



FOUNDATION DESIGN REPORT

for

MERRITT ROAD UNDERPASS

SITE NO. 34-460

HIGHWAY 406 FOUR-LANING

GWP 280-99-00

CITY OF THOROLD, ONTARIO

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Figure 1 - Abutment on Compacted Fill Showing Granular A Core

Appendix A – Results of Slope Stability Analyses

FOUNDATION DESIGN REPORT

for
Merritt Road Underpass
Site No. 34-460
Highway 406 Four-Laning
G.W.P. 280-99-00
City of Thorold, Ontario

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding the design and construction of the foundations and approach embankments for the proposed new Merritt Road Underpass at Highway 406 in the City of Thorold, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

Highway 406 is being twinned at the new structure location. The proposed underpass will carry the realigned Merritt Road traffic lanes over the proposed Highway 406 northbound and southbound lanes at approximate Sta. 15+853 and 15+830, respectively (new Highway 406 chainages).

The proposed underpass will be a two span structure with a total length of approximately 72 m between abutments based on Drawing S7303-301-001 General Arrangement (GA) drawing dated January 2009 prepared by MRC.

Earth stockpiles about 5.7 and 5.5 m high were placed on the alignments of the east and west approach embankments prior to 2001. The limits of the east and west stockpiles are outlined in the Foundation Drawings MR-1 and MR-2 appended to the Foundation Investigation Report. The limits and height of the stockpiles should be checked for detail design. These fills were intended to be used as advance fills for the approach embankments to the structure. The eastern stockpile was placed to the approximate east abutment location. The western stockpile is located partially within the proposed Merritt Road alignment and about 40 m away from the proposed west abutment. The existing N-W off-ramp from Highway 406 crosses the location of the future approach embankment for the proposed Merritt Road underpass structure.



It is estimated that up to 2.5 m of additional fill is typically required over the existing advance east embankment fill and 3.0 to 4.0 m of fill is required over the existing west fill stockpile level to accommodate the proposed road grade. Between the west stockpile limit and the proposed west bridge abutment about 8 m of fill will be required.

In summary, the soil cover included 0.3 to 1.2 m thick fill units and the two fill stockpiles placed to the east and west of the proposed structure site. These fills covered continuous cohesive typically stiff to very stiff silty clay unit of relatively uniform thickness (up to 9.9 m), which in turn overlaid a 0.6 to 2.2 m thick very loose to compact silt layer. The upper 2.7 to 4.0 m thick zone of the silty clay exhibits a very stiff consistency at the structure location. Below the silt, the stratigraphy included 8.4 to 16.7 m thick layers of firm to hard cohesive clayey silt/clayey silt till which were underlain by a layer of 8.0 to 16.1 m thick compact to very dense cohesionless silt till/silt and sand till locally interbedded with a 5.2 m thick stiff silty clay layer (west abutment borehole 104). The silty clay deposits contain scattered layers of wet silt and silty sand which lowered some of the standard penetration test N values. These N values typically ranged from 0 to 3 blows and were considered not to be representative of the actual soil consistency. Dolostone bedrock of low to medium strength was encountered below the native soils at depths of 36.1 to 37.4 m.

Based on the encountered subsurface conditions and the height of the embankments over the existing ground surface, the bridge may be founded on lightly loaded, maximum 2.0 m wide spread footings placed on the upper zone of the native silty clay or structural fill. Deep foundations using steel H-piles driven to refusal on the very dense sand and silt till or bedrock underlying the site are considered to be feasible.

The loading from the existing fill stockpiles have contributed to consolidate the local cohesive and compressible soils and reduce the estimated 50 mm of consolidation settlement of the clayey soils that would otherwise be expected from an 8 m high approach embankment fill loading. Further comments including pre-loading requirement and recommendations are included in this report.

Re-use of the existing advance fill stockpiled at the approaches for the construction of the new approaches is considered to be feasible. However, the materials will have to be removed and recompacted due to the inferred non-uniform compaction effort during the previous placement.



The "red flag" issues outlined in the preceding paragraphs and the recommended methods of overcoming these issues as noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

All the elevations in this report are reported in meters. A list of the Ontario Provincial Standard documents Referenced in this report is enclosed in Table 1.

2. FOUNDATIONS

2.1 General

It is considered that the fill and organic soils are compressible, therefore are not suitable to support the bridge foundations.

Lightly loaded footings placed on the desiccated upper zone of the native silty clay or on structural fill placed on the same deposit may be used for semi-integral or conventional abutment design. Founding the proposed underpass on steel H-piles driven to practical refusal on the dense to very dense silt till or bedrock is also considered feasible.

Drilled caissons bearing on the glacial till or on the bedrock to support the underpass are not considered to be practical due to the presence of cobbles and boulders in the till, loose to very loose cohesionless silt as well as a groundwater table higher than the anticipated founding levels.

Conventional, semi-integral and integral abutments are considered feasible at this site based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundations engineering perspective, use of integral abutments supported on piles driven to very dense silt till or bedrock is the preferred type of foundation abutments.



All footings and/or pile caps subject to frost action should be provided with 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover.

