

**FOUNDATION INVESTIGATION AND REMEDIATION DESIGN
FOR FAILED EMBANKMENT
HIGHWAY 17 FROM STATION 12+100 TO 12+400
TOWNSHIP OF BOYS
WEST OF KENORA
Geocres No.: 52E-50
AGREEMENT NO.: 6005-A-000167**

**July 17, 2007
GS-TB-007607**

**Prepared For:
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Ministry of Transportation

Foundation Investigation and Remediation Proposal for Failed Road Embankment

DST Reference No.: GS-TB-007607, July 16, 2007

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Executive Summary

On June 29th or 30th, 2007, the south side of an embankment on Highway 17 west of Kenora failed over a length of 180 m, resulting in closure of the eastbound lane. A geotechnical investigation was fast tracked with drilling carried out July 5 to 7, followed by laboratory classification tests, analyses and a draft report on July 11, 2007. Field and office works was carried out with close liaison with the Ministry of Transportation (MTO) Pavement and Foundations Section.

At the site the highway grade falls at a 4% slope from west to east. The land at the south toe is relatively flat, dipping gently to the south. The north toe is higher and falls to the east at a flatter gradient than the highway. The embankment, which has 3h:1v side slopes, may be considered a side-hill fill. Interviews with a long time area resident indicate that this stretch of highway has a long history of movements for more than 50 years. The failure occurred a day after a severe rainstorm. Failure observations indicate no indications of movements beyond the toe and fill slope bulging at some locations.

The investigation found that the embankment overlies a complex stratigraphy and unusual groundwater conditions. The site is in transitional terrain between lacustrine clay deposits and shallow bedrock overlain by tills, some of which have been modified by post glacial lake action. This has resulted in a thick clay deposit at the south toe, and generally sandy soils below the embankment with thin clay interbeds. Bedrock outcrops to the north. The groundwater table level within the embankment is midway between the ditch levels.

A back analysis was conducted by modeling the most severe cross-section and the estimated post-failure geometry. It was found that in order to explain a failure, the water table within the embankment must have been considerably higher than at the time of investigation. This would be a

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very unusual condition likely involving recharge along the embankment. The analyses established estimated soil parameters and groundwater conditions to be used in design.

A stable configuration for rapid reconstruction of the highway was designed, incorporating 6 m wide bi-level granular counter berms which include an allowance for long term settlement of the underlying clay. This requires embankment excavation and reconstruction extending not less than 1 m beyond the failure zone at highway level. Construction excavations should not extend longer than 30 m at any one time, measured at the bottom of excavation parallel to the highway centerline, where it should not be left open overnight. Traffic wheel loads should not be allowed closer than 1.6 m from the edge of excavation and no parking on the westbound lane should be allowed. Temporary slopes should not exceed 2h:1v, but at the top of the embankment may be steeper up to 1:1 over heights of up to 3 m. Improved drainage in the north ditch is also recommended in order to lower storm water levels. The above should be reviewed and confirmed once the topographic survey currently in progress is complete.

Given the uncertainties associated with the complex subsurface conditions, the limited soil information and testing, the long history of movements in this area, the unusual failure trigger conditions, the possibility of undetected failure mechanisms, the possibility that all the slide mass may not be removed and the possibility of even more severe rainfall events, it is recommended that in the long term further improvements to the embankment's stability be provided. Options that can be assessed in more detail include groundwater and stormwater drainage improvements, additional reinforcement of the foundation soils and relocation of the embankment to more stable ground to the north. In the meantime, frequent visual and survey monitoring is recommended including additional hand auger holes at the toe, and with an allowance for contingencies of further maintenance as a result of unexpected movements in the south slope and shoulder.

1.0 INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation of Ontario (MTO) to conduct a foundation investigation at a slope failure on Highway 17 located 16 km west of the west Kenora Bypass junction. The purpose is to determine the cause of failure and propose a suitable remediation option to rapidly re-establish a safe highway embankment as an emergency response.

Authorization to proceed with this work was received from Doug Cooper, P.Eng, head of the Geotechnical Section of MTO, Northwestern Region.

This report has been updated with clarifications from a report originally issued on July 13, 2007.

In order to determine the cause of failure and explore remedial options suitable for rapid stabilization, field Investigation and stability analyses are required. The following is the scope of the task discussed and agreed to with MTO during discussions on site with senior foundation engineer Tae Kim of MTO's Pavement and Foundation Section and DST's Dr. M.W. Bo.

- carry out 6 boreholes and associated in-situ tests on the crest and toe of the embankment.
- carry out 6 hand auger holes at or near the toe of embankment.
- carry out 2 shallow auger drill holes at 12 m from the centre line of existing road alignment on the north side of highway to explore the possibility of road widening.
- carry out relevant laboratory tests on the collected samples.
- estimate geotechnical parameters from field data, laboratory testing and back analyses.
- propose appropriate remediation options to stabilise the embankment.
- provide recommendations for a temporary detour by means of road widening, if required.
- propose a suitable monitoring scheme.

This scope is limited at this time due to time constraints and the need to re-establish a safe highway configuration with minimal delay. Not all of the work has been prepared to normal MTO guidelines and presentation standards in order to expedite its delivery (eg. some borehole locations/elevations are estimated, no laboratory strength and compressibility testing, no stratigraphic profiles/sections plotted).

This report addresses the field investigation, laboratory testing program and slope stability analyses for the failed slope and proposed remedial slope.

Complete topographic survey data for the site had not yet been received from the MTO at the time this report was issued. It is therefore recommended that the conclusions and recommendations be reviewed by DST and confirmed once this data is available.

2.0 SITE DESCRIPTION

The road embankment affected by the slope failure is between 12+110 and 12+290. The existing highway embankment varies in height from approximately 4.0 m to 11.0 m within the affected area and appears to be constructed on a natural slope. The alignment of the highway rises from 338.6 m at 12+360 to 353.6 m at 11+860 at the rate of 0.042 m/m. The geometry of the highway embankment is not symmetrical with a lower height at the northern side and much higher height at the southern side. Elevations of the toe at the northern side are 4.5 m to 8 m higher than the elevations of the toe at the southern side. The gradient of the south embankment slope is between 2.8 h to 1 v and 3.2 h to 1 v. The engineered fill slope is vegetated with grass.

The slope on the northern side of the highway is covered by some bedrock exposures dipping towards the ditch at the toe of the embankment and there is a valley formed between the road embankment and the hilly slope. A drainage ditch is located about 30 m from the centre line of the highway and it is parallel to the road embankment. It has a very shallow gradient from west to east and water is discharged through two ~700 mm culverts installed underneath the road embankment at 12+360 and 12+380. A running water of a few ten millimetres deep was noticed in the ditch between 12+250 and 12+380.

The southern side of the highway is sloping down to a flat plain. The plain slopes gently over about 100 m towards a lakeshore. Part of the embankment side slope is formed with engineered fill and the lower quarter of the slope at the toe is formed by a natural soil formation with similar gradient. The natural ground is generally heavily vegetated with medium high trees. Water ponding at the toe of the slope was found between about 12+250 and 12+290.

3.0 FAILURE OBSERVATIONS

The failure was discovered in the morning of 30th June, 2007, after heavy rain occurred on the night of 28th June. The slope failure area is located between 12+110 and 12+290 and the total length of affected area is about 180 m along the road embankment. There are two main areas of tension cracks. The smaller one is a 25 m length on the road shoulder with a crack line extended 4 m into the road surface between 12+110 and 12+135. The crack width is about 100 mm. The larger crack has a length of 140 m between 12+150 and 12+290 with the crack line extending up to the centre line of the road. This major crack has two near parallel cracks indicating progressive failure. No heave or evidence of movement at or outside the toe was noticed but bulging was observed at two or three places on the slope within the engineered fill.

Cross sections of the embankment at several borehole locations were surveyed by MTO and have been plotted up in Appendix 'D'.

A few pictures of the site taken at the slip site can be found in the Appendix 'C'.

An interview with a long time area resident indicates that this stretch of highway has a long history of movements for more than 50 years. A well vegetated sinkhole-like feature was also observed north of the highway, a further indication of possible ground instability in the area. The area to the east has previously been investigated for movements and some degree of stabilization measures applied.

4.0 INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between July 5 and July 7, 2007 utilizing a CME 750 equipped for geotechnical drilling and operated by DST. Eight boreholes (BHs) at 7 locations were put down to depths ranging between 1.1 and 15.35 m. Six auger holes (AHs) were also sunk using portable equipment at or near the toe of the embankment on the flat plain to depths ranging between 1.5 m and 5 m.

Borehole locations are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 1, 2, 4 and 5 are located at the edge of the shoulder of the roadway while Boreholes 3 is located near the toe of the existing embankment. Boreholes 7 and 8 were located on the northern side slope of the embankment and were 12 m from the centreline of the road embankment. The boreholes were advanced with hollow stem augers well into the natural ground.

Soil samples were obtained from the auger flights and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 51 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 310 mm is known as the standard penetration blow count (N) which provides an indication of the denseness or consistency of the soil. Representative soil samples are obtained from within the sampler. Borehole and auger hole logs are presented as Enclosures 1 to 14.

Ground surface elevations at the BH borehole locations were surveyed by MTO and DST, and estimated for AH1 to AH6. Auger hole locations AH1 to AH6 were estimated using GPS coordinates. Offsets for borehole locations at and beyond the south toe were estimated based on cross-sections.

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.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included natural moisture contents, Atterberg limit tests and gradation analyses. Laboratory test results are presented on the Borehole Logs and Enclosure 15 to 17.

5.0 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 Published Engineering Geology

The Quaternary and bedrock geology of the general area under investigation, as reported by D.R. Hallett and M.A. Roed in the Northern Ontario Engineering Geology Terrain Study 20, Rat Portage Bay Area (NTS52E/NE and part of 52E/NW) District of Kenora, consists of a discontinuous mantle of Quaternary surficial deposits overlying crystalline bedrock of Precambrian age. Ground moraine occurs as a dominant unit patchy throughout the bedrock terrain. The northern half and western part of the area are underlain by early Precambrian felsic igneous and metamorphic rocks mainly gneiss. During deglaciation, a variety of superficial materials were deposited. Till was deposited directly by the ice in the form of ground moraine. Deposit of glaciolacustrine silt and clay occur wherever the lake covered the land. At the end of the Pleistocene Stage of the Quaternary period, modern streams have developed alluvial flood plains and organic deposits have accumulated in wet depressions. These deposits are non-glacial in origin, and together with the various glacial materials, comprise the variety of Quaternary unconsolidated deposits that form a discontinuous mantle over the bedrock in the Rat Portage Bay area.

On review of the Northern Ontario Engineering Geology Terrain Study Data Base Map (Map 5055, Rat Portage Bay NTS 52E/NE) for the general area, the material landform, topography and drainage generally consists of ground moraine and shallow bedrock with isolated areas of sand and gravel outwash. Organic terrain occurs in small, isolated, poorly drained depressions in bedrock terrain, along streams and near lakes with low relief. This unit overlies glacio-lacustrine silt and clay in depressions and occurs in association with bedrock knobs. Bedrock terrain is found at the higher ground with moderate relief and well drained.

At this specific site, the mapping indicates a predominant landform of shallow bedrock knobs. Subordinate landforms are indicated to consist of sandy till (ground moraine) as well as silt and clay

(a glacio-lacustrine plain).

5.2 Field Observations

The generalized stratigraphy of the site based on findings at the borehole locations on the highway consists of fine to medium sand fill underlain by a natural deposit of medium to coarse sand with some gravel under the embankment. Auger refusal occurred in two boreholes (BH 2 and 3) within the fill and in BH 8 in the natural ground. Refusal within the fill is likely to be floating boulders or rockfill. The refusal material in the natural ground at Borehole 8 was not confirmed by diamond drilling techniques and therefore could be boulders or bedrock.

The highway fill, as identified by Boreholes 1, 4 and 5, sunk at the edge of the highway, consists of fine to medium granular fill to 8.2 to 9.1 m. A thin layer of silty clay of about 400 mm was found near the base of the fill at 9.1 m depth in BH 5. Based on the all borehole data fill material is generally underlain by compact to dense medium sand with some gravel.

Based on the hand auger holes sunk at or a few meters away from the toe on the flat plain to the south, 100 mm of organic soil is underlain by 1 to 1.5 m thickness of silty sand at AH 4 and 6. Hand auger refusal was encountered at 1 to 1.5 m depth. At about 3 m away from the toe of the embankment at AH 5, 4 m thickness of silty clay is found underlying a meter thickness of fine sand. However total thickness of silty clay was unable to determine due to the limitation of hand auger capacity. At auger holes 1, 2 and 3 which were penetrated about 20 m away from the toe of embankment, 2.5 to 4.8 m thickness of silty clay was found underlying 100 to 200 m thickness of organic. The total thickness of silty clay was not determined due to the limitation of hand operated auger capacity.

Based on the above, the embankment appears to be constructed over an area of transition from lacustrine silty clay to glacial till and shallow bedrock. This results in a complex stratigraphy, with

clay interbeds and zones of sandy till materials modified by post glacial lake action below and near the embankment.

Furthermore, some clay layers originally underneath the embankment might have pushed out decades ago during the original embankment construction or compressed, or a combination of both might have occurred.

