

**MATERIALS ENGINEERING AND RESEARCH OFFICE**  
PAVEMENTS AND FOUNDATIONS SECTION

**GWP 5427-06-00**  
**HWY 522**

**DISTRICT: 54**  
**STR SITE: 44-051**

Trout Creek Bridge Replacement

**DISTRIBUTION**

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# Foundation Design Report

**FOUNDATION DESIGN REPORT**  
**For**  
**Trout Creek Bridge Replacement**  
**GWP 5427-06-00, Site 44-051, Highway 522, North Bay**

**DISCUSSION AND RECOMMENDATIONS**

**General**

This report provides Foundation engineering discussion and recommendations for the design and construction of the foundations and approach embankments for the proposed new bridge that will replace the existing Trout Creek Bridge on Hwy 522 in the Township of South Himsforth, North Bay.

The existing bridge is a 12m clear single span structure. The distance between back faces of pile caps of the existing span is 16.5m. Extension to the existing bridge was carried out in or after 1937. It appears from an old GA drawing from Contract No. 37-30 dated May 18, 1937 that the existing bridge is supported on deep foundations.

The new bridge will be a 20m single span structure. The new bridge will be prestressed concrete box (B800) with 150mm concrete topping slab and 90mm asphalt water proofing. The width of the new bridge will be more or less the same as the existing structure. There will be one lane of traffic in each direction on the new bridge. The new bridge will be constructed at the same alignment and location. There will be no grade raise. The highway will be closed during construction so; stage construction or temporary modular bridge will not be required. Consideration will be given to supporting the bridge on integral abutments.

The subsurface investigation for this project was carried out by DST Consulting Engineers under MTO Agreement No: 5011-E-0035 (MTO Northeastern Region Geotechnical Retainer).

In summary, the subsurface condition, based on DST's report, consists of more than 20m deep mainly non cohesive soil comprised of silt, sand, gravel and cobbles. The relative density of the soil ranged from very loose to compact. Bedrock was encountered at depths 21.3m to 22.3m below approach embankments (Elevation 291.3m to 292.3m). Details of the soil condition are provided in DST's Report GS-TB-018036 dated May 22, 2014.

## **Bridge Foundation**

Shallow foundation is not feasible for the new bridge foundation due to low geotechnical resistance of the native soil. Deep foundation such as augured piles (caissons) may also not be a good option for this site due to non-cohesive overburden and high water table. Steel tube piles were not considered because they are high displacement piles and typically experience high penetration resistance. Therefore, driving steel tube piles to the required penetration depths and capacity would be difficult.

Based on the soil conditions encountered in the boreholes, deep foundations using steel H-piles driven to refusal on bedrock is the preferred foundation option. Due to the presence of potentially sloping granite bedrock and boulders the steel piles should be equipped with rock points (Titus H Bearing Points Rock Injector Model).

As the new abutments will be located outside the foot print of the existing abutments, there will be no obstruction in pile driving from the existing buried piles. It is expected that the tip of the piles will reach to elevation 291.3 m at the north abutment and 291.8 m at the south abutment (Table 1). However, the pile tip elevations may vary significantly in the field as the soil and bedrock information between the boreholes could be different.

Table 1 – Bedrock Elevation in the Boreholes

<b>Location</b>	<b>Boreholes</b>	<b>Refusal</b>	<b>Depth (m)</b>	<b>Elevation (m)</b>
South Abutment	BH2	Bedrock (Cored)	21.8	291.8
South Abutment	BH3	Probable Bedrock	21.3	292.3
North Abutment	BH4	Bedrock (Cored)	22.3	291.3
North Abutment	BH5	Probable Bedrock	21.8	291.8

Two steel H-pile sections (HP 310 x 110 and HP 360 x 152) are considered. The following factored geotechnical resistance at ULS for the noted sections of steel piles are recommended:

<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 reaction at SLS allows for 25 mm of the settlement. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement criteria since the loading required for producing a 25 mm settlement would be larger than the factored geotechnical resistance at ULS.

To facilitate pile driving particle sizes of any fill placed beneath the pile locations should be restricted to 75mm. The piles should be advanced with a hammer capable of developing sufficient energy to drive the piles.

In view of the layers of cobbles and boulders encountered in the boreholes, 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 essential than is called for in OPSS 903.

