

**FOUNDATION INVESTIGATION AND DESIGN REPORT
GWP 392-98-00
PROPOSED EMBANKMENT WIDENING
HIGHWAY 17 FROM STATION 30+025 TO 30+150
TOWNSHIP OF HEAD**

**December 19, 2005
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 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 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, laboratory testing program and design for the proposed widening at the Township of Head, Station 30+025 to 30+150, 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 2 m to 8 m within the proposed construction.

The north side of the embankment alongside the guardrail is very steep at mid-section and gradual at both the west and east ends, and is covered with shot rock. The embankment is tree covered located approximately 4 m north of the guardrail. Generally, the surface runoff from the highway flows down along the embankment slope. It appears that the surface water from the westerly most point of the study area (at the west-most point of the guardrail) flows southeast, and the surface water from the east of the study area (at the east-most point of the guardrail) flows northwest, which would indicate the total surface water converges and flows in a northward direction.

A picture of the site taken from Station 30+150 looking west is shown below.



3.0 INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between November 15 and December 20, 2004 utilizing a CME 750 drill rig equipped for geotechnical drilling and operated by DST, and an excavator operated by a local contractor. Eight testholes were put down to depths ranging between 0.6 to 2.3 m.

Testhole locations and a stratigraphic profile and section are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 28 and 29 are located in the shoulder of the roadway while Testpits 68A to 73 are located between the toes of the existing and proposed embankments. The boreholes were advanced with hollow stem augers to auger refusal at 0.75 m below existing grade. Numerous attempts were unsuccessful in penetrating the rockfill. The testpits were terminated on bedrock at depths varying between 0.6 and 2.3 m below existing grade (elevation 180.5 m to 184.7 m).

Soil samples at the borehole locations were obtained from the auger flights. Borehole Logs are presented as Enclosures 1 and 2. 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 relative density of compact.

Ground surface elevations at the borehole locations were surveyed by DST and referenced to the highway centreline at Station 30+100. The elevation of the centreline was taken as 189.876 m from K Smart Associates Limited's survey of the site.

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 9. 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, 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 testpit locations off the highway consist of topsoil overlying sand and silt. All testpits were terminated on bedrock varying between 0.6 m and 2.3 m below existing grade.

4.3 Embankment Fill

Penetration through the embankment fill was not achieved. Auger refusal occurred at Boreholes 28 and 29 at 0.75 m in rockfill. The base material of sand and gravel is 0.35 to 0.37 m in thickness. From the cross sections of the site the embankment height varies from 2.6 to 8.8 m.

4.4 Organics

A layer of topsoil, 0.2 m in thickness, exists at surface at all testpit locations. The surface elevations of the testpits vary between 181.2 m and 189.8 m.

4.5 Sand and Silt

From beneath the topsoil noted above, a sand and silt exists to the bedrock at between 0.7 and 2.3 m below existing grade. Gradation analyses conducted on representative samples retrieved from the field investigation at Testpits 68A, 70 and 72 indicate 0 to 2% gravel content, 42% to 45% sand and 55% to 58% silt (Enclosure 9). The cone penetration test (CPT) results (greater than 2000 kPa) generally indicate a compact state of denseness.

4.6 Bedrock

At testpit locations the bedrock was visually identified and varies between elevations 180.5 m to 184.7 m.

4.7 Groundwater

All testpits were dry on completion. The groundwater design level should be taken at 1 m below surface.

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+025 and 30+150. 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 2.6 to 8.8 m with side slope grades generally between 1.4 and 2.3 horizontal to 1 vertical. Side slope grades at stations 30+030, and 30+150 are 3.4 and 5.3 horizontal to 1 vertical, outside the norm for this site.

The subsurface conditions below the existing embankment shoulder consists of up to 0.8 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 0.3 m topsoil overlying up to 2.3 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, $c' = 0 \text{ kPa}$

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 should be constructed from rockfill, in order to keep the new embankment materials consistent with the existing embankment materials. However, granular fills may be used for the widening with the existing rockfill chinked prior to placement of the 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.	Unless costly rigid foam type insulation is considered for lightweight fill, settlement reduction will be negligible. Foam type insulation is not recommended below the water level. Some fill will be required above and below the insulation. Not warranted due to the competency 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, <u>not warranted</u> .	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 and rockfill cannot be acquired.	High costs, not warranted.	High cost, however the costs must be compared to the cost of land procurement and availability of rockfill.	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 2.6 to 8.8 m height) and be 6.75 m in width. The existing side slope is constructed at a slope of 1.4 to 2.3 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+025 to 30+150 in the Township of Head. The existing highway embankment is approximately 2.6 to 8.8 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 0.3 m of topsoil/peat overlying up to 2.3 m of sands and silts underlain by bedrock.