The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

2.2 Spread Footings on Native Soils

The reference founding levels for spread footings placed on the desiccated upper zone of native silty clay at the south and north abutments and pier are provided on the following table:

FOUNDATION UNIT	SUBGRADE TYPE	BOREHOLE	REFERENCE LEVELS	
			ELEVATIONS	DEPTHS* (m)
West Abutment	Very stiff Silty Clay	2	179.6	0.8
		104	179.0	0.8
Pier	Very stiff Silty Clay	105	178.9	1.2
East Abutment	Very stiff Silty Clay	1	179.1	1.4
		106	179.7	0.7

* Depth from existing ground surface. The minimum 1.2 m foundation frost protection was not considered except at the pier location.



The following geotechnical resistances should be used for the design of the spread footings:

FOUNDATION UNIT	SUBGRADE TYPE	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL RESISTANCE AT SLS (kPa)
West Abutment	Very Stiff Silty Clay	375	250
Pier			
East Abutment			

A maximum footing width of 2.0 m and groundwater at about elevation 175.7 were considered for the ULS computation of footing on native soils. A maximum 2.0 m footing width is recommended to prevent overstressing the underlying firm to stiff silty clay layers. This limitation may make the spread footing alternative impractical for structural design.

Construction of the footings should be performed and monitored in accordance with SP 902S01 to verify the competency of the founding surface.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the founding soils or bedrock. The following parameters should be used for sliding resistance of cast-in-place concrete spread footings on native soils.

PARAMETER	VERY STIFF CLAYEY SILT
Friction angle, degrees	0
Cohesion, kPa	150
Unit weight, kN/m ³	20.0



2.3 Spread Footings on Structural Fill

Construction of the abutment footings on structural fill placed in the approach embankment to support the abutment foundation loads is also feasible. The structural fill should comprise Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

Footings should not be constructed on rockfill. However, rockfill may be placed adjacent to the Granular A core. The recommended bearing resistance for 2.5 m wide footings constructed on structural fill is as follows:

Factored Geotechnical Resistance at ULS, kPa	900
Geotechnical Resistance at SLS, kPa	350

A minimum thickness of structural fill pad of 2.5 m was used for the computation. The structural fill pads should be founded at the same reference levels indicated for the footings on native soils, Section 2.2. The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.70 is recommended for footings placed on granular fill.



2.4 Pile Foundation

It is considered feasible to have the abutments and the center pier founded on steel H-piles. It is anticipated that the piles will reach refusal in the very dense sand and silt till in all reference boreholes except in boreholes 1 and 104 drilled near the southern end of the east and west abutments, respectively where the piles will likely reach the Dolostone bedrock.

The pile foundation design recommendations for conventional and semi-integral abutments are provided on the following paragraphs followed by additional recommendations for integral abutment foundations in Section 2.5.

At the abutments, the piles will find practical refusal on the very dense sand and silt till or on the underlying bedrock based on the subsoil conditions encountered in the boreholes. At the pier, the piles will likely find refusal on the very dense till. The estimated reference founding levels for piles for the abutment and pier locations at the borehole locations are given on the following table:

LOCATION	FOUNDING UNIT	FOUNDING DEPTH (m)	PILE FOUNDING ELEVATION	RELEVANT BOREHOLE
West Abutment	Very dense sand and silt till	34.0	146.4	2
	Bedrock	36.1	143.6	104
Pier	Very dense sand and silt till	34.4	146.0	105
East Abutment	Bedrock	36.7	143.8	1
	Very dense sand and silt till	34.0	146.4	106

The reference depths are taken from the existing ground surface at the borehole locations to the top of the founding stratum. About 1.5 to 2.0 m for pile embedment at refusal on the glacial till deposit and 0.5 m into the Dolostone bedrock should be allowed. For design purposes, the pile lengths should be estimated based on a linear interpolation between the two founding conditions. A contingency allowance for additional pile driving should be included in the contract.



The piles will be driven through native soils containing compressible clayey soils at the abutment locations. The east abutment site has been pre-loaded (as indicated in Section 1) and consequently the additional 1.5 m of fill for the approach embankment will only cause minor consolidation of the underlying native soils. The existing grade at the west abutment will be raised about 8.1 m above the existing grade. Consequently, the development of negative skin friction on the piles should be considered to affect the geotechnical axial resistance for the west abutment piles only.

Based on very dense sandy and silt till at the pile tips at the west abutment and low to medium strength bedrock anticipated at the east abutment, the preliminary factored axial resistance at ultimate limit states (ULS) for a steel HP 310x110 pile is 1,600 kN. The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the maximum 36.7 m pile length required, the design is not expected to be governed by settlement since the required load causing that magnitude of deformation of the pile (1,900 kN) is larger than the ULS factored geotechnical resistance.

The capacity of the HP 310x110 piles for the west abutment should be reduced to allow for negative skin friction of 260 kN if the area is not preloaded and/or surcharged as recommended in Section 4 of this report.

Refer to Section 4 for a discussion and recommendations on the treatment of approach embankment settlements.

Although boulders and cobbles were not encountered at the borehole locations, a NSSP should be prepared to advise the contractor of the potential presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in SP 903S01, if boulders and cobbles are encountered.

The compacted granular fill pad placed as a working platform for construction equipment during installation of the abutment piles should comprise Granular A material to allow installation of the piles without damage. Alternative granular materials such as Granular B Type II could be employed provided the maximum particle size does not exceed 75 mm.



The piles will be driven through very dense soils or set on or into bedrock and should be equipped with driving shoes. SP 903S01 calls for the use of OPSD-3000.100 (Driving Shoe Details for H-piles) or Titus H Bearing Pile Points Standard Model on piles driven to bedrock.

