

GEOCRETS No. _____

DIST. _____ REGION _____

W.P. No. _____

CONT. No. _____

W. O. No. _____ 93-11003

STR. SITE No. _____ 14-76

HWY. No. _____ Local

LOCATION _____ Vidal Street Bridge

_____ Retaining Wall

No of PAGES - _____ Sarma

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____



Ministry
of
Transportation

FILE No. _____ **DATE** _____

REMARKS _____

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MEMORANDUM



To: A.A. Witecki
Municipal Engineer
Approvals Section
7th Floor, Atrium Tower

Date: May 7, 1993

Attn: A.J. Lee, Sr. Project Engineer

From: Foundation Design Section
Room 315, Central Bldg.

Tel: 235-3731
Fax: 235-5240

Re: Review of Retaining Wall
Slope Stability Analysis
Vidal Street Bridge, Sarnia
W.O. 93-11003, Site 14-76
District 1, Chatham

Further to your memo dated May 3, 1993, the slope stability analysis prepared by Golder Associates Ltd. has been reviewed by this office.

Based on our review, it is our opinion that the overall stability of the proposed slopes beneath the Vidal Street Bridge is considered to be acceptable based on the total and effective stress analyses and provided appropriate precautions are taken during construction. However, it should be noted that the length of the open construction cut for retaining wall installation, at any given time, be limited to 10 m between chainages 9+640 and 9+700 as proposed by Golder Associates Ltd. to ensure the stability during construction. It is also recommended that field monitoring of lateral and vertical movement for both the bridge piers and the slopes should be carried out before, during and after construction.

We have no further comments. If you have any questions, please contact this office.

Tae C. Kim
Tae C. Kim, P. Eng.
Senior Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

MD\TCK\jb

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (416) 567-4444
Fax (416) 567-6561



April 12, 1993

921-1322V

Hatch/Mott MacDonald Joint Venture
2800 Speakman Drive
Mississauga, Ontario
L5K 2R7

Attention: Mr. P. Tavares, P. Eng.

**RE: SLOPE STABILITY ANALYSIS
VIDAL ST. BRIDGE - SARNIA APPROACH
ST. CLAIR RIVER TUNNEL PROJECT
SARNIA, ONTARIO**

Dear Sirs:

As requested, this letter addresses item 2 of a memorandum, dated February 23, 1993, from the Ministry of Transportation Foundation Design Section to their Structural office concerning the Vidal St. Bridge. The item requests assessment of the slope stability of the Sarnia approach cut, at the bridge location, during and after construction.

INTRODUCTION

Site investigation was carried out by Golder Associates for the St. Clair River Tunnel Project in the summer of 1992 and is summarized for the Sarnia approach cut in Golder Associates report 921-1322B-1. In addition, two associated reports (921-1322B and 921-1322B-3) were issued concerning the geotechnical aspects of the open cut design for the Sarnia approach. These reports include drained and undrained stability analyses of existing and proposed slopes on both the north and south sides of the proposed cut. However, the slope area directly under the Vidal St. Bridge was not included in our terms of reference, although sections immediately east and west of the bridge at Chainages 9+600 and 9+708 were analyzed.

This letter presents and discusses the results of slope stability analyses carried out for the slope beneath the Vidal St. Bridge. A summary of strength parameters and modelling assumptions used in the analyses are also provided.

DESIGN PARAMETERS

General design parameters for the St. Clair Till derived from the site investigation are shown in Table 1.

For short term stability of slopes (i.e. during construction), the most relevant design parameter is the undrained shear strength of the St. Clair Till. For long term stability (i.e. after construction), the design factors are the 'drained' strength parameters of the St. Clair Till, together with the steady-state seepage condition.

Undrained Shear Strength

The undrained shear strength of a clay can be established by a number of tests. The most representative tests for the St. Clair Till are considered to be:

- In situ Field Vane Testing
- In situ Piezo-cone Penetration Testing (CPT)

The undrained shear strength design line is shown on Figure 1. The design line is based on test data and theoretical considerations. Selected results of field vane and piezo-cone penetration test data, in the area of the Sarnia approach contract area, are also shown on Figure 1. CPT 92-17 is located in the vicinity of the Vidal St. Bridge.

The design strength line below Elevation 165 m is based on the theoretical relationship:

$$s_u = 0.22 \sigma'_p$$

where s_u = undrained shear strength

σ'_p = vertical effective stress for normally consolidated clay.

Drained Strength

Direct shear test results reported by Klohn Leonoff Ltd. (Volumes 1 and 2A, 1991 and 1992) indicate that the near normally consolidated St. Clair Till has an effective cohesion intercept (c') of zero and effective friction angle (ϕ') of about 26° to 28° , with a residual (large strain) angle (ϕ'_r) of about 22° . The limited test data available in the desiccated crust indicates that the till in this zone has an effective cohesion intercept (c') of about 9.5 kPa and an effective friction angle (ϕ') between about 31° and 33° .

The effective stress parameter ranges are summarized in Table 1. For analysis and design purposes, the upper desiccated clay is characterized as $c' = 9.5$ kPa and $\phi' = 31^\circ$, while the remaining clay below Elevation 176 m is characterized as $c' = 0$ and $\phi' = 26^\circ$.

SLOPE STABILITY ANALYSES

General

Slope stability analyses for three sections located at Chainages 9+697, 9+673 and 9+655 are presented. The sections have been taken perpendicular to the track alignment. Detailed drawings of the proposed cut and retaining wall systems at the Vidal St. Bridge are shown, in particular on Hatch Associates/Letham Ltd. drawing numbers 21635-B1-C-110, 21635-B1-C-113 and 21635-B1-C-156.

The stability analysis has been carried out using the commercially available program SLOPE/W, using a limiting equilibrium method of analysis as described by Morgenstern and Price (1965). The program utilizes numerous trial "failure" circular and non-circular surfaces in order to compute minimum factors of safety for a plane strain condition. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The effect of the interaction between the piled foundations and the slope has not been considered in the analysis.

For the 'undrained' slope stability analyses, a target factor of safety of 1.3 was selected. This is a low factor of safety to use for undrained slope analysis in a situation where the consequence of failure would be severe. However, the calculated factor of safety for the existing slopes is in the range of 1.2 to 1.5. The satisfactory performance of the existing slopes removes some of the uncertainties normally involved in an undrained slope analysis. In this case a target factor of safety of 1.3 is, therefore, considered acceptable.

For the 'drained' slope stability analyses, a target factor of safety of 1.3 was also used for those analyses based on peak shear strengths. Selected sections were also checked using residual shear strength parameters. Where residual shear strengths were used, the target factor of safety was 1.0.

For analysis, the depth of potential tension cracks, z , used in the undrained stability case was calculated from:

$$z = 2s_u/\gamma$$

where: s_u = undrained shear strength (kPa)
 γ = unit weight (kN/m³)

Using this equation, it was calculated that a tension crack could penetrate to the upper desiccated clay (i.e. from ground surface to about Elevation 176 m). For all undrained stability analyses, a water filled tension crack zone was modelled from ground surface to Elevation 176 m.

Analysis Results

Table 2 summarizes the results obtained for undrained analyses (plane strain or two-dimensional case) carried out at Chainages 9+673 and 9+697. Chainage 9+697 which is located on the west side of the bridge was analyzed for the construction condition (at the maximum excavation depth) and for the final condition (after wall completion). Chainage 9+673, located under the north slope of the bridge, was additionally analyzed for the existing slope condition. The analysis results for Chainage 9+673 are shown on Figures 2, 3 and 4 for the existing, construction and final conditions, respectively. For both analyzed sections, the factor of safety for the construction cut condition is about 1.2, which is below the target factor of safety of 1.3. However, it is generally recognized that an increase in stability of at least 10 percent can be realized through three-dimensional effects. Such three-dimensional effects can be obtained by restricting the length of the open construction cut, at any given time, to 20 m. Therefore, allowing for three-dimensional support effects, the temporary construction condition is considered acceptable.

The results of a drained analysis at Chainage 9+673 is shown on Figure 5. This figure indicates a calculated factor of safety of 1.32 for a deep seated failure beneath the retaining wall system. The phreatic surface used in the analysis was estimated for steady-state seepage conditions. An analysis was carried out using residual strength for the same slope configuration; the calculated factor of safety reduced to 1.06. These calculated factors of safety, for the assumptions used, are acceptable for drained analysis.

Undrained analyses were also carried out for a section at Chainage 9+655 which is located on the east side of the bridge. The analyses included a line load at the crest located to simulate the effects of a spread footing abutment load. The bridge foundation locations and type were obtained from as-constructed drawings provided by Hatch Associates Ltd. The footing load and

founding elevation for the spread footing, designated Pier 4 (north abutment), was obtained from a report entitled "Soil Investigation for Vidal Street Bridge, Sarnia, Ontario" by Dominion Soil Investigation Ltd., dated May 2, 1967 to Nisbet Letham Ltd. This report gives the design dead load as about 275 kN/m and the live load as 91 kN/m for the abutment footings at a founding elevation of about 181.7 m.

The undrained analysis results for Chainage 9+655 are given on Figure 6 for the existing, construction and final wall constructed conditions and plotted as calculated factor of safety against the value of the Pier 4 line load. Figure 7 shows, as an example, the calculated factor of safety as 1.24 for the final wall constructed condition, using a line load of 366 kN/m.

The results on Figure 6 indicate that at Chainage 9+655 and for a total pier load of 366 kN/m the calculated factor of safety for the construction cut is about 1.15. It improves to 1.24 for the final condition. These factors of safety compare to 1.37 for the existing cut adjacent to the bridge (i.e. no line load). Given the low level of the calculated factor of safety for the construction cut, we recommend that the length of open construction cut, at any given time, be reduced to 10 m between Chainages 9+640 and 9+700 to further enhance three-dimensional support effects. For the final condition, the overall slope factor of safety, east and west of the Vidal St. Bridge, is above 1.3; it would reduce to about 1.25 only over a length of about 30 m. Therefore, this localized case is considered acceptable.

CONCLUSIONS

The stability of the proposed slopes beneath the Vidal St. Bridge is considered acceptable, based on both drained and undrained analyses and provided appropriate precautions are taken during construction. To enhance stability during construction, we recommend that the length of the open construction cut for retaining wall installation, at any given time, be 10 m between Chainages 9+640 and 9+700.

