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HWY. No. 132

LOCATION CULVERT REPLACEMENT

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

FILE COPY

Ministry of Transportation and Communications

Ontario

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
PAVEMENT & FOUNDATION DESIGN SECTION

WP 58-80-01 DIST 10

HWY 132 STR SITE

Culvert Replacement

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FOUNDATION INVESTIGATION REPORT

For

Culvert Replacement
Hwy. 132, W.P. 58-80-01
District 10, Bancroft

INTRODUCTION

This report contains the results of a foundation investigation carried out at the site of the above mentioned project. The fieldwork was carried out on June 10, and 11, 1980 and consisted of two sampled boreholes advanced by means of hollow stem augers to depths of 32 feet below the original ground surface and 57 feet below the Hwy. 132 embankment grade.

SITE DESCRIPTION AND GEOLOGY

The site is located on Hwy. 132, 0.2 mi. west of the Renfrew town limits, in the Township of Admaston, County of Renfrew.

The creek under concern originates some 2 miles south of the crossing and empties into the Bonnechere River some 0.5 mi. north of the site. At the Hwy. 132 crossing the creek is located in a valley some 300-500 feet wide, up to 25 feet deep with valley slopes in the order of 1.5H:1V. Outside the valley the topography is generally flat with land use being agricultural and rural residential along the highway frontage. At the time of investigation the creek bed was undefined as a result of temporary culvert installment and a hydrological failure of the existing culvert. Upstream of Hwy. 132 the water had backed-up at the time of the fieldwork forming a pond about 10 feet deep. The top of the existing culvert is barely visible on the downstream side while on the upstream side no evidence of the culvert opening was observed.

Physiographically the site is located in the Ottawa Valley Clay Plains. This region is characterized by extensive deposits of generally soft to stiff marine clay deposits known as Leda clay.

SUBSURFACE CONDITIONS

Parent subsurface conditions consist of a surficial deposit of firm to sensitive grey clay about 25 feet thick. Insitu vane testings gave shear strengths ranging from 320 to 1000 P.S.F., generally in the order of 650 to 800 P.S.F., and sensitivites as high as 38 in this cohesive deposit. The results of the vane shear strength testing are shown on Figure 1. This deposit contains a trace of sand, silt and some organic inclusions and from past experience may be described as a clay of medium to high plasticity, CI to CH zone on the Atterberg Limit chart. This material is known to be only slightly overconsolidated and very highly compressible. Laboratory consolidation testing is presently being carried out.

Underlying this clay deposit is either bedrock or a 6.5 foot thick layer of dense sand and gravel overlying bedrock. The bedrock was not proven during the investigation but is inferred to be at the point of refusal to augering.

The parent subsurface conditions are overlain downstream of the highway embankment by layers, collectively up to 5 feet thick, composed of loose fine to medium sand or loose gravel and cobbles. These surficial deposits are believed to be recent deposits originating from the highway shoulders or existing culvert backfill or from recent remedial measures that were washed through the culvert and deposited downstream of the embankment. The highway embankment itself is composed of a clay fill of medium to high plasticity with 'N' values of 10 to 11 blows per foot which indicates the embankment fill material has undergone a moderate compactive effort.

The boundaries between the various subsoil types are shown on the stratigraphical cross-section on Drawing 588001-A which has been inferred from the borehole information. The location and elevation of the boreholes are shown on Drawing 588001-A as well.

All site photos, borehole log sheets, and laboratory test results will be retained in our files. Copies of this information will be issued to those who so request it.

DISCUSSION AND RECOMMENDATIONS

The Ministry is proposing to replace the culvert under Hwy. 132 located about 0.22 miles west of the Renfrew town limits.

According to available information the existing pipe is a 74 foot long 36" C.S.P. with a 32 foot extension right and a 34 foot extension left. The downstream end of the pipe is effectively covered by the recent sand and gravel deposits mentioned previously. Shovel probing into the pipe indicates the downstream end of the C.S.P. is filled with sand and gravel to a depth of about 2 to 2.5 feet. The probing indicates this sand and gravel debris extends for a minimum distance of five feet into the C.S.P. As noted previously the source of this material is not known but it is suspected the material originated from the granular shoulder on the upstream side of the pipe or from pipe bedding material or as a result of remedial measures.

