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REPORT ON

**FOUNDATION INVESTIGATION
SETTLEMENT OF EAST APPROACH OF HIGHWAY
401 EBL/27 STRUCTURE
PROJECT NO. 2002 – 2000
TORONTO, ONTARIO**

Submitted to:

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January 2004

03-1111-032



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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation for the area of distress on the eastbound lanes (EBL) adjacent to the new east abutment of the Highway 401 overpass structure over Highway 27.

The terms of reference for the scope of work were outlined in Golder's proposal P31-1423 dated 28 October 2003. The scope of work was modified during a meeting at Morrison Hershfield's site office on 5 November 2003 attended by Ms. Anne Poschmann (Golder), Mr Tony Verta (MH), Mr. Terry Choo-Kang (MH) and Ms. Betty Bennett (MTO). The modifications essentially involved a reduction in the number of boreholes from three (3) to one (1), plus a slight modification to the initially proposed laboratory testing.

This report presents the results of the field investigation, laboratory testing and analysis completed to assess the factor(s) contributing to the distress on the Highway 401 EBL adjacent to the east abutment of the overpass structure at Highway 27. The objectives of this investigation include gaining an understanding of the embankment fill materials and underlying native soils, and provide a discussion on the possible causes of distress.

2.0 SITE DESCRIPTION

The site is located on the eastbound lane of the newly constructed Highway 401 overpass of Highway 27 in Toronto, Ontario (Figure 1). For purposes of this report and as referenced in the construction drawings supplied by MH, Highway 401 is orientated in an east-west direction, with Highway 27 orientated in a northeast-southwest direction.

The previous Highway 401 overpass consisted of a five span post-tensioned concrete structure. The existing abutments, adjacent to the foreslopes at inclinations of approximately 4H:1V on the east side and 6H:1V on the west side of Highway 27, are supported on piles apparently driven to found on the hard till/shale bedrock.

The new Highway 401 overpass, which will be wider than the previous structure, will be a two span pre-stressed box girder structure with central piers located between the north and southbound lanes of Highway 27, and closed abutments located on the east and west sides of Highway 27. The new abutments are at, or close to the locations of the previous Piers 1 and 3, and are constructed within a Reinforced Soil System (RSS). The approach embankments constructed using RSS and extending to the previous abutments, involve fill placement over the previous foreslopes.

The widening of the Highway 401 overpass requires a staged approach to demolishing and construction in order to maintain continued flow of traffic along Highways 401 and 27. Figure 2 (Sheet 529 of the construction drawings) includes a plan and sections of the overpass structure, noting the previous and new pier and abutment locations, the region where settlement is observed to be occurring, plus the approximate location of the borehole recently put down for this investigation. For simplicity and unless otherwise indicated, the area of distress which includes the southern third of the Highway 401 eastern abutment (which encompasses approximately 2/3rds of the Highway 401 EBL), will be referenced as the "SE Abutment". The Central and Northern thirds of the eastern abutment will be referenced as CE Abutment and NE Abutment respectively as also illustrated in Figure 2.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for this investigation was carried out during the late evening and early morning hours of 14 November 2003 and 15 November 2003, during which one (1) borehole (BH-1) was put down in the north portion of the newly constructed eastbound lanes, just east of the new approach slab as illustrated in Figure 2.

The field investigation was carried out using a tire mounted CME 55 drill rig supplied by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole was advanced using a 8 1/4" O.D. hollow stem auger. Continuous soil sampling was undertaken using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Continuous sampling was carried out within the fill materials and underlying native soils. The borehole was terminated at a depth of 46.5 ft (14.2 m) within residual soil. The Record of Borehole sheet for this investigation is included in Appendix A.

The samples from the field investigation were returned to Golder Associates' laboratory facilities in Mississauga for further inspection and testing. Appropriate samples were selected for water content determination, grain size analysis, Atterberg Limit and organic content determination. The laboratory testing results are included with the record of borehole sheet in Appendix A.

3.2 Subsurface Conditions

3.2.1 Pavement Structure

About 150 mm of asphalt underlain by the 250 mm thick unreinforced concrete base slab was encountered at the road surface. The concrete is underlain by 1.4 m of sand and gravel associated with the base and subbase materials for the road. Measured SPT N values of 14 and 20 blows per 0.3 m of penetration reflect a compact state of packing for the base and subbase materials.

3.2.2 Fill

The pavement structure is underlain by fill materials typically consisting of clayey silt with some sand and gravel, and extends to about a depth of 11 m (Elevation 141.5 m). To a depth of 8.5 m, the clayey silt is brown in colour and generally has a stiff consistency, with SPT N values ranging from 9 through 14 blows per 0.3 m of penetration. From a depth of 5.5 m to 7.3 m however, the clayey silt had a very soft to soft consistency, with SPT N values ranging from 2 to 4 blows per 0.3 m of penetration.

Water contents measured on samples of the fill increased gradually with depth, measuring 9% at a depth of 1.9 m, increasing to 13% at a depth of 8.4 m. The Atterberg Limit values of the samples also showed a slight increase with depth, with the Plastic Limit (PL) and Liquid Limit (LL) values of 12% and 21% at a depth of 1.9 m, increasing slightly to 13% and 24% respectively at a depth of 8.5 m.

Between depths of 8.5 m and 10.4 m, the fill consisted of clayey silt with sand and trace gravel, was mottled brown and grey in colour and had a very stiff to hard consistency, with SPT N values ranging from 16 to 40 blows per 0.3 m of penetration. The water contents on samples of this fill layer were less than that encountered above, with values ranging from 7% to 11%.

A thin layer of silty sand with trace to some gravel was encountered from a depth of 10.4 m to 11 m. The layer, which is considered as fill due to the presence of asphalt pieces, was grey and has a compact state of packing with an SPT N value of 29 blows per 0.3 m penetration.

3.2.3 Organic Clayey Silt

About 1.6 m of organic silty clay to clayey silt with some sand and trace gravel was encountered at a depth of about 11 m. This deposit, which transitioned from dark grey to brown grey with depth, had measured organic contents of 6% at a depth of 11.4 m and 2.6% at a depth of 12.2 m. This deposit had a stiff to very stiff consistency, with SPT N values ranging from 11 to 21 blows per 0.3 m of penetration. Measured water contents of the two samples were 19% within the higher organic content portion, and 15% at the base of this layer. The Atterberg Limits within this deposit also decreased with depth, with PL and LL values of 22% and 38% respectively near the top of the deposit, and 15% and 26% respectively at the bottom of the deposit.

3.2.4 Clayey Silt (Till)

A thin layer of clayey silt, with sand and trace gravel was encountered from a depth of 12.6 m to 13.4 m. This layer was mottled brown and grey, with a hard consistency having an SPT N value of 42 blows per 0.3 m of penetration. The water content of the one sample taken in this layer was about 9%.

3.2.5 Residual Soil

A clayey silt deposit was encountered from a depth of about 13.4 m (Elevation 139.1 m), through to the borehole termination depth of 14.2 m (elevation 138.3 m). The deposit contains trace sand and shale fragments and is considered to be formed through weathering of the underlying shale bedrock (ie. residual soil). This deposit was found to have a hard consistency, with SPT N values of two samples exceeding 100 blows per 0.3 m of penetration. Water contents of two samples

within this deposit measured 11% and 8%. The PL and LL values of a sample near the top of this layer were 15% and 24% respectively.

3.2.6 Groundwater Conditions

The water level within the borehole upon completion of drilling was found at a depth of 10.7 m (Elevation 141.8 m). This water level is within the silty sand layer encountered at the base of the fill described in Section 3.2.2 above.

4.0 EAST ABUTMENT AND APPROACH EMBANKMENT DESCRIPTION

4.1 Construction Details

The Highway 27 grade at the site is at Elevation 145.2 m, about 7 m below the Highway 401 grade (approximately Elevation 152.5m).

Based on correspondence with Terry Choo-Kang of MH, the approximate schedule for the construction works in the area of the SE Abutment is as shown in Table 1. As indicated on Figure 3, a temporary soldier pile and lagging wall (roadway protection) was installed to allow excavation for the new abutment footings and to ultimately retain the fill to be placed forming the approach embankment. The approximate 3 m of fill placed beneath the central “CE” portion of the existing bridge (north of the soldier piles, and supporting the scaffolding beneath this portion of the bridge), was placed to an Elevation of about 148.0 m.

It is understood that the soldier piles were to be installed on about 3 m centers with 4” x 6” timber lagging, however based on site photographs there appears to be considerable field variation with some soldier piles at spacings exceeding about 4 m. Excavation within the region of the SE Abutment was required for construction of the new abutment footings (PC 7 & PC 8), as well as for placement of the geogrid mat which was designed by MTO to provide suitable support above the existing sewer for the RSS walls. The memo dated 17 February 2003 prepared by MTO summarizing the design of this geogrid mat is included in Appendix B. This memo also includes sketches referencing the dimensions of the geogrid mat, as well as the locations of the deadmen and anchor rods associated with the soldier pile and lagging wall. The RSS walls (location shown on Figure 7) form the wing and front facing walls to the new east abutment, and extend along the south side of the Highway 401 from the new east abutment to the previous abutment location.

The geogrid mat foundation was installed between 25 February and 28 February 2003. Figure 3 shows schematically the approximate location of this geogrid mat based on review of the construction drawings and correspondence provided by MH. The approximate location of the borehole put down in November 2003 as part of this investigation (BH-1) is also shown on the figure. The borehole was positioned in order to be outside the newly constructed approach slab and the geogrid mat, and avoid the deadman anchors, but still focus on the area where the maximum surface settlement is occurring. This figure also highlights specific areas at the site which will be referenced in subsequent discussions on establishing the boundary where new fill was placed for the SE Abutment.

4.2 Settlement Information

Settlement data as provided by MH for this study is presented in Table 2. The settlement points are located on the north edge of the newly constructed approach slab as well as the concrete “base slab”, which extends from the approach slab to join in with the existing pavement structure at the previous east abutment location. Figure 4 illustrates the locations of the settlement points along the approach and base slab, and Figure 5 illustrates the settlement data versus time, with the key construction events noted. Although settlement may have occurred during and following the construction of the RSS wall and associated fill placement for the approach abutment, the date of “zero” settlement is taken as the dates when the base and approach slabs were completed. The actual start of settlement is likely to be anytime between the commencement of fill placement for the SE Abutment (circa 4 March 2003 for the RSS fill; 14 March 2003 for the general abutment fill) and the date of initial traffic loading (21 June 2003).

Figure 6 illustrates the settlement profile along the north edge of the eastbound lanes. Two separate patching/padding exercises since June 2003 have been carried out to maintain a suitable profile for traffic. It should also be emphasized that the settlement profile of the underlying soil is probably masked due to the fixity of the base and approach slabs on the old and new abutments respectively, plus the relative stiffness of these slabs. Hence, although the settlement trough is measured at the juncture between the base and approach slabs (performing as a hinge connection), the maximum settlement of the underlying soil may actually be occurring elsewhere.

5.0 INTERPRETATION OF DATA AND RESULTS

5.1 Volume of Settlement Trough

Figure 6 illustrates the settlement profile as measured at the survey points along the north edge of the new eastbound lanes. Based on site observations and discussions with MH staff, it is estimated that up to about 50 mm (approximately 30% of the settlement along the north edge of the eastbound lanes) of settlement has occurred along the south edge of the eastbound lanes. Assuming that the shape of the settlement trough along the south edge is similar to that at the north edge of the eastbound lane, the volume of the settlement trough at the road surface is estimated to be about 11 m³ as shown in Table 3. As previously mentioned, this settlement trough is a reflection of the movement behaviour of the base and approach slabs, which likely masks the true settlement profile (and calculated volume of settlement) of the underlying soils.

5.2 Deformation of the Roadway Protection

Some of the volume of the settlement trough may be attributed to the performance of the contractor's soldier pile and lagging support system (roadway protection) along the north edge of the newly constructed lanes. The three mechanisms associated with the contractor's roadway protection, which could have contributed to the surface settlement include:

1. Tilting of the soldier piles towards the center portion of the existing bridge upon initiation of traffic loading
2. Bowing of the lagging between the soldier piles
3. Flowing/washing of soil through the lagging

It is understood that the soldier piles were installed adjacent to, but not in contact with, the existing bridge. At present, however, the tops of the soldier piles are in contact with the adjacent bridge deck, and at the time of this report were apparently transferring load to the bridge deck given the reports of spalling/cracking of the concrete in contact with the soldier piles. It is further understood that the contractor did not tension the anchor rods of the deadmen within the fill behind the soldier piles until recently, some time after the SE section of the new Highway 401 overpass was constructed and open to traffic.

Calculations carried out to determine the potential volume of the settlement trough which may be attributed to the tilting of the soldier piles against the center portion of the existing bridge, plus bowing of the lagging, are shown in Table 4. The calculations assume that the soldier piles tilted towards the central portion of the existing bridge about 25 mm. The calculations also assume that

the lagging has bowed a maximum of 50 mm. Although this amount of bowing disagrees with the approximate 100 mm of bowing which was reported by MH staff, visual observations (notably after the contractor installed additional bracing) indicate that 50 mm of bowing may be more representative.

It is estimated that about 3.1 m^3 of the some 11 m^3 (some 25%) of the settlement trough may be attributed to tilting of the soldier piles and bowing of the lagging. Although information has not been obtained regarding the amount of soil which flowed through the lagging, based on site observations and photo records, it is possible that up to about 1 m^3 of material may have been lost in this manner. Hence, it is estimated that about 4 m^3 ($\approx 3.1 \text{ m}^3 + 1 \text{ m}^3$) or 36% of the 11 m^3 may be attributed to the contractor's roadway protection system installed along the north side of the newly constructed eastbound lanes.

The ensuing discussion presents information supporting that the remainder of the volume of the settlement trough (about 7 m^3) is attributed to settlement of the new fill for construction of the SE Abutment.

5.3 Delineation of Limits of New Fill for SE Abutment

5.3.1 Background Plans and Sections

Figure 7 shows the approximate location of the recently drilled Borehole BH-1 at the new east approach, the approximate limits of the RSS structural fill and the geogrid mat, and the existing Metro sewer alignment. The locations of two boreholes put down by others during a previous investigation in order to obtain information regarding the nature of the sewer trench backfill (borehole logs and associated lab test results by others are included in Appendix B) are also shown.

Figure 8 is a section along the southern face of the RSS wall illustrating the approximate location of the Metro sewer on this section, and the proposed benching for the RSS wall and fill placement.

Figure 9 is a section along the north side of the construction showing the sewer alignment in profile at this location, the schematic location and details of the previous east abutment, and the measured settlement profile along the north edge of the SE Abutment.

Figure 10 superimposes the location of the geogrid mat onto the above Figure 9 and also shows the interpreted excavated ground surface profile for the geogrid mat construction based on correspondence with MH, plus independent review of select construction photos. This figure also includes the profile of SPT N values for the recently drilled Borehole BH-1.

5.3.2 Estimated Lower Boundary of Newly Placed Fill

It is understood that the existing Metro sewer which crosses the new abutment alignment was installed in the mid 1960's. The sewer was reportedly installed using open trench techniques using temporary side slopes above wooden shoring supporting the lower portion of the excavation depth. During subgrade preparation for the current RSS wall construction, two boreholes were put down through the sewer trench backfill in order to obtain subsurface information for design of the geogrid mat (Appendix B).

Borehole BH-100 put down on the west side of the sewer encountered clayey silt fill from the ground surface of Elevation 144.7 m, to a depth of 6.6 m or Elevation 138.0 m. The fill within the upper 3 m was generally found to have a stiff to very stiff consistency, with SPT N values generally between 10 to 12 blows per 0.3 m of penetration, except one value of 21 blows per 0.3 m of penetration. From a depth of 3 m to 6 m, the fill displayed a firm consistency, with SPT N values measuring between 5 and 7 blows per 0.3 m of penetration. The higher SPT N values at shallower depths are likely reflective of the increased compaction effort applied to the fill as backfill proceeded and the trench width increased with increased elevation. From a depth of about 6 m (Elevation 138.6 m) through to 8.1 m (Elevation 136.6 m), the SPT N values exceeded 50 blows per 0.3 m penetration, which as noted on the borehole log from a depth of 6.6 m onwards reflects the wood and gravel probably associated with the shoring for the sewer trench construction.

