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STR. SITE No. 34-111

HWY. No. 58

LOCATION CNR OVERPASS

No OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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WP 422 - 97 - 01

FOUNDATION INVESTIGATION AND DESIGN REPORT FOR HWY 58 - CNR OVERPASS WELLAND, ONTARIO MTO SITE 34-111, DISTRICT 4 BURLINGTON - CENTRAL REGION

Ref. No. G-98.1003
November1998

Prepared for:

MTO c/o R.V. Anderson Associates Limited
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Distribution

10 Copies - R.V. Anderson Associates Limited
2 Copies - Geo-Canada Ltd.

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FOR HWY 58 - CNR OVERPASS
WELLAND, ONTARIO
MTO SITE 34-11[illegible] DISTRICT 4
BURLINGTON - CENTRAL REGION

FOUNDATION INVESTIGATION REPORT

FOR HWY 58 - CNR OVERPASS

WELLAND, ONTARIO

MTO SITE 34-111 DISTRICT 4

BURLINGTON - CENTRAL REGION

1.0 INTRODUCTION

This report summarizes the factual information obtained from a foundation investigation program performed at the above-mentioned structural site. The work was carried out at the request of the Ministry of Transportation of Ontario and authorization to proceed with the investigation was received from the Ministry's Structural Consultant, R.V. Anderson Associates Limited.

The field work was carried out on November 9, 10 and 11, 1998, and consisted of two (2) sampled boreholes and a test pit. The results of the investigation are described in the following sections of this report.

2.0 SITE AND GEOLOGY

The site is located at the crossing of Hwy 58 and the CNR Canal Spur Line, Mile 2.10 off Mile 20.86, Cayuga Subdivision, in the southern outskirts of the City of Welland. The

.../...

highway, at this location, runs on an about 8 m high embankment and crosses the CNR tracks on a three-span bridge structure, approximately 300 m north of Forks Rd. The area in the vicinity of the site has a rural character with widely spaced scattered residential homes along both sides of the road.

The topography around the site is flat and level, with the exception of the highway embankment. Vegetation in the area consists of some large trees mixed with copses of small trees and shrub.

Physiographically, the site is located in the Haldimand clay plane which is characterized by deep deposits of lacustrine clays. The clay, which is of the order of 30 m thick, was laid down by glacial Lake Warren. The glacial lake phase was possibly interrupted by two major retreats of the ice front which resulted in two different deposits: a non or faintly stratified, relatively silty, homogeneous deposit laid down with the ice front fairly close; and heavily stratified, very clayey deposits laid down when the ice front had retreated some distance. The deposit is lightly over-consolidated. The pre-consolidation pressure is about 100 to 150 kPa above present overburden pressure. The clay is underlain by sandy-gravelly glacial drift, followed by shales and dolomites of the Paleozoic era.

.../...

3.0 METHOD OF INVESTIGATION

The investigation in the field consisted of putting down two (2) exploratory boreholes in front of the existing south bridge pier. Borehole locations are shown on Drawing No. 1. Boreholes could not be drilled at the north pier because of its proximity to the railroad tracks. The field work was under the full time supervision of a technologist and the borings were advanced with a bombardier mounted power auger machine. The boreholes were extended to a depth of 9.6 m, to which depth soil samples were recovered at 0.75 m intervals from the ground surface to a depth of 6 m, and at 1.5 m intervals below. In-between the soil samples, the undrained shear strength of the soil was measured in situ by field vane tests. In one of the boreholes, a piezometer was installed to monitor the groundwater level. The boreholes were backfilled with soil cuttings, placing bentonite clay plugs at regular intervals as an additional means of sealing the borehole.

On November 11, 1998, a test pit was dug adjacent to the south bridge pier. The purpose of the test pit was to examine the foundation level of the existing retaining wall and pier footings, the type of soil or fill below the retaining wall foundations, and the condition of the concrete below the ground surface. In the test pit, measurements were taken of the existing foundations, the soil beneath the retaining wall footing was sampled and in situ strength measurements of the soil were taken with a pocket penetrometer. The test pit was dug with a small rubber tired backhoe under the supervision of senior engineers from R.V. Anderson Associates and Geo-Canada Ltd. and the protection of a CN flagman.

.../...

The samples recovered from the boreholes and the test pit were forwarded to Geo-Canada's laboratory, where they were re-examined and selectively tested for their natural moisture contents and consistency limits.

4.0 SUMMARIZED SUBSURFACE CONDITIONS

In general, reasonably uniform subsurface conditions were encountered in the boreholes. Underlying the ground surface is a reddish-brown to grey coloured silty clay to clay deposit with a layered structure. The clay has a very stiff to hard consistency near the ground surface, but is becoming stiff to firm with increasing depth. Although the boreholes were terminated 9.6 m below ground surface in the clay deposit, from previous borings it is known that the deposit extends to a depth of 24 m, where it is underlain by a thin layer of gravelly sand, followed by the Paleozoic bedrock, a dolomitic limestone.

Further details of the subsurface conditions are shown on the individual borehole logs (Enclosures 1 and 2) and are summarized in the form of a section through the boreholes on Drawing No. 1.

5.0 DESCRIPTION OF SOIL STRATA

5.1 Silty Clay to Clay

The significant native soil deposit underlying the site is a reddish-brown to reddish-grey coloured silty clay to clay, which extended to the full depth of the investigation. The clay has a layered structure, confirming its lacustrine origin. It also contains seams and pockets of silt.

