

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 557 GRADE RAISE AT DUBORNE LAKE  
AGREEMENT No.: 5006-E-0071  
GEOCRES No. 41J-77  
ASSIGNMENT No. 8**

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**Prepared For:  
Ministry of Transportation  
Engineering Office  
Northeastern Region  
447 McKeown Avenue, Suite 301  
North Bay, Ontario P1B 9S9**

6 Copies - Ministry of Transportation, North Bay, ON  
1 Copy - DST Consulting Engineers Inc., Thunder Bay

**DST CONSULTING ENGINEERS INC.**  
605 Hewitson Street, Thunder Bay, Ontario P7B 5V5  
Phone: 1-807-623-2929 Fax: 1-807-623-1792

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**PART 1: FACTUAL INFORMATION**

**1.0 INTRODUCTION**

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation, Geotechnical Section, Northeastern Region to conduct a geotechnical investigation to evaluate the feasibility of raising the grade of Highway 557, adjacent to Duborne Lake (Upper Cranberry Lake) located 17 km north of Highway 17 at Blind River. A total of 1 testpit and 8 boreholes were advanced, 7 of which are located beyond the toe of slope staggered left and right of the centreline every 50 m and 2 are located at the edge of shoulder.

This report addresses the field investigation undertaken, laboratory test program performed and summarizes the factual information on subsurface conditions encountered and provides conclusions and recommendations for the grade raise. This work was carried out under Agreement No. 5006-E-0071.

The purpose of this investigation is to provide recommendations regarding the feasibility of raising the embankment by 1.0 m to reduce the impact of a high water level. Particular issues for this investigation are stability and settlement of the roadway embankment if supported on soft ground. The terms of reference that in the case of soft ground conditions sufficient field information is required to support a second phase of work with a more detailed evaluation on stability and settlement.

**2.0 SITE DESCRIPTION**

The geotechnical investigation extends for 300 m and is located 17 km north of Highway 17 on Highway 557. The culvert at the south end of the project was assigned as Sta 10+000. Along this section of highway alignment the embankment elevation is between 0.3 and 2.7 m above the elevation of the surrounding muskeg terrain. Fill from a previous Highway 557 alignment is

encountered at Sta 10+105 to the west of current alignment where it extends to Sta 10+265 and crosses to the east of current alignment progressing to the northern extent of the investigation. At Sta 10+185 a transition from swamp to higher elevation is encountered with bedrock and shallow overburden beginning at Sta 10+250. The site vegetation consists of tag alders within the muskeg terrain and mixed forest beyond the ditch line as elevations increase at the northern and southern extents.

At the time of investigation, the area of study was mostly cleared of snow to the peat layer. A layer of snow and ice of 0.4 m was only encountered at Borehole 3. The adjacent terrain was covered with 0.5 m to 1.0 m of snow. Pictures of the site from Google Earth™ and the northern and southern extents are shown below.



Satellite Imagery of Highway 557 at Duborne Lake



North end of site facing south



Highway 557 Proposed Grade Raise – At north limits looking south





South end of site facing north



Highway 557 Proposed Grade Raise – At south limits looking north, in front of rock out crop

### **3.0 INVESTIGATION PROCEDURES AND LABORATORY TESTING**

Site work was carried out between February 19<sup>th</sup> and February 25<sup>th</sup>, 2008 utilizing a CME 45 drill rig mounted on a towable chassis, equipped for geotechnical drilling and operated by DST as well as a Case 580 rubber tire backhoe operated by JI Enterprises. Six boreholes and one test pit located beyond the toe of slope were advanced to depths ranging from 2.0 to 15.5 m at 50 m spacing. Two boreholes were advanced at the edge of the paved shoulder to depths of 13 m and 2.4 m at Sta 10+100 and Sta 10+250 respectively. All boreholes and test pits were located as indicated on the Site Plan provided by the Ministry of Transportation in Agreement # 5006-E-0071, Assignment # 8 Order received in January 2008 from Jason Wright.

Boreholes 3 through 6 located from Sta 10+003 to Sta 10+200 beyond the toe of slope were advanced through expected peat and clay layers. All these boreholes encountered refusal on probable bedrock between 7.0 m and 15.0 m except Borehole 3 where drilling proceeded to a depth of 15.5 m without refusal. Borehole 2 and Test Pit 1 located at Sta 10+150 and Sta 10+250 respectively west of the current alignment and Borehole 7 located at Sta 10+300 east of the current alignment were advanced through materials which included fill from a previous highway alignment. Test pit 1 and Boreholes 2 and 7 encountered bedrock at 2.0 m, 6.3 m and 3.4 m respectively. Boreholes 8 and 9 located at Sta 10+250 and Sta 10+100 edge of paved shoulder respectively were advanced through the current embankment material. Borehole 8 was advanced to 2.3 m where refusal on bedrock was encountered. Borehole 9 was advanced to 13.0 m without encountering bedrock.