According to Maintenance personnel, the C.S.P. joints have, in the past, separated. The District Maintenance personnel have also indicated that the pipe has settled some 10"± since construction in 1960.

In the spring of 1980 the culvert became effectively blocked and the water backed-up within inches of the profile grade. A temporary relief C.S.P. was installed about 10 feet above the average ground surface to reduce the backed-up water level.

It is suspected that the culvert failure was a result of substantial differential settlements occuring since original construction. The differential settlements would cause a separation in the C.S.P. joints resulting in eventual misalignment of the C.S.P. sections. This combined with an influx of bedding and backfill would eventually reduce the hydraulic capacity of the culvert.

The fill embankment itself is in the order of 25 feet high. No evidence of instability was observed at the time of the field investigation.

Our recommendations for the embankment stability and culvert design are given in the sections to follow.

Embankment Stability

Stability analysis in terms of total stress were carried out to assess the long term stability of the fill embankment. The parameters used in the analysis were based on the field borings and are given below.

	Elevation Feet	Bulk Density γ P.C.F.	Cohesion C P.S.F.	Internal Friction Angle ϕ o
Fill Material	405 - 380	130	1500	0
Clay	380 - 350	100	700	0
Bedrock	Below 350	150	5000	0

Tension crack was assumed to be 15 feet.

The results of the stability analysis indicates the embankment is marginally stable with side slopes of 2:1.

Culvert

There is a possibility that piping failure has occurred along the existing culvert. For this reason the existing culvert should be entirely removed prior to placing the new culvert.

The new culvert should be provided with a minimum of 12 inches of compacted granular for bedding purposes. The bedding width should be at least D+2 feet. Above the bedding, granular should be shaped to a height of D/10 to accommodate the new pipe shape.

Temporary excavations for pipe removal and placement should be carried out using slopes not steeper than 1:1. Furthermore, the base of the temporary excavations should be limited to 12 feet minimum. Backfill for the culvert and temporary excavations should be native material. There is a possibility that the recent high water levels has saturated portions of the embankment fill material to such an extent so as to make its use as native backfill impractical. The acceptability of this material would be dependent on actual field conditions; it may be necessary to utilize limited amounts of borrow or granular material for backfill purposes. It is important to note that all granular bedding and backfill, if necessary, be provided with a two foot thick clay seal on the upstream side to prevent piping.

Finally, it will be necessary to provide a camber to the new pipe. As previously mentioned we are currently carrying out consolidation tests to assess the time rate settlement of the embankment. These results are not available at the time of writing; however, they will be supplied to the Region in about two weeks time. For contract purposes a note could be added that "the pipe camber will be specified by the M.T.C. field engineer at the time of construction".

