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W.P. No. 112-86-01 (B)

CONT. No. 91-60

W. O. No.

STR. SITE No. 16-45

HWY. No. 15

LOCATION Morton Creek Bridge

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 112-86-01 (B) DIST 8
HWY 15 STR SITE 16-45

CONT 91-60

Morton Creek Bridge Replacement

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DATE MAY 25 1990

FOUNDATION INVESTIGATION REPORT
For
Morton Creek Bridge Replacement
W.P. 112-86-01 (B), Site No. 16-45
Highway 15, District 8, Kingston

INTRODUCTION

This report presents the results of the final foundation investigation carried out for the construction of a replacement bridge structure at Morton Creek, Highway 15, and for the proposed detour construction on the west side of the existing structure.

A preliminary foundation investigation was carried out at this site in 1988 and the results were reported in the foundation report dated 1989 07 14. The data of the preliminary investigation are also incorporated in this report.

SITE DESCRIPTION

The site is located just north of Morton on Highway 15 at Morton Creek, in the Township of South Crosby in the County of Leeds. The site is quite hilly. Rock ridges are all around this site and are as close as 50m to the creek (on the southwest side).

The physiological region is Leeds Knobs and Clay Flats and typically consists of knobs of granite and other Precambrian rocks with areas between filled with very weak calcareous clay left by Champlain Sea (Reference: Champman and Putnam, 'The Physiography of Southern Ontario; 3rd Edition, 1984)

INVESTIGATION PROCEDURES

Earlier in 1988, a preliminary foundation investigation was carried out at this site. The investigation consisted of drilling four sampled boreholes (Boreholes 1,2,3 and 4). The locations of these boreholes are shown on the enclosed drawing 1128601B-A.

In the earlier investigation all boreholes were advanced with hollow stem augers except at Borehole 1 (the deepest borehole), where hollow stem augers were followed by BW casings. The boreholes were advanced to depths ranging from 9.6m (B.H. 4) to 61.4m (B.H. 1) below ground surface. All boreholes were terminated either in the overburden or at presumed bedrock.

Except in Borehole 1, where silty sand was encountered at a depth of 60.7m below the ground surface all boreholes encountered an extensive stratum of clayey silt with frequent silty clay layers and occasional sand and silt seams.

The silty clay to clayey silt was found to be underlying the surficial granular fill and ranged from 8m to 58m in thickness.

The standard penetration N - value generally ranged from 0 to 5 blows per 0.3m. Generally, the N - value was 0 and 1.

The shear strength based on unconfined compression tests ranged from 15 to 50 kPa and based on field vanes it ranged from 12 to more than 96 kPa.

Only in Borehole 1 silty sand was encountered below the silty clay to clayey silt stratum at a depth of 60.7m below ground surface. The borehole was terminated after it penetrated 0.7m in this stratum. A split spoon sample was obtained from the surface of this stratum. The N - value 32 blows per 0.3m suggested that the stratum was dense at the surface.

The final (recent) foundation investigation was carried out between 90 03 12 and 90 03 27 and comprised of drilling five boreholes. Samples from overburden were obtained from four boreholes and one borehole (Borehole 13) was advanced to the bedrock and subsequently bedrock core was obtained at this location.

The boreholes for the final investigation are identified as B.H. 11, 13, 16, 17 and 18.

Borehole 11 was advanced to 61.6m (deepest borehole). Continuous flight hollow stem auger and N-casings were used to advance this borehole. Borehole 13 was advanced by driving 'N' casings to the bedrock without taking samples in the overburden. Further, B-casings were advanced to obtain a 3.3m bedrock core.

Borehole 16, 17 and 18 were advanced by continuous flight hollow stem auger to depths ranging from 7.7m to 16.2m. These boreholes were drilled to obtain information for the analyses of settlement due to detour construction.

Survey details were provided by the Eastern Region Survey and Plans Section. The elevations given in this report are geodetic.

The sampling program consisted of split spoon and thin wall samples (shelby tubes). The Standard Penetration Tests N-values were used for the assessment of the in-situ state of compaction of the non-cohesive material. Information obtained from the Field Vane tests were used to determine the shear strength and sensitivity of the cohesive material. The shear strength was used to determine the consistency of the cohesive material. These samples also provided material for identification purposes.

The laboratory testing program for representative samples consisted of:

- grain size analyses
- natural moisture content determinations
- Atterberg Limit determinations
- unit weight determinations
- consolidation tests, and
- unconfined compression tests.

SUBSURFACE CONDITIONS

The record of Borehole Sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes, along with stratigraphical profiles based on the borehole data are shown on Drawing No. 1128601B-A.

Underlying the fill material or topsoil the soil strata consists of silty clay to clayey silt material which is underlain by silty sand with gravel and occasional boulders. The silty sand layer overlies the bedrock.

On the northwest side of the bridge (B.H. 11) the granular fill was underlain by a 1.8m thick organic soil. Organic material was also encountered in Borehole 1 (preliminary investigation, 1988) which was drilled on the northeast side of the bridge. In the recent (final) investigation continuous samples were obtained to determine the thickness and nature of the organic material.

Underlying the fill material (up to 4m thick) or the surficial topsoil, all boreholes encountered silty clay to clayey silt material. The investigation revealed that the silty clay to clayey silt layer ranges in thickness from 5m to 36m. The thickness of this stratum on the south side of the bridge is about 27m. In the centre of the creek the deposit is estimated to extend 21m below the creek bed and on the immediately northwest side of the bridge it is 36m thick. It should be noted that in the earlier investigation (1988) the borehole on the northeast side of the bridge discovered a 58m thick clayey silt layer. The great variation in stratigraphical surface in a short distance suggests that the subsurface condition at this site is quite variable and that the bedrock is steeply dipping towards the north.

In Borehole 13 (centre of the creek) and Borehole 11 (northwest side of the bridge) a silty sand layer was encountered which was underlying the silty clay to clayey silt stratum. At Borehole 13 (centre of the creek) the silty sand layer was found to be overlying the bedrock. The thickness of this layer at this location was estimated to be 16m. At Borehole 11 the full depth of this stratum was not determined. The borehole penetrated 22m into this material after which further penetration was not possible.

The overburden thickness on the south side of the bridge therefore, is expected to be 32m, in the centre 36m and on the north side of the bridge in excess of 62m.

It has been found out that the bedrock slopes down from the south side of the bridge to the north side and at any point north of the bridge its surface rises up. It is therefore, concluded that the deepest bedrock surface at this site is not in the centre of the bridge but on the north side of the bridge.

Based on the information obtained in Borehole 11 and the assumed soil stratigraphy in Borehole 13 (since no samples were obtained from the overburden) it is expected that the boundary line between cohesive and non-cohesive material slopes to the north (in correlation to the bedrock slope) and therefore the thicknesses of the cohesive and non cohesive material increase from the south side to the north side of the bridge.

Based on the information obtained in Boreholes 16, 17 and 18 it has been determined that the thickness of silty clay to clayey silt layer decreases on the north side of the bridge. This is because the bedrock surface rises up on the north side.

Following are the detailed descriptions of the soil strata encountered.

Fill Material/Topsoil

A non-cohesive fill material was encountered in Boreholes 11, 17 and 18. The fill material was placed for the construction of approaches and shoulders of Hwy 15. The approach fill adjacent to the northwest side of the bridge (B.H. 11) was 2.1m thick and was found to be overlying a layer of organic contained soil. The soil containing organics was about 1.8m thick and consisted of dark brown clayey silt with sand and wood fragments in the upper zone and was almost like a topsoil in the lower zone.

The Standard Penetration test 'N' value of 25 blows/0.3m in the non-cohesive fill suggest that this is in compact state. The N-value in the organic soil (clayey silt with wood fragments and decomposed organic) ranged from 2 to 5 blows/0.3m, which suggest it is in soft to firm state.

Clayey Silt

This cohesive material was encountered in all boreholes. This material was immediately underlying fill material or topsoil. This layer was fully penetrated in Boreholes 11, 13 and 18.

This material is a lacustrine deposit of clayey silt, with frequent silty clay layers and contained frequent sand and silt seams and traces of sand and gravel.

The thickness of this stratum as encountered in the boreholes ranged from 5m to 36m. The stratum was only 5m thick at Borehole 18 which was located at about 100m north of the existing bridge. The overall thickness of this stratum is quite variable. On the south side of the bridge the thickness is expected to be 27m, in the centre of the bridge 21m and on the northwest side of the bridge 36m.

Typical properties of the material, as determined by laboratory tests of representative samples from the boreholes, are summarized as follows:

	<u>Range</u>	<u>Average</u>
Natural Moisture Content (w)	22-45%	30%
Liquid Limit (wP)	18-51%	29%
Plastic Limit (wL)	12-23%	15%
Unit Weight (γ)	17.3-20.4 kN/m ³	19 kN/m ³

Figure 1 illustrates a typical plasticity envelope for this material.

Figure 2 illustrates a typical grain size distribution envelope for this material.

The standard penetration test in this material recorded N-values ranging from 0 to 9 blows/0.3m but generally they ranged from 1 to 2 blows/0.3m. According to the field vane, the undrained shear strength of this material ranged from 21 kPa to more than 100 kPa which suggest that the consistency of the material is soft to very stiff, but generally is firm.

The field vane was used frequently to determine the in-situ undrained shear strength of the soil. Selected samples obtained in the shelby tubes were tested in the laboratory for unconfined shear strength. The strength characteristics of this material are as follows:

	<u>Range</u>	<u>Average</u>
Shear Strength (unconfined compression test)	15-50 kPa	32 kPa
Shear Strength (based on Field Vane)	12-100+ kPa	49 kPa

Consolidation tests were carried out in the laboratory on selected samples. The results are as follows.

	<u>Range</u>	<u>Average</u>
Initial Void Ratio (e ₀)	0.769-1.18	0.96
Preconsolidation (P _c)	111-320	229 kPa
Compression Index (C _c)	0.18-0.66	0.43

The consolidation tests carried out on selected samples show that the deposit under the existing Hwy 15 alignment and along the proposed detour alignment is over consolidated.

The results of the consolidation tests are shown on Figures 3 through 7.

Silty Sand

This non-cohesive stratum underlies the silty clay to clayey silt stratum. This layer was encountered in Boreholes 11 and 13.

In Borehole 11 this material was encountered at 40.1m below the ground surface (road level). In Borehole 13 no sample was obtained from this deposit. However, based on the penetration record of the N-casings it is assumed that the sand layer was encountered at 23m below the water level in the creek.

The thickness of this layer in the centre of the bridge is estimated to be 16 m. At Borehole 11 the full depth of this stratum was not determined although the borehole was advanced 22m in this material.

The silty sand stratum contained gravel and occasional boulders. Based on the split spoon tests in Borehole 11 the silty sand layer is in compact to very dense state. The compactness increases with increasing depth.

Due to difficulty in the drilling operation the split spoon was advanced with a 63.5kg hammer (in the same way used for standard penetration test) from depths 56m to 62m below the ground surface to estimate the density of this stratum. It is determined that N-values in excess of 100 blows/0.3m could be obtained in this stratum at depths 60m or more below the existing ground surface.

Bedrock

Bedrock was encountered at Borehole 13 at a depth of 38.7m below the water level in the creek. Bedrock was proved by coring from depths 38.7m to 42.0m below the water level in the creek.

The bedrock is described as Marble of Late Precambrian Age.

As stated in our preliminary report the bedrock elevations at this site are quite variable.

It has been determined that the bedrock surface slopes down from south to north at this site. On the north side of the bridge the borehole was terminated within the overburden at a depth of 62m. Therefore, the bedrock at that location is expected to be at a depth in excess of 62m below the ground surface.

The bedrock surface on the south side of the bridge is about 32m below the ground surface. In the centre of the bridge the bedrock is expected to be encountered at 36m below the creek bottom and on the north side of the bridge the bedrock is expected to be at a depth in excess of 62m below the ground surface.

Groundwater

The groundwater was measured in open boreholes and also in a piezometer installed in Borehole 16. The groundwater in Borehole 11 and the piezometer had stabilized and was found to be matching with the water level in the creek (Elev. 92.0m). Water levels in Boreholes 17 and 18 were not stabilized at the time of measurement.

It should be noted that groundwater levels are subject to seasonal fluctuations and may therefore change as the water level in the creek changes.

DISCUSSION

General

It is proposed to replace the existing bridge at Morton Creek on Highway 15 with a new bridge. The new bridge will be a 3 span structure and will be of the same length as the existing bridge. The alignment of the new structure will be the same as the existing and no modification in the existing grade will be made.

A preliminary foundation investigation was carried out at this site in 1988. Based on the earlier information available, it had been proposed to construct the new structure on friction piles. The purpose of the final investigation was to determine if the proposed structure could be supported on end bearing piles in order to facilitate construction at the piers by utilizing pile bents and thus minimizing dewatering requirements.

During construction of the new bridge the traffic will be re-routed through a detour which will be constructed on the west side of the existing Highway 15. Near the centre of the bridge, the centreline of the proposed detour will be 13.5m west of the centreline of the existing Highway 15.

The bailey bridge for the detour will be founded on timber friction piles.

The existing bridge structure is performing well except for minor deterioration. The foundation appears to be performing well as no settlement or distruction to the bridge took place.

As described in the previous sections, the subsurface conditions at this site are quite variable.

The subsurface investigation has revealed that the bedrock slopes down from the south side of the bridge to the north side of the bridge and then at any point north of the bridge the bedrock surface rises up. This is evident from the bedrock outcrop at a distance of about 120m north of the bridge.

It is estimated that the bedrock surface slopes down from the south side of the bridge to the north at an angle of about 30 degrees.

The bedrock surface on the south side of the bridge is about 32m below the ground surface, in the centre of the bridge 36m below the creek bottom and on the north side of the bridge the bedrock is expected to be at depth in excess of 62m below the ground surface.

RECOMMENDATIONS

Structure Foundations

Alternative No. 1

As recommended in our preliminary report dated 89 07 14 the proposed structure can be supported on friction piles.

It is understood it is proposed to construct the bailey bridge on friction piles.

For the purposes of the O.H.B.D.C., and assuming an embedded length of 15m the following values are recommended for No. 36 timber piles.

Factored Axial Capacity at U.L.S. = 265 kN/pile (conditional)**
Axial Capacity at S.L.S. Type II = 175 kN/pile (conditional)**

If consideration is given to friction piles it would be prudent if the entire structure were to be supported on friction piles, unless the bridge can accomodate differential movements. If the structure is partially supported by end bearing piles differential settlements may result.

