

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
GWP 392-98-00  
PROPOSED EMBANKMENT WIDENING  
HIGHWAY 17 FROM STATION 30+450 TO 30+600  
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-11**

6 Copies  
1 Copy

- JMC Transportation Group, St. Catharines, ON  
- 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



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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 and provide recommendations 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 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 west bound 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 and Design are 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.

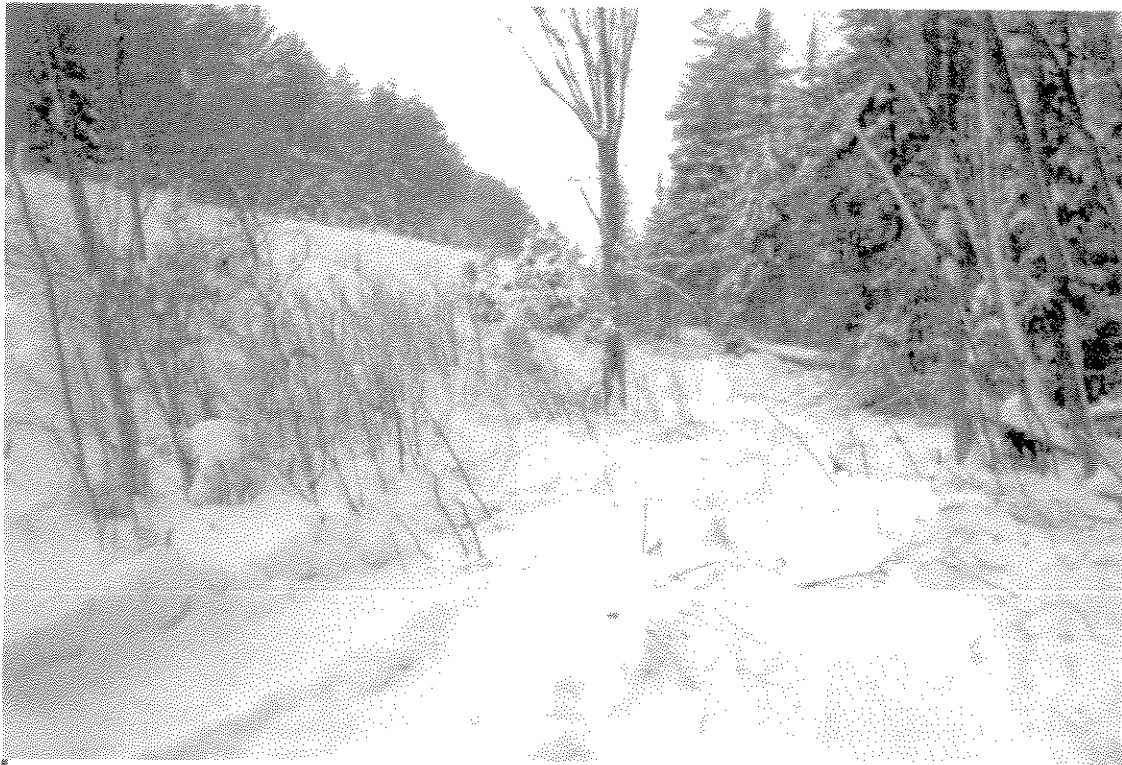
This report addresses the field investigation and laboratory testing program for the proposed widening at the Township of Head, Station 30+450 to 30+600, as defined by the Ministry as the Foundation Investigation and Design Report.

## **2.0 SITE DESCRIPTION**

Along this section of highway under investigation for the proposed widening, the existing highway embankment varies in height from approximately 3 m to 7.8 m within the proposed construction.

On the north side of the embankment, the surface water drainage appears to be to the northwest. The tree line is approximately 14 to 17 m from highway centreline.

A picture of the site taken from Station 30+510 looking west is shown below.



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

Site work was carried out between November 15 and December 22, 2004 utilizing a track mounted 75 drill rig equipped for geotechnical drilling and operated by DST, and an excavator operated by a local contractor. Nine testholes were put down to depths ranging between 0.6 to 6.9 m.