M MacLean

M. MacLean, P. Eng.
Project Foundations Engineer

M. D. MacLEAN

M Devata

M. Devata, P. Eng.
Senior Foundations Engineer

June 25, 1980

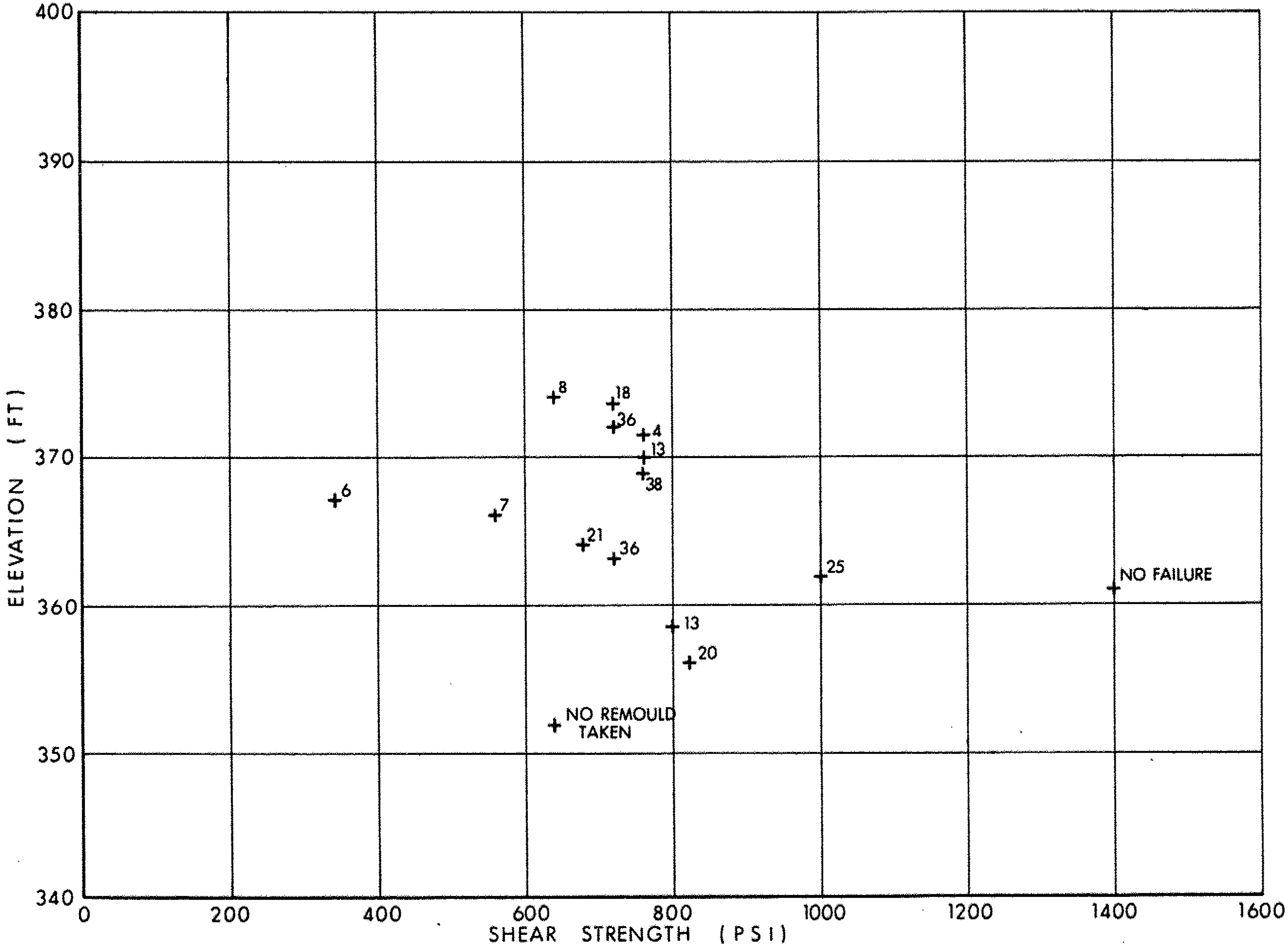
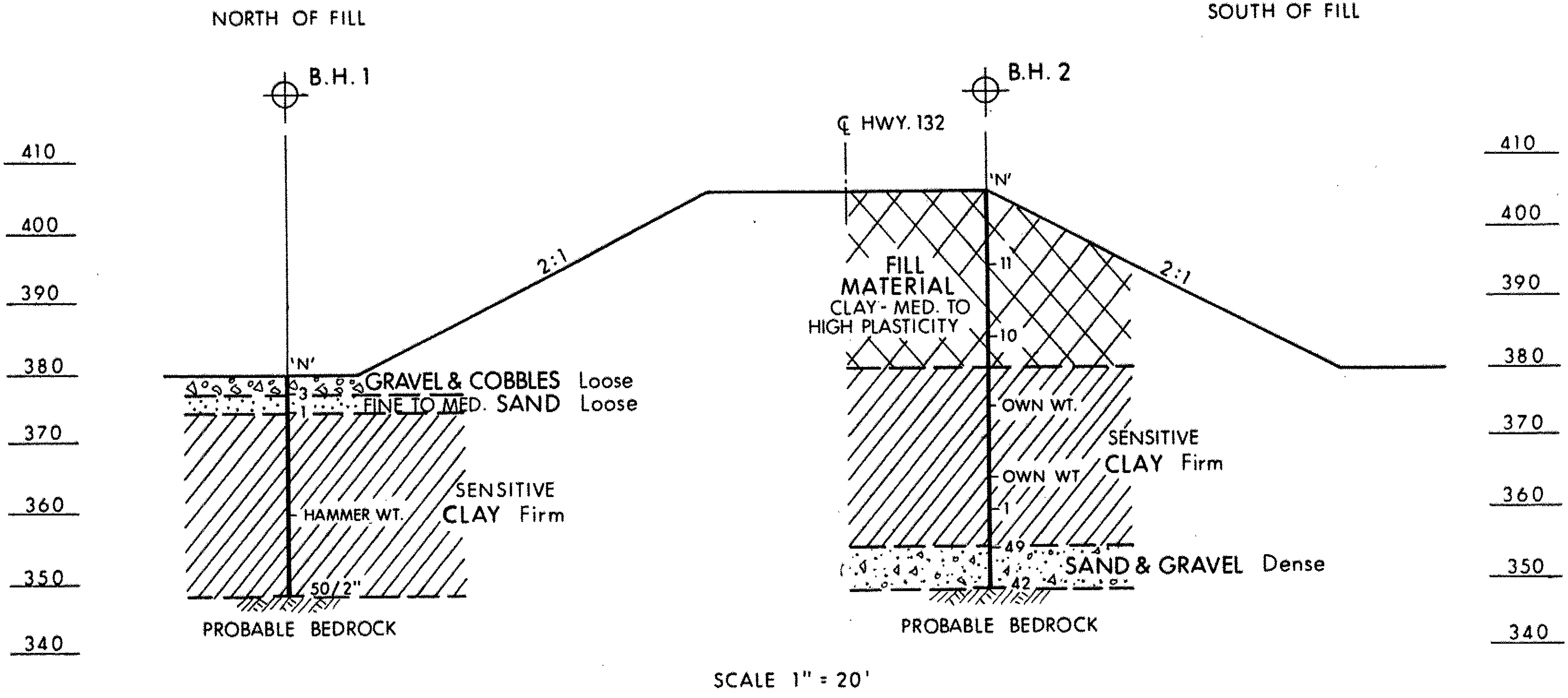