A minimum of 1.5m earth cover should be provided on the pile caps for frost protection.

The piles should be treated with preservatives as per MTO Standards.

Alternative No. 2

The structure may be supported by end-bearing steel H-piles driven to bedrock or very dense material. The following are recommended design values, as per the O.H.B.D.C.

For Steel Piles HP 310 X 79:

Factored Axial Capacity at ULS = 1150 kN/pile (conditional)**
Axial Capacity at SLS Type II = 825 kN/pile (conditional)**
Ultimate Pile Capacity (for Hiley Formula) = 2475 kN/pile

For Steel Piles HP 310 X 110:

Factored Axial Capacity at ULS = 1600 kN/pile (conditional)**
Axial Capacity at SLS Type II = 1150 kN/pile (conditional)**
Ultimate Pile Capacity (for Hiley Formula) = 3450 kN/pile

TABLE 1

Foundation Location	Assumed Cut off Elevation (m)	Estimated Tip Elevations (m)	Approximate Founding Depths below Ground Surface (m)
North Abutment	93	30	63
North Pier	93	42	51
South Pier	93	52	41
South Abutment	93	59	34

The piles should be driven to bedrock. However, in some cases refusal may be encountered above bedrock. In those cases Hiley Dynamic Pile Driving Formula as per MTO Standards SS 103-10 or SS 103-11 should be used to determine the final founding elevations.

The piles should be advanced with a driving hammer capable of developing a minimum energy of 50,000 J per blow. The pile tip elevations and depths are provided in Table 1 for estimation purposes only.

It is estimated that the bedrock surface slopes down from south to north at an angle of about 30 degrees. It is therefore, recommended that the H-piles should be fitted with Rock Points.

Driving of pile equipped with Rock Points will be as specified in Section 903.07.02.05 (Steel H Piles with Rock Points) of Ontario Provincial Standard Specification for Construction.

**

Due to the construction of the detour on the west side of the existing bridge some settlement will take place which will reduce the bearing capacity of the piles due to down drag. It is therefore, recommended that either the fill for the detour should be placed atleast three months before the construction of the new bridge or the bearing capacity as shown above (for friction and end bearing piles) should be reduced by 10 percent.

Lateral Resistance

The horizontal component of battered piles may be used for the calculation of resistance to lateral forces. The lateral resistance of vertical piles can be assumed to be negligible due to the weak nature of the upper soils.

Dewatering

Dewatering will be required for the construction of pile caps below the prevailing groundwater elevation. Due to relatively impervious nature of the silty clay to clayey silt soil, seepage from the silt and sand seams can be removed by ordinary sump pumping techniques. The proposed construction may also require temporary diversion of the creek to facilitate construction of the pile caps.

If steel H-piles are used, consideration should be given to constructing pile bents instead of pile caps. This will minimize requirement for dewatering. Concrete filled steel tubes should be incorporated into the design to enhance rigidity and offer protection against damage. These tubes should extend from underside of the deck to below frost level or below scour level whichever is deeper.

Lateral pressure on Abutments

Backfill to abutments should consists of Granular 'A' or Granular 'B' material for which the following properties are recommended.

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ$	$K_a = 0.27$	$K_o = 0.43$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$	$\phi = 30^\circ$	$K_a = 0.33$	$K_o = 0.50$

Lateral pressure should be computed in accordance with Section 6.6.1.2.1 of the O.H.B.D.C. From a geotechnical perspective an yielding foundation condition may be assumed and hence the active condition will govern the design. If the structure cannot tolerate movement then the at rest condition will govern the design.

Stability of Slopes and Settlement

Since there will be no change in the grade of existing Hwy 15, no deep seated stability problems are anticipated.

For the construction of the detour, there would be no stability problem for fills up to 2m high.

Total stress analyses have been carried out assuming that a 2m thick granular fill will be placed for the detour construction and 4m thick fill will be placed temporarily along the existing highway alignment to minimize the long term settlement. The results show that there will be no stability problem. The results are shown on the attached Figures 8 & 9.

All slopes should be protected against surficial erosion by establishing vegetation covers.

It has been reported that significant settlement took place at the north approach. Our recent subsurface investigation has revealed that, underlying the granular approach fill, there is about 1.8m of organic soil. It is expected that majority of the reported settlement took place because of the consolidation of this deposit.

It is expected that most of the long term settlement would have already taken place at the existing approaches. However, after the detour is constructed it is recommended that the organic material be removed before placing granular material for the reconstruction of the north approach. It is estimated that the thickness of organic soil ranges from 0.3m to 1.8m and extends about 10m towards north from the north abutment. However, exact amount will have to be determined at the time of topsoil removal. If the scheduling permits it is desirable that the north approach may be loaded with a surcharge load (2m of fill material) and left for at least three months to accelerate consolidation and minimize long term settlement.

It is estimated that settlement up to 15 to 20 cm will take place along the detour line. It is predicted that 90 percent of this settlement will occur within 12 years. At least 50 percent of this total settlement is expected however, to occur within a period of 2.5 years. Such settlement would not affect the integrity of the new structure. However, if the fill for the proposed detour is not placed at least three months prior to the construction of new bridge, it will reduce the bearing capacity of the piles as has been stated in earlier sections.

Construction Concerns

It is understood that the new piers and abutments will be more or less at the same location as the existing ones. Therefore, driving of new piles will be difficult due to the presence of existing piles.

This problem may be solved by locating the new footings away from the existing one. Consideration could be given to a two span bridge rather than a three span or by selecting new locations for all piers and abutments.

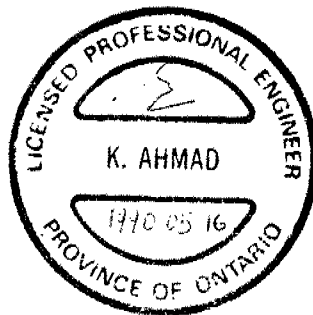
The non-cohesive layer underlying the cohesive layer contains occasional boulders. It is possible that such boulders may be encountered during pile driving and may cause difficulties in pile driving. This possibility should be noted in the contract.

MISCELLANEOUS

The field work for this project was carried out under the supervision of Ken Ahmad.

The equipment used was owned and operated by Marathon Drilling Co. Ltd.

The report was written by Ken Ahmad, Foundation Engineer, reviewed by D. Dundas, Senior Foundation Engineer and approved by M. Devata, Chief Foundation Engineer.



A handwritten signature in cursive script, reading "Ken Ahmad".

Ken Ahmad, P. Eng.
Foundation Engineer

A handwritten signature in cursive script, reading "M. S. Devata".

M. S. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

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

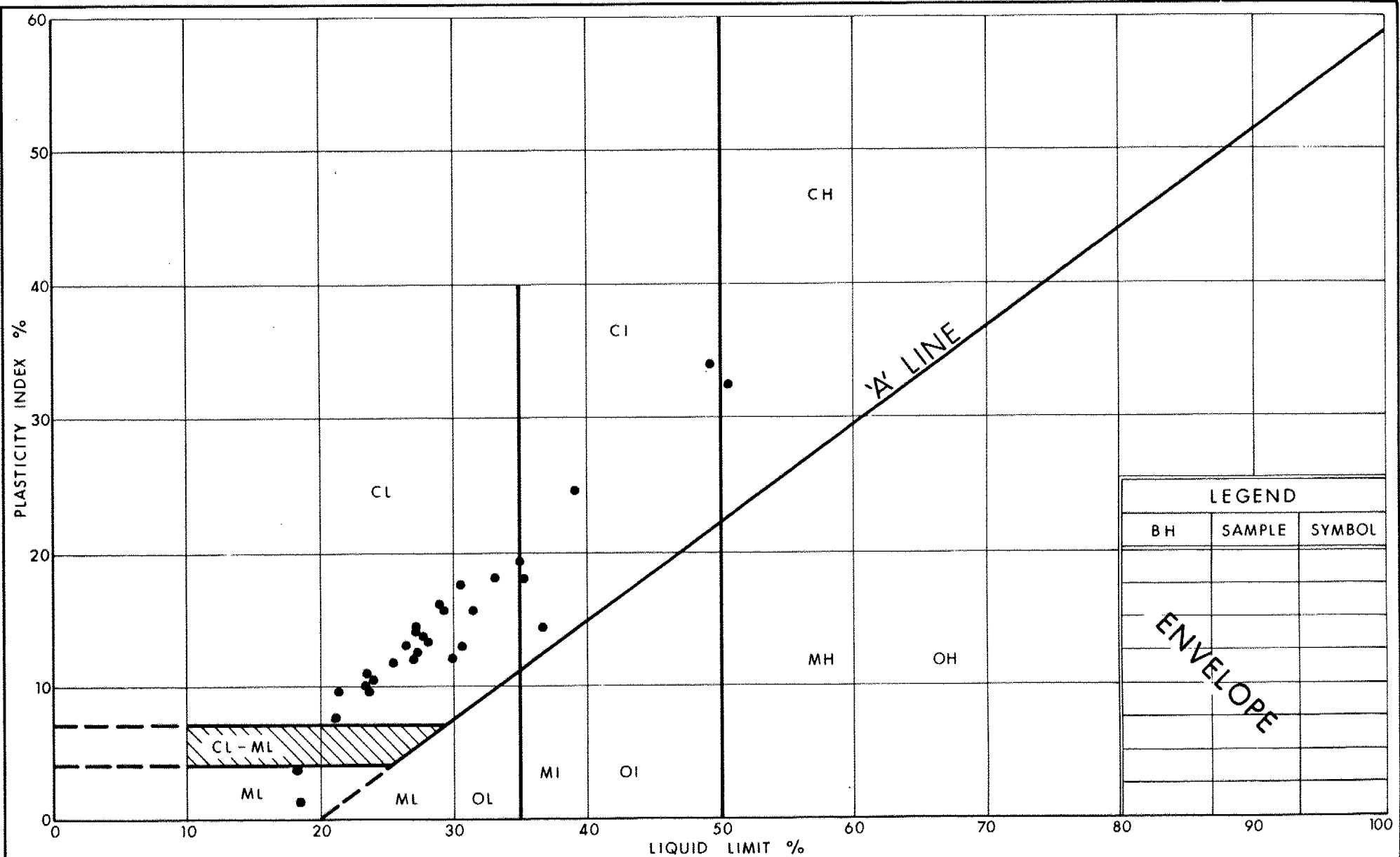
m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
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 = $\frac{c_u}{\tau_r}$

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

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 = $w_L - w_p$	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^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						



Ontario

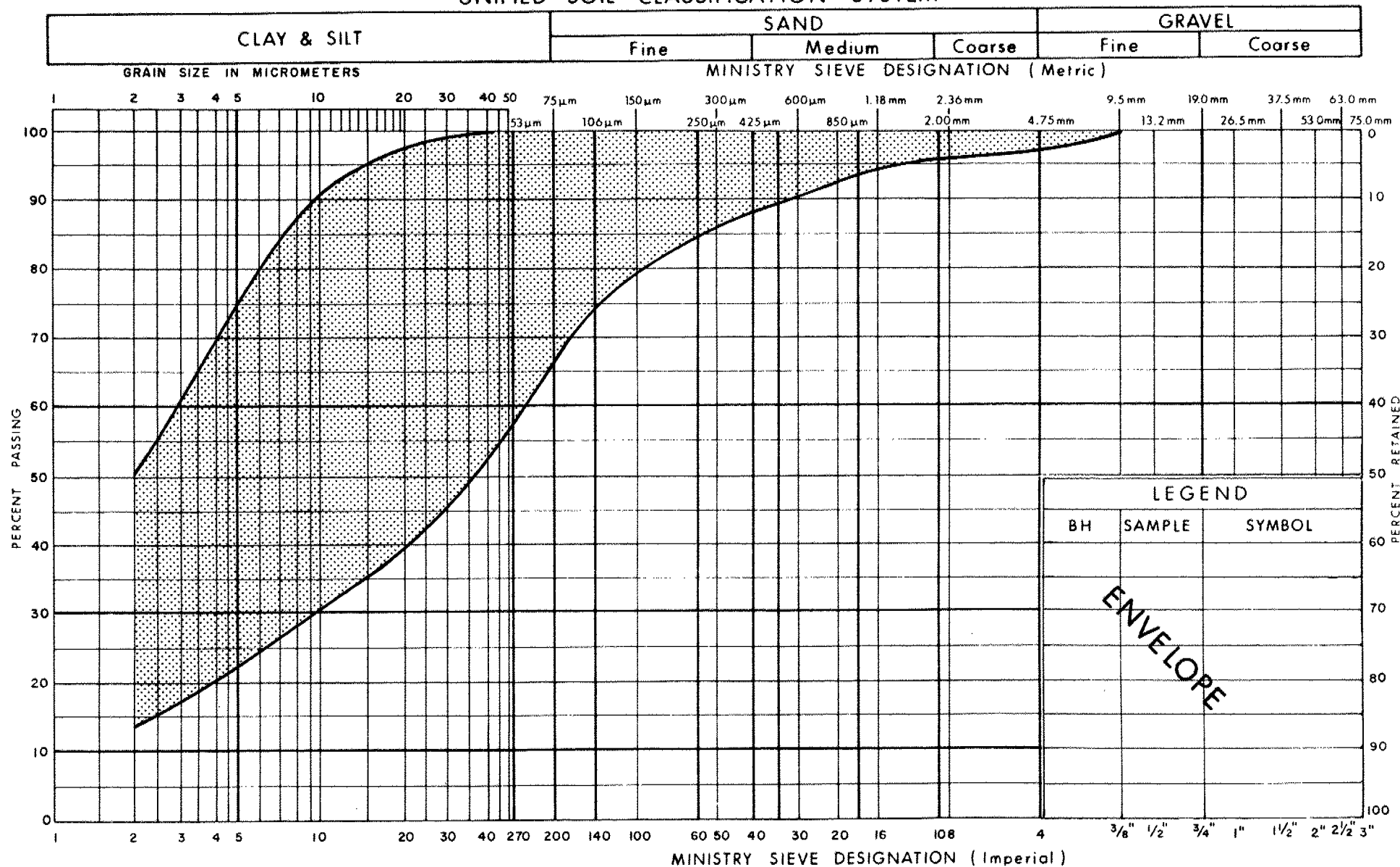
Ministry of
Transportation

PLASTICITY CHART CLAYEY SILT, WITH FREQUENT LAYERS OF SILTY CLAY AND OCCASIONAL SILT SEAMS

FIG No 1

W P 112-86-01(B)