5.3 Embankment Fill

The fill in the highway embankment consists of fine to medium sand with some gravel, and sand and gravel as the sub-base materials. The sub-base materials vary in thickness from 0.3 to 0.4 m. Rockfill zones may also exist. The bottom of the fill is located at elevation between 333.5 m and 337.0 m at the borehole locations, although noting it is difficult to absolutely determine the boundary between fill and native soils, given their similarity. In particular, the fill may be shallower than estimated on the logs.

The Standard Penetration Test (SPT) results generally indicate a very loose to very dense state of denseness (N values vary from 3 blows/0.3 m to 52 blows/0.3 m). Significant reductions of SPT values to 3 to 5 may indicate the shear zone after failure.

A thin clay layer or pocket incorporated within the fill was encountered at BH 4.

Gradation analyses (Enclosure 16 and 17) conducted on samples from Borehole 1, 2 and 4 at various depths indicates 20 to 33% gravel, 50 to 58% sand and 15 to 20% fines (silt and clay fraction) with the exception in BH 5 at 1.5 m depth in which the sample has a high gravel content of 60 %, sand content of 35 % and fines content of 5 %.

5.4 Silty Clay

A thin silty clay layer is present below a 100 to 200 mm depth underlying decomposed organics in all auger holes (AH 1 to AH3) located about 20 m away from the toe of the Highway embankment. At AH 4 which is 3 m away from the toe of the embankment a silty clay layer was found underlying a 1.5 m thickness of fine sand. The thickness of the silty clay layer up to the end of auger holes varies between 2.5 and 4.8 m but the total thickness was not confirmed by the auger holes.

The silty clay has a water content varying between 51 and 89 % with plasticity indices ranging between 34 and 59 %, showing high plasticity (Enclosure 15).

The undrained shear strength of the silty clay at AH 1 as measured by field vane equipment was found to be as low as 29 kPa at a 1.5 m depth indicating a firm consistency. Considering the relatively high moisture content with low undrained shear strength, the silty clay is expected to be in a state of normally consolidated or lightly consolidated condition likely due to aging and dessication.

Drained friction angles of 24 to 28 degrees are estimated based on published correlations with plasticity (Kenney, 1959).

5.5 Natural Sand

Beneath the granular fill a compact to very dense layer of fine to coarse silty sand was found between 8.2 and 9.1 m depths. Gradation analyses conducted on representative samples retrieved from the field investigation at Boreholes 1 to 4 indicate 12 to 20% gravel content, 60 to 68% sand and 18 to 20% silt (Enclosure 18).

The Standard Penetration Test (SPT) results generally indicate a compact to very dense state of denseness (N values vary from 12 blows/0.3 m to 56 blows/0.3 m).

At Borehole 2, 3 and 8 only, auger refusal was met at 3.6 m, 1.1 m and 4.3 m respectively.

5.6 Groundwater

The groundwater levels taken on completion of drilling and measured from temporary standpipes are noted on the Borehole Logs, Enclosures 1 to 14. The groundwater level in the boreholes carried out on the edge of highway noted during our field investigation varied between 5.38 m and 9.1 m below existing grade. At Boreholes 1, 4 and 5 the water level noted was 5.38 m (elevation 336.13 m), 7.4 m (elevation 335.94 m) and 9.1 m (elevation 337.8 m) respectively below existing grade.

Observations indicate that the groundwater is likely near surface at both embankment toes. At BH 1, 4 and 5 the water level is roughly midway between ditch levels. For design purposes, the groundwater profile advised from back analysis should be taken as the worst case scenario for past groundwater levels. Higher levels could conceivably occur as a result of more severe precipitation events than have occurred in the recent past.

Of interest is that the water levels within the embankment at the time of failure were likely much higher than the north ditch level, indicating recharge was likely occurring along the embankment in permeable zones from uphill. Unpredictable recharge conditions may also occur as a result of groundwater conducted through highly fractured zones of shallow bedrock. Alternatively, the high water levels could be from infiltration when water in the north ditch was temporarily much higher during storm runoff. Furthermore, both of the above could possibly be a cause of the high water table.

6.0 DISCUSSIONS

DST Consulting Engineers Inc. (DST) has been retained by MTO to conduct a foundation investigation for a failed embankment of Highway 17 in the Township of Boys between Stations 12+110 and 12+290. It was understood that failure was noticed in the morning of 30th June after the incident of heavy rainfall occurred in the evening of 28th June, 2007. No corresponding earthquake events were reported.

Options considered for rapid implementation of remedial work were as follows and are discussed in detail within this section:

- Reconstruct with a flatter slope or counterbalancing berm;
- Lower the water table;
- A combination of the above.

Longer term options were also considered, as discussed in Section 8.0.

6.1 Soil Parameters

As an overview, this 180 m long site is located within an area of transition between deep glacio-lacustrine and shallow ground moraine terrain. This results in a complex stratigraphy below the highway.

The subsurface conditions at the existing embankment (a side-hill fill) consist of granular fills up to 9.1 m at the boreholes. The fill is underlain by native silty sands with occasional silt and clay seams. In general, the subsurface conditions beyond the existing south toe of the embankment were found to consist of silty clay of thickness greater than 5 m.

Engineering properties for embankment fill have been estimated as follows based on results of

laboratory and field results, and further refined in the back analysis of failure mechanisms

(described later herein).

Sand Fill

- Unit Weight, $\gamma = 19 \text{ kN/m}^3$
- Drained Angle of Internal Friction, $\phi' \geq 30^\circ$
- Drained Angle of Internal Friction in the weak zone, $\phi' = 28^\circ$
- Drained Cohesion Intercept, $c' = 0 \text{ kPa}$

Clay Soils

For the normally consolidated silty clay, consolidation properties have been estimated based on natural moisture content measured between 56 and 71 % and liquid limit measured between 51 and 89 %. Strength parameters are based on limited field measurement and published correlations for large strains with laboratory test results.

- Void Ratio (e) varies from 1.48 to 1.89.
- Compression Index (C_c) varies from 0.37 to 0.56.
- Secondary Compression Index (C_α) varies from 0.018 to 0.028.
- Unit Weight (γ) = 18.0 kN/m^3 .
- Drained Angle of Internal Friction, $\phi' \geq 25^\circ$ (From back analyses and estimated from correlation with plasticity index (24 to 28°), (Kenney 1959))
- Drained Cohesion Intercept, $c' = 0 \text{ kPa}$
- Undrained cohesion, $c = 30 \text{ kPa}$

Native Sand

Engineering properties for silty sand have been estimated as follows:

- Unit Weight, $\gamma = 18 - 20 \text{ kN/m}^3$

- Drained Angle of Internal Friction, $\phi' \geq 35^\circ$
- Drained Cohesion Intercept, $c' = 0$ kPa

6.1 Back Analyses and Failure Cause

In order to assess before-failure geotechnical parameters within the fill, the environment at the time of failure and the mechanism which caused the slip, a series of back analyses were carried out using Slope/W software. As Morgenstern & Price's method satisfies force equilibrium, overall moment equilibrium and inter slice moment equilibrium as well as providing consistent results for all groundwater conditions (Bo 2003, Bo & Choa 2004), this method was applied and FOSs (factors of safety) from this method have been reported here.

Several cross sectional profiles were surveyed and provided by MTO, Kenora office. The highest embankment with the steepest embankment slope from the bigger slip (where excessive bulging was observed) was taken as the worst section to be analyzed. BH 4 was used as a reference borehole for the selected section. Some low SPT blow counts encountered in BH 4 from 6.8 to 7.8 m depth were considered as after-failure characteristics, and a weak zone with a slightly lower angle of internal friction of 28 degrees was applied. A soft silty clay of 5 m thickness and which wedged out underneath the embankment slope was also assumed. A groundwater level was assumed to be 7.4 m below the present grade (335.94 m) at the BH 4 location, and at the surface at the northern and southern toes of the embankment. With this measured groundwater profile, calculations indicate that the embankment has sufficient FOSs for stability both under short and long term conditions.

The analyses found, however, that the embankment would fail if the groundwater level rises 6 m above the present level within the embankment. Both drained and undrained conditions were applied to the clay for this case. The geometry of the failure slip is mainly in the fill material and very

much similar to that observed in the field. This condition is possible considering the much higher embankment on the western side together with the limited drainage outlet and gentle gradient of ditch parallel to the highway embankment. This valley or ditch is the only surface water drainage outlet for that area and runoff is likely to be high due to significant exposures of rock outcrops.

Analyses were also repeated for the smaller failure area at the BH 5 location using geotechnical parameters from the back analysis at BH 4.

FOSs obtained for various conditions are shown in Table 1.

Table 1. Factor of Safety obtained from back analyses

Reference Section	Groundwater Condition	FOS		Remarks
		Undrained In Clay	All Drained Parameters	
BH4	With measured level	1.36	1.23	
BH4	With high GW within the embankment	0.99	0.87	See Enclosures 19 and 20
BH5	With measured level	1.34	1.48	
BH5	With high GW within the embankment	1.07	0.97	See Enclosures 21 and 22

In addition to the circular type failures described above, a failure mechanism where part of the embankment slides over a thin clay layer near the base of the fill was considered. The results of this translational analysis indicate that even with the high water levels required to cause a circular failure (described above), the translational mode is unlikely to occur.

As a sensitivity analysis, additional analyses were carried assuming the unlikely possibility of a 5 m thick, normally consolidated clay layer extended beneath the entire south lane. In this case the

embankment also has sufficient FOS with existing groundwater levels but the embankment has a marginally low FOS of slightly below unity when the groundwater level rises 6 m above current level.

In conclusion, the back analysis combined with post-failure observations found that the most likely cause of failure was a raised water table within the embankment. This would likely have been caused by a high recharge rate through permeable embankment zones, either from uphill within the embankment or from a very high north ditch storm water level or both, associated with the heavy rainfall 1 to 2 days previous. The above may also have been associated with seepage out of the south slope, involving internal erosion and/or unstable flow of fine sand materials. Other possible contributors include weak clay or organic zones that are softer and thicker than those detected in the boreholes, and undetected artesian pressures.

6.2 Stability Analysis With A Counter Berm

The stability analyses were carried out with a remedial profile for short and long term conditions under the high groundwater level in the embankment established by the back analysis. The parameters determined from the back analysis were used as input data. In the proposed remedial profile a slope of 3 h to 1 v was assumed on the southern side of the slope in order to achieve the stability within the embankment fill. In the analyses, a counter berm at the toe of 6 m width and approximately 3 m height was provided in order to improve the stability. As the elevation of the toe of the embankment elevation varies between 334.1 m and 335.65 m, it is recommended that the elevation of the counter berm would be kept between elevation 338 and 341 m (which includes an allowance for future settlement) depending on the height of the existing embankment.

The above noted configurations provide a minimum design factor of safety of at least 1.33 for end of construction and final long term drained conditions.

The following table tabulates the FOSs obtained from the analyses with proposed remedial design:

Table 2. Factor of Safety with proposed remedial design

Reference Section	Groundwater Condition	FOS		Remarks
		Undrained	Drained	
BH4	With high GW level within the embankment	1.69	1.72	See Enclosure 23 and 24
BH5	With high GW Level within the embankment	1.33	1.39	See Enclosure 25 and 26

A series of sensitivity analyses were also carried out for the clay layer under long term drained condition with the proposed remedial profile. In the sensitivity analyses the drained internal friction angle of silty clay was reduced down to 22 degrees. The FOS is found to be greater than 1.3 even with a low drained friction angle of 22 degrees for both circular and non-circular slip.

6.3 Drainage Stabilization Options

Analyses were carried out assuming a 3h to 1v re-constructed slope with no counter berm and a lowered water table within the embankment. This configuration was found to be adequate (FOS > 1.5) when the water table at the centerline is at the level of the south toe. To achieve this level, the water table would need to be lowered. There is, however, insufficient subsurface information for the detailed design of drainage measures to achieve this.

6.4 Settlement

The existing embankment is unlikely to settle further considering both the very thin nature of the clay layer encountered in boreholes underneath the embankment and the age of the embankment. Because the clay layer below the embankment is thin, all primary consolidation will have been completed.

The new fill at the counter berm locations is likely to extend over a firm clay deposit. It will induce primary and secondary consolidation settlement. Maximum settlements induced by the proposed counter berm fills due to primary and secondary consolidation of any clay deposits have been roughly estimated (without the benefit of laboratory consolidation tests).

The following table indicates the settlements calculated for construction as per Drawing 2.

Selected C_c	Primary Consolidation of Silty Clay (mm)	Secondary Consolidation of Silty Clay Over 25 yrs. (mm)	Total Settlement After 25 years (mm)
Maximum	570	23	593
Minimum	400	18	418

It is expected that most of the primary consolidation of the silty clay will occur over a period of 8.5 years. It is likely that 35% of the primary consolidation will have occurred within the first year. Secondary consolidation has been calculated over a 25 year period.

The settlements along the longitudinal profile of the counter berm will vary and be roughly proportional to the thickness of the silty clay deposit below the counter berm.

Settlements attributed to the sands within and below the fill have been estimated to be less than 25 mm and are expected to occur during construction.

Considering an estimated settlement of greater than 500 mm, part of which will occur during construction, an over-height berm design of 500 mm is recommended and this has been incorporated into the design.

Settlement as a result of a lowered water table as a potential remedial embankment drainage measures has not been assessed at this time.

