



FINAL REPORT

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

*Highway 401/Flook and Hinton Drain Bridges (Site Nos. 13X-0229/B1 & B2)
County of Kent, Ontario MTO GWP 3024-18-00, Assignment No. 3018-E-0011*

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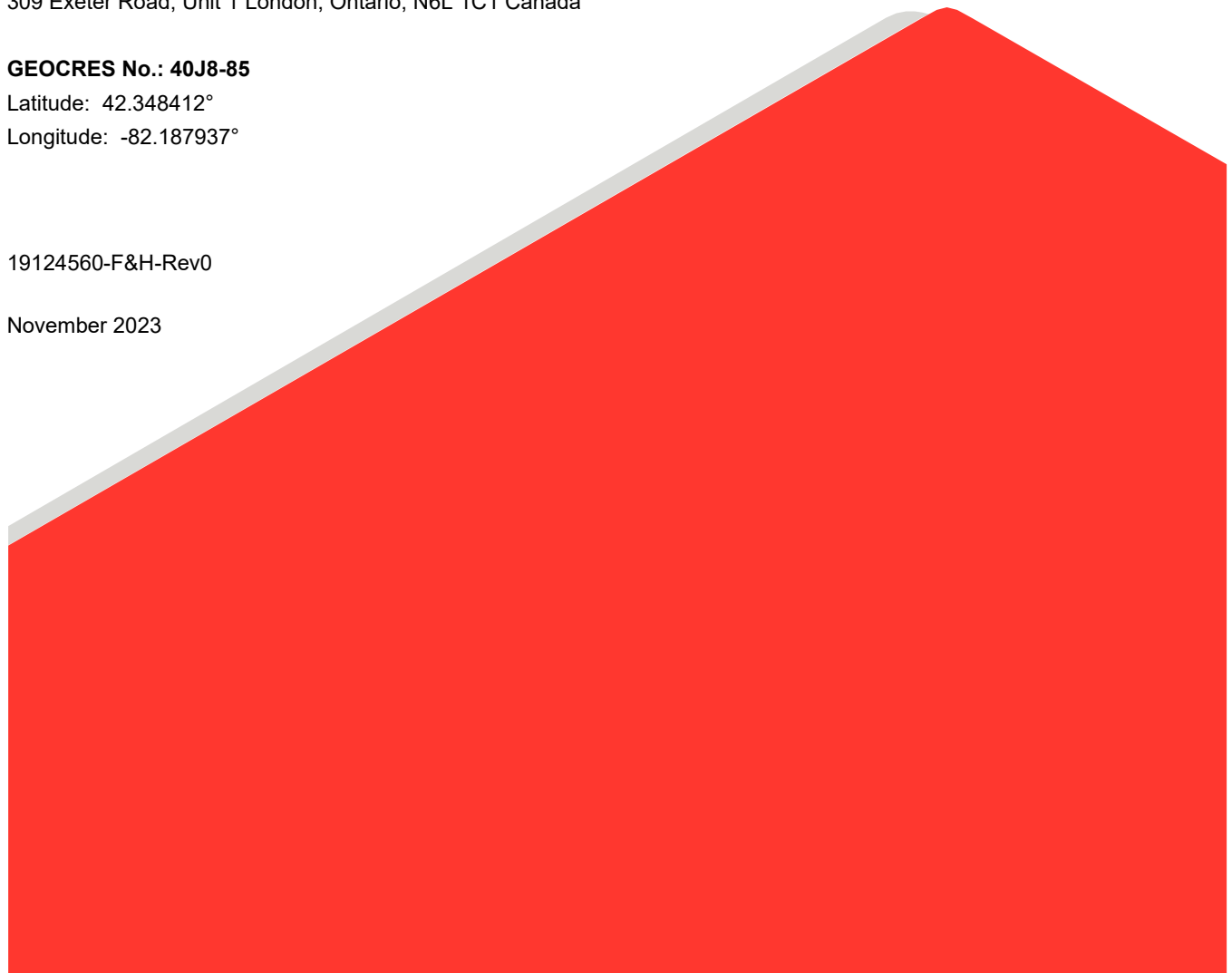
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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/FLOOK AND HINTON DRAIN BRIDGES
(SITE Nos. 13X-0229/B1 & B2)
COUNTY OF KENT, ONTARIO
MTO GWP 3024-18-00, ASSIGNMENT No. 3018-E-0011**

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., amalgamated with WSP Canada Inc. as of January 2023) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the expansion of Highway 401 from Tilbury to London, Ontario as part of MTO Assignment No. 3018-E-0011.

The overall project is divided into six GWPs. The Flook and Hinton Drain bridges, the subject of this report, are part of GWP 3024-18-00, which consists of widening Highway 401 towards the median to create a six-lane cross section in conjunction with the installation of new median tall wall barrier. As such, both the eastbound and westbound Flook and Hinton Drain bridges will be widened.

This report presents the results of a foundation investigation conducted in support of preliminary design for the widening of the Flook and Hinton Drain bridge structures (MTO Structure Site Nos. 13X-0229/B1 and 13X-0229/B2) on Highway 401. The purpose of the work is to assess the subsurface conditions at the location of the proposed bridge widenings by drilling a limited number of boreholes, completing in situ testing and completing geotechnical and analytical laboratory testing on selected soil samples obtained from the boreholes to support the development of foundation engineering recommendations for the design. These data are supplemented by four boreholes advanced as part of a 1959 investigation by others (GEOCREC No. 40J08-009) for the original construction of the Highway 401 eastbound and westbound bridges. Additional information is available from GEOCREC No. 40J08-037 related to the crossing of the drain to the south on Seventh Line West.

The terms of reference for the scope of work are provided in MTO's Request for Proposal for Assignment No. 3018-E-0011, dated December 2018, Section 3.7.1 – Foundation Engineering of Stantec's Technical Proposal.

2.0 SITE DESCRIPTION

The Flook and Hinton Drain bridges are located approximately 430 m west of the Highway 401/Bloomfield Road bridge and interchange in the County of Kent, Ontario. The orientation (i.e., north, south, east, west) stated in the text of this report is referenced to project north and therefore may differ from magnetic north. For the purposes of this report, Highway 401 is considered to be oriented east-west and the Flook and Hinton Drain to be oriented north-south.

Two single-span concrete rigid frame bridges, one for the eastbound lanes and one for the westbound lanes, carry Highway 401 traffic over the Flook and Hinton Drain. The existing rigid frame bridges were constructed in 1961, and have decks about 11.3 and 13.7 metres (m) wide and clear spans of about 17.2 m. The spread footings supporting the existing abutments and the median retaining walls between the abutments are about 2.5 m wide and are founded at about Elevation 175.4 m. Steel sheet piles were planned to be installed in front of the existing abutment and retaining wall footings, as shown on the 1959 design drawings (refer to Appendix D).

Highway 401 has been constructed on a low embankment with its grade at approximately Elevation 181.0 m at the structure site, and the low point of the grassy median swale is at approximately Elevation 180 m to the west and east of the bridges, above the crest of the slopes toward the Drain. The existing embankment slopes are on the order of 2 m high adjacent to the outer lanes and the median swale is about 1 m deep. No displacements or distortion of the pavements or approach embankments were noted. The channel base is at approximately Elevation 175.5 m at the structure site, with the typical watercourse level at approximately Elevation 176.1 m. The topography

in the area of the existing Highway 401 crossing of the Flook and Hinton Drain consists of flat to gently undulating agricultural land.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation was carried out between August 11 and 13, 2020, during which time two sampled boreholes (designated as Boreholes BH-801 and BH-802) were advanced as near as practicable to the footprint of the proposed abutments for the median widening structures. The locations of the boreholes are shown on Drawing 1 following the text of this report.

The boreholes were drilled using track-mounted drilling equipment supplied and operated by specialist drilling contractors. The boreholes were advanced through the overburden and bedrock to depths of 24.7 m to 25.0 m using HW casing and mud rotary techniques in the overburden and HQ size core barrels in the bedrock. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50-millimetre (mm) outer diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)¹ and using 76 mm O.D. thin-walled 'Shelby' Tube samplers (ASTM D1587-00)² to obtain relatively undisturbed samples in the cohesive soils. Field vane shear tests were carried out in the cohesive soils for assessment of undrained shear strength (ASTM D2573)³ using an MTO standard N-size vane. The results of the in situ field tests (i.e. SPT "N" values and undrained shear strengths from the field vane tests) as presented on the borehole records in Appendix A and in Section 4 are uncorrected.

The mud rotary drilling techniques precluded observations of the groundwater conditions and water levels in the boreholes. Upon completion of drilling and sampling, the boreholes were backfilled with bentonite grout in accordance with Ontario Regulation 903 (as amended).

The field work was observed by a member of WSP Golder's technical staff who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to WSP Golder's London geotechnical laboratory where the samples underwent further visual examination. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. One-dimensional consolidation (oedometer) testing was carried out on two samples of the cohesive deposits in WSP Golder's Mississauga, Ontario laboratory. All of the geotechnical laboratory tests were carried out to MTO and/or ASTM Standards, as applicable, and the results are presented in Appendix B.

The as-drilled borehole locations and the ground surface elevations were obtained using a GPS (Trimble XH 3.5G) having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The locations given on the borehole/drillhole records and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 11) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations including both northing and easting coordinates and geographic coordinates of latitude and longitude, ground surface elevations and drilled depths are summarized below.

¹ ASTM D1586-08a - Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.

² ASTM D1587-15 - Standard Practice For Thin-Walled Tube Sampling Of Fine-Grained Soils For Geotechnical Purposes.

³ ASTM D2573-15 - Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

| Borehole No. | Location (MTM NAD 83, Zone 11) | | Ground Surface Elevation (m) | Borehole Termination Depth (m) | Borehole Termination Elevation (m) |
|--------------|--------------------------------|-------------------------------|------------------------------|--------------------------------|------------------------------------|
| | Northing (m) (Latitude, °) | Easting (m) (Longitude, °) | | | |
| BH-801 | 4,689,932.4 (42.348572) | 330,527.6 (-82.187730) | 180.6 | 25.0 | 155.6 |
| BH-802 | 4,689,895.8 (42.348283) | 330,495.3 (-82.188125) | 180.5 | 24.7 | 155.8 |

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This project lies within the physiographic region of southwestern Ontario known as the Bothwell Sand Plain which was the delta of the Thames River in prehistoric glacial Lake Warren. The Bothwell Sand Plain primarily consists of a thin layer of sand, approximately 1 metre thick, over the clay.⁴ Quaternary geology mapping indicates that surficial materials consist primarily of glaciolacustrine deposits of clayey silt and silty sand overlain by glaciolacustrine silty sand and sand.⁵ The mapping also indicates that a “till moraine” is present immediately south of the site. Based on geologic mapping the underlying bedrock surface is estimated to be about 25 m below the ground surface or at about Elevation 160 m.⁶ The rock is described as limestone, dolostone and shale of the Hamilton Group of middle Devonian age.⁷

Although the mapping provides a general indicator of the geologic conditions of the site, these maps only address the most recent phase of the region’s glacial geology based on near-surface materials and may not characterize the geologic complexity of the site at greater depths. In southwestern Ontario, the most significant prehistoric glacial features are associated with the last advance and retreat of ice through the area. As the ice receded from the region, a number of moraines and lakes were formed near the retreating ice front. In some areas, such as the Windsor-Chatham-Wallaceburg area, the clayey silt or silty clay deposits have a grain size distribution consistent with that of a cohesive glacial till although the density and strength of the materials are not consistent with deposition below a grounded ice sheet as commonly assumed for materials described as glacial till. Some of the soils described as glacial till were likely deposited from the underside of floating ice through a shallow water depth as a diamict (broadly graded mud) and, therefore, the soil carried little or no weight of the overlying ice while in other areas, the ice sheet may have been grounded and produced hard cohesive glacial till. Further, there are also likely areas where the ice may have been floating or partially floating which has resulted in complex conditions.

⁴ Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

⁵ Kelly, R.I., 1991: Quaternary geology of the Chatham-Wheatley area; Ontario Geological Survey, Open File Map 163, scale 1:50 000.

⁶ Sado, E.V. and Faught, R.B. 1981: Drift Thickness of Chatham Area, Southern Ontario; Ontario Geological Survey Preliminary Map P.2453, Drift Thickness Series. Scale 1:50 000.

⁷ Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

4.2 Subsurface Conditions

The detailed subsurface soil, rock and groundwater conditions, as encountered in the boreholes advanced during the current investigation, together with the results of the geotechnical laboratory tests and in situ testing carried out, are presented on the borehole records in Appendix A; the geotechnical laboratory test sheets are provided in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile and cross-sections on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change and, moreover, the interpreted stratigraphy shown on Drawing 1 represents a simplification of the subsurface conditions. Further, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions in the area of the bridges consist of fill materials underlain by a thick deposit of firm to very stiff clayey silt, which is underlain by limestone bedrock. A more detailed description of the subsurface conditions encountered in the boreholes from the current investigation is provided in the following sections.

4.2.1 Fill

Sand and gravel fill was encountered at ground surface in both boreholes. These layers were about 400 mm to 450 mm thick.