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
CLAYEY SILT, WITH FREQUENT LAYERS OF SILTY CLAY
OCCASIONAL SAND AND SILT SEAMS

FIG No 2

W P 112-86-01(B)

VOID RATIO - PRESSURE CURVES

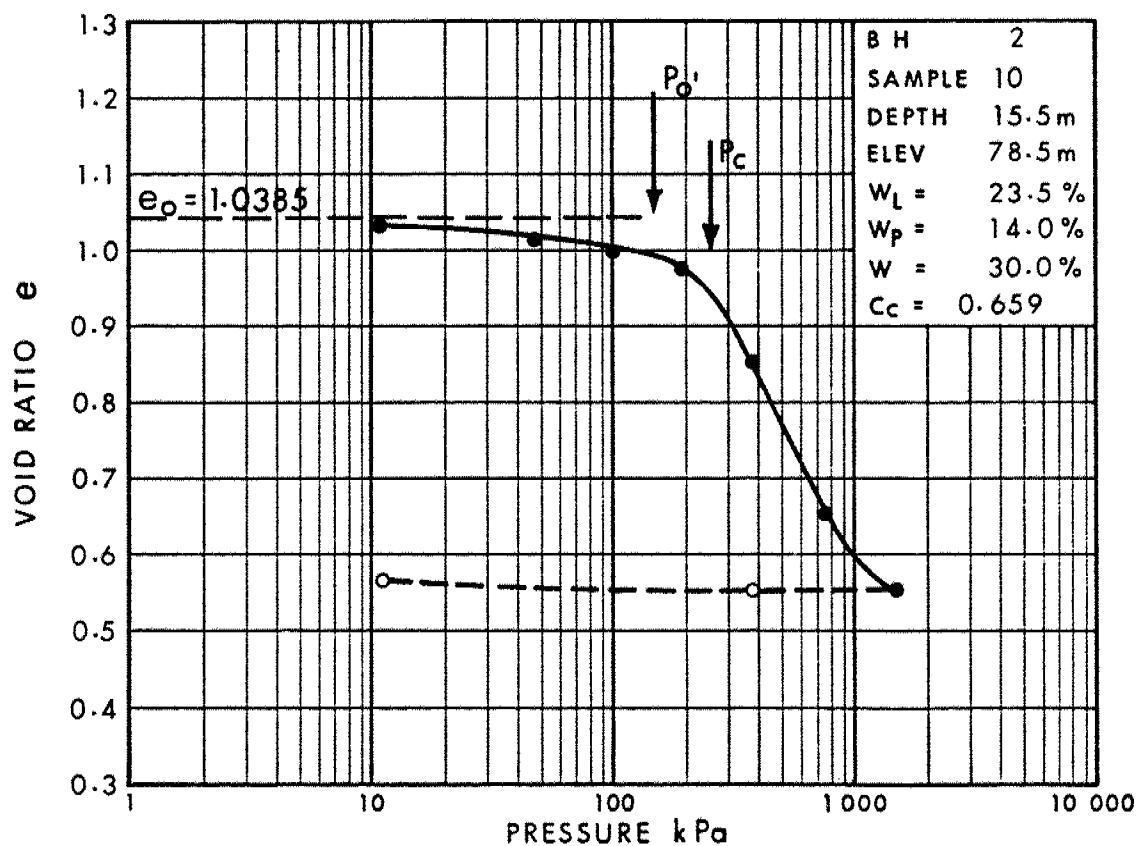
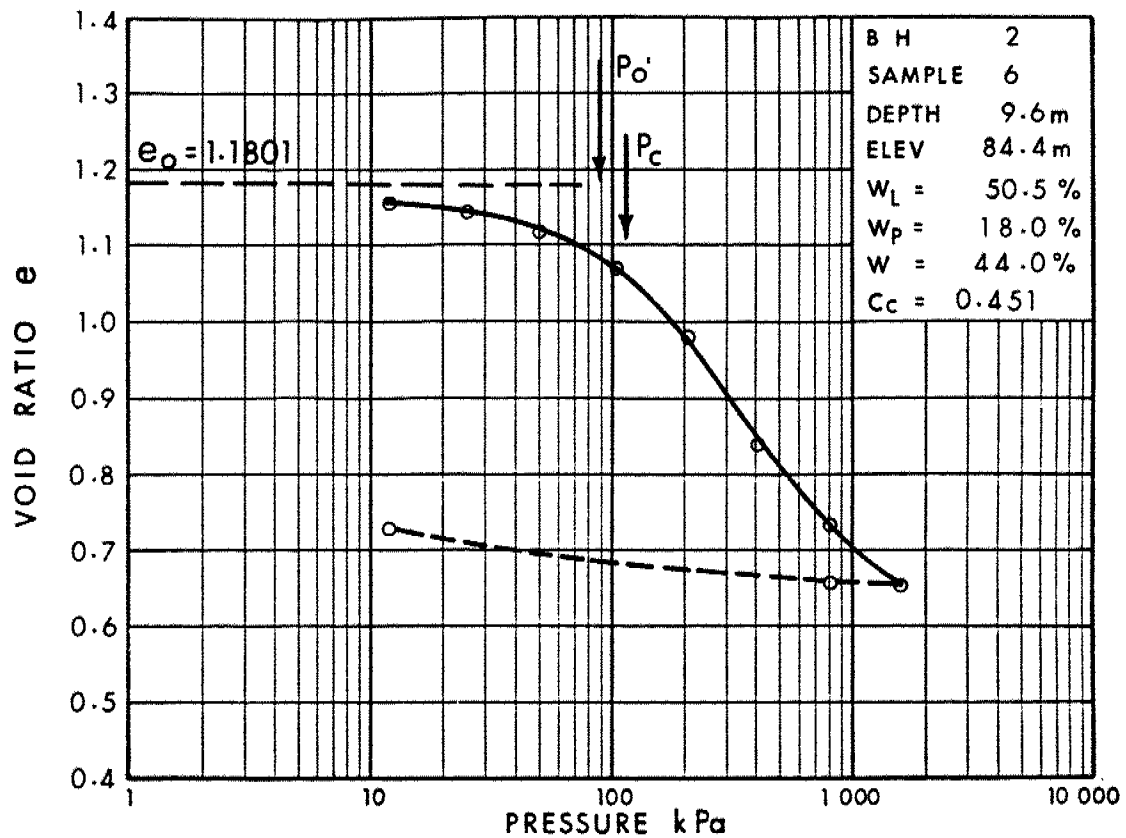


Fig 3

W P 112-86-01 (B)

VOID RATIO - PRESSURE CURVES

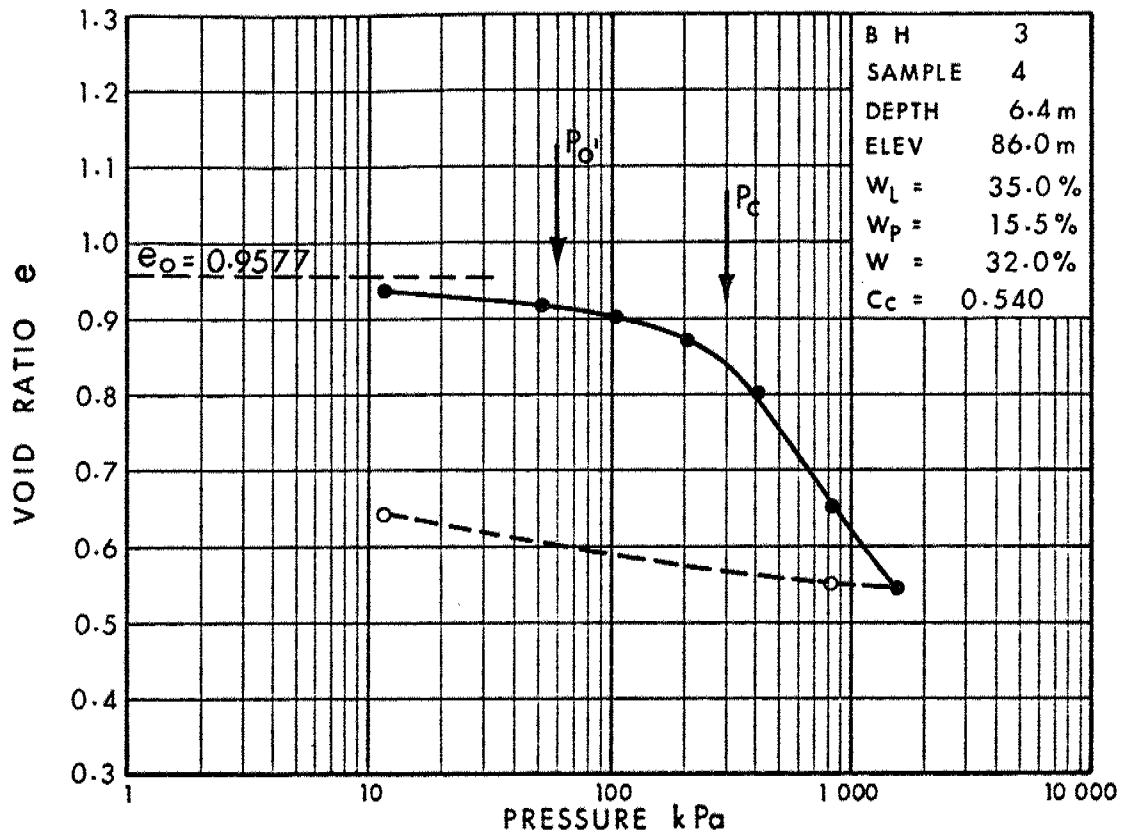


Fig 4

VOID RATIO - PRESSURE CURVES

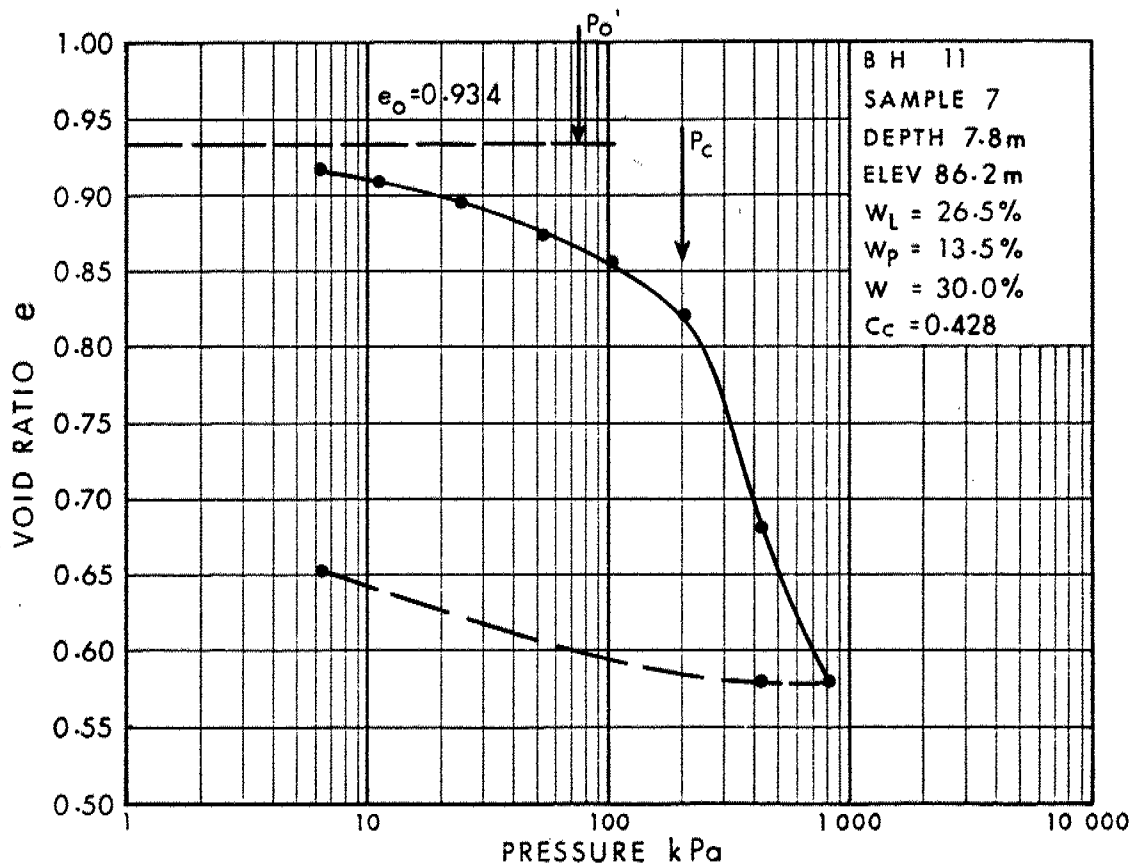
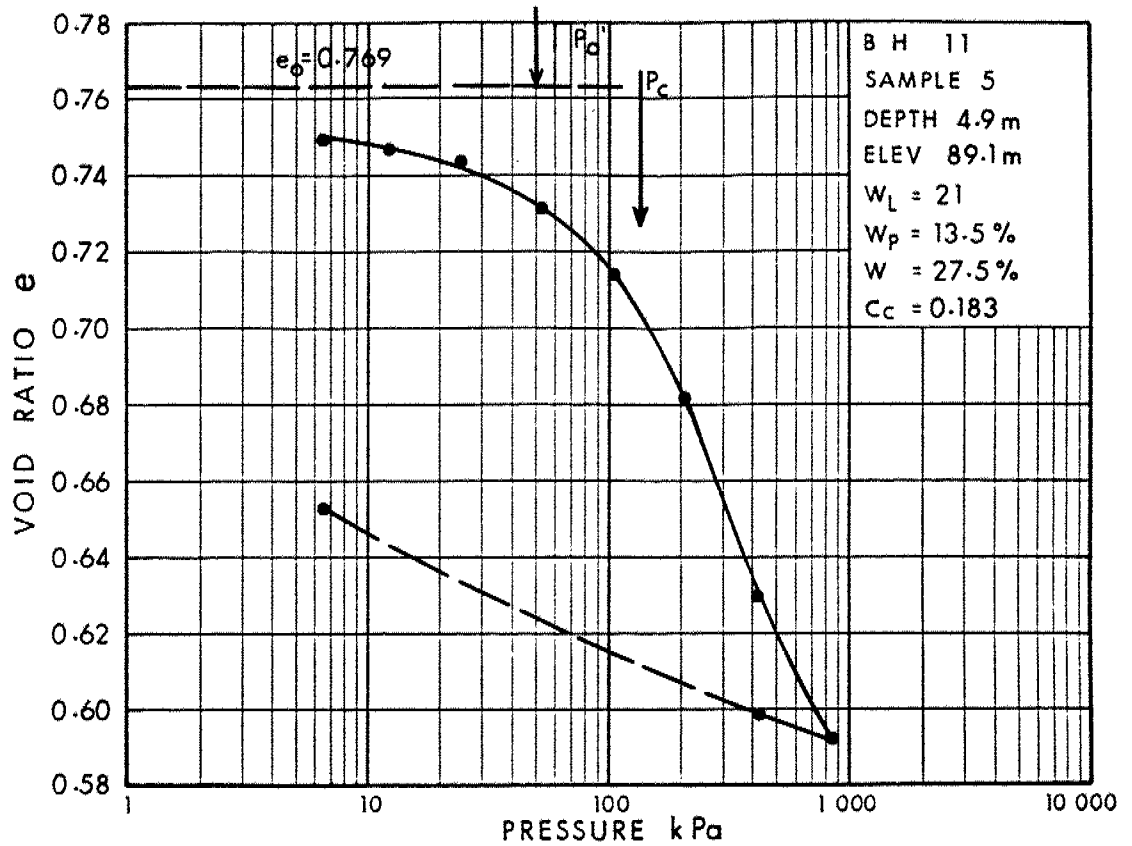