FIG. 1 - SHEAR STRENGTH VS DEPTH

WP 58-80-01



CROSS-SECTION HWY. 132
SUBSURFACE STRATIGRAPHY

DWG. № 588001-A

EXPLANATION OF TERMS USED IN REPORT

'N' VALUE: AN INDICATOR OF SUBSOIL QUALITY. IT IS OBTAINED FROM THE STANDARD PENETRATION TEST (CSA STD. A119.1). SPT 'N' VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 2 INCH O.D. SPLIT-BARREL SAMPLER TO PENETRATE 12 INCHES INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WEIGHING 140 POUNDS, FALLING FREELY A DISTANCE OF 30 INCHES. FOR PENETRATIONS OF LESS THAN 12 INCHES 'N' VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. 'N' VALUES CORRECTED FOR OVERBURDEN PRESSURE ARE DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST (CSA STD. A119.3): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (2" O.D. 60 CONE ANGLE) DRIVEN BY 350 FT-LB IMPACTS ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 12 INCH ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOIL QUALITY: SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH AS FOLLOWS:

S_u (PSF)	0 - 250	250 - 500	500 - 1000	1000 - 2000	2000 - 4000	> 4000
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF SPT 'N' VALUES AS FOLLOWS:

'N' (BLOW/FT)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCK QUALITY: 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 DRILLED IN THAT CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE NATURALLY FRACTURED CORE PIECES, 4"+ 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	2"	2" - 12"	1' - 3'	3' - 10'	> 10'
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS & SYMBOLS

LABORATORY TESTING

TRIAXIAL TESTS ARE DESCRIBED IN TERMS OF WHETHER
THEY ARE CONSOLIDATED (C) OR NOT (U)
ISOTROPICALLY (I) OR NOT (A)
AND SHEARED DRAINED (D) OR UNDRAINED (U)
WITH PORE PRESSURE MEASUREMENTS (BAR OVER SYMBOLS)
EG. $\overline{CIU}$ = CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENT
UNLESS OTHERWISE SPECIFIED IN REPORT ALL TESTS ARE IN COMPRESSION

FIELD SAMPLING

S S	SPLIT SPOON
W S	WASH SAMPLE
S T	SLOTTED TUBE SAMPLE
B S	BLOCK SAMPLE
C S	CHUNK SAMPLE
T W	THINWALL OPEN
T P	THINWALL PISTON
O S	OSTERBERG SAMPLE
F S	FOIL SAMPLE
R C	ROCK CORE
P H	T.W. ADVANCED HYDRAULICALLY
P M	T.W. ADVANCED MANUALLY

