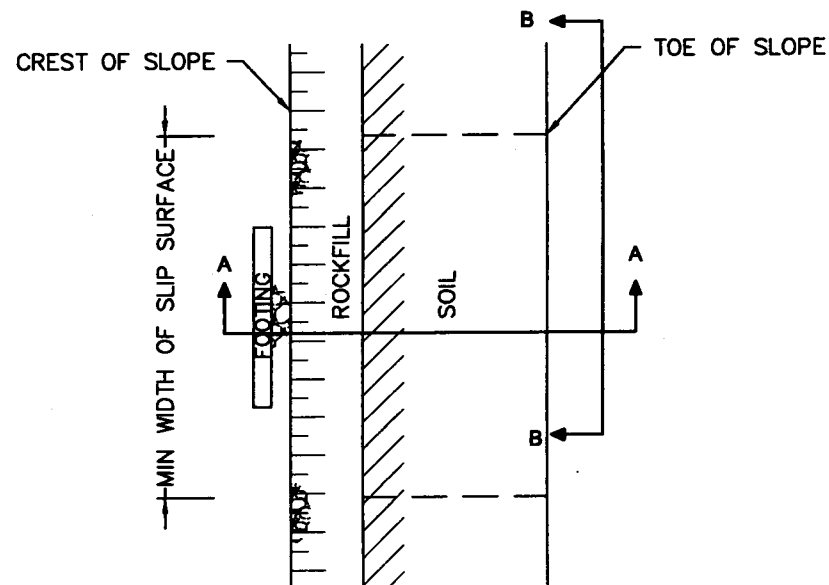


$$q_{20} = \frac{q_f \times B}{W} = \text{EQUIVALENT FOUNDATION LOAD FOR TWO DIMENSIONAL MODEL}$$

SECTION B-B

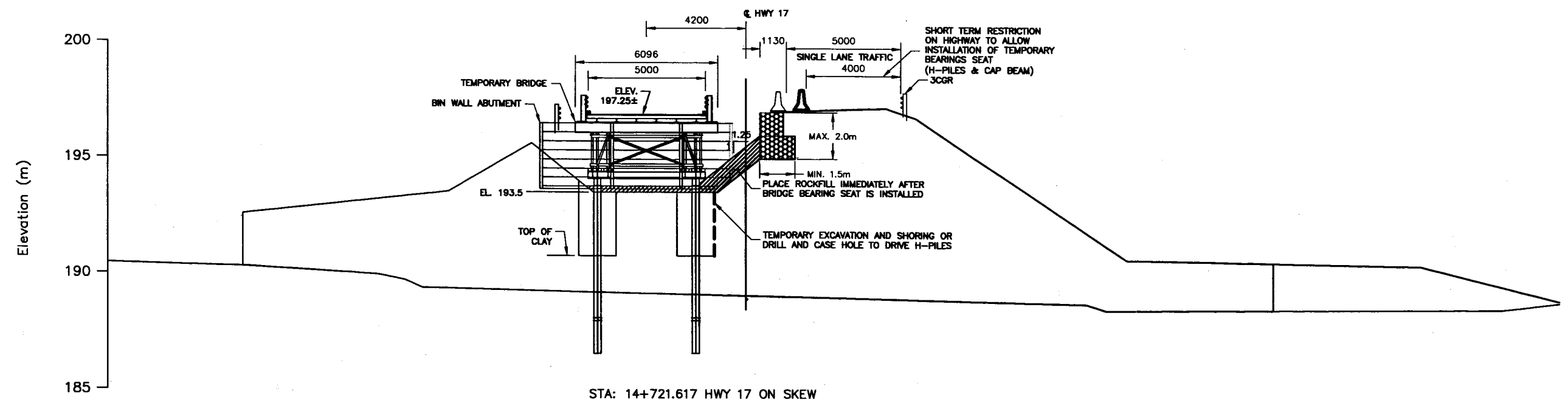


SECTION A-A



PLAN

OPTION 2



DATE DEC 2002		SCALE 1 : 200	
DRAWN BY T.G.	APPROVED BY G.M.	PROJ. TG02019	
CLIENT PHILIPS ENGINEERING LTD			
DWG. REF. NO. MCKELLAR-SEC-2			

NOTES: OPTION 2 BASED ON DRAWING PROVIDED BY PHILIPS ENGINEERING LTD.

