



FOUNDATION DESIGN REPORT

for

HIGHWAY 637 UNDERPASS

SITE NO. 46-511

HIGHWAY 69 FOUR-LANING

W.P. 5265-05-01

DISTRICT 54, SUDBURY

***PHASE 2: STA. 15+180 TO 22+346.5, TOWNSHIP OF SERVOS
STA. 10+000 TO 11+300, TOWNSHIP OF BURWASH***

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Table 1 – List of Ontario Provincial Standard Documents referenced in report

Table 2 – Gradation Specification for Sand Fill in Pre-augered Holes at Integral Abutments

Figure 1 - Abutment on Compacted Fill Showing Granular 'A' Core

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WP 5265-05-01
District 54, Sudbury

*Phase 2: Sta. 15+180 to 22+346.5, Township of Servos
Sta. 10+000 to 11+300, Township of Burwash*

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and the approach embankments for the proposed Highway 637 Underpass at the realigned Highway 69 about 41 km south of Sudbury, Ontario. The investigation was conducted for AECOM on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed underpass consists of a 76 m long, two-span structure. The approach embankments to the west and east abutment will be raised about 2.0 to 3.0 and 8.0 m above existing grades, respectively.

In summary, the subsurface soil stratigraphy revealed in the boreholes generally comprised a surficial topsoil unit overlying an upper layer of cohesionless of gravelly sand/silty sand underlain by cohesive clayey silt/silty clay/clay deposits which cover localized lower layers of cohesionless sand/sandy gravelly sand at the east abutment. In the supplementary auger probes APE5 to APE10 drilled at east abutment, the depth of the soil cover varies from 2.1 to 4.7 m. The soil cover generally consists of surficial 200 to 300 mm thick peat overlying a 1.9 to 4.4 m thick layer of silty clay with a localized 0.7 m thick sand layer mantling probable bedrock. Scattered layers of cobbles and boulders are encountered within the silty clay deposit. The relative density/consistency of the soil cover is variable and typically in the loose to compact/very soft to very stiff range. Bedrock outcrops at the west abutment (borehole APW2), pier (boreholes E11, E12, APP1 to APP3) and east abutment location (boreholes APE2 and APE4).



The bridge site abuts the western limit of a swamp crossing which extend easterly from Sta. 10+040 (Highway 637 chainage) to beyond approach embankment limit. The recommended swamp treatment for the embankment construction in the swamp crossing was full excavation of soft soils according to OPSD 203.020.

The bedrock surface in the boreholes drilled for the west abutment and pier of the proposed Highway 637 was found at 3.2 and 3.1 m below the proposed bridge deck grade respectively.

The level of the bedrock surface in the boreholes drilled for the east abutment varied from 7.3 to 13.8 m below the bridge deck grade.

About 7.0 m rock excavation will be required at and between the west abutment and pier. The thickness of the bedrock excavation will taper off to 0.0 m between the pier and the east abutment where the bedrock is typically covered by up to 5.7 m of soil.

The underpass may be founded on spread footings placed on the bedrock underlying the site. Spread footings placed on structural fill at the east abutment are also feasible although excavation of up to 5.7 m of soil and groundwater control would be required.

Alternatively, deep foundations may be used for the abutment foundations provided that the rock is excavated at the west abutment to allow for an adequate pile length and the piles are driven to bedrock at the east abutment. The installation of driven piles for the east abutment will have to consider the steeply sloping bedrock surface which was measured in the boreholes to be 15 to 44° from west to east and 3 to 28° from north to south with localized steeper slopes up to 49° across the abutment area. The boulders encountered within the east abutment area should be excavated for the installation of piles for the alternative integral abutment design.

The presence of numerous boulders within the native soils at the east abutment indicates that the installation of drilled cast-in-place concrete caissons will not be practical.