Beneath the sand and gravel fill, layers of clayey silt fill were encountered. The cohesive fill was noted to contain varying quantities of topsoil and rootlets. The cohesive fill was about 2.5 and 4.0 m thick in Boreholes BH-801 and BH-802, respectively. The Standard Penetration Test (SPT) “N”-values measured within the cohesive fill ranged from 4 blows to 13 blows per 0.3 m of penetration indicating that the cohesive fill has firm to stiff consistency. An Atterberg limits determination carried out on a sample of the cohesive fill indicated a liquid limit of about 33%, a plastic limit of about 18% and a plasticity index of about 15%, confirming that the tested sample consists of clayey silt of low plasticity. Samples of the cohesive fill had water contents ranging from about 16% to 27% with an average water content of about 22%. The results of grain size distribution tests carried out on three samples of the clayey silt fill are shown on Figure B-1 in Appendix B.

4.2.2 Clayey Silt

A deposit of clayey silt containing some sand and trace gravel was encountered below the cohesive fill material in Boreholes BH-801 and BH-802 at about Elevations 177.6 m and 176.1 m, respectively. The cohesive deposit was about 17.3 m to 18.7 m thick as encountered in these boreholes.

The SPT “N”-values measured within the cohesive deposit range between 4 blows and 21 blows per 0.3 m of penetration. In situ field vane tests carried out within the cohesive stratum measured undrained shear strengths generally greater than 96 kilopascals (kPa) with a zone in Borehole BH-802 between about Elevation 164 m and 160 m having undrained shear strengths ranging from about 64 kPa to 75 kPa with a calculated sensitivity between about 1.7 and 2.3. The field vane test results along with the measured SPT “N”-values indicate that the clayey silt deposit has a firm to very stiff consistency, but is generally stiff to very stiff.

The results of grain size distribution tests carried out on five samples of the clayey silt deposit are shown on Figure B-2 in Appendix B. Atterberg limits tests were carried out on five samples of this deposit which indicated liquid limits ranging between about 21% and 35%, plastic limits ranging between about 15% and 18%, and plasticity indices ranging between about 13% and 18%. These results, which are plotted on the plasticity chart on Figure B-

3 in Appendix B, indicate that the cohesive deposit can be classified as a clayey silt of low plasticity. Samples of the clayey silt had water contents that ranged from about 13% to 33% with an average water content of about 19%.

Laboratory consolidation testing was carried out on two samples of the clayey silt obtained from Borehole BH-802. The consolidation test results are included on Figures B-4 and B-5 in Appendix B, and the test results are summarized below. The compressibility characteristics will vary with depth in accordance with the water content and shear strength profiles.

| Borehole and Sample No. | Sample Depth/Elevation | σ_{vo}' (kPa) | σ_p' (kPa) | $\sigma_p' - \sigma_{vo}'$ (kPa) | OCR | C_c | C_r | e_o |
|-------------------------|------------------------|----------------------|-------------------|----------------------------------|-----|-------|-------|-------|
| BH-802, Sa 14 | 14.0 m / 166.5 m | 170 | 280 | 110 | 1.6 | 0.153 | 0.013 | 0.54 |
| BH-802, Sa 18 | 20.1 m / 160.4 m | 260 | 360 | 100 | 1.4 | 0.140 | 0.021 | 0.47 |

* For stress range between the approximate in situ effective overburden stress and final stress due to proposed embankment construction.

where: σ_{vo}' is the effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
OCR is overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is initial void ratio
 c_v is the coefficient of consolidation in cm^2/s

4.2.3 Limestone Bedrock

Bedrock was encountered underlying the clayey silt in Boreholes BH-801 and BH-802 at about Elevation 159.0 m and 158.9 m, respectively, and was cored for approximately 3.4 m prior to terminating the boreholes. The bedrock consists of dark grey, medium strong to very strong limestone. The total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) for each core run are summarized as follows:

| Borehole No. | Core Run | Core Depth Interval (m) | TCR (%) | SCR (%) | RQD (%) |
|--------------|----------|-------------------------|---------|---------|---------|
| BH-801 | 1 | 21.6 to 23.2 | 95 | 92 | 83 |
| BH-801 | 2 | 23.2 to 24.7 | 96 | 93 | 93 |
| BH-801 | 3 | 24.7 to 25.0 | 108 | 108 | 83 |
| BH-802 | 1 | 21.7 to 23.2 | 100 | 90 | 77 |
| BH-802 | 2 | 23.2 to 24.7 | 98 | 0* | 0* |

* vertical fracture throughout core

4.3 Groundwater and Natural Gas Conditions

The boreholes were drilled using mud-rotary drilling techniques which introduces water and drilling fluids into the boreholes precluding direct observations of groundwater conditions. Based on the soil colour change from brown to grey, the inferred groundwater level in the clayey silt is at approximately Elevation 176.5 m. The groundwater level will be subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

Suspected natural gas pockets were noted in both boreholes at approximately Elevation 165 m (corresponding to a depth of approximately 15 m below existing ground surface), based on observation of moderate gas bubbling in the drilling fluids.

4.4 Analytical Testing of Soil Samples

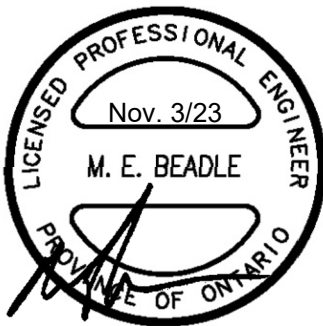
Two soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soils to steel and concrete, and the results are summarized as follows:

| Parameter | BH-801 Sample 4 (Elev. 177.2 m) | BH-802 Sample 6 (Elev. 175.7 m) |
|---------------------------------|---------------------------------------|---------------------------------------|
| pH | 7.88 | 7.32 |
| Resistivity (ohm-cm) | 1,160 | 412 |
| Electrical Conductivity (mS/cm) | 0.864 | 2.43 |
| Chlorides (µg/g) | 276 | 44 |
| Soluble Sulphates (µg/g) | 182 | 2,560 |

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Michael Beadle, P, Eng., a Senior Principal geotechnical engineer with WSP Golder. Ms. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP Golder, conducted an independent review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 401/FLOOK AND HINTON DRAIN BRIDGES
(SITE Nos. 13X-0229/B1 & B2)
COUNTY OF KENT, ONTARIO
MTO GWP 3024-18-00, ASSIGNMENT No. 3018-E-0011**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides discussion and preliminary recommendations on geotechnical/foundation aspects for the design of the Flook and Hinton Drain bridge widenings. The recommendations are based on WSP Golder's interpretation of the factual information obtained during the field explorations and geotechnical laboratory testing. The discussion and recommendations presented are intended to provide the designers with information to assess the feasible design and construction alternatives and to design the bridge foundations.

This Preliminary Foundation Design Report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor or design-build proponents. Contractors undertaking the work must make their own interpretation based on the factual data presented in Preliminary Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects which could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

As part of this project, Highway 401 will be widened from four to six lanes with the new lanes constructed towards the median. Currently, two single-span rigid frame bridges carry Highway 401 traffic over the Flook and Hinton Drain: one structure for westbound traffic and one structure for eastbound traffic. According to the 1959 design drawing (Raleigh Township Bridge No. 6, General Plan, dated November 1959), the existing bridges and the retaining walls between the bridges in the median are supported on spread footings bearing on the native clayey silt at about Elevation 175.4 m. Steel sheet piles were planned to be installed in front of the existing abutment and median retaining wall footings during construction, as shown on the design drawing dated November 1959, which are contained in Appendix C for reference.

Based on the preliminary General Arrangement drawing prepared by Stantec, the existing WBL and EBL bridges will be widened by approximately 5.9 m and 5.7 m toward the median, respectively, as measured along the abutments. The existing embankment slopes are on the order of 2 m high adjacent to the outer lanes and the median swale is about 1 m deep. No displacements or distortion of the existing pavements or approach embankments were noted at the time of the foundation investigation. As the existing bridge structures will remain in place, no grade raise is proposed on the existing Highway 401 approach embankments, which are at approximately Elevation 181.0 m. Minor filling on the order of 1 m will be required in the existing median swale, which has a current low point at approximately Elevation 180 m, to match the Highway 401 pavement grades.

Both shallow and deep foundation options have been considered for support of the median widening of the existing Flook and Hinton Drain bridges. A summary of the advantages and disadvantages associated with each foundation option is presented below.

- **Strip or spread footings founded within the firm to stiff clayey silt deposit to match the existing footings:** This option would require excavation to a depth of about 5.5 m to 6.5 m relative to the Highway 401 median and pavement grade for construction of the new widened abutment footings to match the existing at approximately Elevation 175.4 m, via removal and replacement or modification of the existing retaining wall footings. Temporary protection systems will be required along the median edges of the

Highway 401 EBL and WBL to facilitate this excavation and construction. It is estimated that there is potential for up to about 10 mm to 15 mm of differential settlement of the widened portion of the structure foundations relative to the existing structures, provided the new footings are structurally connected to the existing footings; this settlement is anticipated to occur within about two to three years following construction of the widenings.

- **Steel H-piles or pipe piles driven to found on limestone bedrock:** Driven steel H-piles or pipe piles are feasible for support of the abutments for the median widening; this would entail end-bearing piles that terminate at approximately Elevation 158 m to 159 m, approximately 22 m to 23 m below the Highway 401 grade. If the pile caps are founded at the same level as the spread footings for the existing bridges, the piles would be approximately 15.5 m to 16.5 m in length; this approach would require deeper excavations and protection systems similar to that for spread footings, as discussed above. The pile caps could be “perched” within the approaches above the Drain grade, and in this case could permit integral or semi-integral abutments; however, the existing structures may not be able to be modified to be compatible, and given the relatively narrow widening it is understood that integral abutments are not considered to be advantageous from a structural perspective compared to shallow footings. If structurally compatible, a perched pile cap would be advantageous in minimizing the depth of excavation compared to founding spread footings to match the existing bridge footings, and so minimize the requirements for temporary protection systems; in addition, differential settlement between the existing structures and the median widening would be negligible for this foundation type.
- **Drilled shafts (caissons) founded on limestone bedrock:** Caissons extended to bedrock are feasible for support of the abutments for the median widening. This option could be constructed with removal and replacement of the existing retaining wall footings to implement a pile cap consistent with the existing abutment footings, with similar requirements for depth of excavation and protection systems as noted above. Alternatively, caisson construction could be completed behind the existing retaining wall in conjunction with a perched pile cap, potentially with the existing retaining walls maintained to retain the soils; although this option could minimize depth of excavation, protection system and removal requirements, it poses structural compatibility challenges and risks associated with continued use of the existing retaining walls between the abutments.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the structure widening on spread footings founded on the very stiff to hard silty clay, to match and structurally connect to the existing footings. This may include either removal and replacement of the existing retaining wall footings, or modification of the existing retaining wall footings as may be required following removal of the existing retaining walls. The use of driven steel piles is also considered a viable foundation alternative for the median widening at this structure site.

6.3 Design Considerations

6.3.1 Consequence and Site Understanding Classification

The proposed Highway 401/Flook and Hinton Drain bridges carry large volumes of traffic with the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2019 *Canadian Highway Bridge Design Code* CAN/CSA S6-19 and its Commentary (CHBDC 2019), the proposed bridge and its foundation system is considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree

of site understanding in Section 6.5 of the CHBDC (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the CHBDC have been used for design.

6.3.2 Seismic Design

6.3.2.1 Seismic Parameters

The new bridge is in Seismic Performance Category (SPC) 1 and therefore seismic analysis of bridges in SPC 1 is not a requirement of the CHBDC (Clause 4.4.5.1). However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clauses 4.4.10.2 and 4.4.10.5.

6.3.2.2 Seismic Hazard Assessment and Liquefaction Potential

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.⁸ The characteristics of the cohesive soils indicate that they are not susceptible to liquefaction. Although layers of saturated granular materials are present, they are relatively thin. The liquefaction potential is considered low based on the soil profile type, age of the deposits, relative density/consistency and the historically low regional seismicity. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted.

6.3.2.3 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation and in situ testing. Based on the energy-corrected average penetration resistance, \bar{N}_{60} below the founding level and the measured undrained shear strengths, the site may be classified as Site Class D in accordance with Table 4.1 of the 2019 CHBDC, in the absence of any geophysical testing. Geophysics testing, if carried out, may provide a more favorable Site Class designation.

The 2019 CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The recommendations in this report have been developed based on the 5th generation seismic hazard maps developed by the GSC, which were made available for public use in December 2015. The values in the following section can be updated based on the 6th generation seismic hazard maps if warranted.

6.3.2.4 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the 2019 CHBDC, the peak ground acceleration (PGA), peak ground velocity (PGV) and 5 per cent damped spectral response acceleration ($S_a(T)$) values for Site Class C are presented below.