The natural moisture content of the deposit ranges from 22 to 40% and is generally between the plastic and liquid limits of the soil. Atterberg tests indicate liquid limits of 50 to 63%, plastic limits of 21 to 27% and plasticity indices of 29 to 36. These test results indicate a clay of high plasticity.

The undrained shear strength of the deposit was measured in the field by in situ field vane tests. Tests performed near the surface gave shear strength values greater than 100 kPa. With depth, the values gradually decreased, reaching a minimum value of about 28 kPa at a depth of 6.5 to 8.5 m before increasing again. The sensitivity of the clay was measured to range from 1.3 to 3.0, indicating a clay of low to medium sensitivity. The in situ shear strength values are plotted on the borehole logs. From these, the consistency of the clay is inferred to range from very stiff to firm. From the standard penetration test (SPT) results which gave 'N' values ranging from 42 to 4 blows per 0.3 m, the range of consistency is inferred to be hard to firm.

Based on previous work carried out in the vicinity on the same clay deposit, the clay is known to be over-consolidated. The pre-consolidation pressure is 100 to 150 kPa above the existing overburden pressure. The coefficient of compressibility, m_v, in the pre-consolidation range is known to range between 0.00015 and 0.00009 m^2/kN.

.../...

5.2 Groundwater Conditions

Based on observations made in the field and the monitoring of the piezometer, the position of the groundwater table is believed to be at a depth of about 2.9 m below ground surface. This is also confirmed by the colour change of the soil from reddish-brown to reddish-grey at about the same depth.

6.0 TEST PIT

The observations made in the test pit dug adjacent to the east column of the south pier are described on the Test Pit Log presented as Enclosure 3. The log shows the south face of the test pit where the footing of the retaining wall was exposed. On the other sides of the excavation, very stiff to stiff clay was exposed, which stood unsupported as a vertical cut. Seepage through the soil was minimal but water entered the excavation from a 150 mm thick crushed stone layer beneath the retaining wall footing. Initially, the flow was heavy but rapidly decreased with time.

Due to the proximity of the north pier to the rail tracks, a test pit at that location could not be dug.

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FOR HWY 58 - CNR OVERPASS
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FOUNDATION DESIGN REPORT

FOR HWY 58 - CNR OVERPASS

WELLAND, ONTARIO

MTO SITE 34-111 DISTRICT 4

BURLINGTON - CENTRAL REGION

GWP 420-97-00

7.0 DISCUSSION AND DESIGN RECOMMENDATIONS

7.1 The Proposed Works

A condition survey carried out in 1997 indicated that both the north and south piers are in poor condition. The concrete in both piers is weathered, cracked and with areas of delamination, honeycombing and scaling. As part of the repair to the piers, it is proposed to install two additional columns between the existing pier columns.

We have been informed that the total dead and live load carried by each of the piers is of the order of 2450 kN. This load is supported on three approximately 2 m by 2 m size footings under which the average bearing pressure was calculated to be about 200 kN/m^2, excluding any pressure from the retaining wall. After the repairs, the total load on the piers will increase to 2790 kN, an increase of about 14%. Due to the structural stiffness of the pier, this additional load will be carried almost evenly by the old and new footings supporting the five

.../...

pier columns. The resulting pressure, in this case, under the column footings is estimated to be about 140 kPa, excluding again any pressure from the retaining wall. On the other hand, if all the additional load is assumed to be carried by the new columns and footings, then the pressure under these 2 m x 2 m footings will be of the order of 42.5 kPa.

Behind the piers on each side, there is a reinforced concrete earth retaining structure. The base of the footing of the retaining wall is about 1875 mm above the foundation level of the pier. The toe of the retaining wall footing extends over the footing of the pier and, as shown on Figure 1, the gap between the pier column and the retaining wall footing is only 90 mm.

The width of the retaining wall footing is unknown and, therefore, the pressure under the wall footing cannot be calculated.

The purpose of the geotechnical review is:

- to provide recommendations for the foundation design of the reconstructed bridge pier;
- estimate the settlements of the pier after the reconstruction;
- provide estimates of the horizontal earth pressures acting on the retaining wall and on temporary earth support systems during construction;
- analyze the stability of the embankment during construction; and

.../...

- make recommendations for carrying out the excavation for the repair works.

7.2 Foundations

Based on the measured shear strength profile, the factored geotechnical bearing resistance at the level of the existing pier foundations is 530 kPa at ULS. The allowable bearing pressure at the SLS is 200 kPa for the 2 m wide footings.

It is recommended that the new pier column footings be placed at the same level as the existing pier footings, i.e. about 3.2 m below present ground surface. At this depth, they will have adequate earth cover for frost protection.

7.3 Settlement

Depending on the load distribution between the existing and the new pier columns, the effective stress beneath the foundation level after the repair work is estimated to either decrease or to increase by less than 50 kPa. As the pre-consolidation pressure of the Haldimand clay in the Welland area is about 100 to 150 kPa above the existing overburden pressure, this pressure increment is not expected to cause settlements in excess of 15 mm. This, in our opinion, should be within the acceptable limits of the existing bridge structure.

.../...

It is believed that the relatively large settlements observed in the past have been caused by the dewatering activities during the construction of the nearby Welland Canal and the resulting increases in the effective stresses in the soil.