Borehole BH-101 put down on the east side of the sewer main encountered clayey silt fill from the ground surface of Elevation 144.6 m, to a depth of 7.8 m or Elevation 136.9 m. The upper 3.5 m of this fill exhibited a firm to stiff consistency, with SPT N values ranging from 7 to 9 blows per 0.3 m of penetration, except one value of 30 blows per 0.3 m of penetration. From a depth of 3.5 m to 7.8 m the fill exhibited a variable consistency with SPT N values varying from 9 to over 50 blows per 0.3 m of penetration. It is believed that the high blow counts are associated with gravel, cobbles, and wood within the fill. Wood was first noticed in the fill from a depth of about 5 m (Elevation 139.6 m), which is about 1 m higher than observed in Borehole BH-101. Weathered shale was encountered in Borehole BH-101 from a depth of 7.8 m (Elevation 136.9 m) through to the termination depth of 135.4 m.

Based on the photograph of Figure 3, the contractor excavated a sufficient amount of fill within the region of the former foreslope, to expose a significant depth of the previous abutment face. Given that the lagging of the soldier pile wall at the time of the photograph in Figure 3 was constructed to Elevation 148 m, it is estimated that excavation immediately adjacent to the previous abutment proceeded to at least Elevation 147.5 m as shown in Figure 3.

It is understood through correspondence with MH, that after the geogrid mat was constructed with its top at about Elevation 143 m, a near vertical cut about 1.5 m in height was present at the east limit of the geogrid mat. Above this step, there was a gradual slope up to the old abutment. Given the estimated lip of the step at Elevation 144.5 m, coupled with the estimated excavation surface at the old abutment of about Elevation 147.5 m, the estimated boundary between the old fill for the former foreslopes, and new fill associated with this approach abutment, is as illustrated in Figure 10.

5.3.3 Influence of Existing Sewer

As shown on Figure 8, the existing Metro sewer alignment is located almost directly beneath the juncture between the approach slab and the concrete base slab along the south side of the eastbound lanes. As shown on Figures 7, 9 and 10 the sewer traverses beneath the east abutment at the location of one of the pile caps (footing PC 8). Although the Metro sewer and associated backfill could influence surface settlement along the south side to a greater extent than the north side of the eastbound lanes, little to no surface settlement, distress to the road surface or RSS panels is observed in this region. In addition, the settlement trough as evident at the road surface, although influenced by the fixity/rigidity of the base and approach slab, effectively diverges away from the sewer alignment.

Given the configuration of the sewer alignment in relation to the settlement trough, and the fact that little to no settlement is observed along the south edge of the eastbound lanes, it can be concluded that the geogrid mat is performing adequately, and the Metro sewer backfill as well as the underlying foundation materials are not controlling the observed settlement. Rather, the settlement is believed to be controlled by the newly placed fill bounded by the inside edge of the RSS wall to the south and west, the roadway protection to the north, and the previous abutment to the east.

5.3.4 Settlement of Newly Placed Fill

Up to 155 mm of settlement has occurred at the monitoring points located along the north limit of the newly constructed eastbound lanes. Prorating of the settlement (assuming linear from 155 mm at the north edge, to 0 mm at the south edge), it is estimated that about 120 mm of settlement could have occurred at the location of the recent Borehole BH-1. Settlement calculations have been carried out to assess the potential magnitude of settlement of the fill materials based on empirical relationships between the SPT N value and the elastic moduli for the soil encountered in the borehole (i.e. $SPT\ N\ value = E\ in\ MPa$). The new fill as encountered in the location of the borehole was divided into four layers, based on the SPT N values, with average stresses and average SPT N values calculated for each layer as shown in Table 5. Strains within each of the layers were determined based on the computed stress changes and elastic moduli from which the

settlement was calculated. As shown in Table 5, about 130 mm of settlement is estimated within the 8.5 m of newly placed fill materials as encountered in Borehole 1. Of this calculated settlement, 79 mm (or 61%) occurs within the lower 2 m of fill where SPT N values of 2 to 4 blows per 0.3 m of penetration were measured in the borehole. These low blow counts in the fill reflect compaction values of less than 95% Standard Proctor Density.

Two key points are worth highlighting with respect to the above assessment. Firstly this settlement calculation neglects the time component, and some of this settlement may have occurred during construction of the embankment and prior to casting of the base slab and approach slab. Secondly, empirical formulations for estimation of elastic moduli can vary significantly. Our selection of elastic moduli (E (MPa)/SPT N ratio = 1), is within the middle to upper range of a variety of published relationships. However, there are many references which quote E (MPa)/SPT N ratios much lower than unity (i.e. NAVFAC¹ quotes 0.4 to 0.7 for fine grained soils). It should be noted that selecting NAVFAC's upper value of 0.7 for the E (MPa)/SPT N ratio for fine grained soils would result in a 43% increase in the amount of settlement predicted.

It should also be highlighted that the above settlement calculation is based on a new newly placed thickness of about 8.5 m as encountered at the recent borehole. The fill thickness above the geogrid mat and beneath the approach slab measures about 9.5 m. Extrapolating the approximate 130 mm of settlement to a fill thickness of 9.5 m yields about 145 mm of settlement.

¹ NAVFAC, Soil Mechanics (DM 7.1), Naval Facilities Engineering Command, Alexandria, 1982, 355p.

6.0 CONCLUSIONS

Based on the above discussion, the following preliminary conclusions are provided:

1. The average SPT N value measured within the fill below Elevation 142 m (the base of the geogrid mat) in the two boreholes (BH-100 and BH-101) put down through the sewer backfill is 28 blows per 0.3 m of penetration. The fill had been in place since the mid 1960's and under the current construction configuration, has had greatest increase in load applied in the southern half of the eastbound lanes; east of the previous Pier No. 1 alignment area.
2. Settlement of the RSS wall forming the south wing wall has not been observed to date, even where the sewer traverses beneath the RSS wall (also coincident with the juncture between the base and approach slab). Based on this observation, it can be concluded that the geogrid mat is performing satisfactorily, and that the previously existing fill and underlying native soils within the area of the new east approach are not undergoing significant consolidation.
3. The performance of the contractor's roadway protection along the north face of the new east approach (tilting of soldier piles, bowing of lagging, flowing of soil through lagging) may have contributed about 4m³ or 36% of the estimated 11 m³ of the settlement trough volume.
4. Based on the correspondence provided by MH and the borehole information recently gathered, it is estimated that there is about 8.5 m of newly placed fill at the recently drilled borehole location. Based on empirical formulations, it is calculated that about 130 mm of settlement could be attributed to this 8.5 m thickness of fill. Considering that there is actually up to about 9.5 m of fill placed between the geogrid mat and the approach slab, and assuming similar overall consistency of the lower 1 m of fill materials as at the borehole, up to 145 mm of settlement (extrapolated) could be attributed to consolidation settlement of the newly placed fill materials. The majority of this settlement occurs within the zone of fill with SPT N values of 2 to 4 blows per 0.3 m.

5. Given the hypothesis of little to no settlement within the foundation fills and underlying native soils, the analysis suggests that the settlement observed at the road surface can be attributed to settlement of the recently placed fill, plus deflection of the contractor's roadway protection system.

Yours truly,

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TABLE 1
Summary of Construction Operations - East Abutment and Approach

Date(s)	Operation
Sept 2002 - Oct 10, 2002	Fill and scaffolding support placed beneath central portion of Highway 401 overpass
Oct 22, 2002	Traffic routed to center portion of bridge
Oct 22 – 1 Nov 2002	Southern third of previous Highway 401 overpass demolished
Nov 1- Nov 20, 2002	Soldier piles installed for support system along the north face
Nov 21, 2002 - Feb 03, 2003	Excavation for pier foundations and RSS wall footprint
Feb 25 – Feb 28, 2003	Install Geogrid foundation in RSS footprint over old sewer trench
Mar 4 – Mar 21, 2003	Construct RSS abutment and wing wall to full height
Mar 14 – Apr 2, 2003	Place fill between RSS walls and previous abutment
Mar 27, 2003	Place concrete for abutment beam cap
May 13, 2003	Cast east approach slab
May 27, 2003	Place concrete base to east of approach slab
June 16, 2003	Pave bridge and approaches
June 21, 2003	Switch traffic to newly constructed EBL bridge and road
July 9, 2003	Settlement observed
July 11, 2003	Settlement monitoring begins
July 10, 2003	Bowing of and seepage through lagging of soldier pile wall
July 16, 2003	Begin installing cross-bracing to lagging

TABLE 2
Summary of Settlement Data Along North Edge of New East Approach


Point	Station		O/S from CL	Proposed Top of Concrete (m)	Date		Date		Date		Date		Date		Date		Date	
					11-Jul-03	Difference	14-Jul-03	Difference	15-Jul-03	Difference	16-Jul-03	Difference	17-Jul-03	Difference	18-Jul-03	Difference	21-Jul-03	Difference
					(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
O/S Zone of Influence (4 m east of point 1)	11+	542.01																
1	11+	538.01	8.608	152.344	152.337	-7	152.330	-14	152.331	-13	152.335	-9	152.325	-19	152.329	-15	152.330	-14
2	11+	535.03	8.626	152.334	152.300	-34	152.296	-38	152.298	-36	152.304	-30	152.297	-37	152.296	-38	152.293	-41
3	11+	532.04	8.624	152.326	152.243	-83	152.240	-86	152.240	-86	152.244	-82	152.240	-86	152.243	-83	152.236	-90
4	11+	529.05	8.644	152.315	152.204	-111	152.201	-114	152.201	-114	152.194	-121	152.191	-124	152.188	-127	152.188	-127
5	11+	528.05	8.587	152.308	152.196	-112	152.195	-113	152.194	-114	152.187	-121	152.187	-121	152.184	-124	152.182	-126
6	11+	525.04	8.630	152.300	152.257	-43	152.256	-44	152.255	-45	152.258	-42	152.251	-49	152.248	-52	152.248	-52
7	11+	522.02	8.637	152.294	152.307	13	152.305	11	152.306	12	152.306	12	152.303	9	152.300	6	152.297	3
Point	Station		O/S from CL	Proposed Top of Concrete (m)	Date		Date		Date		Date		Date		Date		Date	
					23-Jul-03	Difference	24-Jul-03	Difference	25-Jul-03	Difference	28-Jul-03	Difference	29-Jul-03	Difference	13-Aug-03	Difference	15-Aug-03	Difference
					(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
O/S Zone of Influence (4 m east of point 1)	11+	542.01																
1	11+	538.01	8.608	152.344	152.329	-15	152.327	-17	152.329	-15	152.331	-13	152.331	-13	152.330	-14	152.330	-14
2	11+	535.03	8.626	152.334	152.290	-44	152.290	-44	152.290	-44	152.292	-42	152.292	-42	152.292	-42	152.292	-42
3	11+	532.04	8.624	152.326	152.234	-92	152.232	-94	152.232	-94	152.235	-91	152.234	-92	152.235	-91	152.234	-92
4	11+	529.05	8.644	152.315	152.185	-130	152.182	-133	152.184	-131	152.184	-131	152.183	-132	152.182	-133	152.183	-132
5	11+	528.05	8.587	152.308	152.182	-126	152.175	-133	152.176	-132	152.182	-126	152.182	-126	152.180	-128	152.182	-126
6	11+	525.04	8.630	152.300	152.248	-52	152.243	-57	152.243	-57	152.248	-52	152.249	-51	152.246	-54	152.245	-55
7	11+	522.02	8.637	152.294	152.300	6	152.293	-1	152.299	5	152.300	6	152.299	5	152.301	7	152.300	6
Point	Station		O/S from CL	Proposed Top of Concrete (m)	Date		Date		Date		Date		Date		Date		Date	
					11-Sep-03	Difference	18-Sep-03	Difference	25-Sep-03	Difference	3-Oct-03	Difference	9-Oct-03	Difference	30-Oct-03	Difference	18-Nov-03	Difference
					(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
O/S Zone of Influence (4 m east of point 1)	11+	542.01																
1	11+	538.01	8.608	152.344	152.330	-14	152.330	-14	152.329	-15	152.327	-17	152.326	-18	152.321	-23	152.315	-29
2	11+	535.03	8.626	152.334	152.295	-39	152.295	-39	152.293	-41	152.291	-43	152.290	-44	152.282	-52	152.281	-53
3	11+	532.04	8.624	152.326	152.235	-91	152.236	-90	152.235	-91	152.233	-93	152.230	-96	152.223	-103	152.216	-110
4	11+	529.05	8.644	152.315	152.181	-134	152.182	-133	152.180	-135	152.176	-139	152.175	-140	152.162	-153	152.160	-155
5	11+	528.05	8.587	152.308	152.180	-128	152.181	-127	152.180	-128	152.176	-132	152.174	-134	152.163	-145	152.163	-145
6	11+	525.04	8.630	152.300	152.242	-58	152.241	-59	152.240	-60	152.244	-56	152.243	-57	152.243	-57	152.240	-60
7	11+	522.02	8.637	152.294	152.300	6	152.299	5	152.300	6	152.301	7	152.301	7	152.299	5	152.301	7


TABLE 3
Estimate of Volume of Surface Settlement Trough

Assuming Settlement at South Edge of EBL is 50 mm (2") or 30% of Corresponding Settlement Pins on North Edge

Point	Distance	Settlement		X-Sectional Area	Average Volume
		"Actual" Northern Shoulder EBL, 18 November 2003 (m)	"Estimated" Southern Travelled EBL, "12.42 m Away" (m)		
				m ²	m ³
0	19.988	0.000	0	0.00	
1	15.988	-0.029	-0.0087	-0.23	-0.47
2	13.016	-0.053	-0.0159	-0.43	-0.98
3	10.020	-0.110	-0.033	-0.89	-1.97
4	7.034	-0.155	-0.0465	-1.25	-3.19
5	6.036	-0.145	-0.0435	-1.17	-1.21
6	3.025	-0.060	-0.018	-0.48	-2.49
7	0.000	0.007	0.0021	0.06	-0.65
Total Volume					-11.0

TABLE 4
Estimate of Volume of Ground Loss Deflection of Soldier Piles & Lagging

Rotation of Soldier Piles								
Schematic 	Pile Number	Lateral Distance (design) (m)	Comments	Estimated Height of Soldier Pile above Excavation for Geogrid Mat Construction (m)	Deflection at Top of Soldier Pile (mm)	Deflection at Bottom of Soldier Pile (mm)	Area m ²	Volume m ³
	4	0	Edge of previous abutment	3.5	25	0	0.044	
	5	3		4.5	25	0	0.056	0.150
	6	6		6.0	25	0	0.075	0.197
	7	9		6.5	25	0	0.081	0.234
	8	12		8.0	25	0	0.100	0.272
	9	15		8.0	25	0	0.100	0.300
	10	18		8.0	25	0	0.100	0.300
	11	21	At pilecap	8.0	25	0	0.100	0.300
	Volume Through Deflection of Soldier Piles							