The construction of the 2 to 3 m thick west approach embankment on the encountered bedrock outcrop is considered to be straightforward. For the east approach embankment, the existing clayey soils should be excavated and the embankment fill should be surcharged to mitigate the settlements behind the abutment as further discussed in this report. This treatment will match the recommended treatment for the adjacent swamp crossing.

The "red flag" issues outlined in the preceding paragraphs and the recommended methods of overcoming these issues 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 elevations in this report are expressed in metres. A list of the Ontario Provincial Standard documents referenced in this report is enclosed in Table 1.

2. FOUNDATIONS

2.1 General

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 foundation engineering perspective, use of semi-integral abutments supported on footings placed on excavated bedrock at the west abutment and footings placed on structural fill or driven piles at the east abutment is the preferred type of foundation abutments.

All footings and/or pile caps subject to frost action should be provided with 2.0 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. Footings bearing directly on bedrock do not require protection from frost.



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).

2.2 Shallow Foundations

It is considered that the loose sandy silt/gravelly sand units and very soft to stiff silty clay/clayey silt/clay are compressible, therefore are not suitable to support the bridge foundations. The unit of gravelly sand, cobbles and boulders encountered at depth in the east abutment boreholes E19 and APE3 may contain loose zones and is also not considered an adequate founding subsoil.

As indicated in the previous Section 1 of this report, it is considered feasible to support the foundations of the proposed west abutment and centre pier on spread footings constructed on bedrock. To construct the east abutment on spread footings bearing on bedrock, a stepped footing design should be used or mass concrete should be installed to provide a level subgrade, in view of the encountered steep bedrock slope and grade variation at the east abutment.

2.2.1 Footings on Bedrock

The reference founding levels for spread footings placed directly on the bedrock at the west and east abutments and pier are provided on the following table:

FOUNDATION UNIT	SUBGRADE TYPE	REFERENCE ELEVATIONS	DEPTHS* (m)
West Abutment	Bedrock	234.5 to 236.2	0.0 – 1.7
Pier	Bedrock	233.7 to 235.9	0.0 – 0.7
East Abutment	Bedrock	225.6 to 231.9	0.0 – 5.7

* Depth from existing ground surface. The minimum 2.0 m frost protection was not considered for footings placed directly on bedrock.

The founding surfaces of sloping or uneven bedrock should be made level or stepped for the placement of the footings, where required. Alternatively, mass concrete should be placed on the uneven bedrock, as further recommended in this section of this report.



The outside edge of the footings perched on the top of the rock cut for the west abutment should be set back at least 1.0 m from a plane inclined upwards and away from the cut at 2V:1H from the toe of the road cut.

A factored bearing resistance at ULS of 10,000 kPa should be used for the design of the spread footings considering the high strength bedrock subgrade. The geotechnical resistance at SLS is not applicable for footings founded on bedrock, since the bedrock is considered to be non-yielding. The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The design west abutment foundation is at approximately elevation 236.0, therefore the existing bedrock is within 0.5 m of the proposed footing subgrade. At the pier, the bedrock will be excavated below the road grade elevation 230.0 and the pier footing should be founded at this level or lower on the exposed bedrock surface. For the east abutment, where the bedrock surface slopes down easterly with a maximum relief of about 6 m across the foundation area, the footing subgrade should be made level using mass concrete or structural fill placed over mass concrete where required to bring the grade over the bedrock above the water table, as discussed in Section 2.2.2.

Mass concrete could be placed to provide a level founding surface for the footings and/or to also raise the subgrade to the design founding level of the footings. The need to expand the plan area at the base of the mass concrete to provide for stress distribution (2V:1H), place reinforcing steel in the mass concrete and/or use high strength concrete to prevent overstressing of the mass concrete will be dictated by the actual thickness of the mass concrete and structural design considerations. Tremie concrete should be used below the water table.

Subject to these comments, the bearing resistance provided for footings bearing on bedrock is considered to be appropriate for mass concrete with an unconfined compressive strength of at least 30 MPa. If the actual bearing pressure is less than 10,000 kPa, the compressive strength of the concrete could be reduced in direct linear proportion to the actual bearing pressure (minimum value of 25 MPa).



