



**FOUNDATION DESIGN REPORT**

**for**

**PICKEREL RIVER BRIDGE NORTHBOUND  
HIGHWAY 69 FOUR-LANING, SITE NO. 44-429/1  
W.P. 5267-05-01 (PART OF G.W.P. 5378-02-00)  
SUDBURY AREA, ONTARIO**

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for  
Pickerel River Bridge Northbound  
Highway 69 Four-Laning, Site No. 44-429/1  
W.P. 5267-05-01 (Part of G.W.P. 5378-02-00)  
Sudbury Area, Ontario

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**1. INTRODUCTION**

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and approach embankments for the proposed construction of a bridge to carry northbound traffic on the realigned Highway 69 over the Pickerel River, south of Sudbury, Ontario. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The northbound lanes (NBL) bridge is at approximate Station 19+975, new Highway 69 median chainage. The bridge is proposed to be a three span structure with a total length of 138 m and width of 14 m. The middle span will be 61 m long, reduced by the inclination of the north and south pier legs (ref. Drawing 1 'Highway 69 – Pickerel River Bridge' prepared by MRC in October 2009).

The centre section of the bridge deck is envisaged to be placed under the bridge on a barge and hoisted in place using cranes perched on each end of the previously completed bridge deck sections. The erection of the centre section of the bridge deck will likely require that the abutments be anchored to the bedrock behind the bridge to resist the horizontal forces due to the hoisting operations.

The road grade on Highway 69 at the bridge location is planned to be at elevation 192.4 at the south abutment and elevation 196.3 at the north abutment. The approach embankments to the structure are envisaged to be 9 to 13 m high at the south abutment and 1 to 12 m high at the north abutment (interpolated from ground surface elevations and the road grade shown on the MRC drawing referred to above). It is noted that the fill adjacent to the north abutment will be placed along some 15 m section and the bedrock cut beyond this section to construct the approach embankment.



In summary, the subsurface stratigraphy revealed in the boreholes drilled at the site comprised surficial topsoil and/or fill mantling bedrock. Cobbles and boulders were encountered in 7 boreholes. The bedrock surface was contacted at depths of 0.0 to 1.4 m.

The water level in the Pickerel River was at elevation 178.0 in April 2007, with the 100-year high at elevation 179.25.

The depth to and surface elevation of the bedrock identified in the boreholes drilled at this site are summarised in the following table:

Location	Borehole No.	Depth to Rock (m)	Bedrock Elevation
South Approach	N1	0.0	185.5
South Abutment	N2	0.0	183.5
	N3	1.4*	179.5*
	N4	0.4*	181.7*
	N4A	0.0	182.2
	N5	0.1*	181.9*
	N6	0.1*	181.8*
	N7	0.7	180.1
	APS-N1	0.0	185.0
	APS-N2	0.5	180.6
	APS-N3	0.0	184.4
	APS-N4	0.6	180.3
	APS-N5	0.6	180.5
South Pier	N8	0.3*	179.5*
	N9	0.8*	179.6*
	N9A	0.0	180.4
	N10	0.5*	179.1*
	N11	0.5*	179.0*
	N12	0.2	179.3
	N13	0.0	179.6
North Pier	N14	0.0*	180.9*
	N15	0.0*	179.3*
	N15A	0.0	179.1
	N16	0.0*	179.9*
	N17	0.0	180.6
	N18	0.0	181.3
	N19	0.0*	179.1*
	N19A	0.0	180.2



North Abutment	N20	0.0*	184.1*
	N20A	0.0	183.6
	N21	0.0*	184.9*
	N21A	0.0	185.6
	N22	0.0*	187.5*
	N22A	0.0	185.7
	N23	0.0	186.9
	N24	>0.0	<184.9
	N24A	0.0	186.3
	N24B	0.0	184.4
	N25	0.0*	188.1*
	N25A	0.0	187.7
	N25B	0.0	188.6
	APN-N1	0.3	181.0
	APN-N2	>0.0	<181.7
	APN-N3	0.0	181.7
North Approach	N26	0.0	199.5

\* confirmed by rock coring

## 2. FOUNDATIONS

### 2.1 General

#### 2.1.1 South Abutment

The design road grade at the south abutment is near elevation 192.4, about 9 to 13 m above the ground surface.