Borehole locations and a stratigraphic profile and section are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 30 and 31 are located in the shoulder of the roadway while Testholes 62 to 68 are located between the toes of the existing and proposed embankments. Boreholes 30 and 31 reached shallow auger refusal of 1.2 and 0.7 m. Numerous attempts were unsuccessful in penetrating the rockfill. Boreholes 65 to 68 were advanced with hollow stem augers to auger refusal which varied from 1.6 to 6.9 m. Testholes 62 to 64 were excavated with a backhoe to depths of 0.6 m to 1.5 m where bedrock was encountered.

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 condition of the soil. Representative soil samples are obtained from within the sampler. Borehole and Testpit Logs are presented as Enclosures 1 to 9.

At the Testpit locations, representative disturbed samples were obtained from the backhoe bucket and/or the walls of the testpit at regular intervals. The in situ characteristics of the soil were estimated using a static cone penetration test (CPT). A cone shaped probe is advanced through the soil and the force required to push the cone is measured with a load cell. At the testpit locations,

hand operated equipment was used which is limited to a maximum penetration resistance of approximately 2000 kPa. This is indicative of a soil with a relative density of compact. The penetration resistance obtained from the field tests were greater than 2000 kPa.

Ground surface elevations and 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 10. 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 Northern Ontario Engineering Geology Terrain Study**

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 consists of sand and gravel outwash plain with moderate local relief and dry.

### **4.2 Field Observations**

Numerous cobbles and boulders were noted on surface throughout the investigated area. Bedrock was visible at surface in the area of Borehole 65.

The generalized stratigraphy based on the off highway testhole locations consist of topsoil overlying sand and silt that in turn is underlain with sand. Refusal to penetration occurred at all locations. Auger refusal in Boreholes 65 to 68 occurred 1.6 m to 6.9 m below existing grade. The refusal material was not confirmed by diamond drilling techniques, therefore could be either bedrock or boulders. At Testpit locations 62 to 64 the bedrock at depth of 0.6 m to 1.5 m was visually confirmed in the field.



### **4.3 Embankment Fill**

Penetration through the highway fills was not achieved. Auger refusal occurred at 1.2 m and 0.7 m in Boreholes 30 and 31. Numerous attempts were made in the vicinity of the boreholes but could not penetrate past 1.2 m. The initial 0.35 to 0.45 m consisted of base material. The subbase material to a depth 1.2 m consisted of a sand with a trace to some gravel, trace silt. Beneath this depth it is assumed rockfill exists.

From the highway cross sections provided, the height of the embankment in the vicinity of the boreholes vary between 6.5 and 7.5 m indicating a possible thickness of rockfill between 5.5 and 6.5 m.

### **4.4 Organics**

A topsoil layer exists at surface at the off highway testholes from EL 178.2 m to 180.7 m varying in thickness from 0.2 m to 0.35 m. Beneath the topsoil a discontinuous layer of clay with some organics was present in Borehole 67 to a depth of 1.2 m (EL 177.5 m). At Borehole 68 a layer of sand with some organics was present to a depth of 0.6 m (EL 178.6 m).

### **4.5 Sand and Silt**

At Testholes 62, 67 and 68 beneath the topsoil, a layer of sand and silt exists. This layer is in a compact to dense state of denseness according to N values from the standard penetration test of 17 to 49 blows per 0.3 m. At Borehole 68 at a depth of 0.75 m, a N value of 6 was obtained indicating a loose state of denseness.

Gradation analysis conducted on representative samples obtained from the field program

(Enclosure 10) indicate 0 to 2% gravel, 45% to 58% sand and 37% to 48% silt.

#### **4.6 Sand**

Beneath the sand and silt or topsoil at Testholes 63, 64 and 65 a sand exists with varying amounts of silt and gravel. Gradation analyses conducted on representative samples (Enclosure 10) indicate 15% to 39% gravel and 6% to 22% fines.

The results of the CPT testing (tip resistance greater than 2000 kPa at the testpit locations) indicate the density of the sand is in at least a compact state of denseness. The N values from the SPT testing generally ranged from 11 to 49 blows per 0.3 m indicating a compact to dense state of denseness.

#### **4.7 Bedrock**

At Testpits 62, 63 and 64 the bedrock was visually confirmed and varies in elevation from 178.2 m to 179.7m.