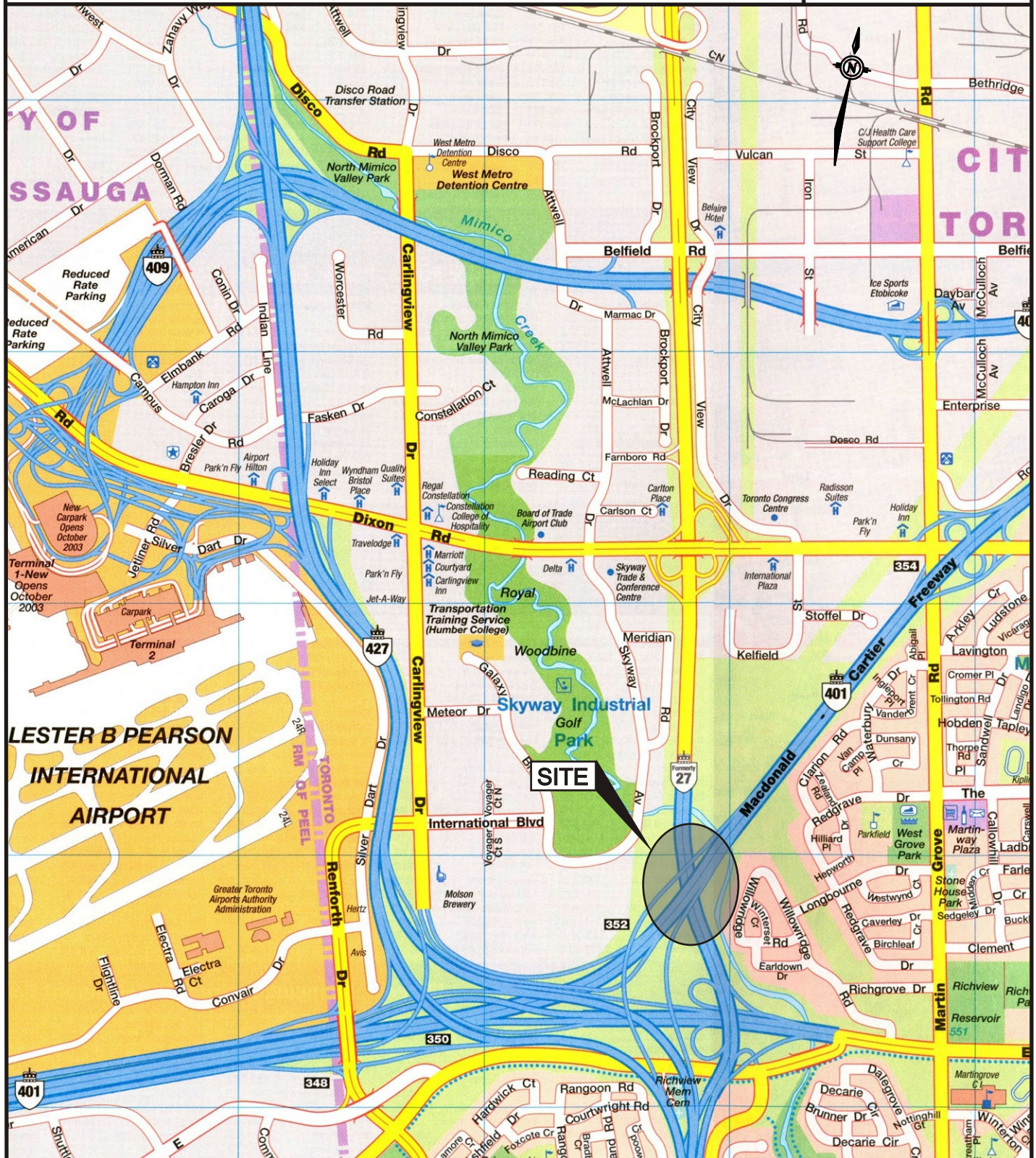
Bowling of Lagging								
Schematic 	Pile Number	Lateral Distance (design) (m)	Comments	Estimated Height of Soldier Pile above Excavation for Geogrid Mat Construction (m)	Exposed Height of Lagging (m)	Assumed Outward Bowling Between Piles (mm)	Plan Area of Deflection m2	Volume of Deflection Between Soldier Piles m3
	4	0	Edge of previous abutment	2.0	2	0	0.000	0.000
	5	3		4.0	4	10	0.015	0.045
	6	6		6.5	4	30	0.045	0.180
	7	9		6.5	4	50	0.075	0.300
	8	12		6.5	4	50	0.075	0.300
	9	15		6.5	4	50	0.075	0.300
	10	18		6.5	4	30	0.045	0.180
	11	21	At pilecap	6.5	4	10	0.015	0.060
	Volume Through Bowling of Lagging							

Total Resulting Volume of Through Deflection & Bowling 3.12

TABLE 5
Estimate of Settlement Due to New Fill Placement Plus Traffic Loading

LAYER	DEPTH	Comment	THICKNESS	DENSITY	AVERAGE N	Δ STRESS IN LAYER DUE TO NEW FILL	Δ STRESS DUE TO TRAFFIC LOADING	TOTAL CHANGE IN STRESS	COMPUTED ELASTIC MODULUS (SPT N = E)	COMPUTED STRAIN ϵ	COMPUTED SETTLEMENT
	(m)		(m)	(KN/m3)	(kPa)	(kPa)	(kPa)	(kPa)	(Mpa)		(mm)
Layer 1	0 to 1.8	Sand & Gravel (New Fill)	1.80	21	17	19	10	29	17	0.2%	3
Layer 2	1.83 to 5.48	Clayey Silt (New Fill)	3.65	21	11	76	10	86	11	0.8%	29
Layer 3	5.48 to 7.31	Clayey Silt (New Fill)	1.83	21	3	134	10	144	3	4.3%	79
Layer 4	7.31 to 8.53	Clayey Silt (New Fill)	1.22	21	12	166	10	176	12	1.5%	19
										Total	129

/document



REFERENCE:
THIS FIGURE WAS CREATED FROM A MAPART
MAP BOOK TITLED "TORONTO & AREA"
2004 EDITION.

0 .5 1 KILOMETRE

Date: **JANUARY 2004**

Project: **03-1111-032**

Golder Associates

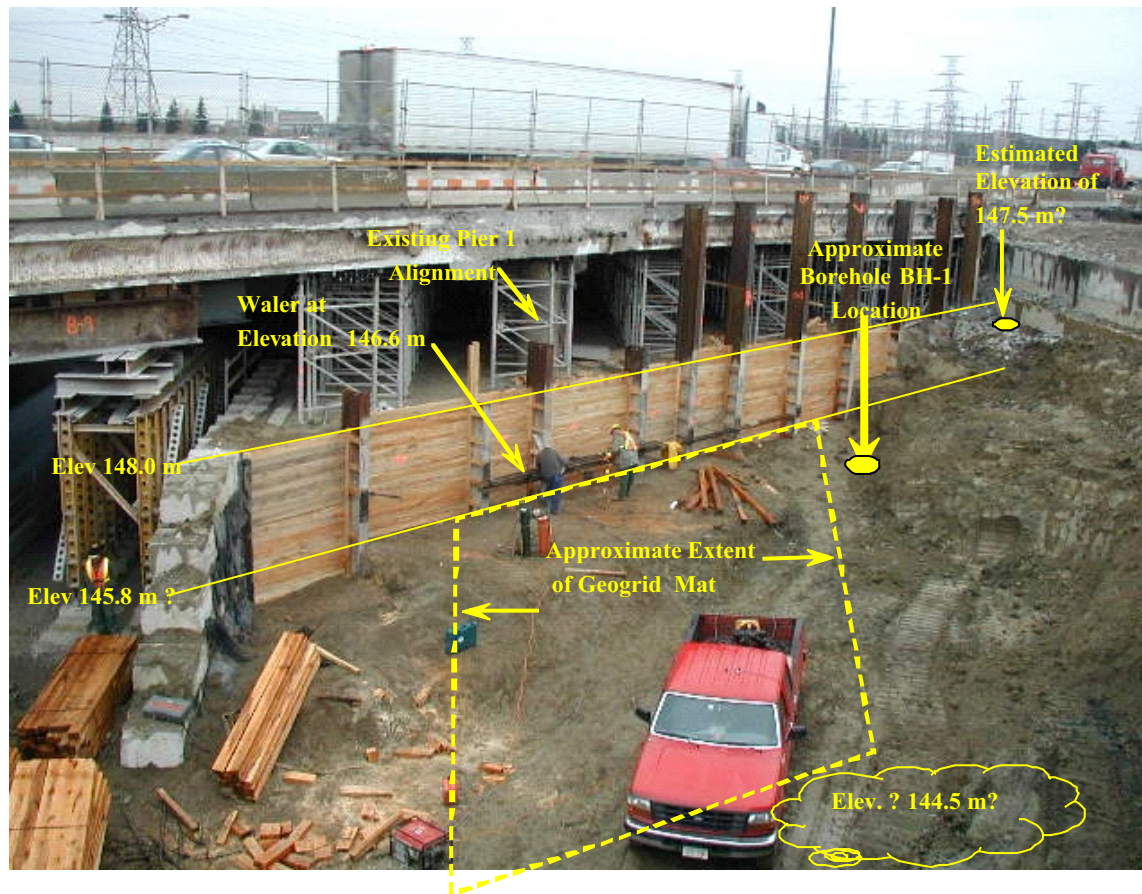
Drawn: **RJ**

Chkd: **RPS**



View of SE Abutment Area Following Installation of
Soldier Piles and Prior to SE Abutment Footing Excavation
(Looking Northeast)

FIGURE 3



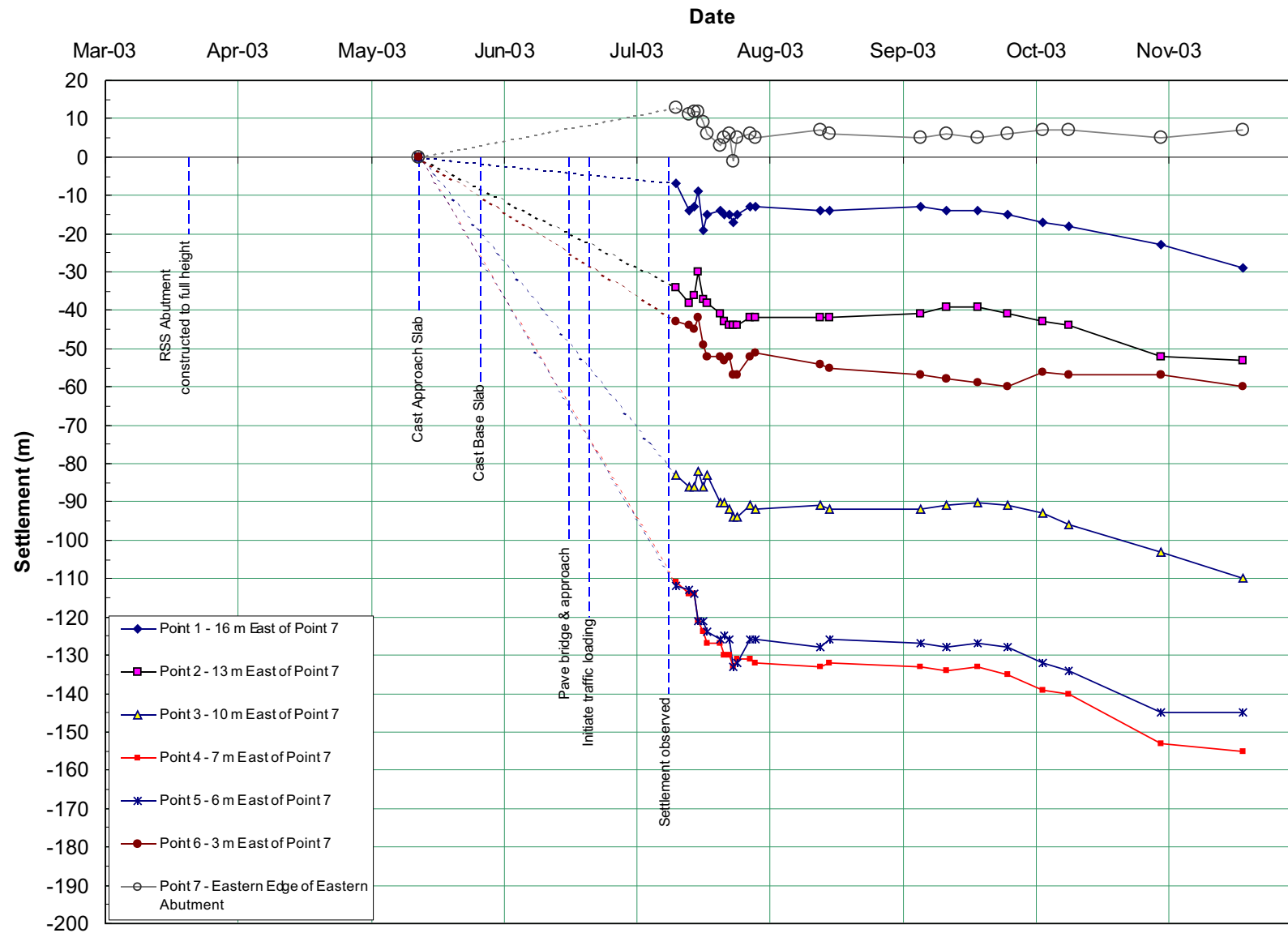
Location of Settlement Points Along Line of Soldier Piles
(Looking East)