Comments concerning excavation of the bedrock, if required to found the footings on sound bedrock at a level lower than indicated previously, are provided in section 6 of this report.

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 bedrock.

An unfactored friction factor of 0.7 is considered to be suitable at this site on the “rough bedrock surfaces” (asperity height of at least 25 mm). The factored horizontal resistance at ULS of the bedrock is considered to be 5,000 kPa.

The lateral resistance of footings founded on bedrock could be increased, if required, by installing shear keys, sockets or anchors into the bedrock (SP 999S26). The increased lateral resistance will be provided by the shear strength of the steel dowels, the horizontal resistance of the bedrock and the horizontal component of tensile forces developed in any inclined anchors. A greater frictional resistance between the footing and rock may be achieved if the anchors are prestressed to increase the vertical pressure.

If dowels are employed, a NSSP should be included in the tender documents to provide specific direction for the contractor during installation and testing of the dowels. Fractured rock should be removed from these areas. A NSSP should also be prepared for the shear keys.

Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 (CHBDC). If anchors are installed, a factored bond stress at the rock/grout interface of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) is recommended for design. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length.



2.2.2 Footings on Structural Fill

Structural fill for the west abutment footings is not required because the spread footing subgrade at this location is within 0.5 m of the bedrock surface level.

Structural fill placed in the east approach embankment could be employed to support the east abutment footings at the design elevation 232.8. . The structural fill should comprise Granular A material placed in maximum 200 mm thick layers, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

Since the majority of the existing native soil is a wet to saturated swampy deposit, the native soils should be totally removed to construct the structural fill on the underlying bedrock surface. The engineered fill should extend out to a plane inclined downwards at 45° to the horizontal originating at least 1 m away from the top of the footing. This scheme is illustrated in the appended Figure 1. The extent of the fill should be established by a site specific survey prior to placement of the fill.

It is recommended that the structural fill should not be placed below the water table. To this end, it is recommended that mass concrete (tremie type concrete if placed below water) be placed to a level of about elevation 228.8, which is the level encountered during the investigation. The mass concrete would be required at the location of boreholes E15 to E17 and E19 and auger probes APE3 and APE5 to APE10 advanced in the eastern and southern areas of the abutment footing footprint. The average thickness of the required mass concrete would be about 1.5 m with a maximum thickness of 3.2 m at the location of borehole E19.

The thickness of the structural fill placed as recommended above is about 4.0 m over the area covered with mass concrete becoming thinner westerly to about 0.9 m over the bedrock at the location of auger probe APE2.

Footings should not be constructed on rock fill. However, rock fill may be placed adjacent to the Granular 'A' core shown in Figure 1.



The recommended bearing resistance for a minimum 2.0 m wide footing constructed on structural fill (bearing resistance independent of fill thickness at this location because the engineered fill should be placed directly on the bedrock or mass concrete) is as follows:

Factored Bearing Resistance at ULS	900 kPa
Bearing Resistance at SLS	350 kPa

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 2.0 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 0.7 is recommended for footings on the granular structural fill.

2.3 Deep Foundations

2.3.1 Pile Foundations

It is considered feasible to have the east abutment founded on piles driven to bedrock. As mentioned in Section 1 of this report, the bedrock slopes down easterly across the foundation area at relatively steep angles of 15 to 44° southerly at flatter angles of 3 to 28° with areas of steeper angles up to 49°. The bedrock is locally overlain by cobbles and boulders at the southern section of the footing (borehole E19).