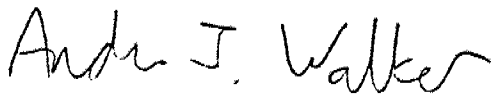
It should be noted that the deformation of slopes is controlled by the overall factor of safety. As the factor of safety decreases, deformation of the ground increases. The factor of safety for the proposed cut is lower than the corresponding existing condition. Therefore, some lateral and vertical deformation of the ground is anticipated as a result of cutting the new slope. This implies that additional loads will be experienced by the piled pier foundations. An estimate of the movement/load has not been made at this stage, since this analysis is sensitive to both the assumed soil parameters and the past history of the slope; neither are sufficiently defined to allow a reasonable estimate to be made. If required, it would be advantageous to carry out this analysis

after data has been obtained from the adjacent slope excavation. In any event, field monitoring of lateral and vertical movement for both the bridge foundations and the slopes should be carried out.

If you have any questions regarding this report or if we can be of further assistance, please do not hesitate to call us.

Yours truly,

GOLDER ASSOCIATES LTD.



A. J. Walker, M.S.



D.E. Becker, P.Eng.

Associate

AJW/DEB/JNS/pds

Att: References
Tables 1 and 2
Figures 1 to 7

REFERENCES

- Golder Associates Ltd. (1992). Geotechnical Aspects of Open Cut Design, Chainage 9+950 to 10+000, St. Clair River Tunnel, Sarnia, Ontario. Report No. 921-1322B submitted to Hatch/Mott MacDonald Joint Venture.
- Golder Associates Ltd. (1992). Site Investigation Report, CN - St. Clair River Tunnel Project, Sarnia Approach Cut, Sarnia, Ontario. Report No. 921-1322B-1 submitted to Hatch/Mott MacDonald Joint Venture.
- Golder Associates Ltd. (1992). Geotechnical Aspects of Open Cut Design, Sarnia Approach to Ch. 9+850, CN-St. Clair River Tunnel Project, Sarnia, Ontario. Report No. 921-1322B-3 submitted to Hatch/Mott MacDonald Joint Venture. -
- Klohn Leonoff Ltd. in association with Giffels Associates Limited (1991). St. Clair River Tunnel, Feasibility Assessment of the Construction of a New Tunnel, Design and Project Implementation, Volume 1. Report to Canadian National Railways.
- Klohn Leonoff Ltd. in association with Giffels Associates Limited (1991). St. Clair River Tunnel, Feasibility Assessment of the Construction of a New Tunnel, Geological and Geotechnical Aspects, Volume 2. Report to Canadian National Railways.
- Klohn Leonoff Ltd. in association with Giffels Associates Limited (1992). St. Clair River Tunnel, Feasibility Assessment of the Construction of a New Tunnel, Addendum to Geological and Geotechnical Aspects, Volume 2A. Report to Canadian National Railways.
- Morgenstern, N.R. and Price, V.E. (1965). The Analysis of the Stability of General Slip Surfaces. Geotechnique, Vol. 15, pp. 7-93.

TABLE 1
GEOTECHNICAL DESIGN PARAMETERS FOR ST. CLAIR TILL

PARAMETER	GREY CLAY	BROWN CLAY (CRUST)
Grading	Silty clay, trace sand and gravel	Silty clay, trace sand and gravel
Water Content (%)	5 - 60	5 - 40
Unit Weight (kN/m ³)	18 - 21	20 - 22
Effective Cohesion, c' (kPa)	0	9.5
Effective Friction angle ϕ' (deg)	26	31
Residual Effective Friction Angle ϕ_r' (deg)	22	22
Undisturbed Shear Strength (S_u)	c.f Figure 1	c.f. Figure 1
Deformation Modulus, E' (MPa)	10 - 20	10 - 20
Poisson's Ratio ν	(0.3 - 0.5)	(0.3 - 0.5)
Coefficient of Earth Pressure at Rest K_0	0.6 - 0.8	> 1
Hydraulic Conductivity, k (m/s)	10^{-11} - 10^{-9}	10^{-9} - 10^{-7}

Notes:

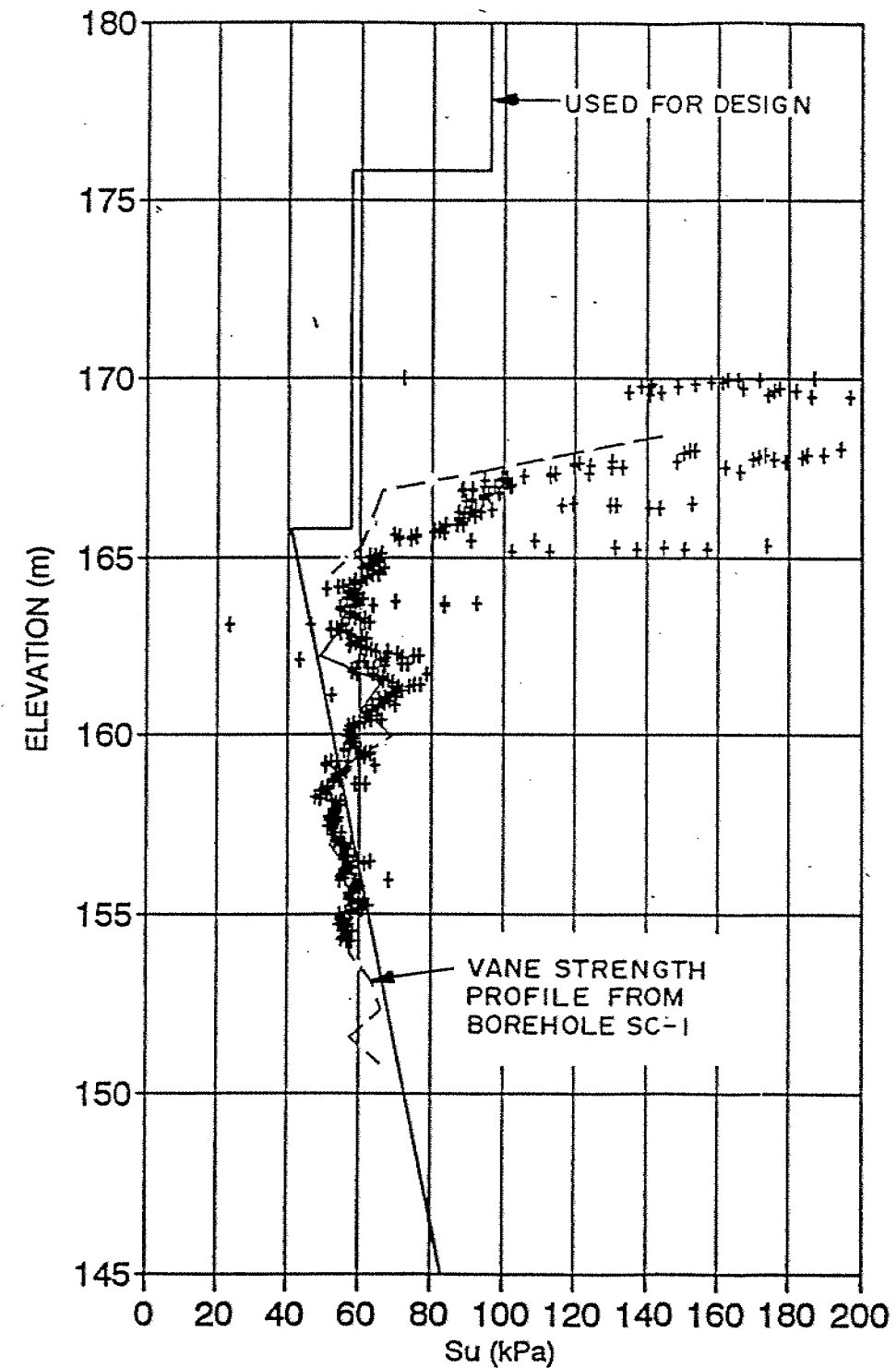
Values in brackets are geotechnical properties estimated by experience.

TABLE 2
SUMMARY OF UNDRAINED STABILITY ANALYSIS RESULTS

SECTION CHAINAGE	ANALYSIS CONDITION	MINIMUM CALCULATED FACTOR OF SAFETY
9 + 697	Construction Cut	1.20
	Final Condition	1.30
9 + 673	Existing	1.48
	Construction Cut	1.23
	Final Condition	1.33

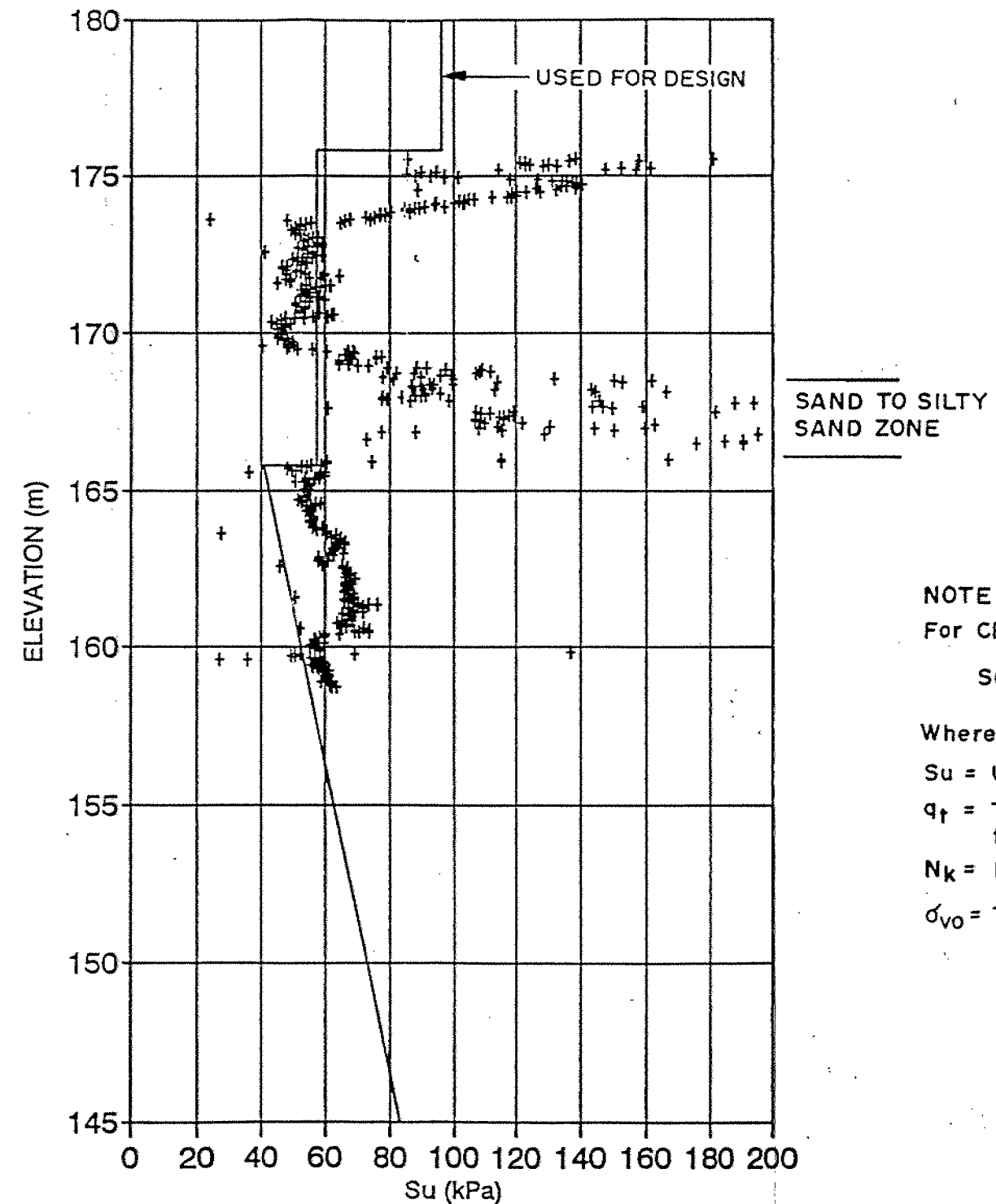
UNDRAINED SHEAR STRENGTH

CPT 92-7 STRENGTH PLOT



UNDRAINED SHEAR STRENGTH

CPT 92-17 STRENGTH PLOT



NOTE:

For CPT Strength interpretation

$$S_u = \frac{q_t - \sigma_{v0}}{N_k}$$

Where:

 S_u = Undrained Shear Strength q_t = Tip resistance adjusted for pore pressure $N_k = 12.5$ σ_{v0} = Total Vertical Soil Pressure

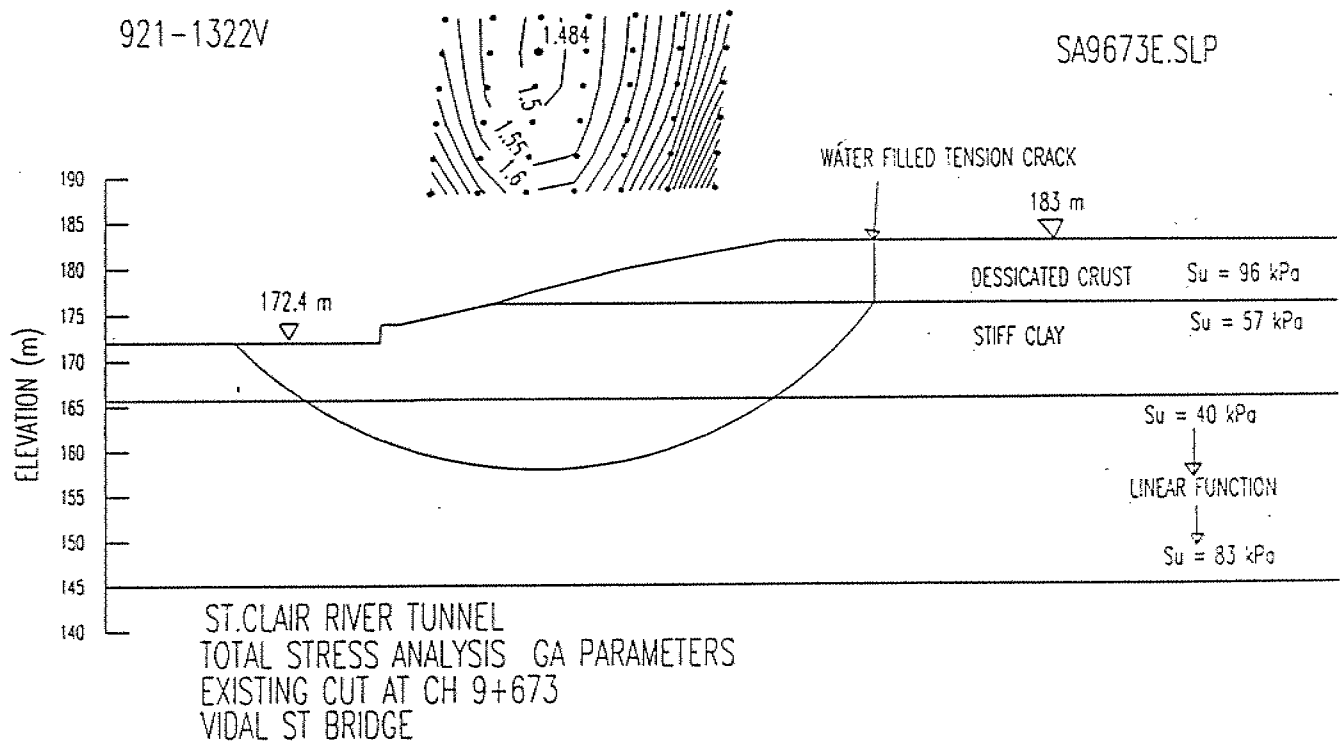
Date: APR. 8, 1993
 Project: 92I-1322V

Golder Associates

Drawn: MHW/SK
 Chkd: AJW

UNDRAINED ANALYSIS
EXISTING SLOPE CONFIGURATION
CH. 9 + 673

FIGURE 2



Date APR. 8, 1993

Project 921-1322V

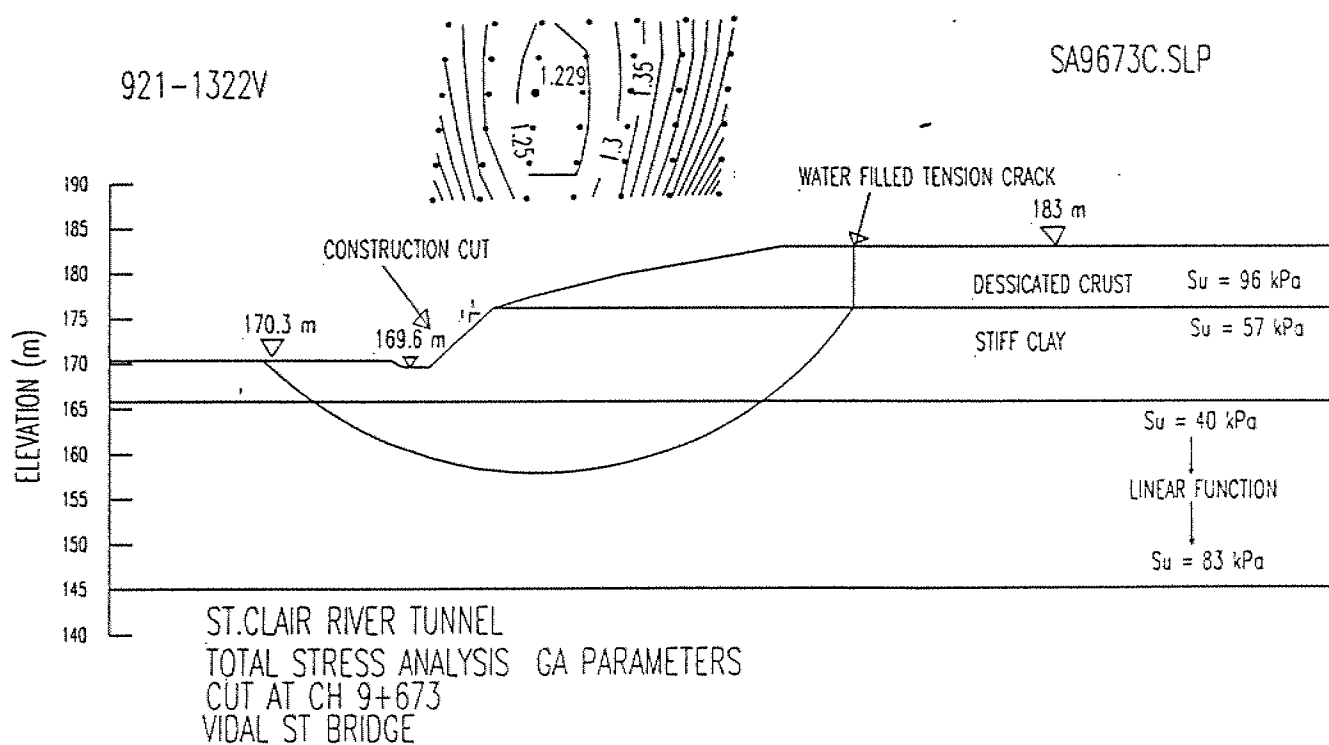
Golder Associates

Drawn DRM

Chkd. AJW

UNDRAINED ANALYSIS
CONSTRUCTION SLOPE CONFIGURATION
CH. 9 + 673

FIGURE 3



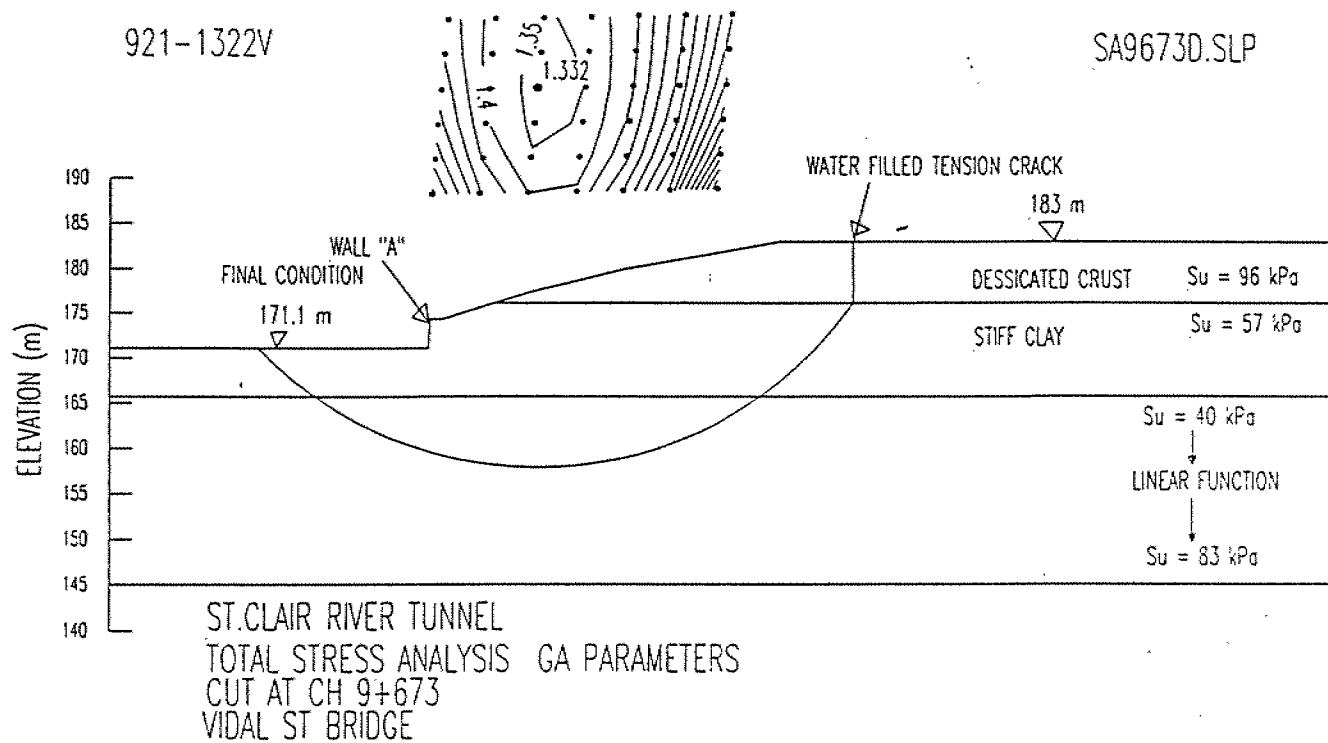
Date APR. 8, 1993

Project 921-1322V

Golder Associates

Drawn DRM

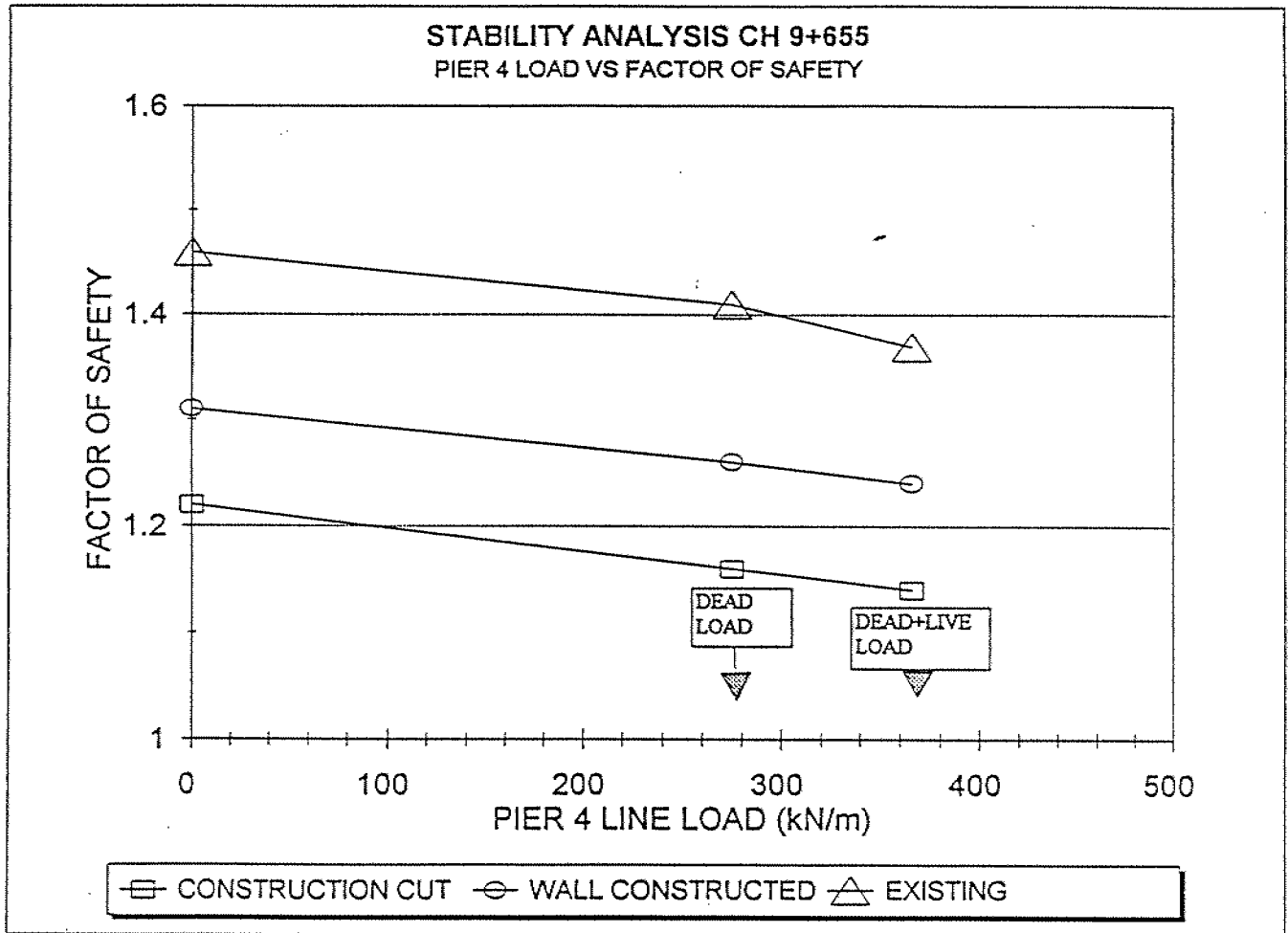
Chkd. AJW



Date APR. 8, 1993
Project 921-1322V

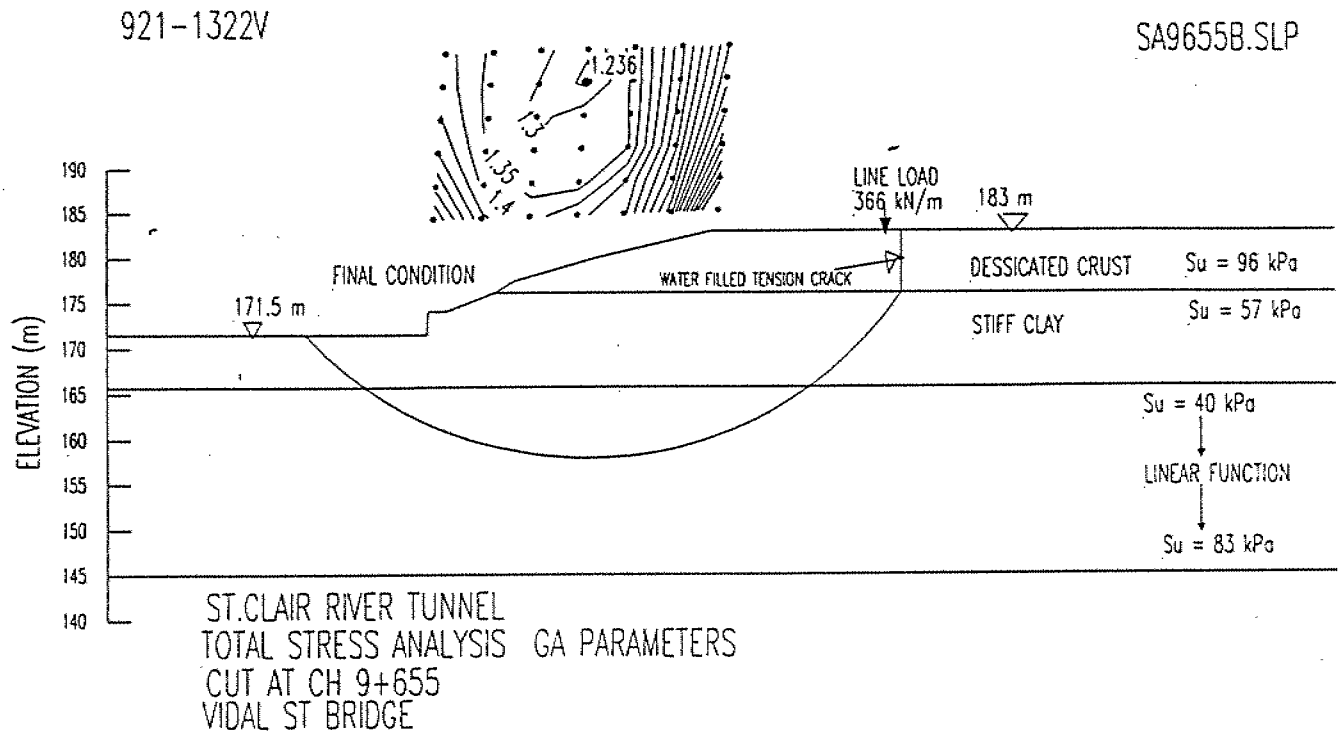
Golder Associates

Drawn ORM
Chkd. AJW



UNDRAINED ANALYSIS
FINAL CONDITION
CH. 9 +655

FIGURE 7



Date APR. 8, 1993
Project 921-1322V

Golder Associates

Drawn DRM
Chkd. AJW



Structural Office
7th Floor, Atrium Tower
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8
Telephone: (416)235-4960
Fax: (416)235-4068

March 29, 1993

Mr. M.J. Bosher, P. Eng.
Letham Ltd.
Consulting Engineers
133 Kendall St.
Point Edward, Sarnia, Ontario
N7V 4G6

**Re: City of Sarnia
St. Clair River Tunnel Project
Vidal Street Bridge Pier Footing Modification
M.T.O. Site No. 14-76**

Dear Sir:

We have reviewed the revised final plans for the above project which you submitted on March 10th, 1993. The plans are acceptable to this Ministry, provided that:

- The temporary timber lagging support system should be checked and approved by your engineer on the site prior to construction.
- A copy of a confirmation letter from the project geotechnical consultant for the overall stability of your proposal should be sent to this Section for our records.

Yours truly,

A.A. Witecki, P. Eng.
Municipal Engineer,
Approvals Section

AAW:at

cc: T. Swan
A. Ho
T.C. Kim (Foundation Design) ✓



memorandum



To: A.A. Witecki, Municipal Engineer
Approvals Section
Structural Office
7th Floor, Atrium Tower

Date: 1993 02 23

Attn: A.J. Lee, Project Engineer

From: Foundation Design Section
Room 315, Central Building

Re: Review of Retaining Wall
Vidal Street Bridge, Sarnia
W.O. 93-11003, Site 14-76
District 1, Chatham

Further to your memo dated February 19, 1993, the construction drawings of the above retaining walls provided by Letham Ltd. and the Foundation Investigation Report prepared by Dominion Soil Investigation Inc., have been reviewed by this office.

Based on our review, the following comments should be made on the above mentioned drawings and report.

1. The base of all footing excavations should be proof-rolled and covered immediately upon exposure with a 150 mm thick working slab of lean concrete to protect the exposed silty clay from disturbing and softening.
2. Since the existing slope will be cut to add one more line of railway, the overall slope stability during and after construction should be reviewed and assured by a geotechnical consultant.
3. The existing pile cap for the pier should be placed so as to have a minimum earth cover of 1.2 m or equivalent thickness of styrofoam (4 inches) to allow for frost protection.