Fig 5

VOID RATIO - PRESSURE CURVES

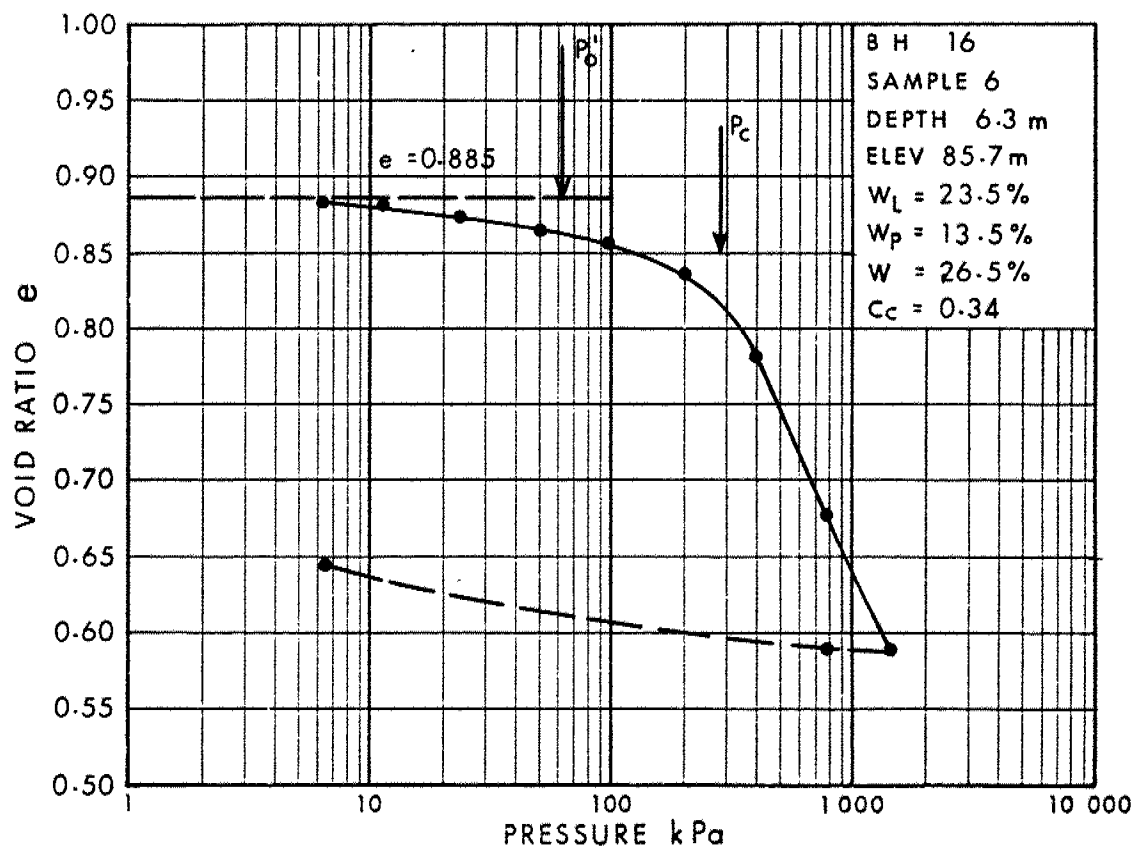
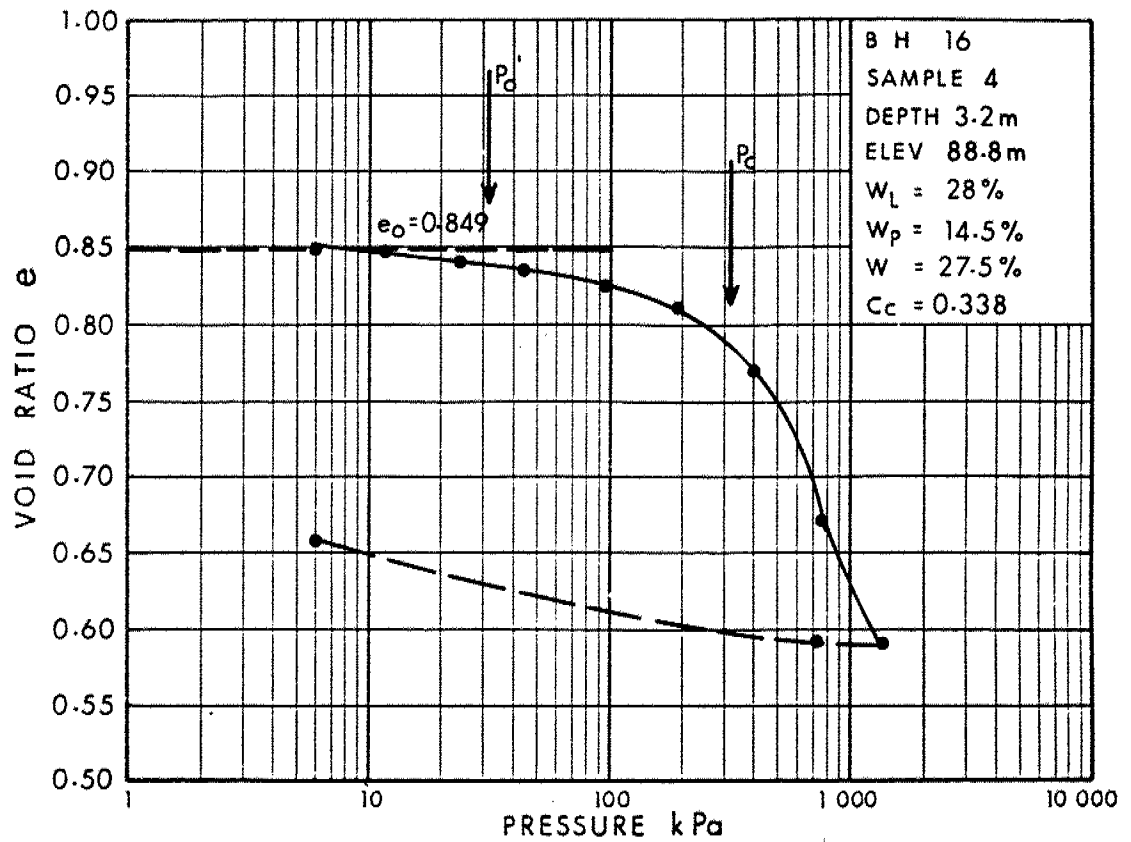


Fig 6

W P 112-86-01(B)

VOID RATIO - PRESSURE CURVES

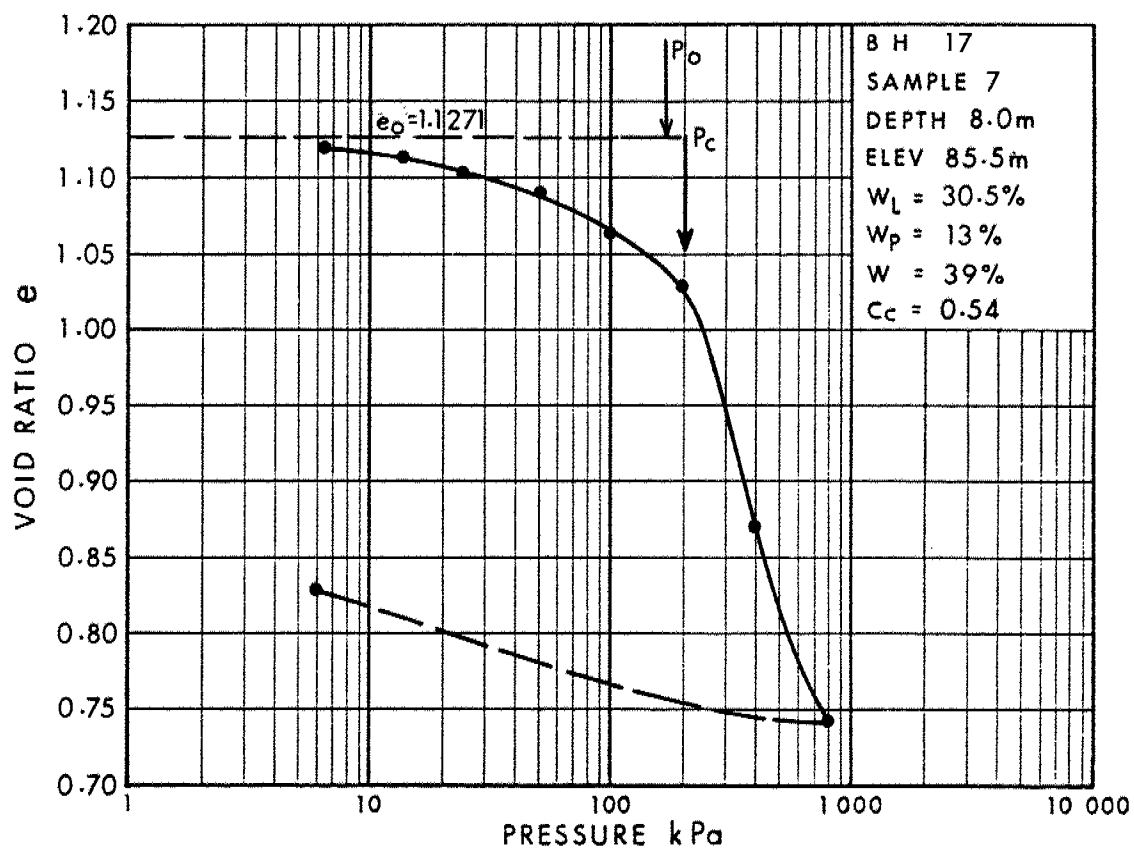
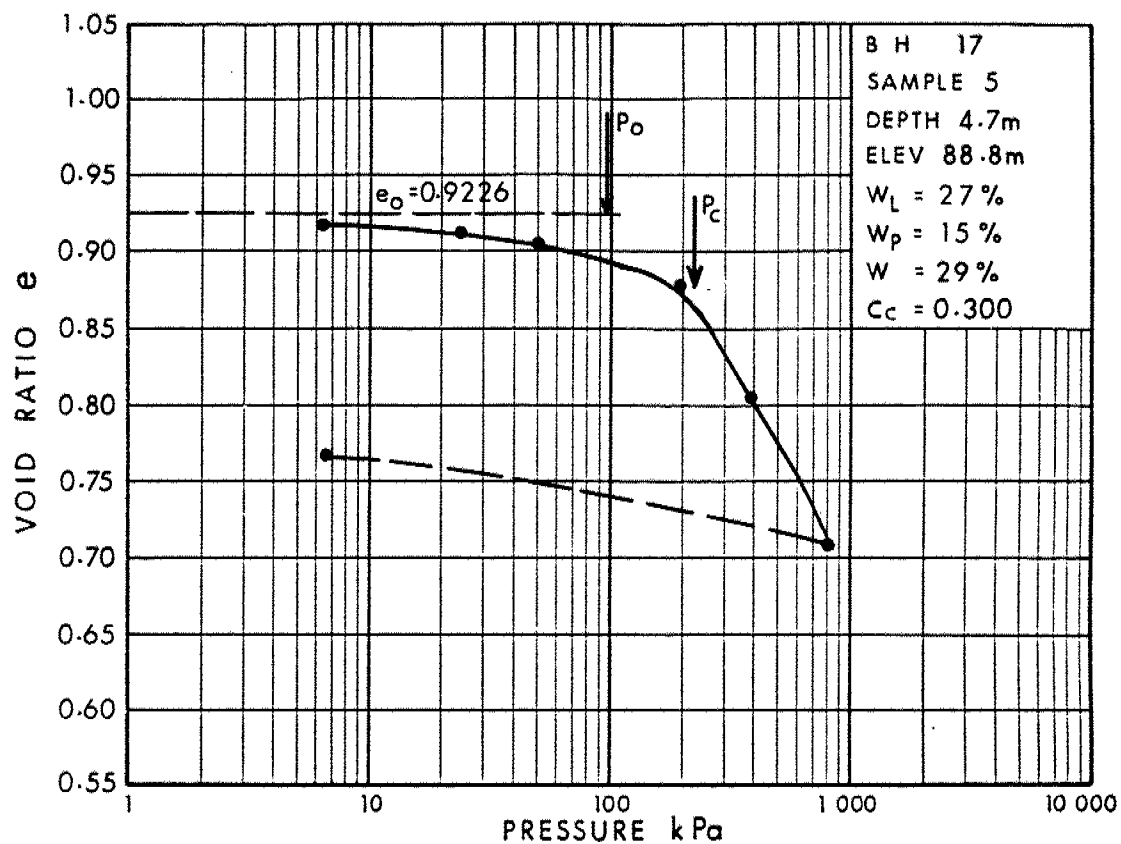
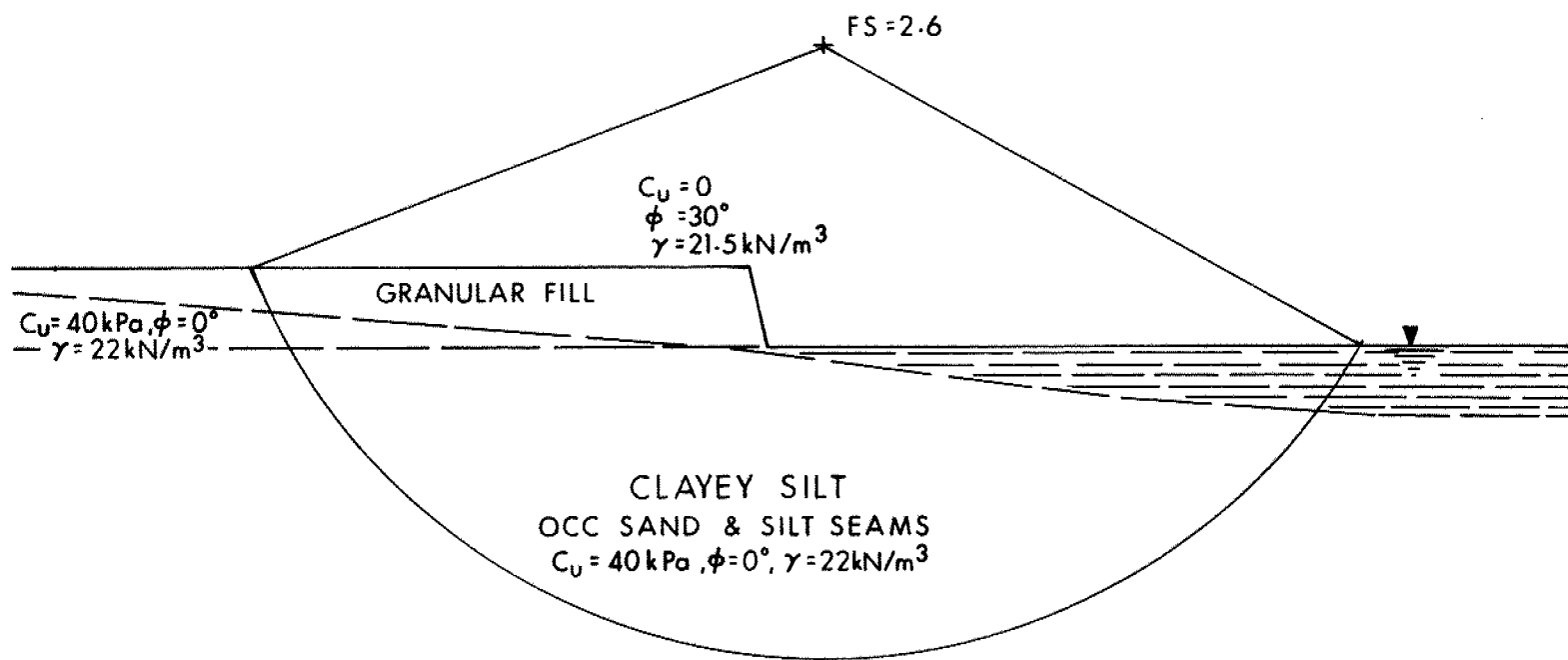