7.4 Horizontal Earth Pressure

Assuming that the backfill behind the retaining wall is clay similar to that found below the foundations of the retaining wall, the total horizontal earth pressure on the 4.4 m high retaining wall measured from the top of the wall to the underside of the footing is estimated to be 74 kN/m length of wall. This value is based on the following soil parameters:

Unit weight of soil	19 kN/m^2
Effective angle of shearing resistance	28°
Effective cohesion	5 kPa
Slope behind retaining wall	2.5H:1V

The earth pressure distribution on the retaining wall can be assumed to be triangular, with the resultant acting 1.5 m above the base of the wall. The direction of the earth pressure can be assumed to be parallel to the slope of the backfill.

On braced shoring systems for the temporary support of excavations, the earth pressure distribution shall be taken to be trapezoidical as shown on Figure 4. The intensity of the

.../...

earth pressure shall be taken as 22 kPa. To this, pressure from any live load (e.g. train) shall be added.

7.5 Stability of Embankment

If the excavation is carried out in short sections as recommended and discussed in the next section of this report then, in our opinion, the stability of the embankment in front of the bridge abutment will not be jeopardized. Although the present factor of safety of the existing embankment is not known and has not been calculated, based on the observed satisfactory past performance of the embankment, it is considered to be adequate. The construction of the repairs, if carried out as recommended, should not cause a reduction in the presently existing safety factor and should not result in the instability of the embankment.

8.0 CONSTRUCTION

In connection with the proposed excavations for the repair work consideration will have to be given to the stability of the retaining wall and the embankment retained by the wall. A 3.2 m deep excavation, if carried out along the full length of the pier could cause the horizontal displacement of the retaining wall and/or the rotational failure of the embankment behind the wall. To prevent this, it is recommended that the excavation be carried out in five stages, each stage being approximately only 2 m wide. The recommended sequence of the construction is described below and is also shown graphically on Figures 2 and 3 .

.../...

8.1 Proposed Construction Sequence

- Install soldier piles on approximately 2 m c/c. Exact location and depth of pre-drilled holes to be determined by contractor's engineer, based on his analysis and his proposed construction method.
- Install raker bracing as required.

- Carry out excavation for Stage I, using a combination of machine and hand excavation. For extent of excavation see Figure 3.
- Install timber lagging between soldier piles as excavation progresses with depth.
- Hand clean base of excavation before pouring concrete, remove water accumulation.
- Backfill excavation with 30 MPa high-early strength concrete to at least 500 mm above the base of the footing.
- Proceed with Stage II as for Stage I above.
- Proceed with excavation to the top of the existing footings in Stages III and IV, when concrete in Stages I and II has reached at least 20 MPa strength.
- Install concrete jacket around existing pier column to the top of retaining wall footing.
- Backfill excavation for Stages III and IV with well compacted granular fill or 2 MPa non-shrink fill.
- Carry out excavation and installation of concrete jacket and backfilling for Stage V, as described for Stages III and IV above.

.../...

- Form and install new pier columns.
- Proceed with rest of the repair work.

8.2 Dewatering

As the native soil and the backfill beneath the retaining wall is clay, only minor seepage is expected into the excavation, mainly from the thin crushed stone layer below the retaining wall. We do not foresee the need for special dewatering measures, however, submersible pumps should be available to remove any water accumulation from the excavation before the concrete is poured.

9.0 STATEMENT OF LIMITATION

The Statement of Limitation, as quoted in Appendix "B", is an integral part of this report.

GEO-CANADA LTD.

Ivan P. Lieszkowszky, P. Eng.

James Ng, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
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Encl.

D15-reports/G-98 1003A RVA

APPENDIX "A"

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

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

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

APPENDIX 'B'

Statement of Limitation

The conclusions and recommendations in this report are based on information determined at the borehole locations. Soil and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation.

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 of alignment and elevations stated in the report. Since all details of the design may not be known to us, in our analysis certain assumptions had to be made. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis. We recommend also that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the boreholes. In cases where these recommendations are not followed, the company's responsibility is limited to interpreting accurately the information encountered at the boreholes.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the design engineer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. 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 conclusions as to how the subsurface conditions may affect their work.

ENCLOSURES

Ministry of Transportation Ontario

ENCL. 1

RECORD OF BOREHOLE No 5

METRIC

W P 422-97-01 LOCATION CNR OVERHEAD, SOUTH PIER, SITE 34-111 ORIGINATED BY L.D.

DIST 4 HWY 58 BOREHOLE TYPE AUGERING (Solid Stem) COMPILED BY I.P.L.

DATUM N.A. DATE NOVEMBER 9, 1998 CHECKED BY I.P.L.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Ground Surface							
0.0	SILTY CLAY to CLAY		1	SS	18			
			2	SS	19			
	Reddish Brown		3	SS	10	▼	-2	
	Grey		4	SS	27			
	Some silt pockets laminated structure.		5	SS	42			
			6	SS	24		-4	
	very stiff to hard		7	SS	13			
	stiff		8	SS	9		-6	
	firm		9	SS	6			
			10	SS	4		-8	
			11	SS	4			Date W.L. Nov.9 1.8
9.9	End of Borehole.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ

+3, ×5 : Numbers refer to Sensitivity

20 15 ⊕ 5 (%) STRAIN AT FAILURE 10

Ministry of Transportation
Ontario

ENCL. 2

RECORD OF BOREHOLE No 6

METRIC

W P 422-97-01 LOCATION CNR OVERHEAD, SOUTH PIER, SITE 34-111 ORIGINATED BY L.D.

DIST 4 HWY 58 BOREHOLE TYPE AUGERING (Solid Stem) COMPILED BY I.P.L.