We have no further comments. If you have any questions, please contact this office.

Tae C. Kim
Tae C. Kim, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

MD/TCK/jb

c.c. A. Ho

24-2732
ALSO B7-1353 RACEY, HARGREAVES
Report

NISBET LETHAM LIMITED
206 WATER STREET
SARNIA ONTARIO

STRUCTURE SITE No. 14-76

Report on
SOIL INVESTIGATION
for
VIDAL STREET BRIDGE
SARNIA, ONTARIO.

by
DOMINION SOIL INVESTIGATION LIMITED
369 Queens Avenue
LONDON ONTARIO

Reference No 7-3-L11
May 2nd, 1967.

INTRODUCTION

In accordance with authorization from Nisbet Letham Limited, Consulting Engineers, a soil investigation has been carried out in the City of Sarnia where it is proposed to reconstruct a section of Vidal Street. The proposed structure is an overhead road crossing the approach cutting to the St. Clair railway tunnel and a spur line about 300 feet to the north. Vidal Street at this point connects the main city area to the industrial section to the south.

The objects of this investigation as defined by discussions with the client have been:-

-) to reveal the subsurface conditions and to determine the relevant soil properties for the design and construction of the piers and abutments
-) to investigate the stability of the existing railway cutting
-) to recommend the thickness of subbase and base course for the approach road construction

THE GEOLOGY OF THE SITE

The site is situated in the southern part of the City of Sarnia, in an area of little surface relief. The general ground surface elevation is about El. 602 feet Geodetic datum, and the railway cutting extends down to El. 563 where it crosses the centre line of the road.

The physiographic region, known as the St. Clair Clay Plains, consists of deep clay deposits with a thickness of about 120 feet, overlying black shale bedrock. The surface of the bedrock is usually very flat.

FIELD WORK

The field work, consisting of 22 boreholes, was carried out during the period April 5 to 27, 1967, at the locations indicated on the site plan, Enclosure 2. The holes were advanced by a continuous flight power auger, except for those put down in the railway cutting which were advanced by a diamond drill machine.

Standard penetration tests were carried out at frequent intervals of depth, as detailed on Appendix 'A', and the results are recorded on the Geotechnical Data Sheets as 'N' values.

Undisturbed samples were obtained in 2-inch diameter thin-walled Shelby tubes which were sent to the laboratory for testing. The ends of the samples were sealed in the field to prevent loss of moisture.

The undrained shear strength of the cohesive soil was measured insitu by means of a 4-inch long by 2-inch diameter four bladed rotating vane. The vane was pushed 18-inches in the undisturbed soil below the bottom of the borehole and a torque was applied until the soil failed. The vane was then rotated 10 times to remould the soil and after one minute the torque test was repeated. The shear strength of the soil was calculated from the

torque and the dimensions of the vane, and the sensitivity of the material estimated from the ratio of the original torque to the final torque after remoulding.

Elevations were referred to Geodetic benchmarks which were obtained from the City of Sarnia Engineering Department.

SUBSURFACE CONDITIONS

Detailed descriptions of the strata encountered in each borehole are given on the Geotechnical Data Sheets, comprising Enclosures 3 to 19. The following notes are intended only to amplify this data:-

Underlying thin surface layers of topsoil, sand and gravel road ballast, and clay fill, the boreholes penetrated a deep clay stratum which extends down to El. 483. Black shale bedrock was encountered at this elevation and was penetrated 5 feet in borehole 4. A core recovery of 100% was obtained which indicates that the bedrock is in a sound condition.

The clay stratum has an upper weathered zone extending down to El. 597. Below the weathered zone, the clay stratum has a very stiff crust which extends down to El. 589.

Unconfined compression tests carried out on samples taken from the crust zone gave values of undrained shear strength ranging from 2750 to 7400 p.s.f.

The colour of the crust is generally brown, however with increasing depth the colour changes to grey and is characterized by an increase in the moisture content and a decrease in the shear strength. The shear strength reaches a minimum value of 1400 p.s.f. at El. 580. Below El. 580 the undrained shear strength increases gradually, as would be expected due to the overburden pressure.

A shear strength against depth relationship is plotted on Enclosure 20, together with the shear strength profile used for design purposes. The high values of undrained shear strength as measured by insitu vane shear tests could be a result of the vane encountering gravel particles thus increasing the normal resistance which would be attributed to the clay.

Classification tests carried out on samples of the brown and grey clay types are listed as follows:-

	<u>Brown Silty Clay</u>	<u>Grey Silty Clay</u>
Bulk density (p.s.f.)	132 to 137	123 to 124
Dry Density "	114 to 124	98 to 101
Moisture Content %	12 to 16	22 to 27
Plastic Limit %	13 to 15	15 to 18
Liquid Limit %	29 to 33	31 to 38
Plasticity Index	13 to 15	22 to 27
Liquidity Index	0	0.1 to 0.5
Sensitivity	-	1.1 to 2.7

GROUNDWATER CONDITIONS

Over the level part of the site, the groundwater reached equilibrium at an average El. 598. However due to the impermeable nature of the silty clay subsoil a seasonal fluctuation in the water table may be anticipated.

From a visual and tactile examination of the soil samples the groundwater table may be assumed to fluctuate between El. 592 and El. 598.

The groundwater table in the lower part of the railway cutting may be assumed to coincide with the invert elevation of the drainage system.

DISCUSSION AND RECOMMENDATIONS

It is understood that the piers and abutments will be about 100 feet in length, and that where the soil conditions permit, they will be supported on strip footing foundations.

The total of the live and dead loads on the pier footings for spans of 100 feet will be about 38 kips per linear foot, and of this total the live loading will constitute 7 kips per linear foot.

The total of the live and dead loads on the abutment footings will be about 25 kips per linear foot and of this total the live loading will constitute 6 kips per linear foot.

PIERS & ABUTMENTS LOCATED NORTH OF THE RAILWAY CUTTING

The critical factor in the prevailing soil conditions is the magnitude of the total loads, and whether the very stiff brown silty clay crust is strong enough to withstand them without imposing too high a load on the relatively compressible grey silty clay layer below. The footings should also be located below the upper layer of weathered silty clay to minimise settlement.

On the basis of the borehole results the optimum elevation which may be considered applicable to the above comments is at El. 596. However due to other circumstances a deeper footing grade may be necessary, and the following table may be used in estimating allowable soil pressures for various footing dimensions and grades.

<u>Footing Elevation</u> <u>feet</u>	<u>Maximum Allowable Soil Pressure</u> <u>tons per square foot</u>						
	<u>Footing Widths Feet</u>						
	Up to:						
	6	7	8	9	10	11	12
596	3.0	2.9	2.8	2.7	2.6	2.5	2.4
595	2.8	2.8	2.7	2.6	2.5	2.4	2.3
594	2.7	2.6	2.5	2.4	2.3	2.2	2.2
593	2.5	2.4	2.3	2.2	2.2	2.1	2.1
592	2.4	2.3	2.2	2.1	2.1	2.0	2.0
591	2.2	2.2	2.1	2.0	2.0	1.9	1.9
590	2.1	2.0	2.0	1.9	1.9	1.8	1.8

589	1.9	1.9	1.9	1.8	1.8	1.7	1.7
588	1.8	1.8	1.8	1.7	1.7	1.6	1.6
587	1.6	1.6	1.6	1.6	1.5	1.5	1.5
586	1.5	1.5	1.5	1.5	1.4	1.4	1.4

The above soil pressures incorporate a factor of safety of 3 against shear failure of the underlying soil and are calculated on the basis of the undrained shear strength profile shown on Enclosure 20.

Construction

The footing grade should be inspected by a competent soils engineer prior to placing of the concrete and any 'soft' areas removed and replaced by lean concrete.

It is anticipated that seepage will be restricted to any rainfall which occurs during the construction period and in this respect particular care should be taken to protect the footing grade and prevent softening when it is exposed.

Settlement

On the basis of previous laboratory testing carried out on this site and in the same area, the following values of the modulus of compressibility 'K' have been assumed:-

	<u>Modulus of Compressibility tons/sq.foot</u>
Very stiff brown silty clay	140
Stiff grey silty clay	60

Using these values the computed theoretical settlement is about 1.5 inches. However this oedometer value of settlement must be corrected for the effect of rigidity of the foundation and lateral strain taking place during consolidation. Applying these correction factors the most probable value of settlement is estimated to be 0.8 inch. Since the settlement will be due to the long term consolidation of the subsoil, in the settlement analysis only the dead weight of the structure was considered.

A deformation of the clay stratum will take place immediately as the load is applied due to the elastic deformation of the clay, which takes place without dissipation of pore pressure. Based on an assumed modulus of elasticity of 200 tons per square foot it is estimated that the maximum elastic deformation will be about 0.5 inch.

SOUTH ABUTMENT

The layer of weathered silty clay extends down to El. 595.8 at borehole 3 location therefore it is recommended that the footing grade be established at El. 595. For design purposes the allowable soil pressures can be estimated from the table shown in the preceding discussion.

The stability of the railway cutting embankment will depend on the location of the south abutment and approach fill and in this respect calculations indicate that the forward toe of the abutment should be a minimum distance of 10 feet from the edge of the cutting to maintain adequate slope stability.

PIERS LOCATED ON THE SLOPE OF THE RAILROAD CUTTING

The ultimate bearing capacity of foundations on slopes depends on the geometry of the slope and also on the properties of the soil. A method of calculating the ultimate bearing capacity was developed by G. G. Meyerhof which applies a correction to the bearing capacity factor N_{cq} in the standard formula for bearing capacity.

$$q = c + N_{cq} \gamma D$$

where q is the ultimate bearing capacity

c is the undrained shear strength

γ is the unit weight

D is the depth of the footing below
the ground surface

According to Meyerhofs graphs, using values of slope angle of 20 degrees and a height of slope of 40 feet, the value of the bearing capacity factor is 2.0. The ultimate bearing capacity is therefore 2800 pounds per square foot at the surface of the slope.

It is usual to apply a factor of safety of 3 in bearing capacity design, therefore the allowable bearing capacity of footings on the slope will be 930 p.s.f. plus an additional 40 p.s.f. for each foot of cover above the footing grade.

The above design necessitates a footing width of about 40 feet which is impractical, therefore it is recommended that the piers be supported on friction piles which will transfer the load to the stiffer clays at a much lower depth.