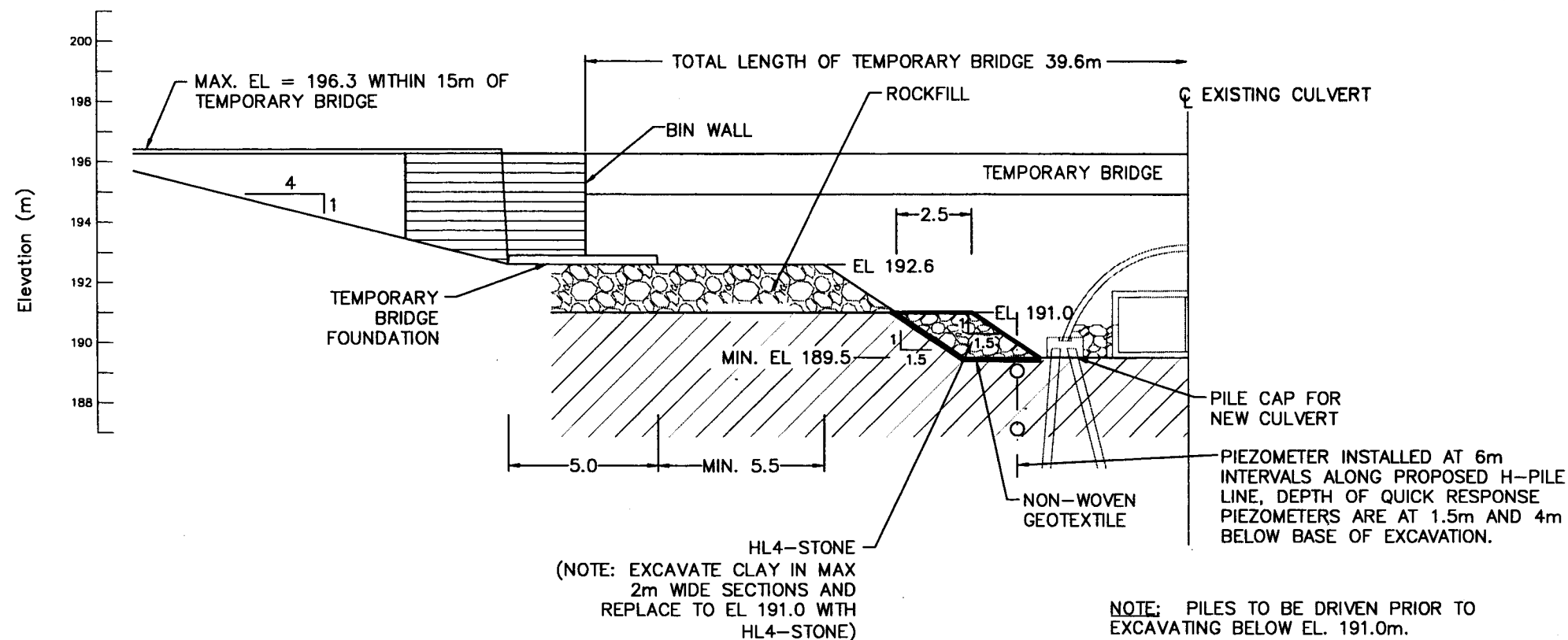
DST
CONSULTING ENGINEERS

TITLE
CONCEPTUAL DESIGN - PROPOSED STAGE 2 CONSTRUCTION
McKELLAR CREEK CULVERT - STATION 14+720
TERRACE BAY, ONTARIO

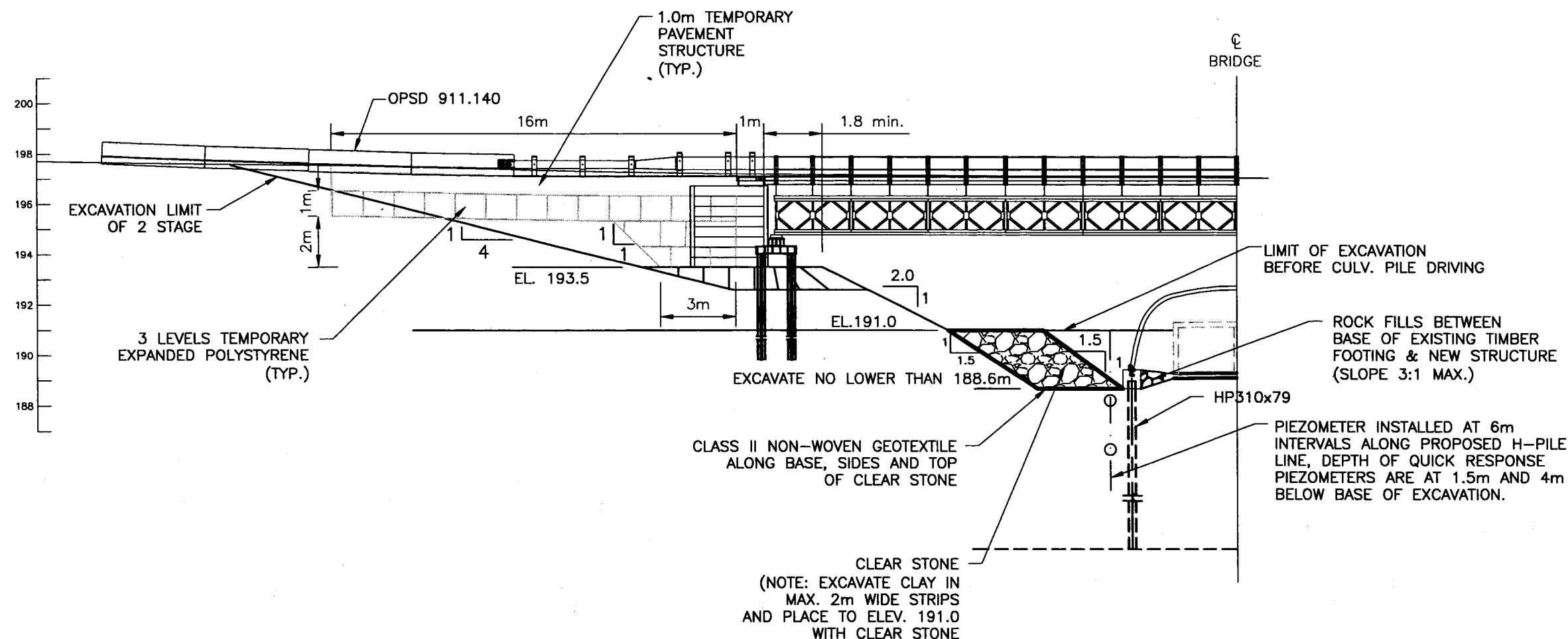
ENCLOSURE

36

OPTION 1



OPTION 2



DATE DEC 2002	SCALE 1:200
DRAWN BY T.G.	APPROVED BY G.M.
CLIENT PHILIPS ENGINEERING LTD	PROJ. TG02019
DWG. REF. NO. MCKELLAR-SEC-2	

NOTES: OPTION 2 BASED ON DRAWING PROVIDED BY PHILIPS ENGINEERING LTD.