2.5 Integral Abutments on Piles

For the integral abutment design, the H-piles should be also driven to refusal on the very dense sand and silt till or bedrock anticipated at the depths/elevations and axial resistance are indicated in the previous section. The minimum 5.0 m long pile length below the abutment stem which should be incorporated in the design will not be a concern at this site.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

2.6 Lateral Resistance

The soil adjacent to the upper section of the piles is expected to consist of compacted granular material in the approach fills. Typically, cohesive very stiff to firm native clayey soils will be locally present at depth below the embankment fill.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. For integral abutment piles, only the length below the annular space referred to previously should be considered. The assessed lateral resistance for the HP 310x110 pile section noted previously is as follows:

	NATIVE SILTY CLAY /CLAYEY SILT	GRANULAR BACKFILL 'A' OR 'B' TYPE II
Factored Lateral Resistance at ULS, kN	120	120
Lateral Resistance at SLS, kN	35	50



The assessed values of lateral resistance assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended. If greater resistance is required, batter piles should be installed.

To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction, k_s (MN/m³) should be computed using the following equation:

Cohesionless Soils (Terzaghi, 1955)

$$k_s = n_h z/b$$

where n_h = coefficient related to soil density
 = 10.0 MN/m³ for granular backfill
 z = depth, m
 b = pile width, m

The cohesive soil parameters apply to the cohesive silty clay and clayey silt units encountered at the abutment and pier locations.

The coefficient of horizontal subgrade reaction, k_s , for the native clay/clayey silt units should be taken as 28,000 kN/m³.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters/widths. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

PILE SPACING IN DIRECTION OF LOADING d = PILE DIAMETER OR WIDTH	SUBGRADE REACTION REDUCTION FACTOR, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25



2.7 Comparison of Foundation Alternatives

Caisson foundations were not considered to be practical in view of installation difficulties. These would be due to high groundwater table and potential for cobbles and boulders in the glacial till stratum. A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives discussed in the preceding paragraphs is presented below.

Spread Footings on Native Soil

Advantages

- Ease of installation
- Lower cost than deep foundations
- Allows use of semi-integral abutments

Disadvantages

- Footing width limited to 2.0 m to prevent overstressing the underlying stiff soils
- Relatively low geotechnical resistances and limited footing width may render alternative structurally impractical

Footings on Structural Fill

Advantages

- Ease of installation
- Reduced height of abutment stem
- Allows use of semi-integral abutments
- Higher geotechnical resistances than footings on native soil

Disadvantages

- Construction of structural fill pad requires wider area than spread footings
- Requires construction of a fill pad

Piles to Bedrock

Advantages

- Higher bearing resistance values than for spread footings
- Allows use of integral abutments

Disadvantages

- Requires construction of a fill pad ahead of the approach embankment construction for pile driving
- Higher installation cost than spread footings

3. ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The fill to the structure will likely consist of Granular A or Granular B Type II material. The lateral earth pressure, p (kPa), may be computed using the



equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation:

$$p = K (\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)
 γ = unit weight of free-draining granular material, kN/m³
 h = depth below final grade, m
 q = surcharge load, kPa, if present
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
 where ϕ = angle of internal friction of retained soil (35° for Granular B Type II)
 δ = angle of friction between the soil and wall (23.5° for Granular B Type II)

The seismic site coefficient for the conditions at this site was provided in Section 2.1.

Hydrostatic pressures were not included in the equation since free-draining granular material or rockfill will be used as backfill behind the wall. The following parameters are recommended for design:

PARAMETER	GRANULAR A, GRANULAR B TYPE II or TYPE III
Angle of Internal Friction, degrees	35
Unit weight, kN/m ³	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient Passive Earth Pressure, K_p	3.69

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).



The magnitude of the passive resistance and active pressure is dependent on the actual lateral movement of the structure toward and away from the adjacent soil, respectively. We refer to Figure C6.16 (Clause C6.9.1) of the CHBDC for these computations. The backfill should be considered as medium dense sand for this project.

A weeping tile system (SP 405F03 and OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to retaining structures should be carried out in conformance with OPSD-3101.150 for granular backfill at abutments.

Retained soil system (RSS) walls were not shown in the preliminary GA Drawing S7303-301-001, provided by MRC, dated January 2009 and consequently, site specific recommendations are not provided in this report. Additional analysis for RSS walls should be carried out if these structures will be included in the project.

RSS walls, which would be required at the abutments, should tolerate the estimated settlements noted in Section 4 Approach Embankments are accommodated. A high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to the NSSP for Retained Soil Systems dated January 2008.

The bearing resistances recommended for spread footings constructed on the native or structural fill in Sections 2.2 and 2.3 of this report should be employed for design of the RSS wall.

The supplier of the RSS should be responsible for the detail design of the structure (reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance, etc.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to SP 105S10 for additional information in this regard.



4. APPROACH EMBANKMENTS

The existing topsoil and other organic or deleterious soils within the backfill zone of the abutments and retaining walls should be excavated prior to placement of the backfill. We refer to the Preliminary Pavement Design Report for further comments in this regard.

The level of the approach embankments will be typically raised about 2.1 m (east approach; borehole 107) and a maximum of about 8.1 m (west approach; borehole 103) above the existing surface. It is anticipated that the new embankments will be constructed with earth fill. Construction of the embankment fill for the abutments on the existing soils is considered to be feasible.