#### **4.8 Groundwater**

The groundwater levels taken on completion of drilling are noted on the Borehole Logs, Enclosures 1 to 13. The groundwater generally varied between 0.3 and 0.8 m below existing grade. The testpits at locations 63 and 64 were dry.

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

## **5.0 DISCUSSION**

DST Consulting Engineers Inc. (DST) has been retained by JMC Transportation Group to conduct a foundation investigation for proposed westbound passing lane of Highway 17 in the Township of Head between Stations 30+450 and 30+600. It is understood that no significant change in grade is expected. The existing embankment width (shoulder to shoulder) is understood to be 12.1 m. With the addition of a new passing lane and the proposed modifications to the lane and shoulder widths, it is understood that the new embankment width from end of shoulder to end of shoulder will be 16.55 m. With a 1.0 m rounding proposed for the passing lane side of the embankment, the distance from existing centerline to end of rounding on the passing lane side will be 11.5 m ( $3.75 + 3.75 + 3.0 + 1$ ). Along the proposed passing lane side, the existing height of the embankment varies from 3.0 to 7.8 m with side slope grades generally between 1.4 and 2.0 horizontal to 1 vertical.

The subsurface conditions below the existing embankment shoulder consists of up to 1.2 m of granular fills overlying rockfill. In general, the subsurface conditions beyond the existing toe of the embankment were found to consist of up to 0.35 m topsoil overlying up to 6.3 m of sands and silts underlain by bedrock.

Engineering properties for embankment fill have been estimated as follows:

- Unit Weight,  $\gamma = 20 \text{ kN/m}^3$
- Drained Angle of Internal Friction,  $\phi' \geq 35^\circ$
- Drained Cohesion Intercept,  $c' = 0 \text{ kPa}$

Engineering properties for sand and silt have been estimated as follows:

- Unit Weight,  $\gamma = 20 \text{ kN/m}^3$

- Drained Angle of Internal Friction,  $\phi' \geq 32^\circ$
- Drained Cohesion Intercept,  $c' = 0$  kPa

Based on the findings of the field and laboratory programs, preliminary engineering assessment of various construction alternatives were carried out to identify the most feasible construction option. The results of this preliminary assessment have been discussed in Section 5.1, Preliminary Assessment of Construction Alternatives.

As indicated in the Draft Supplemental Pavement Design Report, G.W.P. 392-98-00, carried out for this project by Golder Associates Ltd., it is understood that no distressed areas were identified within this section of the highway.

### **5.1 Preliminary Assessment of Foundation Alternatives**

Based on the findings of the field investigation works and engineering assessment, the following table has been prepared summarizing and comparing various foundation alternatives. Through this exercise of comparing the various foundation alternatives, DST has selected a preferred option.

DST recommends stripping and grubbing of the existing topsoil and vegetation below the proposed embankment widening (Foundation Alternative 1 in the following table). Given there is only a surficial layer of topsoil underlain by compact sand and silt and shallow bedrock found below the proposed widening, settlement and stability is not considered a major concern. As such, conventional construction techniques and methodology should prove adequate. The subsequent sections of this report address this foundation alternative in detail.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
1) Fill placement conventional construction techniques	Lowest cost alternative.	The proposed embankment widening can be constructed from rockfill and/or granular fill. Due to right of way limits, and embankment slopes additional property would need to be purchased if granular fill is utilized. Geotextile may be required between the rockfill and sand and silt material.	Lowest cost for construction. Some additional construction costs associated with geotextile if rockfill is used. If granular fill is used additional property would be required.	Low risk given the competency of the existing subsurface material.
2) Lightweight fills.	Can be used to mitigate/reduce settlement issues.	Not warranted due to the competence of the existing subsurface condition.	Construction utilizing rigid foam type insulation is likely cost prohibitive and <u>not warranted</u> .	Existing slopes and subsurface material are adequate. Black ice formation is a known problem where rigid foam type insulations are used.
3) Preload	Can be used to mitigate long term settlement issues.	Not warranted due to the competency of the existing subsurface condition.	High construction costs and delays, <u>not warranted</u> .	Construction delays.
4) Retained Soil System	Can be useful to permit construction with steeper than conventional slopes.	High costs, <u>not warranted</u> unless procurement of additional property is cost prohibitive where a slope of 2:1 is required for granular fill.	High cost, <u>not warranted</u> .	Existing slopes and subsurface material are adequate.

