FOUNDATION INVESTIGATION REPORT
GWP 392-98-00
PROPOSED EMBANKMENT WIDENING
HIGHWAY 17 FROM STATION 28+475 TO 28+700
TOWNSHIP OF HEAD

December 19, 2005
TG04221

Prepared For:
JMC Transportation Group
288 St. Paul Street, Suite 106
St. Catharines, Ontario L2R 3M9
Geocres No.: 31K-14

6 Copies	- JMC Transportation Group, St. Catharines, ON
1 Copy	- DST Consulting Engineers Inc., Thunder Bay, ON

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

TABLE OF CONTENTS

APPENDICES

DRAWINGS

ENCLOSURES

1.0 INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by JMC Transportation Group to conduct foundation investigations at nine locations for the proposed embankment widening for the passing lanes of Highway 17 and a culvert extension in the Townships of Head and Rolph. This report summarizes the factual information for one of these sites.

Authorization to proceed with this work was received from JMC Transportation Group. This work was carried out as part of their Total Project Management Contract for the Ministry of Transportation of Ontario (MTO) under G.W.P. 392-98-00.

The project is located on Highway 17, from 6.2 km west of Renfrew County, Road 635, easterly 18.6 km. As part of this project, it is proposed to construct one new eastbound passing lane from Station 28+250 to Station 30+850, Twp of Head, one new westbound passing lane from Station 29+120 Twp of Head to Station 12+250 Twp of Rolph and to extend the Colton Creek culvert. Within the alignment for the passing lanes, Detailed Foundation Investigation is required at nine locations for embankment widening and for a culvert extension. The specified locations for detailed foundation design are as follows:

Township of Head	Station 28+475 to 28+700
Township of Head	Station 29+175 to 29+350
Township of Head	Station 30+025 to 30+150
Township of Head	Station 30+450 to 30+600
Township of Head	Station 30+750 to 31+100
Township of Rolph	Station 10+400 to 10+675
Township of Rolph	Station 11+250 to 11+550
Township of Rolph	Station 11+725 to 11+825

Township of Rolph Station 12+050 to 12+250

At the proposed passing lane locations, the through lanes and the passing lanes will be 3.75 m wide. Shoulder widths at the passing lanes will be 3.0 m on the side of the new passing lane and 2.3 m on the opposite side. The shoulder rounding on the side of the new passing lane will be 1.0 m.

This report addresses the field investigation, laboratory testing program for the proposed widening at the Township of Head, Station 28+475 to 28+700, as defined by the Ministry as the Foundation Investigation Report.

2.0 SITE DESCRIPTION

Along this section of highway under investigation for the proposed widening, the existing embankment varies in height from approximately 1.5 m to 8.0 m within the proposed construction.

The study area is the south portion of highway between limits indicated and abuts two adjacent flooded areas. The embankment is very steep from guardrail to waterfront, and surface runoff seems to be direct from the surface of the highway to the ponded water below. It is not certain if the two flooded areas are natural water courses or resulted from beaver activities. A beaver hut was observed on the east side of the bush area (the land separating the two flooded areas) at approximately 500 m south of the highway. There is a small beaver dam located near Borehole 82. The bush area separating the two ponds is located approximately between Station 28+580 to 28+625. At the time of the field program the depth of water (including ice thickness) varied from 0.4 m to 1.6 m. The ponded water on the east side is at a higher elevation than the west ponded area.

Pictures of the site taken from Station 28+450 and 28+600 are shown below. Both pictures are looking in the easterly direction.

Station 28+450 looking east

Station 28+600 looking east

3.0 INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between November 15 and December 20, 2004.

The boreholes located in the shoulder of the highway (Boreholes 24, 24A and 25) were drilled with a track mounted CME 75 drill equipped for geotechnical drilling and operated by DST. Due to ponded water and access problems, Boreholes 81 to 89 were put down with portable power equipment. A portable cathead mounted on a 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. A total of thirteen boreholes were put down at this site.

Borehole locations and a stratigraphic profile and section are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 24, 24A and 25 are located in the shoulder of the roadway while Boreholes 81 to 89 are located between the toes of the existing and proposed embankments. The boreholes were advanced with hollow stem augers or B casing to refusal which varied from 0.5 to 6.7 m below existing grade.

Soil samples were obtained from the auger flights and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 50 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 300 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. Borehole Logs are presented as Enclosures 1 to 13.

Ground surface elevations at the borehole locations were surveyed by K. Smart Associates Limited.