DATUM N.A. DATE NOVEMBER 10, 1998 CHECKED BY I.P.L.

SOIL PROFILE: ELEV DEPTH	DESCRIPTION (Ground Surface)	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	SILTY CLAY to CLAY		1	SS	12						
			2	SS	16						
	Some silt pockets laminated structure.		3	SS	13		-2				
			4	SS	24						
			5	SS	21						
	stiff to very stiff — Reddish Brown		6	SS	20		-4				
	stiff — Grey		7	SS	5			+			
	firm		8	SS	6		-6	+			
			9	SS	4			+			
						bentonite seal					
			10	SS	5		-8	+			
						Piezometer					Date W.L. Nov.10 3.. Nov.12 2..
			11	SS	4			+			
9.8	End of Borehole.										

+[3], ×[5] : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

LOG OF TEST PIT

CLIENT: R.V. Anderson

JOB No.: G-98.1003

PROJECT: Hwy 58/CN Spur Line

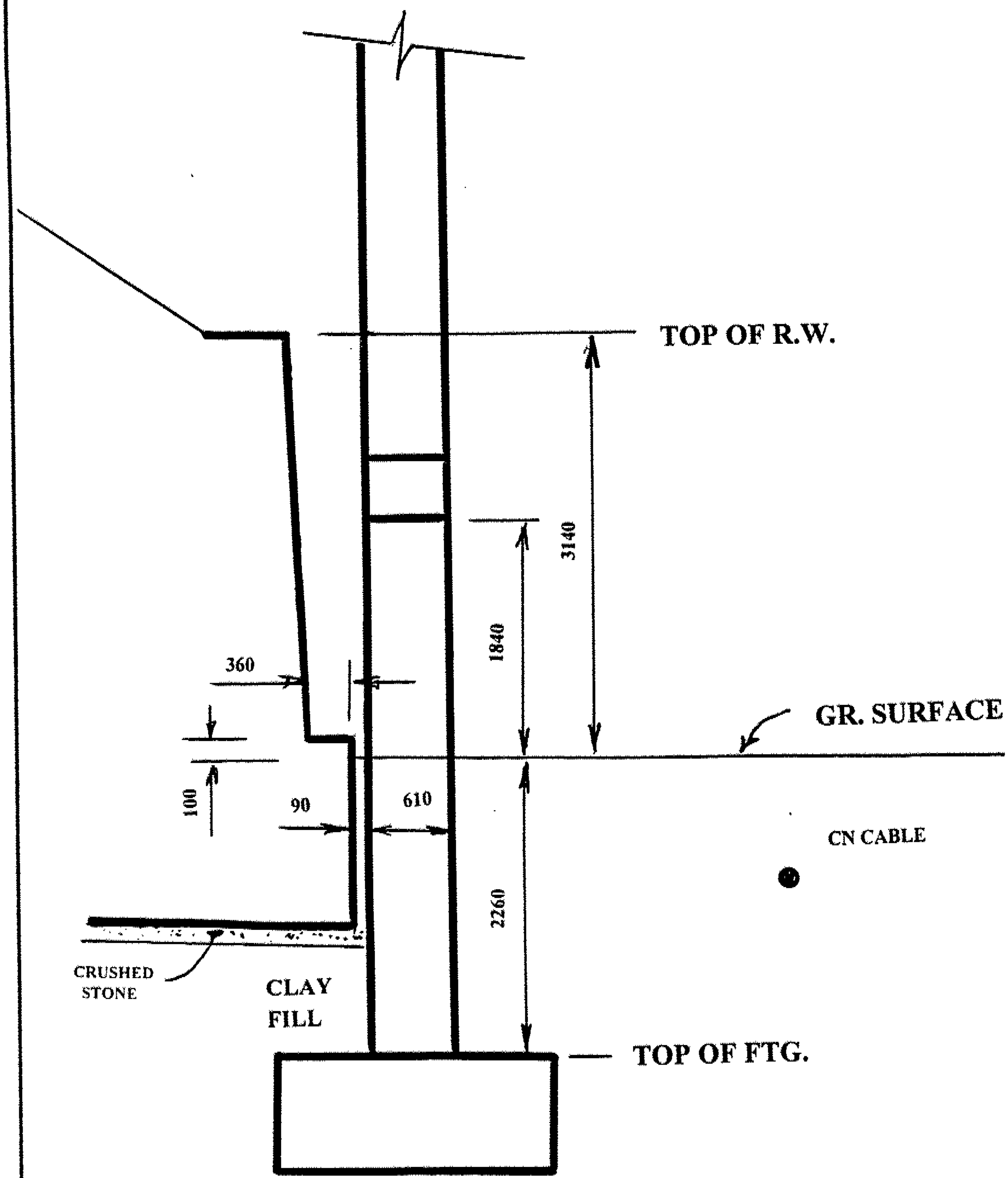
DATE: November 12, 1998

LOCATION: South Pier, Between East and Centre Columns

ELEVATION: N/A

DEPTH mm	DESCRIPTION	SYMBOL	GR. WATER	SAMPLES	q_u TESTS	
	Ground Surface					
0.000	CONCRETE RETAINING WALL					Top of R.W. 3140 mm above ground surface. Top of R.W. footing 100 mm above ground surface
1300	BOTTOM OF R.W. FOOTING					
	CRUSHED STONE		*			* Heavy ground water flow. Rate of flow decreases with time.
1450	CLAY				192	
	FACE OF CONCRETE COLUMN				373	q_u = unconfined compres. strength KPa
	750				258	
2260	TOP OF PIER FTG.				230	
2460	END OF TEST PIT					NOTES: Test Pit 1.8 m x 2.0 m in plan. Pit open for 1 hr. Sides stable at vertical cut.
	ALL DIMENSIONS IN MILLIMETERS					