7.0 RECOMMENDATIONS

7.1 Stabilization

For rapid reconstruction of the failed embankment, a new counter berm construction is proposed from Stations 12+080 to 12+320 at the south embankment toe. The existing highway embankment is approximately 4 to 11 m in height. The dimension of the counter berm should be 6 m in width and about 3 to 4 m in height with gradient of slope not steeper than 3 h to 1v. The following 6 m wide berm design elevations (includes over-height allowance) are recommended:

- Station 12+080 to 12+150 338.0 and 341.0 m (2 levels)
- Station 12+150 to 12+320 338.0 m

While a recompacted embankment fill with slightly gentler slope provides stability within the fill, the counter berm will also improve stability against possible sliding through weak zones below the fill. The additional load of the proposed counter berm will induce primary and secondary consolidation settlement. It may drag down the south shoulder of the embankment to a certain extent but this will be much smaller than settlement in the berm itself. Therefore this impact may be considered insignificant and maintained through routine maintenance.

It is recommended that the counter berm be constructed in accordance with Drawings 2 & 3. The construction sequence should be commenced from the east and proceed towards the west. Each lift of the berm construction should proceed over its entire width as it is built. Excavation and re-profiling of the existing embankment should be carried out only after completion of the construction of the counter berm at the toe.

It is noted that at BH 5 the critical failure mode is with the clay in an undrained condition, based on only the lowest of 2 field vane tests. It is recommended that this data be confirmed during construction with additional hand auger holes and vane tests in the clay plain south of the toe.

7.2 Surface and Groundwater Management

The most likely mechanism triggering the slip is considered to be groundwater within the fill. The stability could be improved if the groundwater level within the embankment can be kept at the current level or lowered. This can be achieved both by surface and groundwater management schemes. Following are a few options by means of surface water management.

- Increase the gradient of the existing ditch (to reduce storm water levels in the ditch).
- Provide lining for the drainage ditch (to minimize infiltration).
- Improve rate of runoff on the upstream slope of embankment (to minimize infiltration).
- Provide an additional discharge culvert to reduce storm water levels in the ditch).

Possible options of managing or lowering the groundwater within the embankment fill are:

- Provide horizontal drainages across the embankment at a suitable spacing.
- Provide permeable trench drains across the embankment at suitable interval, and possibly in strategic locations on the north side.

Details of these options can be further studied and provided if necessary. As a minimum, improving the north ditch gradient is recommended as well as ensuring adequate culvert capacity for the Ministry's design storm.

7.3 Temporary Detour

Based on two additional boreholes (BH 7 and 8) carried out at 12+200 and 12+225 in the recent investigation, the ground at 12 m north from the centre line of existing highway is underlain by sand to at least a 4 to 5 m depth. At BH 7 the auger refusal within fill at 4.7 m. Weak soils below this fill would appear unlikely. Fill widening for fill heights up to 3 m is considered feasible, if carried carefully with expectations that some unpredictable settlement may occur.

The widening of the roadway should be in accordance with Drawing 2 & 3. The gradient of the widened embankment slope should not be steeper than 3h to 1 v. New fill should be keyed in to the existing fill with suitable benching in accordance with drawing OPSD 208.010.

7.4 Construction

The construction methodology must be in accordance with all relevant Ministry guidelines. The contractor's methods and equipment must be suitable for the site conditions and materials used. Furthermore, with the uncertainty with respect to potential destabilization as a result of heavy rainfall during construction, vibration resonance from large compactors or other unexpected mechanisms, full time surveillance of the construction should be carried out by qualified personnel for any signs of impending movements. If noted, the construction should be immediately halted, the embankment stability reassessed, and the methodology reviewed and modified as required.

Equipment and worker traffic over the native subgrade soil should be avoided and the first lift of fill should be placed and compacted immediately after excavation to provide a working surface and protection for the subgrade.

The embankment fill affected by the failure should be sub-excavated and reconstructed by benching in accordance with OPSD 208.010, Benching of Earth Slopes. The excavated material should be recompacted to 98 % of standard proctor compaction with suitable lifts (not more than 500 mm thick) to form a slope gradient of 3 h to 1v as shown in Drawing 2. Excavation should be carried out section by section, and the section length at any one time should not be greater than 30 m, measured at the bottom of excavation along the highway centerline. No section should be left unfinished overnight.

Ideally, sub-excavation and replacement of all slide mass materials should be carried out. This is sound construction practice for slope stabilizations, albeit more critical for fine grained materials than granular materials. However, excavation is proposed only to the top of proposed counter berms in order to minimize detoured work and for the ease of construction, as requested by MTO. As such, it will leave part of the disturbed soils untreated.

Embankment excavation and reconstruction should extend not less than 1 m beyond the failure zone at highway level. Traffic wheel loads should not be allowed closer than 1.6 m to the edge of excavation. Temporary slopes should not exceed 2h:1v, but at the top of the embankment may be steeper up to 1:1 over heights of up to 3 m. No parking should be allowed on the westbound lane until remedial measures are completed.

8.0 LONG TERM STABILITY

Regardless of the results of the above described investigation, several uncertainties remain for this site. In particular, these are associated with the following:

- complex subsurface (bedrock, soil and groundwater) conditions,
- the limited soil information and testing,
- the long history of movements in this area,
- the unusual failure trigger conditions,
- the possibility of other undetected failure mechanisms,
- the fact that all weak slide material is not being removed and
- the possibility of even more severe rainfall events.

It is therefore recommended that in the long term further improvements to the embankment's stability be implemented. Options that can be assessed in more detail include:

- groundwater and stormwater drainage improvements,
- additional reinforcement of the foundation soils and
- relocation of the embankment to more stable ground to the north.

In the meantime, frequent visual and survey monitoring is recommended, with an allowance for contingencies of further maintenance as a result of unexpected movements in the south side slope and shoulder.

9.0 MONITORING DURING AND AFTER CONSTRUCTION

Given the degree of uncertainty described in the foregoing section, the following monitoring scheme is recommended until long term stabilization options have been implemented:

- full time surveillance for signs of instability during construction
- additional hand auger holes and field vane tests at the toe (see Section 7.1)
- weekly visual inspection for cracks, deformation and movement during construction for the first year after construction,
- visual inspection of surface water drainage and adequacy of drainage during storm runoff, and
- measurement and assessment of vertical settlement along the counter berm, shoulder and traffic lane, at not less than 3 sections and at least bi-monthly during the first year after construction.

10.0 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.

11.0 REFERENCES

1. Bo, M.W & Choa, V (2004). Reclamation and Ground Improvement, Thomson Learning.
2. Bo, M.W (2005). "Application of Observational Methods in Embankment Built on Soft Ground to Overcome Limitations of Slope Stability Analyses". *International Conference on Railway Engineering*, London, UK.
3. Hallett, D.R and Roed, M.A. (1980). "Northern Ontario Engineering Geology Terrain Study 20", Ret Portage Bay Area (NTS 52E/NE and Part of 52E/NW), District of Kenora, Ontario Geological Survey.
4. Kenney, T.C., "Discussion" (1959). Proc. ASCE, Vol. 85, no. SM3.
5. Morgenstern, N.R., and Price, V.E. (1965). "The Analysis of the Stability of general Slip Surfaces". *Geotechnique*, Vol. 15.

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MWB:dm



Mike Fabius, P. Eng.
Principal

ENCLOSURES

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. _____ LOCATION 12+230.5 Rt 5.6m ORIGINATED BY A.F
DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
DATUM Geodetic DATE 2007 07 05 - 05.07.07 CHECKED BY M.W.B.

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								
								○ UNCONFINED	✕ FIELD VANE	□ QUICK TRIAXIAL	★ LAB VANE					
								WATER CONTENT (%)								
341.5	Ground Surface															GR SA SI CL
340.9	ASPHALT - 140mm															Standpipe water level measured 12 & 36 hours after completion
341.1	FILL - Sand & gravel															
0.4	FILL - Sand, some gravel, brown, loose		1	AS												
			2	SS	7											
340.0																
1.5	FILL - Sand, silty, some gravel, brown, compact		3	SS	17											33 48 (18)
339.4																
2.1	FILL - Sand, some gravel, occasional cobbles, brown, loose		4	SS	8											
338.8																
2.7	FILL - Sand, silty, some gravel, occasional cobbles, brown, loose		5	SS	5											21 59 (20)
			6	SS	5											
			7	SS	8											
			8	SS	21											
			9	SS	5											
	----- - compact		10	SS	5											
	----- - loose		11	SS	5											Wet @ 6.7m
			12	SS	3											
			13	SS	8											
333.3																
8.2	SAND - Silty, some gravel, occasional cobbles, grey, loose		14	SS	10											
	----- - layered silt & sandy silt, grey, dense		15	SS	34											12 67 (21)
332.1																
9.4	SAND & GRAVEL - Silty, trace cobbles, grey, compact		16	SS	30											
331.5																
10.0	SAND - Silty, some gravel, trace cobbles, grey, compact		17	SS	18											
			18	SS	20											
330.2																
11.3	End of Borehole @ 11.3m															

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\times^3, \star^3 : Numbers refer to Sensitivity \bigcirc 3% STRAIN AT FAILURE

ENCLOSURE 1

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. _____ LOCATION 12+261 Rt 5.6m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 06.07.07 - 06.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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			500	1000	1500	2000	2500					
340.3	Ground Surface																
340.0	ASPHALT - 120mm		1	BS			340										Dry on completion
339.8	FILL - Sand & gravel		2	BS													
339.8	FILL - Sand, some gravel & silt, brown		3	SS	13												25 58 (17)
0.8	- cobbles & boulders																
	FILL - Sand, silty, some gravel, occasional cobbles & boulders, brown, loose to compact		4	SS	5		339										
			5	SS	8		338										
	- grey/brown		6	SS	10		337										25 56 (19) Cave @ 3.0m
336.7	End of Borehole @ 3.6m Auger Refusal																
3.6																	

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

ENCLOSURE 2

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. _____ LOCATION 12+233.6 Rt 34.0m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 06.07.07 - 06.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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		
						500	1000	1500	2000	2500							
						○ UNCONFINED ✕ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE					WATER CONTENT (%)						
						15	30	45	60	75							
335.4	Ground Surface																
335.0	TOPSOIL - Sandy		1	AS													
	FILL - Sand/rockfill, some gravel																
334.3																	
1.1	End of Borehole @ 1.1m Auger Refusal																

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

ENCLOSURE 3

ON_MOT-SCPT_CS-TB-007607 ONTARIO MOT.GPJ DST_MIN.GDT 16/07/07

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. _____ LOCATION 12+192 Rt 5.3m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 06.07.07 - 06.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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			500	1000	1500	2000	2500					
343.3	Ground Surface																
340.0	ASPHALT - 130mm		1	BS			343										Dry on completion
342.8	FILL - Sand & gravel		2	BS													
0.6	FILL - Sand, some gravel & silt, occasional cobbles, brown, compact to dense		3	SS	16		342										20 64 (16)
			4	SS	16												
			5	SS	45		341										
340.3	FILL - Sand & gravel, silty, occasional cobbles, brown, compact		6	SS	30		340										
3.0	- Sand - 150mm, some gravel, brown		7	SS	22		339										
	- Clay - 100mm, trace organics, grey		8	SS	17												
338.1	- Sand - 30mm, brown		9	SS	8		338										
5.2	FILL - Sand, silty, some gravel, occasional cobbles, brown, loose		10	SS	11		337										
337.2	FILL - Sand, some gravel & silt, occasional cobbles, brown, compact		11	SS	6		336										Cave @ 6.6m
6.1	FILL - Sand, silty, some gravel, occasional cobbles, grey, loose to compact		12	SS	6												
336.4	- brown/grey		13	SS	30		335										17 66 (17)
6.9	SAND - occasional gravel & cobbles, brown, compact		14	SS	12		334										
9.1	SAND - Silty, trace gravel, grey/brown, compact to dense		15	SS	35		333										
333.4	- occasional cobbles		16	SS	12		332										
9.9	- Clay, silty, layered, grey - 300mm		17	SS	26		331										
	- Sand & gravel, brown - 100mm		18	SS	40		330										
328.1	SAND & GRAVEL - Silty, occasional cobbles, grey, very dense		19	SS	100+		328										21 60 (19)
15.4	End of Borehole @ 15.35m																

Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ENCLOSURE 4

RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. _____ LOCATION 12+119.6 Rt 5.0m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 06.07.07 - 06.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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			500	1000	1500	2000	2500					
346.9	Ground Surface																
346.0	ASPHALT - 120mm																
346.4	FILL - Sand & gravel																
0.4 346.1	FILL - Sand, some gravel, brown		1	AS													
0.8	FILL - Sand & gravel, occasional cobbles & boulders, brown, very dense		2	SS	52		346										
345.4																	
1.5	FILL - Sand, some gravel & silt, occasional cobbles & boulders, brown, loose to very dense		3	SS	95		345										
			4	SS													
							344										
			5	SS	61												
							343										
			6	SS	9												
							342										
			7	SS	20												
341.6																	
5.3	FILL - Sand, silty, trace gravel, occasional cobbles, brown, loose to very dense		8	SS	10		341										
			9	SS	89												
340.0							340										
6.9	FILL - Sand, some gravel & silt, occasional cobbles, brown, compact		10	SS	24												
	----- - trace gravel & silt		11	SS	23		339										
338.5	----- - sandy silt layer, brown - 30mm																
8.4	FILL - Sand, silty, layered, occasional gravel & cobbles, brown, dense		12	SS	32		338										
337.8																	
9.1	CLAY - Silty, layered, trace sand seams, grey, dense		13	SS	45												
337.4																	
338.8	SAND - Silty, layered, brown																
9.7	SAND & GRAVEL - Silty, brown, very dense		14	SS	56		337										
336.5																	
10.4	End of Borehole @ 10.4m																