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 contents and gradation analyses. Laboratory test results are presented on the Borehole Logs and Enclosure 14. Where samples were non-plastic, grainsize analyses were carried out above the 0.075 mm sieve which is adequate for the purposes of soil classification and assessment of frost susceptibility. Other analyses of the finer fractions of non-plastic soils were not carried out given that soil permeability estimates were not required.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Published Engineering Geology

The Quaternary and bedrock geology of the area under investigation as reported by John F. Gartner and P.F. VanDine in the Northern Ontario Engineering Geology Terrain Study 103, Deep River Area (NTS31K/SW) District of Nipissing and County of Renfrew consists of a discontinuous veneer of ground moraine till over the bedrock. During deglaciation, a number of glaciofluvial deposits were formed (Chapman 1975). Precambrian rocks of the Grenville Structural Province underlie the Deep River map area. The oldest and most abundant rocks of the metamorphic complex are metasediments derived largely from siliceous sandstone and sillstones. These metasediments consist of a variety of gneisses and can be found throughout most of the map area.

The mapping associated with the above report indicates that material, landform, topography and drainage generally consists of sand and gravel outwash plain with moderate local relief and dry.

4.2 Field Observations

The generalized stratigraphy of the site based on the borehole locations off the highway consists of ice and water overlying a thin layer of organics that in turn is underlain by sand and/or silt. Borehole 82 was located on top of a beaver dam and there was no ponded water at surface at Boreholes 83, 84 and 85.

The highway fill as identified in Boreholes 24, 24A and 25 consists of sand with varying amounts of gravel and silt with cobbles. Shallow auger refusal of 1.2 to 3.2 m occurred on several attempts between Station 28+496 to 28+551 inferring rockfill below these depths.

4.3 Embankment Fill

The fill in the highway embankment was investigated at Boreholes 24, 24A and 25. Numerous attempts were made to penetrate the fill in the area of Boreholes 24 and 24A from Station 38+496 to Station 28+550.

Auger refusal occurred in each borehole at depths below existing highway grade from 1.2 to 5.6 m. It is believed with reference to the pertinent cross sections supplied by JMC Transportation Group, the highway fills were penetrated at Boreholes 24A and 25. The base of the fill is at Elevation 171.1 and 169.2 respectively.

Gradation analyses on Sample 3 from Borehole 25 indicated 30% silt, 66% sand and 4% gravel. The Standard Penetration Test (SPT) results generally indicate loose to compact state of denseness (N values vary from 5 blows/0.3 m to 18 blows/0.3 m).

4.4 Ponded Water

Ponded water exists at the toe of the embankment at two locations separated by a mixed bush area. There is evidence of beaver activity in the ponded area to the east. The depth of water varied between 0.4 and 1.6 m. At the time of the field investigation, about 200 mm of ice had formed.

4.5 Organics

A thin layer of organics (100 mm) or sand with some organics is present beneath the ponded water or at surface at Boreholes 83 and 84. Borehole 82 was located on a beaver dam which consisted of organics and wood.

4.6 Sand

Beneath the organics, sand exists with varying amounts of gravel and silt. The sand is in a very loose to dense state of denseness, as indicated by the N values from the standard penetration test varying between 3 and 36 blows/0.3 m. Gradation analyses conducted on representative samples (Enclosure 14) indicate 0 to 15% gravel, 67% to 80% sand and 12% to 18% silt.

4.7 Silt

At Borehole 85 a silt layer exists between the sand strata. The silt is in a very loose to compact state of denseness as indicated by N values ranging from 3 to 11 blows/0.3 m from the standard penetration test.

Gradation analysis (Enclosure 14) indicate that the composition of this layer varies with 0% gravel, 19% to 20% sand and 80% to 81% silt. At Borehole 86, this layer contains 37% sand which indicates a sand and silt.

4.8 Groundwater

Ponded water exists over the majority of the site. The depth of surface water is likely controlled by beaver activities.

Groundwater levels may fluctuate seasonally and in response to climatic conditions.

5.0 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.

Prepared by: Reviewed by:

R.F. Crowley, P. Eng.
Sr. Project Engineer

Mike Fabius, P. Eng.
Principal

RFC:dm

APPENDIX 'A'

LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. 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. Conditions can also change with time. It is recommended practice that DST Consulting 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.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

DST CONSULTING ENGINEERS INC.

DRAWINGS

ENCLOSURES

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE