

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
GWP 392-98-00  
PROPOSED EMBANKMENT WIDENING  
HIGHWAY 17 FROM STATION 12+050 TO 12+250  
TOWNSHIP OF ROLPH**

**December 20, 2005  
TG04221**

**Prepared For:  
JMC Transportation Group  
288 St. Paul Street, Suite 106  
St. Catharines, Ontario L2R 3M9  
Geocres No.: 31K-9**

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**

1.0 INTRODUCTION .....	1
2.0 SITE DESCRIPTION .....	3
3.0 INVESTIGATION PROCEDURES AND LABORATORY TESTING.....	4
4.0 DESCRIPTION OF SUBSURFACE CONDITIONS .....	6
5.0 DISCUSSION.....	9
6.0 RECOMMENDATIONS.....	17
7.0 LIMITATIONS OF REPORT .....	20

## **APPENDICES**

LIMITATIONS OF REPORT .....	'A'
OPSD DRAWINGS.....	'B'
MODIFIED VON POST CLASSIFICATION SYSTEM.....	'C'

## **DRAWINGS**

BOREHOLE LOCATION PLAN.....	DWG 1
-----------------------------	-------

## **ENCLOSURES**

LOG OF BOREHOLES.....	1 - 10
GRAINSIZE ANALYSIS .....	11

## **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 Rolph, Station 12+050 to 12+250, 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 0.7 m to 1.8 m within the proposed construction.

The existing ditch is approximately 1.7 m in width, 12.8 m from the highway centreline and slopes down to the east. The tree line is about 10 m north of the ditch. The ditch is grass covered with bullrushes. A 760 x 24.24 CSP culvert exists at Station 12+225.

A picture of the site taken from Station 12+250 looking west is shown below.



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

Site work was carried out between November 27 and December 7, 2004 utilizing a track mounted CME 75 drill rig equipped for geotechnical drilling and operated by DST. Ten boreholes were put down to depths ranging between 1.5 to 6.1 m.

Borehole locations and a stratigraphic profile and section are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 55 and 60 are located in the shoulder of the roadway while Boreholes 53, 54 and 55A to 59 and 61 are located between the toes of the existing and proposed embankments. The boreholes were advanced with hollow stem augers to auger refusal which varied from 1.5 to 6.1 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 10.

Ground surface elevations at the borehole locations were surveyed by DST and referenced to the highway centreline at Station 12+050. The elevation of the centreline was taken as 175.534 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 11. 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 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 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 consist of topsoil and/or organics overlying a sand. At Boreholes 57 and 61, no sand was identified beneath the organic. 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 55 and 60, consists of granular fill from 2.8 to 3.5 m in thickness.

### **4.3 Embankment Fill**

The fill in the highway embankment consists of sand and gravel. The sand and gravel comprises



the base and subbase materials. The base material is 300 mm in thickness. The subbase varies in thickness from 2.5 to 3.2 m. The bottom of the fill varies between elevations 172.3 m and 171.8 m.

The Standard Penetration Test (SPT) results generally indicate loose to compact state of denseness (N values vary from 5 blows/0.3 m to 25 blows/0.3 m).

At Borehole 60 the base of the fill from 2.0 m to 3.5 m consists of sand mixed with organics. An organic content conducted on sample 4 indicated an organic content of 3.5%.

At Borehole 53 only, located off the highway, a layer of granular fill exists, 0.6 m in thickness underlying the topsoil.

#### **4.4 Organics**

An organic layer is present at surface in all boreholes located off the Highway varying from Elevation 175.3 to 173.9 m except Borehole 53. The organic layer varies in thickness from 0.4 m to 4.0 m. The organic layer has a variable organic content and consists of a mixture of topsoil, root mat, inorganic soil and peat (and organic soil with <75% organics).

Beneath the highway fill, a 0.8 m thick layer of peat was encountered at Borehole 55. At the other locations drilled through the embankment the peat appears to have been subexcavated.

Field vanes carried out within the peat generally indicate a soft consistency with field vane strengths ranging from 15 to 25 kPa. At Borehole 54, one vane test was recorded in excess of 25 kPa.

#### **4.5 Sand**

From beneath the fill and/or organics noted above, sand exists to the extent of penetration refusal between 2.0 and 6.1 m below existing grade. Gradation analyses conducted on representative samples retrieved from the field investigation at Boreholes 53, 55A, 56 and 58 indicate 0% to 27% gravel content, 50% to 96% sand and 4% to 36% silt (Enclosure 11).