Use of conventional spread footings founded on either bedrock or structural fill as well as steel H-piles driven to bedrock through the embankment fill are considered to be suitable methods of supporting the south abutment foundation.

Spread footings founded on bedrock or structural fill are considered to be the preferred foundation system due to the shallow depth to bedrock at the south abutment. It is noteworthy that cobbles and boulders present at this location should be removed before placing the footings or fill.



### 2.1.2 South and North Piers

Since the inclined south and north pier legs will exert significant lateral forces on the pier foundations, use of piles or caissons for the piers is not considered to be feasible.

The south and north pier foundations should be supported by spread footings founded on bedrock.

### 2.1.3 North Abutment

The design road grade at the north abutment is near elevation 196.3, about 1 to 12 m above the ground surface.

Spread footings founded on bedrock or structural fill are considered to be the preferred foundation system due to the shallow depth to bedrock at the north abutment. It is noteworthy that boulders present at this location should be removed before placing the footings or fill.

### 2.1.4 Ancillary Design Considerations

The preferred system employed to support the structure foundations, particularly the south abutment, will be dictated by structural design considerations, economic considerations and construction constraints. Due to the design of the bridge, integral abutments are not considered to be feasible at this site and the abutments should be of conventional or semi-integral design.

Continuous monitoring of the partially completed structure should be implemented during the hoisting operations of the central section of the bridge deck. A NSSP should provide maximum review and alarm limits of the horizontal and/or vertical movement by the structural designer.

The seismic coefficient for the conditions at this site is 1.0 (Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06). The zonal acceleration ratio is 0.05. The bridge site is located in Seismic Performance Zone 1.



All footings 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 600 mm of soil cover. Footings bearing directly on bedrock do not require protection from frost.

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding surface. In addition, a rock engineering specialist should be retained to examine the integrity and/or impact on bedrock below the footings should blasting be required near the bridge foundations.

Further comments and recommendations for design of the foundations are provided in the following sections. A summary of the advantages, disadvantages and the preferred foundation type from a foundation engineering perspective is provided in Table 1. The standard specifications referenced in this report are listed in Table 2.

## **2.2 Spread Footings**

### **2.2.1 Footings Constructed on Bedrock**

As discussed in section 2.1, the foundations for the abutments and piers may be constructed as footings on bedrock. The anticipated depths/elevations to bedrock at these locations are tabulated in section 1. The bedrock surface level within the footprints of the foundation elements ranges from elevation 179.5 to 183.5 at the south abutment, from elevation 179.0 to 180.4 at the south pier, from elevation 179.1 to 181.3 at the north pier and from elevation 183.6 to 188.6 at the north abutment. The inferred surface of the bedrock slopes down to the east at inclinations of 7° to 14° at the south abutment and north pier, dips to the south at a maximum angle of 29° at the north abutment and is roughly level at the south pier.

In summary, the bedrock comprises a moderately weathered to unweathered medium to high strength granitic gneiss and is generally classified as poor to excellent quality (RQD of 28 to 100%) with a core recovery in excess of 72%. Very poor quality rock was identified locally at both abutments, especially at the north pier. It is considered that the rock is capable of adequately supporting the foundation loads.



Footings bearing on the medium to high strength bedrock should be designed using a factored geotechnical bearing resistance of 8,000 kPa at ultimate limit states (ULS). The geotechnical resistance at serviceability limit states (SLS) allows for 25 mm compression of the founding medium. Considering the bedrock to be non-yielding, the design will not be governed by settlement criteria since the loading required to produce 25 mm deformation would be much larger than the factored geotechnical resistance at ULS. The geotechnical bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

Mass concrete could be placed to provide a level founding surface for the footings where required. Mass concrete could also be employed to raise the subgrade to the design 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 will be dictated by the actual thickness of the mass concrete and structural design considerations.

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 35 MPa. If the actual bearing pressure is less than 8 MPa, the unconfined compressive strength of the concrete could be reduced in direct linear proportion to the actual bearing stress to a minimum of 25 MPa.

Comments concerning excavation of bedrock to enable construction of the footings are provided in section 5 of the 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 bedrock. An unfactored friction factor of 0.7 is recommended for footings constructed on rough bedrock surfaces (asperity height of at least 25 mm).

The need to install anchors/dowels to resist sliding if the slope of the bedrock surface referred to previously is unfavourably inclined relative to the direction of the force imposed by the foundation loads should be considered by the structural engineer. As indicated in section 1 of this report,





installation of temporary or permanent prestressed anchors at the abutments is likely to be required. Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 of the CHBDC.

The lateral resistance of footings founded on bedrock could be increased by means of a shear key and/or by installing prestressed anchors into the bedrock (SP 999S26). The increased lateral resistance will be provided by the shear strength of steel dowels if used, the horizontal resistance of the bedrock, the horizontal component of tensile forces developed in any inclined anchors and/or a greater frictional resistance between the footing and rock if the anchors are prestressed to increase the vertical pressure. The factored horizontal resistance at ULS of the bedrock is considered to be 5000 kPa.

If dowels into concrete are employed, installation and testing of the dowels should be done in accordance with SP 999S29. Fractured rock should be removed from these areas.

A NSSP should be included in the tender documents for inspection of the footing subgrade by a specialist rock mechanics engineer. Based on the inspection, rock bolts or grouting and/or local scaling will likely be required, especially at the north pier footing.

If prestressed 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 anchors should extend at least 30 bar diameters into sound bedrock and be spaced at a distance of at least four times the diameter of the anchor hole. 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. Design, installation and testing of the anchors subjected to tensile stresses should be conducted in accordance with SP 999S26 and clause 6.10.4 of the CHBDC.

#### **2.2.2 Footings Constructed on Structural Fill**

Construction of the abutment footings on structural fill placed in the approach embankment may be employed to support the foundation loads. The structural fill should comprise OPSS 1010 Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM



D-689 (standard Proctor) maximum dry density, and extend laterally to a line inclined downwards at 45° to the horizontal originating at least 1 m from the top of the footing. This scheme is illustrated in Figure 1, appended.

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

The recommended bearing resistance for a minimum 2.5 m wide footing constructed on a structural fill pad (bearing resistance independent of fill thickness due to the shallow depth to bedrock) 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 A fill.

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

### **2.3 Piles**

Steel H-piles could be used to support the foundation loads at the south abutment. The piles should be driven to refusal on bedrock anticipated at depths of 0.0 to 1.4 m (elevation 179.5 to 183.5). Based on the proposed bridge abutment grades, it is not likely that local bedrock



excavation to provide for the structurally required minimum 3 to 5 m free pile length will be necessary.

The following factored geotechnical axial resistance at ULS for the following sections of steel piles is considered to be appropriate (refer to notes 5 and 6 in Section 3.3.3 of the Pile Driving Notes in the Structural Manual, June 2002):

<b>Pile Section</b>	<b>Factored Geotechnical Axial Resistance at ULS (kN)</b>
HP 310 x 110	2000
HP 360 x 152	2800

The geotechnical resistance at SLS allows for 25 mm compression of the pile and founding medium. Considering the bedrock to be non-yielding and the pile length needed, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm deformation of the pile and bedrock would be larger than the factored geotechnical resistance at ULS.

The approach fill embankments within the limits of the pile foundation should comprise Granular A or Granular B Type II materials with a maximum nominal size of 75 mm to enable driving and minimise the potential for damage during pile installation.

The piles driven at the south abutment to support the foundation loads are envisaged to be less than 10 m long. The piles will be driven through compacted granular fill materials placed for the approach embankment as the soil cover is practically absent. It is considered, based on our experience with pile driving under similar conditions, 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.

The piles will set on or into bedrock and should be equipped with driving shoes, such as the Titus H-bearing pile points indicated in OPSS 903. Since the piles will be driven to bedrock, a specific set is not provided.



The piles should be installed and monitored in accordance with the requirements of OPSS 903. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 2.0 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Resistance to lateral loads may be provided in part by mobilisation of passive resistance along the pile. The recommended lateral resistance is as follows:

	<b>Granular Backfill</b>	
Pile Section	HP 310	HP 360
Factored Lateral Resistance at ULS, kN	120	170
Lateral Resistance at SLS, kN	50	70

If greater resistance is required, batter piles should be installed.

The coefficient of horizontal subgrade reaction,  $k_s$ , should be computed using the following equation to evaluate the point of contraflexure:

$$k_s = n_h z/b$$

where  $n_h$  = coefficient related to soil density,  $\text{kN/m}^3$   
 = 10,000 for granular backfill

$z$  = depth, m

$b$  = pile width, m



### 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 may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:

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

where  $K$  = coefficient of lateral earth pressure (dimensionless)  
 $\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$   
 $h$  = 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  $\phi$  = angle of internal friction of retained soil ( $35^\circ$  for Granular B Type II)  
 $\delta$  = angle of friction between the soil and wall ( $23.5^\circ$  for Granular B Type II)

The seismic site coefficient and zonal acceleration ratio for the conditions at this site were provided in section 2.1.5.

Free-draining granular material or rockfill should be used as backfill behind the walls. The following parameters are recommended for design:

Parameters	Granular A or Granular B Type II	Rockfill
Angle of Internal Friction, degrees	35	42
Unit Weight, $\text{kN/m}^3$	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 of Passive Earth Pressure $K_p$	3.69	5.04

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  $10^\circ$  to the horizontal. Alternatively, the



material above the top of the wall could be treated as a surcharge load ( $q$  in the preceding equation).

A weeping tile system (SP 405F03 and OPSD 3190.100) should be installed to minimise the build-up of hydrostatic pressure behind the walls. 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 placed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standards specifications for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200).

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 height of fill embankments will be 9 to 13 m at the south approach and 1 to 12 m at the north approach. It is anticipated that the approach embankments will be constructed with earth borrow, granular material or rockfill. Construction of the fill on bedrock is considered to be feasible.

The topsoil identified at the abutment locations and along the alignment of the approach fills within 20 m of the abutments should be stripped prior to placement of the embankment fill. All loose boulders that are exposed at the subgrade level should be removed from each approach embankment footprint.

Backfilling adjacent to the structure abutments should be carried out in conformance to Ontario Provincial Standards specifications for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200). As noted in section 2.2, Granular A or Granular B Type II (maximum particle size of 75 mm) should be employed within the limits of driven piles at the south abutment, if utilised.



The embankments should be constructed in accordance with OPSD 201.020, 202.010 and SP 206S03. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rockfill. A 2 m wide mid-height berm should be provided since the anticipated maximum slope height is 12 to 13 m (OPSD 202.010).

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 guiderail, a granular infilled drainage gap should be provided in accordance with the Northeastern Region Pavement Design Practices and Guidelines as shown in Figure 2, appended. OPSS Granular B Type II should be used for the drainage gaps.

Where the bedrock surface slopes in the east-west direction, it should be benched using a bench between 0.3 and 1.0 m high and a minimum 2.0 m wide to provide stable conditions for the embankment fill construction transversely to the highway alignment.

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Settlement of the road surface will only be governed by 'consolidation' of the newly placed fill (settlement of the embankment fill due to consolidation of the bedrock at both embankments is negligible).

The backfill placed adjacent to the abutments will be about 8 to 13 m thick. The magnitude of 'consolidation' of this fill will be dependent on the workmanship employed by the contractor and, if placed in 200 to 300 mm thick lifts compacted to 100% of the standard Proctor maximum dry density in accordance with the requirements of OPSS 902 and OPSS 501 (Method A), should be in the order of 25 mm at the south abutment and 5 to 30 mm at the north abutment. The settlement of the approach fill surface near the abutments should be essentially complete within 2 to 4 months after placement of the fill.

The total settlements of the south and north approach embankments made up of rockfill beyond the granular fill zone are estimated to be 75 to 155 mm and 10 to 135 mm respectively. The settlements remaining after 6 months following fill placement are 15 to 30 mm at the south approach and 5 to 25 mm at the north approach. The long-term settlements (after 12 months) are



10 to 15 mm and 0 to 15 mm respectively. These estimates are in accordance with the current MTO guidelines.

The embankment platform founded on bedrock should be widened by 1 m in accordance with the Northeastern Region Engineering Directive (NRE 98-200).