Site Class C Spectral Values for Subject Site

| Seismic Hazard Values | 2% Probability of Exceedance in 50 years (2,475-year return period) |
|-----------------------|--|
| PGA (g) | 0.069 |

⁸ FHWA, 1997: “Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles.” *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.

| Seismic Hazard Values | 2% Probability of Exceedance in 50 years (2,475-year return period) |
|-----------------------|--|
| PGV (m/s) | 0.054 |
| $S_a(0.2)$ (g) | 0.114 |
| $S_a(0.5)$ (g) | 0.070 |
| $S_a(1.0)$ (g) | 0.039 |
| $S_a(2.0)$ (g) | 0.019 |
| $S_a(5.0)$ (g) | 0.005 |
| $S_a(10.0)$ (g) | 0.002 |

The values given above are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given above in this report (Site Class D) in accordance with Section 4.4.3 of the CHBDC. The corresponding site-specific Site Class D seismic hazard values given in the table below can be used for design.

Site Class D Spectral Values for Subject Site

| Seismic Hazard Values | 2% Probability of Exceedance in 50 Years (2,475-year) |
|-----------------------|--|
| PGA (g) | 0.089 |
| PGV (m/s) | 0.079 |
| $S_a(0.2)$ (g) | 0.141 |
| $S_a(0.5)$ (g) | 0.103 |
| $S_a(1.0)$ (g) | 0.060 |
| $S_a(2.0)$ (g) | 0.030 |
| $S_a(5.0)$ (g) | 0.008 |
| $S_a(10.0)$ (g) | 0.003 |

6.4 Shallow Foundations

6.4.1 Founding Elevation and Geotechnical Resistances

The abutments for the structure widenings may be founded on conventional spread footings founded at the same elevation and with the same widths as the existing footings; removal and replacement or modification of the existing retaining wall footings will be required. All footings should be provided with a minimum of 1.0 m of earth cover or thermal equivalent for frost protection purposes, per Ontario Provincial Standard Drawing (OPSD) 3090.101.

A factored ultimate geotechnical resistance of 350 kPa and a factored serviceability geotechnical resistance of 150 kPa may be used for abutment footings founded on the very stiff clayey silt at approximately Elevation 175.4 m.

The serviceability geotechnical resistance is based on a settlement criterion of 25 mm; however, it is anticipated that the footing for the widening will be rigidly connected to the existing footings (i.e., dowelled connections and concrete poured neat to the existing footings) which will assist in load distribution, reduce differential settlements and reduce the overall settlement to about half this value. The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width or founding elevation vary from that given above. The factored ultimate geotechnical resistances provided are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, eccentricity and inclination of the load should be considered.

The stiff founding soils could be susceptible to disturbance and degradation on exposure to water and construction traffic and therefore, if this foundation option is adopted in conjunction with removal of the existing retaining wall footings, a concrete working slab is recommended to be placed over the subgrade to protect the integrity of the foundation soils.

6.4.2 Resistance to Lateral Forces/Sliding Resistance

Resistance to lateral forces/sliding between the concrete spread footing and the founding soils should be calculated in accordance with Section 6.10.4 of CHBDC 2019. Assuming that the founding soils are not loosened or disturbed during excavation and footing construction, the unfactored coefficient of friction ($\tan \phi'$) values between the cast-in-place concrete footings and the inspected and approved founding soils may be taken as follows:

| Subgrade Material | Coefficient of Friction, $\tan \phi'$ |
|---|---------------------------------------|
| Cast-in-place footing or working slab on compacted Granular 'A' | 0.70 |
| Cast-in-place footing or working slab on very stiff clayey silt | 0.58 |

6.5 Driven Piles

6.5.1 Tip Elevation and Axial Geotechnical Resistances

The foundations for the bridge widenings could also be founded on steel piles driven to practical refusal on the limestone bedrock. The underside of pile cap should be at Elevation 175.4 m (i.e., the same elevation as the existing spread footings, unless it is feasible to structurally connect a pile cap that is perched higher than this level. Pile caps should be provided with a minimum frost cover of 1.0 metres of soil cover or thermal equivalent.

For preliminary design, a factored ultimate axial geotechnical resistance of 3,000 kilonewtons (kN) per pile may be used for HP 310x110 piles driven to practical refusal on the bedrock at approximately Elevation 159 m. The factored serviceability geotechnical resistance for 25 mm and 10 mm of settlement will be higher than the factored ultimate geotechnical resistance associated with the medium strong to very strong limestone bedrock and, as such, ULS will govern the design. Given the nominal grade changes required to in-fill the median, which are expected to be less than 1 m, downdrag loads on the piles are not expected.

Pile installation should be completed in accordance with OPSS.PROV 903 (*Deep Foundations*). If this foundation option is adopted, the drawings should indicate "Piles to be driven to bedrock", and it is recommended that a tolerance of ± 2 m be included in the pile length to accommodate potential undulations or steps in the bedrock surface.

6.5.2 Other Details – Driving Shoes and Frost Protection

The clayey silt is known to contain cobbles and boulders which may interfere with driving of the piles or cause damage to pile tips. Further, the piles will be driven to the limestone bedrock. As such, all piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles. Driving shoes (such as Titus Standard “H” Bearing Pile points) are preferred over flange plates (OPSD 3000.10: Foundation Piles – Steel H-Pile Driving Shoe).

Pile caps should be provided with a minimum frost cover of 1.0 metres of soil cover or thermal equivalent.

6.5.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. Pile batter as much as 1H:3V is expected to be achievable. The horizontal reaction to the pile can be estimated using the following equation and ranges of soil properties.

$$K_h = \begin{array}{l} \text{coefficient of horizontal subgrade} \\ \text{reaction (MPa/m)} \end{array} = \frac{67 S_u}{d}$$

where

d = pile width or diameter (m)

S_u = undrained shear strength of the soil (MPa)

The following parameters may be incorporated into the calculations of the coefficient of horizontal subgrade reaction (K_h) for structural analysis of a single vertical pile.

| Elevation (m) | S _u (MPa) |
|---------------|----------------------|
| 164 to 175 | 0.150 to 0.200 |
| 160 to 164 | 0.065 to 0.075 |
| 159 to 160 | 0.150 to 0.200 |

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 including Figures C6.22 to C6.24 of the Commentary to the CHBDC (2019).

If the conditions are not met for use of horizontal subgrade reaction moduli, it is recommended that the lateral behaviour and resistance of the piles be assessed using non-linear P-y curves.

6.6 Lateral Earth Pressures for Design

The lateral pressures acting on the widened bridge abutments and associated wingwalls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type II or III should be used as backfill behind the abutments and walls. Compaction and placement should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements

with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirements*) and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).

A minimum compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC Section 6.12.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501. Other surcharge loads should be accounted for in the design, as required.

For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.0 m (equivalent to the depth of frost penetration at this site as interpreted from OPSD 3090.101) behind the back of the wall in accordance with Figure C6.31(a) of the Commentary to the CHBDC (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the Commentary to the CHBDC (2019).

- For restrained walls, the pressures are based on the existing embankment fill materials in the median and the following parameters (unfactored) may be used; the coefficient of lateral earth pressure is based on level ground above/behind the walls, as is applicable for the median widening at this site:

Soil unit weight: 20 kN/m³

Coefficients of lateral earth pressure:
At rest, K_o 0.50

- For unrestrained walls, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed; the coefficients of lateral earth pressure are based on level ground above/behind the walls, as is applicable for the median widening at this site:

| | <u>GRANULAR A</u> | <u>GRANULAR B</u> <u>Type III</u> |
|---|----------------------|--------------------------------------|
| Soil unit weight: | 22 kN/m ³ | 21 kN/m ³ |
| Coefficients of lateral earth pressure: | | |
| Active, K_a | 0.27 | 0.31 |
| Passive, K_p | 3.7 | 3.3 |

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the Commentary to the CHBDC (2019).

If the wall support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures should be assumed for geotechnical design.

If integral abutments are structurally feasible at this site considering structural compatibility and the skew, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In

practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.2 of the CHDBC.

6.7 Analytical Testing for Construction Material Impacts

The results of an analytical test on two samples of the upper materials are provided in Section 4.4 of this report. The analytical test results were compared to CSA A23.1 Table 3 ("Additional requirements for concrete subjected to sulphate attack") for potential sulphate attack on concrete. The sulphate concentration measured in one of the tested samples (2,560 µg/g) is classed as S-2 (Severe), while that in the second is below the exposure class of S-3 (Moderate).

The analytical test results of the soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The resistivity measured in both soil samples and the chlorides measured in one sample indicate strong corrosion potential.

Based on the results of the samples tested and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a "C" type exposure class as defined by CSA A23.1 Table 1. It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 "Durability Requirements" are followed.

6.8 Construction Considerations

6.8.1 Excavations and Temporary Cut Slopes

Excavations for removal of the existing retaining walls and removal/replacement or modification of the existing retaining wall footings will penetrate the existing fill materials and extend into the clayey silt and should be completed in accordance with OPSS.PROV 902 and FOUN0003. The groundwater level is expected to be at about Elevation 176.5 m and will fluctuate seasonally. The excavations may extend below the groundwater level; however, seepage volumes from the cohesive founding soils are expected to be relatively minor, although excavations may encounter some groundwater "perched" within non-cohesive fill layers or pockets. If necessary, groundwater control for such seepage may be achieved by pumping from properly constructed and filtered sumps in the base of the excavation in accordance with OPSS 517 and FOUN0003. Sumps should be maintained outside of the foundation limits. Surface water runoff should always be directed away from the excavations.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey materials would be classified as Type 2 soils.

6.8.2 Temporary Protection Systems

To support the excavation sides and permit the use of vertical cuts, temporary protection systems will be required where space is restricted and will not permit the use of open cuts. These systems are to be designed by and the limits determined by the contractor.

Temporary protection systems could consist of soldier piles and lagging, where the H-piles would be driven or installed within a pre-bored hole to a suitable depth and horizontal lagging installed as the excavation proceeds, or driven steel sheet piling. Support of the system(s) could be in the form of struts and walers in the case of pile cap/abutment excavations or rakers and anchors. The protection system must be designed to accommodate the

loads applied from pressures and surcharge pressures from area line or point loads as well as address the impact(s) of sloping ground behind the system. The lateral movement of the temporary support system should meet Performance Level 2 as specified in OPSS.PROV 539.

6.8.3 Obstructions

Cobbles and boulders should be expected in the soils at the site, which might impact installation of temporary protection systems and/or pile driving operations. An NSSP or Notice to Contractor should be added to the future Contract Documents to alert the contractor to the potential need to deal with cobbles, boulders and other obstructions, such as the steel sheeting that was installed along the Drain during the original bridge construction, during pile installation. Further, if adopted, driven piles would extend to the strong limestone bedrock. As such, all piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles. Driving shoes (such as Titus Standard “H” Bearing Pile points) are preferred over flange plates.

6.8.4 Monitoring of Existing Structures

Given the ground conditions (i.e., firm to very stiff cohesive soils that will attenuate vibrations over relatively short distances), ground vibrations generated by installation of temporary protection systems and/or pile driving are expected to be low for the neighbouring bridge foundations. It is likely not necessary to monitor vibrations full-time during all construction activities, but it is recommended that some limited vibration monitoring be included in the future contract documents to demonstrate that the vibrations are maintained within acceptable limits. Further, it is recommended that an elevation survey be conducted of each of the four corners of the existing bridges immediately prior to and following pile driving to document induced settlement, if any, associated with the pile driving. A visual and photographic survey of the existing bridge conditions should also be conducted immediately before and after pile driving. The visual and photographic survey should document any observed cracks in concrete, evidence of corrosion or other defects and each photograph should include a time and date stamp along with a scale in the photograph to assist with identifying the magnitude of any observable defects.

6.9 Recommendations for Additional Work in Detailed Design

The following additional foundation investigation and design is recommended as part of detailed design, in order to meet the typical requirements in MTO's Guideline for Foundation Engineering Services:

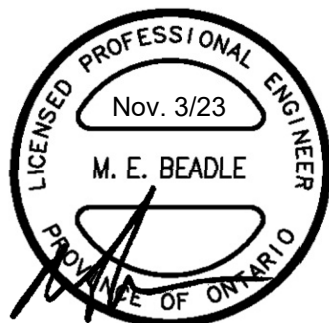
- One additional borehole in the median area near each abutment to provide further assessment of stiffness/consolidation properties of the cohesive soils. This may include conventional thin-walled tube sampling to obtain samples for oedometer testing, pressuremeter testing for assessment of elastic modulus, and/or implementation of CPT testing.
- Geophysics testing such as Multi-Channel Analysis of Surface Waves (MASW) or vertical seismic profiling to measure the site-specific seismic shear wave velocity for potential refinement of seismic site class and input to seismic design; such testing is generally considered helpful given the likely application of the higher values associated with the 6th generation seismic hazard mapping.

The preliminary foundation recommendations provided in this report should be reassessed and updated, as applicable, based on the results of additional investigation in detailed design. In addition, it is recommended that requirements for monitoring of the existing bridges during installation of protection systems and pile driving (if adopted) be assessed further during detailed design.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Michael Beadle, P.Eng. Senior Principal, and an independent technical and quality control review of this report was carried out by Ms. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP Golder.