The Standard Penetration Test (SPT) results generally indicate a very loose to very dense state of denseness (N values vary from 3 blows/0.3 m to 121 blows/0.3 m).

At Boreholes 57 and 61 no sand was found below the organics and auger refusal occurred at 1.5 m below existing grade.

#### **4.6 Groundwater**

The groundwater levels taken on completion of drilling are noted on the Borehole Logs, Enclosures 1 to 10. The groundwater level in the off road boreholes noted during our field investigation varied between 0.2 m and 0.4 m below existing grade. The water level recorded in the boreholes drilled in the shoulder of the road varied between 1.4 and 2.0 m below 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 DISCUSSION**

DST Consulting Engineers Inc. (DST) has been retained by JMC Transportation Group to conduct a foundation investigation for proposed west bound passing lane of Highway 17 in the Township of Rolph between Stations 12+050 and 12+250. 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 0.7 to 1.8 m with side slope grades between 1.6 and 6.0 horizontal to 1 vertical.

The subsurface conditions below the existing embankment shoulder consists of granular fills overlying up to 0.8 m of peat or 1.5 m of sand with trace to organics. The organic layer is underlain by native sands. In general, the subsurface conditions beyond the existing toe of the embankment were found to consist of 0.6 to 4.0 m of peat overlying sand.

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,  $c' = 0 \text{ kPa}$

For the normally consolidated peat outside of the existing embankment the following consolidation properties have been estimated based on highest natural moisture content measured of 588%.

Engineering properties for the organic materials have been estimated as follows:

- Void Ratio ( $e$ ) varies from 8 to 11.
- Compression Index ( $C_c$ ) varies from 4 to 6.
- Secondary Compression Index ( $C_\alpha$ ) varies from 0.4 to 0.6.
- Unit Weight ( $\gamma$ ) = varies from 10.0 to 10.5 kN/m<sup>3</sup>.

For the peat below the embankment, the following consolidation properties have been estimated based on the highest natural moisture content of 132%:

- Void Ratio ( $e$ ) varies from 1 to 3.
- Compression Index ( $C_c$ ) varies from 1.0 to 1.5.
- Secondary Compression Index ( $C_\alpha$ ) varies from 0.1 to 0.15.
- Unit Weight ( $\gamma$ ) = varies from 11.0 to 12.5 kN/m<sup>3</sup>

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.

Engineering properties for sand have been estimated as follows:

- Unit Weight,  $\gamma = 20$  kN/m<sup>3</sup>
- Drained Angle of Internal Friction,  $\phi' \geq 28^\circ$
- 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. 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 there is at least one area of settlement within this section of the highway. It has been reported that a 7 m patch extending across both lanes has exhibited about 50 mm of settlement at Station 12+225. Another area of reported longitudinal distortion was identified from Station 12+150 to 12+165 in the eastbound lane with a stepped crack (up to 35 mm). Other pavement distress included; slight to moderate extensive wheel track rutting, extensive moderate longitudinal wheel track cracking, extensive slight to moderate transverse cracking, extensive slight centerline cracking, and a slight waviness in profile.