Note that light weight fills, preload and/or retained soil system are not warranted for this site.

### **5.2 Stability Analysis For Proposed Passing Lane**

The grade of the proposed new passing lane and associated shoulder will approximately match the height of the existing roadway (approximately 3.0 to 7.8 m height) and be 6.75 m in width. The existing side slope is constructed at a slope of 1.4 to 2.0 H:1V utilizing rockfill. At this time it is not known if the proposed new widening will utilize rockfill (if available), or granular fill. As such, recommendations for both construction materials have been provided. Where the embankment material will consist of rockfill, a side slope of 1.25H:1V can be constructed. Where the embankment material will consist of Granular 'B', Type 1 fill in accordance with OPSS 1010, a side slope of 2:1 should be constructed. The embankment fill should be placed in lifts to the required subgrade level and compacted to a minimum 95% of standard Proctor maximum dry density.

The above noted configurations provide a minimum design target factor of safety of at least 1.3 for end of construction and final long term (drained) conditions.

### **5.3 Settlement of Proposed Passing Lane**

Settlements attributed to the native silt and sand below the proposed widening have estimated to be less than 25 mm and are expected to occur during construction.

## **6.0 RECOMMENDATIONS**

A new westbound passing lane is proposed from Stations 30+450 to 30+600 in the Township of Head. The existing highway embankment is approximately 3.0 to 7.8 m in height and is to be widened to facilitate the construction of the new passing lane and shoulder. The existing native soils at the site consist of up to 0.35 m of topsoil overlying up to 6.3 m of sands and silts underlain by bedrock. At two locations, Boreholes 67 and 68, clay or sand with some organics was encountered to depths of 1.2 m and 0.6 m beneath the topsoil.

The additional load of the proposed passing lane does not adversely affect the stability of the existing highway embankment, nor the passing lane itself. The construction of the passing lane can therefore proceed without special considerations for stabilizing the embankment.

Benching of earth slopes should be carried out in accordance with OPSD 208.010, Benching of Earth Slopes (Appendix 'B'). The existing topsoil, clay and/or sand with organics beneath the proposed widening should be excavated to the native silt and sand which will be in the order of 0.2 to 1.2 m deep as indicated at discrete testhole locations.

In the eventuality that water levels rise the native silt and sand subgrade may become wet and easily disturbed through site traffic of any type. Water ingress into the excavation may need to be controlled and granular fill should be placed and compacted immediately following excavation. Where dewatering cannot be accomplished, excavation of the topsoil and fill placement in the wet should be considered. The use of self compacting clear stone (such as 19mm stone) or rockfill should be used below the water level.

Where the embankment widening material will consist of rockfill, a side slope of 1.25H:1V can be

constructed. Voids on the top of the rockfill embankment should be chinked with rock fragments and spalls to form the subgrade as per OPSS 206.07.08, Rock Embankments.

Where the embankment widening material consists of Granular 'B', Type 1 fill in accordance with OPSS 1010, a side slope of 2:1 should be constructed. Projection of the 2:1 slope extends beyond the current right of way (23.0 from centerline). Additional property will be required to construct these slopes at the aforementioned slope; steeper slopes may be possible with the implementation of a retained soil system (RSS). The embankment fill should be placed in lifts to the required subgrade level and compacted to a minimum 95% of standard Proctor maximum dry density. The slope should be initially overbuilt and cut back to the design slope in order to achieve the specified compaction to the slope face. Where granular fill is placed adjacent to existing rockfill, voids at the rockfill surface should be chinked with rock fragments and spikes.

Where rockfill and/or clear stone fill is used below the water level, a non-woven geotextile (OPSS 1860.07.05.02 Class II) should be placed between the native subgrade and fill to prevent the migration of fines into the void spaces of the fill.

## **6.1 Construction**

The construction methodology must be in accordance with all relevant Ministry guidelines. The contractor's methods and equipment must be suitable for the site conditions and materials used.