WSP Golder



Michael E. Beadle, P. Eng.
Senior Principal Geotechnical Engineer

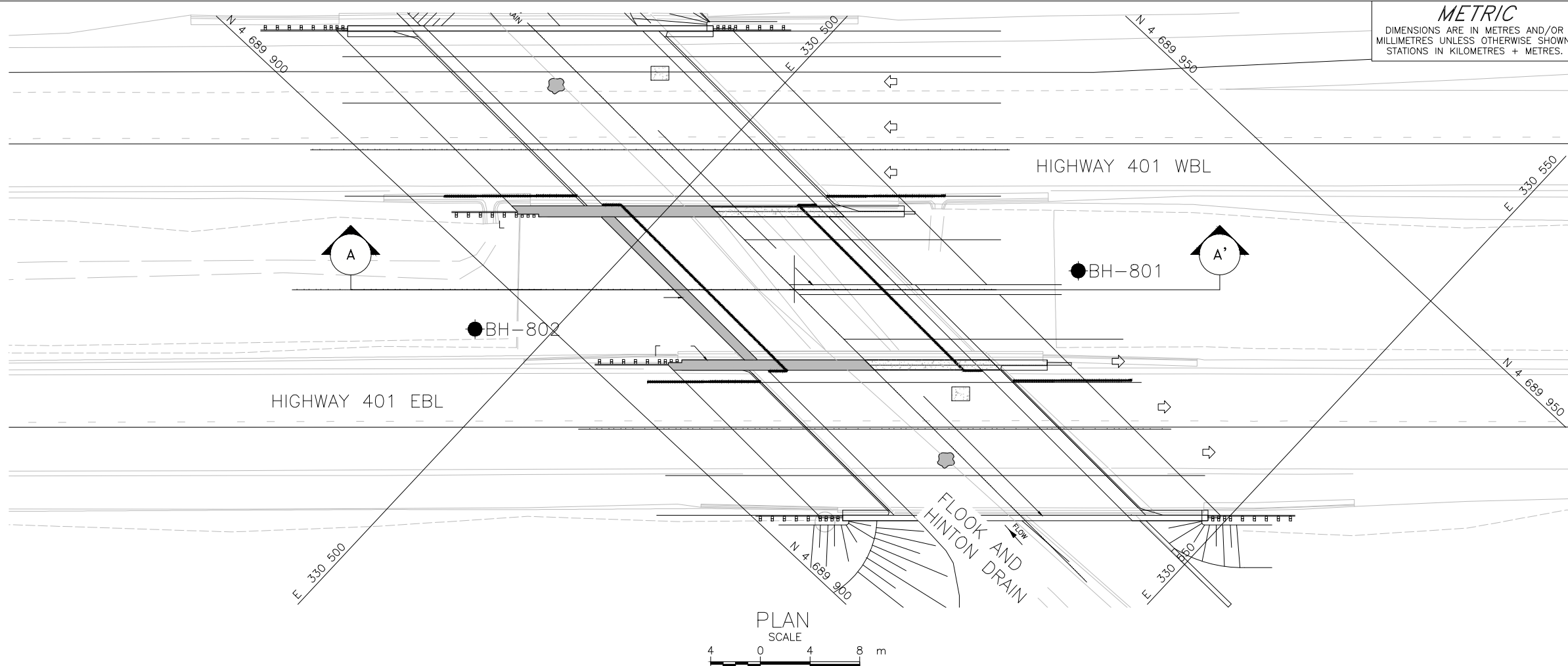


Lisa C. Coyne, P. Eng.
Fellow, MTO Principal Foundations Contact

RA/MEB/LCC/ms

[https://golderassociates.sharepoint.com/sites/110354/project files/6 deliverables/fdns/fidrs to finalize/19124560-f&h-rev0 final pfidr flook hinten \(gwp 3024-18-00\) oct 2023.docx](https://golderassociates.sharepoint.com/sites/110354/project%20files/6%20deliverables/fdns/fidrs%20to%20finalize/19124560-f&h-rev0%20final%20pfidr%20flook%20hinten%20(gwp%203024-18-00)%20oct%202023.docx)

Drawings

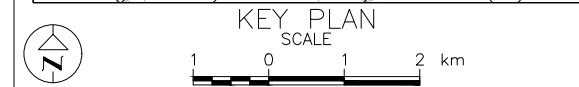
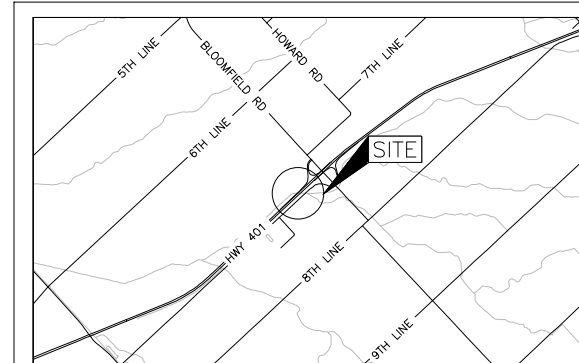


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.3024-18-00

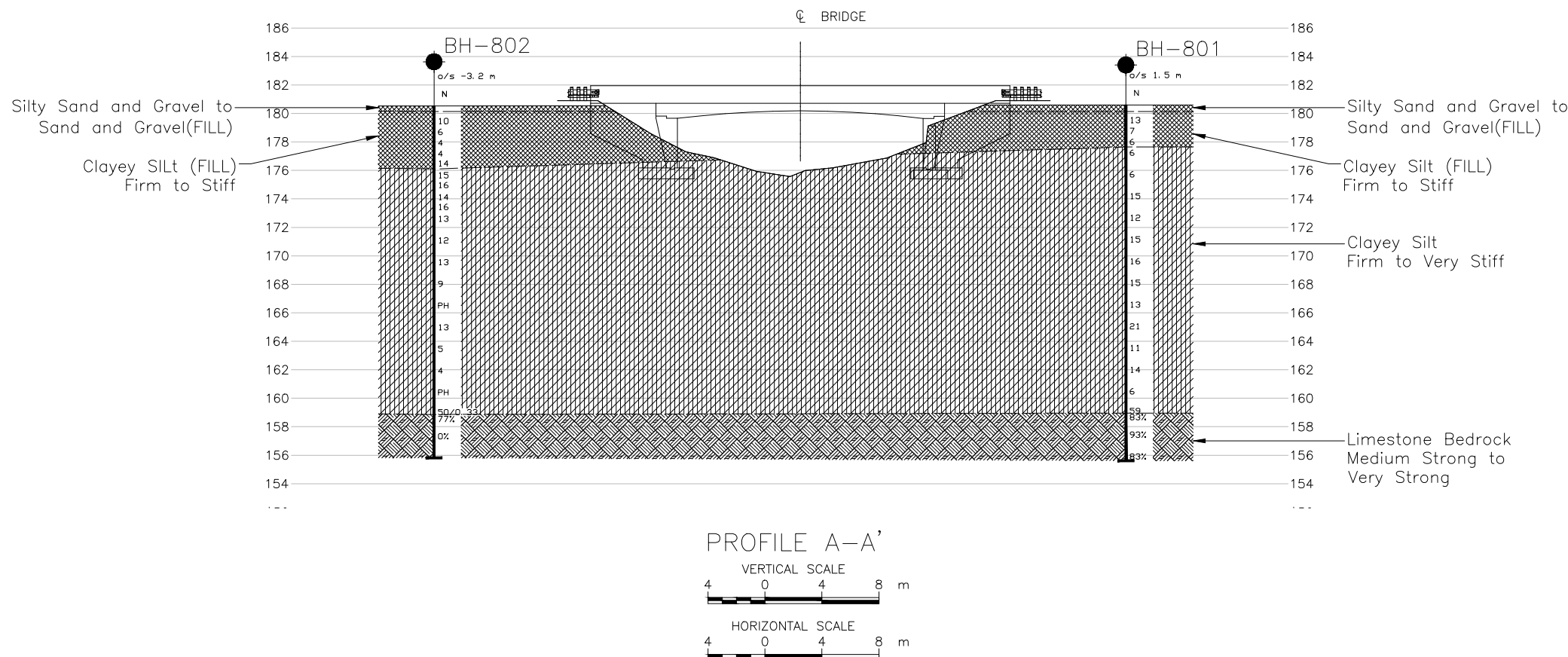
HIGHWAY 401
FLOOK AND HINTON DRAIN BRIDGE
BOREHOLES LOCATION PLAN AND
SOIL STRATA

SHEET



| LEGEND | | | |
|--------|--|--|--|
| | Borehole – Current Investigation | | |
| N | Standard Penetration Test Value | | |
| 16 | Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow) | | |
| 100% | Rock Quality Designation (RQD) | | |

| BOREHOLE CO-ORDINATES | | | |
|-----------------------|-----------|-----------|----------|
| No. | ELEVATION | NORTHING | EASTING |
| BH-801 | 180.6 | 4689932.4 | 330527.6 |
| BH-802 | 180.5 | 4689895.8 | 330495.3 |



NOTES
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE
Base plan provided in digital format by Stantec, drawing file nos. 944_HWY_401_XR_EBP.dwg.
GA provided in digital format by Stantec, drawing file no. 165000981-13-229-1-r2-01, received May 24, 2023.

| NO. | DATE | BY | REVISION |
|---------------------|----------------------|------------------|----------------------|
| | | | |
| Geocres No. 40J8-85 | | | |
| HWY. | PROJECT NO. 19124560 | DIST. | |
| SUBM'D. LCC | CHKD. LCC | DATE: 11/06/2023 | SITE: 13X-0229/B 1&2 |
| DRAWN: SA/DD | CHKD. MEB | APPD. LCC | DWG. 1 |

APPENDIX A

Borehole Records

[illegible]

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE



| PROJECT 19124560 | | | RECORD OF BOREHOLE No BH-801 | | | SHEET 2 OF 2 | | | METRIC | | | | | | |
|-------------------|---|------------|--|------|------------|-------------------------|-----------------|--|--------|--|------------------------------|----------------------------|-----------------------------|--|---------------------------------------|
| G.W.P. 3024-18-00 | | | LOCATION N 4689932.4; E 330527.6 MTM NAD 83 ZONE 11 (LAT. 42.348572; LONG. -82.187730) | | | ORIGINATED BY MR | | | | | | | | | |
| DIST West HWY 401 | | | BOREHOLE TYPE Power Auger; Hollow Stem; Mud Rotary | | | COMPILED BY DCH | | | | | | | | | |
| DATUM Geodetic | | | DATE Aug. 11, 2020 | | | CHECKED BY MEB/LCC | | | | | | | | | |
| 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 (%) |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | | | | | | | | |
| | CLAYEY SILT (CL), trace to some sand, trace gravel Firm to very stiff Grey Moist | | 12 | SS | 21 | | | | | | | | | | |
| | | | 13 | SS | 11 | | | | | | | | | | |
| | | | 14 | SS | 14 | | | | | | | | | | |
| | | | 15 | SS | 6 | | | | | | | | | | |
| 159.7 | | | | | | | | | | | | | | | |
| 20.9 | CLAYEY SILT (CL), trace sand, with sandy silt layers Firm Grey | | 16 | SS | 59 | | | | | | | | | | |
| 159.0 | | | | | | | | | | | | | | | |
| 21.6 | LIMESTONE BEDROCK Dark grey Medium strong to very strong | | | RC | REC 95% | | | | | | | | | | RQD = 83% |
| | | | | RC | REC 96% | | | | | | | | | | RQD = 93% |
| | | | | RC | REC 100% | | | | | | | | | | RQD = 83% |
| 155.6 | END OF BOREHOLE | | | | | | | | | | | | | | |
| 25.0 | NOTE: 1. Borehole dry during drilling on August 11, 2020, prior to introduction of water for bedrock coring. | | | | | | | | | | | | | | |