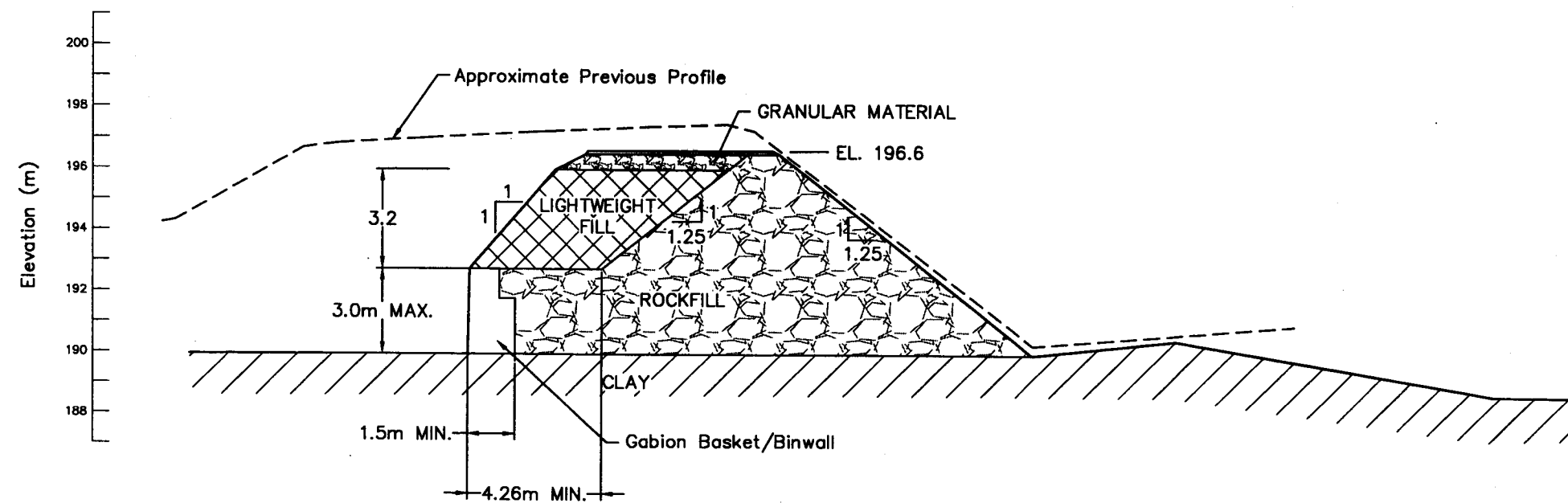
DST
CONSULTING ENGINEERS

TITLE
CONCEPTUAL DESIGN - PROPOSED STAGE 3 CONSTRUCTION
MCKELLAR CREEK CULVERT
TERRACE BAY, ONTARIO

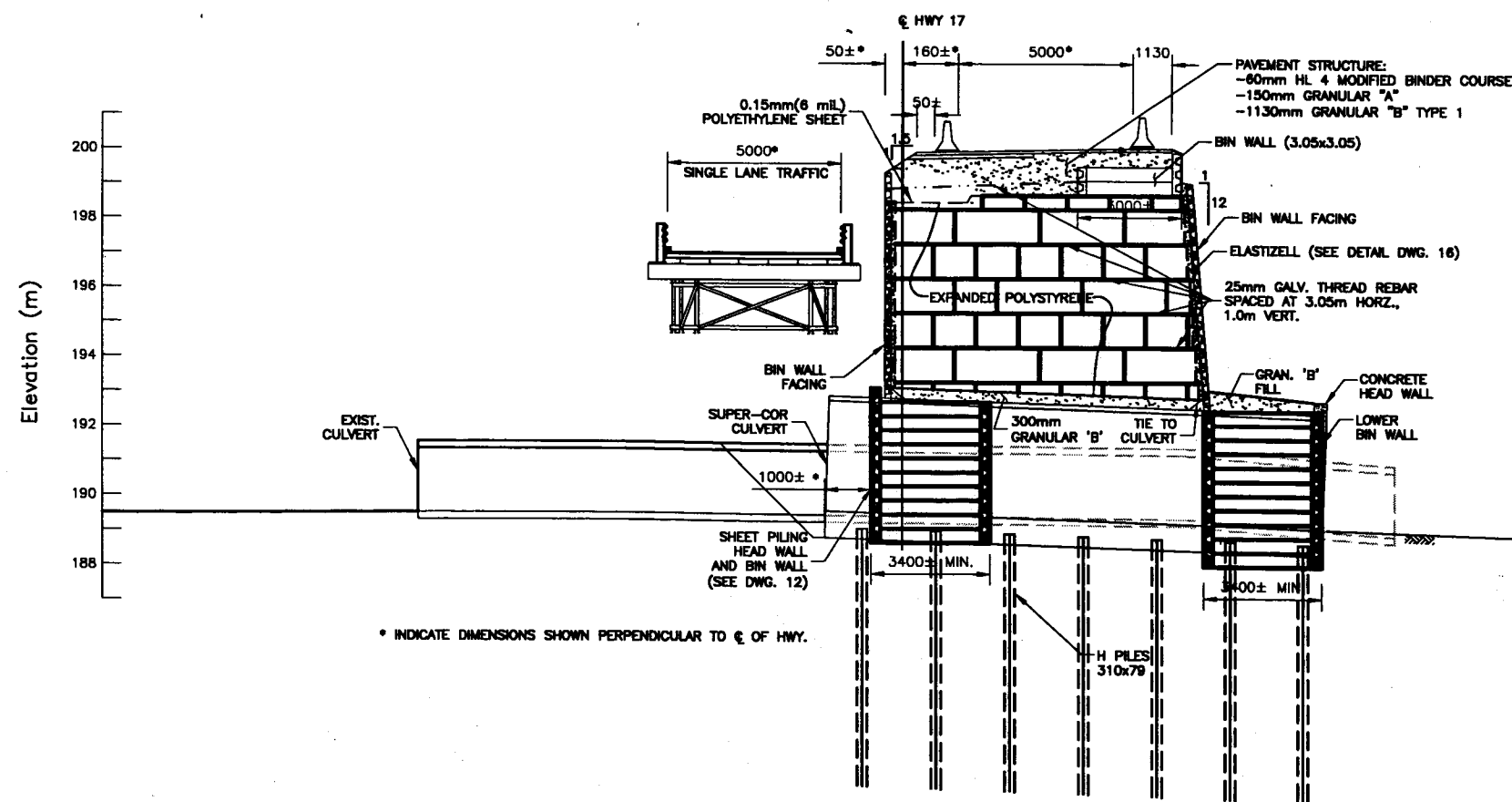
ENCLOSURE

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OPTION 1



OPTION 2



DATE	SCALE
DEC 2002	1:200
DRAWN BY	APPROVED BY
T.G.	G.M.
CLIENT	PROJECT
PHILIPS ENGINEERING LTD	TG02019
DWG. REF. NO.	CROSS-SEC

NOTES: OPTION 2 BASED ON DRAWING PROVIDED BY PHILIPS ENGINEERING LTD.