To facilitate the installation of the piles for the east abutment the bedrock should be exposed in the area of the abutment foundation. This activity should be carried out in conjunction with the construction of the embankment through the adjacent swamp crossing located to the east of the structure. Where piles are to be driven in shallow bedrock areas and the piles need to be extended to a minimum of length required for structural adequacy, the excavation should be



deepened by excavating into the bedrock. It is noted that this preparation for pile driving will be minimised for the alternative integral abutment design and construction because only a single row of piles is required. Where the piles will be installed, the rock excavation (if required) should be backfilled with granular material such as Granular B Type II with a maximum nominal particle size of 75 mm to provide lateral support for the piles. The granular material should be adequately compacted according to OPSS 501.

To found the west abutment on piles, a trench should be excavated/blasted into the existing rock outcrop where the pile length will not be structurally adequate. This bedrock removal will be specifically required to ensure that the free pile lengths will not be less than 5 m below the abutment stem to allow for an integral abutment design for the underpass.

The general pile foundation design recommendations are provided on the following paragraphs followed by additional recommendations for integral abutment foundations.

The estimated range of reference founding levels for piles for the east abutment and the maximum levels of the rock excavation at the west abutment where piles should be driven to refusal on bedrock are provided on the following table:

LOCATION	GROUND SURFACE ELEVATION (m)	PROPOSED UNDERSIDE OF THE ABUTMENT	DEPTH TO ROCK(*) (m)	PILE FOUNDING ELEVATION (m) (**)	RELEVANT BOREHOLES
West Abutment	235.2 to 236.9	235.8	Rock cut to 7.2	Maximum 229.0	E2 to E7, APW1 to APW3
East Abutment	231.3 to 231.9	231.9	Rock cut to 2.9	225.6 to 229.0	E14 to E19 and APE1 to APE4

Notes: (*) Depth is measured from existing ground surface. A 1.0 m variation of the average depth to rock should be allowed for construction estimation purposes.

(**) The estimated highest level for pile foundations is elev. 229.0 to allow for installation of minimum 5 m long piles below the abutment stem.

The actual founding levels of the pile tips are dependent on the structural design of the abutment stem.



The recommended factored axial resistance at ultimate limit states (ULS) for the pile sections listed below is considered to be appropriate.

FACTORED AXIAL RESISTANCE AT ULS, kN

HP 310 x 110	2000
HP 360 x 108	2000

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be non-yielding and the relatively short pile length required, the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile and/or bedrock are much larger than the ULS factored resistance.

It is considered that negligible down drag force will develop on the piles due to negative skin friction since the consolidation of the relatively shallow cohesionless granular materials placed under the embankment loading will occur during construction.

Refer to Section 5 for a further discussion and recommendations on the treatment of approach embankment settlements.

The presence of cobbles/boulders was identified within gravelly sand, silty sand and sand materials below topsoil and above bedrock in boreholes E5, E6 and E19, at the west and east abutments. The risk of damage during driving is considered to be high and the boulders should be removed from the intended pile foundation area before driving the piles, as previously recommended in this section. The excavation should be backfilled with granular materials compacted to 95% of the ASTM D-689 (standard Proctor) maximum dry density.

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



The piles will be driven through the granular fill pad placed on the bedrock surface. It is considered that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed. Since the piles will be driven to bedrock, a specific set is not provided.

The piles will set on or into bedrock and should be equipped with "Rock Points" according to SP 903S01. The Titus H Bearing Pile Points, Rock Injector Model should be used at the east abutment since the slope of the bedrock revealed at the east borehole locations is variable and locally exceeds 30°. The piles for the west abutment (if required), which would be installed in a rock trench, should use the Titus H Bearing Pile Points Standard Model.

2.3.2 Integral Abutments on Piles

The use of integral abutments at the west abutment will require the installation of piles in a trench excavated into the rock. To provide a minimum of 5 m free pile length below the abutment stem, it is estimated that the bottom of the rock trench will be excavated to about elevation 229.0.

Under the east abutment, the bedrock level along the location of the piles (boreholes E16 and auger probes APE1 and APE3) ranges from elevations 227.2 and 230.7. The rock higher than the maximum elevation determined by structural design should be excavated to allow for a 5 m minimum free pile length.