It is considered that the non-organic fill in the existing east and west stockpiles is of adequate quality for use as earth borrow source for embankment fill. For further details we refer to the Preliminary Pavement Design Report. The existing fills were stockpiled with minimal and non-uniform compaction effort and, locally, excessive thickness of topsoil was left in place (refer to boreholes 102 and 107). The existing fill soils should be removed and recompacted in place upon removal of the peat and topsoil layers previously left under the fill.

The embankments should be constructed in accordance with OPSD-200.010, 202.010, 208.010, OPSS 206, 212, 501 and SP 206S03. The contractor should make allowances for adequate handling of the fill soils to minimize wetting of the soils by unfavourable weather conditions which may render the existing soils impractical to use in the embankments in accordance with OPSS 212. The side slopes of earth fill approach embankments should be inclined no steeper than 2 horizontal to 1 vertical (2H:1V) to provide adequately stable slopes, with a long-term factor of safety higher than 1.5. Where the height of the embankment is greater than 8 m for earth fill, a 2 m wide mid-height berm (bench) will be required according to OPSD-202.010. The embankment fill should be placed in accordance with SP 902S01 and OPSS 501 (Method A). It is also considered that a bench drainage ditch should be provided at mid slope on embankments higher than 6.0 m to minimize surface erosion of the predominantly clayey silt embankment slopes. This measure is also recommended in view of the previously encountered erosion problems experienced by MTO in the Highway 406 corridor, including crossing roadway embankments.



Slope stability analyses were carried out for the west and east embankments for short-term and long-term conditions. Based on the soil data and laboratory tests conducted on selected samples, the table below summarizes the soil parameters applied to the analyses for each of the abutment locations:

LOCATION	MATERIAL DESCRIPTION	UNIT WEIGHT (kN/m ³)	EFFECTIVE COHESION (kPa)	EFFECTIVE FRICTION ANGLE (Degree)
West Embankment	Granular Fill	21.5	0	40
	Earth Fill	20	50	0
		20*	5*	28*
	Clayey Soils	19	50	0
		19*	5*	28*
	Silt	19	0	30
East Embankment	Earth Fill	20	50	0
		20*	5*	28*
	Topsoil	13	30	0
	Silty Clay	19	50	0
		19*	5*	28*
	Silt	19	0	30

Notes: * - Refers to soil parameters used for long-term slope stability analyses.

The table below summarized the results of slope stability analyses carried out under for the surcharged (short-term) and final embankment (long-term) scenarios. A surcharge is not considered to be required at the east embankment. The analysis was carried out using the Slope/W software purchased from Geo-Slope International Inc. and the graphs are attached in Appendix A.

LOCATION	SHORT-TERM / LONG-TERM DURATION	FACTOR OF SAFETY (FOS)
West Embankment (Slope 2H : 1V)	Short-term with Surcharge	1.3
	Long-term	1.7
East Embankment (Slope 2H : 1V)	Short-term	1.8
	Long-term	1.9



The factors of safety (FOS) values of 1.3 and 1.8 for the short-term and 1.7 and 1.9 for the long-term conditions at the west and east abutments are considered to be adequate for slope stability considerations.

It is considered that the approach embankments constructed in accordance with the foregoing recommendations will be stable.

Settlement of the road surface during and following completion of construction will result from the consolidation of the existing native soils below the embankment fill and "self-weight" consolidation of the embankment fill.

The estimated magnitude of settlement of new or recompacted earth fill is placed as recommended is in the order of 40 mm near the abutments.

Based on the consolidation test results, the native soils are pre-consolidated and the new fills will cause only limited settlements. The consolidation settlement of the underlying soil is expected to be in the order of 20 mm (east approach) to 60 mm (west approach). Hence, the total consolidation settlement is estimated to be about 60 mm at the east approach and about 100 mm at the west approach. The estimated settlement of cohesive soils at the west abutment is likely to take up to 18 months to occur to 80% completion. The settlement at the east abutment is expected to be 80% complete within six to nine months following placement of the new fill.

In view of the timing of this project, the embankment settlements should be accelerated to allow for the timely driving of the piles and reduce the post-construction settlement of the approach embankments. As an alternative to the 18 month pre-loading, the west approach embankment fill should be placed and the area surcharged with 2 m of granular material. The surcharge should remain in place for a period of about 12 months prior to driving the abutments piles. This surcharge period would reduce the post-construction settlements to about 20 to 30 mm and eliminate or reduce the negative skin friction on the abutment piles. In addition to pre-loading the east abutment or surcharging the west abutment, an advance contract to provide fill placement ahead of the construction season of the structure should be considered to minimize construction time of the structure.



It is considered that earth fill utilizing local native soils will be susceptible to surface erosion, in view of the silty nature of soils. Earth fill slopes should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation. Refer to OPSS 572 for time constraints and type of seed and mulch required. Local areas of concentrated surface water flow should be protected with additional measures, such as rip-rap (OPSS 511).

5. EXCAVATION AND GROUNDWATER CONTROL

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations. For this purpose, the upper fill and cohesive firm to stiff silty clay soils encountered in the boreholes are considered to be Type 3 soils according to OHSA (Ontario Regulation 213/91) criteria. The very stiff to hard silty clay soils are considered Type 2 soils. Any cobbles or boulders exposed on the excavation slope faces must be removed.

Groundwater at the site was encountered at levels deeper than about 4.0 m below ground surface (borehole 104), elevation 175.7 and is not expected to affect the design and construction of the structure foundations.