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

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. _____ LOCATION 12+201 Lt 12.0m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			500	1000	1500	2000	2500		
341.2	Ground Surface													
340.0	TOPSOIL - Sandy, 30mm						341							Dry on completion
	FILL - Sand, some gravel, occasional cobbles, & silt, brown													
340.4														
0.8	FILL - Sand, silty, occasional cobbles, brown		1	AS			340							
	----- - some organics													
339.1	----- - occasional gravel & boulders		2	AS										
2.1	End of Borehole @ 2.1m Auger Refusal													

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

ENCLOSURE 6

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. _____ LOCATION 12+225 Lt 12.0m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			500	1000	1500	2000	2500		
340.2	Ground Surface													
340.0	TOPSOIL - Sandy, 30mm FILL - Sand, silty, brown ----- -occasional gravel & cobbles		1	AS			340							Dry on completion
							339							
			2	AS			338							
	----- -occasional boulders		3	AS			337							
			4	AS			336							Cave @ 4.1m
335.5	End of Borehole @ 4.7m Auger Refusal													
4.7														

\times^3, \star^3 : Numbers refer to Sensitivity \bigcirc 3% STRAIN AT FAILURE

ENCLOSURE 7

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

W.P. _____ LOCATION 12+233.6 Rt 33.5m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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			500	1000	1500	2000	2500					
336.7	Ground Surface																
336.0	TOPSOIL - Sandy, 30mm FILL - Sand, silty, brown - trace gravel & cobbles		1	AS			336										Water level estimated based on wet surface conditions.
	- some gravel, trace cobbles		2	AS			335										
334.2	SAND - some gravel, cobbles, & boulders, brown		3	AS			334										Cave @ 2.2m
2.5							333										
332.4	End of Borehole @ 4.3m Auger Refusal																
4.3																	

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

RECORD OF BOREHOLE No AH1

1 OF 1

METRIC

W.P. _____ LOCATION 12+160 Rt 41.1m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			500	1000	1500	2000	2500		
341.0	Ground Surface													
340.0	ORGANICS													
0.2	CLAY - Silty, soft to firm		1	AS			340							Water level estimated based on wet surface conditions.
							339							
							338							
							337							
336.5			2	AS										
4.5	End of Borehole @ 4.5m													

\times^3, \star^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No AH2

1 OF 1

METRIC

W.P. _____ LOCATION 12+126 Rt 42.7m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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	500	1000	1500	2000					
344.0	Ground Surface															
344.0 0.2	ORGANICS CLAY - Silty, soft to firm		1	AS												Water level estimated based on wet surface conditions.
341.5			2	AS												
2.5	End of Borehole @ 2.5m															

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

RECORD OF BOREHOLE No AH3

1 OF 1

METRIC

W.P.		LOCATION	12+095 Rt 42.7m	ORIGINATED BY	A.F
DIST	0.14	HWY	17	BOREHOLE TYPE	Hand Auger
DATUM	Geodetic	DATE	07.07.07 - 07.07.07	CHECKED BY	M.W.B.
COMPILED BY M.L.					

[illegible]

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

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RECORD OF BOREHOLE No AH4

1 OF 1

METRIC

W.P. _____ LOCATION 12+095 Rt 30.0m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			500 1000 1500 2000 2500						
345.2	Ground Surface													
344.0	ORGANICS - Sandy SAND - trace silt						345							Water level estimated based on wet surface conditions.
343.7			1	AS			344							
1.5	End of Borehole @ 1.5m Auger Refusal													

\times^3, \star^3 : Numbers refer to Sensitivity \bigcirc 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No AH5

1 OF 1

METRIC

W.P.		LOCATION	12+127 Rt 34.6m	ORIGINATED BY	A.F
DIST	0.14	HWY	17	BOREHOLE TYPE	Hand Auger
DATUM	Geodetic	DATE	07.07.07 - 07.07.07	COMPILED BY	M.L.
				CHECKED BY	M.W.B.

[illegible]

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

ENCLOSURE 13

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RECORD OF BOREHOLE No AH6

1 OF 1

METRIC

W.P. _____ LOCATION 12+126 Rt 29.3m ORIGINATED BY A.F
 DIST 0.14 HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY M.L.
 DATUM Geodetic DATE 07.07.07 - 07.07.07 CHECKED BY M.W.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	STATIC 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		
							500	1000	1500	2000	2500						
344.0	Ground Surface																
343.9	ORGANICS																
	SAND - trace silt																
342.4			1	AS													
1.6	End of Borehole @ 1.6m Auger Refusal																

\times^3, \star^3 : Numbers refer to Sensitivity \bigcirc 3% STRAIN AT FAILURE

ATT-MTO GS-TB-007607 ONTARIO MOT.GPJ DST_MIN.GDT 16/07/07

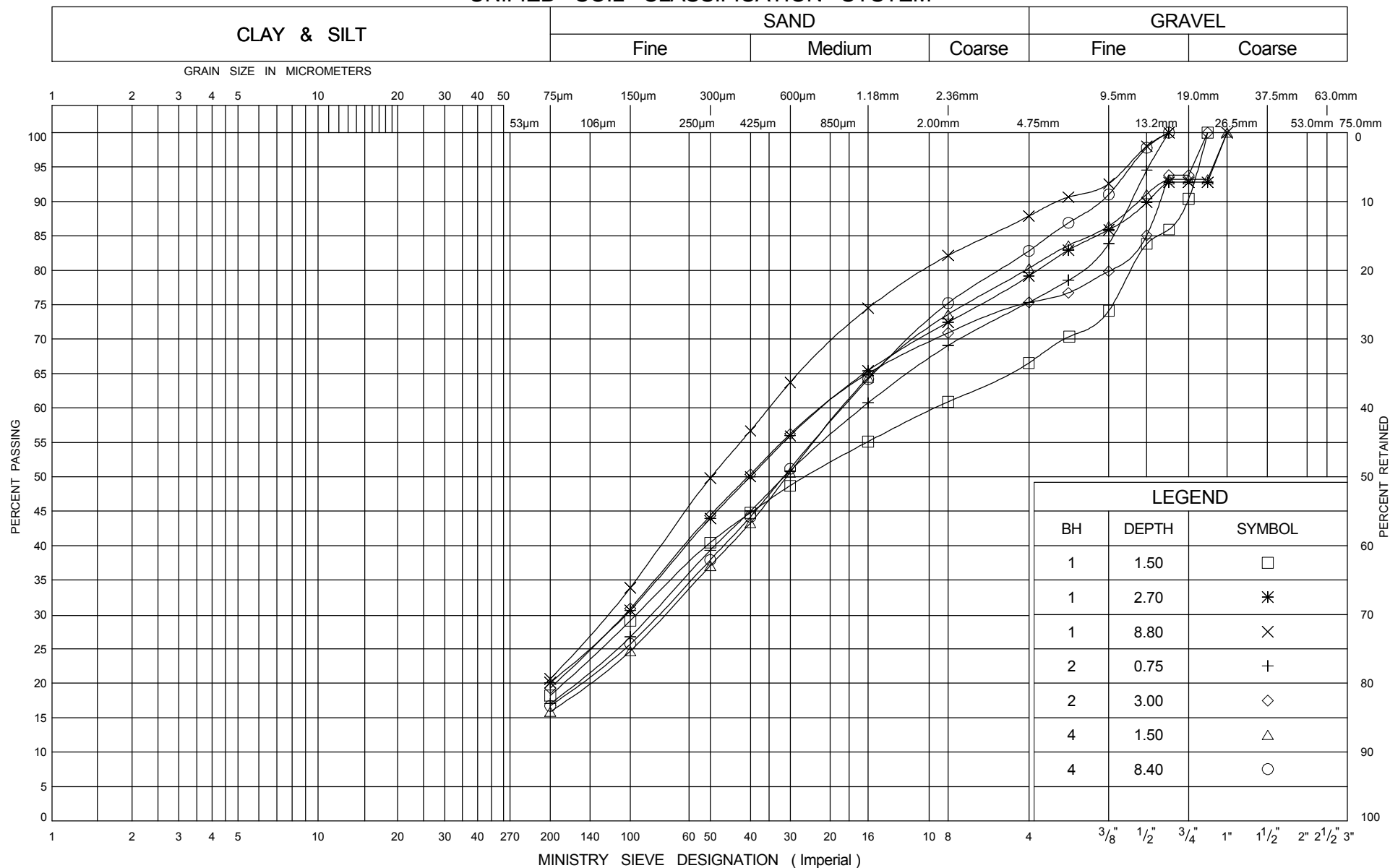


July 2007

Reference No.:

17 - 0.14

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

FILL - Sand, Silty, some gravel

ENCLOSURE 16

W P

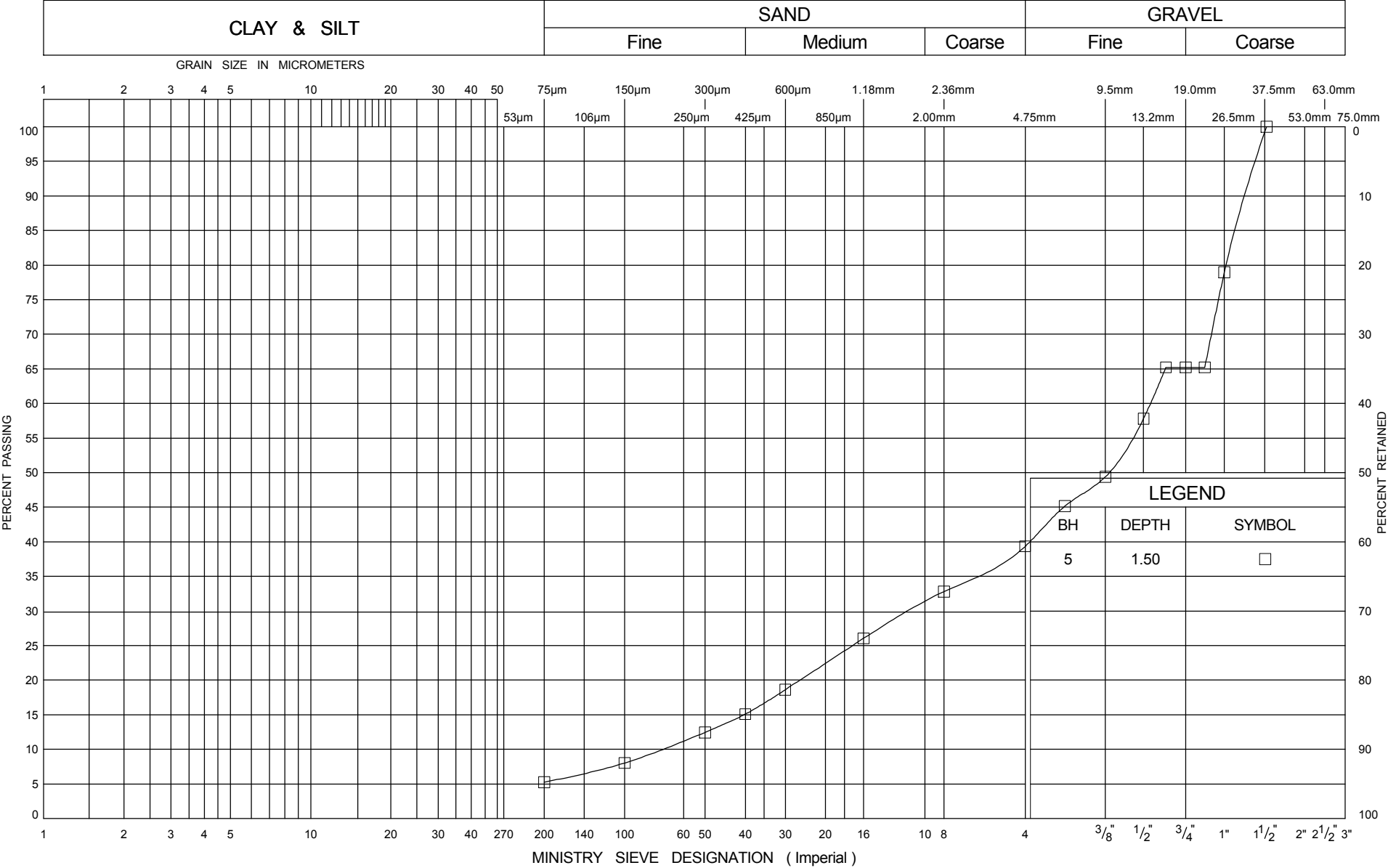
17



Ministry of
Transportation

Ontario

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

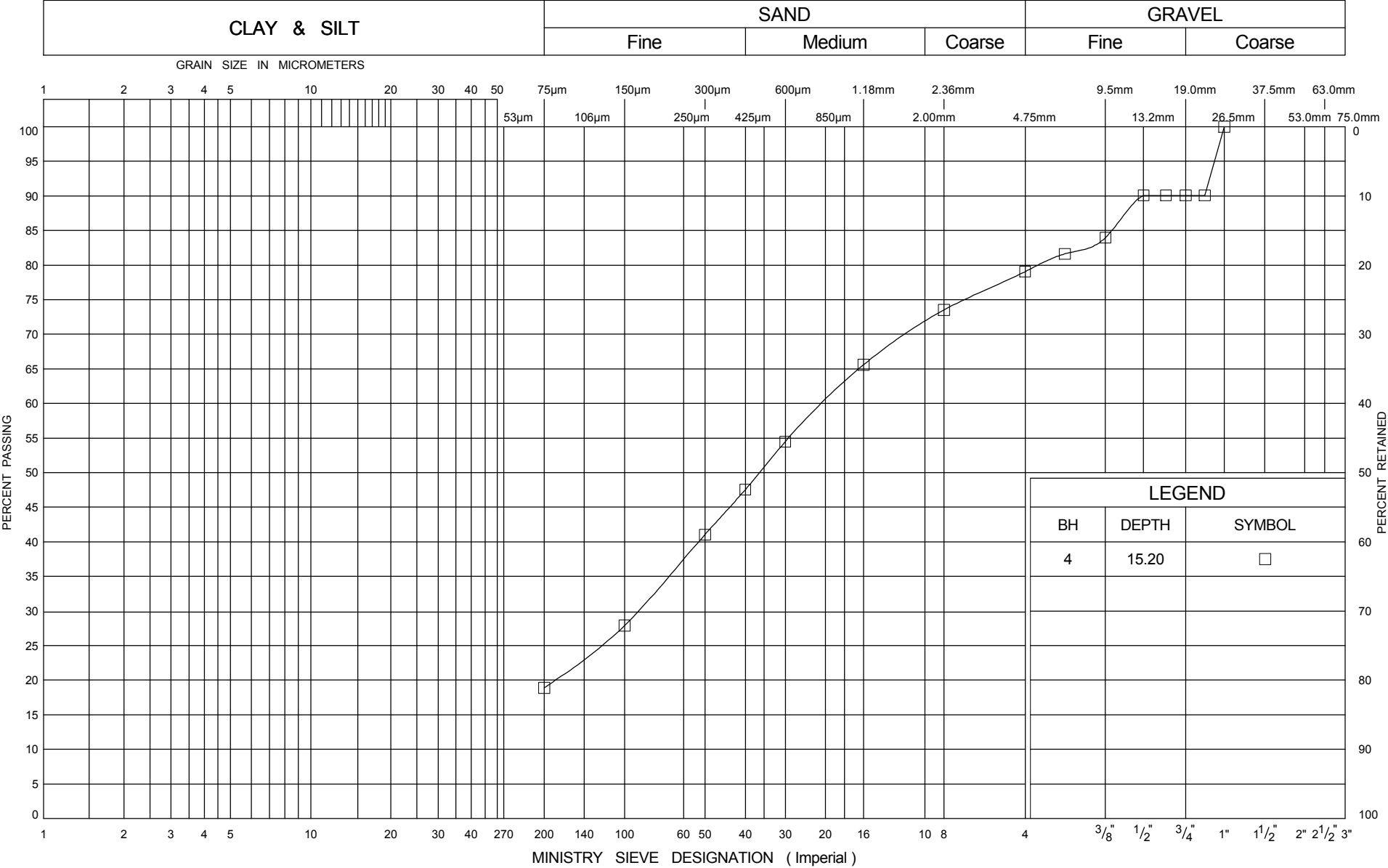
FILL - Sand & Gravel

ENCLOSURE 17

W P

17

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

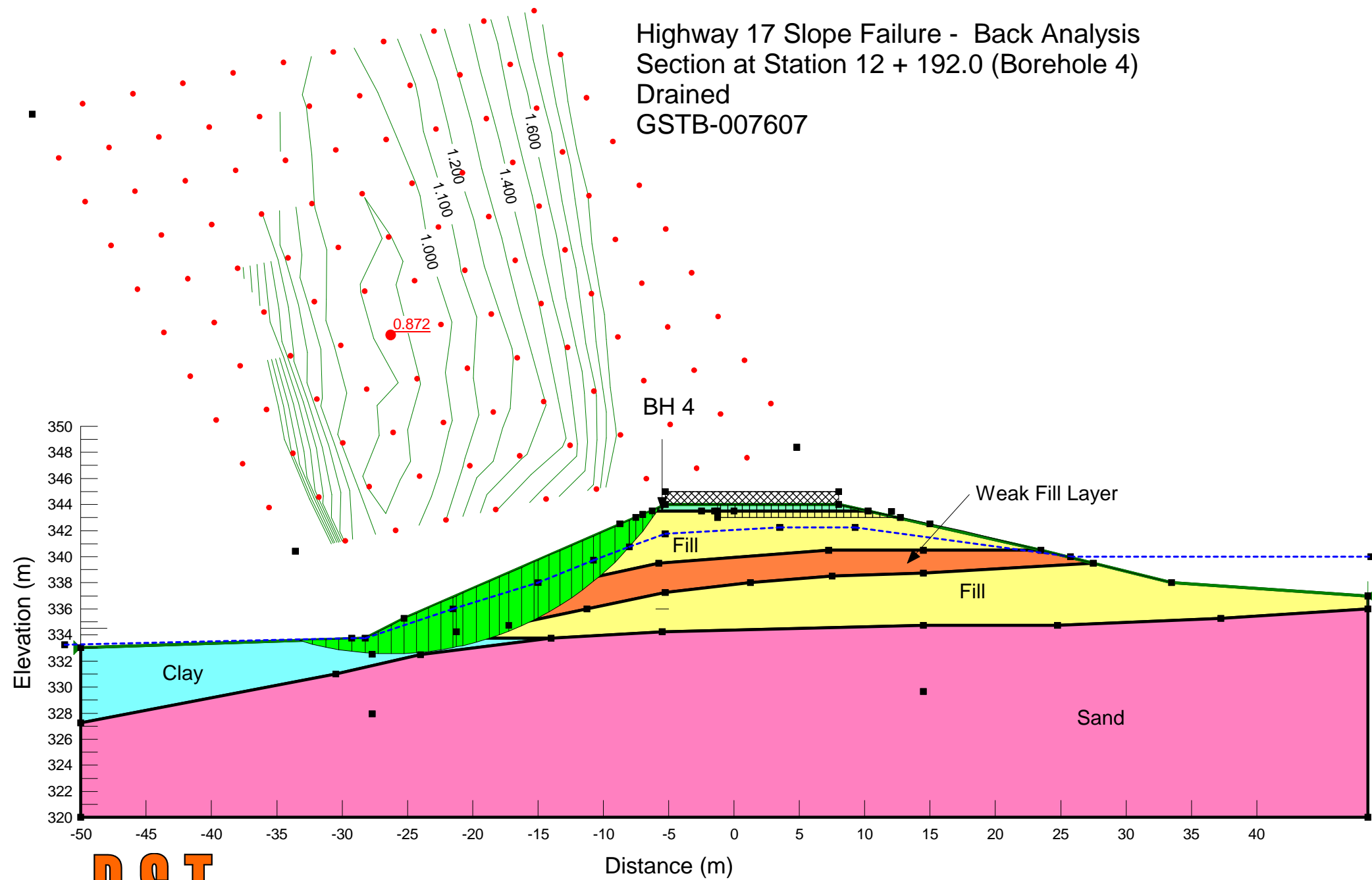
Sand & Gravel - Silty

ENCLOSURE 18

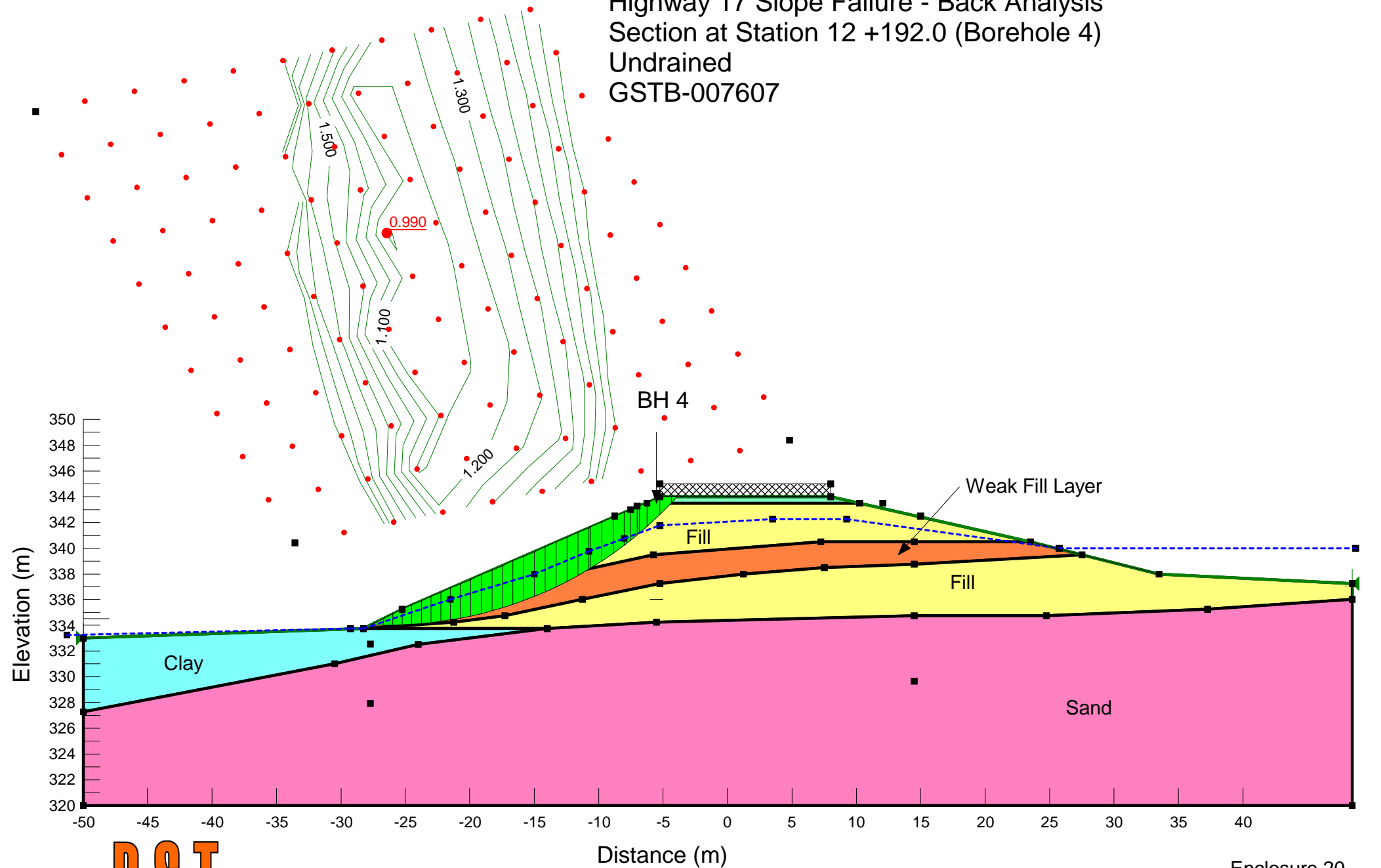
W P

17

Highway 17 Slope Failure - Back Analysis
Section at Station 12 + 192.0 (Borehole 4)
Drained
GSTB-007607



Highway 17 Slope Failure - Back Analysis
Section at Station 12 +192.0 (Borehole 4)
Undrained
GSTB-007607



Highway 17 Slope Failure - Back Analysis
Section at Station 12 + 119.6 (Borehole 5)
Drained
GSTB-007607

0.966

1.500

1.600

BH 5

Fill

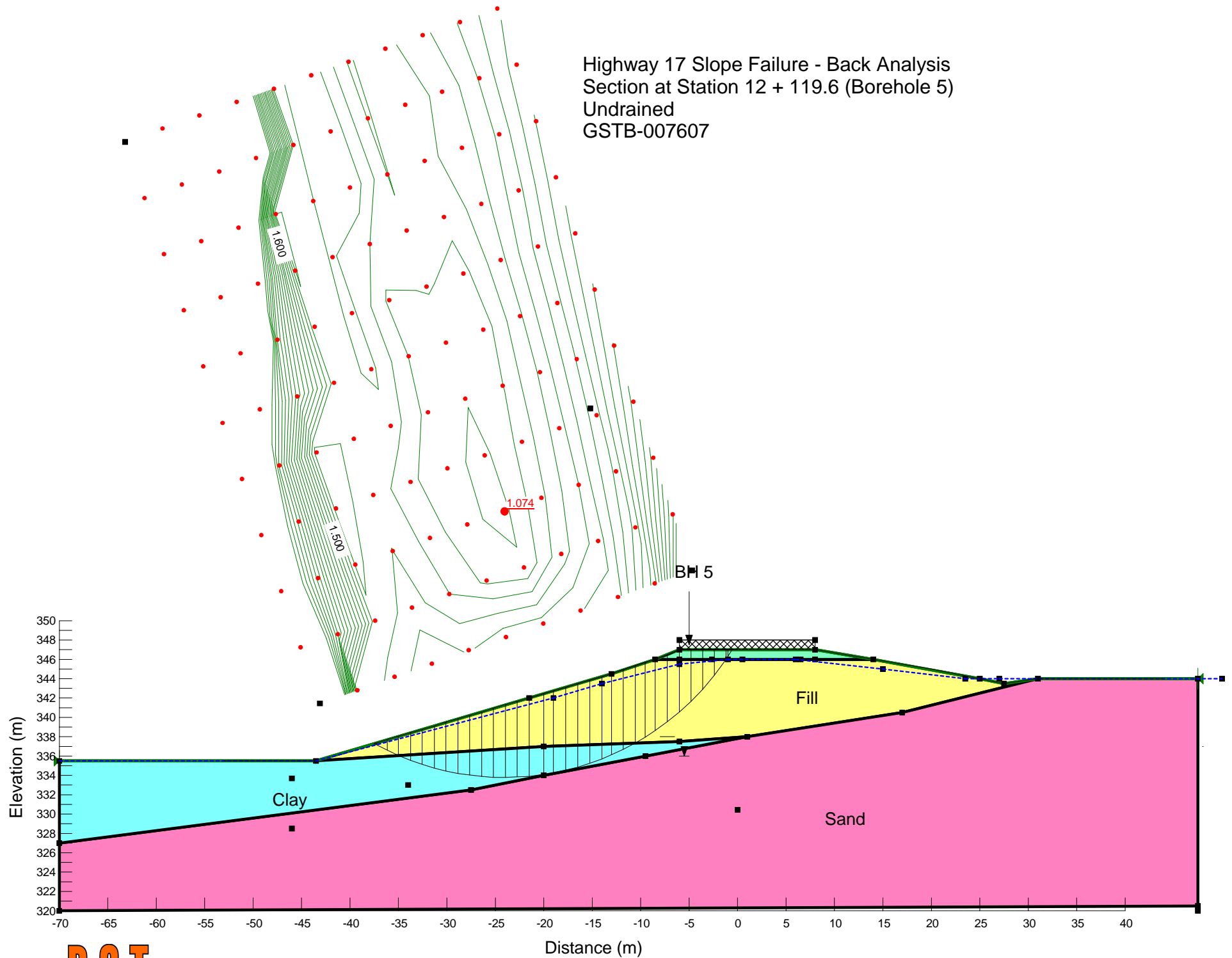
Clay

Sand

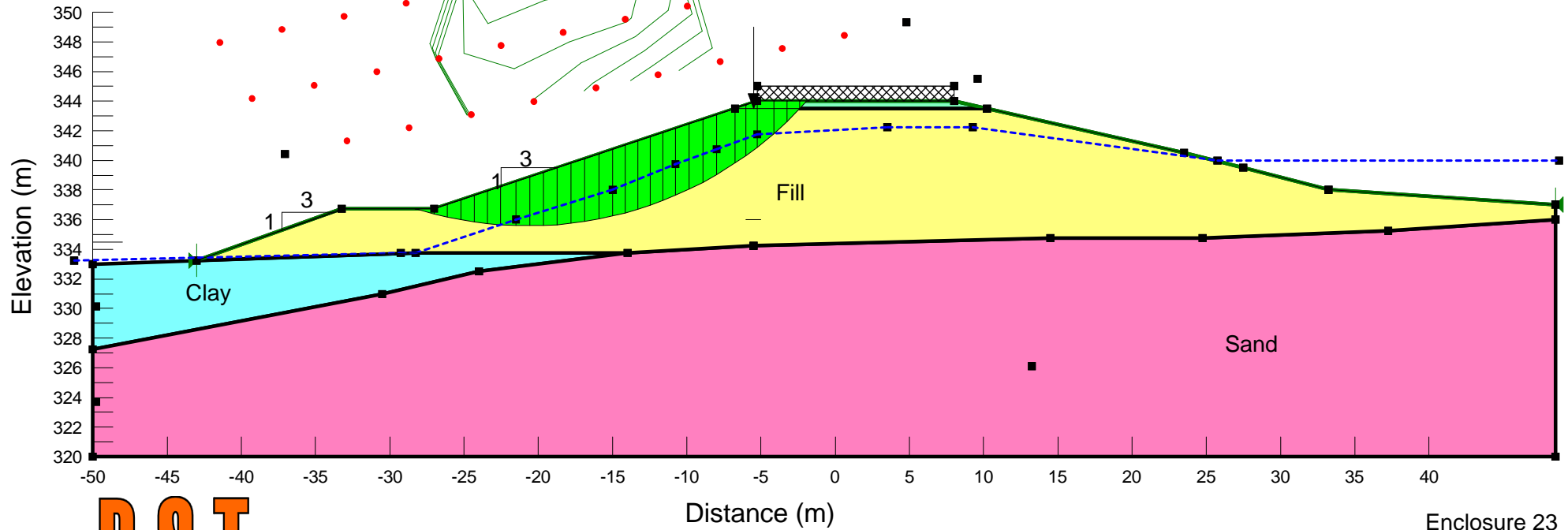
Elevation (m)

Distance (m)

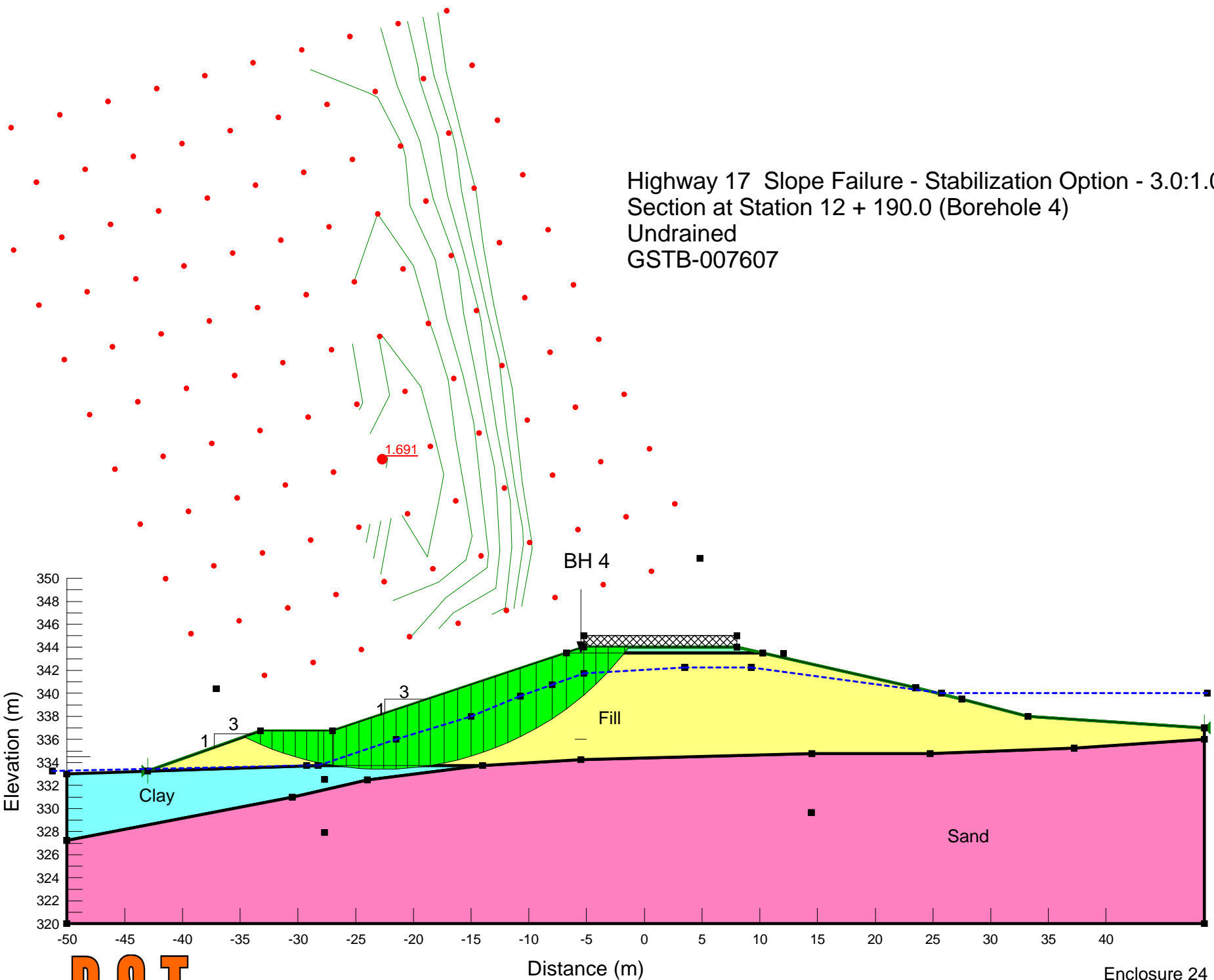
Highway 17 Slope Failure - Back Analysis
Section at Station 12 + 119.6 (Borehole 5)
Undrained
GSTB-007607



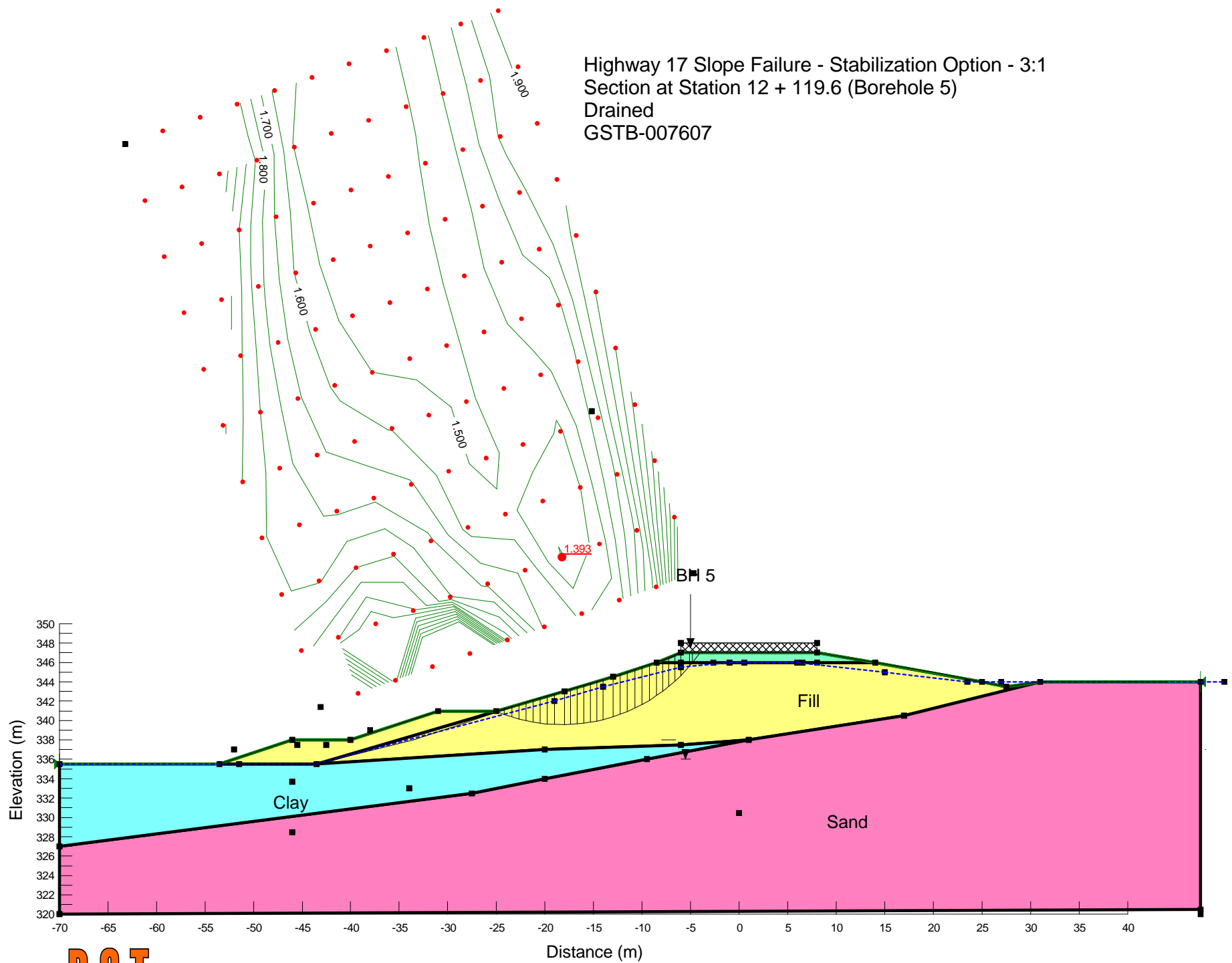
Highway 17 Slope Failure - Stabilization Option - 3.0:1.0
Section at Station 12 + 190.0 (Borehole 4)
Drained
GSTB-007607