FIGURE 4



Settlement Data Versus Time

FIGURE 5



Date JANUARY 2004

Project 03-1111-032

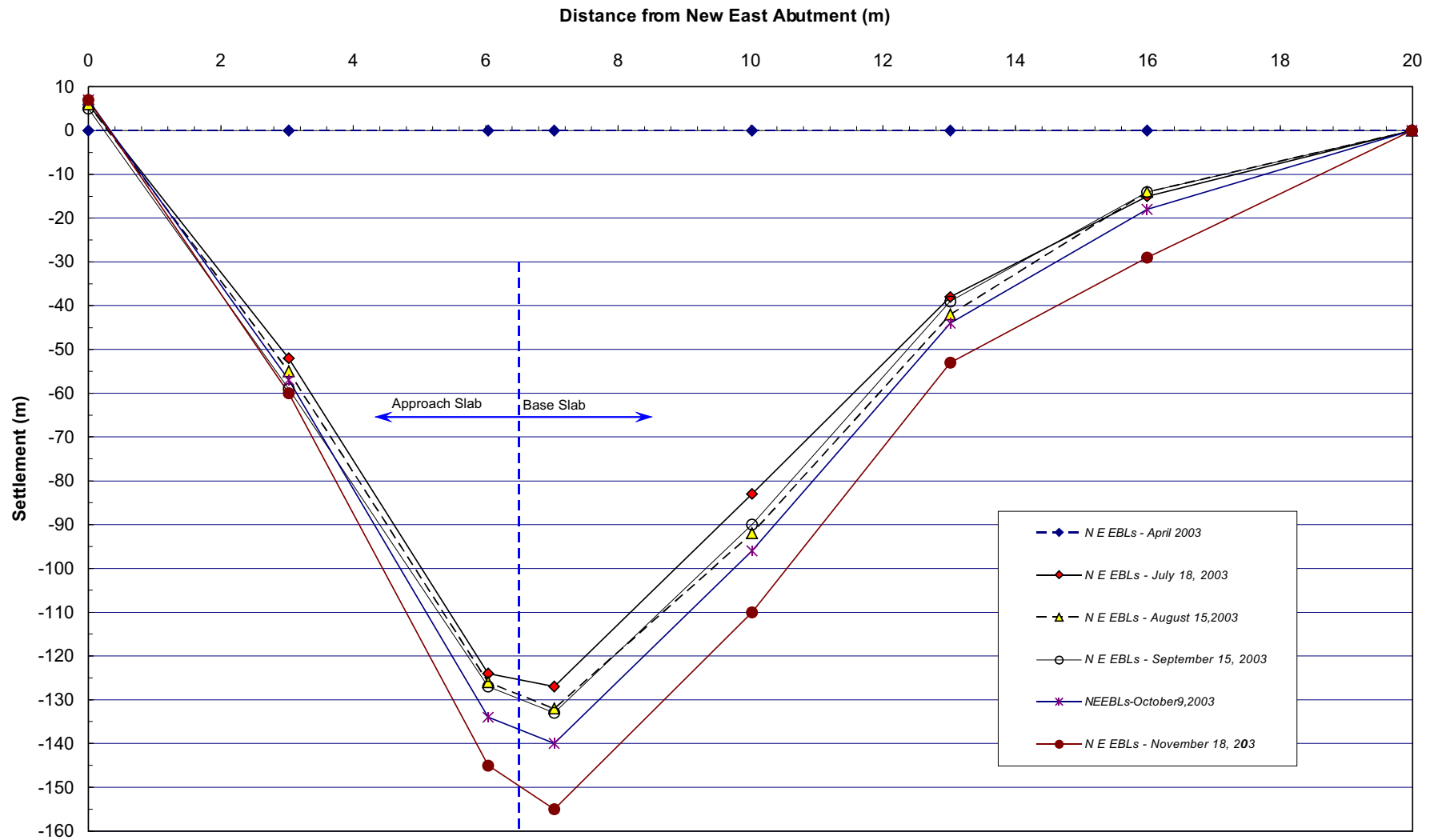
Golder Associates

Drawn RJ

Chkd. RPS

Settlement Trough North Edge of New Eastbound Lanes

FIGURE 6



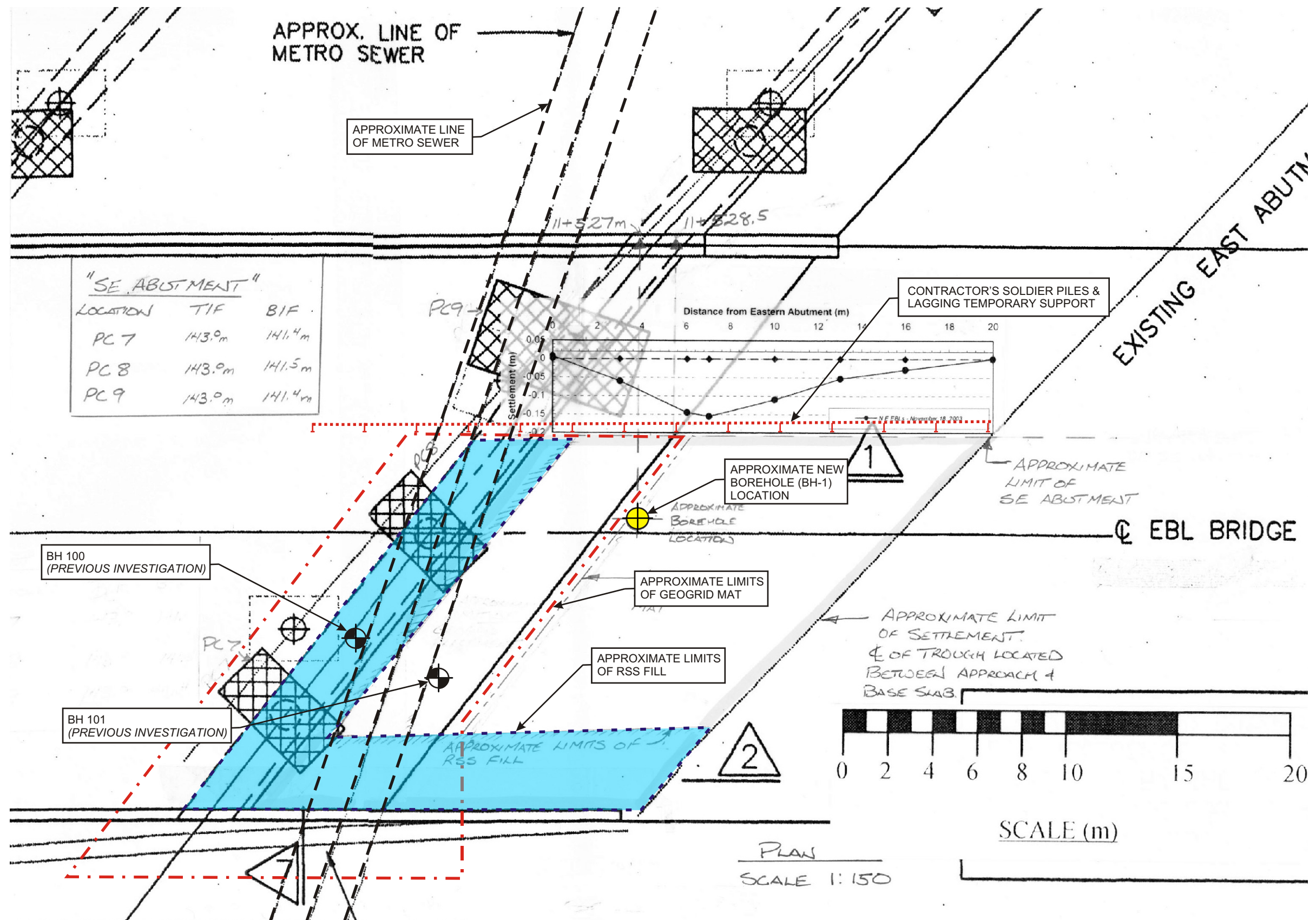
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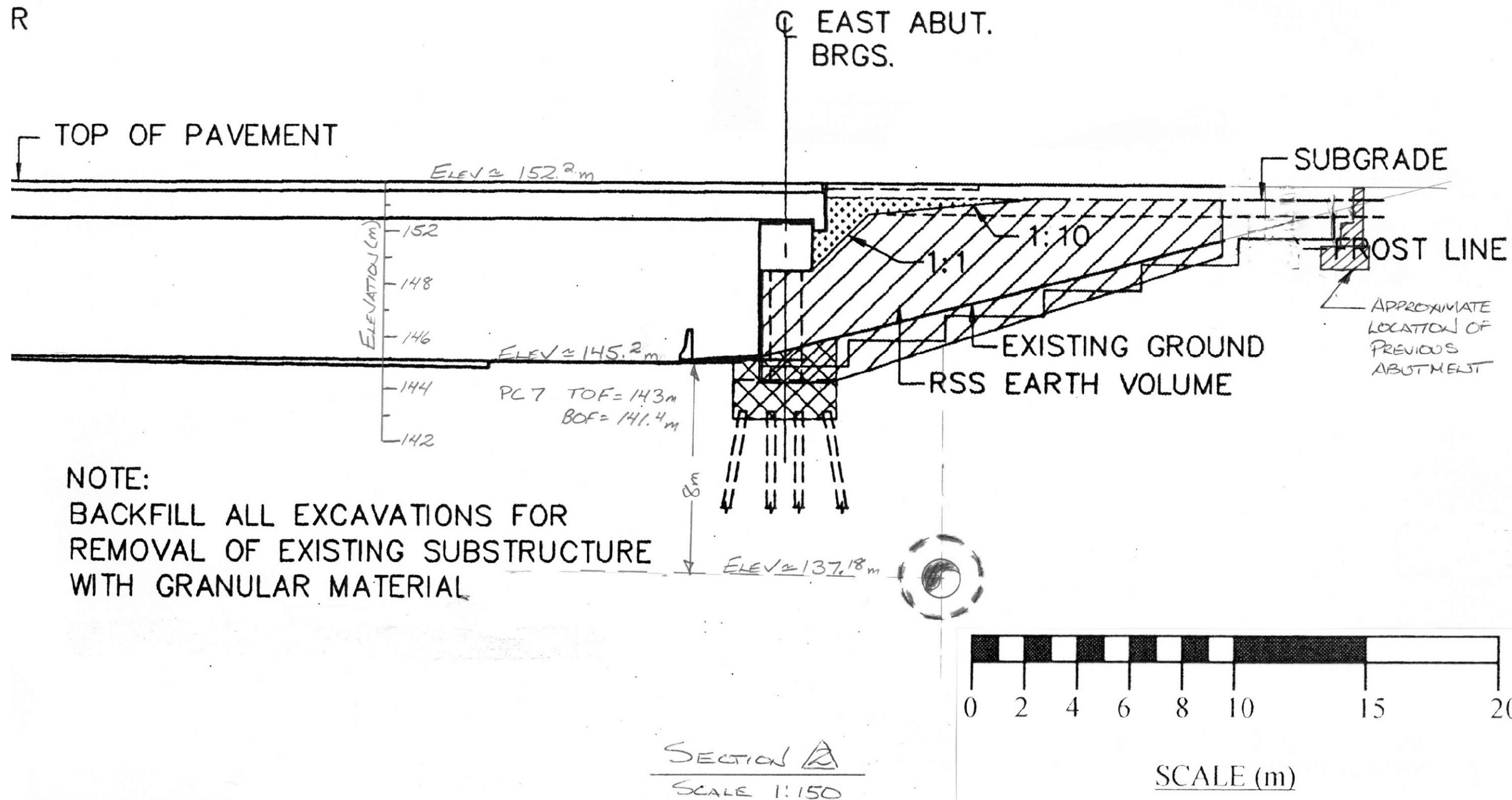
Project 03-1111-032

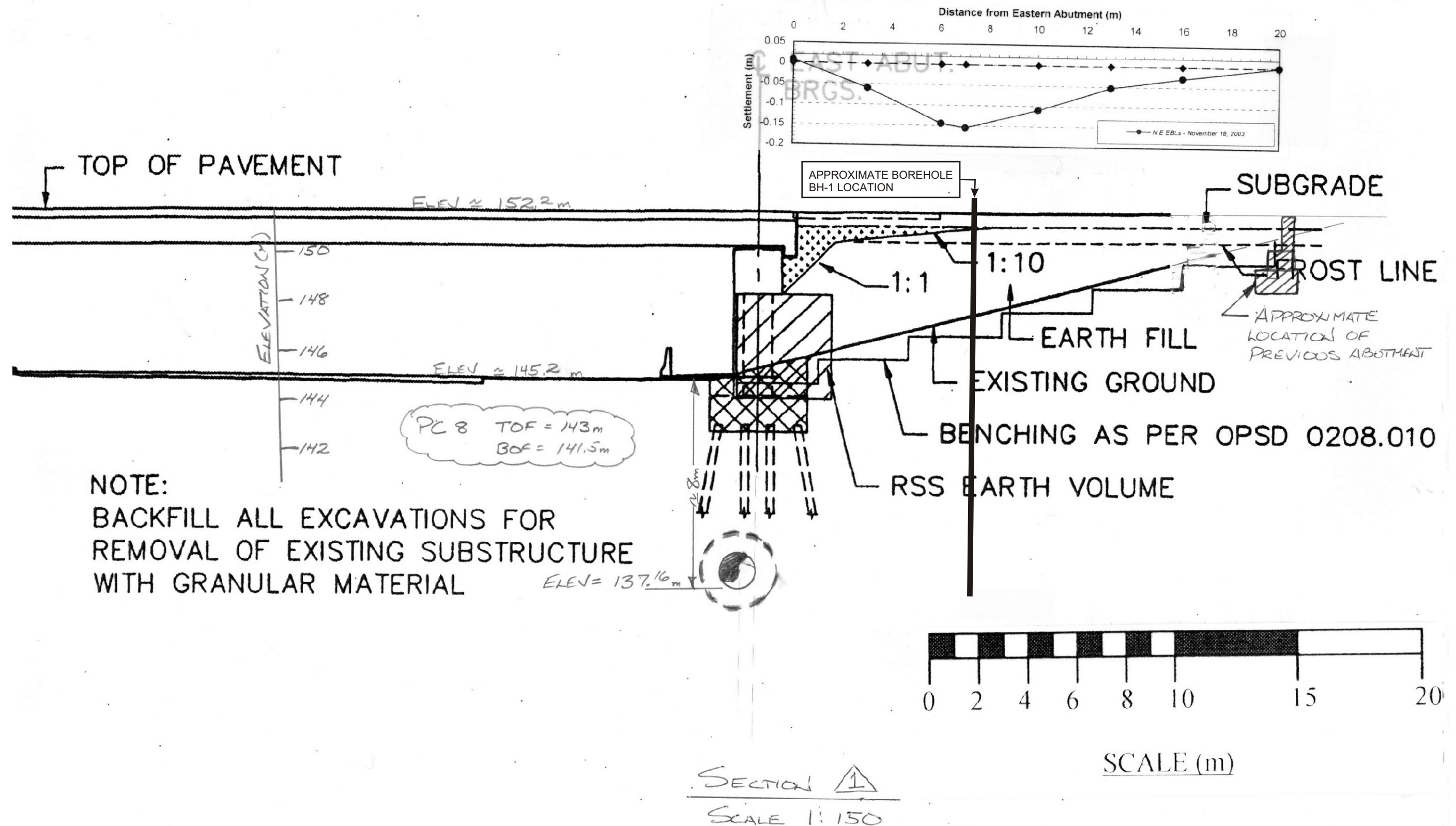
Golder Associates

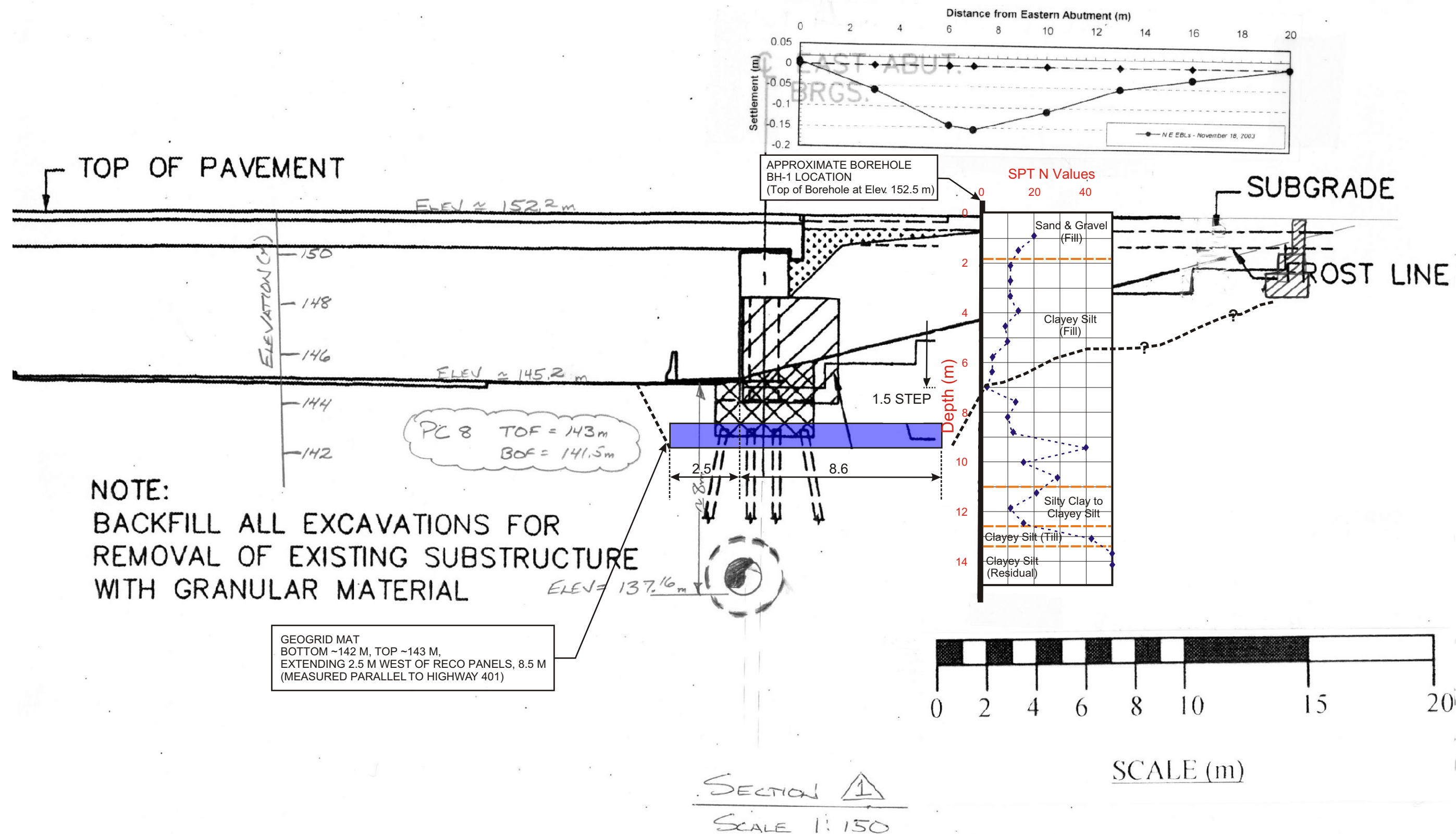
Drawn RJ

Chkd. RPS









APPENDIX A

RECORD OF BOREHOLE LOG (BH-1)
AND LABORATORY TEST RESULTS

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Consistency

	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength = (compressive strength)/2
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

PROJECT <u>03-1111-032</u>		RECORD OF BOREHOLE No 1		1 OF 2	METRIC
W.P. _____		LOCATION <u>STN 11+527 o/s 12 m south from center-line of bridge structure</u>		ORIGINATED BY <u>PKS</u>	
DIST _____ HWY <u>27</u>		BOREHOLE TYPE <u>Truck mounted</u>		COMPILED BY <u>KG</u>	
DATUM <u>Geodetic</u>		DATE <u>November 14, 2003</u>		CHECKED BY <u>RS</u>	

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 (%)			
								○ UNCONFINED	+ FIELD VANE							● QUICK TRIAXIAL	× REMOULDED	
152.5	GROUND SURFACE						20	40	60	80	100							
0.0	Asphalt																	
	Concrete Base																	
0.4	Sand and Gravel, trace silt (FILL)																	
	Compact		1	SS	20													
	Brown		2	SS	24													
	Moist																	
150.7																		
1.8	Clayey Silt, some sand and gravel (FILL)		3	SS	11													
	Stiff becoming soft/ very soft between 5.5 and 7.3 m depth		4	SS	11													
	Brown		5	SS	11													
	Moist to wet between 5.5 and 7.3 m depth		6	SS	14													
			7	SS	9													
			8	SS	10													
			9	SS	4													
			10	SS	4													
			11	SS	2													
			12	SS	13													
			13	SS	10													
144.0																		
8.5	Clayey Silt with sand, trace gravel and asphalt pieces (FILL)		14	SS	12													
	Stiff to very stiff		15	SS	40													
	Brown to grey		16	SS	16													
	Moist																	
142.1																		
10.4	Silty Sand, trace gravel and asphalt pieces (FILL)		17	SS	29													
	Compact																	
141.5	Grey																	
11.0	Moist		18	SS	21													
	Silty Clay to Clayey Silt, some sand, trace rootlets		19	SS	11													
	Stiff to very stiff																	
	Grey / black																	
	Moist		20	SS	16													
139.9																		
12.6	Clayey Silt with sand, trace gravel (TILL)		21	SS	42													
	Hard																	
	Brown to grey																	
	Moist		22	SS	25/0.39													
139.1																		
13.4	Clayey Silt, trace sand with shale (RESIDUAL SOIL)																	
	Hard		23	SS	100/0.15													
138.3	Grey																	
14.2	Moist																	