### **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 subexcavation and replacement of the peat below the proposed embankment widening (Foundation Alternative 2 in the following table). Given relatively shallow, but highly variable peat thickness found below the proposed widening, peat removal is recommended to avoid potential differential and excessive embankment settlements. As this peat thickness is generally less than 2 to 3 m (except at one location where peat was found to a depth of 4 m) costs associated with peat removal are considered to be relatively small given the much improved settlement performance. The subsequent sections of this report address this foundation alternative in detail.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
1) Fill placement without peat excavation	Lowest cost alternative.	Poor settlement performance with highly differential and short and long term settlements given highly variable peat thicknesses. Within the embankment widening, maximum settlements due to primary consolidation of the peat has been estimated at 850 mm, most of which will occur over a period of 1 to 2 months (regrading will be required to account for settlement during this period). Additional settlement due to secondary consolidation of the peat over a 25 year period has been estimated to be in the order of 350 mm. Side slopes flatter than 1.25:1 for rockfill and 2:1 for earth fill would likely be required. Possible peat displacement during fill placement at localized areas; however this risk can be reduced through the use of a geotextile. Typically, staged construction may be required where embankment fills over peat exceed 1.2 m which could add to construction delays.	Lowest cost for construction. Some additional construction costs associated with geotextile and staged construction may be warranted to avoid peat displacement. Highest long term maintenance costs.	High risk of poor settlement performance (both short and long term). Instability and displaced peat will need to be addressed during construction. Zones where peat has been displaced can lead to more severe differential settlements. Risk of higher long term maintenance costs associated with settlements.
2) Fill placement with peat sub-excavation	Short and long term settlement below widening associated with primary and secondary consolidation of the peat is mitigated. Settlements associated with native soils below peat are expected to be negligible. Conventional side slopes of 1.25:1 for rockfill and 2:1 for earth fill will likely prove adequate.	Excavation of the peat will likely be required in the wet with subsequent fill placement. Fill placement below water will require the use of rockfill or 19 mm stone and a geotextile separator.	Costs associated with peat excavation should be considered, but are not expected to be excessive given the peat thickness is less than 2.5 m except for one area where peat found extending to a 4 m depth (as indicated on the test hole logs). It is expected that excavation can be carried out without utilizing specialized equipment such as a drag line. Long term maintenance costs will be reduced given improved settlement performance.	It will be important to ensure all peat is removed especially where excavation in the wet is required. Where zones of peat are left in-place localized areas of differential settlement can be expected. Should peat be found at depths deeper than expected special measures may be required for subexcavation. Potential for instability and/or settlement on existing embankment due to adjacent subexcavation and fill placement.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
3) Lightweight fills without peat excavation	Can be used to mitigate/reduce potential 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 which will lead to primary and secondary consolidation of the peat. For up to 1.0 m of fill placement primary consolidation in the order of 400 mm can be expected with secondary consolidation settlements in the order of 350 mm over 25 years. Regrading will be required during the period of primary consolidation.	Construction utilizing rigid foam type insulation is likely cost prohibitive and <u>not warranted</u> . Some increased long term maintenance cost may be realized to address some differential settlement due to secondary consolidation of the peat.	Black ice formation is a known problem where rigid foam type insulations are used.
4) Preload with or without surcharge or wick drains	Preloading without wickdrains may be considered to reduce/mitigate primary consolidation of the peat.	Preloading would require careful management as fills will need to be imported to restore the grade during preloading. The addition of fill will initiate additional primary consolidation which can extend preload times. Preload durations in excess of 1 to 2 months may be required to effectively eliminate primary consolidation. Secondary consolidations in the order of 350 mm over 25 years would still be expected. A surcharge may be considered to reduce the expected secondary consolidations, however, this would add to the risk of instability of the embankment and would likely require staged construction to place the preload and surcharge adding to construction delays. As such, only light surcharge loads are considered feasible. Wickdrains would not prove economical to accelerate preload duration. Light surcharge loads may be considered.	Additional cost associated with additional regrading required after preloading and for construction delays. Long term maintenance cost will be higher than that of peat removal given some long term settlement can be expected due to secondary consolidation (unless a surcharge is considered). Displacement of the peat may be incurred during construction.	Preload may not prove adequate in eliminating all long term consolidation of the peat which can be highly differential leading to increased maintenance costs.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
5) Retained Soil System	Can be useful to permit construction with steeper than conventional slopes. However, given embankment toes are expected to be within existing ROW, an RSS is <u>not warranted</u> .	High costs, <u>not warranted</u> . Would likely require peat removal to provide necessary foundation capacity and improve settlement performance.	High cost, <u>not warranted</u> .	A retained soil system does not improve settlement performance, <u>not warranted</u> .



## **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 1.2 to 2.6 m height) and be 6.75 m in width. The existing side slope is constructed at a slope of 2.2 to 5.8 H:1V utilizing granular fill. 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. Peat subexcavation is proposed by DST as per OPSD 203.030, or OPSD 203.020 (Appendix 'B') with a side slope of 1.25H:1V for rockfill, or 2.0H:1V for granular fill. The peat subexcavation is to be extended as per Note 1 of these standards.

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.

Under OPSD 203.020, the existing side slope is specified to be temporarily cut at a slope of 1 horizontal to 1 vertical starting at the existing shoulder. This is done to facilitate the removal of more peat possibly located below the existing side slope. It should be noted that sloughing of the 1 horizontal to 1 vertical slope may be experienced during construction. The degree of sloughing will be largely dependant on the duration in which it is left to stand. Backfilling of the slope to design levels should commence immediately after excavation. We recommend rapid excavation and backfilling methods be utilized. The temporary slope should be visually inspected as construction progresses. Should excessive sloughing become problematic, the temporary cut slope angle may be revised to 1.5 horizontal to 1 vertical. However, additional settlements may be incurred below the proposed new shoulder and rounding.