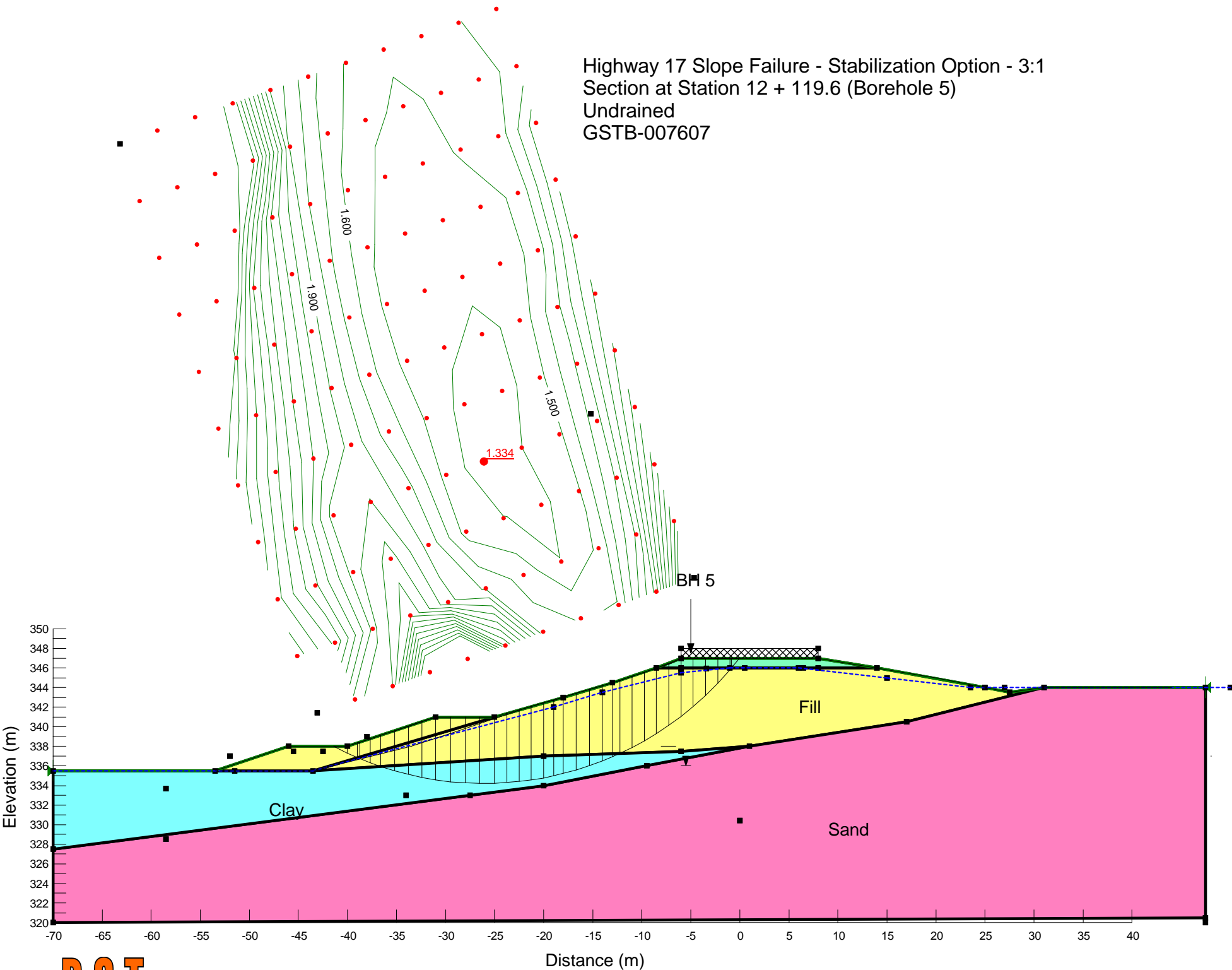
Highway 17 Slope Failure - Stabilization Option - 3.0:1.0
Section at Station 12 + 190.0 (Borehole 4)
Undrained
GSTB-007607



Highway 17 Slope Failure - Stabilization Option - 3:1
Section at Station 12 + 119.6 (Borehole 5)
Drained
GSTB-007607



Highway 17 Slope Failure - Stabilization Option - 3:1
Section at Station 12 + 119.6 (Borehole 5)
Undrained
GSTB-007607



APPENDIX 'A'
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. 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. Conditions can also change with time. 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.

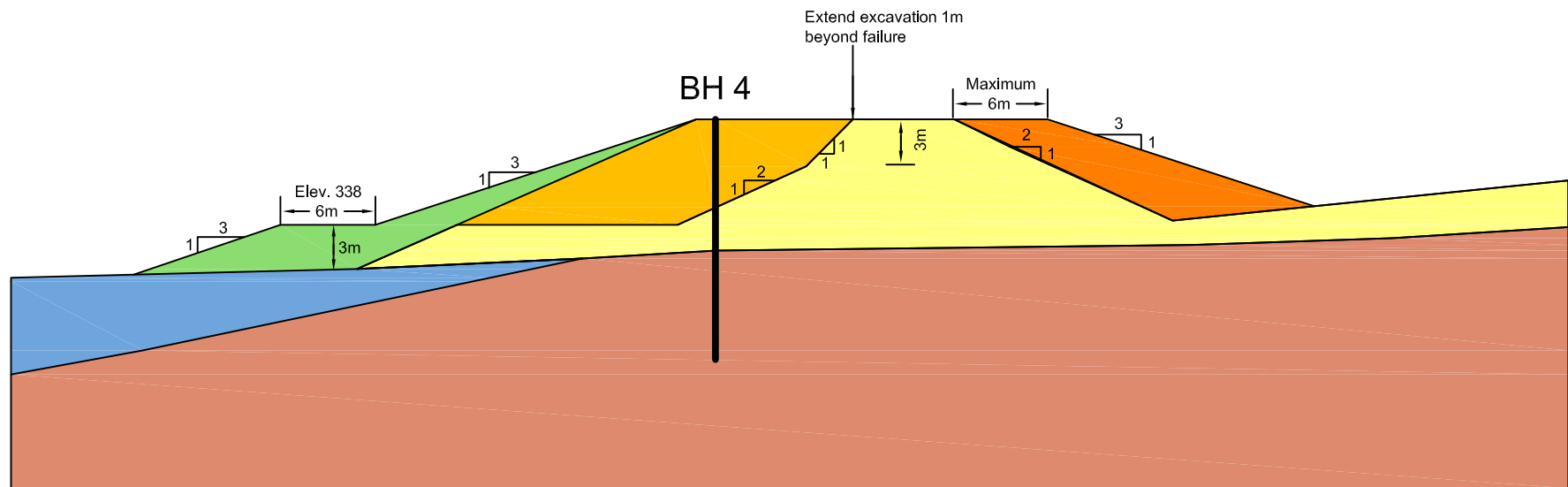
The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage 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.

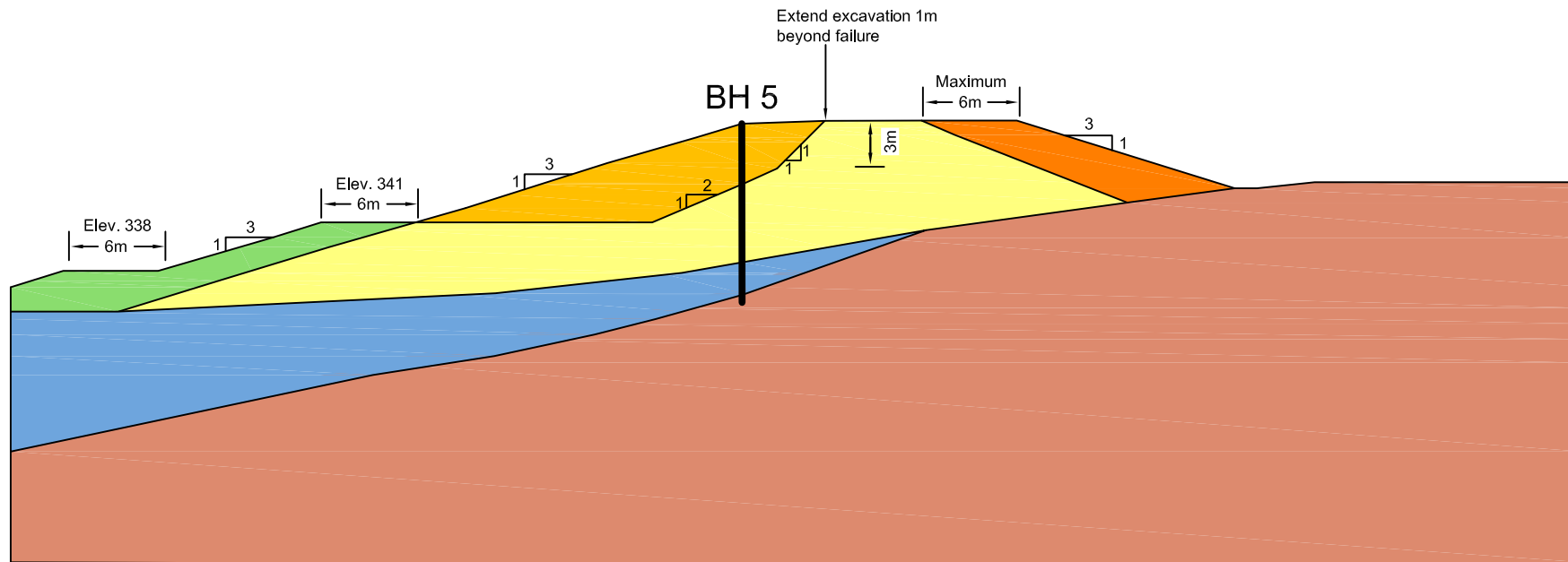
Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

APPENDIX 'B'
DRAWINGS




- EXISTING EMBANKMENT FILL (To be Excavated and Recompactd)
- EXISTING EMBANKMENT FILL
- SILTY CLAY
- DENSE SAND
- PROPOSED FILL
- ADDITIONAL FILL FOR DETOUR

DWG. TITLE: STABILIZATION AT STATION 12+150 TO 12+320		
		
PROJECT: SLOPE STABILIZATION OPTION AT BH4 HIGHWAY 17, BOYS TOWNSHIP		
CLIENT:	MTO	PROJECT NO.: GS-TB-007618
DATE:	JULY 2007	DRAWING 2



- EXISTING EMBANKMENT FILL & SAND (To be Excavated and Recompacted)
- EXISTING EMBANKMENT FILL
- SILTY CLAY
- DENSE SAND
- PROPOSED FILL
- ADDITIONAL FILL FOR DETOUR

DWG. TITLE: STABILIZATION AT STATION 12+080 TO 12+150		
 THUNDER BAY, ONTARIO www.dstgroup.com	PROJECT: SLOPE STABILIZATION OPTION AT BH5 HIGHWAY 17, BOYS TOWNSHIP	
	CLIENT: MTO	PROJECT NO.: GS-TB-007618
	DATE: JULY 2007	DRAWING 3

APPENDIX 'C'

SITE PHOTOGRAPHS



Figure 1. Tension crack along small slip



Figure 2. Relative width and depth of tension crack



Figure 3. Rock exposure on the northern side of Highway 17



Figure 4. Embankment looking towards the west on the southern side



Figure 5. Highway 17 looking towards the east



Figure 6. Highway 17 embankment looking towards the west on the northern side



Figure 7. Rock boulders on the southern side of the slope



Figure 8. Culvert on the southern side of the slope



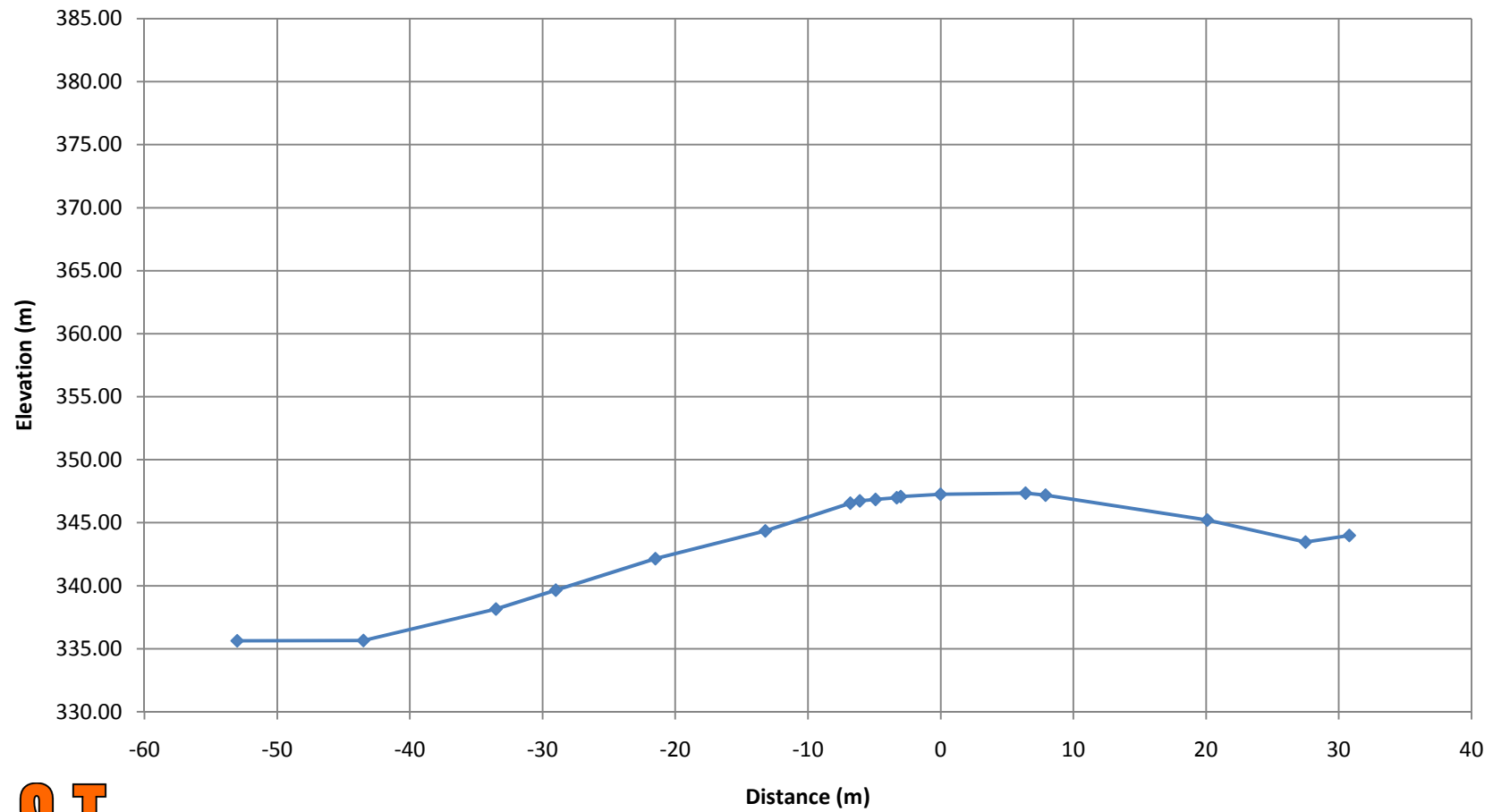
Figure 9. Culvert on the northern side of the slope