FIGURES



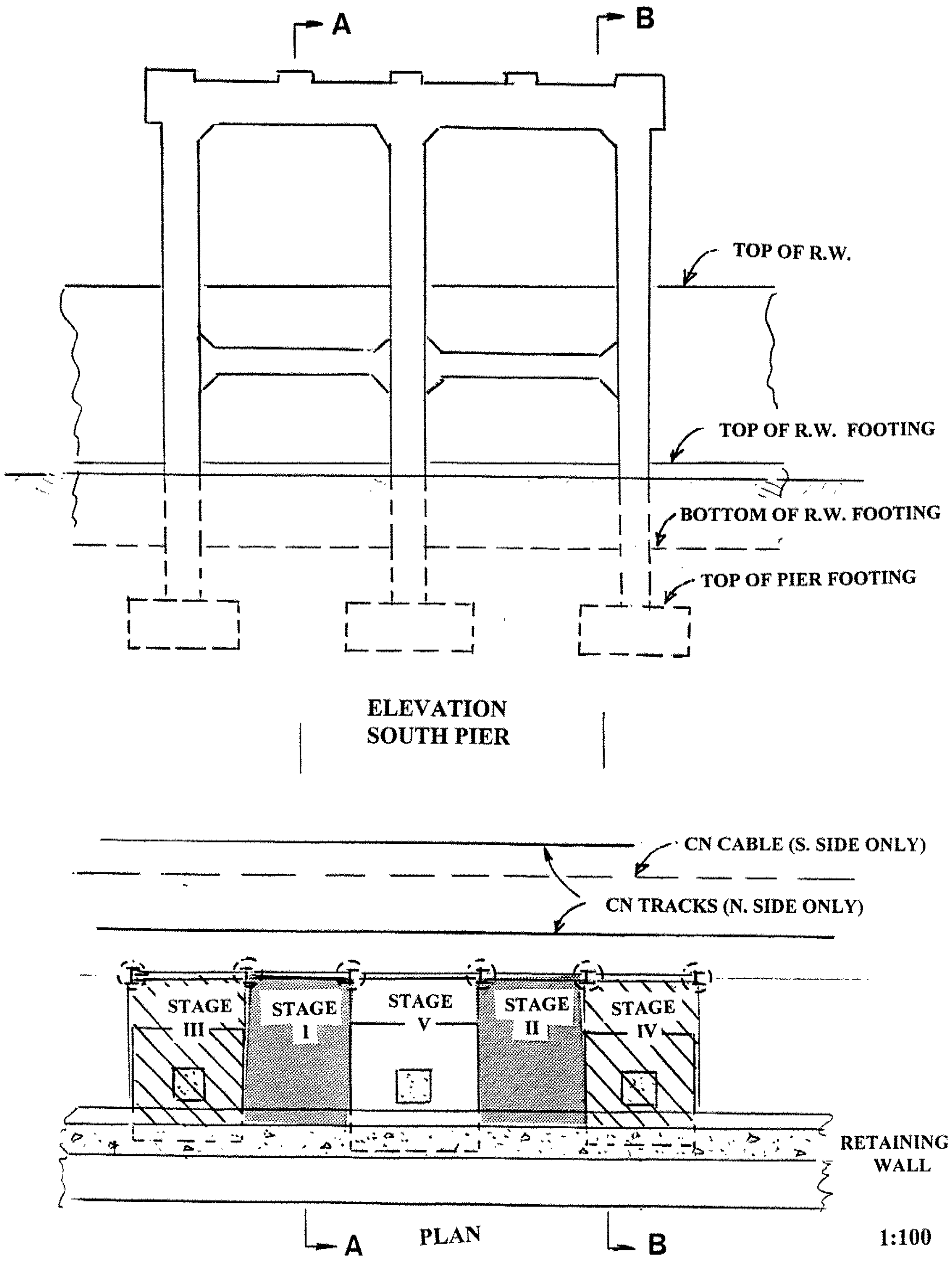
ALL DIMENSIONS
IN MILLEMETERS

1:50

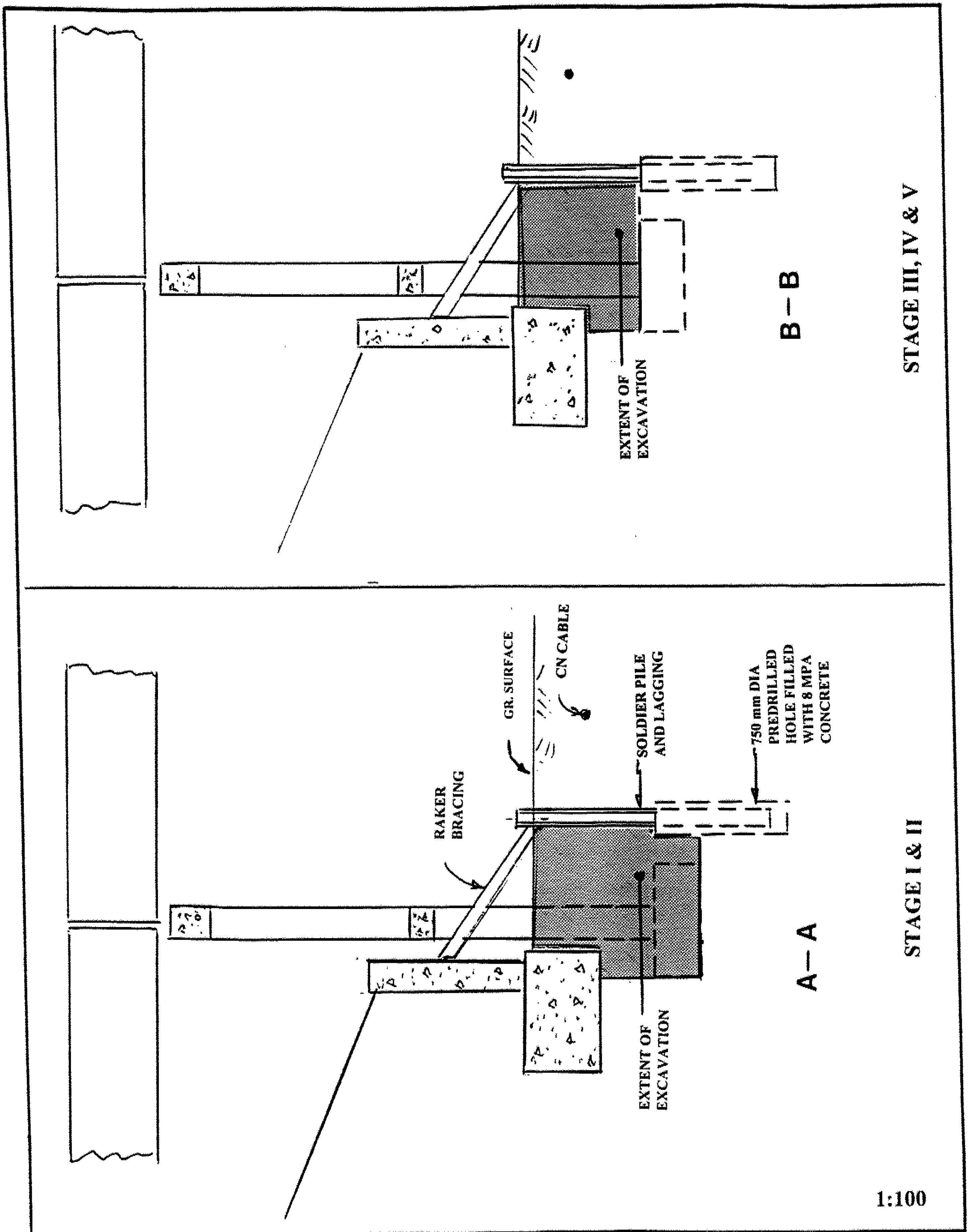
HIGHWAY 58 - CN OVERPASS
SECTION THROUGH S. PIER AND RETAINING WALL

G-98.1003

FIG. 1



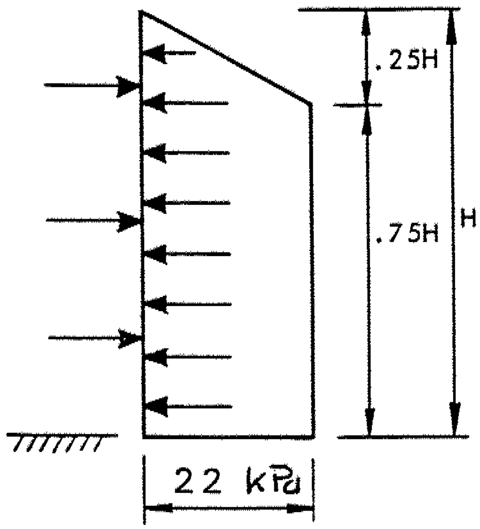
HIGHWAY 58 - CNR OVERPASS	G-98.1003 FIG. 2