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MISS_MTO_031111032.GPJ ON_MOT_GDT_27/1/04

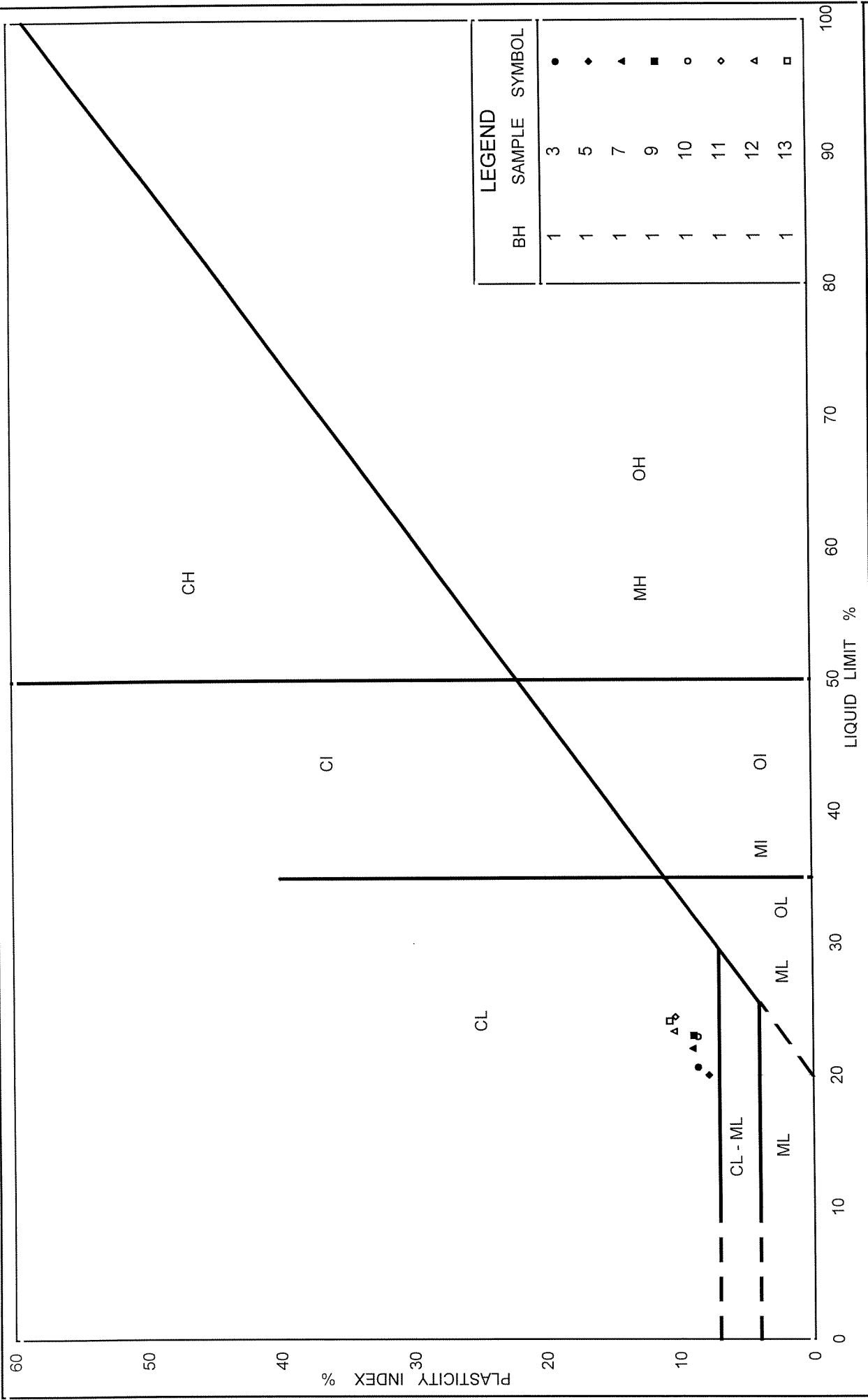
PROJECT <u>03-1111-032</u>	RECORD OF BOREHOLE No 1	2 OF 2	METRIC
W.P. _____	LOCATION <u>STN 11+527 o/s 12 m south from center-line of bridge structure</u>	ORIGINATED BY <u>PKS</u>	
DIST _____ HWY <u>27</u>	BOREHOLE TYPE <u>Truck mounted</u>	COMPILED BY <u>KG</u>	
DATUM <u>Geodetic</u>	DATE <u>November 14, 2003</u>	CHECKED BY <u>RS</u>	

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					W _p	W	W _L							
									○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)						
	--- CONTINUED FROM PREVIOUS PAGE ---									20	40	60	80	100		10	20	30		
	End of Borehole Notes: 1. Water level in open hole at Elev. 141.8 m upon completion of drilling operations																			

MISS_MTO_031111032.GPJ ON_MOT_GDT 27/1/04

TABLE 1**SUMMARY OF WATER CONTENT DETERMINATIONS**

PROJECT NUMBER		031-111032			
PROJECT NAME		MH / Hwy 427 Distress / Toronto			
DATE TESTED		November, 2003			
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
1	2	4.0-6.0	1.22-1.83	6.8%	
1	3	6.0-8.0	1.83-2.44	8.7%	LL=20.6, PL=12.0, PI=8.6
1	4	8.0-10.0	2.44-3.05	10.3%	Organics
1	5	10.0-12.0	3.05-3.66	9.5%	LL=20.0, PL=12.2, PI=7.8
1	6	12.0-14.0	3.66-4.27	10.5%	
1	7	14.0-16.0	4.27-4.88	11.1%	LL=22.0, PL=13.0, PI=9.0
1	8	16.0-18.0	4.88-5.49	12.7%	
1	9	18.0-20.0	5.49-6.10	11.3%	LL=23.0, PL=14.1, PI=8.9
1	10	20.0-22.0	6.10-6.71	11.9%	LL=22.9, PL=14.3, PI=8.6
1	11	22.0-24.0	6.71-7.32	13.1%	LL=24.4, PL=14.1, PI=10.3
1	12	24.0-26.0	7.32-7.92	12.2%	LL=23.3, PL=12.9, PI=10.4
1	13	26.0-28.0	7.92-8.53	13.3%	LL=24.1, PL=13.4, PI=10.7
1	14	28.0-30.0	8.53-9.14	12.5%	LL=24.5, PL=13.5, PI=11.0
1	15	30.0-32.0	9.14-9.75	9.2%	
1	16	32.0-34.0	9.75-10.36	10.8%	
1	17	34.0-36.0	10.36-10.97	7.1%	
1	18	36.0-38.0	10.97-11.58	19.1%	LL=38.8, PL=21.7, PI=17.1
1	19	38.0-40.0	11.58-12.19	20.5%	LL=26.2, PL=16.2, PI=10.0
1	20	40.0-42.0	12.19-12.80	14.9%	LL=19.1, PL=12.4, PI=6.7
1	21	42.0-44.0	12.80-13.41	9.1%	
1	22	44.0-46.0	13.41-14.02	10.9%	LL=24.0, PL=14.5, PI=9.5
1	23	46.0-46.5	14.02-14.17	7.7%	



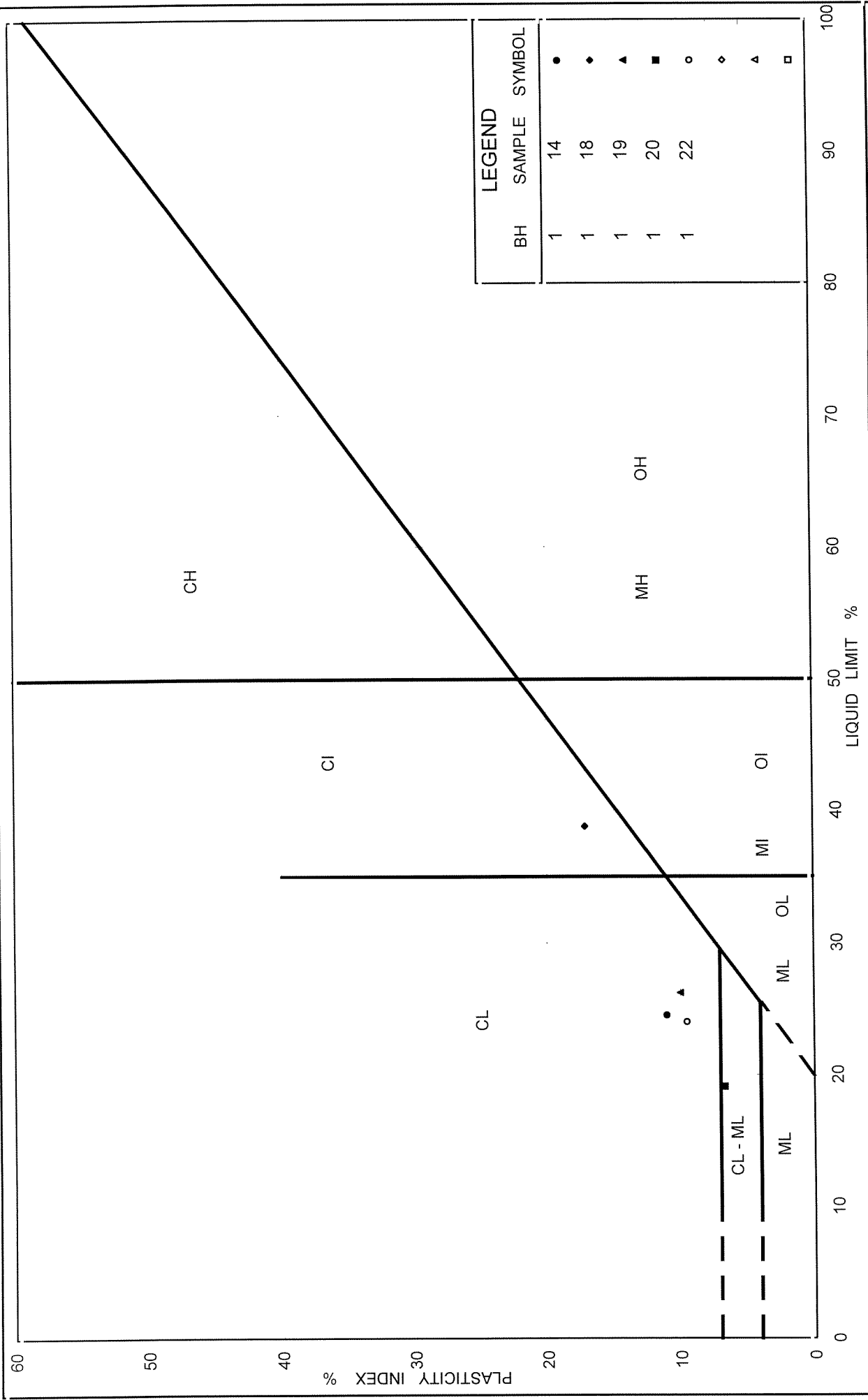


FIG No.

PLASTICITY CHART

Ministry of Transportation

Project No. 031-11032



Ontario

TABLE 1
SUMMARY OF ORGANIC CONTENT DETERMINATIONS

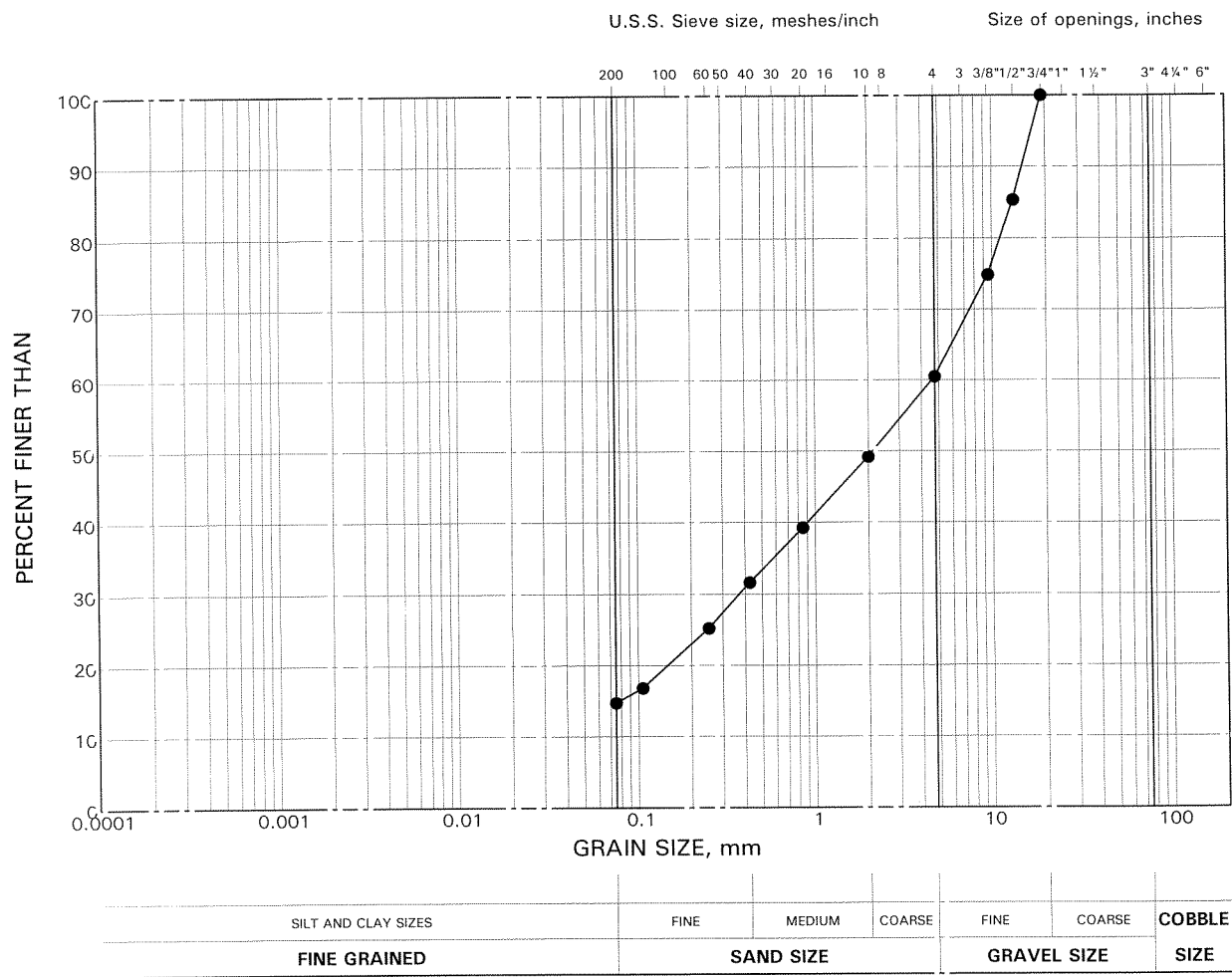
PROJECT NUMBER 031-111032				
PROJECT NAME MH / Hwy 427 Distress / Toronto				
DATE TESTED November, 2003				
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Organic Content (%)
1	18	36.0-38.0	10.97-11.58	6.0
1	19	38.0-40.0	11.58-12.19	2.6

Notes:

1. Samples dried at 110 degree centigrade prior to testing.
2. Test performed according to ASTM D2974-87 Standard, test method C.
3. Organic matter determined by burning the oven dried samples in a muffle furnace at 440 degree centigrade.

