



**FOUNDATION DESIGN REPORT
200m NORTH OF NESTOR FALLS BRIDGE
W.P. 306-85-00
HIGHWAY 71, DISTRICT OF THUNDER BAY, GODSON TOWNSHIP**

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TG98146**

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MTO GEOCRETS NO. 52-F-26



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1.0 INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by Cook Engineering to conduct foundation investigations for five sites along Highway 71. The investigation for four of the five sites has been previously reported in our reports "Foundation Investigation Report and Foundation Design Report, Four Roadway Settlement Areas, W.P. 306-85-00 Highway 71, District of Thunder Bay, Willingdon Township" (January 1999). This report presents the Foundation Report for the fifth site located approximately 200 m north of the Nestor Falls Bridge on Highway 71, adjacent to Kakabikitchiwan Lake.

Authorization to proceed with this work was received from Cook Engineering Co. This work was carried out as part of the Total Project Management project for the Ministry of Transportation, Ontario (MTO) under W.P. 306-85-00.

Details of the method of investigation are provided in Appendix 'B' and are briefly discussed in the following paragraphs. At this site, three of the boreholes (No. 1 to 3) are located in Kakabikitchiwan Lake within the MTO. right-of-way. Access to these locations was not possible in the fall and the drilling program was rescheduled to the winter months after freeze up in order for the holes to be drilled from the ice. Four boreholes on land (No. 4 to 7) were completed in September, 1998. Enclosures 3 to 6 (Borehole Logs) provide the subsurface conditions at the boreholes.

2.0 PROJECT DESCRIPTION

The area to be investigated is located about 200 m north of the Nestor Falls Bridge on a horizontal curve along the section of highway adjacent Kakabikitchiwan Lake. MTO was unable to locate the original construction records for this section of highway. It is understood that the highway was constructed in the 1960's and a cut and fill operation was employed in this section of the highway. The south bound lane required blasting of the bedrock and the resulting rockfill was used for the north bound lane.

Pavement distresses noted by the MTO Patrol through this area included longitudinal cracks and the settlement of the northbound lane with areas of cold mix and hot mix patching. The noted section of highway is reported to have been settling since it was constructed. During construction, the northbound lane grade had to be restored several times as it seemed to be 'sliding' down towards the lake. Continual maintenance has been required through this settlement area including cold mix patching approximately every second year and several hot mix patches have been placed since construction. Resurfacing of this section of highway was completed in 1998.

3.0 SUBSURFACE CONDITIONS

3.1 Roadway

The roadway structure, based on the conditions at Boreholes 4 through 7, includes 50 mm of asphalt overlying sand and gravel (Granular A). The crushed sand and gravel varied in thickness from 250 to 650 mm. Rock fill is present beneath the crushed sand and gravel at Borehole 4. At Boreholes 5, 6 and 7 a brown fine to medium sand exists beneath the base material. The sand contains 10 to 30 percent gravel, and up to 10 percent rockfill. At Borehole 4 beneath the above

noted layer, at a depth of 700 mm, up to 30 percent silt is present in the sand.

Rockfill exists beneath the above noted layers. No further penetration occurred in all boreholes in the rock fill at depths between 0.8 and 2.4 m below roadway surface. The roadway elevation increases to the north and the surface of the roadway is 5 to 7 m above the existing lake level.

Based on all available data the following assumptions with respect to subsurface stratigraphy under the roadway have been made.

- 1) The south bound lane is likely constructed over bedrock since it is adjacent a rockcut and has appeared to be stable.
- 2) The bedrock topography slopes downward into the lake bottom from about the centre of the roadway.
- 3) The construction of the north bound lane involved placing rockfill over the native clays of the lake bottom. A layer of clay may exist between the rockfill and the bedrock surface above lake level.

3.2 Lake Bottom

Soundings of the lake bottom were taken along two profiles (Stations 11+230 and 11+278) from ice level (elevation 328.9 m). The soundings were measured from the shore line east 21 and 18m respectively. The results of the soundings are tabulated below, and are illustrated on the section profiles Enclosure No. 2.

Distance from Shoreline (m)	Offset from Highway Centre Line (m)		Water Depth Below Ice Level (m) (Ice Elevation 328.9 m)	
	Station 11+230	Station 11+278	Station 11+230	Station 11+278
3	17	21	1.0	1.2
6	20	24	2.3	1.2
9	23	27	2.4	1.3
12	26	30	2.5	
15	29	33	2.6	1.4
18		36		1.5
21	35		2.8	

Boreholes 1, 2 and 3 were drilled into the lake bottom. Boreholes 1 and 2 were drilled to refusal on possible bedrock at depths of 8.9 m (EL 320.0 m) and 5.8 m (EL 323.1 m) respectively.