HIGHWAY 58 - CNR OVERPASS SECTIONS

G-98.1003

FIG. 3



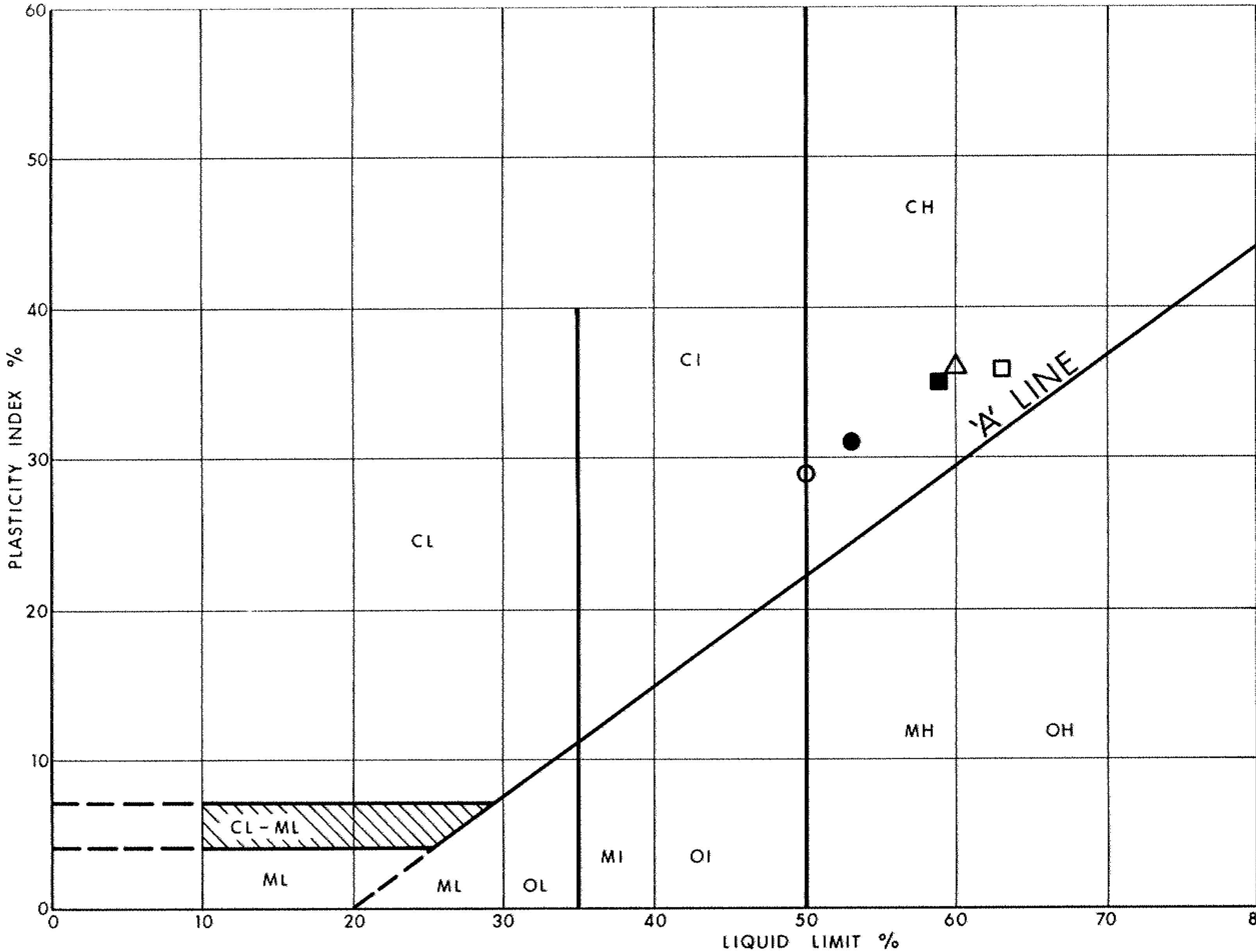
CLAY

NOTES:

1. CHECK SYSTEM FOR PARTIAL EXCAVATION CONDITION
2. IF THE FREE WATER LEVEL IS ABOVE THE BASE OF THE EXCAVATION THE HYDROSTATIC PRESSURE MUST BE ADDED TO THE ABOVE PRESSURE DISTRIBUTION
3. IF SURCHARGE LOADINGS ARE PRESENT AT OR NEAR THE GROUND SURFACE THESE MUST BE INCLUDED IN THE LATERAL PRESSURE CALCULATION.

EARTH PRESSURE DISTRIBUTION ON BRACED SHEETING	G-98.1003 FIG. 4

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
5	3	○
5	5	□
5	7	△
5	10	●
5	11	■