Boreholes were advanced with hollow stem augers. Soil samples were obtained from the auger flights, Shelby tube sampler and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 51 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 310 mm is known as the standard penetration blow count (N) which provides an indication of the denseness or consistency of the soil. Representative soil samples are obtained from within the sampler. A dynamic cone penetration test (DCPT) was also performed in Boreholes 3 and 4 below a 12 m depth. Borehole Logs are presented as Enclosures 1 to 9.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, supervised the drilling, sampling and in-situ testing and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included natural moisture content, gradation and plasticity limits. Laboratory results are presented on the Borehole Logs and Enclosures 10 to 11.

Ground surface elevations at the borehole locations were surveyed by DST personnel, using a benchmark with an assigned elevation of 100.00 m etched into the bedrock 9.5 m left of centreline at Sta 9+995.



## **4.0 DESCRIPTION OF SUBSURFACE CONDITIONS**

### **4.1 Field Observations**

The generalized stratigraphy beyond the toe of slope from Sta 10+003 to Sta 10+2000 consists of peat overlying a thick soft gray clay deposit which extends to bedrock at depths from 6.3 m to greater than 15.5 m below grade. The generalized stratigraphy beyond the toe of slope from Sta 10+200 to Sta 10+300 consists of a mix of fill, clay and silt deposits overlying probable bedrock at depths from 2.0 to 7.0 m below grade.

### **4.2 Topsoil**

A topsoil layer of 0.2 m thickness was encountered at Test pit 1 and Borehole 6.

### **4.2 Asphalt**

An asphalt (surface treated) layer of 50 mm thickness was encountered at Boreholes 8 and 9.

### **4.3 Fill**

A sand fill of 0.9 to 2.6 m in thickness is present in Boreholes 7, 8 and 9. Occasional cobbles were encountered within this deposit.

SPT values between 9 and 17 blows per 0.3 m indicate a loose to compact condition (compactness condition). Gradation analyses conducted on a sample from Boreholes 9 indicates gravel, sand and fines contents of approximately 8%, 76% and 16% respectively. Grainsize distributions are reported on the Borehole Logs and are plotted on Enclosure 10. The moisture contents of samples ranges from 8% to 13%.

### **4.4 Silt**

A silt layer between 0.4 and 1.8 m in thickness is present in Test pit 1 and Boreholes 7 and 8.

An SPT value of 17 blows per 0.3 m measured in Borehole 8 indicates a compact condition (compactness condition). Gradation analyses conducted on a sample from Borehole 8 indicates gravel, sand and fines contents of approximately 3%, 33% and 64% respectively. Grainsize distributions are reported on the Borehole Logs and are plotted in Enclosures 10. The moisture

contents of samples range from 12% to 19%.

#### **4.4 Peat**

Outside the embankment at Boreholes 2, 3, 4 and 5 the peat depth varies from 1.5 to 2.3 m. In general, the data indicates that the base of the peat is relatively horizontal without significant slope.

The peat was classified with respect to the modified Von Post System (see Records of Boreholes). The degree of humification varies widely from insignificant to nearly complete. The peat typically has low to moderate fine fibre content, nil to low coarse fibre content and nil to low wood or shrub remnant content. The water content of the peat was found to be from 149% to 812% for the samples collected.

#### **4.5 Clay and Silt**

In Boreholes 2, 3, 4, 5, 6, and 9 a grey clay deposit of thickness from 4.8 to greater than 13.2 m was encountered. The clay at these boreholes has a consistency of very soft to stiff as indicated by in-situ vane shear strengths varying between 7 kPa and 57 kPa. It is classified as having low sensitivity, indicated by in-situ vane test results with sensitivities of less than 10 and noted in the Borehole Logs. In situations where remolded vane test values were unattainable due to consistency of the material it is noted in the remarks on the Borehole Logs. The clay has a medium plasticity as indicated by Atterberg limit tests (Enclosure 11) conducted on samples from Boreholes 3 and 4 at depths of 4.6 and 9.1 m respectively. The moisture contents of samples clean of organics range from 28% to 97%. In Boreholes 3, 4 and 5 the moisture contents of samples contaminated with organics range from 86% to 170%.