GTA-MTO 001 S:\CLIENTS\MTOWHY_401_CHATAM-LONDON\02_DATA\GINT\HWY_401_CHATAM-LONDON.GPJ GAL-GTA.GDT 11/6/23



| PROJECT | | RECORD OF BOREHOLE | | No BH-802 | | SHEET 1 OF 2 | | METRIC | | | | | | | | | | | | | | |
|--------------|-------|---|------------|---|------|-------------------------|-----------|---------|--|----|----|---|-----|----------------|-------------|----------------|---|---------------------------------------|----|----|----|----|
| G.W.P. | | LOCATION | | N 4689895.8; E 330495.3 MTM NAD 83 ZONE 11 (LAT. 42.348283; LONG. -82.188125) | | ORIGINATED BY | | MR | | | | | | | | | | | | | | |
| DIST | | HWY | | BOREHOLE TYPE | | COMPILED BY | | DCH | | | | | | | | | | | | | | |
| DATUM | | DATE | | Aug. 12-13, 2020 | | CHECKED BY | | MEB/LCC | | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | |
| ELEV | DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | ELEVATION | SCALE | 20 | 40 | 60 | 80 | 100 | W _p | W | W _L | γ | GR | SA | SI | CL | |
| 180.5 | 0.0 | GROUND SURFACE | | | | | | | | | | | | | | | | | | | | |
| 180.1 | 0.4 | SAND and GRAVEL, some silt (FILL) Brown Moist | | | | | | | | | | | | | | | | | | | | |
| | | CLAYEY SILT, trace sand, with topsoil and roots (FILL) Firm to stiff Brown to brown and grey Moist | | 1 | SS | 10 | 180 | | | | | | | | o | | | | 0 | 13 | 56 | 31 |
| | | | | 2 | SS | 6 | 179 | | | | | | | | o | | | | | | | |
| | | | | 3 | SS | 4 | 178 | | | | | | | | o | | | | | | | |
| 177.9 | 2.6 | CLAYEY SILT, some sand (FILL) Firm Brown and grey Moist | | 4 | SS | 4 | 177 | | | | | | | | o | | | 1 | 10 | 66 | 23 | |
| | | | | 5 | SS | 14 | | | | | | | | | o | | | | | | | |
| 176.1 | 4.4 | CLAYEY SILT (CL), some sand, trace gravel Firm to very stiff Grey Moist | | 6 | SS | 15 | 176 | | | | | | | | o | | | | | | | |
| | | | | 7 | SS | 16 | 175 | | | | | | | | o | | | | | | | |
| | | | | 8 | SS | 14 | 174 | | | | | | | | o | | | | | | | |
| | | | | 9 | SS | 16 | 173 | | | | | | | | o | | | | | | | |
| | | | | 10 | SS | 13 | | | | | | | | | o | | | 2 | 26 | 37 | 35 | |
| | | | | 11 | SS | 12 | 172 | | | | | | | | o | | | | | | | |
| | | | | 12 | SS | 13 | 171 | | | | | | | | o | | | | | | | |
| | | | | 13 | SS | 9 | 170 | | | | | | | | o | | | | | | | |
| | | | | | | | 169 | | | | | | | | | | | | | | | |
| | | | | | | | 168 | | | | | | | | o | | | | | | | |
| | | | | | | | 167 | | | | | | | | | | | | | | | |
| | | | | 14 | SS | PH | 166 | | | | | | | | | | | | | | | |

Continued Next Page

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

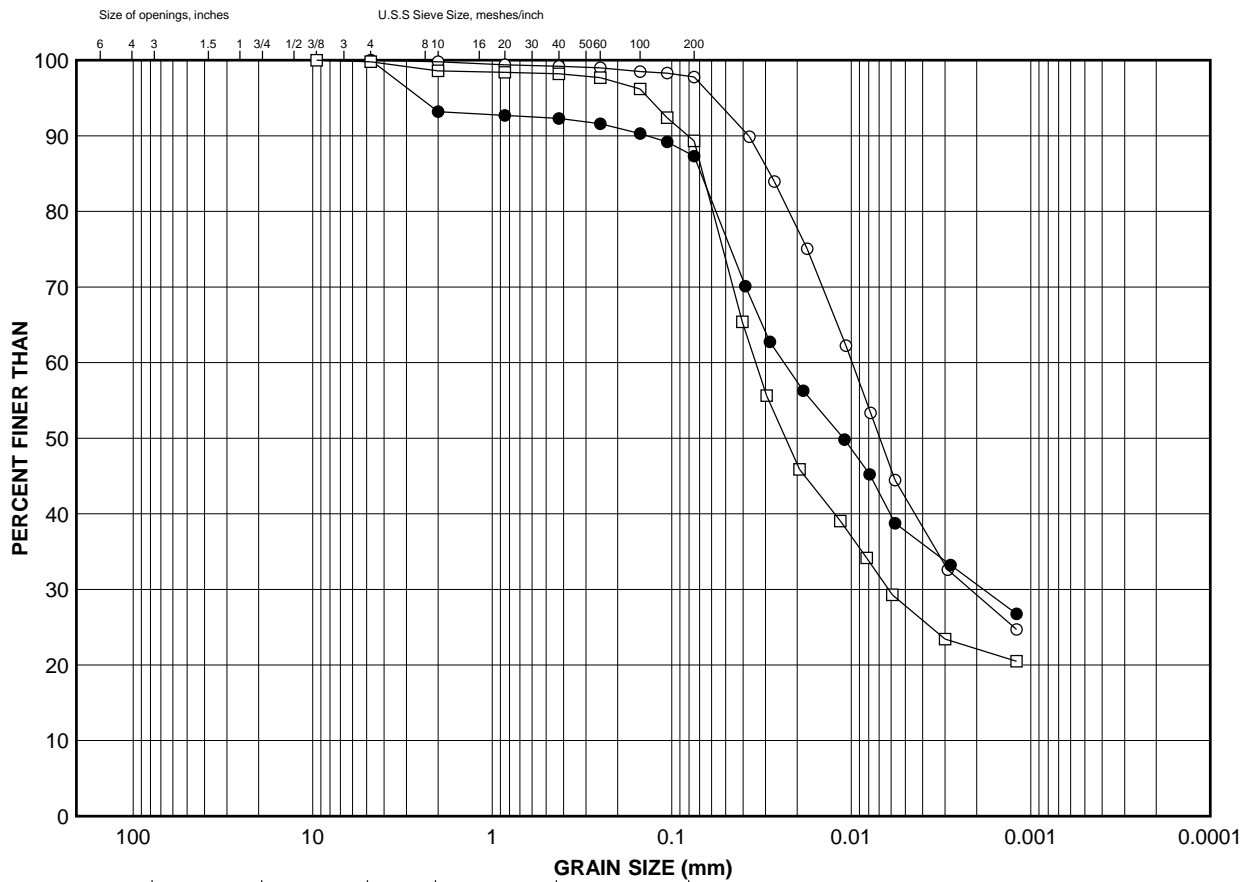
GTA-MTO 001 S:\CLIENTS\MTOWHY_401_CHATAM-LONDON\02_DATA\GINT\HWY_401_CHATAM-LONDON.GPJ GAL-GTA.GDT 11/6/23



| PROJECT | | 19124560 | | RECORD OF BOREHOLE | | No BH-802 | | SHEET 2 OF 2 | | METRIC | | | | | | |
|--------------------------------------|---|--------------|---------|--------------------|------------|---|-----------------|--|--|---------|--|---------------------------------|-------------------------------|--------------------------------|--|---------------------------------------|
| G.W.P. | | 3024-18-00 | | LOCATION | | N 4689895.8; E 330495.3 MTM NAD 83 ZONE 11 (LAT. 42.348283; LONG. -82.188125) | | ORIGINATED BY | | MR | | | | | | |
| DIST | | West HWY 401 | | BOREHOLE TYPE | | Power Auger; Hollow Stem; Mud Rotary | | COMPILED BY | | DCH | | | | | | |
| DATUM | | Geodetic | | DATE | | Aug. 12-13, 2020 | | CHECKED BY | | MEB/LCC | | | | | | |
| 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 (%) |
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | |
| --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | | | | | | | | | | |
| 161.0 | CLAYEY SILT (CL), some sand, trace gravel Firm to very stiff Grey Moist | | 15 | SS | 13 | | | | | | | | | | | |
| 164 | | | 16 | SS | 5 | | | | | | | | | | | 5 24 36 35 |
| 163 | | | | | | | | | | | | | | | | |
| 162 | | | 17 | SS | 4 | | | | | | | | | | | |
| 161 | | | | | | | | | | | | | | | | |
| 160 | | | | | | | | | | | | | | | | |
| 159 | | | 18 | SS | PH | | | | | | | | | | | |
| 158 | | | | | | | | | | | | | | | | |
| 157 | | | | | | | | | | | | | | | | |
| 156 | | | 19 | SS | 50/0.33 | | | | | | | | | | | |
| 155.8 | LIMESTONE BEDROCK Dark grey Medium strong to very strong | | | RC | REC 100% | | | | | | | | | | | RQD = 77% |
| 155.8 | | | | RC | REC 98% | | | | | | | | | | | RQD = 0% |
| 155.8 | END OF BOREHOLE | | | | | | | | | | | | | | | |
| 24.7 | NOTE: 1. Borehole dry during drilling on August 11, 2020, prior to introduction of water for bedrock coring. | | | | | | | | | | | | | | | |

APPENDIX B

Geotechnical Laboratory Test Results

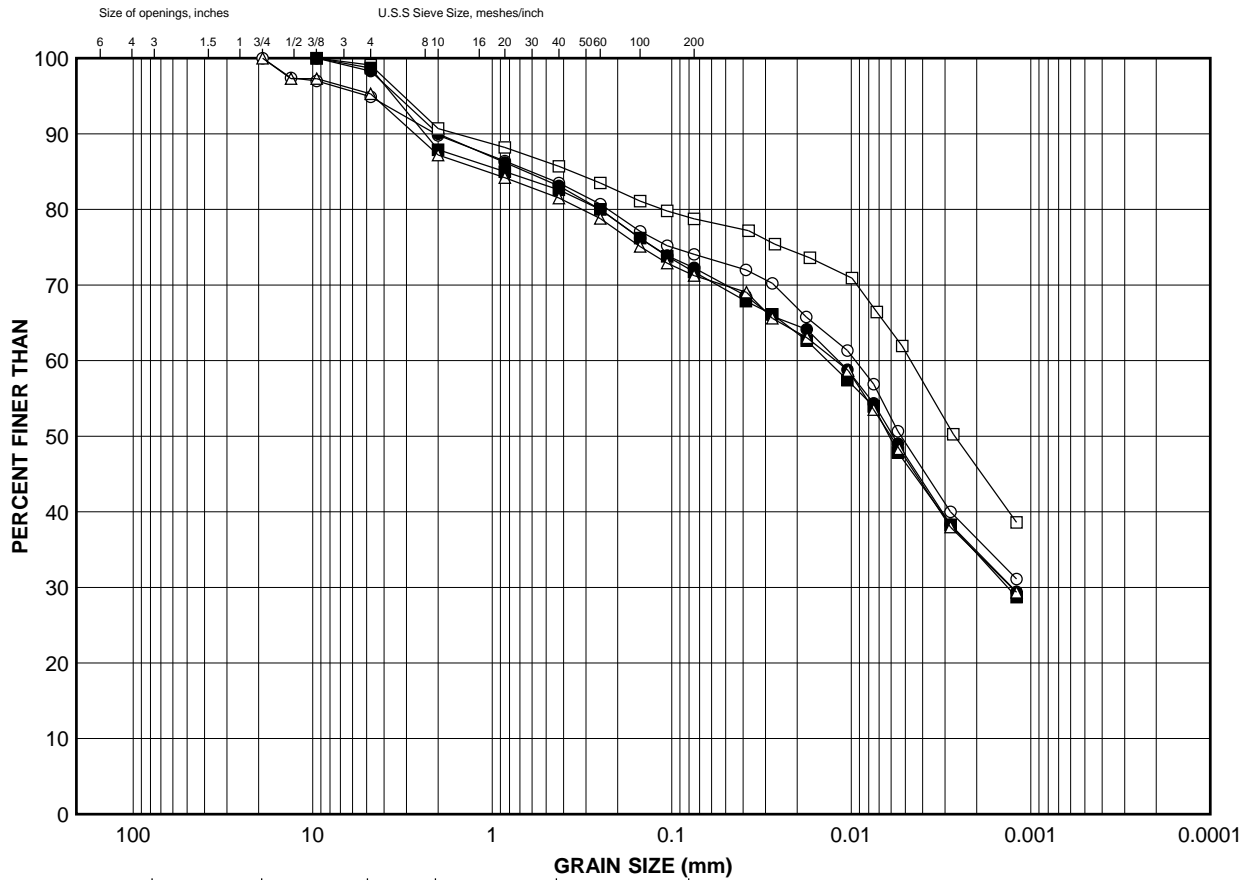


| | | | | | | |
|----------------|-------------|------|-----------|--------|------|---------------|
| Cobble Size | coarse | fine | coarse | medium | fine | SILT AND CLAY |
| | GRAVEL SIZE | | SAND SIZE | | | |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|--------|----------|--------|----------|
| ○ | 801 | 3B | 177.9 |
| ● | 802 | 1 | 179.5 |
| □ | 802 | 4 | 177.2 |


| | | | | | | | | | | | | | | | | | |
|---------|--|--|-------------|--|--|--|--|--|-----------|--|--|---|--|--|-----|--|--|
| PROJECT | | | | | | FLOOK AND HINTON DRAIN BRIDGES HIGHWAY 401 RECONSTRUCTION GWP 3024-18-00 | | | | | | | | | | | |
| TITLE | | | | | | GRAIN SIZE DISTRIBUTION CLAYEY SILT FILL | | | | | | | | | | | |
| | | | PROJECT No. | | | 19124560 | | | FILE No. | | | GSD-v2 - clayey silt fill.xlsm Figure 1 | | | | | |
| | | | DRAWN | | | MEB | | | NOV 28-22 | | | SCALE | | | N/A | | |
| | | | CHECK | | | | | | | | | REV. | | | 0 | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | FIGURE B-1 | | | | | |
| | | | | | | | | | | | | | | | | | |