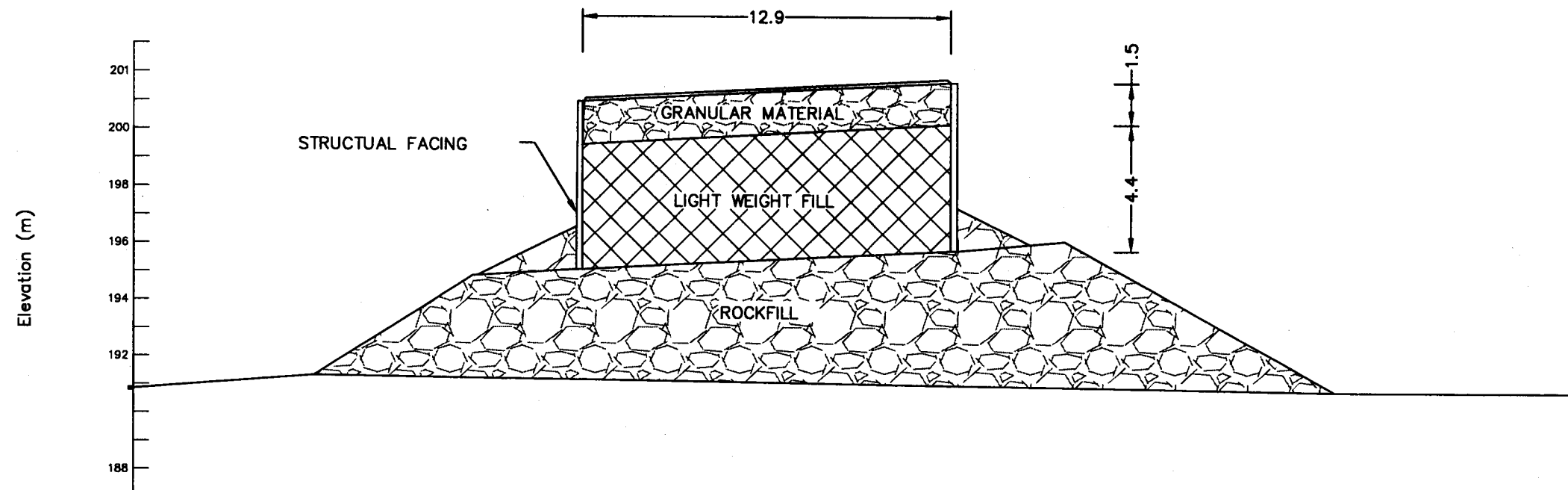
DST
CONSULTING ENGINEERS

TITLE
CONCEPTUAL DESIGN PROPOSED STAGE 4 CONSTRUCTION
McKELLAR CREEK CULVERT - STATION 14+720
TERRACE BAY, ONTARIO

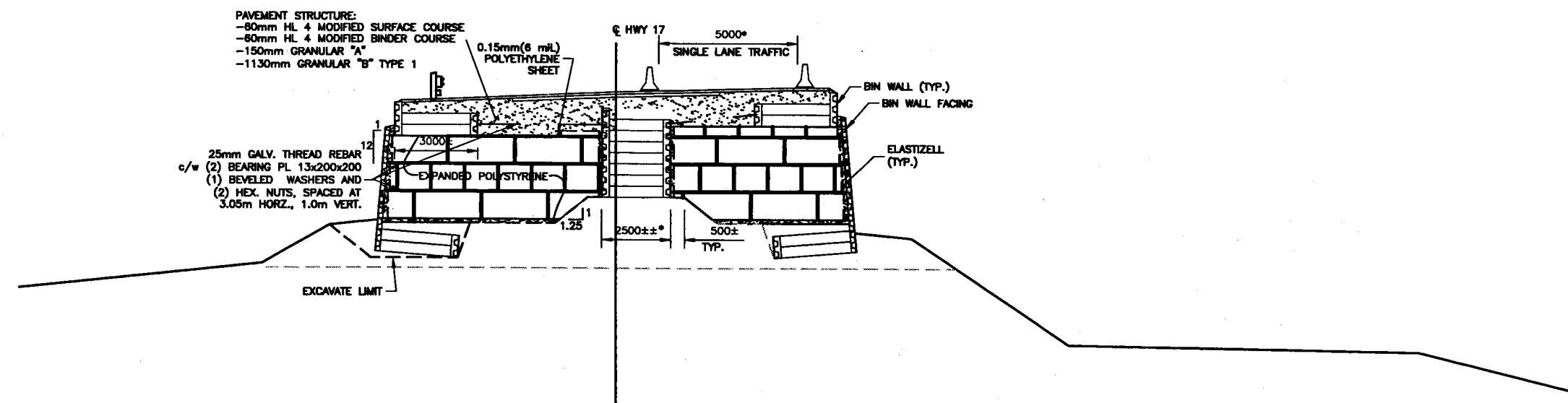
ENCLOSURE

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OPTION 1



OPTION 2



DATE	SCALE
DEC 2002	1:200
DRAWN BY	APPROVED BY
T.G.	G.M.
CLIENT	PROJECT
PHILIPS ENGINEERING LTD	TG02019
DWG. REF. NO. CROSS-SEC	