EARTH PRESSURE TERMS

μ	COEFFICIENT OF FRICTION
δ	ANGLE OF WALL FRICTION
k_o	COEFFICIENT OF EARTH PRESSURE AT REST
k_A	COEFFICIENT OF ACTIVE EARTH PRESSURE
k_p	COEFFICIENT OF PASSIVE EARTH PRESSURE
i	ANGLE OF INCLINATION OF SURCHARGE
ω	SLOPE ANGLE-BACKFACE OF WALL
β	ANGLE OF SLOPE
N_γ, N_q, N_c	BEARING CAPACITY FACTORS
D_f	DEPTH OF FOOTING
B,L	FOOTING DIMENSIONS

INDEX PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G_s	SPECIFIC GRAVITY OF SOLIDS
e	VOIDS RATIO
e_o	INITIAL VOIDS RATIO
e_{max}	e IN LOOSEST STATE
e_{min}	e IN DENSEST STATE
D_r	RELATIVE DENSITY = $\frac{e_{max} - e}{e_{max} - e_{min}}$
n	POROSITY
w	WATER CONTENT
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
w_S	SHRINKAGE LIMIT
I_p	PLASTICITY INDEX = $w_L - w_P$
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
A_c	ACTIVITY = $\frac{I_p \text{ of soil}}{\% < 2\mu m \text{ Soil Fraction}}$
Om	ORGANIC MATTER CONTENT
S_r	DEGREE OF SATURATION
S	SENSITIVITY = $\frac{S_u \text{ (undisturbed)}}{S_u \text{ (remoulded)}}$

STRENGTH PARAMETERS

ϕ	ANGLE OF SHEARING RESISTANCE
τ_f	PEAK SHEAR STRENGTH
τ_R	RESIDUAL SHEAR STRENGTH
c	COHESION INTERCEPT
$\sigma_1, \sigma_2, \sigma_3$	NORMAL PRINCIPAL STRESSES
u	PORE WATER PRESSURE
u_e	EXCESS u
r_u	PORE PRESSURE RATIO
q_u	UNCONFINED COMPRESSIVE STRENGTH
s_u	UNDRAINED SHEAR STRENGTH
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO
E	MODULUS OF ELASTICITY
G	MODULUS OF SHEAR DEFORMATION
k_s	MODULUS OF SUBGRADE REACTION
m,n	STABILITY COEFFICIENTS
A,B	PORE PRESSURE COEFFICIENTS

NOTE: EFFECTIVE STRESS PARAMETERS ARE DENOTED BY USE OF APOSTROPHE ABOVE THE SYMBOL, THUS:
ϕ' = EFFECTIVE ANGLE OF SHEARING RESISTANCE;
σ' = EFFECTIVE NORMAL STRESS

HYDRAULIC TERMS

h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
j	SEEPAGE FORCE PER UNIT VOLUME
η	COEFFICIENT OF VISCOSITY
k	COEFFICIENT OF HYDRAULIC CONDUCTIVITY
k_h	k IN HORIZONTAL DIRECTION
k_v	k IN VERTICAL DIRECTION
m_v	COEFFICIENT OF VOLUME CHANGE
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX
C_r	RECOMPRESSION INDEX
d	DRAINAGE PATH DISTANCE
T_v	TIME FACTOR
U	DEGREE OF CONSOLIDATION
O_r	OVERCONSOLIDATION RATIO (OCR)

ADDENDUM

To

FOUNDATION INVESTIGATION REPORT

For

Culvert Replacement
Hwy. 132, W.P. 58-80-01 NR
District 10, Bancroft.

A foundation investigation was carried out in June, 1980 for the above project. Because of the urgency for construction a foundation investigation report was issued on July 2, 1980, which as a result of time constraints did not include the consolidation test results and camber recommendations for culvert design pruposes. Upon completion of consolidation testing and subsequent analysis, a teletype was sent on July 23, 1980 to the Region detailing appropriate camber recommendations. This Addendum will serve to document the consolidation test results and analysis and to confirm the camber recommendations.