The depth of excavation of a trench into rock to accommodate the use of integral abutments will be dictated by structural design details. The excavation width should be at least 1 m plus width of the pile or as required to achieve adequate compaction of the backfill material; side slopes in the soil cover and in the rock should be excavated as indicated in section 5 of this report. The excavation should be backfilled with Granular A, following the procedures outlined in the section titled "Approach Embankments". Further comments concerning bedrock excavation are provided in the section titled "Excavation and Groundwater Control".



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.

2.3.3 Caisson Foundations

The use of caissons to support foundations on rock at the east abutment is not practical due to the presence of numerous boulders in the soil cover and sloping bedrock that would cause installation difficulties.

2.3.4 Lateral Resistance

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile below the annular space referred to previously for integral abutment design. The assessed horizontal passive resistance values for the pile sections noted previously driven through the granular backfill materials are as follows:

Pile Section	HP 310	HP 360
Factored Lateral Resistance at ULS, kN	120	170
Lateral Resistance at SLS, kN	50	70

The assessed values of lateral resistance assume that the piles are driven through compacted granular materials placed as recommended previously. It is considered that the native soils will be removed to facilitate the installation of driven piles, as discussed in Section 2.3.1 of this report. 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/m^3) 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/m^3 for granular backfill
 z = depth, m
 b = pile width, m

The cohesionless soil parameter is applicable to all granular fill. It is considered that the native cohesionless and cohesive soils would be removed for the installation of driven pile foundations.

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

2.4 Comparison of Foundation Alternatives

Footings on native soils and caisson foundations were not considered to be feasible due to weak soils, high water level and the presence of numerous boulders above the bedrock. A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives (Footings on Bedrock, Footings on Structural Fill and Piles to Bedrock) discussed in the preceding sections is presented below.



Footings on Bedrock

Advantages

- Ease of installation
- Lower cost than deep foundations
- May be used with semi-integral abutments

Disadvantages

- Requires removal of all boulders from foundation footprint (east abutment)
- Possible need to remove fractured rock
- Possible need for mass concrete and/or bedrock excavation to provide level founding surfaces

Footings on Structural Fill

Advantages

- Ease of installation
- Lower cost than deep foundations
- Reduced height of abutment
- Fractured rock need not be excavated
- May be used with semi-integral abutments

Disadvantages

- Construction of structural fill pad requires wider area than footings on bedrock
- Requires construction of a fill pad
- Requires placement of mass concrete to provide subgrade above water table
-
- Requires removal of all existing soils at east abutment
- Potentially requiring tremie concrete for under water construction

Piles to Bedrock

Advantages

- High bearing resistance
- Used for integral abutments, semi-integral abutments or conventional abutments

Disadvantages

- Requires construction of a fill pad ahead of the approach embankment construction
- Higher installation cost than spread footings
- Requires excavation of trench in rock where free pile length is less than 5.0 m (for integral abutment design)
- Requires rock points for installation on steeply sloping bedrock at east abutment



Based on the foregoing the preferred alternative from the foundation engineering perspective is the spread footing foundation on bedrock for the west abutment and pier and using structural fill for the east abutment.

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 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/m^3
 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 according to OPSD 3121.150. The following parameters are recommended for design:

PARAMETER	GRANULAR A, GRANULAR B TYPE II or TYPE III	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m^3	22.8	18.0
Coefficient of Active Earth Pressure, K_a	0.27	0.20
Coefficient of Earth Pressure At Rest, K_o	0.43	0.33
Coefficient Passive Earth Pressure, K_p	3.69	5.04



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, OPSD 3102.100 and 3190.100) or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. Weeping tiles should be installed where there is a potential for flooding conditions. 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 and 3101.200 for granular or rock backfill at abutments.

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. RETAINED SOIL SYSTEM WALLS

A retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 5 Approach Embankments are accommodated where applicable. A



high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to SP 599S22.