Equipment and worker traffic over the subgrade soil should be avoided and the first lift of fill should be placed and compacted immediately after excavation to provide a working surface and protection for the subgrade.



## **7.0 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.

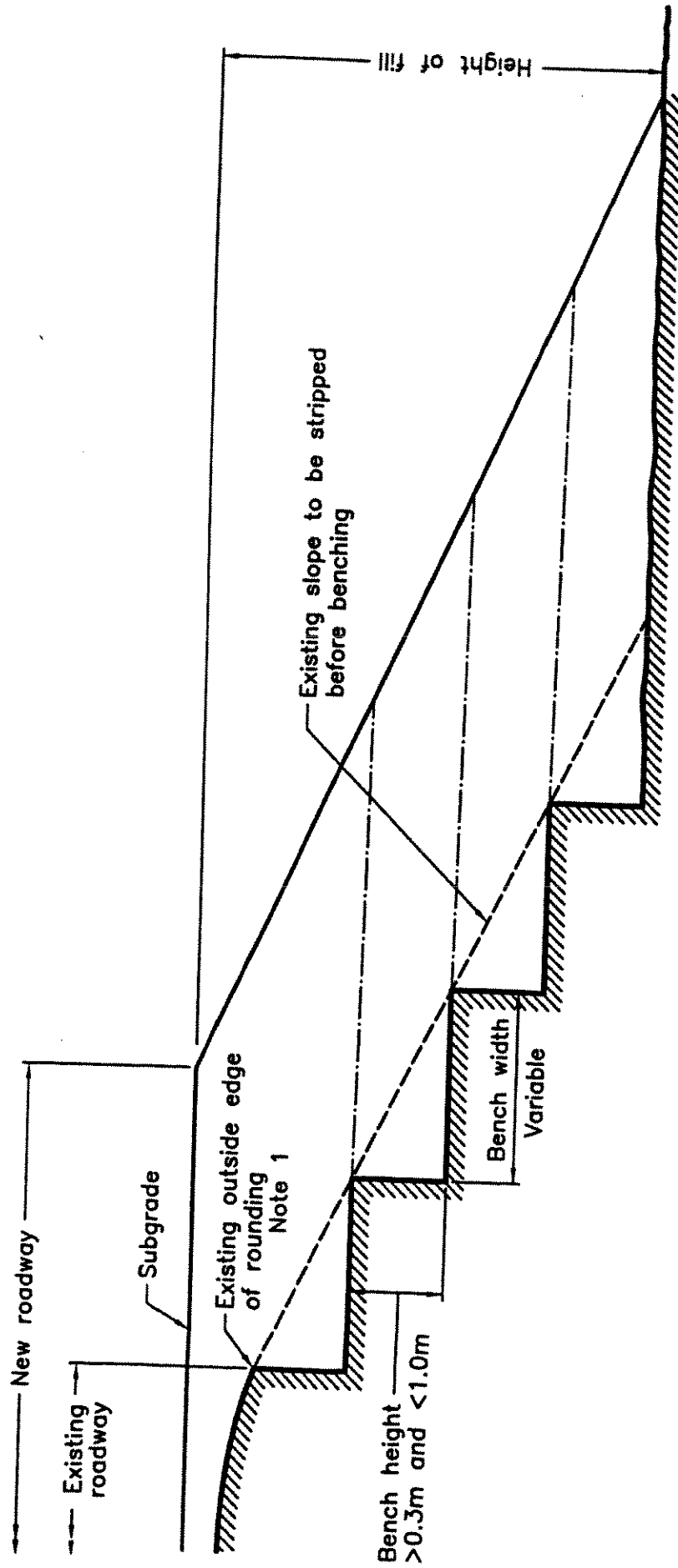
Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

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.

# **A P P E N D I X 'B'**

## **OPSD DRAWING**



**NOTES:**

1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

A Benching is not required on existing slopes flatter than 3H:1V.

B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2003 Rev 1



**BENCHING OF EARTH SLOPES**

OPSD - 208.010

# **DRAWINGS**

DST CONSULTING ENGINEERS INC.

# **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
$C_\alpha$	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
$\sigma'_{v0}$	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
$S_r$	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						