Fig 7

NORTH

SOUTH



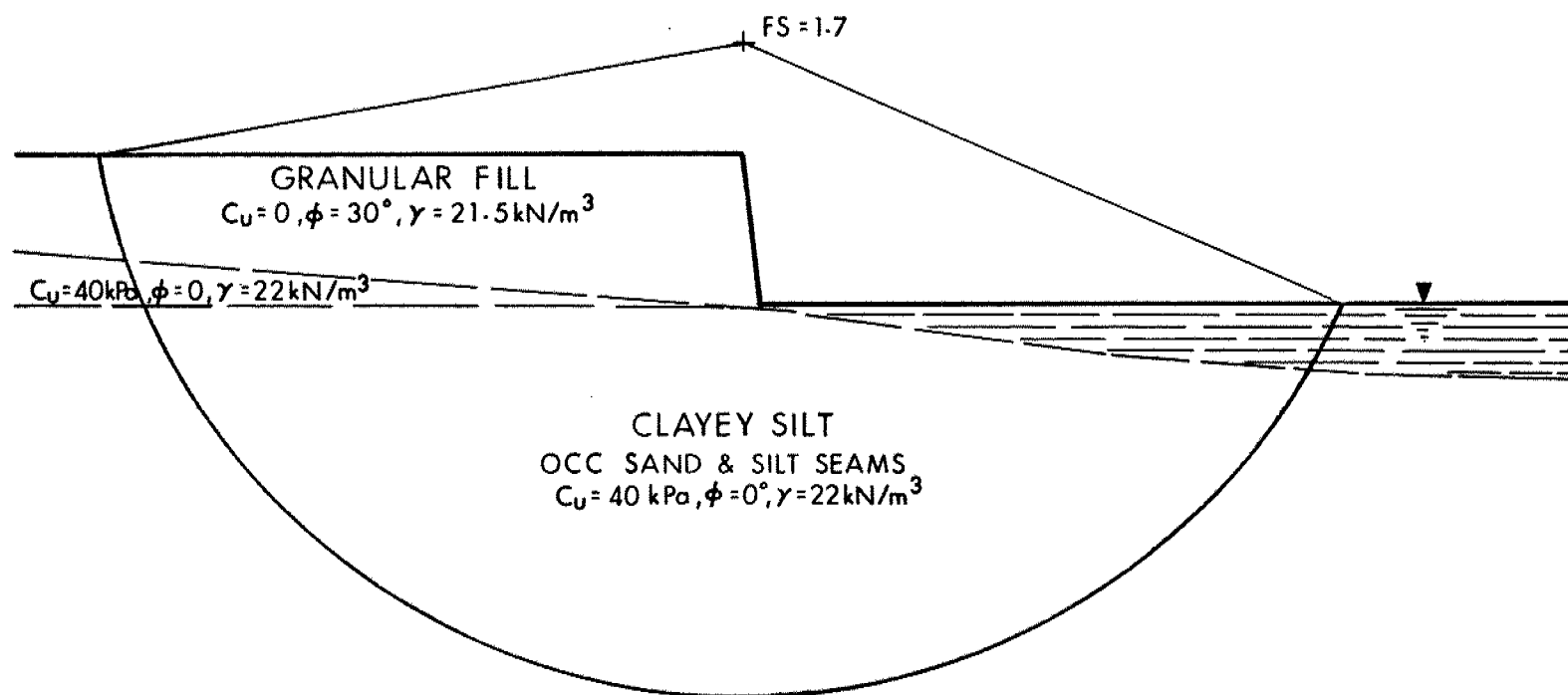
SCALE 1:400

FIG 8 - TOTAL STRESS ANALYSIS

WP 112-86-01(B)

NORTH

SOUTH



SCALE 1:400

FIG 9 TOTAL STRESS ANALYSIS

WP 112-86-01(B)

RECORD OF BOREHOLE No 1 (1 of 2)

METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 147.0; E 328 788.0 ORIGINATED BY MS

DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.); BW Casing COMPILED BY MS

DATUM Geodetic DATE 88 06 23 to 28 CHECKED BY AM

[illegible]

30.2

Continued

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 1 Cont'd (2 of 2) METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 089.2; E 328 775.7 ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.); BW Casing COMPILED BY MS
 DATUM Geodetic DATE 88 06 23 to 28 CHECKED BY MS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
63.8 30.2	Continued		17	SS	5								
	Clayey Silt . Frequent Silty Clay Layers, Occasional Sand and Silt Seams Trace Sand Firm / Stiff (Lacustrine)		18	SS	0								
			19	SS	5								
			20	SS	5								
			21	SS	55								
48.6 45.4	Occasional Silty Clay Seams Hard												
33.3 60.7 32.6	Silty Sand Dense		22	SS	32								
61.4	End of Borehole												

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 089.2; E 328 775.7 ORIGINATED BY MS
DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
DATUM Geodetic DATE 88 06 29 - 30 CHECKED BY MM

[illegible]

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 3 (1 of 2)

METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 184.2; E 328 773.0 ORIGINATED BY MS
DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
DATUM Geodetic DATE 88 06 30 & 88 07 04 CHECKED BY *Ma*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
92.4	Ground Surface													
0.0	Top Soil						92							
			1	SS	3		90							0 0 86 14
			2	SS	1		88							0 1 70 29
	Clayey Silt Frequent Silty Clay Layers Occasional Sand and Silt Seams		3	SS	2		86							
	Trace Sand		4	TW	*		84							
	(Lacustrine)		5	TW	PH		82							
	Stiff		6	SS	0		80							
			7	SS	4		78							
			8	SS	2		76							
			9	SS	0		74							
76.7			10	SS	0		72							
15.7	End of Borehole						70							
	Presumed Clayey Silt Frequent Silty Clay Layers Occasional Sand and Silt Seams						68							
	Trace Sand						66							
	(Lacustrine)						64							
62.2														
30.2														

Continued

* Pushed by Self-Weight

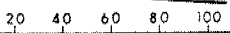
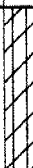
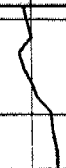
+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 3 Cont'd (2 of 2) METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 184.2; E 328 773.0 ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
 DATUM Geodetic DATE 88 06 30 & 88 07 04 CHECKED BY MS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
62.2	Continued												
30.2	Presumed Clayey Silt Frequent Silty Clay Layers, Occasional Sand and Silt Seams Trace of Sand (Lacustrine) Stiff						62						
58.8							60						
33.6	End of Cone Test												

RECORD OF BOREHOLE No 4

METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 063.6; E 328 766.6 ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
 DATUM Geodetic DATE 88 07 04 CHECKED BY *MS*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
94.1	Ground Surface													
0.0	Gravelly Sand (Fill)													
92.6	Very Loose to Compact*													
1.5	Trace Organics		1	SS	2									
	Clayey Silt Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace Sand (Lacustrine)		2	SS	2									0 11 58 31
			3	SS	0									
			4	SS	0									0 0 77 23
	Firm / Stiff		5	SS	0									
84.5			6	SS	14									
9.6	End of Borehole													
	Presumed Bedrock													
	* Trace Organics													

RECORD OF BOREHOLE No 11

1 OF 3

METRIC

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 142.2; E 328 779.6 ORIGINATED BY KA
 DIST 8 HWY 15 BOREHOLE TYPE Hollow Stem Auger, NW Casing COMPILED BY KA
 DATUM Geodetic DATE 90 03 12 to 90 03 20 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
94.0	Ground Level												
0.0	Gravelly Sand Fill Compact		1	SS	25								
			2	SS	3								
	Dk. Brown Clayey Silt with wood fragments/topsoil Soft to Firm		3	SS	2								
90.1			4	SS	5								
3.9			5	TW	PH								
			6	SS	0								
			7	TW	PH								
			8	SS	1								
			9	TW	*								
			10	SS	0								
			11	TW	*								
			12	TW	*								
	Clayey Silt, Grey with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand and Gravel (Lacustrine) Soft to Stiff (Generally Firm)		13	SS	1								
			14	SS	4								
			15	SS	2								
			16	SS	0								
63.5													
30.5													

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 11

2 OF 3

METRIC

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 142.2; E 328 779.6 ORIGINATED BY KA
DIST 8 HWY 15 BOREHOLE TYPE Hollow Stem Auger, NW Casing COMPILED BY KA
DATUM Geodetic DATE 90 03 12 to 90 03 20 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
30.5	Continued		17	SS	5								
	Clayey Silt, Grey with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand and Gravel (Locustrine) Soft to Stiff (Generally Firm)		18	SS	4								0 0 80 20
54.0			19	SS	0								
40.0			20	SS	18								
	Silty Sand with Gravel Occasional Boulders Grey, Wet Compact to Very Dense		21	SS	20								
			22	SS	17								
			**	SS	16								
			**	SS	23								
			**	SS	72								
			**	SS	94								
			**	SS	93								
			**	SS	115								
			**	SS	122								

Continued

+3, x5: Numbers refer to Sensitivity

20 15-5 (%) STRAIN AT FAILURE 10

Continued

RECORD OF BOREHOLE No 11

3 OF 3

METRIC

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 142.2; E 328 779.6 ORIGINATED BY KA
 DIST 8 HWY 15 BOREHOLE TYPE Hollow Stem Auger, NW Casing COMPILED BY KA
 DATUM Geodetic DATE 90 03 12 to 90 03 20 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	w _p	w		
32.4	Continued		**	SS	132											
61.6	End of Borehole															
	* Shelby Tube sank with the weight of the A-rod. ** Split spoon sampler was advanced continuously (like cone test) to determine the density of the soil. The energy used to drive the spoon was the same as used in the Standard Penetration Test N - values may not be representative															

RECORD OF BOREHOLE No 13

1 OF 2

METRIC

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 121.7; E 328 773.8 ORIGINATED BY KA
DIST 8 HWY 15 BOREHOLE TYPE NW Casing, BW Casing, BX Core COMPILED BY KA
DATUM Geodetic DATE 90 03 22 to 90 03 27 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40						60	80
82.0	Water Level															
0.0	Water (Morton Creek)															
89.9																
2.1																
	Presumed Clayey Silt, Grey, Firm with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand (Lacustrine)															
14.0																
69.1																
22.9																
	Presumed Silty Sand with Gravel Occasional Boulders Grey, Wet Compact to Very Dense															
61.3																
30.5																

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 13

2 OF 2

METRIC

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 121.7; E 328 773.8 ORIGINATED BY KA
 DIST 8 HWY 15 BOREHOLE TYPE NW Casing, BW Casing, BX Core COMPILED BY KA
 DATUM Geodetic DATE 90 03 22 to 90 03 27 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100	W _P	W	W _L		
	Continued															
30.5	Presumed Silty Sand with Gravel Occasional Boulders Grey, Wet Compact to Very Dense					60										
						58										
						56										
						54										
53.3	Bedrock MARBLE		1	RC	REC	97%										RQD 84%
38.7			2	RC	REC	98%										RQD 100%
50.0	End of Borehole															
42.0																

RECORD OF BOREHOLE No 16

1 OF 1

METRIC

W.P. 112-86-01 (8) LOCATION Co-ords: N 4 933 155.7; E 328 771.3 ORIGINATED BY KA
DIST 8 HWY 15 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KA
DATUM Geodetic DATE 90 03 21 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
92.0	Ground Level																
0.0	Topsail 15 cm.		1	SS	2												
			2	SS	2												0 3 72 25
			3	SS	2												
			4	TW	PH												19.6 0 2 73 25
			5	SS	1												
			6	TW	PH												19.6 0 2 73 25
	Clayey Silt, Gray with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand (Lacustrine) Soft to Stiff (Generally Stiff)		7	SS	2												
			8	SS	3												
			9	SS	2												0 6 56 38
			10	SS	1												
75.8																	
16.2	End of Borehole																
* GROUND WATER CONDITIONS																	
PIEZO. NO.			GROUND WATER ELEVATION (Metres)														
1			92.0														