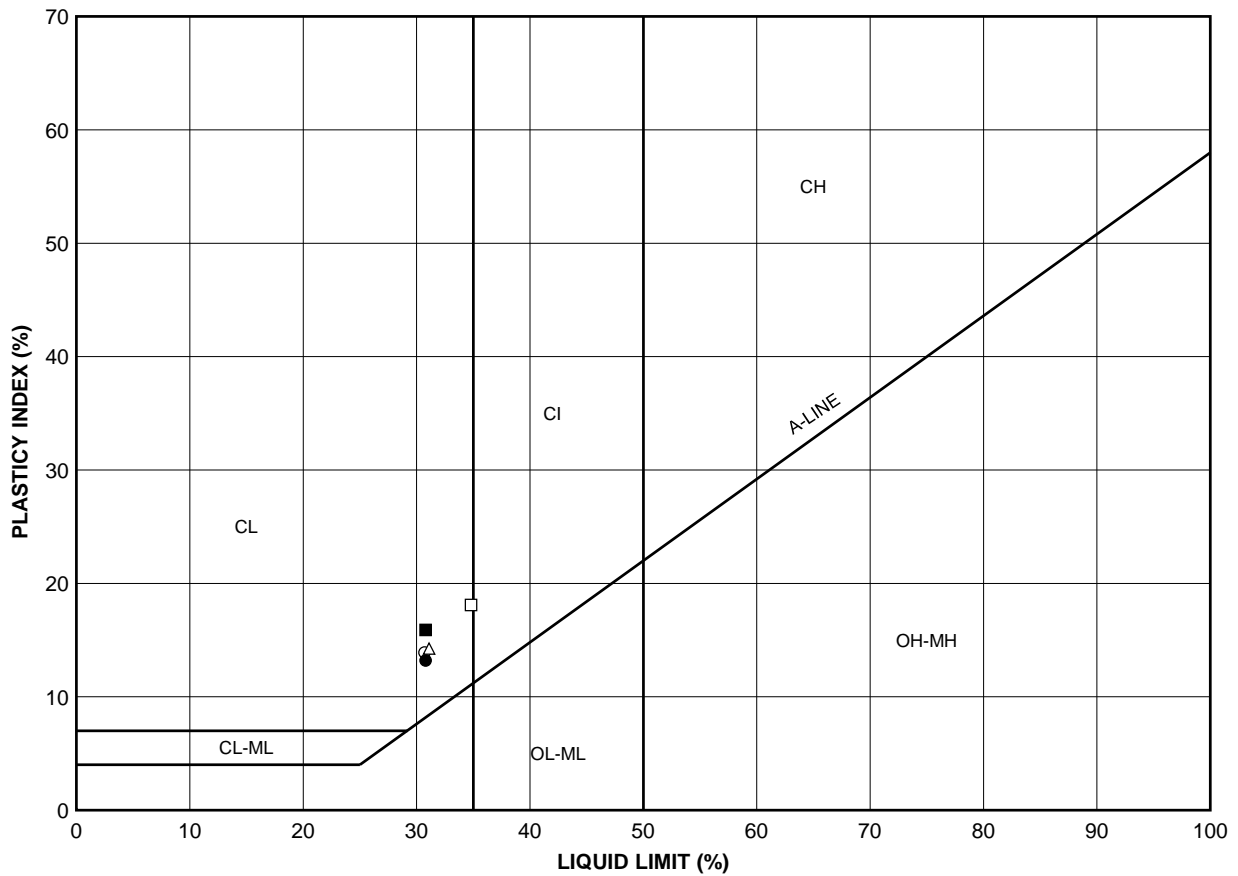


| | | | | | | |
|----------------|-------------|------|-----------|--------|------|---------------|
| Cobble Size | coarse | fine | coarse | medium | fine | SILT AND CLAY |
| | GRAVEL SIZE | | SAND SIZE | | | |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|--------|----------|--------|----------|
| ○ | 801 | 7 | 172.7 |
| ● | 801 | 11 | 166.6 |
| □ | 801 | 15 | 160.5 |
| ■ | 802 | 10 | 172.6 |
| △ | 802 | 16 | 163.5 |

| | | | | | | | | | | |
|--|--|-------------|-----|--|--|------------|--|---------------------------------------|--------|--|
| PROJECT | | | | FLOOK AND HINTON DRAIN BRIDGES HIGHWAY 401 RECONSTRUCTION GWP 3024-18-00 | | | | | | |
| TITLE | | | | GRAIN SIZE DISTRIBUTION CLAYEY SILT | | | | | | |
|  GOLDER | | PROJECT No. | | 19124560 | | FILE No. | | GSD-v2 - clayey silt xism Figure 1 | | |
| | | | | | | SCALE | | N/A | REV. 0 | |
| | | DRAWN | MEB | NOV 28-22 | | FIGURE B-2 | | | | |
| | | CHECK | | | | | | | | |
| | | | | | | | | | | |




SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

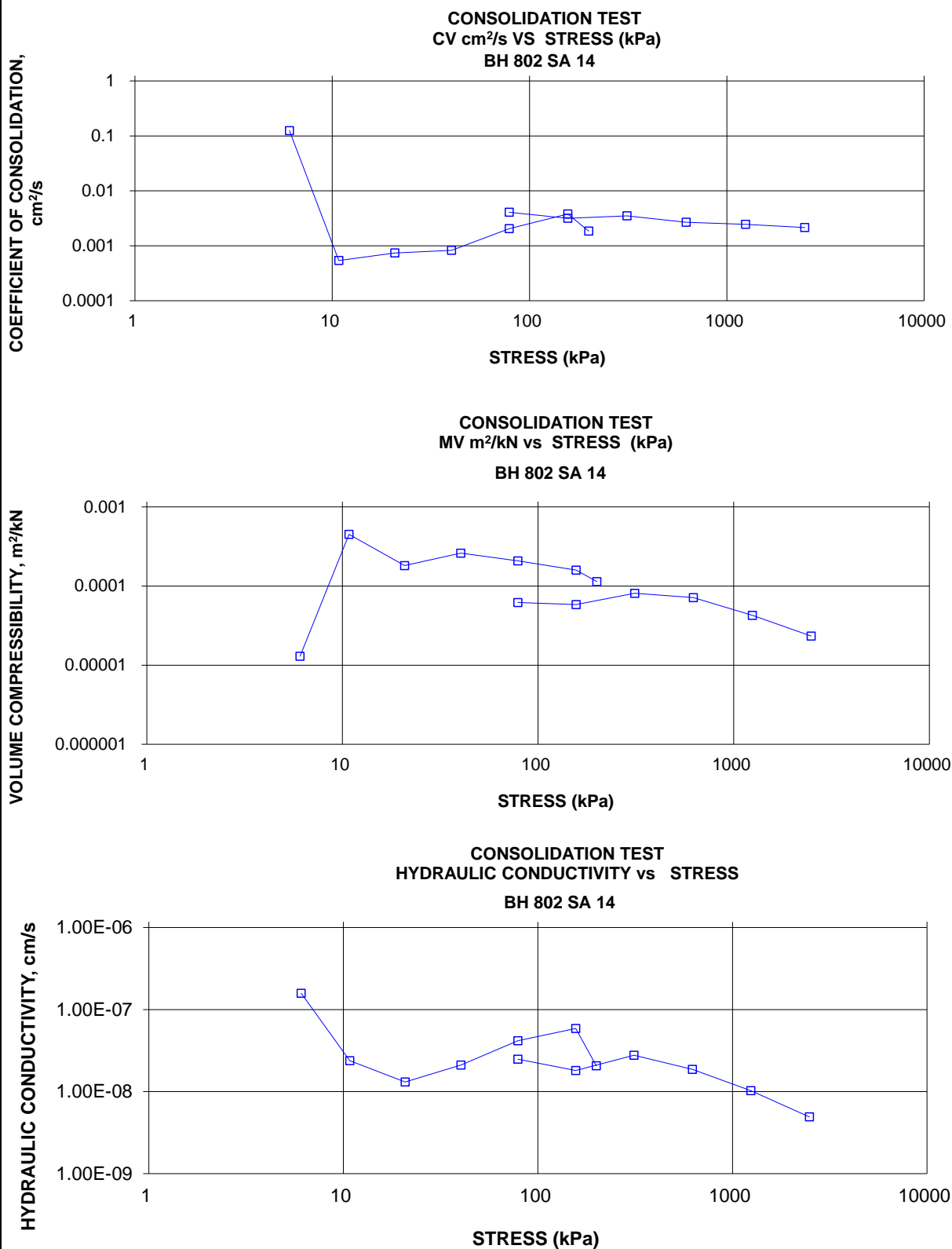
PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) | LL(%) | PL(%) | IP(%) |
|--------|----------|--------|----------|-------|-------|-------|
| ○ | 801 | 7 | 172.7 | 30.7 | 16.8 | 13.9 |
| ● | 801 | 11 | 166.6 | 30.8 | 17.6 | 13.2 |
| □ | 801 | 15 | 160.5 | 34.8 | 16.7 | 18.1 |
| ■ | 802 | 10 | 172.6 | 30.8 | 14.9 | 15.9 |
| △ | 802 | 16 | 163.5 | 31.1 | 16.9 | 14.3 |

| | | | | | | | |
|--|--|----------|--|--|--|---------------------------------------|--|
| PROJECT | | | | FLOOK AND HINTON DRAIN BRIDGES HIGHWAY 401 RECONSTRUCTION GWP 3024-18-00 | | | |
| TITLE | | | | PLASTICITY CHART CLAYEY SILT | | | |
| PROJECT No. | | 19124560 | | FILE No. | | Limits-MTO - clayey silt xls Figure 1 | |
| DRAWN | | MEB | | SCALE | | N/A | |
| CHECK | | | | REV. | | 0 | |
|  | | | | FIGURE B-3 | | | |