NOTES: OPTION 2 BASED ON DRAWING PROVIDED BY PHILIPS ENGINEERING LTD.

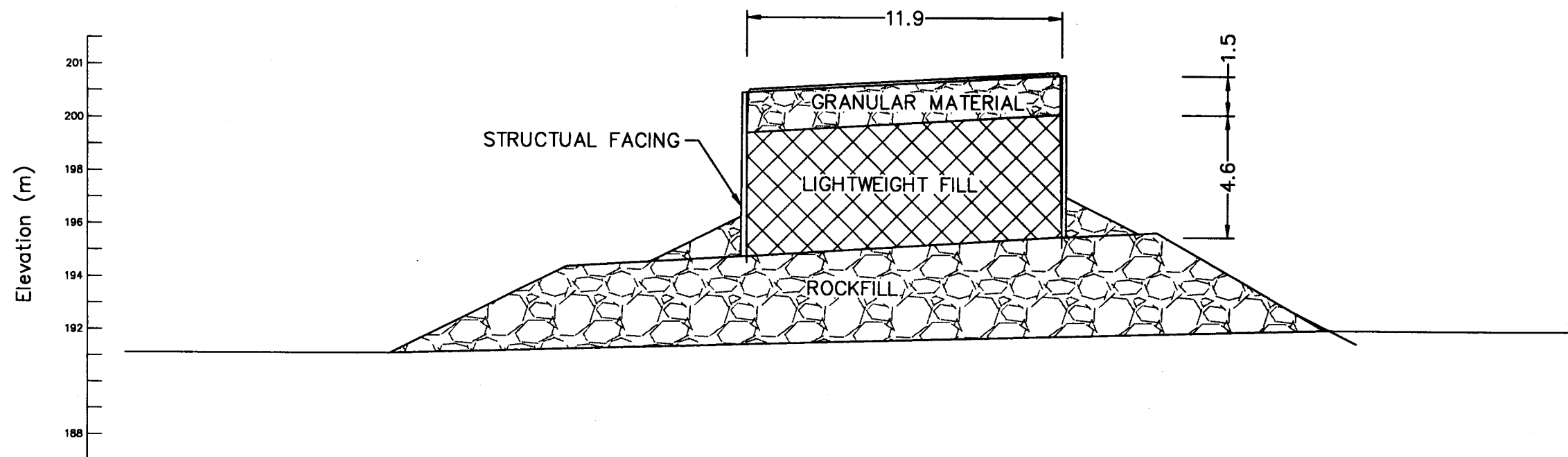
DST
CONSULTING ENGINEERS

CONCEPTUAL DESIGN PROPOSED STAGE 6 CONSTRUCTION
McKELLAR CREEK CULVERT - STATION 14+680
TERRACE BAY, ONTARIO

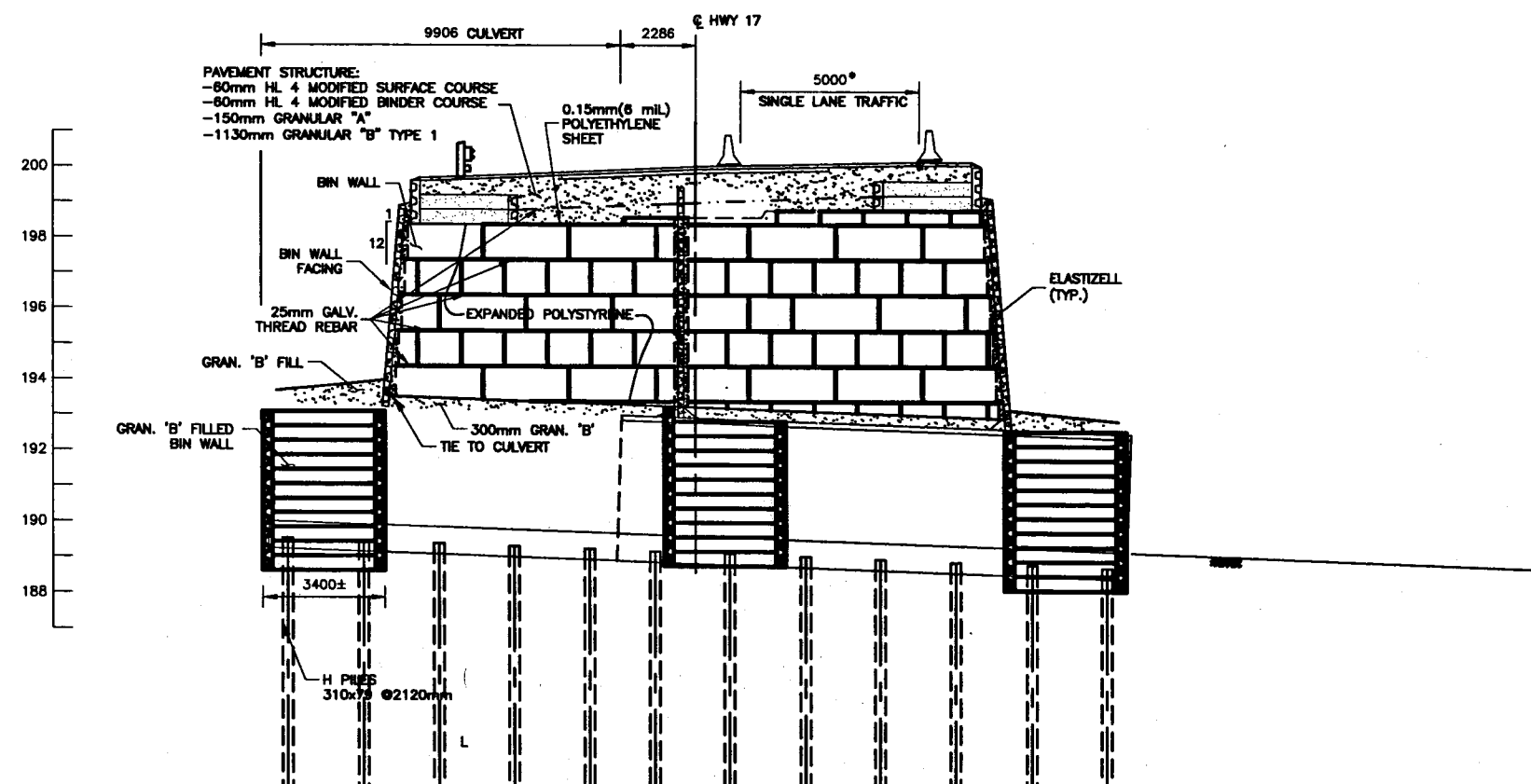
ENCLOSURE

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OPTION 1



OPTION 2



DATE	SCALE
DEC 2002	1:200
DRAWN BY	APPROVED BY
T.G.	G.M.
CLIENT	PROJECT NO.
PHILIPS ENGINEERING LTD	TG02019
DRAW. REF. NO.	CROSS-SEC

NOTES: OPTION 2 BASED ON DRAWING PROVIDED BY PHILIPS ENGINEERING LTD.

DST
CONSULTING ENGINEERS

CONCEPTUAL DESIGN PROPOSED STAGE 6 CONSTRUCTION
McKELLAR CREEK CULVERT - STATION 14+750
TERRACE BAY, ONTARIO

ENCLOSURE

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