METRIC

CON	PROF	PR	EE	DYNAMIC CONE PENETRATION			
-----	------	----	----	--------------------------	--	--	--

+ 3, x 5: Numbers refer to Sensitivity

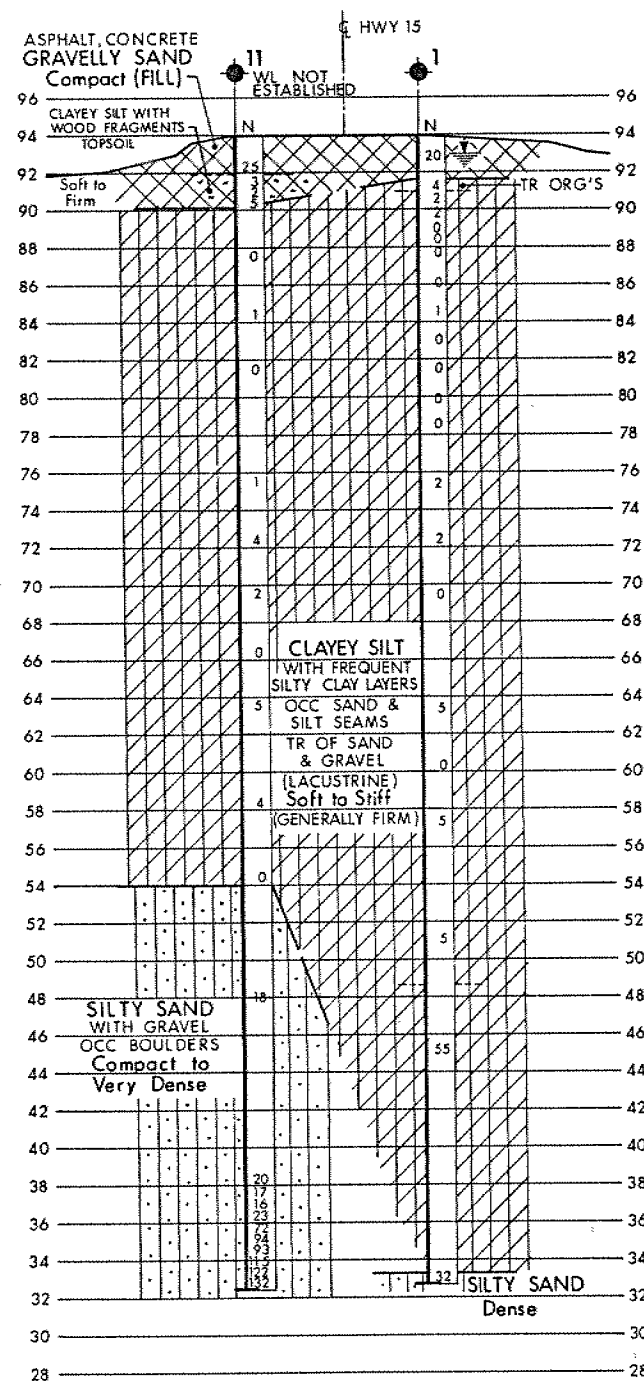
RECORD OF BOREHOLE No 18

1 OF 1

METRIC

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 245.6; E 328 778.5 ORIGINATED BY KA
 DIST 8 HWY 15 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KA
 DATUM Geodetic DATE 90 03 21 CHECKED BY DD

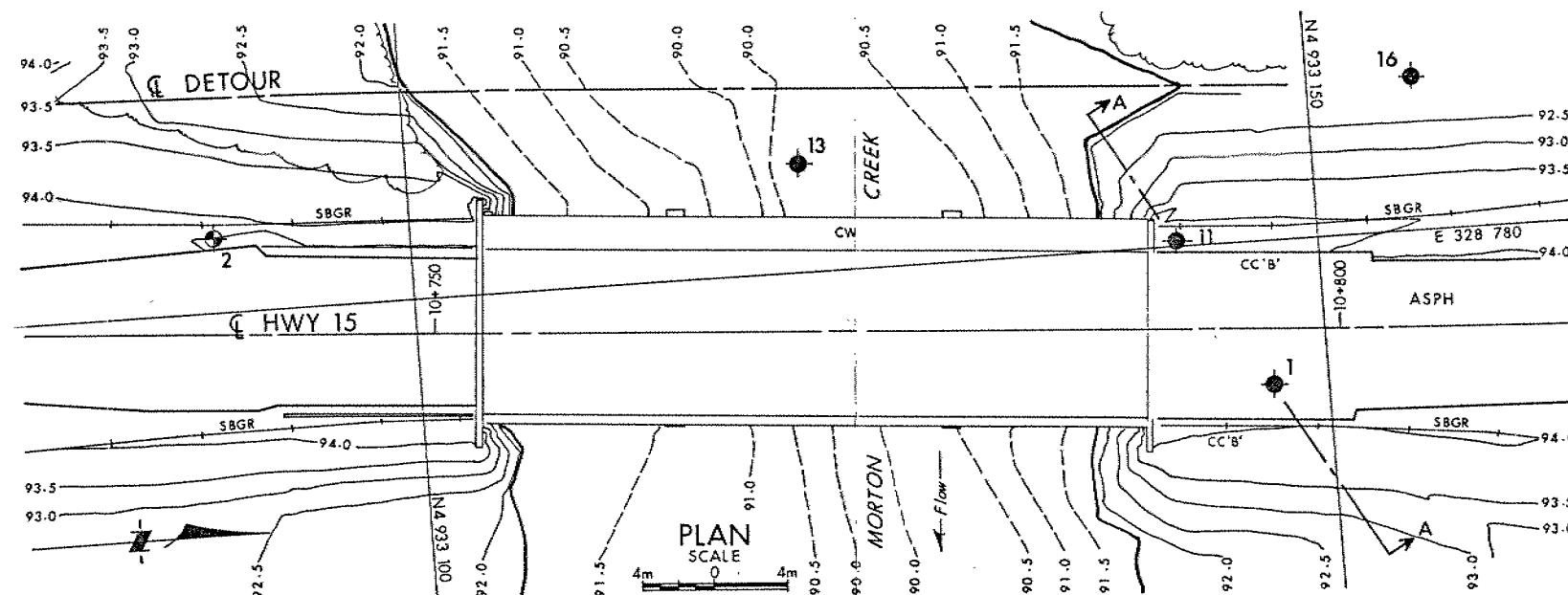
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
93.8	Ground Level															
0.0	Silty Sand with a Trace of Clay (Fill) Very Loose to Loose		1	SS	3											
			2	SS	1											
91.3			3	SS	11											0 34 49 17
2.5	Clayey Silt, Greyish Brown/Gray with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand and Gravel (Locustrine) Firm to V. Stiff		4	SS	9											
			5	SS	1											
			6	SS	1											
			7	SS	1											
88.1			8	SS	50	/Bcm										
7.7	End of Borehole															
	Hammer Bouncing at 7.7m depth Presumed Bedrock															



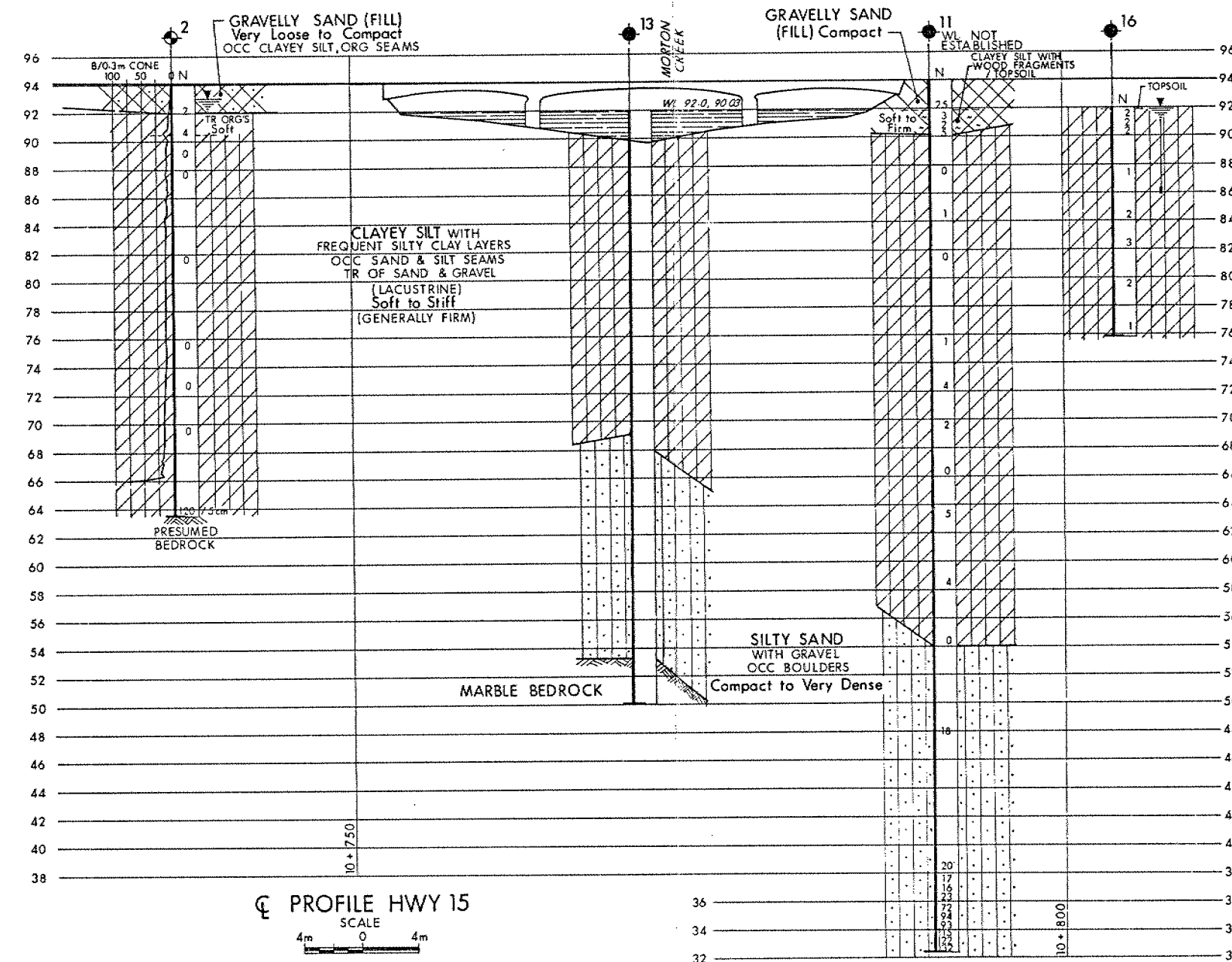
SECTION A-A

SCALE
4m 0 4m

Note:
Subsoil information for BH's 3, 4, 17 & 18
Refer to Record of Borehole



PLAN
SCALE
4m 0 4m



PROFILE HWY 15

SCALE
4m 0 4m

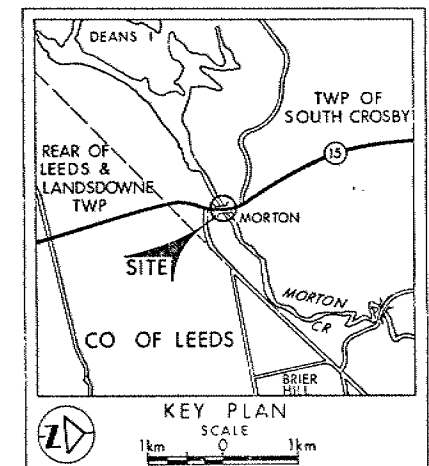
CONT No
WP No 112-86-01(B)

MORTON CREEK BRIDGE

SHEET

BORE HOLE LOCATIONS & SOIL STRATA

METRIC DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN.



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation
88 06, 88 07 and 90 03
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	94.0	4 933 147.0	328 788.0
2	94.0	4 933 089.2	328 775.7
3	92.4	4 933 184.2	328 773.0
4	94.1	4 933 063.6	328 766.6
11	94.0	4 933 142.2	328 779.6
13	92.0	4 933 121.7	328 773.8
16	92.0	4 933 155.7	328 771.3
17	93.5	4 933 203.0	328 779.3
18	93.8	4 933 245.6	328 778.5

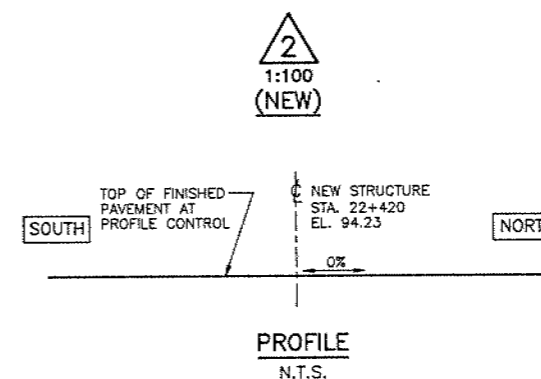
NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

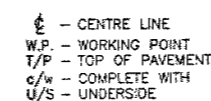
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			
16			
17			
18			
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•DD 3503 MINIMUM GRANULAR BACKFILL REQUIREMENTS

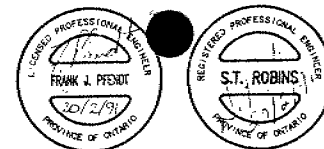


DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING



B.M. ELEV. 95.829
TABLET SET IN TOP OF ROCK
11.78 LT OF STA. 10+656.795
ROUTE 249

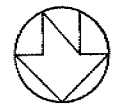
REVISIONS									
	DATE	BY		DESCRIPTION					
DESIGN	F.J.P.	CHK	S.T.R.	CODE	OHBOC 83	LOAD CLASS A	DATE	19 FEB 9	
DRAWN	P.C.M.	CHK	S.T.R.	SITE	18-45	STRUCT	SCHEME	DWG.	1