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

Maximum settlements induced by the proposed passing lane fills due to primary and secondary consolidation of any peat remaining below the existing embankment and side slope have been estimated based on two scenarios as follows:

1. During peat subexcavation, the existing side slope is maintained, as per OPSD 203.030.
2. During peat subexcavation, the existing side slope is cut at 1H:1V, as per OPSD 203.020.

The following table indicates the settlements calculated for construction as per OPSD 203.030.

<b>Location</b>	<b>Primary Consolidation of Peat (mm)</b>	<b>Secondary Consolidation of Peat Over 25 yrs. (mm)</b>	<b>Total Settlement After 25 years (mm)</b>
C/L Existing Roadway	< 5	80	80
Existing Shoulder	< 5	80	80
Midpoint Passing Lane	< 5	80	80
Passing Lane Shoulder	800	350	1150

The following table indicates the settlements calculated for construction as per OPSD 203.020.

<b>Location</b>	<b>Primary Consolidation of Peat (mm)</b>	<b>Secondary Consolidation of Peat Over 25 yrs. (mm)</b>	<b>Total Settlement After 25 years (mm)</b>
C/L Existing Roadway	< 5	80	80
Existing Shoulder	< 5	80	80
Midpoint Passing Lane	< 5	80	80
Passing Lane Shoulder	N/A	N/A	N/A

The following table indicates the settlement calculated for construction of existing side slope cut at 1.5H:1V.

<b>Location</b>	<b>Primary Consolidation of Peat (mm)</b>	<b>Secondary Consolidation of Peat Over 25 yrs. (mm)</b>	<b>Total Settlement After 25 years (mm)</b>
C/L Existing Roadway	< 5	80	80
Existing Shoulder	< 5	80	80
Midpoint Passing Lane	< 5	80	80
Passing Lane Shoulder	220	200	420

Estimated settlements due to consolidation of organic soils for the above scenarios have been presented in the following tables. The settlements have been calculated for a passing lane embankment constructed to the same grade as the existing highway shoulder over the thickest deposit of peat found below the proposed embankment widening. Settlements have been calculated at the existing centerline, existing shoulder, approximate midpoint of passing lane, and the approximate shoulder of the new passing lane. It is expected that most of the primary consolidation of the peat will occur over a period of 1 to 2 months. For a 2 month construction period, we expected that at least 90% of the primary consolidation will have occurred (actual degree of primary consolidation during construction but will depend fill placement methods and scheduling). Secondary consolidation has been calculated over a 25 year period.

The settlements along the longitudinal profile of the passing lane will vary and be roughly proportional to the thickness of the peat deposit below the embankment.

Settlements attributed to the native sands below the proposed widening have been estimated to be less than 70 mm and are expected to occur during construction.

## **6.0 RECOMMENDATIONS**

A new west bound passing lane is proposed from Stations 12+050 to 12+250 in the Township of Rolph. The existing highway embankment is approximately 0.7 to 1.8 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 4 m of peat (generally less than 2.5 m) overlying sands.

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. It is recommended the passing lane be constructed in accordance with OPSD 203.020, Embankments Over Swamp, Existing Slope Excavated to 1:1, to avoid potentially excessive and differential settlement along the proposed shoulder (due to consolidation of the peat underling the existing side slope).

It is expected that at least 90% of the primary consolidation settlements within organic soils, as provided in Section 5.3, will occur during construction. The remaining long term settlements will begin during fill placement for the passing lane and will continue at a decreasing rate. As such, some minor long term maintenance of these areas may be required as the new fills continue to settle. This may involve occasional grading/filling of shoulders and patching of asphalt.

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 0.5 to 1.3 m deep as indicated as discrete borehole locations. The width of swamp excavation should include the additional excavation as indicated in Note 1 of OPSD 203.020.

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 dewatering cannot be accomplished, excavation of the peat and fill placement in the wet should be considered. The use of self compacting rockfill or clear stone (such as 19mm stone) 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 current ground water level is at or near the surface of the peat, 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 peat subexcavation 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 and to ensure adequate fill placement to above the water level.