APPENDIX 'D'

POST-FAILURE CROSS-SECTIONS

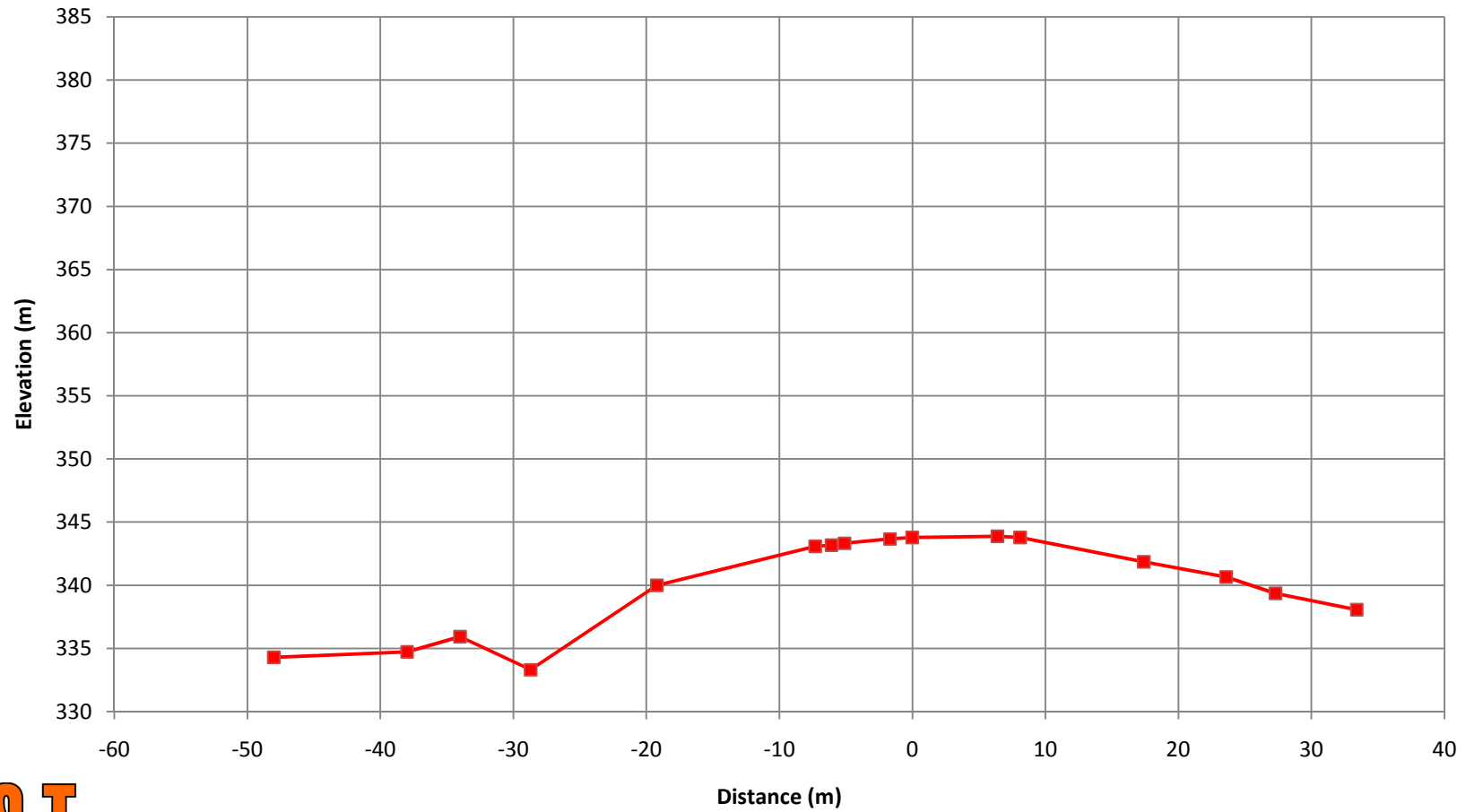
Section A-A
Highway 17 Slope Failure
Cross Section @ 12 + 119.6

—◆— BH5



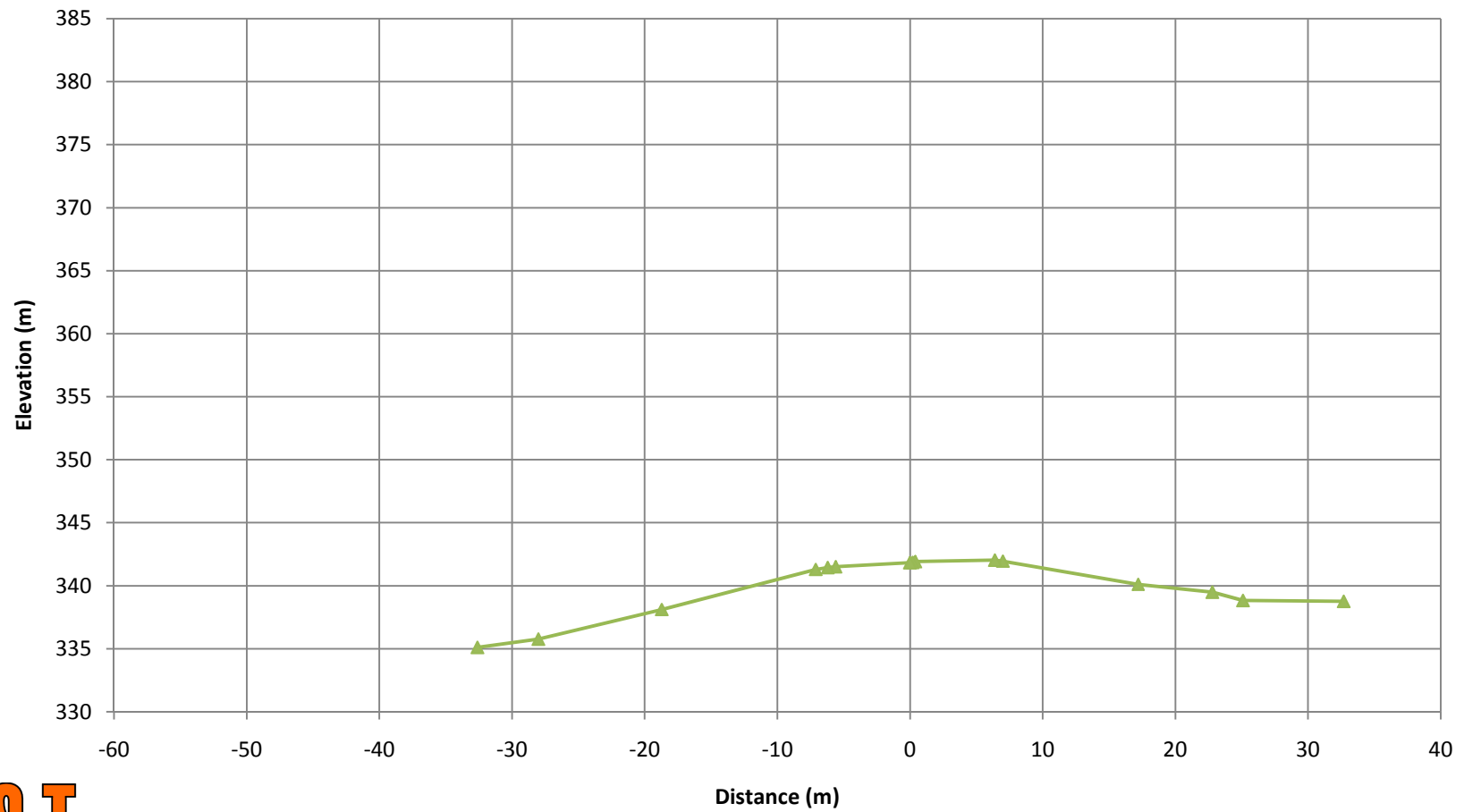
Section B-B
Highway 17 Slope Failure
Cross Section @ 12 + 192.0

—■— BH4



Section C-C
Highway 17 Slope Failure
Cross Section @ 12 + 230.5

—▲ BH 1

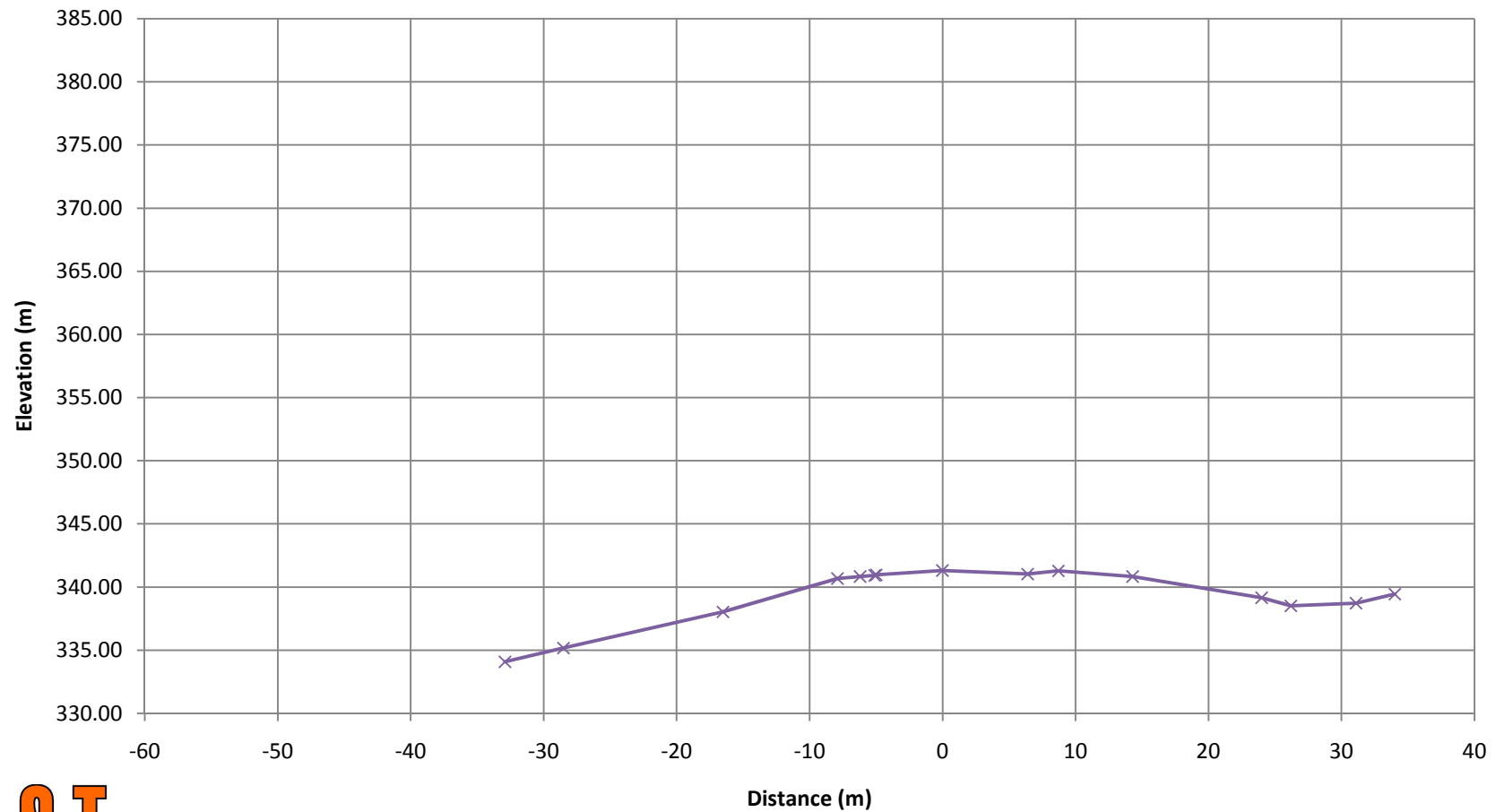


Data provided by MTO

Enclosure D3

Section D-D
Highway 17 Slope Failure
Cross Section @ 12 + 233.6

—x— BH3



Section E-E
Highway 17 Slope Failure
Cross Section @ 12 + 261.0

—*— BH2