Subject to the groundwater level at the time of construction, it is considered feasible to employ sump pumps to control groundwater seepage into the excavations for pile caps and footings during construction.

Surface water run-off should be diverted away from the excavations to ensure that the foundations are constructed in the dry.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.



6. **CLOSURE**

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., Project Manager with the assistance of Mr. N. Rahman, B.A.Sc. Mr. B.R. Gray, M.Eng., P.Eng., MTO Designated Principal Contact carried out an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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Merritt Road Underpass, Site No. 34-460
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W.P. 288-99-00, Index No.: 130FDR
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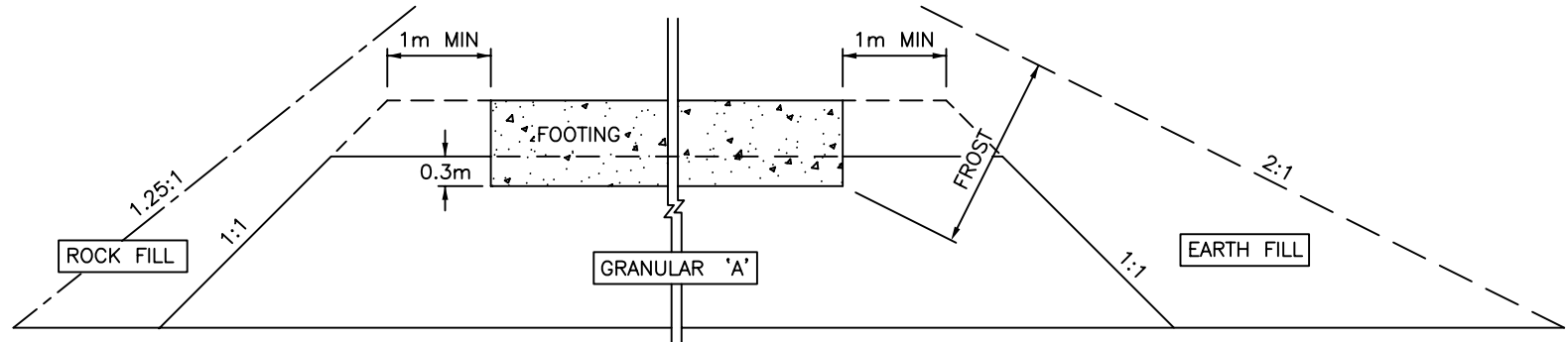
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 206	Construction Specification for Grading
OPSS 212	Construction Specification for Borrow
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 571	Construction Specification for Sodding
OPSS 572	Construction Specification for Seed and Cover
SP 105S10	Construction Specification for Compaction
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 902S01	Excavation and Backfilling of Structures
SP 903S01	Construction Specification for Piling
OPSD-200.010	Earth/Shale Grading - Undivided Rural
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD-208.010	Benching of Earth Slopes
OPSD-3000.100	Foundation Piles Steel H-Pile Driving Shoe
OPSD-3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail
NSSP	Retained Soil Systems



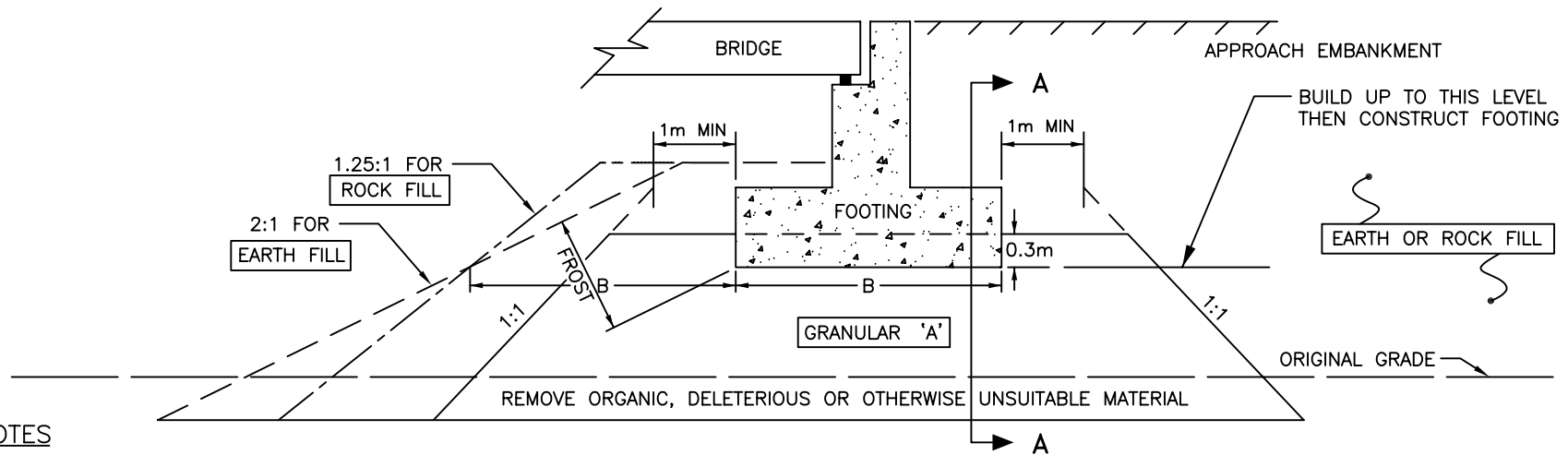
TABLE 2
GRADATION SPECIFICATION FOR SAND FILL IN
PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 µm	#30	80 – 100
425 µm	#40	40 – 80
250 µm	#60	5 – 25
150 µm	#100	0 – 6



CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



APPENDIX A

Results of Slope Stability Analysis

RESULTS OF SLOPE STABILITY ANALYSES

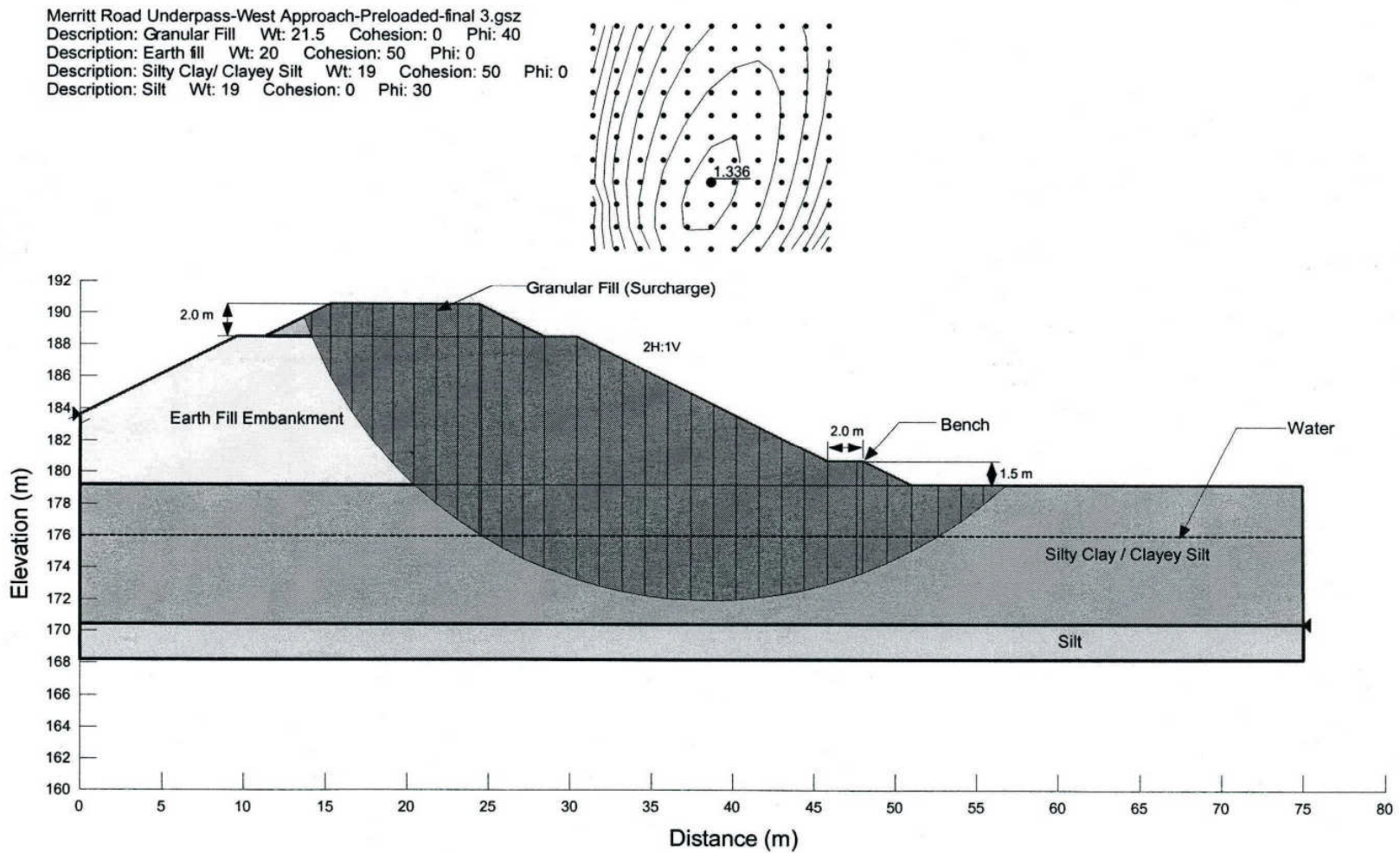


FIGURE 1

RESULTS OF SLOPE STABILITY ANALYSES

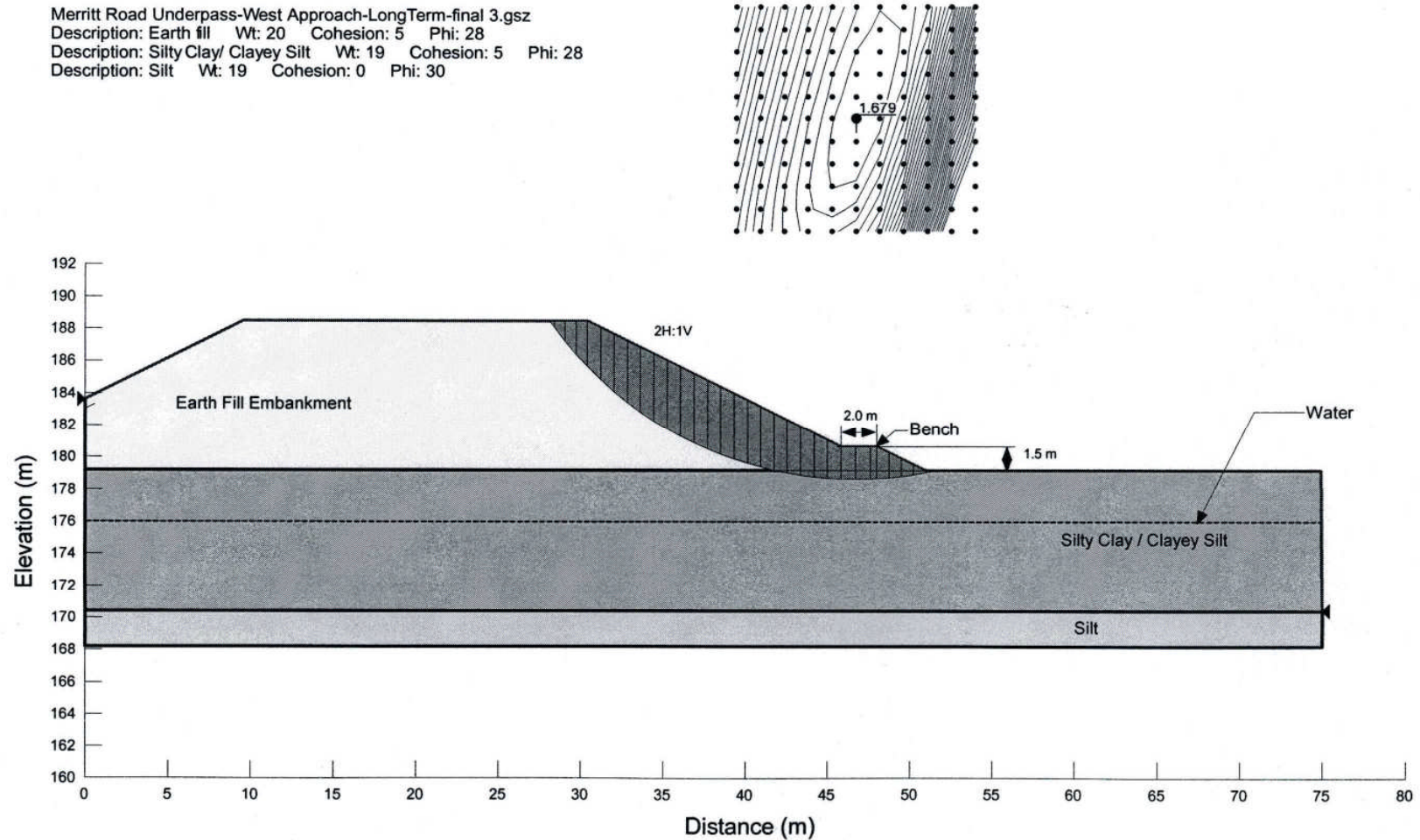


FIGURE 2

RESULTS OF SLOPE STABILITY ANALYSES