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 112-86-01

MORTON CREEK BRIDGE

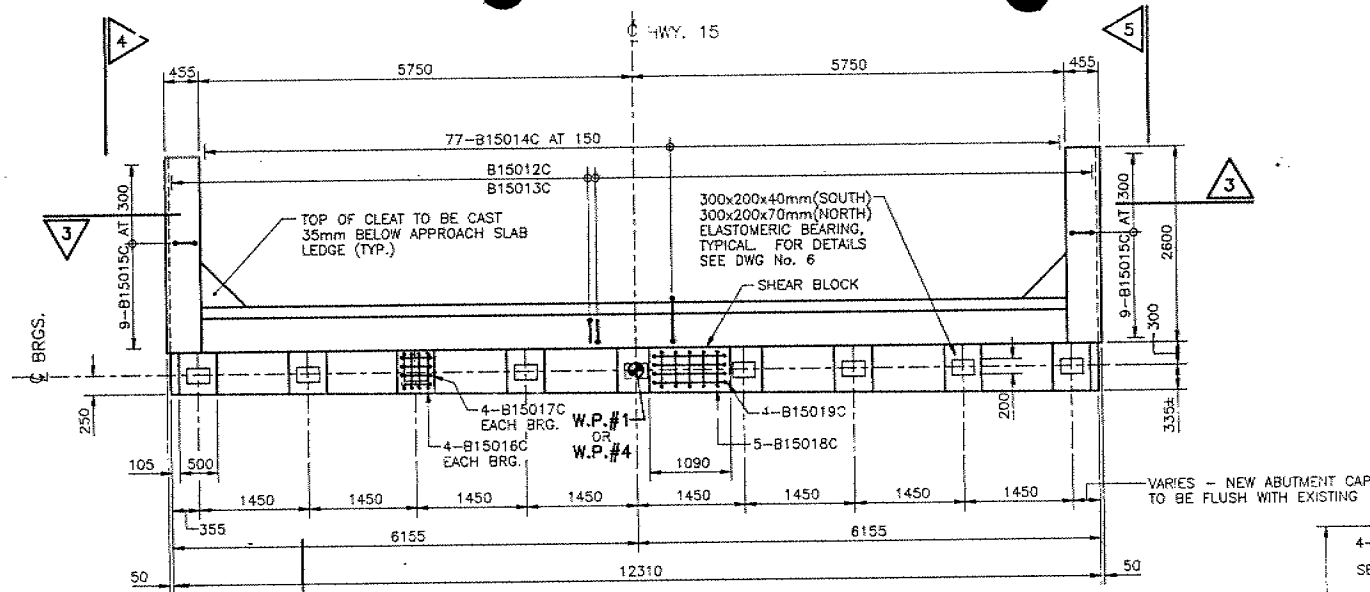


SHEET

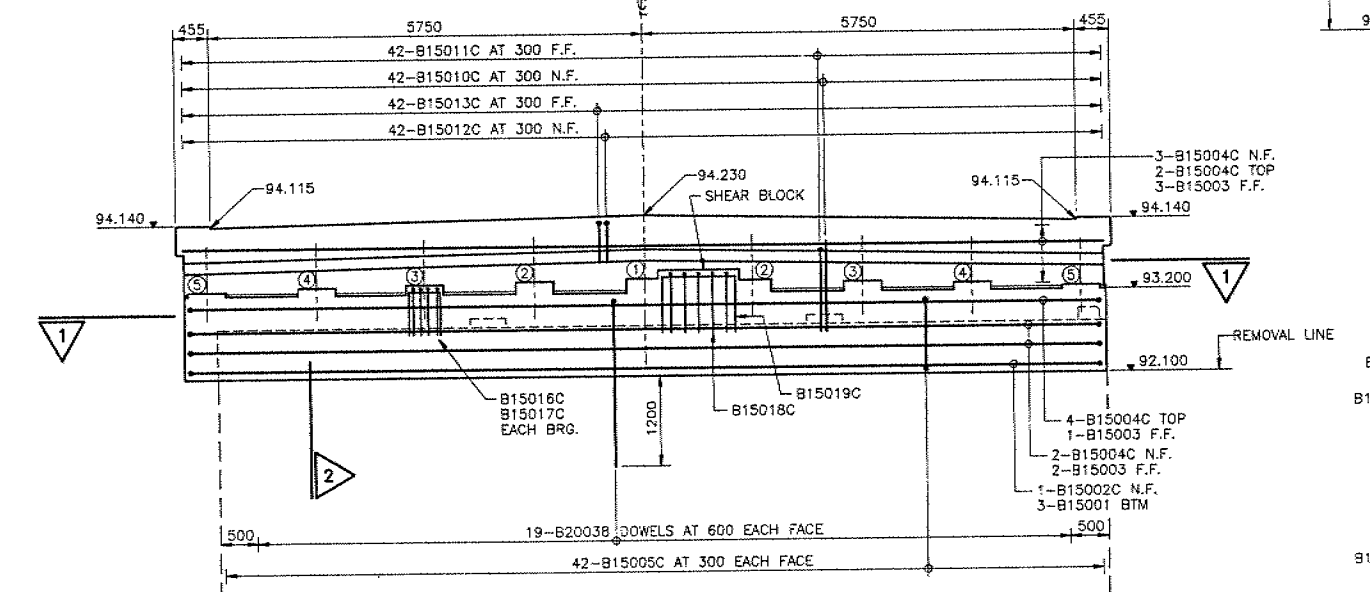
ABUTMENTS

**McNEELY ENGINEERING
& STRUCTURES LTD.**
CONSULTING ENGINEERS
815 PRINCESS STREET
KINGSTON, ONTARIO K7L 1G8
PHONE (613) 549-0500

NOTE:
SOUTH ABUTMENT SHOWN ON
THIS DRAWING.
NORTH ABUTMENT SIMILAR
EXCEPT OPPOSITE HAND

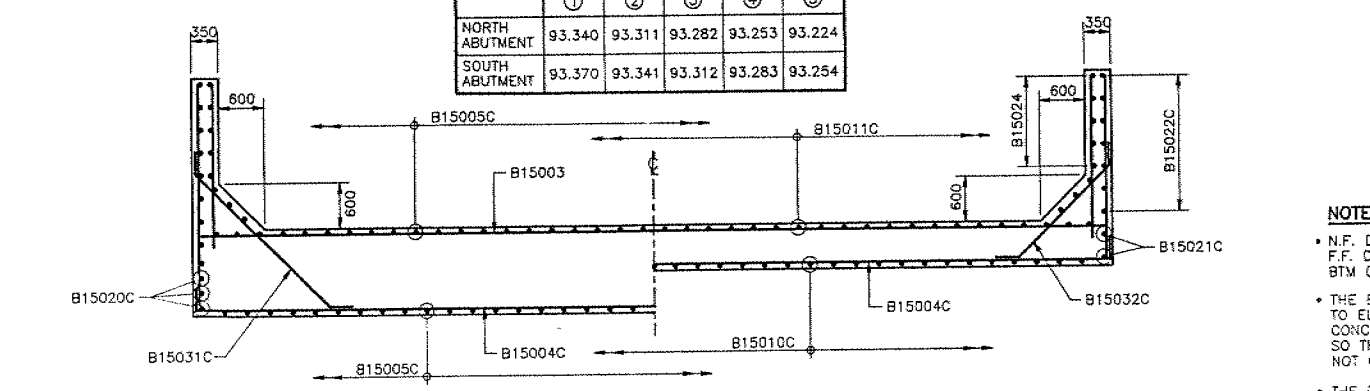


PLAN
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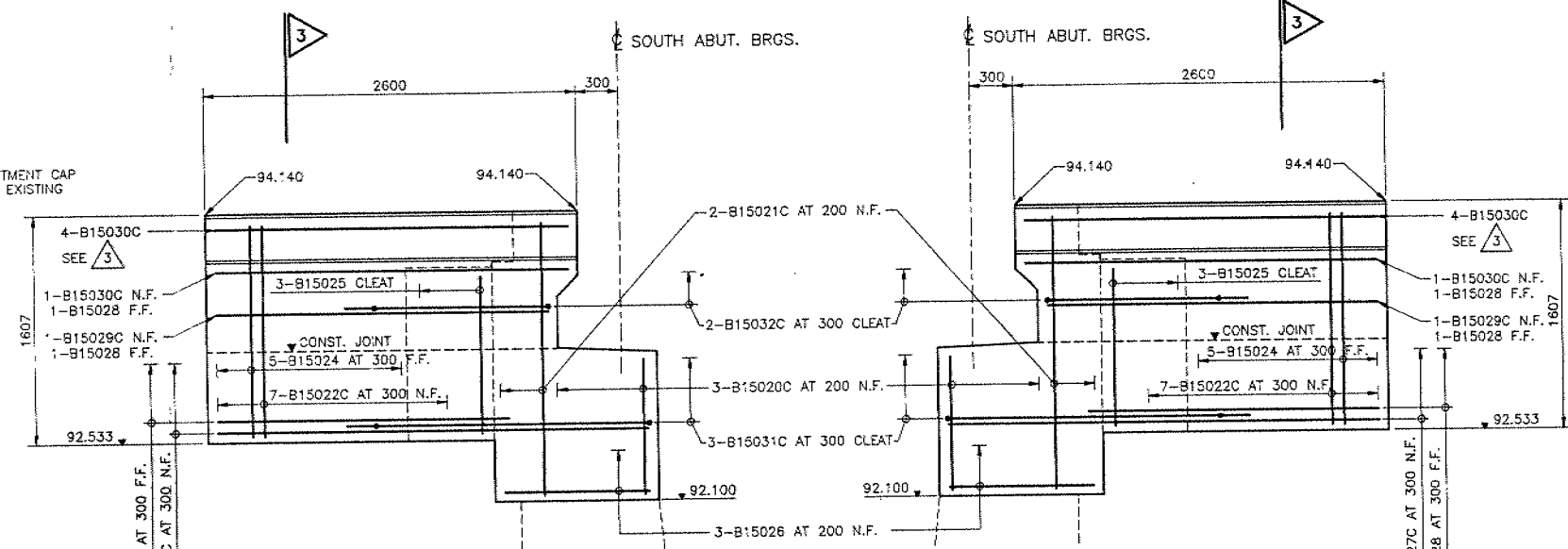


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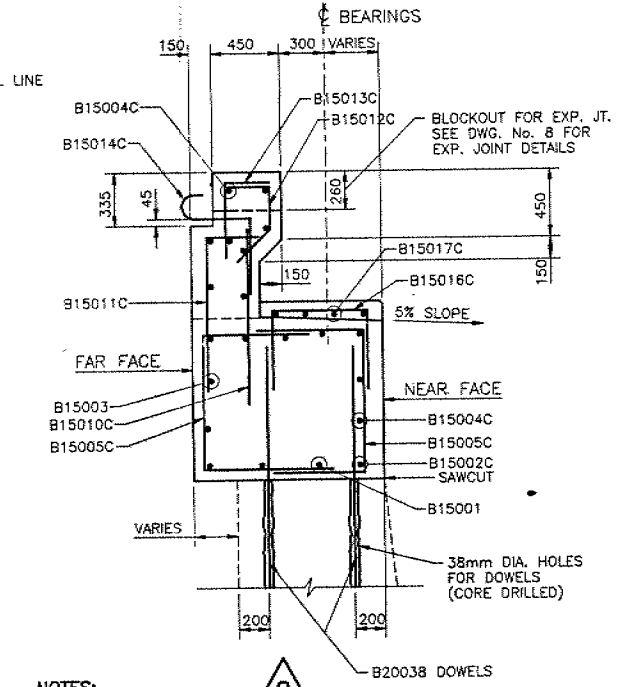
BEARING SEAT ELEVATIONS					
	①	②	③	④	⑤
NORTH ABUTMENT	93.340	93.311	93.282	93.253	93.224
SOUTH ABUTMENT	93.370	93.341	93.312	93.283	93.254



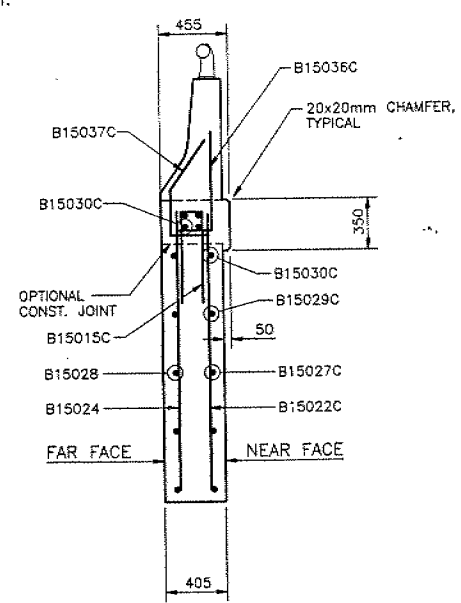
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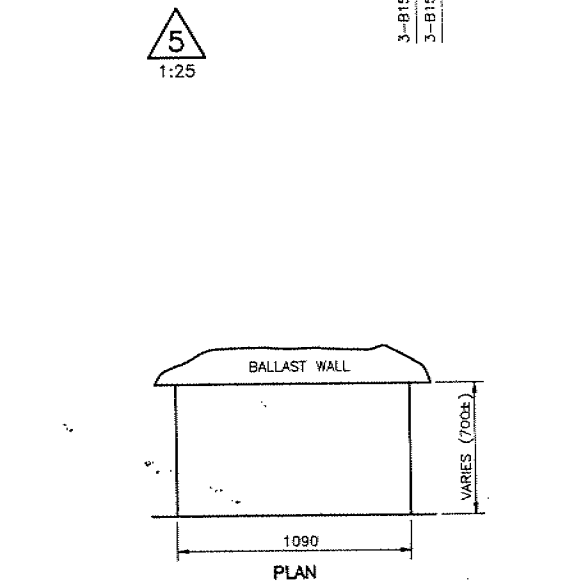
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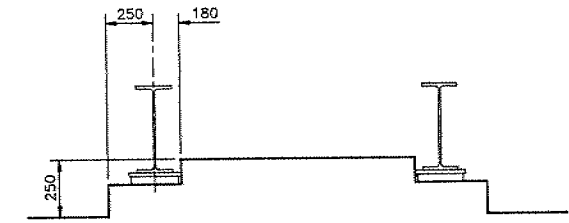
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PLAN



**ELEVATION
SHEAR BLOCK DETAIL**
N.T.S.

- NOTES:**
- N.F. DENOTES NEAR FACE
F.F. DENOTES FAR FACE
BTM DENOTES BOTTOM
 - THE EXISTING ABUTMENTS ARE TO BE REMOVED BY SAWCUTTING AND LINE DRILLING TO ELEVATION 92.100. PRIOR TO PLACING NEW CONCRETE, THE EXISTING CONCRETE SURFACE IS TO BE CLEAN, FREE OF LAITANCE AND INTENTIONALLY ROUGHENED SO THAT THE DEPTH OF INDENTATION IS 5mm AND THE SPACING OF INDENTATION IS NOT GREATER THAN 15mm.
 - THE DOWELS SHALL BE GROUTED IN PLACE USING NON-SHRINK GROUT APPLIED IN ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS.
 - SAWCUTS IN CONCRETE, WHERE DESIGNATED, SHALL BE 100mm DEEP OR THROUGH THE FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS GREATER.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN F.J.P./CHK S.T.R./CODE QHBC-83 LOAD CLASS A/DATE 19 FEB 91
DRAWN P.C.M./CHK S.T.R./SITE 16-45 STRUCT SCHEME DWG. 3

MEMORANDUM

(416) 235-3731

To: I. Husain
Design Engineer
Structural Office
7th Floor, Atrium Tower
1201 Wilson Avenue, Downsview

1991 11 25

From: Foundation Design Section
Room 315, Downsview, Ontario

Re: Mortan Creek Bridge
W.P. 112-86-01, Site 16-45
District 8, Kingston

We have reviewed copies of the D4, Special Provisions and half size print.