The lake bottom generally consists of clay of intermediate to high plasticity. The consistency of the clay generally varies from soft to firm as indicated by the majority of in situ field vane shear tests varying between 10 and 50 kPa. The sensitivity of the clay ranges from sensitive to extra sensitive. Atterberg limits conducted on samples from Borehole 1 at depths of 3.8, 6.1 and 7.6m depths are shown on Enclosure 8. The results indicate the clay is intermediate to highly plastic with liquid limits ranging from 46 to 112 and plasticity indices from 33 to 79. The natural moisture content of the clay varies from 30 to 105 percent.

At Borehole 1 a thin layer (75 mm) of sand is present on the lake bottom and a layer of gravel exists at the base of the borehole.

At Borehole 3, a sand stratum is present beneath the clay at elevation 319.9 m (9 m). The sand is compact to dense with 'N' values from the standard penetration test of 11 and 31 blows per 0.3m. The borehole was terminated in this layer.

4.0 DESIGN PARAMETERS FOR STABILIZING ANALYSIS

Representative stratigraphy has been interpreted from the borehole data and is presented in the Foundation Investigation Report. The stratigraphy was modelled using the results of field and laboratory analysis and our knowledge of the site.

A back analysis of the existing embankment conditions was carried out to confirm the soil parameters required for design of the remedial measures. Since the area is unstable, by definition the existing configuration has a factor of safety against failure of close to unity. The analysis was carried out utilizing the software Slope/W by Geo-slope International. The Bishop simplified method of analysis was used for rotational type failure while the Janbu method was used to analyse block type failures.

The following soil parameters were used.

Material	Density (kN/m ³)	Angle of Internal Friction (degrees)	Cohesion (C) kPa (undrained case)
Granular Fill	22	35	0
Rock Fill	20	40	0
Clay	18	24	20 - 45
Sand and Gravel	22	30	0

The traffic loading used in the analysis is taken from the Ontario Highway Bridge Design Code.

5.0 DISCUSSION

The section of highway investigated has historically showed evidence of distress and has had patch work completed in the past. The section was resurfaced last year. Prior to this resurfacing MTO personnel noted longitudinal cracking and distortions. The Ministry has indicated that this cracking and distortion was only in the north bound lane. The south bound lane has been relatively stable.

Two options have been reviewed as possible solutions: 1) A flanking berm and 2) Re-alignment. For the flanking berm option, stability analyses have been conducted on various berm configurations to investigate stability improvements to the existing embankment. The increase in the Factor of Safety was compared to our back analysis of the current embankment configuration. For the realignment option, the shoulder rounding of the new roadway should be aligned with the existing centre line. These options are discussed in the following sections.

5.1 Stability Analysis (Flanking Berms)

Stability analyses for the construction of flanking berms have been completed for the investigated area. These were carried out using the Bishop simplified method of analysis for rotational type failures and Janbu method of analysis for lateral block type failure using the software Slope/W by Geo-slope. The critical section for analysis was taken in the vicinity of Station 11+230 where the lake bottom clay was the thickest based on our boreholes. Boreholes could not penetrate through the rockfill beneath the existing roadway. From the boreholes put down in lake and historical data given the assumed stratigraphy seems reasonable.

Based on all available data the following assumptions with respect to subsurface stratigraphy under the roadway have been made.

- 1) The south bound lane is likely constructed over bedrock since it has appeared to be stable.
- 2) The bedrock topography slopes downward into the lake bottom from about the centre of the roadway.
- 3) The construction of the north bound lane involved placing rockfill over the native clays of the lake bottom. A layer of clay was assumed to underlie the existing fill under the north bound shoulder, (although its properties are unknown).

For each berm configuration, two cases were analysed, case one, during construction (undrained strength parameters) and case two, long term conditions (drained parameters). Different lengths and thicknesses of berms were analysed to optimize the design.

The following table indicates the calculated Factors of Safety and increase in stability for different berm configurations analysed.

at the centre line of the existing highway.

6.0 RECOMMENDATIONS

There are two suitable options to rectify the stability problem which can be considered. One option is realignment of the highway to the west such that both lanes are based on shallow rock. The second option is the construction of flanking berms east into the Lake which will provide added stability to the north bound lane. These are discussed below.