The topsoil encountered at west abutment should be removed from the RSS wall footprint together with all soil containing organic materials. The RSS footing should be placed on the rock outcrop at levels ranging from elevations 234.5 to 236.2.

The RSS walls at the east abutment will be placed on the bedrock or on a structural fill layer over the bedrock at levels about elevation 232.0. The structural fill should be compacted to 100% of the standard Proctor maximum dry density. The bearing resistances recommended previously for spread footings founded on bedrock or structural fill constructed on bedrock are considered to be suitable for the RSS wall footings. The anticipated width of the RSS footing is 600 mm.

A transition treatment between the RSS wall fill and the rockfill should be provided to minimize the movement of granular fill into the voids of the rockfill. For this purpose, the surface of the rockfill at the interface should be chinked or a geotextile should be provided between the two types of fill. The same geotextile used for subdrains (Section 3) is considered adequate.

The earth pressure coefficients provided previously are considered to be appropriate for the RSS wall. The horizontal force at the base of the RSS will be resisted in part by the friction force developed through the granular backfill or along the interface between the granular backfill and the founding soil, subject to site specific design details. An unfactored friction factor of 0.7 is considered to be appropriate for both situations at this site.

The RSS supplier should be responsible for specifying the type of backfill material employed, taking into consideration the engineering properties of the proprietary product, the design life of the structure, the pullout resistance required and drainage requirements. The RSS wall designer should note that the MTO Northeastern Region requires that all fill to the structures comprises OPSS Granular B Type II for rockfill embankments. The RSS wall should be designed to withstand the estimated settlements of the native soils under the embankment loads indicated in the Approach Embankments section.



The supplier of the RSS should also 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.

5. APPROACH EMBANKMENTS

The west approach embankment will be about 2.0 to 3.0 m high and placed over a partially exposed bedrock outcrop. Construction of this approach embankment will be straightforward, requiring the removal of topsoil and any other soil cover before placing the embankment fill.

The east approach embankment will be about 8.0 m high above existing grade level and extend over a known swamp crossing which extends easterly from approximately Sta. 10+040 (Highway 637 chainage) to beyond approach embankment limit. As recommended for the treatment of the swamp crossing, full excavation of the native soft and compressible soils should be carried out under the footprint of the east approach embankment. The excavation should be backfilled with rockfill except at the locations where the piles will be installed or the alternative structural fill will be constructed for the foundation of the east abutment. The maximum total thickness of the rockfill at the east abutment will be about 14.0 m, considering the required subexcavation to the underlying bedrock.

The embankments should be constructed in accordance with OPSD 201.020, 202.010, 208.010 and SP 206S03. The side slopes of approach embankments should be inclined no steeper than 2 horizontal to 1 vertical (2H: 1V) for earth fill and 1.25 H : 1 V for rockfill. Where the height of the embankment is greater than 8 or 10 m for earth fill or rock fill, respectively, a 2 m wide mid-height bench will be required.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. Where slopes are flattened to eliminate the need for a guide rail, a granular infilled drainage gap should be provided. OPSS Granular B Type II or Type III should be used for the drainage gap.



Since the new embankments will be constructed directly on bedrock the platform width should be widened by a minimum of 1 m each side in accordance with the Northeastern Region Engineering Directive (NRE 98-200).

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Settlement of the embankment fill due to consolidation of the underlying bedrock will be negligible.

Settlement of the road surface away from the abutments during and following completion of construction will result from two mechanisms – consolidation of the existing native soils below the embankment fill and “self weight” consolidation of the embankment fill.

The estimated settlement of new 2.0 to 3.0 m thick rock fill at the west approach should be in the order of 10 to 15 mm if placed in accordance with the requirements of SP 902S01 and OPSS 501 (Method A). For the east approach embankment, the total settlement is estimated to be in the 110 mm order of magnitude. About 50% of the settlement or 55 mm will occur during the initial 12 months after installation and the remaining 55 mm settlement will occur over the following 10 years. Similarly to the adjacent swamp treatment, the east approach embankment should be surcharged for a period of 12 months with a 2 m high load above the proposed approach embankment and abutment area. The resulting estimated settlements after the surcharge application are about 70 mm and the long-term settlements will be about 40 mm. Immediately behind the abutment where the backfill will include compacted Granular B Type II material the post-construction settlements are estimated to be in the order of 20 mm.