## **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.

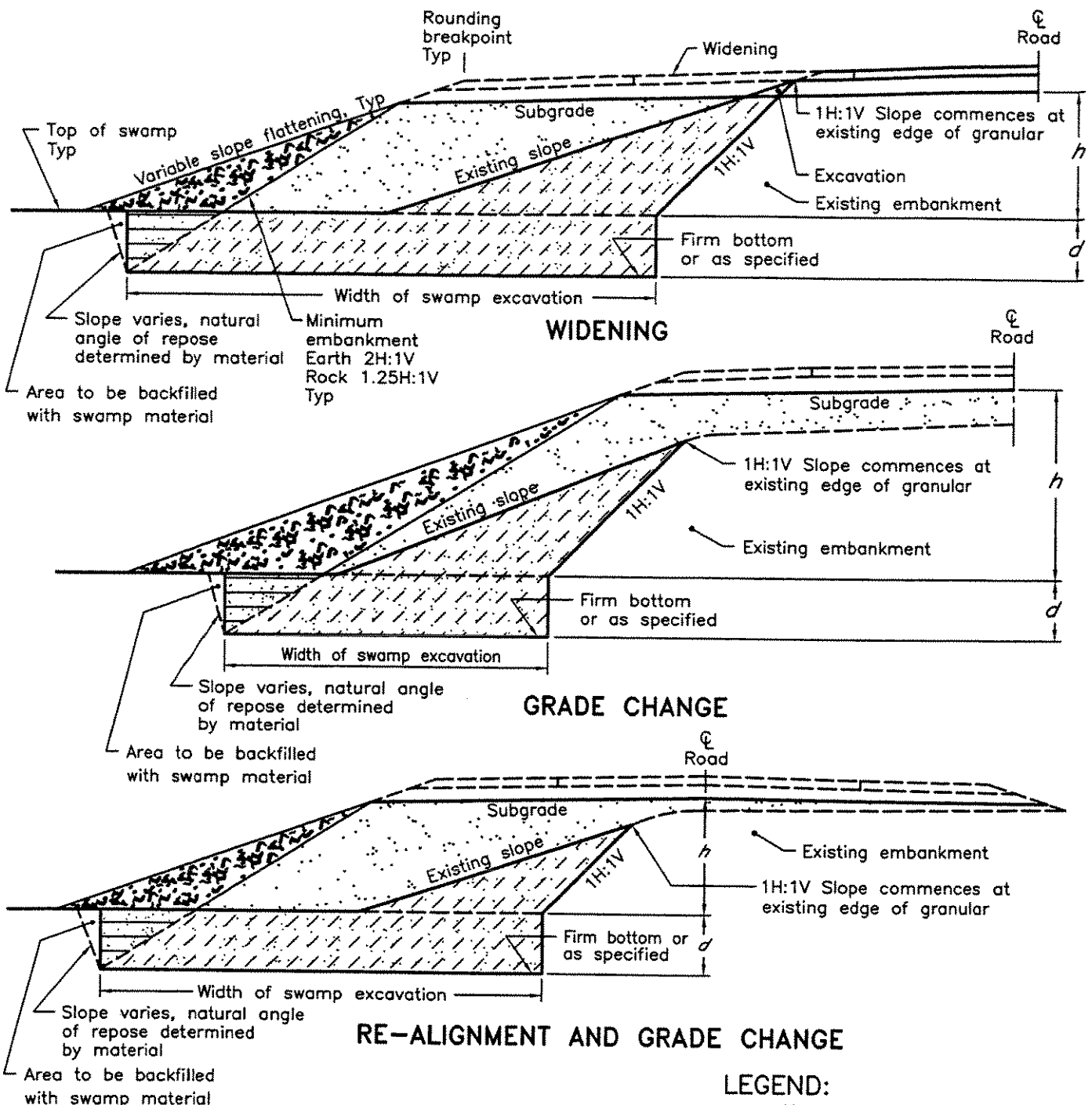
The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage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



#### NOTES:

- For given limits of height  $h \leq 4.5\text{m}$  and depth  $d \leq 6.0\text{m}$ , both requirements shall be met in order to apply.
- Height of fill is the vertical difference between top of subgrade and top of swamp elevation measured at new road centreline.
- Widening of existing earth embankments shall be benched according to OPSD-208.010.
- All dimensions are in millimetres unless otherwise shown.