Consolidation Test Results

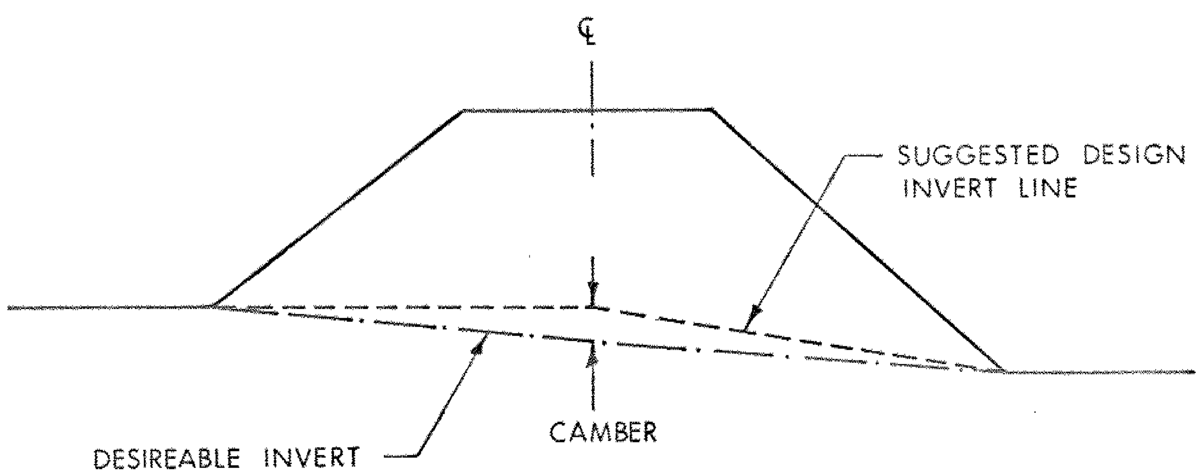
Four consolidation tests were carried out on relatively undisturbed samples obtained from the extensive clay deposit at the site. One test was carried out on a sample obtained beneath the highway embankment; the other tests were performed on samples obtained outside the embankment limits. The test results are shown on the attached Figure 1, and summarized briefly in the following table:

	Outside Embankment Limits			Beneath Embankments
Sample Depth Below Original Ground (feet)	11	16	26	10
Initial Void Ratio	1.20	1.27	1.08	0.96
Preconsolidation Pressure Pc TSF	1.7	2.1	2.1	2.5
Overburden Pressure Po TSF	0.6	0.8	1.3	2.1
Coefficient of Consolidation Cc	0.63	0.90	0.67	0.58

The consolidation results indicate that the stratum is highly compressible and will consolidate under imposed loads greater than the overburden pressure.

Discussion and Recommendations

The consolidation test data was used to estimate settlement under the centre of a 25 foot high earth embankment. The Purdue Method of Stress Distribution was employed to calculate induced pressures due to the embankment loading. Two sets of computations were carried out, one using the consolidation test data from the sample obtained directly under the embankment, the other using the consolidation test data obtained outside the existing area. The computations indicate that due to the orginal embankment loading the underlying clay would consolidate about 16 inches at the centre of the embankment and that since the embankment has been constructed, 70% of the anticipated consolidation has occurred. It is therefore concluded that an additional 5 to 6 inches of consolidation of the clay material will occur in future years.



HWY. 132 CROSS-SECTION
SHOWING CAMBER REQUIREMENTS
(NOTE: DISTORTED SCALE)

In view of the foregoing, a camber of 0.5 feet (6 inches should be applied to the replacement culvert under the centerline of the embankment). See sketch above.

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Project Foundations Engineer