GRAIN SIZE DISTRIBUTION

FIGURE

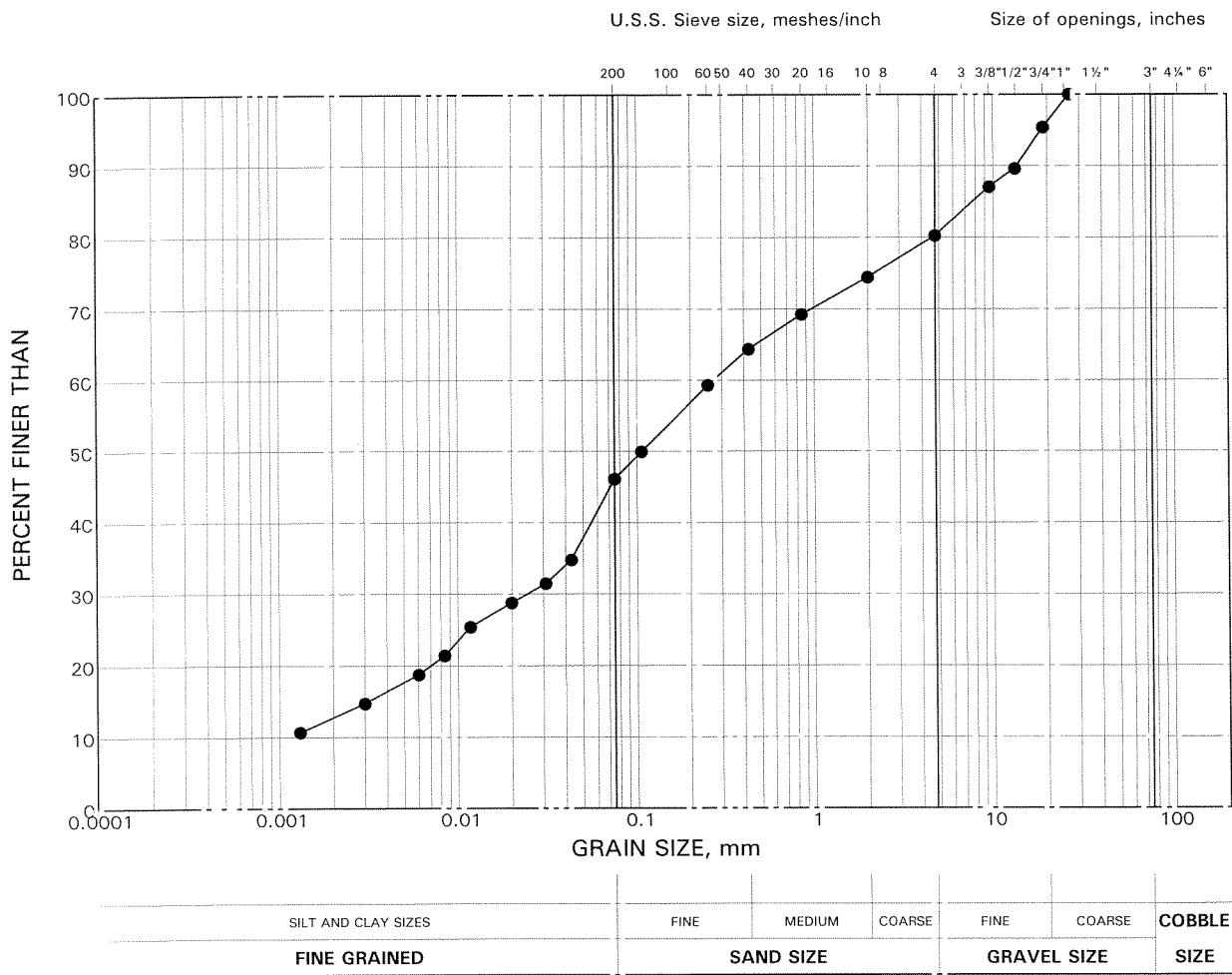


LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	1	2	1.2-1.8

GRAIN SIZE DISTRIBUTION

FIGURE

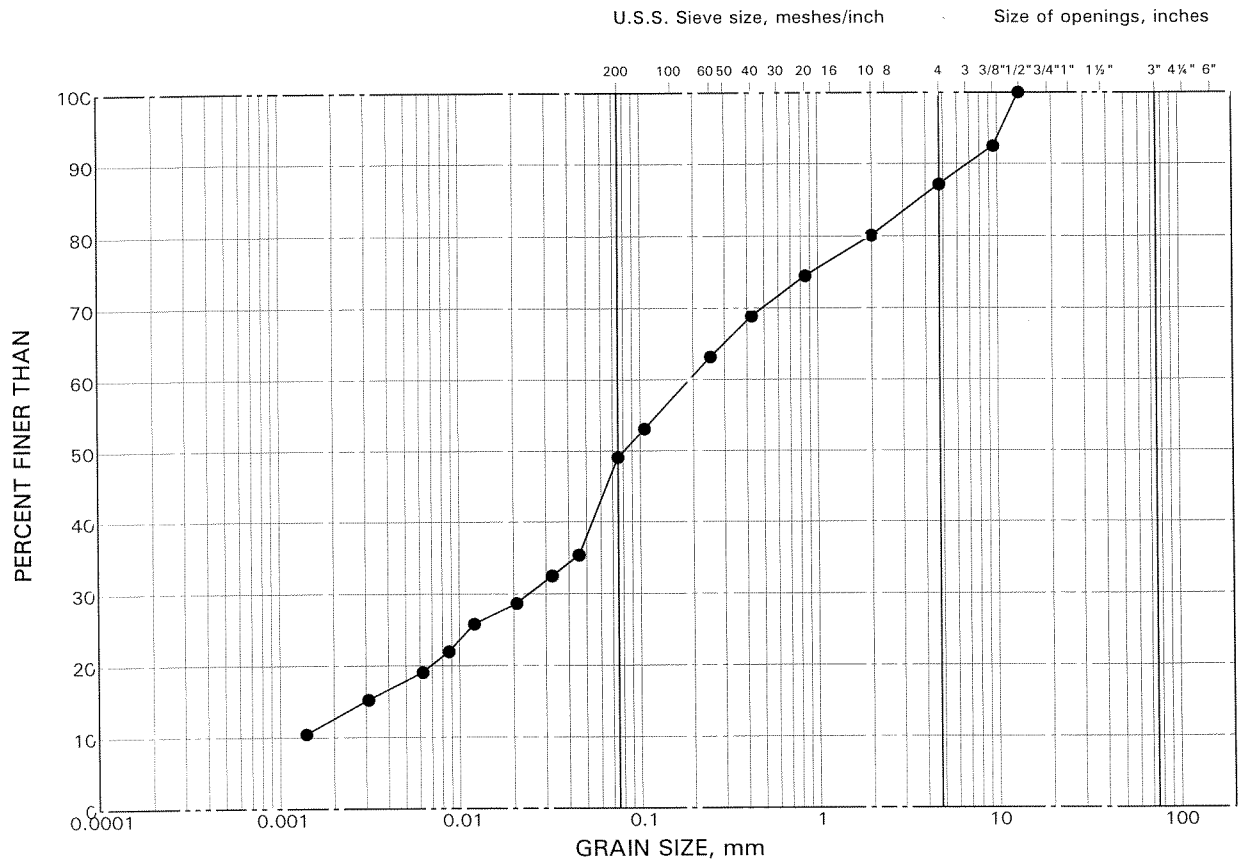


LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	1	7	4.3-4.9

GRAIN SIZE DISTRIBUTION

FIGURE



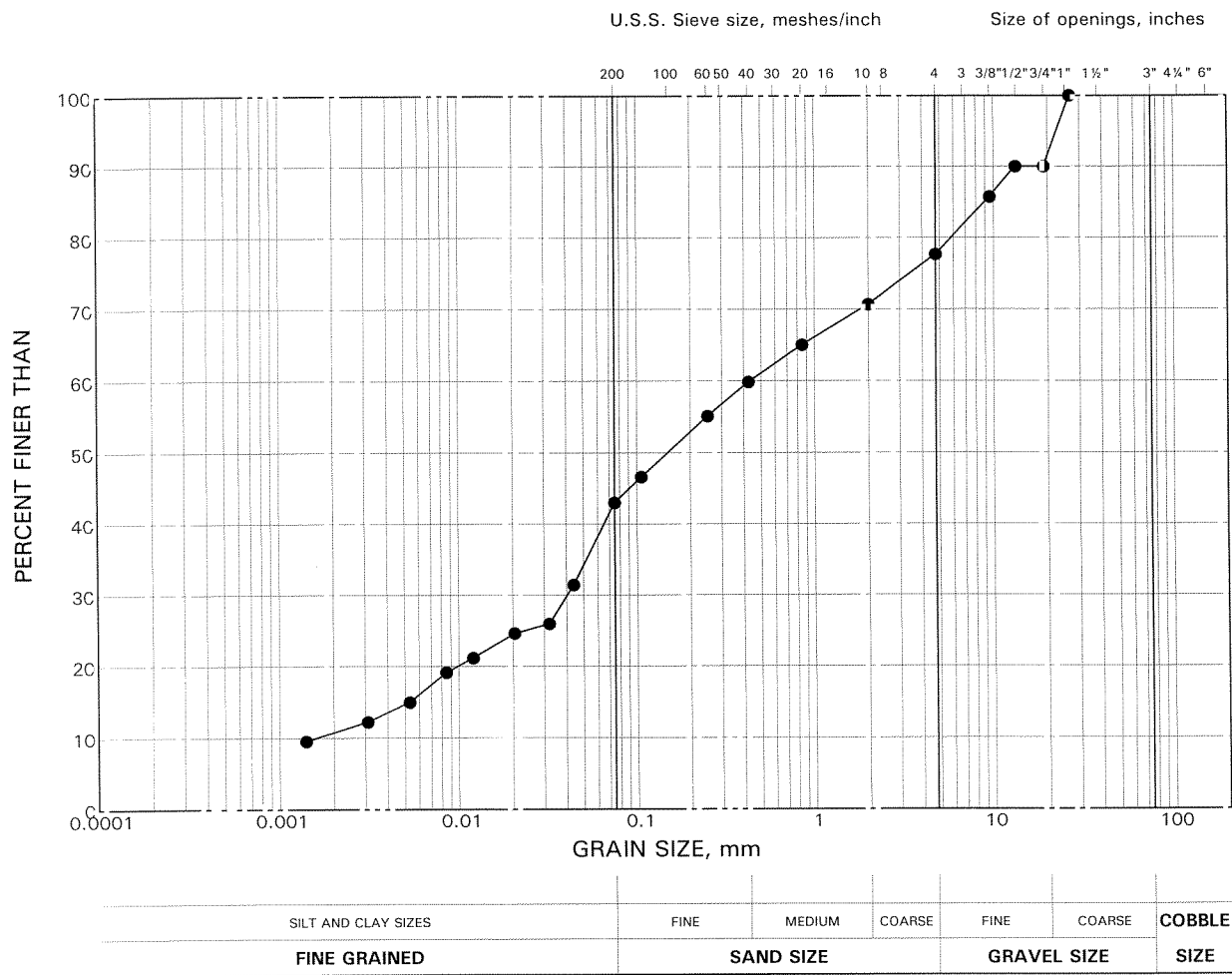
SILT AND CLAY SIZES		FINE		MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE				GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	1	10	6.1-6.7

