DST CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN 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 and provides 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 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 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, laboratory testing program and design for the proposed widening at the Township of Head, Station 28+475 to 28+700, 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 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 DISCUSSIONS

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 28+475 and 28+700. 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 1.5 to 8.0 m with side slope grades between 1.4 and 2.0 horizontal to 1 vertical.

The subsurface conditions below the existing embankment shoulder consists of up to 5.6 m of granular fills. In general, the subsurface conditions beyond the existing toe of the embankment were found to consist of 0.1 m organics and/or sand with organics overlying sands and silts.

Engineering properties for embankment fill have been estimated as follows:

- Unit Weight, $\gamma = 20$ kN/m^3
- Drained Angle of Internal Friction, Ø' ≥ 35°
- Drained Cohesion Intercept, c' = 0 kPa

Engineering properties for sand have been estimated as follows:

- Unit Weight, $\gamma = 20$ kN/m^3
- Drained Angle of Internal Friction, Ø' ≥ 30°

- Drained Cohesion Intercept, c' = 0 kPa

Engineering properties for silt have been estimated as follows:

- Unit Weight, $\gamma = 20$ kN/m^3
- Drained Angle of Internal Friction, Ø' ≥ 30°
- 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. Assessments for stability and settlement have been carried out for static conditions and are reported in subsequent sections of this report for the preferred option.

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 settlement 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 organics and vegetation below the proposed embankment widening (Foundation Alternative 1 in the following table). Given there is only a surficial layer of organics underlain by loose to compact sand and silt found below the proposed widening, settlement and stability is not considered a major concern. The thickness of organic material at the beaver dam is not known, however it will need to be removed in its entirety. 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. Granular fills may also be used for the widening. 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.	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 not warranted.	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, not warranted.	Construction delays.
4) Retained Soil System	Can be useful to permit construction with steeper than conventional slopes. Applicable where the cost of procuring additional property is more expensive than the retaining system.	High costs, not warranted, for stability.	High costs, however costs should be compared to the cost of acquiring additional property.	Existing slopes and subsurface material are adequate.

Note that lightweight 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 1.5 to 8.0 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 granular fill and possibly 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 (or flatter) 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 (or flatter) can 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 sands 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 28+475 to 28+700 in the Township of Head. The existing highway embankment is approximately 1.5 to 8.0 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 0.1 m of organics/peat overlying sands and silts.

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. The existing organics and deleterious material beneath the proposed widening should be excavated to the native sand which will be in the order of 0.1 m deep as indicated as discrete borehole locations. The thickness of organic material at the beaver dam is not known, however it will need to be removed in its entirety.

Where standing water is not present the native sand subgrade will likely be wet and easily disturbed through site traffic of any type. Water ingress into the excavation will need to be controlled and granular fill should be placed and compacted immediately following excavation. Where standing water is present the water will need to be removed. Removal of the standing water may entail the removal of the beaver dam and installation of temporary ditching. However, if these measures prove ineffective, stripping of the organics 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 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 material consists 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 slope should be initially overbuilt and cut back to the design slope in order to achieve the specified compaction to the slope face.

Where rockfill and/or clear stone fill is used to replace the excavated peat, 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. Given the areas of standing water and the existence of the beaver dam careful planning for removal of organics and placement of fill should be considered to optimally make use of the beaver dam and its eventual removal. Advanced dewatering methods may be required during excavation of the peat to permit excavations in the dry. It may prove more economical to carry out stripping and fill placement in the wet. Where construction in the wet is considered, tight quality control will be required to ensure adequate subexcavation has been achieved.

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.

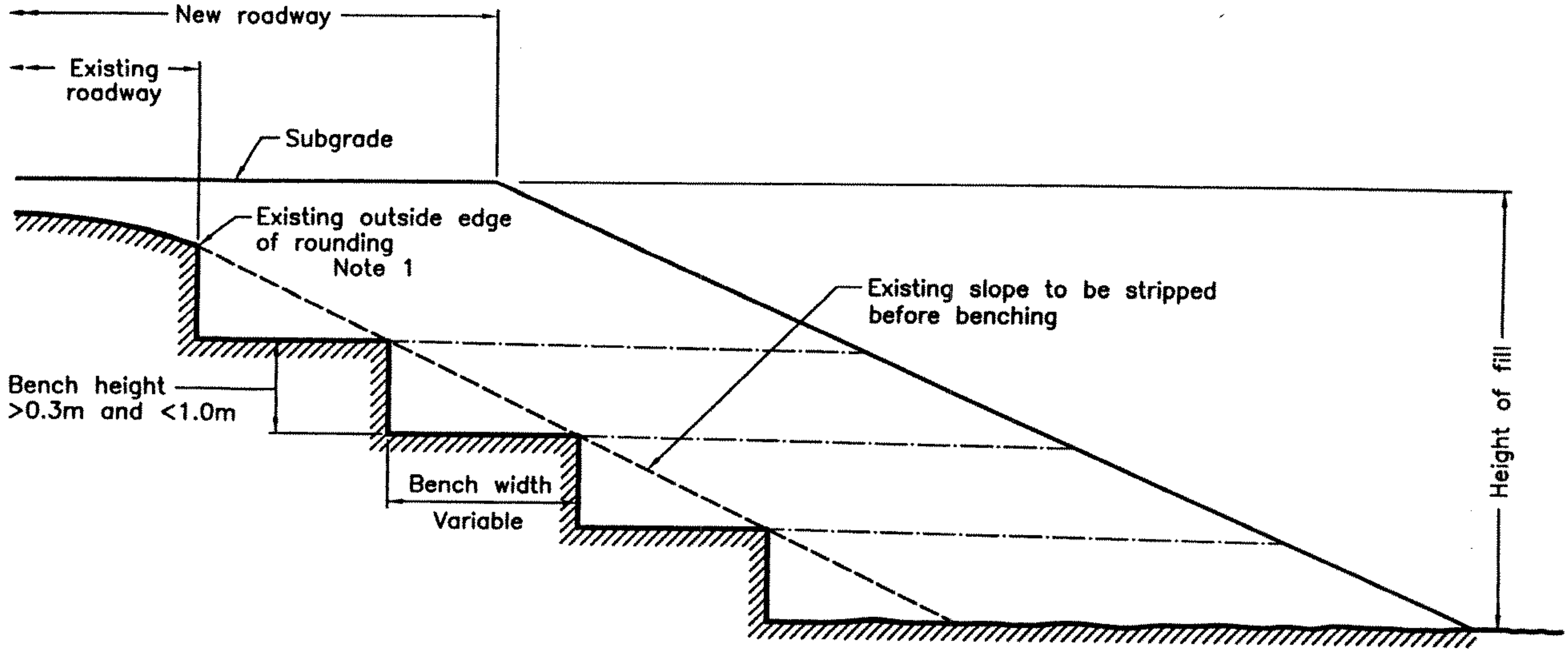
The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil 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.

DST CONSULTING ENGINEERS INC.

APPENDIX '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		

D R A W I N G S

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