



**FOUNDATION INVESTIGATION REPORT  
GWP 392-98-00  
PROPOSED EMBANKMENT WIDENING  
HIGHWAY 17 FROM STATION 30+750 TO 31+100  
TOWNSHIP OF HEAD**

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TG04221**

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## **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 east bound 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 and laboratory testing program for the proposed widening at the Township of Head, Station 30+750 to 31+100, which includes a culvert extension at Station 30+930, 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 highway embankment varies in height from approximately 1.8 m to 2.9 m within the proposed construction. The embankment will be widened on the north side.

The ditch along the north side of the highway within the limits of investigation generally is grass covered to the east of the culvert and to the west, cattails exist. The site slopes downward from east and west toward the culvert. The culvert is a 3.65 x 1.7 x 22.7 concrete rigid frame culvert. The north tree line at the culvert location is approximately 23 m from centreline of highway. The culvert diverts Colton Creek under the highway, flowing south to north. An abandoned foot bridge exists 12 m north of the culvert.

A picture of the site taken from Station 30+750 looking east is shown below.



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

Site work was carried out between November 11 and December 9, 2004 utilizing a CME 750 and track mounted super 75 drill rig equipped for geotechnical drilling and operated by DST. Fifteen boreholes were put down to depths ranging between 2.7 to 10.2 m. In addition, two dynamic cone penetration tests (DCPT) were conducted in the vicinity of the culvert at Station 30+930.

Borehole locations and a stratigraphic profile and section are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 32, 32A and 34 are located in the shoulder of the roadway while Boreholes 35 to 39 and 41 to 45 are located between the toes of the existing and proposed embankments. Boreholes 33 and 40 and DCPT 1 and 2 were put down at the culvert location to provide soil data for the proposed extension. All boreholes were advanced with hollow stem augers to auger refusal which varied from 2.7 to 10.2 m below existing grade.

Auger refusal occurred in the highway fill at a depth of 1.0 m. Numerous attempts were made in the immediate area but could not penetrate past 1.0 m. The borehole was moved west from Station 30+870 to Station 30+797 where penetration through the fill was achieved.

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 17.

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, gradation analyses and organic content. Laboratory test results are presented on the Borehole Logs and Enclosures 18 to 20.

## **4.0 DESCRIPTION OF SUBSURFACE CONDITIONS**

### **4.1 Published Engineering Geology**

The Quaternary and bedrock 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 siltstones. 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 the soil, landform, topography and drainage generally consist of sand and gravel outwash plain with moderate local relief and dry.

### **4.2 Stratigraphy Overview**

The generalized stratigraphy of the site based on the borehole locations off the highway (Boreholes 35 to 38 and 42) consist of an organic layer overlying sand and silt. At Boreholes 43, 44 and 45, the organic layer consists of topsoil. A discontinuous silty clay seam was identified at Borehole 44 starting at a depth of 0.75 m and extending to a depth of 1.5 m. Auger refusal occurred in all boreholes. The refusal material was not confirmed by diamond drilling techniques, therefore could be boulders or bedrock.

The highway fill, as identified by Boreholes 32, 32A, 33 and 34, consists of granular fill varying between 2.3 and 3.8 m deep. Below the granular fill to auger refusals of 6.8 m to 10.2 m a sand and/or silt exists.



The stratigraphy at the culvert location was identified by Boreholes 33 and 40 and DCPT 1 and 2. Borehole 33 is located on the highway and consists of highway granular fill with trace organics to a depth of 2.3 m which is underlain with sands and silts. Borehole 40 is located approximately 4.5 m beyond the end of the existing culvert and consists of fills comprised of topsoil and sand with organics to a depth of 2.1 m. The fill is underlain by sands and silts.

#### **4.3 Embankment Fill Within the Shoulder**

Fill within the embankment shoulders consists of sand and gravel to silty sand as indicated at Boreholes 32, 32A, 33, and 34. The granular fill comprises the base and subbase materials and varies in thickness from 2.3 to 3.8 m. The base materials vary in thickness from 0.3 to 0.4 m. The bottom of the fill varies between elevations 169.3 m and 170.4 m.

The Standard Penetration Test (SPT) results generally indicate very loose to compact state of denseness (N values vary from 1 blows/0.3 m to 11 blows/0.3 m).

The granular fill generally consists of sand with varying amounts of silt and gravel. At Borehole 33, near the existing culvert, the sand fill contained trace amounts of organics. Rockfill was noted in Borehole 34 at a depth of 1.5 m with occasional cobbles below 3.5 m. At Borehole 32, auger refusal was met within the fill at a depth of 1.0 m.

#### **4.4 Fill At or Beyond Embankment Toe**

Fill was encountered at surface in Boreholes 39, 40 and 41 varying in thickness from 0.6 to 1.0 m. The fill consists of 0.3 to 0.45 m of topsoil overlying sand fill with rockfill and/or organics extending to depths of 1.0 to 3.1 m in thickness.

The Standard Penetration Test (SPT) results generally indicate very loose to loose state of denseness (N values vary from 1 blows/0.3 m to 4 blows/0.3 m).

A significant organic content was identified within the sand fill at Boreholes 39, 40 and 41 (within close proximity of the propose culvert extension). The organic content was measured to be up to 19% with a natural moisture content as high as 160 %.

#### **4.5 Organics**

Only a trace of organics was found (in the split spoon sample from Boreholes 33 and 34) beneath the road fill, at 2.4 m and 3.8 m depth respectively.

Off the highway, a peat/organic layer is present at surface in Boreholes 35 to 38 and below surficial topsoil at Borehole 42. The organic layer varies from Elevation 170.5 to 170.8 m and varies in thickness from 0.8 m to 1.4 m. The organic layer consists of a mixture of root mat, inorganic soil and peat.

Primary and secondary consolidation properties of the organics have been estimated based on published correlations for peat.

Based on published experience with peat soils, a drained angle of internal friction of 30° has been estimated for design purposes. This applies to large deformations which eliminate the effects of fibers.

#### **4.6 Sand and/or Silt**

From beneath the fill and/or organics noted above, interbedded layers of sand and silt exist to auger

refusal varying between 3.8 and 10.2 m below existing grade. Gradation analyses conducted on representative samples retrieved from the field investigation at Boreholes 33 and 34 indicate 0% gravel content, 92% to 93% sand and 7 to 8% silt (Enclosure 18).

The Standard Penetration Test (SPT) results generally indicate a very loose to compact state of denseness (N values vary from 2 blows/0.3 m to 21 blows/0.3 m). At Boreholes 35 and 36 at depths of 3.0 and 4.5 m, N values of 72 and 36 blows for 0.30 m were achieved indicating a dense condition.

#### **4.7 Clay**

A discontinuous silty clay seam was identified at Borehole 44 starting at a depth of 0.75 m and extending to a depth of 1.5 m. Atterberg limits carried out on a sample of the clay from Borehole 44 at a depth of 0.75 m indicates the clay is of low plasticity (Enclosure 20). The natural moisture content of 29% approaches the liquid limit of 30%.

The Standard Penetration Test (SPT) results indicate at least a firm consistency (N value of 5 blows/0.3 m).

#### **4.8 Groundwater**

The groundwater levels taken on completion of drilling are noted on the Borehole Logs, Enclosures 1 to 15. The groundwater level in the off road boreholes noted during our field investigation varied between 0.1 m and 1.5 m below existing grade. The majority of the recordings are within 0.3 m from existing grade. For design purposes, the groundwater level should be taken at the natural ground surface outside the embankment.

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.

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**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.

# **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 (RQD), 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
$u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						