Earth fill slopes where employed should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

## **5. EXCAVATION AND GROUNDWATER CONTROL**

It is expected that excavation for construction of spread footings founded on bedrock will be minimal. At the south abutment location excavation will extend through the existing fill to a depth not exceeding 2 m.

The fill is classified as Type 3 soil according to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes over the full depth of excavation should therefore be inclined at an angle of 45° to the horizontal. The need to excavate flatter sideslopes if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should be considered.

Bedrock is classified as Type I soil. Near vertical sidewalls may be utilised for excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workmen.

Mechanical means such as a large excavator equipped with a tiger-toothed bucket in conjunction with a jack-hammer or hoe ram is the preferred method of excavation to shallow depths in rock scaling at foundation locations (SP 299F03). Conventional rock excavation techniques such as blasting (OPSS 120), controlled blasting (SP 299F06) and trim blasting (SP 299F04) are likely to be required as well. The actual equipment required and method of excavation within the bedrock





will be dependent upon the geometry of cut and the relative depth of excavation into the bedrock. Mass concrete could be employed to level minor variations in the bedrock surface.

If blasting is required, a NSSP should be prepared to provide specific direction to the contractor to control the blasting / rock excavation activities to prevent fracturing and/or disturbance of the bedrock surface on which footings will be founded; require that a blasting specialist be retained to establish the charge to minimise overbreak; advise the contractor that any overblasting/overexcavation will be the sole responsibility of the contractor and require that loosened rock resulting from blasting operations be removed by mechanical means.

In addition, the Department of Fisheries Guideline for the use of explosives in or near Canadian Fisheries Water (Canadian Technical Report of Fisheries and Aquatic Sciences 2107 dated 1998) should be followed when conventional rock excavation techniques such as blasting are required near the Pickereel River to protect the water environment and fish habitat.

Groundwater was not observed in any of the boreholes during or upon completion of drilling. It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the foundation excavations. Groundwater levels are subject to seasonal fluctuations and precipitation patterns.

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**

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

**NOTE: Hard copies signed  
and stamped**

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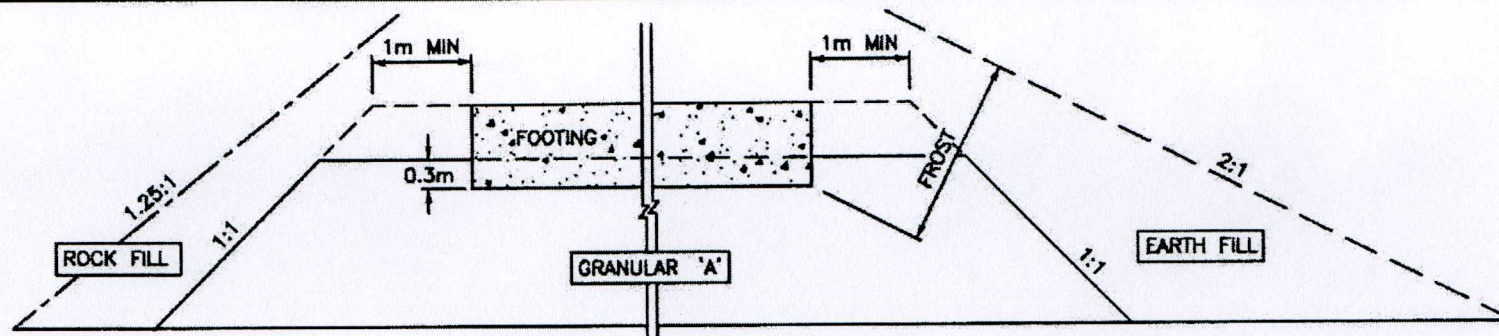
**TABLE 1**  
**SUMMARY OF ADVANTAGES, DISADVANTAGES AND RECOMMENDED FOUNDATIONS**