The estimated fill settlements at the abutments will be about 10 mm at the west and 30 mm at the east side since the granular materials used in these sections will be well compacted.

Earth fill slopes, if utilized, 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.



6. EXCAVATION AND GROUNDWATER CONTROL

Excavation of bedrock is anticipated under the west abutment and pier foundations. Excavation for construction of footings founded on the bedrock at the west abutment and pier will extend through about 0.0 to 1.6 m and 0.0 to 0.8 m of rock, respectively. At the east abutment, 0.0 to 5.7 m thick soil cover will be excavated. Cobbles and boulders should be expected in the excavations.

The gravelly sand, silty sand, cobbles and boulders at the west abutment and pier, and the compact sand and sandy gravel, and stiff to very silty clay/clayey silt/clay soils materials at the east abutment are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. All cohesionless soils below water table are considered Type 4 soils. The higher soil Type number should be considered for planning of the excavation slopes.

A large excavator equipped with a tiger-toothed bucket in conjunction with a jackhammer or hoe ram is the preferred method of excavation to shallow depths in rock scaling. Conventional rock excavation techniques such as blasting (OPSS 120) may also be required. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of the cut and relative depth of excavation into the bedrock. Mass concrete could be employed to level variations in the bedrock surface, as mentioned previously.



It is important that blasting of the rock (SP 299F06) is controlled to prevent fracturing and/or disturbance of the bedrock surface on which footings will be founded. Any overblasting/overexcavation should be made the sole responsibility of the contractor and all loosened rock resulting from blasting operations is to be removed by mechanical means.

Near vertical sidewalls may be utilized in excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workers.

Groundwater was not encountered at the west abutment and pier. At the east abutment and at the east approach embankment areas groundwater was encountered at 1.5 to 2.7 m depths below ground surface, elevations 228.6 to 229.9. 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 construction of foundations. A temporary earth fill dam may be required adjacent to the east abutment if excavation in the dry are required at this location.

Surface water run-off should be diverted away from excavation 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.



7. CLOSURE

The report was prepared by Mr. I. Sadoun, MSc, P. Eng. and Mr. C. M. P. Nascimento, P.Eng. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact, carried out an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