The following estimates of working loads and pile penetrations are based on the shear strength profile shown on Enclosure 20. It is also assumed that the minimum spacing of the piles in each pile cap will be 3 times the pile diameter.

Timber Piles

An adhesion corresponding to 75% of the undrained shear strength of the soil has been assumed for the design of a timber pile foundation.

It is estimated that nominal 12-inch diameter timber piles will achieve a suitable set corresponding to a working load of 30 tons with a penetration of 35 feet. Additional penetration will increase the working load by 1 ton per foot of penetration, however if a working load in excess of 30 tons is proposed, pile loading tests should be performed.

Steel Tube Piles

An adhesion corresponding to 60% of the undrained shear strength of the soil has been assumed for the design of a steel tube pile foundation.

It is estimated that 12-inch diameter tube piles will develop working loads of 40 and 50 tons capacity with penetrations of 50 and 60 feet respectively.

Steel H-piles

The friction load which may be applied to a steel H-pile will depend on the dimensions of the pile rather than the weight of section used.

The following estimates of working load have been made for different pile sizes and penetrations:-

<u>Type of Pile</u>	<u>Penetration (feet)</u>	<u>Working Load (tons)</u>
BP 8	50	40
BP 8	60	50
BP 10	50	50
BP 10	60	62

Settlement of Piled Foundations

In estimating the consolidation settlement of friction pile foundations, the load from the structure may be considered to act as a spread footing at a depth equal to two thirds of the penetration distance of the pile. Therefore in the estimation of settlement it has been assumed that the load will act at about El. 540.

Based on the assumed modulus of compressibility 'K' of 60 tons per square foot, the estimated consolidation settlement of a typical pile group is 2.0 inches. However this oedometer value of settlement must be corrected for the effect of lateral strain taking place during consolidation, which results in a most probable value of settlement of 1.5 inches.

STABILITY OF THE RAILWAY CUTTING

The stability of the slope of the existing railway cutting was calculated using Taylors stability chart and using the following data:-

Slope angle 20 degrees
 Height of Slope (H) 40 feet
 Unit weight 124 lb. per cu. ft.

2.7-1

The average undrained shear strength for a factor of safety of 1.0 is 750 p.s.f. therefore based on the minimum shear strength value of 1400 p.s.f. taken from the shear strength profile, it can be assumed that the actual factor or safety is 1.9. This does not take into account the higher shear strength which may be attributed to the very stiff crust and can therefore be considered a conservative value.

The stability of three slip circles were calculated taking into account the load from the south abutment footing and the weight of fill above it. The geometrical layouts of the slip circles are shown on the cross-section of the railway cutting, Enclosure 21.

The results of the slip circle analyses are as follows:-

<u>Slip Circle No.</u>	<u>Factor of Safety</u>
1	2.2
2	2.0
3	1.7

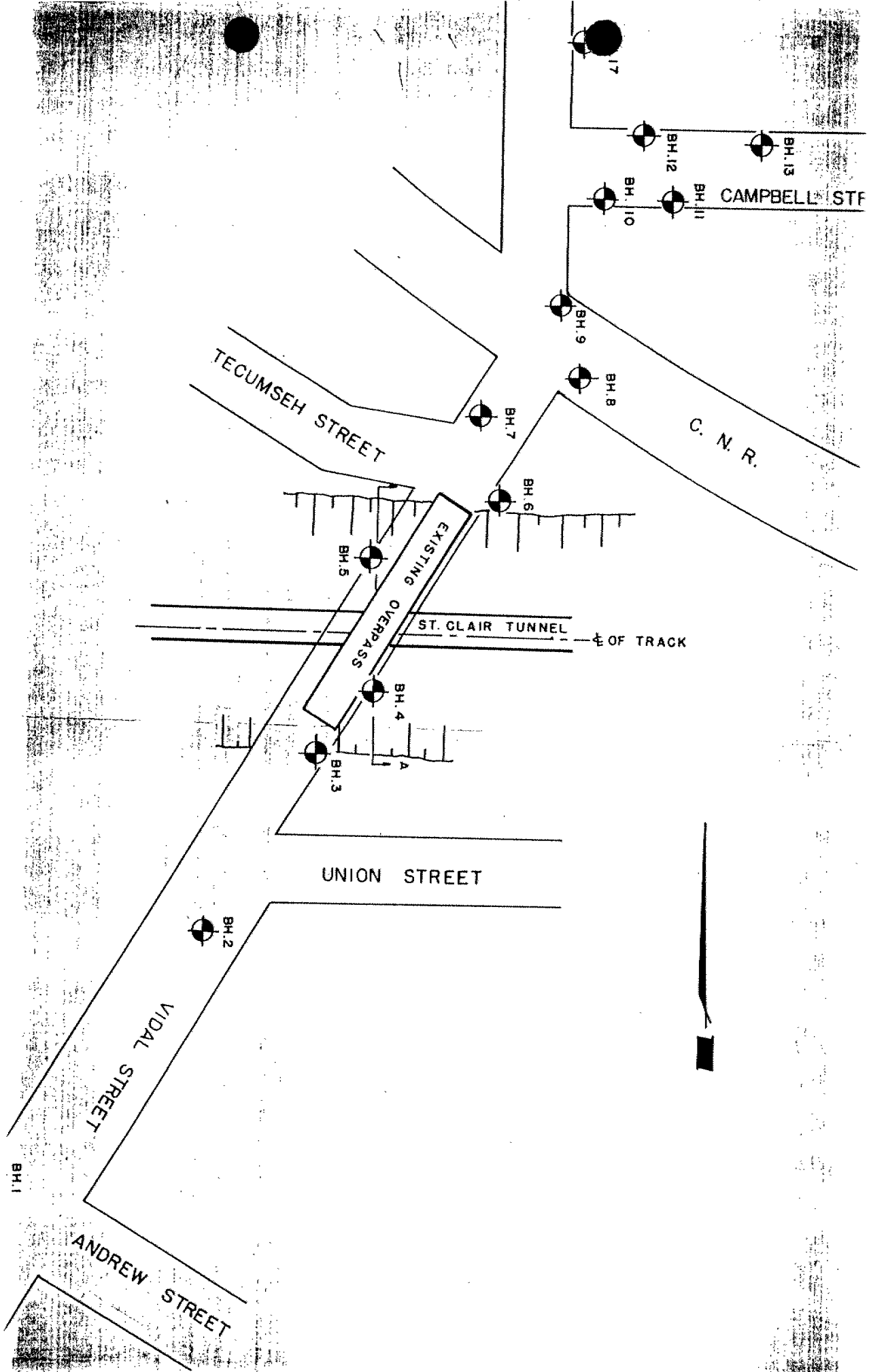
From these results it can be assumed that the critical surface is a deep-seated slip with the arc passing below the toe of the slope and intersecting the ground surface in the level part of the cutting.

The overturning moment due to the load from the abutment footing and approach fill, as calculated in Slip Circle No. 3, amounted to 11.5% of the total overturning moment, therefore the reduction in factor of safety from 1.9 (Taylor's Chrt) to 1.7 may be attributed to the additional loading from the abutment and approach fill.

CONSTRUCTION OF APPROACH FILLS

The main body of the approach fills may be constructed with granular (sand and gravel) or cohesive silty clay material possessing suitable compaction characteristics. The fill should be placed in layers of six to eight inches in thickness and compacted to at least 95% of the maximum Standard Proctor dry density of the particular material used.

It is usual for granular material to be compacted with a vibratory roller and cohesive material with a sheeps-foot roller to achieve the required compaction. Insitu density tests should be made on the compacted fill to determine whether the required density is being achieved, and to ensure that the moisture content compares favourably with the optimum moisture content for the particular material.



OUR REFERENCE NO. 7-3-1

GEOTECHNICAL DATA SHEET FOR BOREHOLE

CLIENT: Robert Letham Limited
 PROJECT: Vidal Street Bridge,
 LOCATION: Sarnia, Ontario,
 DATUM ELEVATION:

METHOD OF BORING: Auger
 DIAMETER OF BOREHOLE: 4-inch
 DATE: April 6, 1967

BH3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %		
				NUMBER	TYPE	Advance- ment of Sampler	20	40	60	80	100	PL	W	
603.8	0.0	Ground Surface					10	20	30	40	50	10	20	30
		4" Asphalt												
	2.5	Sand & Gravel												
600		Stiff grey/brown weathered silty clay trace of organics		1	SS	8								
				2	SS	8								
595	8.0	Very stiff silty clay, Brown traces Grey of sand		3	SS	29								
				4	SS	31								
590				5	SS	17								
				6	SS	7								
585	20.0	and gravel												
		Compact to dense grey silt		7	SS	26								
580														
575	27.0	Stiff to very stiff grey silty clay, traces of gravel.		8	SS	31								
				9	SS	9								
570														
				10	TW									
565														
				11	SS	5								
560														
				12	SS	5								
555														
				13	SS	12								
550		End of Borehole												

VERTICAL SCALE: 1 IN TO

FT.

DORNINGTON SCALE IS CONSIDERED CORRECT FOR THIS PROJECT

MAL

OUR REFERENCE NO. 7-3-L11

GEOTECHNICAL DATA SHEET FOR BOREHOLE 4.1.1

CITY: Nisbet Latham Limited
 PROJECT: Vidal Street Bridge
 LOCATION: Sarnia, Ontario.
 DATUM ELEVATION:

DATE: April 26 & 27, 1967
 BORING: Washboring
 DIAMETER OF BOREHOLE: 3/4 (3-inch)

BH4

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %	
				NUMBER	TYPE	Advance- ment of Sampler	20	40	60	80	100	PL	W
							SHEAR STRENGTH 100 lbs. sq. ft.						
							10	20	30	40	50		
582.4	0.0	Ground Surface											
		4" Topsoil											
580		Stiff	T	1	SS	11							
		to		2	TW								
575		very		3	SS	9							
		stiff	T	4	TW								
570		gray		5	SS	8							
		silty	T	6	SS	10							
565		clay,		7	TW								
		containing	T	8	SS	10							
555		traces		9	SS	26							
550		of	T	10	SS	21							
545		sand		11	SS	20							
		and	T										
540	42.5	gravel	T										
		End of Borehole											