GRAIN SIZE DISTRIBUTION

FIGURE

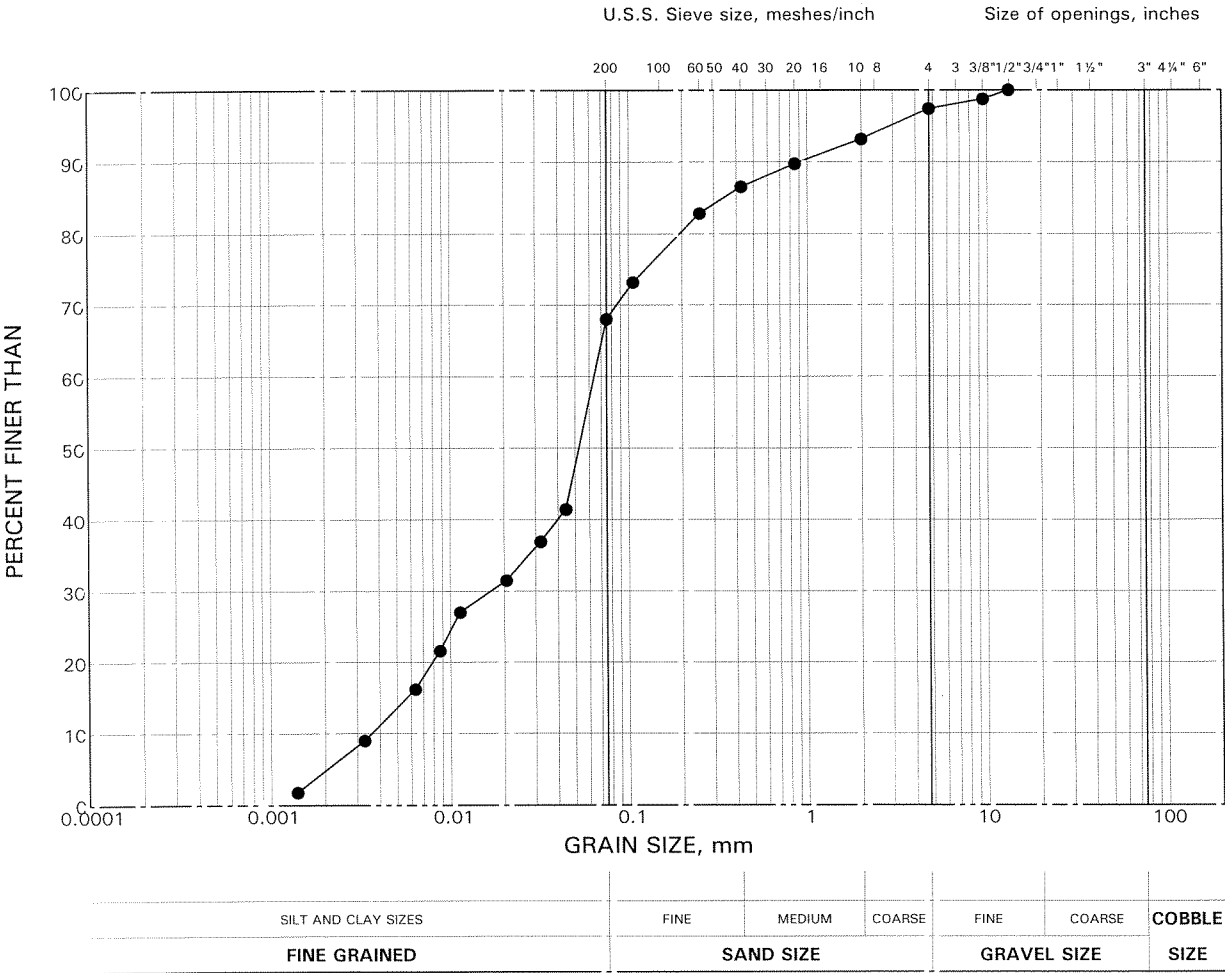


LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	1	12	7.3-7.9

GRAIN SIZE DISTRIBUTION

FIGURE

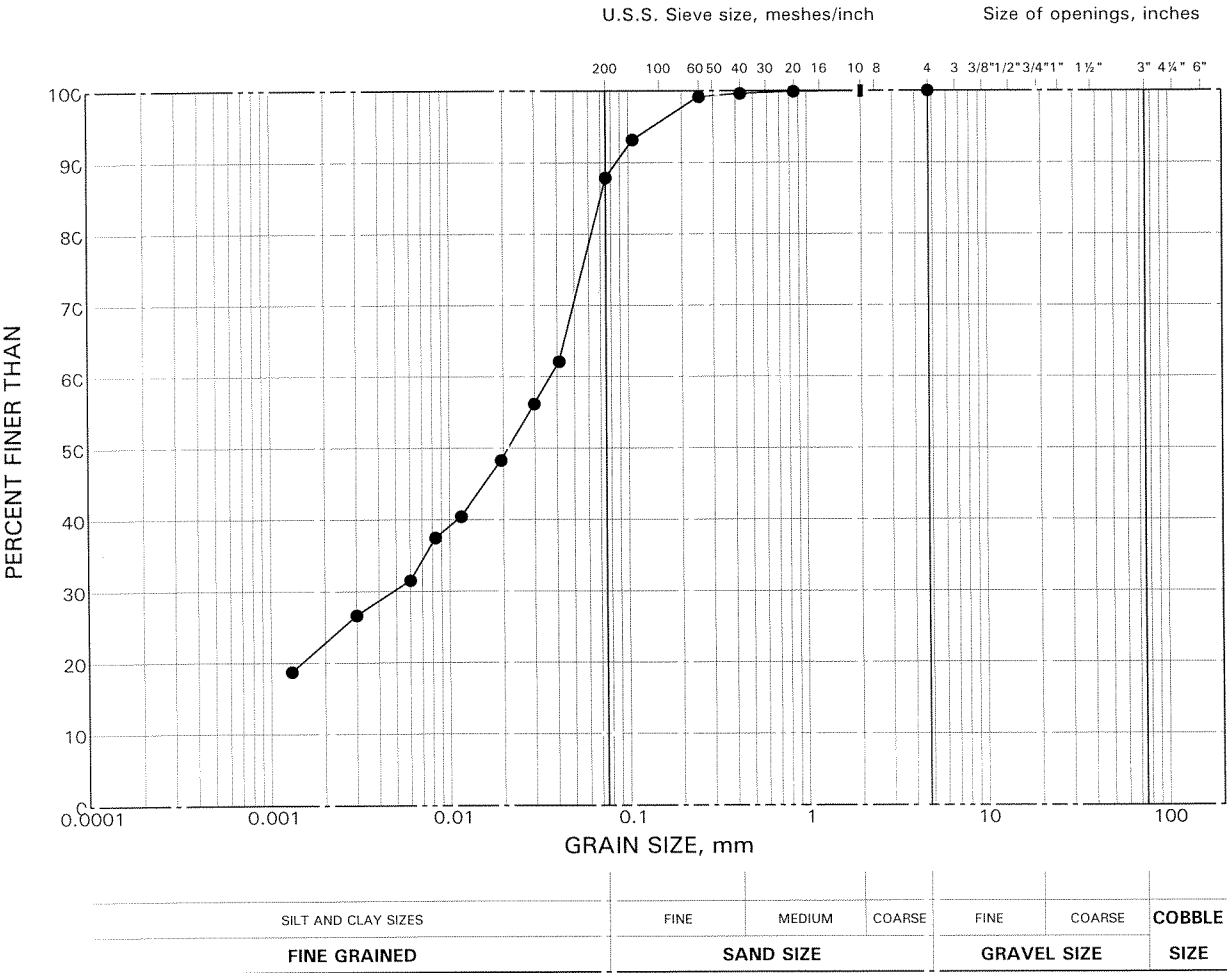


LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	1	14	8.5-9.1

GRAIN SIZE DISTRIBUTION

FIGURE

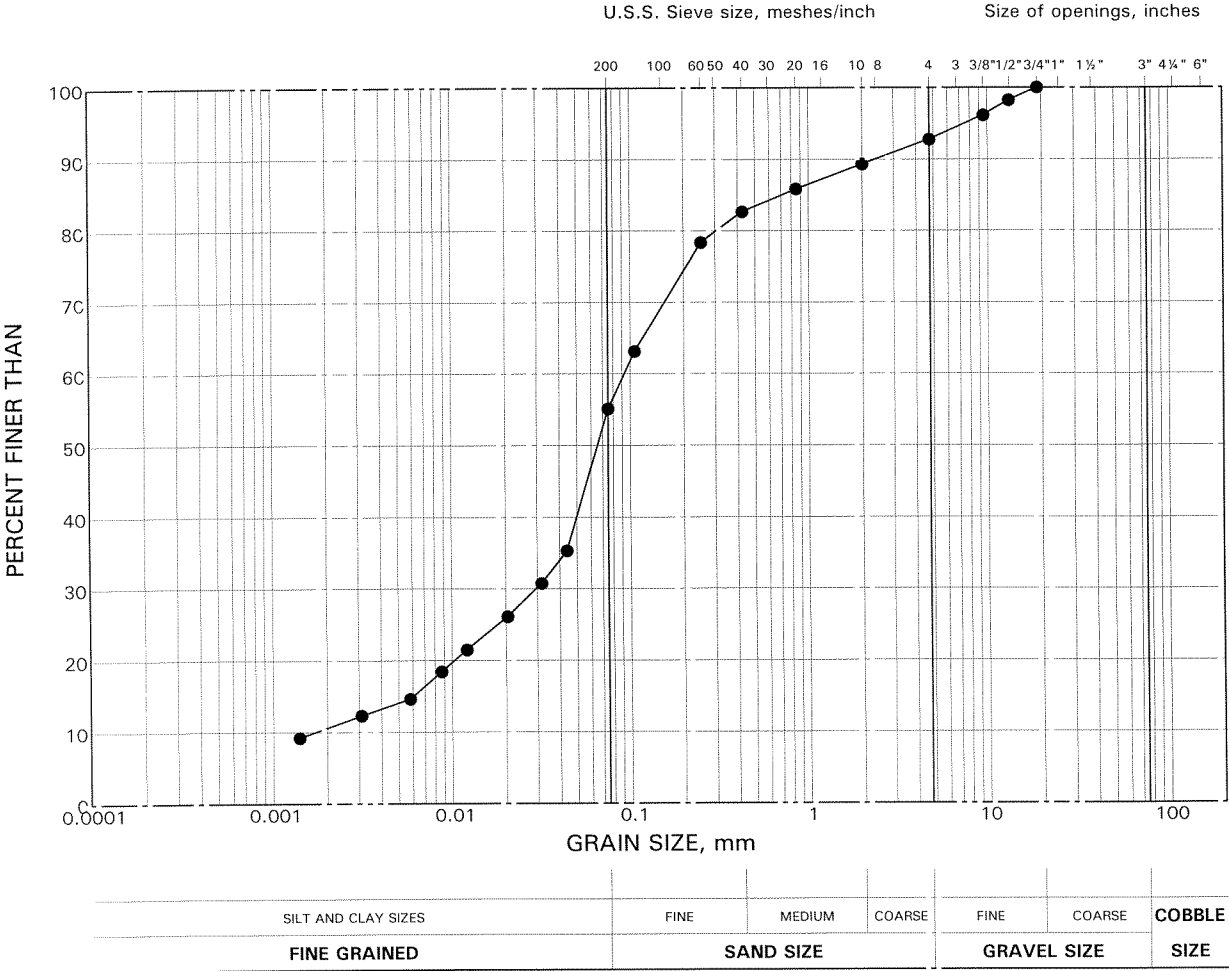


LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	1	18	11.0-11.6

GRAIN SIZE DISTRIBUTION

FIGURE



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	1	20	12.2-12.8

APPENDIX B

**DESIGN MEMORANDUM FOR GEOGRID MAT
PLUS
RECORD OF BOREHOLE LOGS
(BH 100 AND BH 101) AND
ASSOCIATED LABORATORY RESULTS
BY OTHERS**

memorandum



To: Jim Vanbiesbrouck
Construction Office
Central Region

February 17, 2003

From: Pavements and Foundations Section
Room 223, Central Building

Phone: (416)235-4333
Fax: (416) 235-3919

Re: RSS Abutment over Sewer Backfill
Highway 27 Overpass, East Abutment
Hwy 401, Contract 2002-2000
Central Region

Further to the foundation recommendations for the RSS abutment construction over the sanitary sewer, outlined in our memo dated January 30, 2003, the following additional site preparation details are provided. The details are the result of discussion and consensus developed during a site meeting held on February 10, 2003 attended by representatives of MTO, the CA Morrison Hershfield, the contractor Graham, RSS supplier RECO, and geogrid supplier Terrafix.

The original memo should be referred to for a synopsis of the subsurface conditions in the vicinity of the sanitary sewer.

The preferred alternative is to distribute the RSS backfill loading across the site using geogrid, as shown in Figure 1. The installation requires that an area extending 2m beyond the panel walls, and for the proposed extent of the reinforcing strips, ~6.5m, be excavated to Elevation 142.0. The preference would be to remain above the water table, which was measured at 141.4 on January 10, 2003. It is expected that the excavation could be carried out at a temporary slope of 1H:1V, but it is necessary to install the geogrid and backfill as quickly as possible. For this reason, 19mm clearstone should be used since compaction is not required. If the 1H:1V slope cannot be accommodated adjacent to Highway 27, then the extent of geogrid could be reduced to 1.5m beyond the wall alignment.

The recommended geogrid is Tensar High Strength Uniaxial Grid 1600 and 1700 supplied by Terrafix Geosynthetics, (416-674-0363). The UX 1700HS should be placed directly on the native material at El. 142.0, with the strong axis perpendicular to the abutment wall alignment, followed by placement of 200mm of clearstone. The UX1600 HS should be laid at El. 142.2, as per the first layer. Clearstone should be brought up to the top of footing elevation, which is shown on the drawings as 143.0.

To address the possible differential settlement of the RECO panel wall where it rests on the top of the pier footings, it was agreed that an additional biaxial geogrid layer should extend across the top of footings PC7 to PC8 (Figure 2).

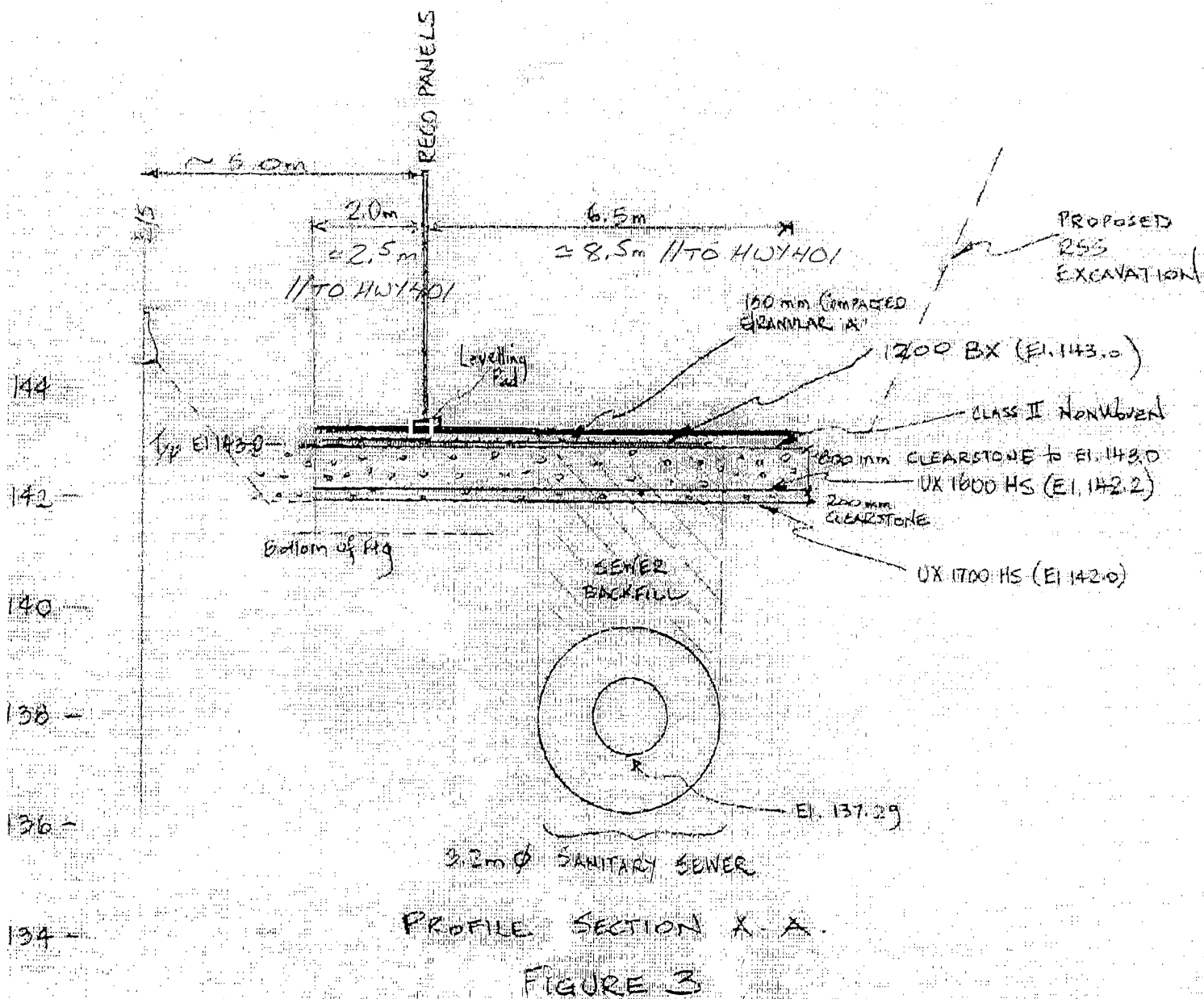
To minimize the movement of fines into the clearstone, a layer of Class II Non-woven geotextile should be placed across the entire plan area of the clearstone.

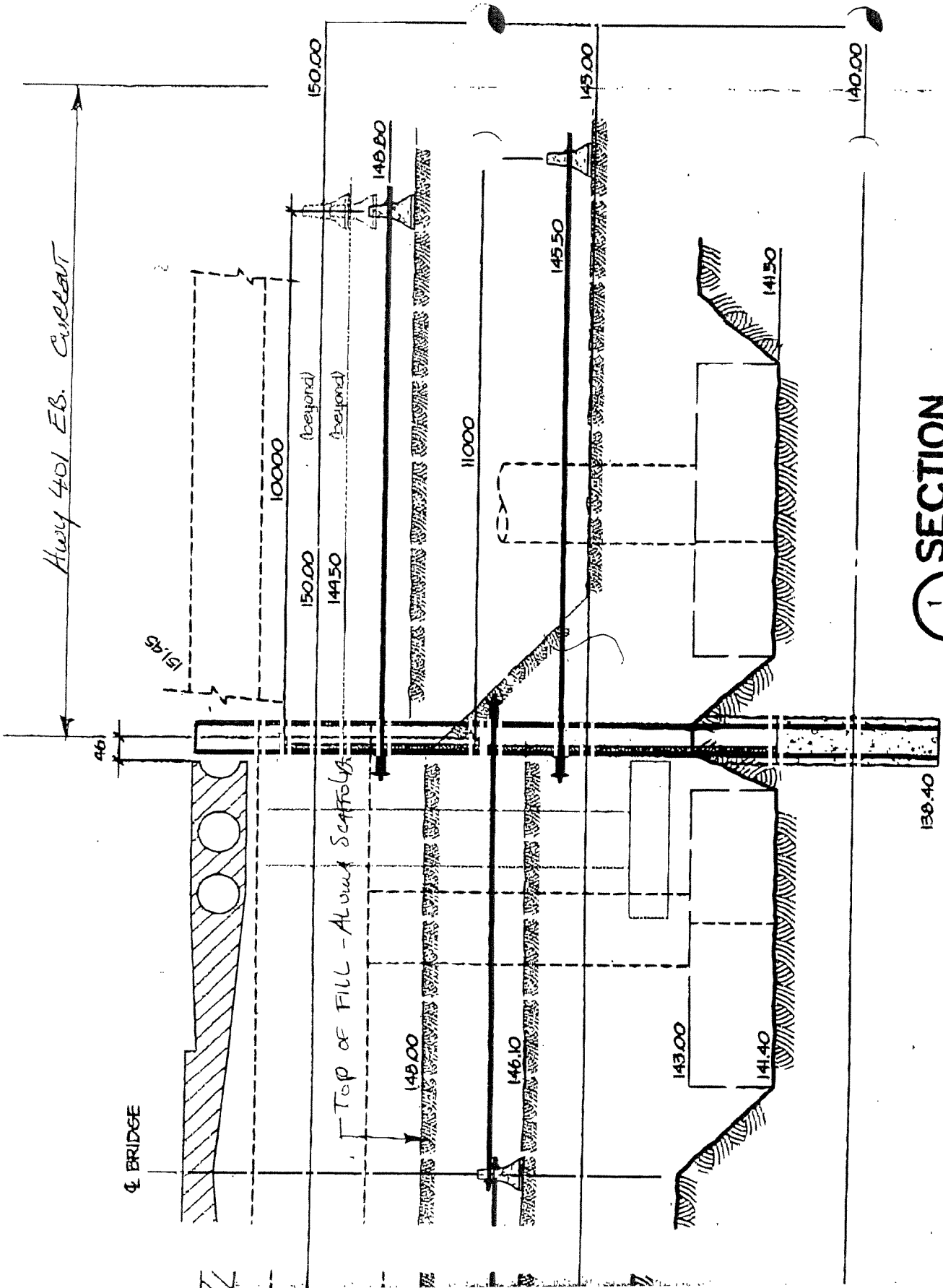
The biaxial grid should be Tensar BX 1200 and installed directly on the geotextile. The geogrid is not required across the entire excavation but limited from 2m beyond the RSS panel wall (highway side) to 2m beyond the pier footings (within the RSS abutment). A 150mm cushion of compacted Granular A should be placed across the excavation to the proposed elevation of the RSS levelling pad. Refer to Figure 3 for a profile section.

As mentioned in previous correspondence, the installation of the geogrid layers, the configuration and sequence should be followed as closely as possible. This may be difficult since dimensions in the field may be slightly different, especially where there may be interference with the unshrinkable backfill areas and the actual proximity of the excavation to the highway, temporary shoring and existing structures.

If there are any questions or concerns, please let me know.