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 organics and deleterious material beneath the proposed widening should be excavated to the native silt and sand which will be in the order of 0.2 to 0.3 m deep as indicated at discrete borehole 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 (22.8 from centerline). Additional property will be required to construct these slopes at the aforementioned slope; steeper slopes may be possible with the implementation of an 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.

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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.

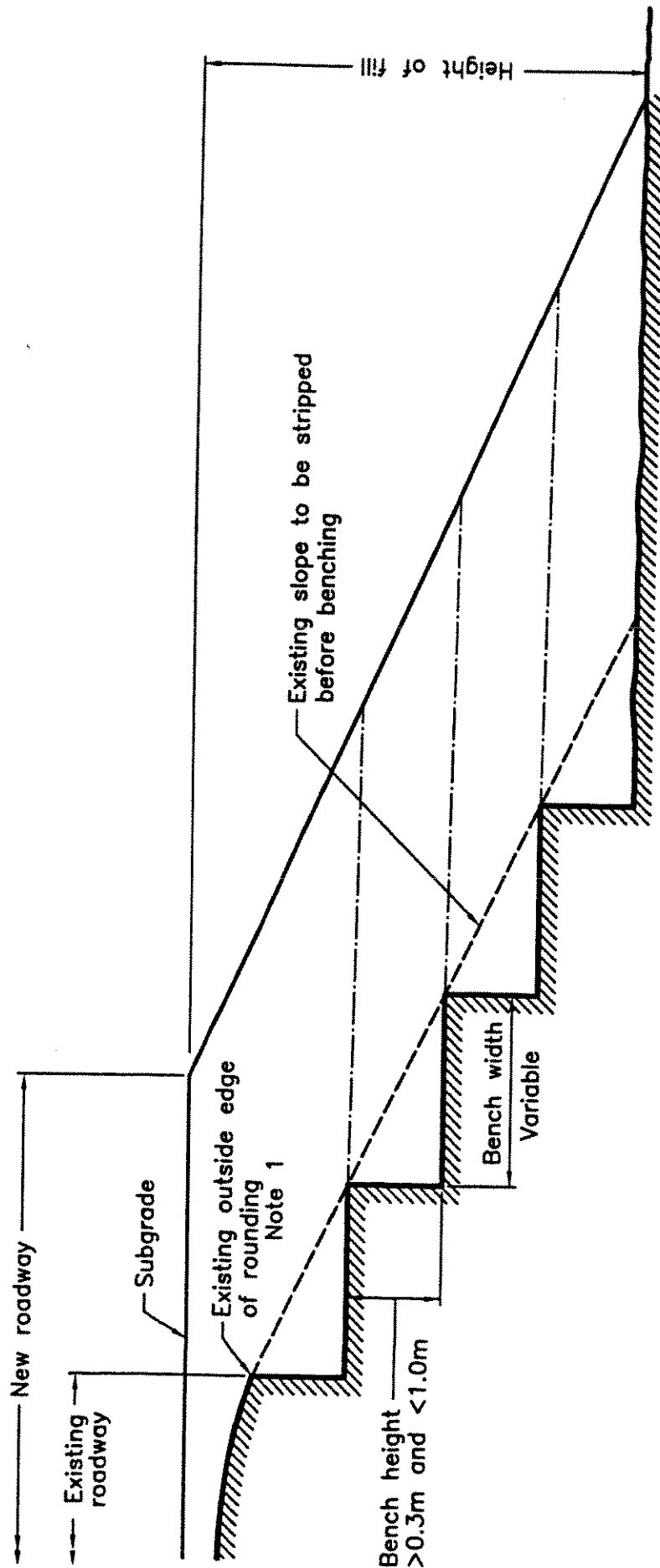
The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage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

A P P E N D I X 'B'
OPSD DRAWINGS



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

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
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_r	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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