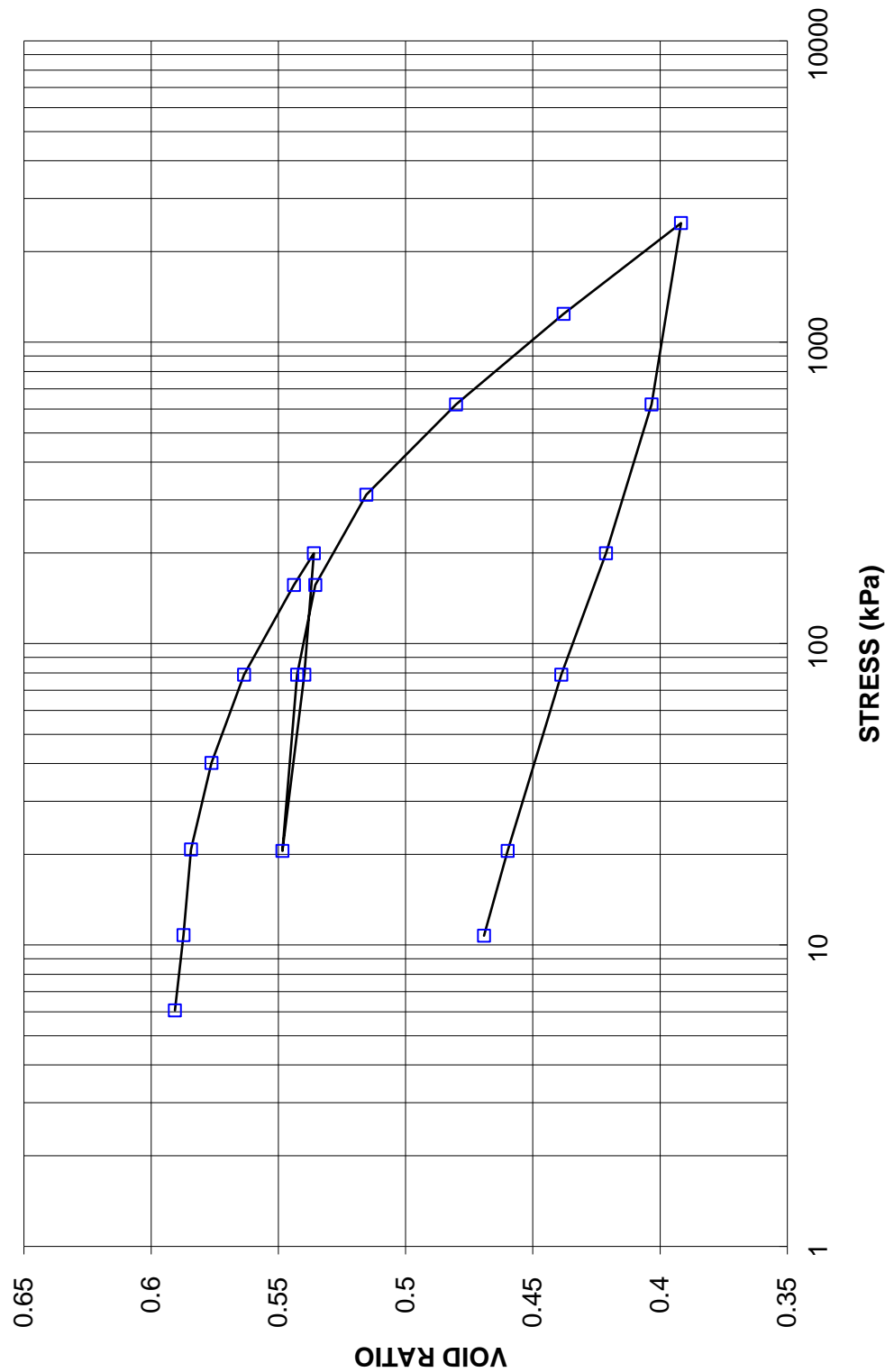
| | | | | | | | |
|--|---------------------|------------------------------------|-------------|--------------------------|---------------------|--------------------|----------|
| CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M | | | | | FIGURE B-4.1 | | |
| SAMPLE IDENTIFICATION | | | | | | | |
| Project Number | 19124560 | Sample Number | 14 | | | | |
| Borehole Number | 802 | Sample Depth, m | 13.72-14.33 | | | | |
| TEST CONDITIONS | | | | | | | |
| Test Type | Laboratory Standard | Load Duration, hr | 24 | | | | |
| Oedometer Number | 11 | | | | | | |
| Date Started | 11/11/2020 | | | | | | |
| Date Completed | 11/26/2020 | | | | | | |
| SAMPLE DIMENSIONS AND PROPERTIES - INITIAL | | | | | | | |
| Sample Height, cm | 2.54 | Unit Weight, kN/m ³ | 20.23 | | | | |
| Sample Diameter, cm | 6.34 | Dry Unit Weight, kN/m ³ | 16.64 | | | | |
| Area, cm ² | 31.55 | Specific Gravity, measured | 2.70 | | | | |
| Volume, cm ³ | 80.20 | Solids Height, cm | 1.598 | | | | |
| Water Content, % | 21.56 | Volume of Solids, cm ³ | 50.41 | | | | |
| Wet Mass, g | 165.47 | Volume of Voids, cm ³ | 29.78 | | | | |
| Dry Mass, g | 136.12 | Degree of Saturation, % | 98.5 | | | | |
| TEST COMPUTATIONS | | | | | | | |
| | Corr. | Average | | | | | |
| Stress | Height | Void | Height | t ₉₀ | cv. | mv | k |
| kPa | cm | Ratio | cm | sec | cm ² /s | m ² /kN | cm/s |
| 0.00 | 2.542 | 0.591 | 2.542 | | | | |
| 6.07 | 2.542 | 0.591 | 2.542 | 11 | 1.25E-01 | 1.30E-05 | 1.58E-07 |
| 10.79 | 2.536 | 0.587 | 2.539 | 2535 | 5.39E-04 | 4.50E-04 | 2.38E-08 |
| 20.76 | 2.532 | 0.584 | 2.534 | 1848 | 7.37E-04 | 1.82E-04 | 1.31E-08 |
| 40.23 | 2.519 | 0.576 | 2.525 | 1636 | 8.26E-04 | 2.61E-04 | 2.11E-08 |
| 78.88 | 2.498 | 0.564 | 2.509 | 653 | 2.04E-03 | 2.09E-04 | 4.18E-08 |
| 156.47 | 2.467 | 0.544 | 2.483 | 346 | 3.78E-03 | 1.59E-04 | 5.87E-08 |
| 199.67 | 2.455 | 0.536 | 2.461 | 694 | 1.85E-03 | 1.14E-04 | 2.06E-08 |
| 78.88 | 2.461 | 0.540 | 2.458 | | | | |
| 20.53 | 2.474 | 0.548 | 2.467 | | | | |
| 78.88 | 2.465 | 0.543 | 2.470 | 317 | 4.08E-03 | 6.20E-05 | 2.48E-08 |
| 156.47 | 2.454 | 0.535 | 2.459 | 406 | 3.16E-03 | 5.83E-05 | 1.80E-08 |
| 311.81 | 2.422 | 0.515 | 2.438 | 360 | 3.50E-03 | 8.10E-05 | 2.78E-08 |
| 622.16 | 2.365 | 0.480 | 2.393 | 454 | 2.68E-03 | 7.14E-05 | 1.87E-08 |
| 1243.23 | 2.298 | 0.438 | 2.332 | 470 | 2.45E-03 | 4.28E-05 | 1.03E-08 |
| 2484.88 | 2.224 | 0.392 | 2.261 | 505 | 2.15E-03 | | 4.92E-09 |
| 622.16 | 2.243 | 0.403 | 2.233 | | | | |
| 199.56 | 2.271 | 0.421 | 2.257 | | | | |
| 78.88 | 2.299 | 0.439 | 2.285 | | | | |
| 20.53 | 2.333 | 0.460 | 2.316 | | | | |
| 10.74 | 2.348 | 0.469 | 2.340 | | | | |
| Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t ₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 12-17cm from bottom of the tube. | | | | | | | |
| SAMPLE DIMENSIONS AND PROPERTIES - FINAL | | | | | | | |
| Sample Height, cm | 2.35 | Unit Weight, kN/m ³ | 21.26 | | | | |
| Sample Diameter, cm | 6.34 | Dry Unit Weight, kN/m ³ | 18.02 | | | | |
| Area, cm ² | 31.55 | Specific Gravity, measured | 2.70 | | | | |
| Volume, cm ³ | 74.07 | Solids Height, cm | 1.598 | | | | |
| Water Content, % | 17.99 | Volume of Solids, cm ³ | 50.41 | | | | |
| Wet Mass, g | 160.61 | Volume of Voids, cm ³ | 23.65 | | | | |
| Dry Mass, g | 136.12 | | | | | | |
| Prepared By: LH | | | | Golder Associates | | Checked By: MM | |



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B-4.3

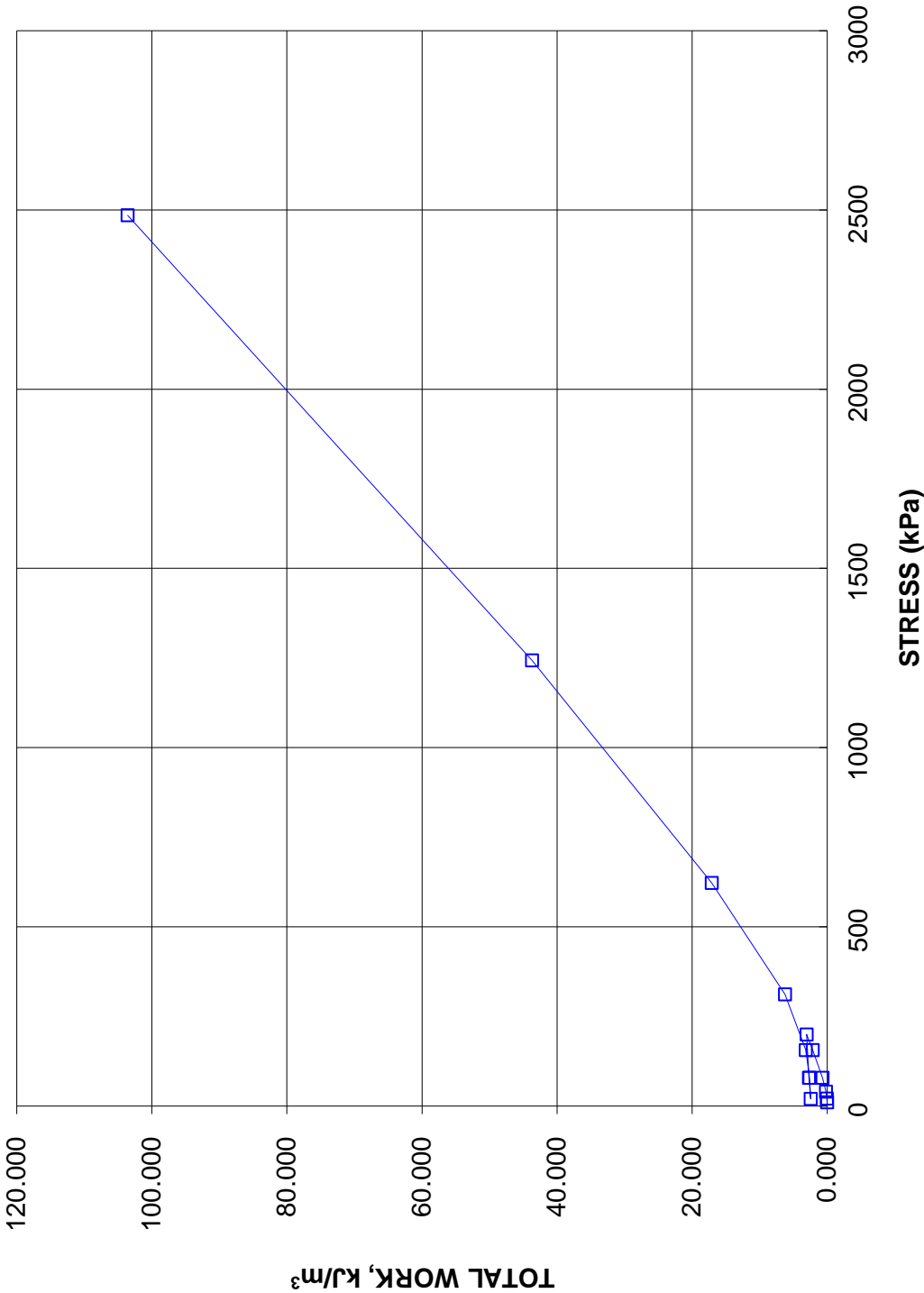
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 802 SA 14



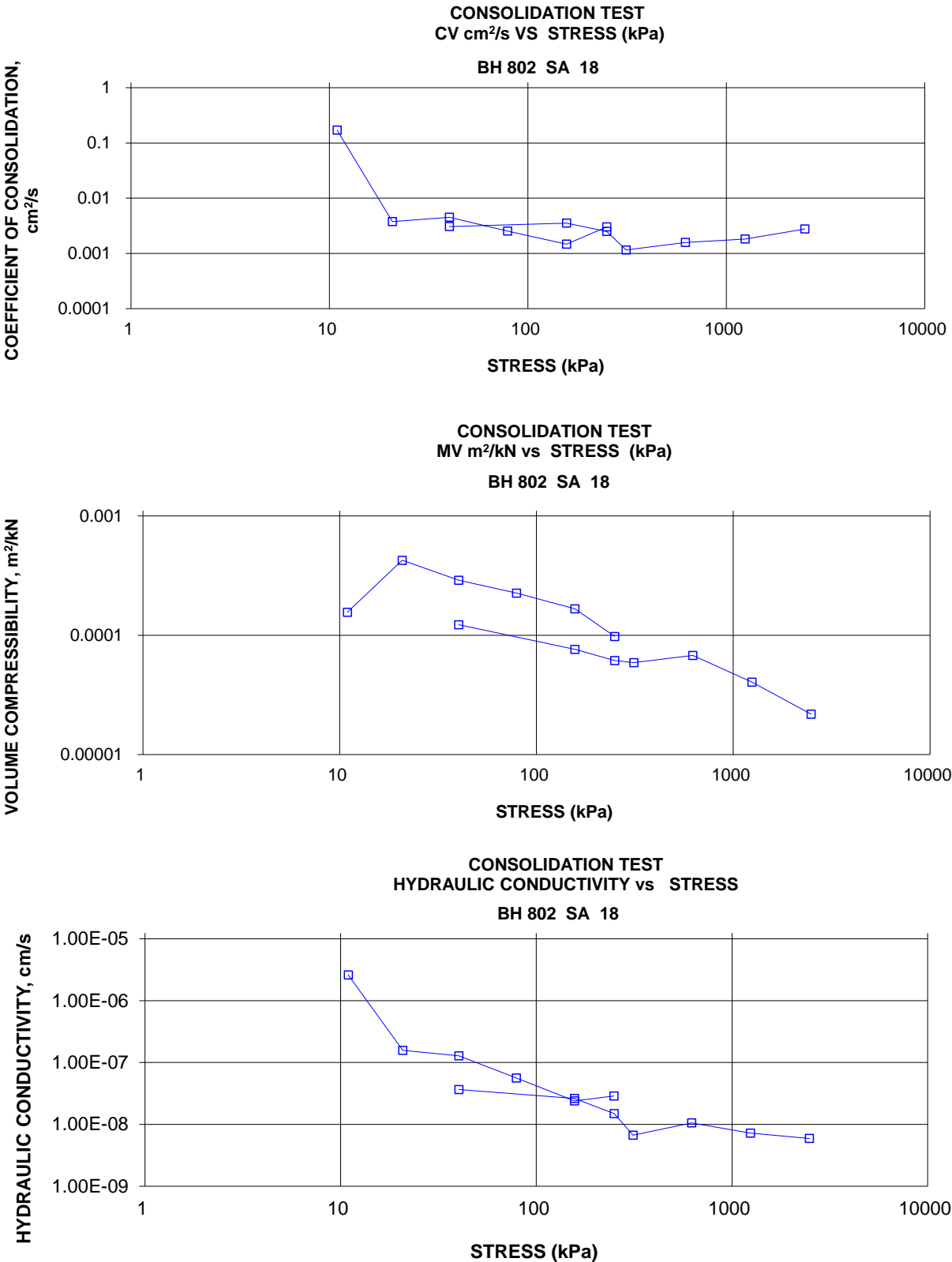
CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B-4.4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH 802 SA 14