**NOTE: Hard copies signed
and stamped**

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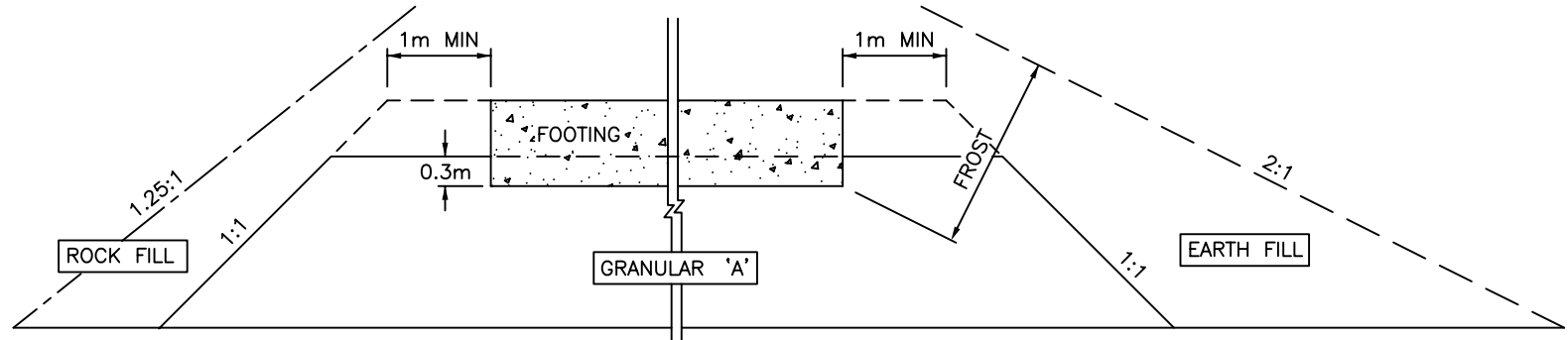
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	
OPSS 120	General Specification for the Use of Explosives
OPSS 501	Construction Specification for Compacting
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 299F06	Rock Excavation (Controlled Blasting)
SP 405F03	Construction Specification for Pipe Subdrains
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)
SP 902S01	Excavation and Backfilling of Structures
SP 903S01	Construction Specification for Piling
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock
OPSD 201.020	Rock Grading-Divided Rural
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment
OPSD 203.020	Embankments Over Swamp – Existing Slope Excavated to 1H:1V
OPSD 208.010	Benching of Earth Slopes
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD 3101.200	Rock Backfill Requirements - Abutments
OPSD 3102.100	Walls Abutment Backfill Drain
OPSD 3121.150	Minimum Granular Backfill Requirements - Walls Retaining
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail
NRE 98-200	Northeastern Region Directive - Platform Widening
NSSP	Dowels Into Concrete
NSSP	Shear Keys



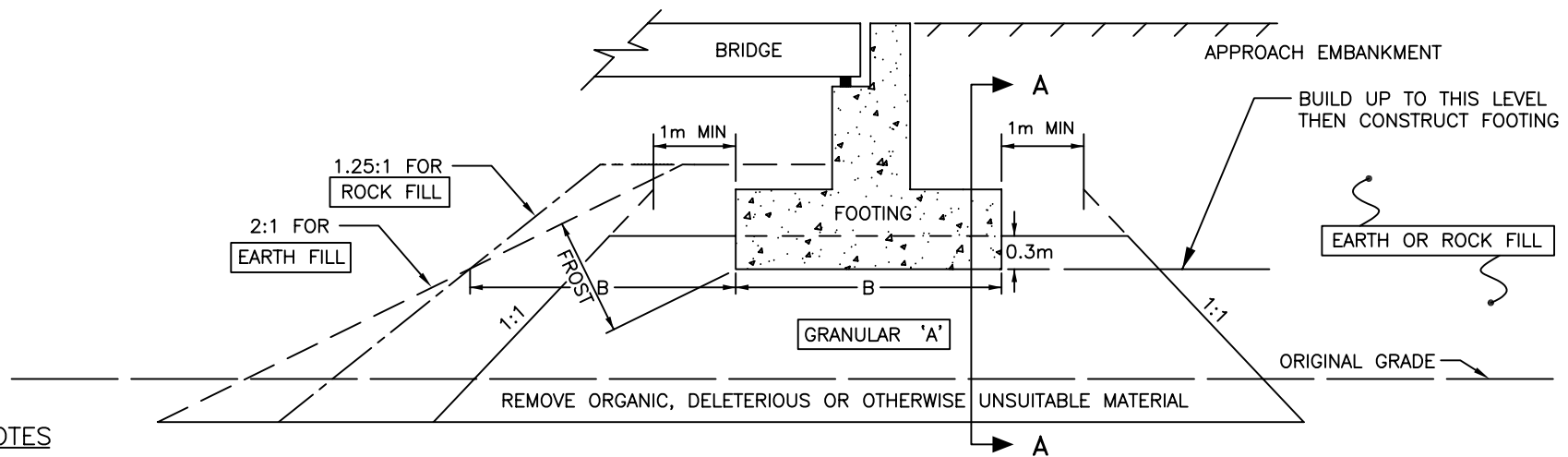
TABLE 2
GRADATION SPECIFICATION FOR SAND FILL IN
PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

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