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.

### **Integral Abutment**

This site is feasible for integral abutments. If integral abutments are considered for this site, it will be constructed on steel piles driven to bedrock. The piles for integral abutments should be installed in one row.

Two concentric CSPs that extend at least 3m below the bottom of the abutment stem 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 0.6m diameter and 3m deep CSP filled with loose uniform sand can be used. Following is the NSSP for the special sand used for integral abutments.

### **NSSP - Backfill to Integral Abutment-Augured Hole**

The annular space between the CSP pipe and the pile shall be backfilled with uniformly graded sand. The gradation for the uniformly graded sand shall be as follows:

<b>MTO SIEVE DESIGNATION</b>	<b>PERCENTAGE PASSING BY MASS</b>
2 mm (#10)	100
600 mm (#30)	80 - 100
425mm (#40)	40 - 80
250 mm (#60)	5 - 25
150 mm (#100)	0 -6

Commercially available materials which meet the above gradation may be considered.

## **Lateral Resistance**

Resistance to lateral load may be provided in part by mobilization 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/pile	120	170
Lateral Resistance at SLS, kN/pile	50	70

<b>Native Sandy Soil</b>		
Pile Section	HP 310	HP 360
Factored Lateral Resistance at ULS, kN/pile	100	140
Lateral Resistance at SLS, kN/pile	40	60

If integral abutments are not considered and greater resistance is required then the lateral capacities of the piles may be supplemented by the horizontal component of the battered piles.

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 or native sand till  
 $z$  = depth, m  
 $b$  = pile width, m

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:

<b>Pile Spacing in Direction of Loading D=Pile Diameter or Width</b>	<b>Subgrade Reaction Reduction Factor, R</b>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## **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  $\emptyset$  = 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)

Free draining granular material such as Granular 'A' or 'B', or rockfill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure build-up.

If rockfill is used for approaches, special care will be required to avoid damaging to the abutment. It would be preferable to place a 0.3m cushion of Granular 'A' or smaller rockfill (with diameter of less than 300mm), between the structure and the main mass of rock fill. Granular material may also be used at the approaches.

For design purposes, the following properties for backfill are recommended:

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

The coefficient of earth pressure at rest ( $K_o$ ) 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 (OPSS 405 and OPSD 3190.100) should be installed to minimize 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.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standards Drawings for granular or rock backfill at abutments (OPSD 3101.150 and/or 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 OPSS 501 for additional information in this regard.

Backfill shall not be placed behind the abutments until the deck ends have been constructed and have acquired design compressive strength. Backfill shall be placed simultaneously behind both abutments keeping the height of the backfill approximately the same. At no time the difference in the elevations shall be more than 500mm.

### **Frost Protection**

A soil cover of 1.9m (OPSD 3090.101) or equivalent will be required for frost cover for pile caps. The pile caps should be constructed below the frost depth or the scour depth whichever is greater. A 25mm thick layer of polystyrene insulation is thermally equivalent to 600mm of soil cover.

### **Excavation and Dewatering**

Excavation for the pile caps or abutment walls will take place within silty sand to sand deposit slightly below the water level in the creek (Elevation 309.4m). Excavation for the pile caps can be carried out within the confines of continuous steel sheeting driven into the silty sand to sand deposit. For basal stability, the sheeting should extend a minimum of  $0.5 B$  below the base of the excavation (where  $B$  is the width of the excavation). For design of the sheeting, an earth pressure coefficient of  $K=1.0$  should be used. Sump pumps may be used to control the seepage and lowering the water level within the cofferdams. Clear stone may be placed at the base of the excavation to act as construction platform.

### **Embankment Stability and Settlement**

The heights of the existing embankments close to the abutments are about 5 m. The side slopes of the existing embankments are approximately at 2H:1V. As the grade will not be raised, there will be no additional loading on the approach fills. Therefore, there will be no stability or settlement concerns at the approach embankments.

## **MISCELLANEOUS**

The Foundation Investigation for this project was carried out by DST Consulting Engineers. The factual portion of the report was produced by DST Consulting Engineers. The Discussion and Recommendation part of the Foundation report was written by Ken Ahmad, Foundations Engineer of the Pavements and Foundations Section.

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