#### LEGEND:

- $h$  - Height of fill
- $d$  - Depth of sub-excavation
- Embankment materials as specified
- Excavated swamp material
- Excavate and backfill
- Excavate and backfill with swamp material

ONTARIO PROVINCIAL STANDARD DRAWING

## EMBANKMENTS OVER SWAMP

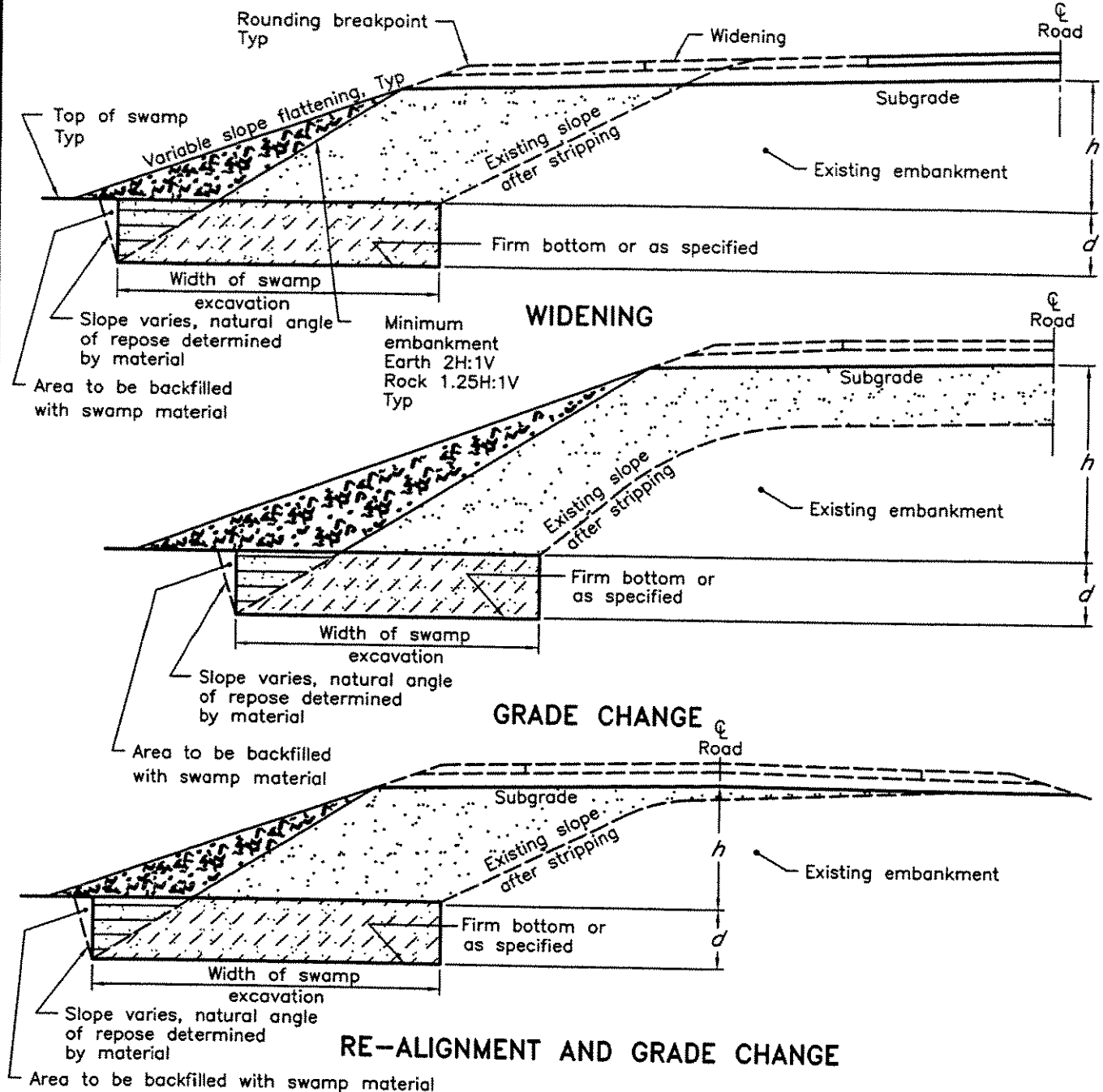
EXISTING SLOPE EXCAVATED TO 1H:1V

Nov 2004

Rev 1



OPSD - 203.020



#### NOTES:

- For given limits of height  $h \leq 4.5\text{m}$  and depth  $d \leq 6.0\text{m}$ , both requirements shall be met in order to apply.
- Topsoil shall be stripped from existing slopes.
- Height of fill is the vertical difference between top of subgrade and top of swamp elevation measured at new road centreline.
- Widening of existing earth embankments shall be benched according to OPSD-208.010.
- All dimensions are in millimetres unless otherwise shown.