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RECOMMENDED FOUNDATION TYPE
SOUTH AND NORTH ABUTMENTS			
Spread footings on rock	<ul style="list-style-type: none"><li>• Ease of construction</li><li>• High bearing resistance relative to footings on engineered fill</li><li>• Minimal requirement for rock excavation</li><li>• No requirement to provide erosion protection</li></ul>	<ul style="list-style-type: none"><li>• Need to place mass concrete to provide a level surface</li></ul>	Spread footings
Spread footings on engineered fill pad	<ul style="list-style-type: none"><li>• Ease of construction</li><li>• No requirement for rock excavation</li></ul>	<ul style="list-style-type: none"><li>• Difficulty placing engineered fill on undulating / sloping rock surface</li><li>• Lower bearing resistance than for other alternatives</li><li>• Need to provide erosion protection</li></ul>	
Driven piles (not practical at north abutment)	<ul style="list-style-type: none"><li>• High capacity</li></ul>	<ul style="list-style-type: none"><li>• Short piles</li><li>• High cost relative to footings</li></ul>	
Caissons	<ul style="list-style-type: none"><li>• High capacity</li></ul>	<ul style="list-style-type: none"><li>• Special construction methods on sloping bedrock</li><li>• High cost relative to other alternatives</li></ul>	
SOUTH AND NORTH PIERS			
Spread footings on rock	The only practical solution		Spread footings on rock
Spread footings on engineered fill pad	Not appropriate		
Driven piles	Not appropriate		
Caissons	Not appropriate		



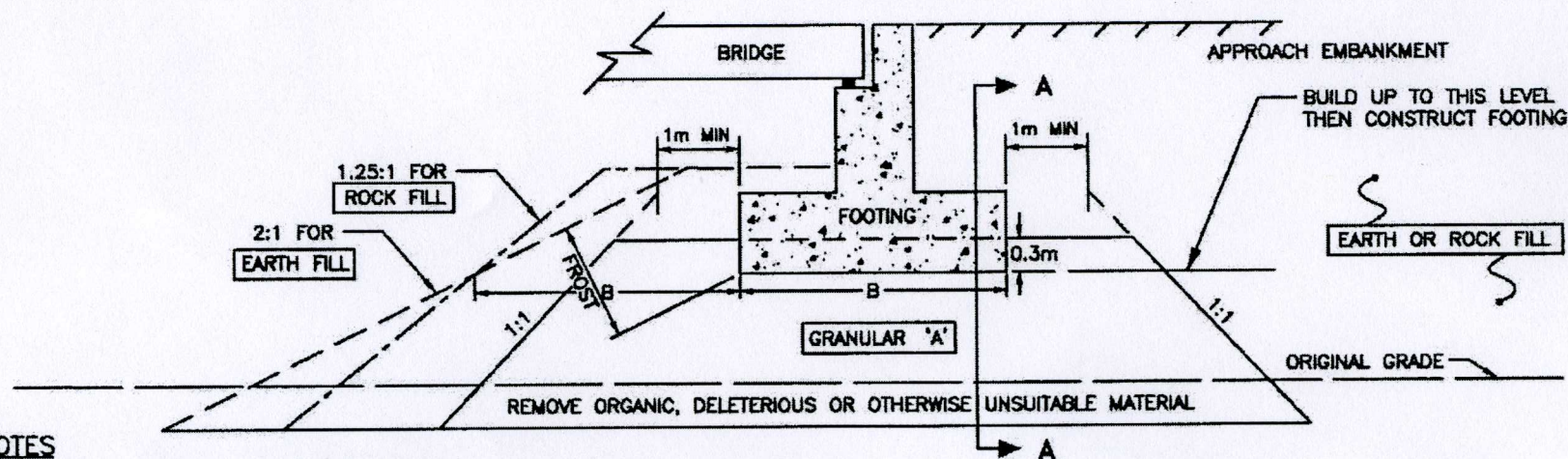
**TABLE 2**  
**LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT**

<b>DOCUMENT</b>	<b>TITLE</b>
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
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Piling
OPSS 1010	Material Specification for Aggregates
SP 105S10	Construction Specification for Compaction
SP 206S03	Construction Specification for Grading
SP 299F03	Rock Excavation (Machine Scaling)
SP 299F04	Rock Excavation (Trim Blasting)
SP 299F06	Rock Excavation (Controlled Blasting)
SP 405F03	Construction Specification for Pipe Subdrains
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock
SP 999S29	Dowels into Concrete
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-3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail
NRE 98-200	Northeastern Region Directive - Platform Widening