Betty Bennett, P.Eng.
Foundation Engineer



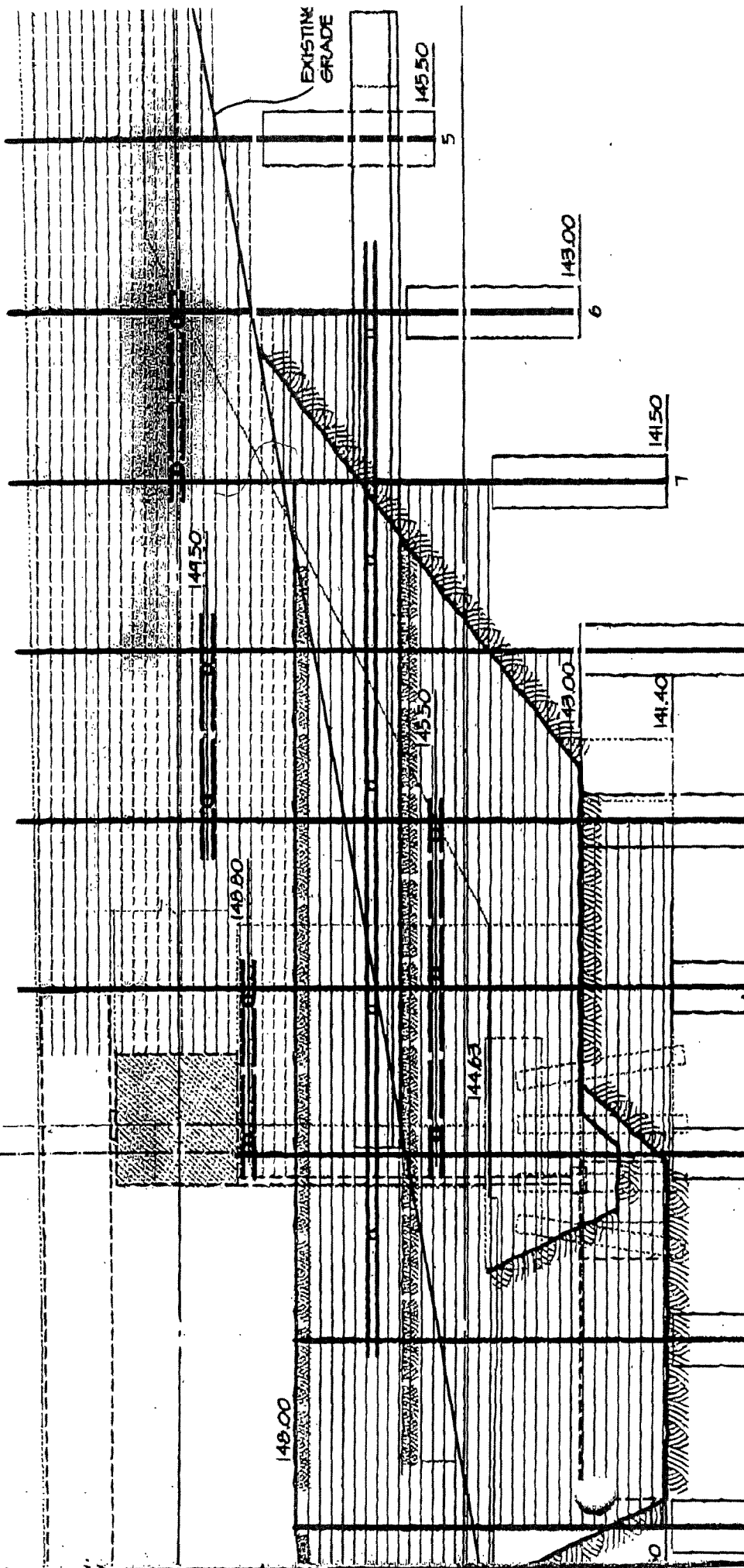


Hwy 401 EB. Culvert

BRIDGE



PROPOSED S.E. ABUTMENT



S.E. ABUTMENT - NORTH ELEVATION

139.85±

12A

EXISTING 1372 I.D. SANITARY SEWER

12

11

9

8

141.40

141.50

143.00

145.50

149.00

148.80

149.50

EXISTING GRADE

PROPOSED S.E. ABUTMENT

IDENT

APPENDIX B

BOREHOLE LOGS

RECORD OF BOREHOLE No BH02-100

1 OF 2

METRIC

W.P. 47-99-00 LOCATION 4,837,476.2 N; 298,567.0 E ORIGINATED BY JS
DIST 6 HWY 401/27 BOREHOLE TYPE Solid Stem Auger COMPILED BY JS
DATUM Geodetic DATE 19/12/2002 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT 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 (%)					
							\circ UNCONFINED	$+$ FIELD VANE	\square QUICK TRIAXIAL	\times LAB VANE	w_p	w	w_L				
144.7 0.0	FILL - CLAYEY SILT, trace to some gravel, trace sand, stiff to very stiff, brown		1	SS	10												
			2	SS	10												
			3a	SS	21												
142.7 1.9	FILL - CLAYEY SILT, some sand and gravel, occasional cobbles, very stiff to stiff, grey		3b	SS													
			4	SS	12												
141.8 2.9	FILL - CLAYEY SILT, some sand and gravel, occasional cobbles, firm, grey		5	SS	7												
			6a	SS													
140.7 4.0	TOPSOIL - sandy, loose, black		6b	SS	7												
	FILL - CLAYEY SILT, some sand and gravel, firm, grey		6c	SS													
			7	SS	6												
			8	SS	5												

Continued Next Page

+ 3, X 3

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH02-100

2 OF 2

METRIC

W.P. 47-99-00 LOCATION 4,837,476.2 N; 298,567.0 E ORIGINATED BY JS
DIST 6 HWY 401/27 BOREHOLE TYPE Solid Stem Auger COMPILED BY JS
DATUM Geodetic DATE 19/12/2002 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT			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	W _p	W	W _L			
138.0 6.6	WOOD AND GRAVEL - Assumed edge of former shoring system. Note: SPT's artificially high due to driving through old wood shoring.		9	SS	52												
			10	SS	68												
136.6 8.1	END OF BOREHOLE AT 8.08 m DUE TO CONCERNS WITH AUGERING THROUGH THE EXISTING WOOD SHORING BOREHOLE OPEN UPON COMPLETION OF DRILLING. STANDPIPE PIEZOMETER INSTALLED TO DEPTH OF 7.92 m WATER LEVEL MEASURED AT ELEVATION 141.9 m UPON INSTALLATION. WATER LEVEL MEASURED AT ELEVATION 141.4 m ON JAN 10, 2003.		11	SS	57												

+ 3, X 3

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH02-101

1 OF 2

METRIC

W.P. 47-99-00 LOCATION 4,837,478.0 N; 298,569.9 E ORIGINATED BY JS
 DIST 6 HWY 401/27 BOREHOLE TYPE Solid Stem Auger COMPILED BY JS
 DATUM Geodetic DATE 19/12/2002 CHECKED BY IC

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 (%)
144.6 0.0	FILL - CLAYEY SILT, some sand and gravel, firm to hard, brown		1	SS	7													
					2	SS	8											6 27 55 12
			3	SS	30													
142.5 2.1	FILL - CLAYEY SILT, some sand and gravel, stiff, grey																	
					4a	SS	9											
141.9 2.7	FILL - CLAYEY SILT, some organics, black, (Possible Topsoil)																	
					4b	SS												
141.7 2.9	FILL - CLAYEY SILT, some sand and gravel, occasional cobbles, stiff, grey																	
					5a	SS	8											
					5b	SS												
141.1 3.5	FILL - CLAYEY SILT, some sand and gravel, occasional cobbles, hard, grey																	
					6	SS	46											7 36 48 8
			7	SS	73													
139.6 5.0	FILL - CLAYEY SILT, some sand and gravel, occasional cobbles, some wood pieces, stiff, grey																	
					8	SS	15											

Continued Next Page

+ 3 x 3: Numbers refer to 0 3% STRAIN AT FAILURE
Sensitivity

METRIC

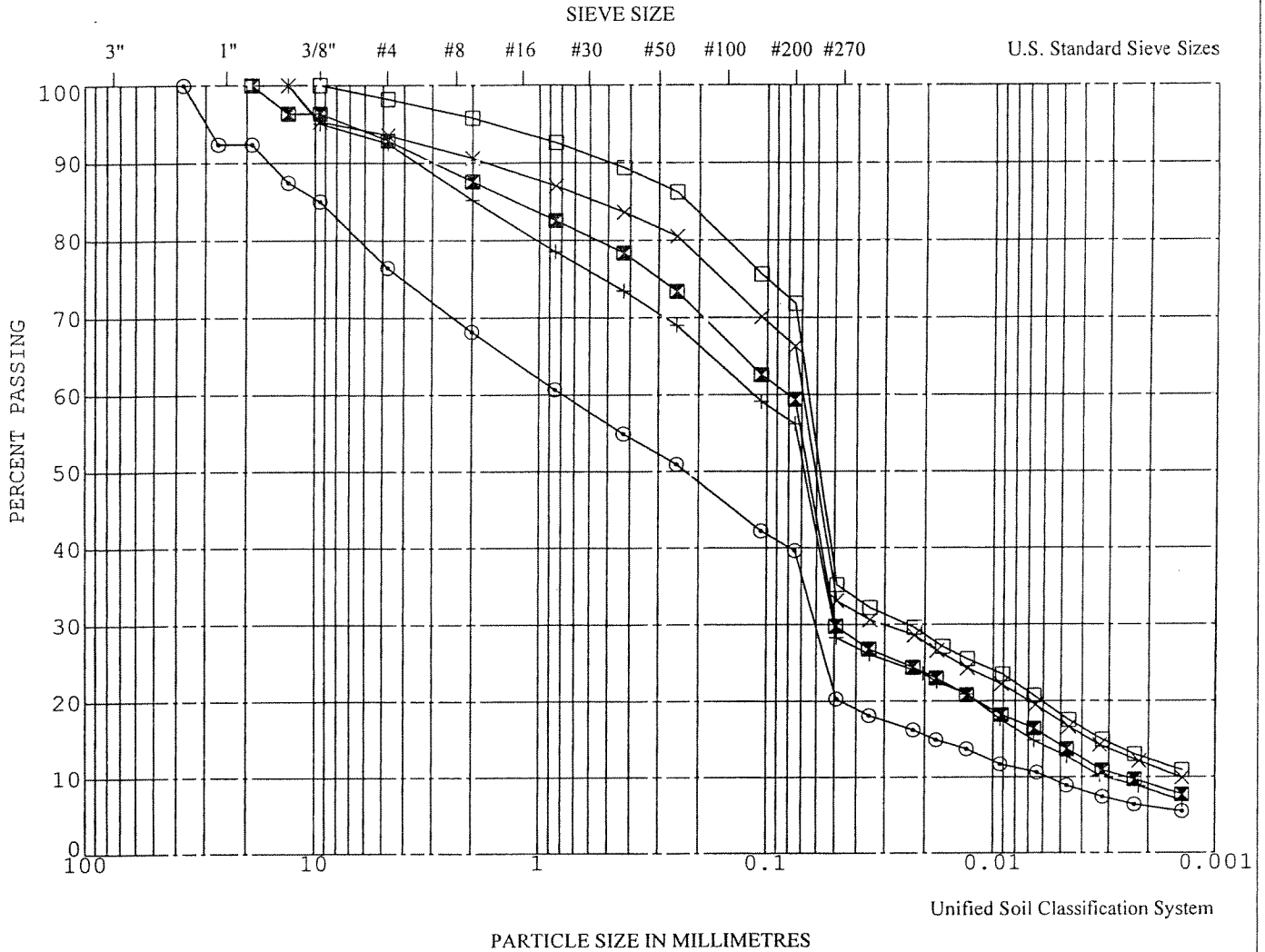
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L			
136.9 7.8	WEATHERED SHALE		9	SS	9		138								
10			SS	11	137										
11a			SS												
11b			SS	80											
12			SS	60											
135.4 9.3	END OF BOREHOLE AT 9.27 m.		13	SS	50										
	BOREHOLE OPEN UPON COMPLETION OF DRILLING. STANDPIPE PIEZOMETER INSTALLED TO DEPTH OF 7.62 m. WATER LEVEL MEASURED AT ELEVATION 141.9 m UPON INSTALLATION. WATER LEVEL MEASURED AT ELEVATION 141.3 m ON JAN 10, 2003.														

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX C

LABORATORY GRAIN SIZE DATA

GRAIN SIZE ANALYSIS CLAYEY SILT FILL



COB'L	GRAVEL		SAND			SILT & CLAY
	Coarse	Fine	Coarse	Medium	Fine	

LEGEND:

- Sample BH02-100 Depth (m) 0.99
- Sample BH02-100 Depth (m) 4.80
- × Sample BH02-101 Depth (m) 0.99
- + Sample BH02-101 Depth (m) 4.04
- Sample BH02-101 Depth (m) 7.09

January 2003

Reference No.: MG02-077

HIGHWAY 401/27
Toronto, Ontario

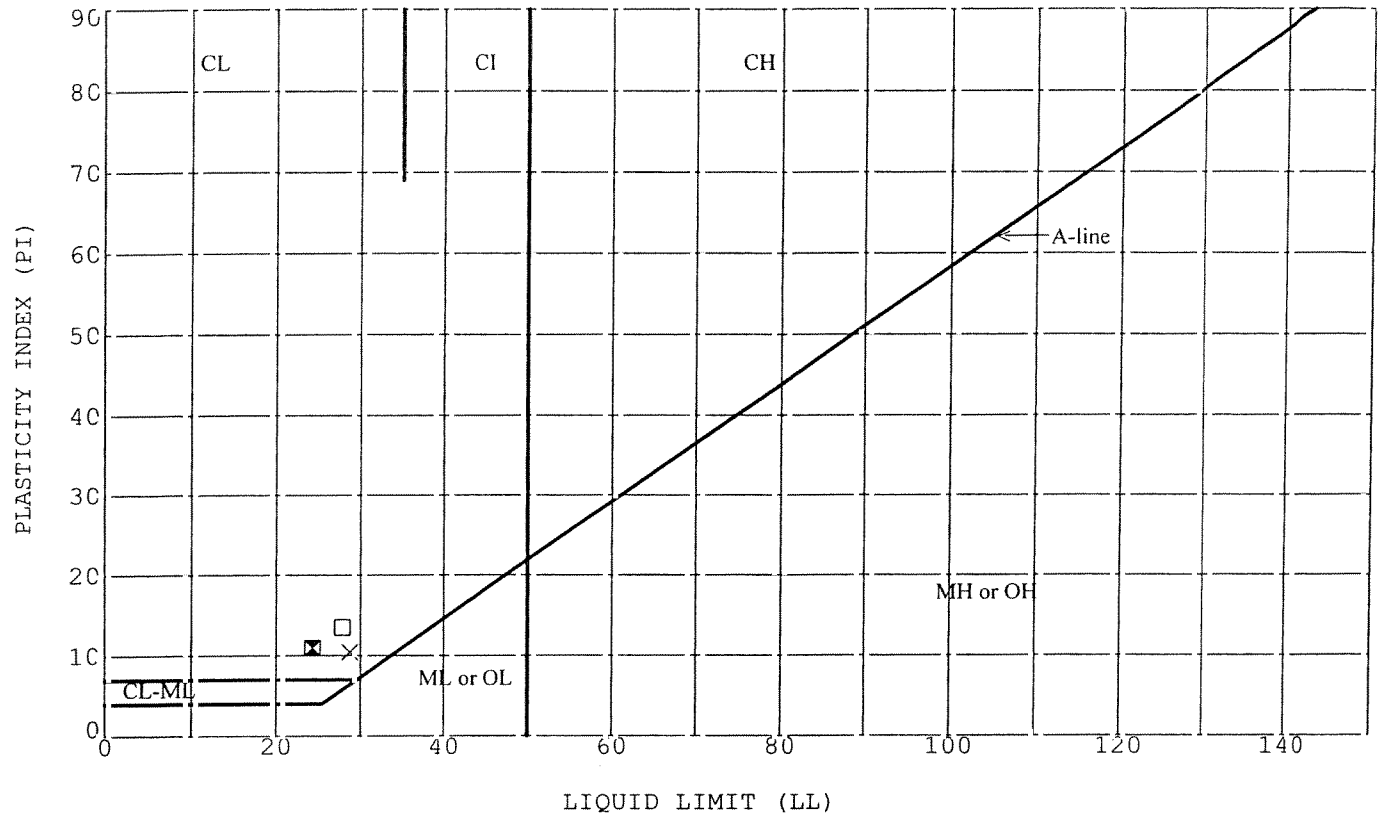
DST CONSULTING ENGINEERS INC.

Figure C1

APPENDIX D

SOIL PLASTICITY CHARTS

ATTERBERG LIMIT TEST RESULTS CLAYEY SILT FILL



LEGEND:

□	BOREHOLE BH02-100	DEPTH 0.99
■	BOREHOLE BH02-101	DEPTH 0.99
×	BOREHOLE BH02-101	DEPTH 7.09

W_L	W_P	PI	W
28	14	14	16
24	13	11	13
29	18	10	16

January 2003

Reference No.: MG02-077

HIGHWAY 401/27 - Toronto, Ontario

DST CONSULTING ENGINEERS INC.

Figure D1