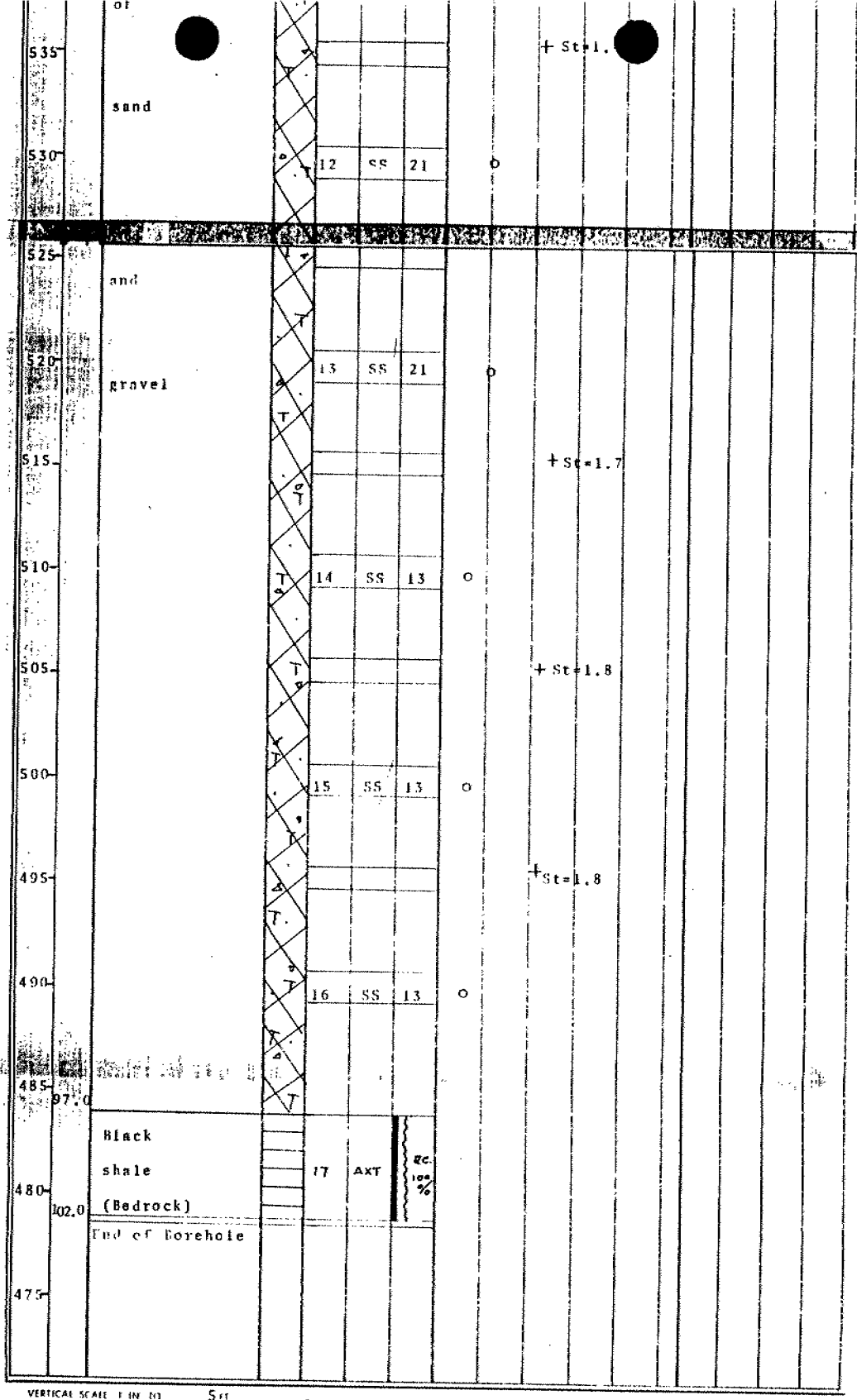
VERTICAL SCALE: 1 IN. TO

5 ft.

DOMINION STATE ENGINEERING CONSULTANTS LTD.

MAD

BH 4
CONT'D



VERTICAL SCALE 1 IN. = 10

5 ft

MAD

REFERENCE NO. 7-3-111
 CLIENT: Met Letham Limited
 PROJECT: Canal Street Bridge
 LOCATION: Sarnia, Ontario.
 DATUM ELEVATION.

METHOD: Washboring
 DIAMETER OF HOLE: 8x (3-inch)
 DATE: April 25 & 26, 1967

BH5

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE					CONSISTENCY	
				NUMBER	TYPE	1/2 in. Adv. of Sampler	30	40	60	EP	100	PL	water content %
							SHEAR STRENGTH 100 lbs. sq. ft.					10 20 30 40 50	
580.3	0.0	Ground Surface											
		1" Topsoil											
		Stiff		1	SS	24							
575		brown		2	SS	13							
		to		3	SS	6							
		grey		4	TW								
570				5	SS	4							
		very		6	TW								
565		stiff		7	SS	10							
560		silty		8	SS	12							
555		clay,		9	SS	11							
550		containing		10	TW								
545		traces		11	SS	21							
540		of											
535		sand		12	SS	21							
530													
525													

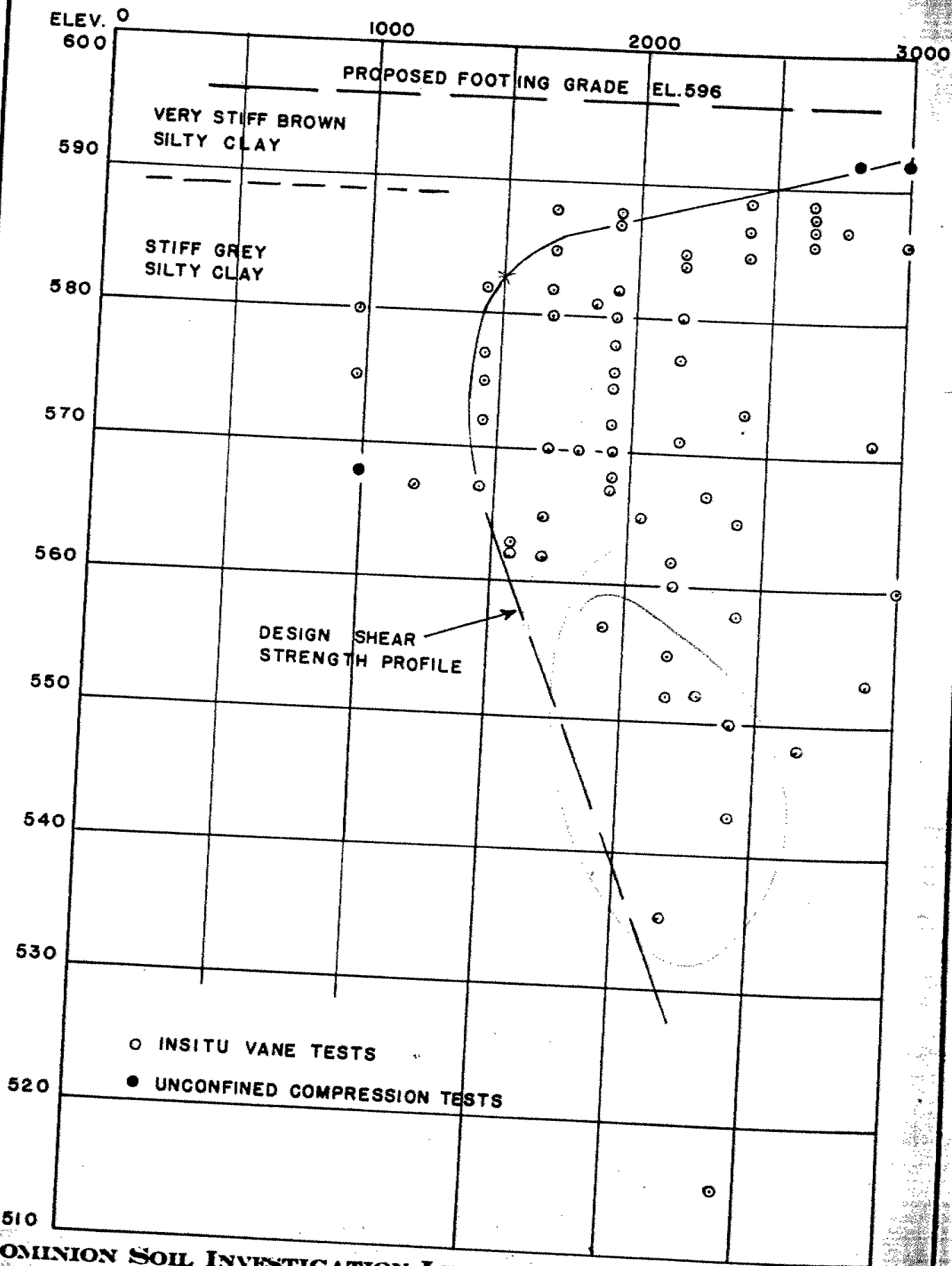
Calculated by 524 to 535 ft. sand

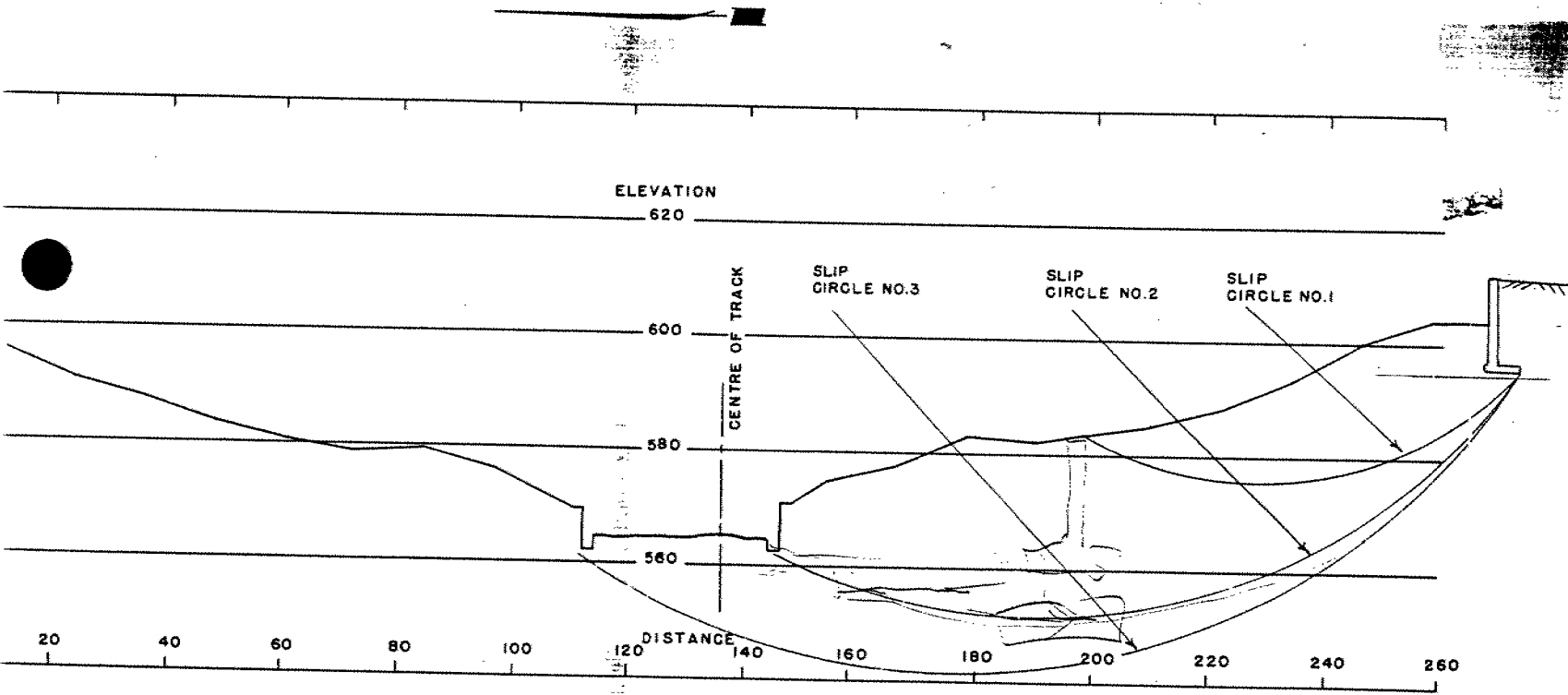
CURVE: Nisbet Letham Limited
PROJECT: Vidal Street Bridge,
LOCATION: Sarnia, Ontario.
DATUM ELEVATION:

METHOD OF BORING Auger
DIAMETER OF BOREHOLE 4-inch
DATE April 5, 1967

BH 6

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE <small>blows per foot</small>					CONSISTENCY <small>water content %</small>	
				NUMBER	TYPE	$\frac{N-1}{2}$ Average of Sample	20	40	60	80	100	PL	W
604.1	0.0	Ground Surface											
		8" Topsoil											
600	4.0	Loose black cinder Fill		1	SS	9	O						
	7.5	Stiff brown weathered silty clay	T	2	SS	9	O						
595		Silty clay containing traces of sand and gravel	T	3	SS	16	O						
			T	4	SS	24	O						
590			T	5	SS	12	O						
			T	6	SS	12	O				+ St=1.3		
585			T								+ St=1.4		
			T	7	SS	8	O				+ St=2.0		
580			T	8	SS	11	O				+ St=2.7		
575	29.0	Very dense grey silt	x x x	9	SS	77	O						
570	33.5	Stiff grey silty clay trace of fine gravel	T	10	SS	8	O				+ St=1.8		
565			T										
560	42.5	End of Borehole	T	11	SS	4	O				+ St=2.0		





SECTION A-A ST. CLAIR TUNNEL
HOR. SCALE 1"=20'
VERT. SCALE 1"=20'

NG TRACK

1.5
1

T/R VARIES

FOR ROAD BED CONSTRUCTION DETAILS, SEE DRG. No. 81-C-113

SECTION A-A

NEW TRACK

OUTLINE OF PORTION OF
PILE CAP TO BE REMOVED

U/S EXISTING PILE CAP
EL. 172.87

CUT AND REMOVE THIS PORTION
OF EXISTING PILE # 18

PILE #18 CUT OFF EL. 169.76

BUILD PILE # 18 INTO WALL FOOTING

150 mm LEAN CONCRETE
MASS CONCRETE

UNDISTURBED
SUBGRADE

THIS PORTION OF EXISTING PILE # 18
TO BE RETAINED

do not need
prob rolling

WALL VERT P.O.C. = 174.38

EXISTING PILE CAP

32
(MIN)

ELEVATIONS AT JUNCTION

SEE DETAIL 'A'

MTC DOWNSVIEW
RECEIVED
FEB 12 1993
STRUCTURAL OFFICE
M. T. O.

HILTI HVA M20 X 240 ADHESIVE ANCHORS
C/W OVERSIZE WASHERS, ALTERNATE FLUTES
WALL 'B' ONLY

CMR L6S PERMANENT SHEET PILING
OR EQUAL APPROVED.

REINFORCED CONCRETE WALL 'B'

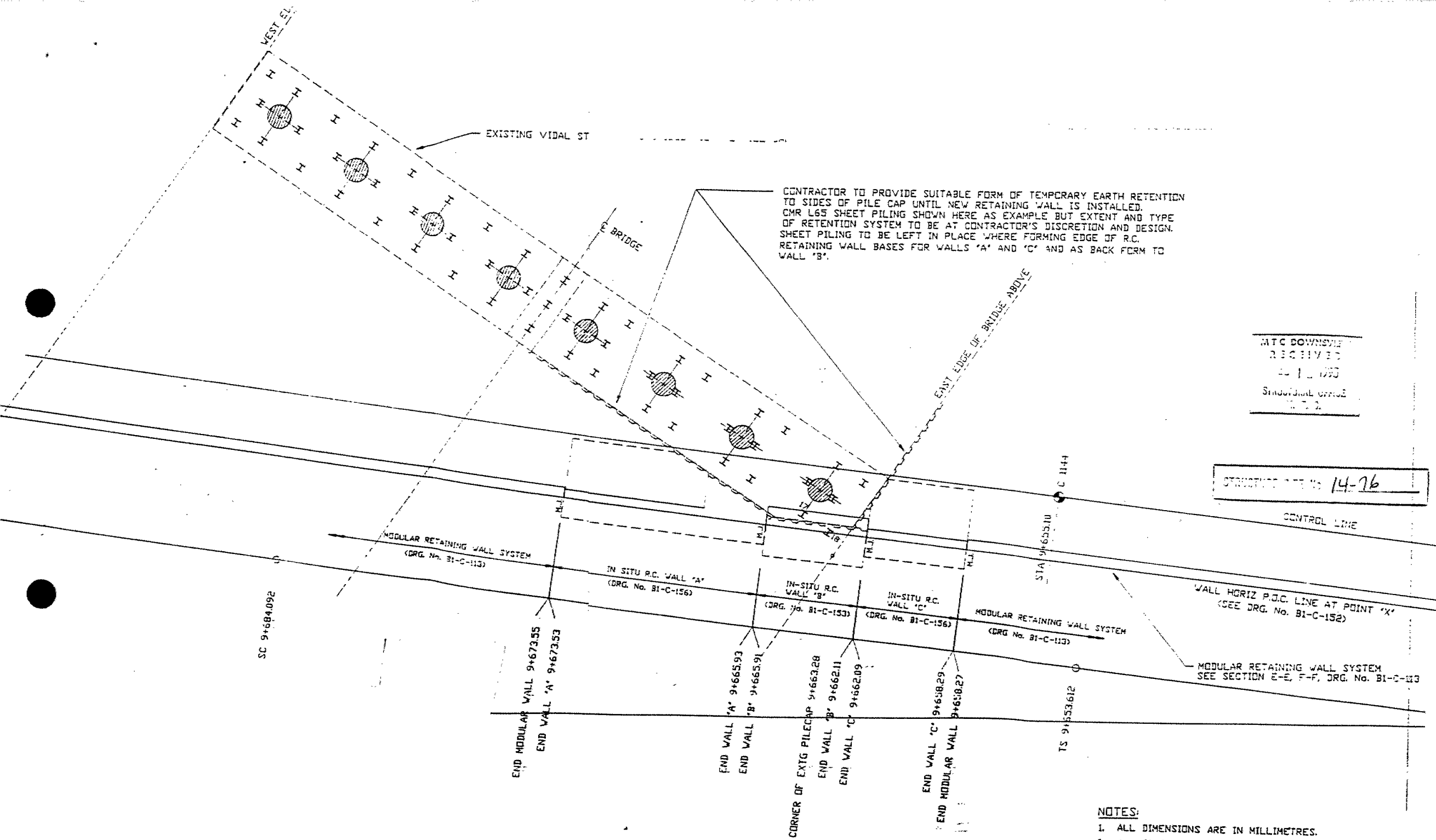
POINT 'X' = WALL HORIZ P.O.C.
4000 FROM NEW ALIGNMENT 'E'

100 MM THICK POLYSTYRENE INSULATION SHAPED
AND FIXED TO PILE FLUTES, WALL 'B' ONLY

STRUCTURAL CIVIL NO. 14-76

U/S CONCRETE RETAINING WALL EL. 169.39

BOTTOM OF SHEET PILING ALONG CUT EDGE
OF PILE CAP = 165.89



MTC DOWNSIDE
RECEIVED
12 1 1993
STRUCTURAL OFFICE
M.T.C.

STRUCTURE NO. 14-76

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES.

PLAN ON RETAINING WALL

CORNER OF EXTG
PILE CAP

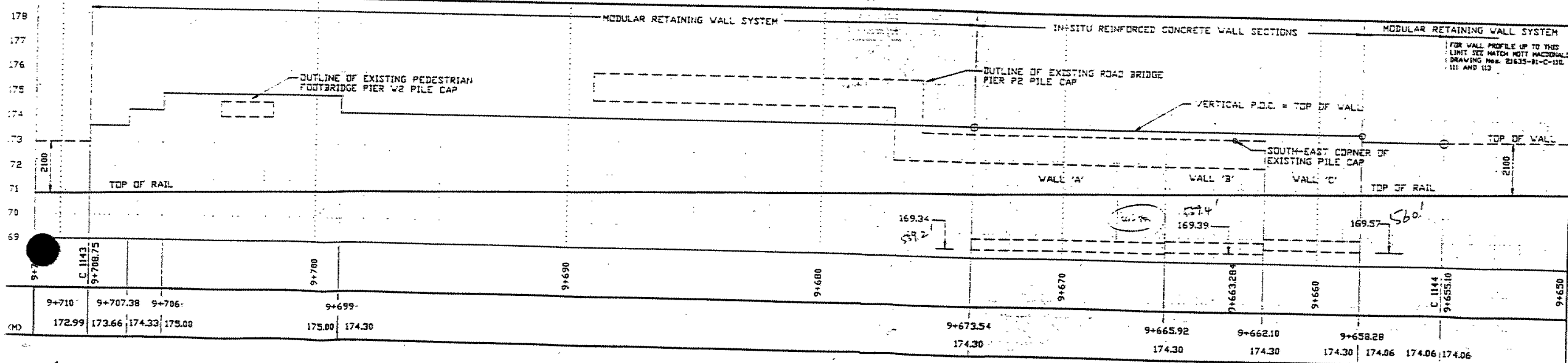
TS 9+53.612

9+63.284

SC 9+681.092

9+708.75

NO DETAILS BEYOND
MOTT MACDONALD
-106



WALL PROFILE

NOTES:

1. ALL DIMS
2. ALL ELE