Since the new bridge will be constructed on existing piers and abutments we have no comments on the proposed construction. As mentioned in the Foundation investigation report dated 90 05 25 and our subsequent memo dated 90 11 20 a deposit of organic material was discovered under the north approach which had caused significant settlement at the north approach. We recommend that this organic layer should be removed in order to reduce the settlement.

Should you have any further questions, please advise.



D.H. Dundas, P. Eng.
Sr. Foundations Engineer

For

M. Devata, P. Eng.
Chief Foundations Engineer

memorandum



To: K.G. Bassi
Head, Structural Section
Central Region
7th Floor, Atrium Tower

Date: 1990 11 20

Attn: Dr. I. Husain

From: Foundation Design Section
Room 315, Central Building

RE: Norton Creek Overpass
W.P. 112-86-01, Site 16-45
District 8, Kingston

We have reviewed the General Arrangement Drawing No. P1 for the above noted structure. Our comments are as follows:

We understand that the new bridge will be constructed on existing piers and abutments. Since the details of the existing foundation is unknown, it can only be assumed that the existing foundations can support as much load as they are currently carrying. We trust that this matter has been incorporated in the design of the new bridge.

As reported in the foundation investigation report dated 90 05 25, a deposit of organic material was discovered under the north approach. It was recommended that this organic material should be removed in order to minimize the reported settlement. We believe this will be noted on the final drawing.

A handwritten signature in cursive script, appearing to read "K. Ahmad".

K. Ahmad, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Sr. Foundation Engineer

DD/KA/jb

MEMORANDUM

(416) 235-3731

To: Geotechnical Section
Eastern Region, Kingston

1990 10 01

Attn: Robert Scott

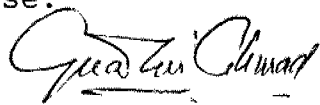
From: Foundation Design Section
Room 315, Central Region, Downsview, Ontario
Central Region

Re: Morton Creek Bridge
W.P. 112-86-01 (B)
Highway 15, District 8 - Kingston

This is in response to your memorandum dated Oct 01, 1990. Following are our comments, in sequence, to some of the issues addressed in your memo.

1. Rock fill may be used for detour construction and along the existing highway alignment.
2. The same recommendation, as in (1), applies for the reconstruction of the north approach.
3. Rock fill can be used as a surcharge load for the north approach. However, it will be desirable that if construction schedule permits, the surcharge load is left in place for a minimum period of four months, instead of three months (as recommended in the Foundation Report, page 12. 4th paragraph).
4. The detour approach fill can be left in place after the construction, provided, the reduced bearing capacity is used for the pile design as discussed in the Foundation Report (Page 10, last paragraph).
5. Before placing any fill, topsoil or any organic material should be removed.

Should you have any further questions, please advise.


Ken Ahmad, P. Eng.
Foundation Engineer

For

D.H. Dundas, P. Eng.
Sr. Foundation Engineer

memorandum



(613) 544-2220 Ext. 4163

To: Mr. M. Devata
Chief Foundation Engineer
Foundations Office
Downsview

ATTN: Mr. K. Ahmad

FROM: Geotechnical Section
Eastern Region, Kingston

RE: W.P. 112-86-01, Highway 15
Morton Creek Bridge
District 08, Kingston

Date: October 1, 1990

Our Planning & Design Section requires clarification of recommendations made in the Foundation Design Report dated May 25, 1990.

Items requiring clarification:

1. Page 12, 1st paragraph: Can rock fill be used for the detour construction and along the existing highway alignment. If not, then what type of granulars are to be used?
2. Page 12, 4th paragraph: Can rock fill be used for the reconstruction of the north approach. If not, then what type of granulars are to be used?
3. Page 12, 4th paragraph: Can rock fill be used for the surcharge load for the north approach. If not, then what type of granulars are to be used?
4. Can the detour approach fills remain in place after construction?
5. Is stripping required in fill sections?

A speedy response would be greatly appreciated.

A handwritten signature in dark ink, appearing to read "Robert Scott".

Robert Scott
Soils Supervisor

RS/dka

c.c.: G. Chaput



memorandum



To: M. Devata
Foundation Design Section
Downsview

Date: July 25, 1990

Attention: Dave Dundas

From: Structural Section
Eastern Region, Kingston



Re: Morton Creek Bridge, Site 16-45
W.P. 112-86-01

The consultant, McNeely Engineering and Structures Ltd., has expressed concerns regarding the end bearing pile foundations. The consultant considers that there is some risk of the pile tips not "catching" on bedrock or in a suitable end bearing stratum, due to the steeply dipping irregular bedrock surface. Note that this concern is only for the pier pile bents.

The consultant wishes to address this risk, and consider as an alternative a friction caisson pile foundation.

Please arrange a meeting as early as possible to discuss the risks and the alternative proposal.

A handwritten signature in dark ink, appearing to read "H. Kleywegt".

H. Kleywegt
for E.C. Lane
Head, Structural Section

ECL:HK:sw

c.c. I. Husain
D. McNeely - McNeely Engineering & Structures

(613) 544-2220 Ext. 4163

July 23, 1990

Mr. D. Kimmett
Head, Planning & Design
Eastern Region, Kingston

ATTN: Mr. D. Huddle
FROM: Geotechnical Section
Eastern Region, Kingston
RE: W.P. 112-86-01, Highway 15
Morton Creek Bridge Replacement
Site 16-45, District 08, Kingston

At the onset of this project, the Geotechnical Section requested field investigations and associated recommendations for detour structure approaches be made by the Foundation Design Section due to poor soil conditions.

We have reviewed the "Foundation Investigation and Design Report" for the project and believe the Geotechnical concerns have been fully addressed.

If there are any outstanding items, please contact this section.

Robert Scott Iaka
Robert Scott
Soils Supervisor

RS/dka

c.c.: M. Devata ✓
H. Kleywegt



SEND
TO

DAVE OUNOAS

FOUNDATION DESIGN SECTION

DOWNS VIEW

FAX 235-5240

FROM

HAROLD KLEYWEGT

DEPT.

STRUCTURAL

DATE

90-04-23

SUBJECT

MORTON CREEK REPLACEMENT BRIDGE

W.P. 112-86-01

FURTHER TO OUR TELEPHONE CONVERSATION, THIS DATE, I WILL CONFIRM HERE MY RATIONALIZATION IN SUPPORT OF END BEARING PILES FOR MORTON CREEK BRIDGE. AS NOTED IN MY MEMORANDUM OF JAN 10, 1990 TO YOURSELF, IT WILL BE POSSIBLE TO AVOID PIER COFFEROAMS BY UTILIZING END BEARING STEEL PILE BENTS FRAMING INTO THE SUPERSTRUCTURE. THE COST OF UNWATERING ON A SIMILAR STRUCTURE WITH COFFEROAMS IN TWO METRES OF WATER WAS \$150,000.00 - SEE CONTRACT 89-206 IN DISTRICT 16.

THE UNFACTORED LOAD ON THE PIERS WILL BE OF THE ORDER OF 3000 KN EACH. A TOTAL OF 30 FRICTION PILES (TIMBER)

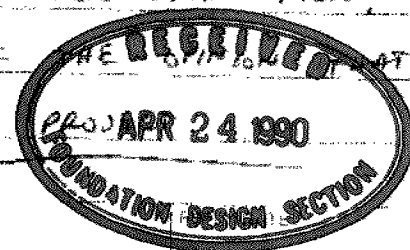
REPLY AT 15m EACH OR 450 m OF PILE WOULD HAVE TO BE DRIVEN FOR A FRICTION PILE FOUNDATION. ALTERNATIVELY, ASSUMING AN AVERAGE 40 m LENGTH END BEARING PILE, ONLY 160m TO 200 m OF END BEARING PILE WOULD HAVE TO BE DRIVEN FOR EACH PIER.

FOR THE ABOVE REASONS I AM STRONGLY OF THE OPINION THAT END BEARING PILES SHOULD BE SPECIFIED FOR THIS

REGARDS,

REPLY FROM

H. Kleywegt



MEMORANDUM

To: E.C. Lane, P. Eng.
Head, Structural Section
Structural Section
Eastern Region, Kingston

Date: 1990 04 12

Att'n: H.S. Kleywegt

From: Preliminary Foundation Design Section
Room 315, Central Region, Downsview, Ontario

Re: Foundation Recommendations
Morton Creek Bridge
W.P. 112-86-01 (B)
Highway 15, District 8 - Kingston

The final Foundation Investigation for the above-captioned project has been completed. The fieldwork was carried out for the proposed structure and detour, and consisted of drilling five sampled boreholes. A preliminary foundation investigation was carried out at this site in 1988 which consisted drilling of four boreholes.

This memo contains preliminary foundation recommendations that should provide sufficient detail for the preparation of your structural planning report.

The subsurface conditions were found to be almost the same as reported in our preliminary report dated 1989 07 14. Our present investigation confirmed the presence of a 1.8m thick layer of organic soil underlying the surficial granular fill material at the north approach. It is expected that most of the settlement at the north approach took place because of the consolidation of the organic soil. This organic layer should be removed during the new construction.

It has been found out that the bedrock slopes down at an estimated 30 degree angle from the south side of the bridge to the north side.

The bedrock surface on the south side of the bridge is about 32m below the ground surface, in the centre of the bridge about 36m below the creek bottom and on the north side of the bridge the bedrock is expected to be at depth in excess of 62m below the ground surface.

As recommended in the preliminary report for this project the proposed structure can be supported on friction piles.

For the purposes of the O.H.B.D.C., and assuming an embedded length of 15m the following values are recommended for No. 36 timber piles.

Factored Axial Capacity at U.L.S. 265 kN/pile (conditional)**
Axial capacity at S.L.S. Type II 175 kN/pile (conditional)**

Alternatively the structure may be supported by end-bearing steel H-piles driven to bedrock or very dense material. The following are recommended design values, as per the O.H.B.D.C.

For Steel Piles HP 310 X 79:

Factored Capacity at ULS = 1150 kN/pile (conditional)**
Bearing capacity at SLS Type II = 825 kN/pile (conditional)**
Ultimate Pile Capacity (for Hiley Formula) = 2475 kN/pile

For Steel Piles HP 310 X 110:

Factored Capacity at ULS = 1600 kN/pile (conditional)**
Bearing capacity at SLS Type II = 1150 kN/pile (conditional)**
Ultimate Pile Capacity (for Hiley Formula) = 3450 kN/pile

**

The design values will apply only if the detour is constructed at least three months before piles are driven for the new bridge construction. Otherwise, the capacities as shown above (for friction and end bearing piles) should be reduced by 10 percent.

The groundwater table was found to be matching with the water level in the creek. A minimum of 1.5m of earth cover is required for frost protection. No major dewatering will be required for the construction.

Backfill to abutments should consists of Granular 'A' or Granular 'B' material for which the following properties are recommended.

Granular 'A'	Y = 22.8 kN/m ³	= 35	kA = 0.27
Granular 'B'	Y = 21.2 kN/mu ³	= 30	kA = 0.33

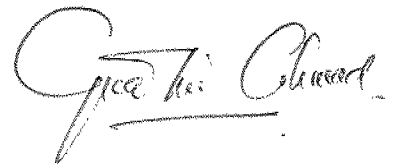
Lateral pressure should be computed in accordance with Section 6.6.1.2.1 of the code. An yielding foundation condition may be assumed.

For the construction of the detour, there would be no stability problem for fills up to 2m high.

The full Foundation Investigation Report, complete with Record of Borehole Sheets and detailed soils descriptions, will be issued as soon as possible.

Regarding the selection of the timber piles or H-piles, we recommend that the most economical alternative should be adopted.

If there are any questions, please contact the undersigned.

A handwritten signature in cursive script, appearing to read "Ken Ahmad", with a horizontal line underneath.

Ken Ahmad, P. Eng.
Foundation Engineer

For

D.H. Dundas, P. Eng.
Sr. Foundation Engineer

memorandum

(613) 544-2220 Ext. 4163



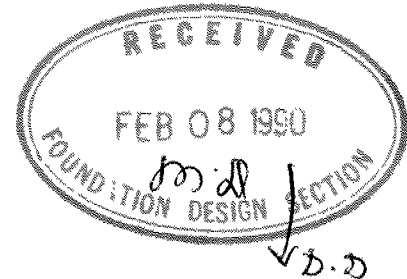
To: Mr. M. Devata
Foundation Design Section
Engineering Materials Office
Downsview

ATTN: Mr. Dave Dundas

FROM: Geotechnical Section
Eastern Region, Kingston

RE: W.P. 112-86-01, Highway # 15
Morton Creek Bridge - Site 16-45
District 08, Kingston

Date: February 07, 1990



Subsequent to my correspondence of 89-01-10 (attached) requesting your Section conduct field investigation for the structure approach fills in conjunction with the investigation for the structure footings, it has been decided to maintain the existing alignment for the new structure and to build a detour around the construction site to the west. I understand plans and profiles have recently been submitted to your office by our Structural Section.

As detailed in the Preliminary Foundation Investigation and Design Report dated 89-07-14, settlement in the order of 15 to 20 cm can be expected along the detour alignment at approaches to the structure.

In view of the above, we would like to modify our original request to include the detour alignment.

Also, the project has surplus rock which can be economically utilized as fill material along the detour alignment; and if left in place, will the proximity of the detour adversely affect long term performance of the existing roadway alignment. These design considerations should also be addressed in your report.

A handwritten signature in cursive script, appearing to read "Robert Scott".

Robert Scott
Soils Supervisor

RS/dka
Attachment
c.c.: H. Kleywegt
G. Chaput