In Boreholes 6, 7, and 8 a brown clay deposit of thickness from 0.6 to 2.8 m was encountered. The clay at these boreholes has a consistency of firm to hard as indicated by in-situ vane shear strengths varying between 44 kPa and greater than 220 kPa. It is classified as having low sensitivity, as indicated by in-situ vane test results with sensitivities of less than 10. The clay has a medium plasticity as indicated by Atterberg limit tests (Enclosure 11) conducted on samples from Boreholes 6 and 7 at depths of 0.75 and 1.5 m respectively. The moisture contents of samples range from 23% to 48%.

#### **4.6 Groundwater**

Water depths on completion of drilling are indicated on the individual borehole logs and on the soil profile. In all boreholes temporary standpipes were installed to measure the groundwater table. The groundwater level stabilized between elevations 96.85 m and 98.35 m and typically would be expected to reflect the water level in the adjacent ditches. It should be noted that groundwater levels will fluctuate seasonally and in response to climatic conditions.

Borehole No.	Water Level			
	Depth Below Grade		Elevation	
	On Completion (m)	Post Drill (m)	On Completion (m)	Post Drill (m)
TP 1	NR	NR	NR	NR
BH 2	0.30	0.30	98.35	98.35
BH 3	1.00	0.40	96.98	97.58
BH 4	0.30	0.30	97.31	97.31
BH 5	0.00	0.00	97.58	97.58
BH 6	2.00	0.40	95.99	97.59
BH 7	2.00	1.50	97.73	98.23
BH 8	2.00	1.85	98.20	98.35
BH 9	1.80	1.20	96.25	96.85

NR - Not Recorded

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**PART 2: DISCUSSION AND RECOMMENDATIONS**

**5.0 DISCUSSION**

DST Consulting Engineers Inc. (DST) has been retained by Ministry of Transportation (MTO) to conduct a foundation investigation and design to evaluate the feasibility of raising the grade of Highway 557, adjacent to Duborne Lake (Upper Cranberry Lake). The site is located 17 km north of Highway 17 (Blind River), just past Old Steel Road.

For much of this 300 m section, the road is as little as 0.3 m above the elevation of the surrounding muskeg terrain to the north and south (lake side) of the road. The purpose of the 1.0 m grade raise is to reduce water impacts on the road.

Key design criteria are that the existing road should not be unduly affected, that the new raised roadbed remains stable over the long term and that future road settlement is acceptable from an operational point of view.

The past history of the site is not well documented. The embankment was likely constructed many years ago. Given the very low elevation of the road and the deep soft clay deposits in one section of the road, it appears likely that the embankment has suffered settlement under long term consolidation of the supporting soil deposits. It is understood that in recent years this section of the road was raised 0.3 m.

The subsurface conditions below the road consist of 1.5 to 2.6 m of sand fill overlying a deep clay deposit identified at Boreholes 9 and 8. In general, the subsurface conditions beyond the existing embankment toes consist of up to 2.3 m of peat over very soft to stiff clay. Two distinctive clay deposit thicknesses are present, 8.0 to 13.2 m thick from Stations 10+000 to 10+100, and 2.1 to 6.8 m thick from Stations 10+100 to 100+300.

The subsurface conditions below the embankment at Borehole 9 indicate that the embankment was likely constructed by the displacement method over the muskeg terrain and was initially slightly shifted to the north of the existing alignment. Where the underlying clay was too weak to support the weight of the fill materials, the fill punched through the peat (see Figure 1 as a typical illustration) and displaced both peat and clay until reaching equilibrium at a depth in the clay where its strength was sufficient to support the fill. A low mud wave adjacent the embankment would have resulted. With time, the clay strength would have increased as a result of consolidation under the weight of the fill.

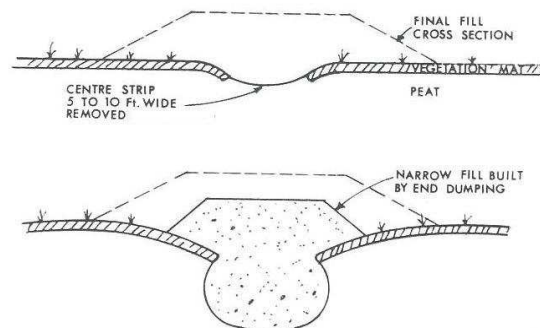


Figure 1 Typical Displacement Method

Experience with peat displacement methods is that often zones of peat remain confined within or below the fill, and also that if the base of the peat does not slope significantly, the fill remains directly under the location where it is placed as a 'bulb'. The undrained shear strengths estimated for the clay below the fill using field vane tests when compared with vertical overburden stress are also consistent with such a bearing capacity type of displacement. The thick nature of the underlying clay means that considerable consolidation settlement under the fill load is possible albeit very slowly over many years, given the thick nature of the deposit. Calculations completed using published correlations and the limited field data indicate that the clay is likely a normally consolidated clay deposit.