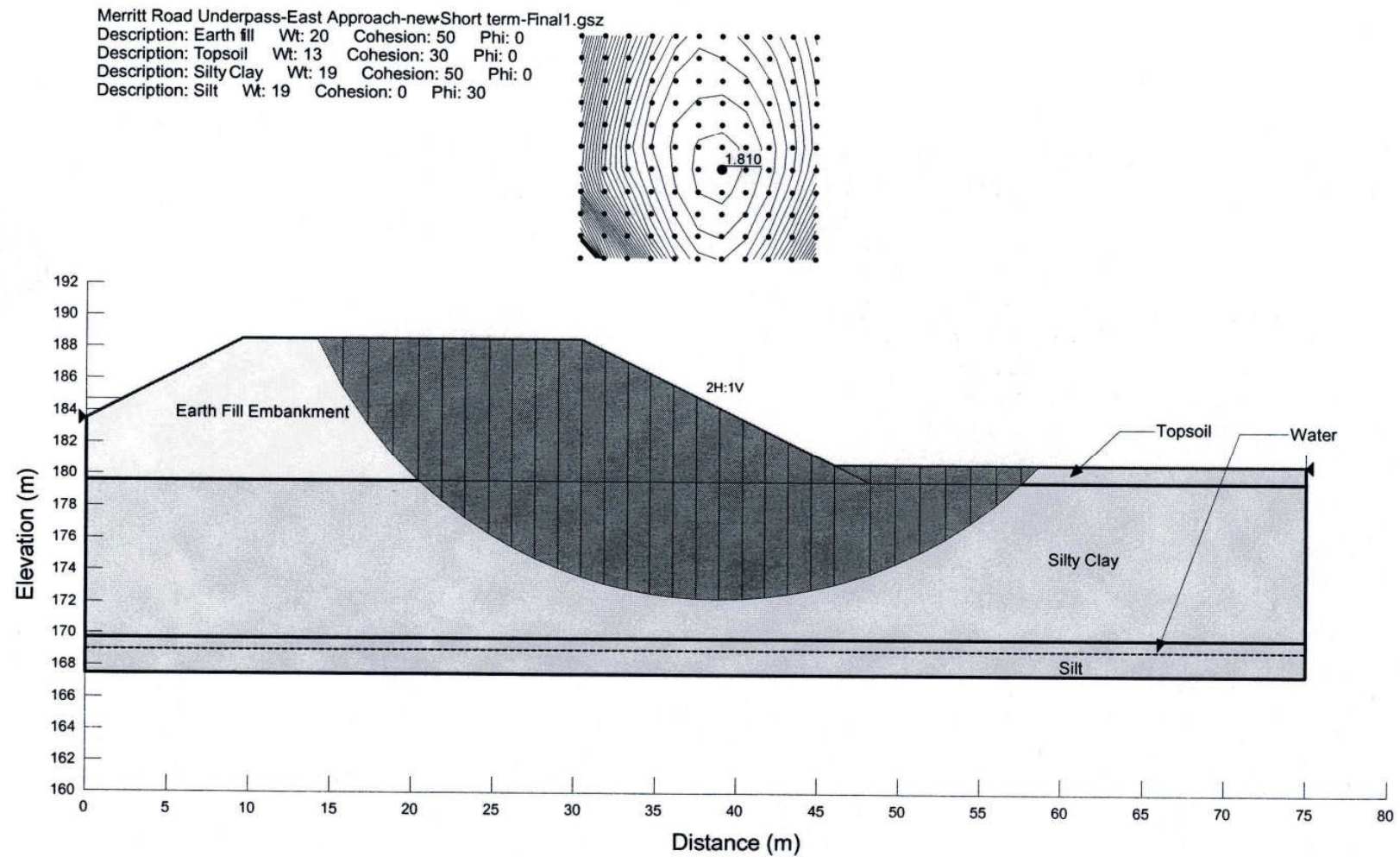


FIGURE 3

RESULTS OF SLOPE STABILITY ANALYSES

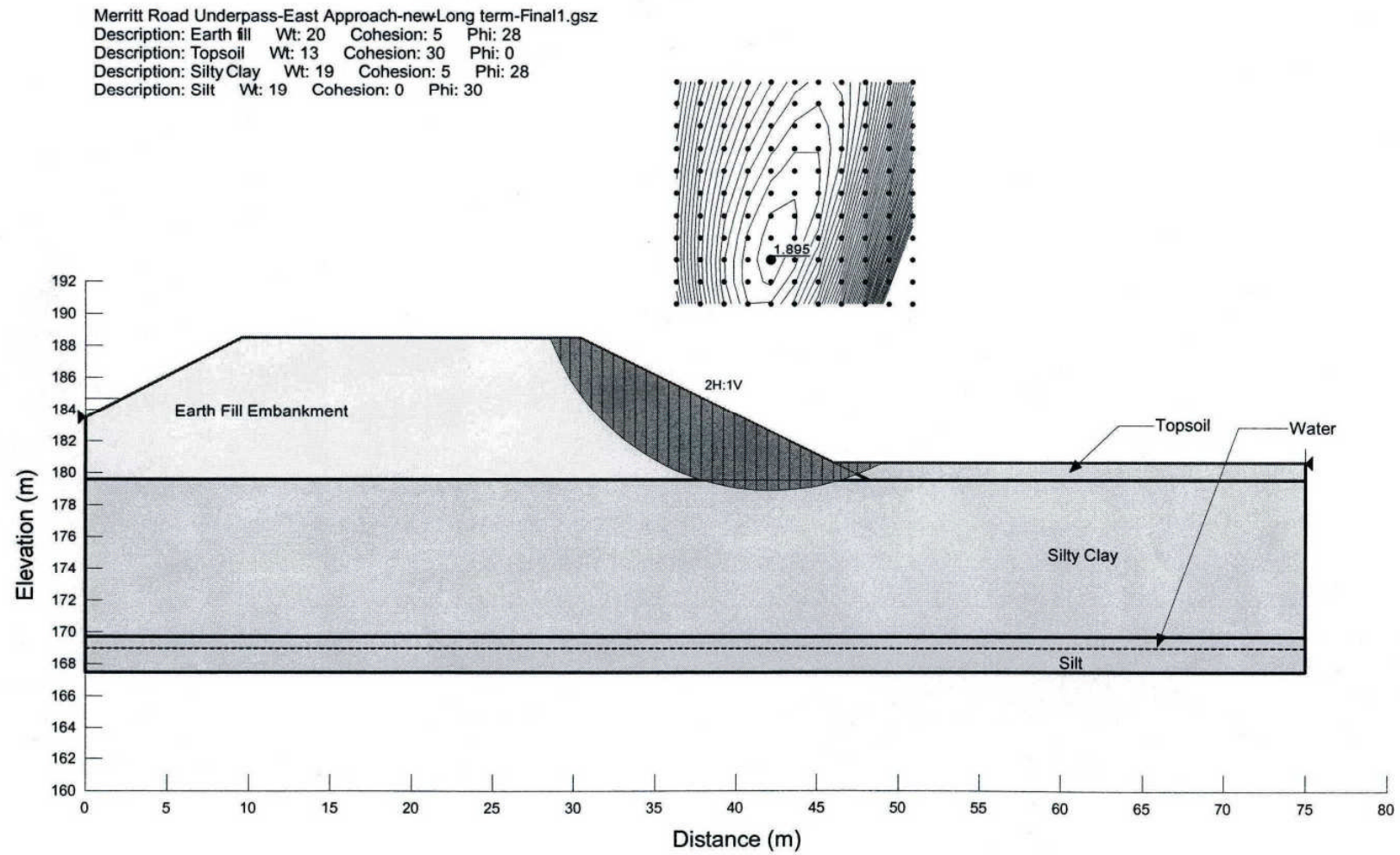


FIGURE 4