**CROSS SECTION A-A**

NOT TO SCALE



**LONGITUDINAL SECTION**

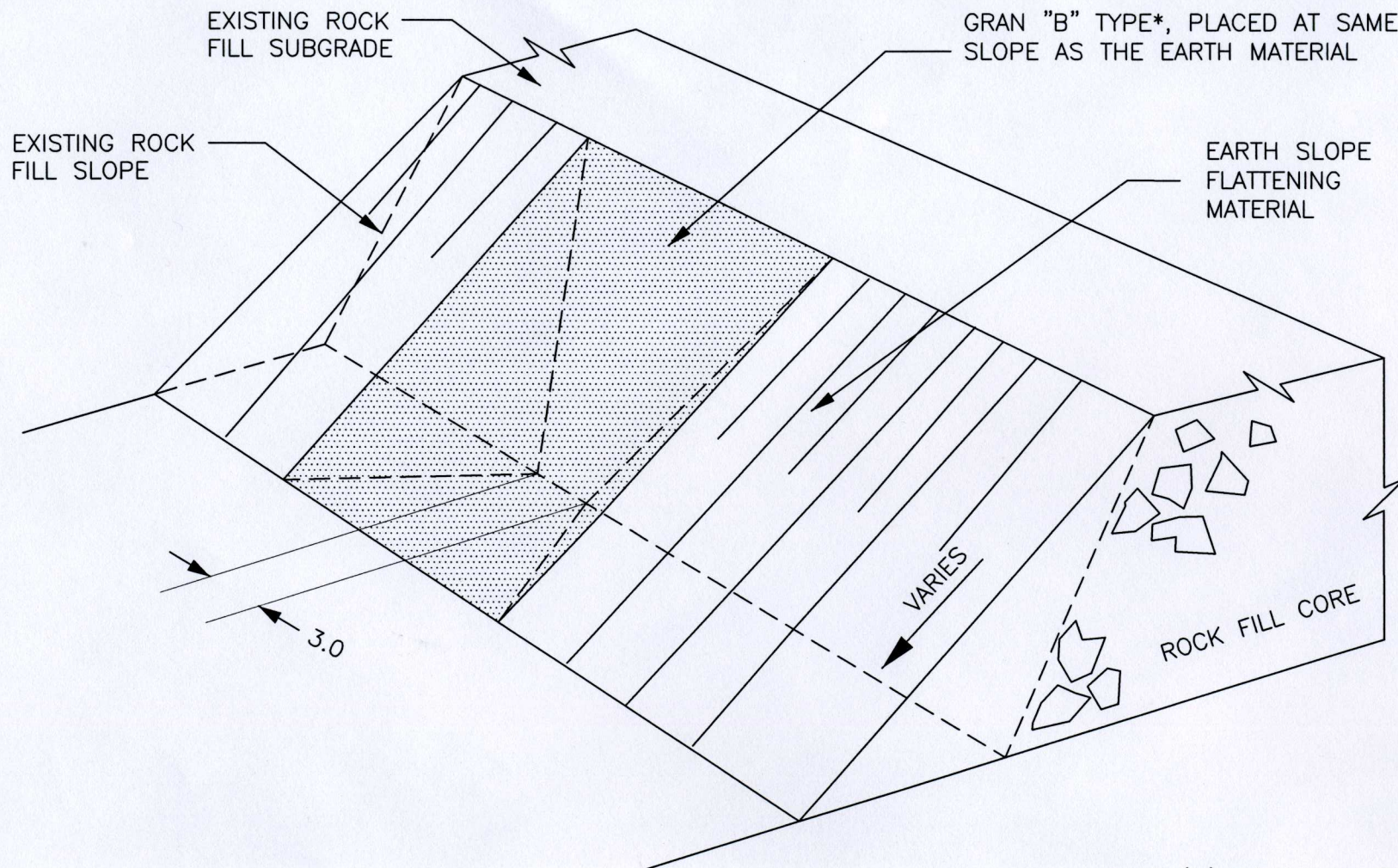
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**NOTES**

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**





\* GRAN 'B' TYPE I OR TYPE II AS RECOMMENDED FOR PROJECT.

FIGURE 2: ROCK FILL DRAINAGE IN SLOPE FLATTENED AREAS

NOT TO SCALE