#### LEGEND:

- $h$  - Height of fill  
 $d$  - Depth of sub-excavation
- Embankment materials as specified
  - Excavated swamp material
  - Excavate and backfill
  - Excavate and backfill with swamp material

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2004

Rev 1

**EMBANKMENTS OVER SWAMP**

EXISTING SLOPES MAINTAINED



**OPSD - 203.030**

**A P P E N D I X 'C'**

**MODIFIED VON POST CLASSIFICATION SYSTEM**

### **MODIFIED VON POST CLASSIFICATION SYSTEM**

Reference: Landva and Pheeney, Canadian Geotechnical Journal, 1980, Pg. 416.  
Peat is classified using the following categories.

#### **Humification (H):**

The degree of humification is graded on a scale from 1 to 10 and designated H<sub>1</sub> to H<sub>10</sub>. The various degrees of humification are recognized as shown in Table 1.

DEGREE OF HUMIFICATION	DECOMPOSITION	PLANT STRUCTURE	CONTENT OF AMORPHOUS MATERIAL	MATERIAL EXTRUDED ON SQUEEZING (Passing between fingers)	NATURE OF RESIDUE
H <sub>1</sub>	None	Easily identified	None	Clear colourless water	
H <sub>2</sub>	Insignificant	Easily identified	None	Yellowish water	
H <sub>3</sub>	Very slight	Still identifiable	Slight	Brown muddy water, no peat	Not pasty
H <sub>4</sub>	Slight	Not easily identified	Some	Dark brown muddy water, no peat	Somewhat pasty
H <sub>5</sub>	Moderate	Recognizable but vague	Considerable	Muddy water, some peat	Strongly pasty
H <sub>6</sub>	Moderately strong	Indistinct (more distinct after squeezing)	Considerable	About 1/3 peat squeezed out, dark brown water	
H <sub>7</sub>	Strong	Faintly recognizable	High	About 1/2 of peat squeezed out, very dark brown water	
H <sub>8</sub>	Very strong	Very indistinct	High	About 2/3 of peat squeezed out, some pasty water	Mainly roots and fibres
H <sub>9</sub>	Nearly complete	Almost not recognizable		Nearly all peat squeezed out, has a fairly uniform paste	
H <sub>10</sub>	Complete	Not discernable		All peat squeezes between fingers, no free water visible	

#### **Water Contents (B)**

In the field the water content of the peat is on a scale from 1 (dry) to 5 (very high) designated B<sub>1</sub> to B<sub>5</sub>. In terms of actual water contents, by dry weight, these are as follows:

- B<sub>1</sub> dry
- B<sub>2</sub> up to 500%
- B<sub>3</sub> 500 to 1000%
- B<sub>4</sub> 1000 to 2000%
- B<sub>5</sub> greater than 2000%

#### **Fine Fibres (F)**

Fine fibres are defined as fibres and stems smaller than 1 mm in diameter or width. Shrub rootlets may also be included, specified as F (N). No special designation is indicated for plant root hairs as such.

- F<sub>0</sub> nil
- F<sub>1</sub> low content
- F<sub>2</sub> moderate content
- F<sub>3</sub> high content

#### **Coarse Fibres (R)**

Coarse fibres are defined as fibres, stems and rootlets greater than 1 mm in diameter or width.

- R<sub>0</sub> nil
- R<sub>1</sub> low content
- R<sub>2</sub> moderate content
- R<sub>3</sub> high content

#### **Wood (W) and Shrub (N) Remnants**

- W<sub>0</sub> nil
- W<sub>1</sub> low content
- W<sub>2</sub> moderate content
- W<sub>3</sub> high content

# **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 (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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	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
$S_i$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	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

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	e	1. %	VOID RATIO	$e_{min}$	1. %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1. %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	w	1. %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	$e_{max}$	1. %	VOID RATIO IN LOOSEST STATE	j	$\text{kN}/\text{m}^2$	SEEPAGE FORCE
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						