Preliminary analysis of the existing embankment indicates that the embankment has likely experienced gradual settlement as a result of consolidation in the deep clay layer.

In order to raise the roadway some options may be considered while providing mitigative measures for the settlement and stability concerns. In general, a grade raise conventional fill will require very careful attention to embankment stability as well as continuing long term settlement. Geosynthetics can address stability but not settlement in this case. Alternatively, lightweight fill can be used to reduce mass, associated gravitational driving forces and vertical stresses that play a key factor in stability and settlement of the final solution. Lightweight materials that have been used in embankment construction include chipped bark, sawdust, dried peat, fly ash, slag, cinders, cellular concrete, lightweight aggregates, expanded polystyrene geofoam, shredded tires, and sea shells, of these polystyrene is the only practical material. Construction considerations would include partial removal of the existing embankment to accommodate the new road structure. Another alternative is improving the shear strength and compressibility characteristics using ground improvement techniques in combination with road raise. As part of the detailed analysis these options may be considered into the assessment

Because of the very high costs associated with the lightweight fill and ground improvements, the preferred option is grade raise with conventional fill materials, on the assumption that its feasibility will be confirmed with a detailed assessment.

### **Grade Raise**

The following options were considered for raising the existing road grade:

Option 1: Raise the existing road grade in one lift giving sufficient allowance for future settlements.

Option 2: Raise the existing road grade in increments. This method is appropriate for a 0.50 m or lower raise, or if the 1.0 m lift is to be completed in two or more increments.

For both options, a symmetrical widening of the existing road structure it is recommended to minimize the additional stresses on the existing native soils.

Using the parameters from the soil investigation a preliminary assessment was made of a grade raise of 0.50 and 1.0 m. For the 0.50 m raise the embankment widening required to achieve this is



expected to be minimal. For the 1.0 m raise the embankment widening required may produce significant settlements particularly in the organic deposits along both sides of the existing road embankment.

An immediate settlement of up to 180 mm was estimated for the clay as a result of the 1.0 m grade raise. Furthermore, the settlement from primary consolidation in the clay is expected to be greater than 150 mm over most of the embankment length. The peat consolidation will likely occur over several months, and may require some maintenance work, given the potential variable distribution of this deposit in both sides of the embankment. The sand fill consolidation will be insignificant. The deep clay consolidation may take place over several decades. Additional analyses and calculation should be carried out to accurately assess the above including time lines, secondary consolidation and stability.

At Borehole 5 a very soft clay material was found between 4.6 and 7.6 m below surface with an estimated shear strength of only between 4 and 5 kPa. However, this section of the highway may not require a full 1.0 m raise. A maximum 0.3 m raise may be feasible for the section with no significant impact to the culvert at Sta 10+000.

## **6.0 RECOMMENDATIONS AND CONCLUSIONS**

The results of the soil investigation indicate that this 300 m long embankment is supported over very weak ground, essentially a peat deposit up to 2.3 m thick overlying a deep deposit of very soft or soft clay. Our preliminary analyses indicate that the settlement which the existing embankment has experienced over the years is likely a result of long term consolidation in the clay, perhaps still continuing today. In addition, there may have been a component caused by slow creep within overstressed zones in the clay.

A preliminary assessment of raising the grade by 0.5 m or 1.0 m using conventional fill material indicates that this is expected to be feasible with respect to stability. The new shoulders as a result of the necessary widening will likely experience considerable and variable settlement as a result of the intermittent peat layer remaining near the existing shoulders, although this effect is considerably less with a 0.5 m raise compared to a 1.0 m raise. Long term slow settlement of the entire embankment will nevertheless continue as a result of deep consolidation within the clay.

To provide a geotechnical design for the grade raise, a detailed assessment is recommended. The field data available from this investigation is considered adequate to complete this. Further laboratory testing for strength and compressibility characteristics (e.g. consolidation and direct shear tests) as well as detailed analyses of options for stability and settlement (Using computer modeling software) would be required.

Where new fill material is required over peat, a woven class II geotextile is recommended as an interface between organics and fill material to evenly distribute the load of the fill and provide high tensile strength and adequate infiltration capability (filtration opening size -F.O.S.- no less than 300 micrometres). Once the snow and ice are gone the toe of the embankment should be accurately located.