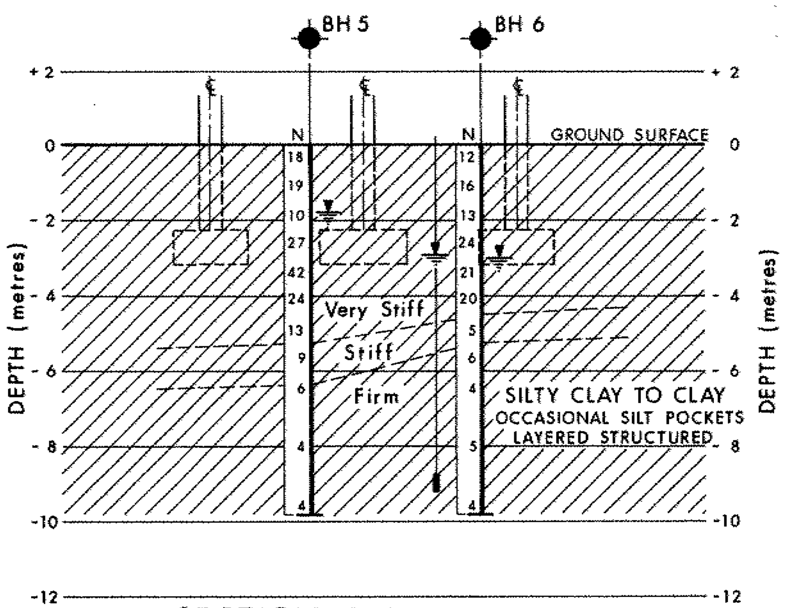
Ministry of Transportation
Ontario

PLASTICITY CHART
CLAY

FIG No 5
WP 422-97-01

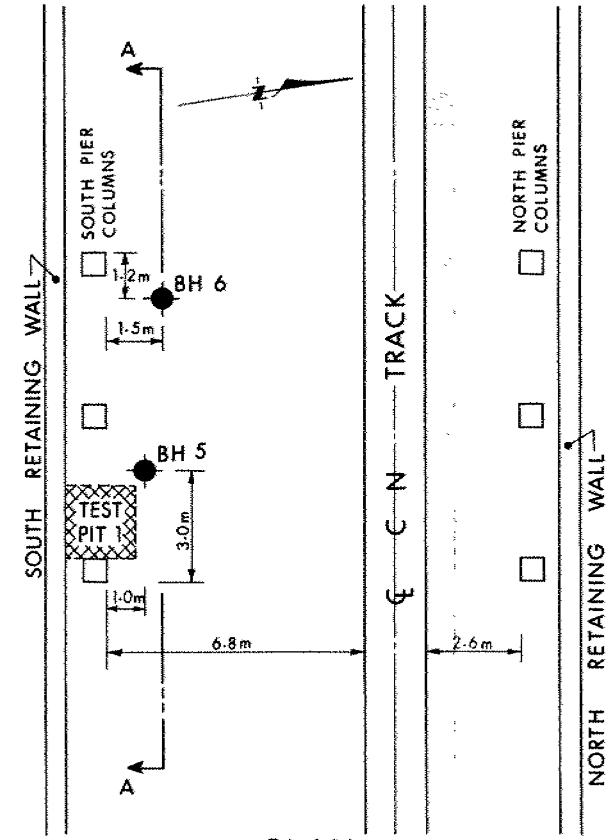
DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10



SECTION A-A (SOUTH PIER)

SCALE 2m 1 0 2 4m



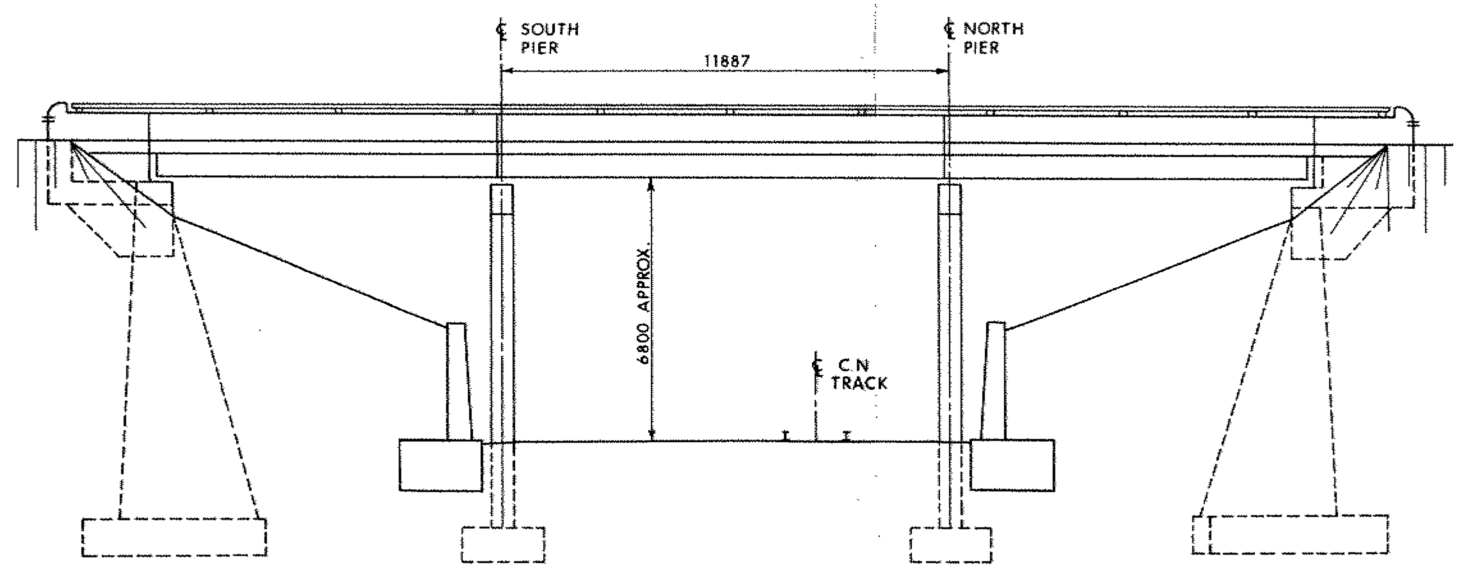
PLAN

SCALE 2m 1 0 2 4m

METRIC

DIMENSIONS ARE IN METRES AND / OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

NOTE:
For detailed information of Test Pit 1 refer to Log of Test Pit, enclosure 3.



ELEVATION

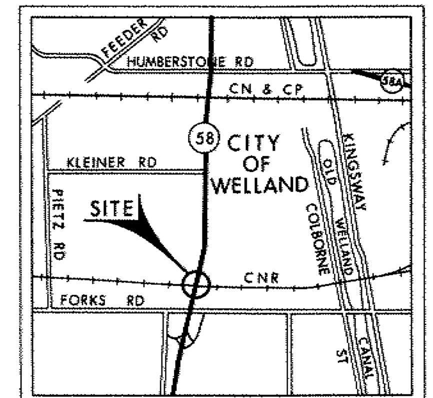
N. T. S.

CONT No
WP No 422-97-01

C N R OVERHEAD
PIER REHABILITATION
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

Geo-Canada Ltd.



KEY PLAN

SCALE 0·5 km 0 0·5 1 km

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)
- W L at time of investigation Nov 1998
- W L in Piezometer
- Piezometer

No	ELEVATION	STATION	OFFSET
FOR BORE HOLE LOCATIONS REFER TO PLAN			

=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

Geocres No

HWY No 58			DIST 4
SUBM'D IPL	CHECKED IPL	DATE Nov 28, 1998	SITE 34-111
DRAWN	CHECKED	APPROVED	DWG 1