M. Devata

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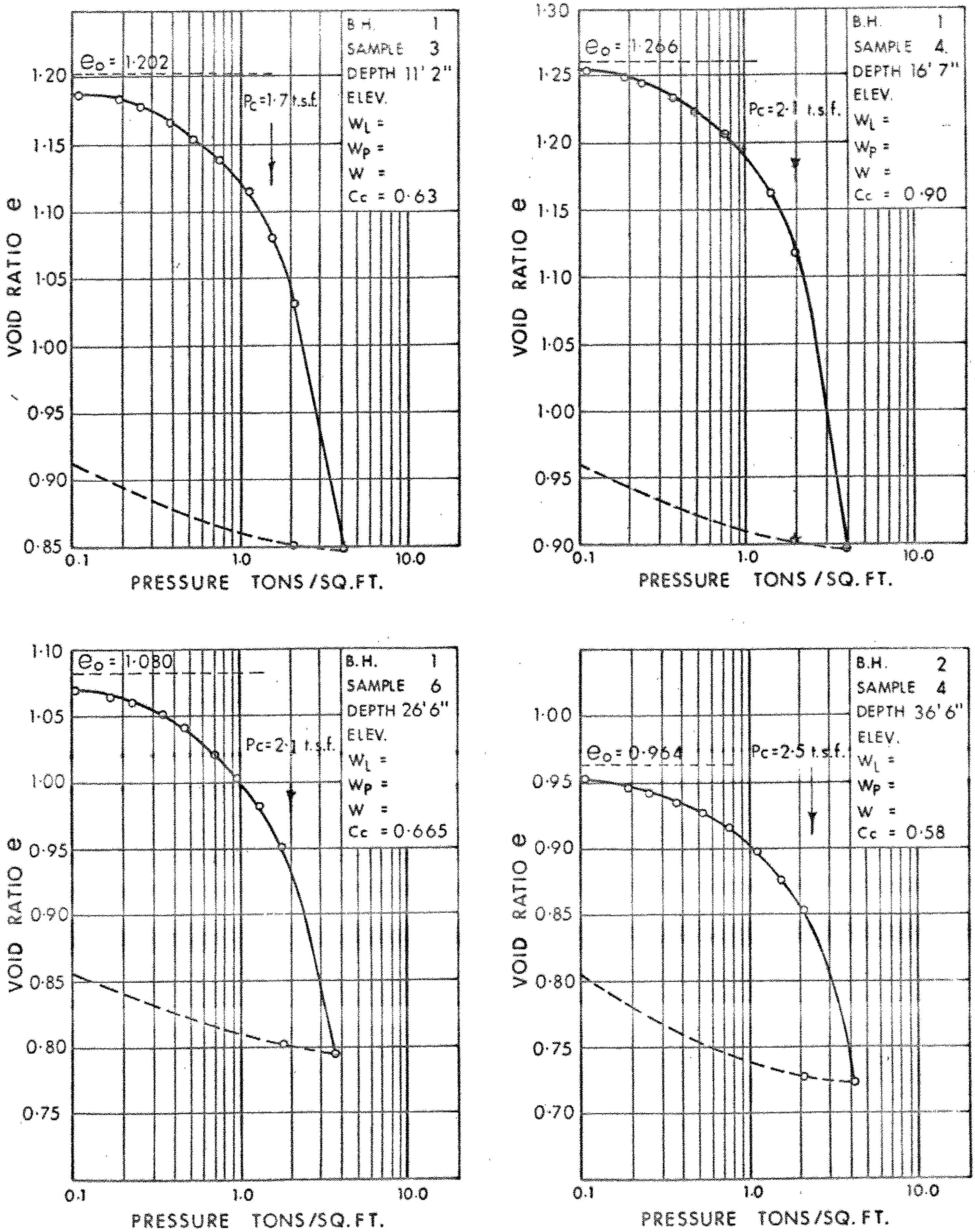
Senior Foundations Engineer

MM/MD/mkq



B.H. 1
SAMPLE 3
DEPTH 11' 2"
ELEV.
W_L =
W_P =
W =
Cc = 0·63
e_o = 1·202
P_c = 1·7 t.s.f.
VOID RATIO e
PRESSURE TONS/SQ.FT.
B.H. 1
SAMPLE 4
DEPTH 16' 7"
ELEV.
W_L =
W_P =
W =
Cc = 0·90
e_o = 1·266
P_c = 2·1 t.s.f.
VOID RATIO e
PRESSURE TONS/SQ.FT.
B.H. 1
SAMPLE 6
DEPTH 26' 6"
ELEV.
W_L =
W_P =
W =
Cc = 0·665
e_o = 1·080
P_c = 2·1 t.s.f.
VOID RATIO e
PRESSURE TONS/SQ.FT.
B.H. 2
SAMPLE 4
DEPTH 36' 6"
ELEV.
W_L =
W_P =
W =
Cc = 0·58
e_o = 0·964
P_c = 2·5 t.s.f.
VOID RATIO e
PRESSURE TONS/SQ.FT.

FIG. 1