6.1 Realignment of the Highway

It is understood from MTO that this option is feasible. It would require blasting of the existing rock face to the west and relocation of utilities. This option assumes the roadway structure for the south bound lane is founded on rock.

For this section of the highway based on the design speed and AADT, we understand the required shoulder width is 2.0 m and the clear zone to the rock face is 4 m from the shoulder or 7.5 m from the new centre line of roadway. With this configuration, the rockface will have to be cut back 3 to 6 m. With the new highway configuration, the existing slope should be flattened to fit between the existing toe and the rounding of the new shoulder. This will reduce destabilizing loadings.

The construction of the new alignment should be in accordance with Ontario Provincial Standard Drawings (OPSD). The following drawings may have relevance to the proposed work.

DRAWING NUMBER (OPSD)	DESCRIPTION
205.02	Transition Treatment, Rock Cut to Rockfill
205.03	Transition Treatment, Rock Cut to Earthfill
205.04	Transition Treatment, Earth Fill to Rockfill
205.05	Transition Treatment, Rock Cut to Earth Cut
205.010	Benching of Earth Slopes
209.01	Widening Granular Base
209.02	Widening Existing Rock Cuts

6.2 Construction of Flanking Berms

The construction methodology must be in accordance with all relevant Ministry guidelines. The construction of the flanking berms would require in-filling the lake. Placing of fill into the lake will require sediment control measures in accordance with OPSS 577. The berms will be placed within the MTO right-of-way but Ministry of Natural Resources and the Department of Fisheries and Oceans must be consulted prior to construction.

The berms should be constructed of rock fill. Some displacement of the lake bottom will occur.

The minimum dimensions for construction of the flanking berm should be 6 m in width and built to an elevation of 329.0 m. A wider berm will increase the factor of safety with less potential future distortion of the highway.

Displacement of the lake bottom is expected to be minimal provided the height of berm does not exceed an elevation of 330.0 m. During construction the height of the leading edge should be restricted to a maximum of one metre above design elevation.

The application of the berm may induce some additional road settlement due to the consolidation of the underlying clays. This is expected to be tolerable, although the amount and rate of settlement should be monitored.

Construction of the berm should not take place by end dumping fill from the existing shoulder. A roadway should be constructed along the toe of the existing slope and berm construction shall advance from the new road.

7.0 CONCLUSION

Both of the options considered will provide solutions to arrest the current instability. The selection of which option to employ should consider costs of construction and environmental impacts of the works.

8.0 LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

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DST CONSULTING ENGINEERING INC.

APPENDIX 'A'

LIMITATIONS OF REPORT

APPENDIX 'A'

LIMITATIONS OF REPORT

The conclusions and recommendations presented in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. It is recommended practice that competent engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

APPENDIX 'B'
METHODS OF INVESTIGATION

DST CONSULTING ENGINEERS INC.

APPENDIX 'B'

METHOD OF INVESTIGATION

Site work for this investigation was carried out in two stages. The boreholes located adjacent to the highway were completed during the fall of 1998. These boreholes were drilled with a CME 55 drill rig equipped with solid flight and hollow stem augers. The boreholes were drilled to auger refusal.

Due to the logistics, the boreholes required in the lake bottom were scheduled to be completed after freeze up. The ice thickness in the study area (less than 300 mm) necessitated the use of portable power equipment to complete the required work. A portable cat head on the tripod system was used to drive the split spoon sampler and 'B' casing to the required depth. Wash boring techniques were used to clean and advance the casing. In situ testing, consisting of standard penetration tests and field vane shear tests, were performed in each of the boreholes at regular interval. In addition "Shelby" tube samples were recovered at selected depths.

Lake bottom soundings at Stations 11+230 and 11+278 were also undertaken. The soundings extended 18 to 21 m east from the shore line. In addition, three cross sections of the roadway were surveyed by DST and are shown on Enclosure 8.

Ground surface elevations were undertaken by DST. The elevations were referenced to MTO benchmark GBM-53-U-576K. The elevation of the benchmark as given by MTO is 331.187 m.

The site work was carried out under the supervision of P. Raynak and A. Finke from DST's Thunder Bay office. Soil samples were returned to our Thunder Bay Laboratory. Classification and index tests were subsequently performed in the laboratory on selected representative samples. Laboratory tests included natural moisture contents and Atterberg limits. Laboratory results are presented in the Borehole Logs where applicable and Enclosure 2 to 5 and 7.