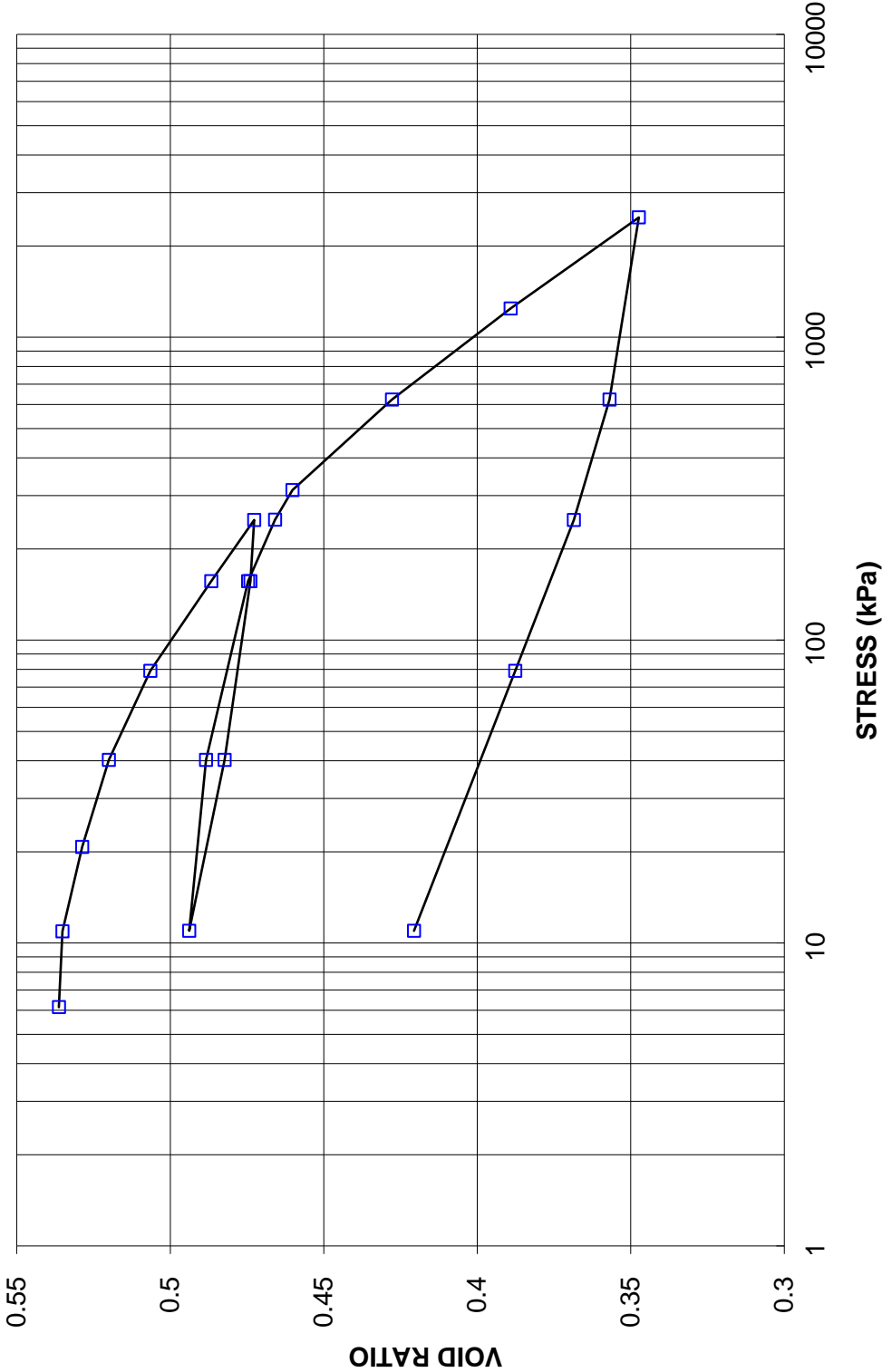
| CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M | | | | | FIGURE B-5.1 | | |
|--|--|---------------------|---------|------------------------------------|--------------------|--------------------|-------------------|
| SAMPLE IDENTIFICATION | | | | | | | |
| Project Number | | 19124560 | | Sample Number | | 18 | |
| Borehole Number | | 802 | | Sample Depth, m | | 19.81-20.42 | |
| TEST CONDITIONS | | | | | | | |
| Test Type | | Laboratory Standard | | Load Duration, hr | | 24 | |
| Oedometer Number | | 4 | | | | | |
| Date Started | | 10/02/2020 | | | | | |
| Date Completed | | 10/17/2020 | | | | | |
| SAMPLE DIMENSIONS AND PROPERTIES - INITIAL | | | | | | | |
| Sample Height, cm | | 2.54 | | Unit Weight, kN/m ³ | | 20.49 | |
| Sample Diameter, cm | | 6.34 | | Dry Unit Weight, kN/m ³ | | 17.20 | |
| Area, cm ² | | 31.53 | | Specific Gravity, measured | | 2.70 | |
| Volume, cm ³ | | 80.02 | | Solids Height, cm | | 1.649 | |
| Water Content, % | | 19.10 | | Volume of Solids, cm ³ | | 52.00 | |
| Wet Mass, g | | 167.20 | | Volume of Voids, cm ³ | | 28.03 | |
| Dry Mass, g | | 140.39 | | Degree of Saturation, % | | 95.7 | |
| TEST COMPUTATIONS | | | | | | | |
| Stress | | Corr. | Average | | | | |
| Height | | Void | Height | t ₉₀ | cv. | mv | k |
| kPa | | Ratio | cm | sec | cm ² /s | m ² /kN | cm/s |
| 0.00 | | 2.538 | 0.539 | 2.538 | | | |
| 6.14 | | 2.534 | 0.536 | 2.536 | | | |
| 10.94 | | 2.532 | 0.535 | 2.533 | 8 | 1.70E-01 | 1.56E-04 2.60E-06 |
| 20.76 | | 2.521 | 0.529 | 2.526 | 359 | 3.77E-03 | 4.25E-04 1.57E-07 |
| 40.25 | | 2.507 | 0.520 | 2.514 | 296 | 4.53E-03 | 2.89E-04 1.28E-07 |
| 79.13 | | 2.484 | 0.507 | 2.496 | 519 | 2.54E-03 | 2.26E-04 5.63E-08 |
| 156.71 | | 2.452 | 0.487 | 2.468 | 877 | 1.47E-03 | 1.67E-04 2.40E-08 |
| 249.57 | | 2.429 | 0.473 | 2.440 | 420 | 3.01E-03 | 9.76E-05 2.87E-08 |
| 156.71 | | 2.431 | 0.474 | 2.430 | | | |
| 40.25 | | 2.445 | 0.482 | 2.438 | | | |
| 10.98 | | 2.464 | 0.494 | 2.454 | | | |
| 40.25 | | 2.454 | 0.488 | 2.459 | 420 | 3.05E-03 | 1.22E-04 3.66E-08 |
| 156.71 | | 2.432 | 0.475 | 2.443 | 359 | 3.52E-03 | 7.61E-05 2.63E-08 |
| 249.77 | | 2.417 | 0.466 | 2.425 | 501 | 2.49E-03 | 6.14E-05 1.50E-08 |
| 312.65 | | 2.408 | 0.460 | 2.413 | 1069 | 1.15E-03 | 5.89E-05 6.66E-09 |
| 622.49 | | 2.355 | 0.428 | 2.381 | 759 | 1.58E-03 | 6.79E-05 1.05E-08 |
| 1243.76 | | 2.291 | 0.389 | 2.323 | 628 | 1.82E-03 | 4.05E-05 7.22E-09 |
| 2486.00 | | 2.222 | 0.347 | 2.256 | 390 | 2.77E-03 | 2.18E-05 5.92E-09 |
| 622.49 | | 2.238 | 0.357 | 2.230 | | | |
| 249.56 | | 2.257 | 0.369 | 2.247 | | | |
| 79.24 | | 2.288 | 0.388 | 2.273 | | | |
| 10.99 | | 2.343 | 0.421 | 2.316 | | | |
| Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t ₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen swelled under 6.14kPa Specimen taken 13-19cm from bottom of the tube. | | | | | | | |
| SAMPLE DIMENSIONS AND PROPERTIES - FINAL | | | | | | | |
| Sample Height, cm | | 2.34 | | Unit Weight, kN/m ³ | | 21.72 | |
| Sample Diameter, cm | | 6.34 | | Dry Unit Weight, kN/m ³ | | 18.64 | |
| Area, cm ² | | 31.53 | | Specific Gravity, measured | | 2.70 | |
| Volume, cm ³ | | 73.87 | | Solids Height, cm | | 1.649 | |
| Water Content, % | | 16.51 | | Volume of Solids, cm ³ | | 52.00 | |
| Wet Mass, g | | 163.57 | | Volume of Voids, cm ³ | | 21.87 | |
| Dry Mass, g | | 140.39 | | | | | |
| Prepared By: LH | | | | Golder Associates | | Checked By: MM | |



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B-5.3

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 802 SA 18

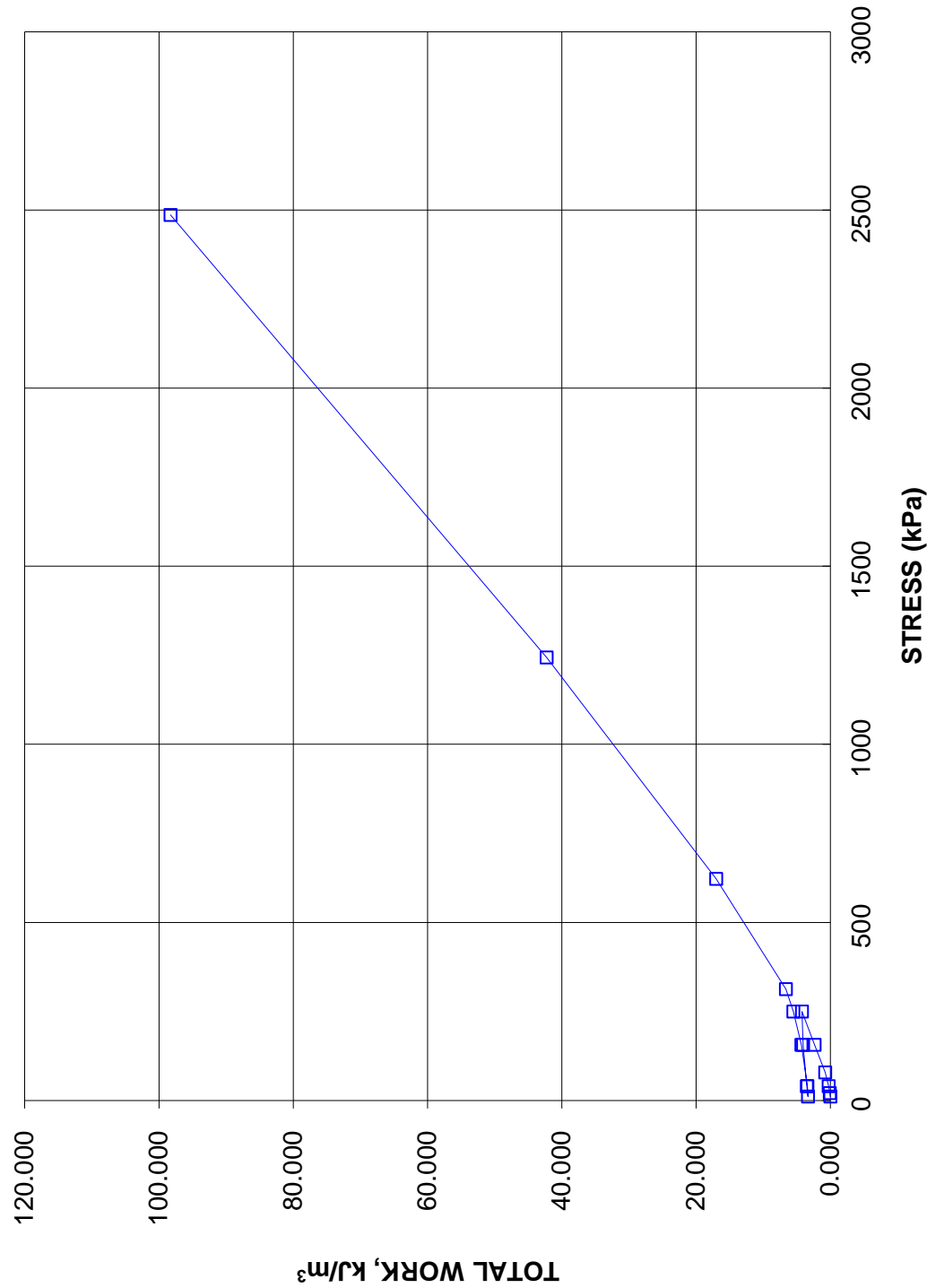


**CONSOLIDATION TEST
TOTAL WORK VS STRESS**

FIGURE B-5.4

**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS**

BH 802 SA 18



APPENDIX C

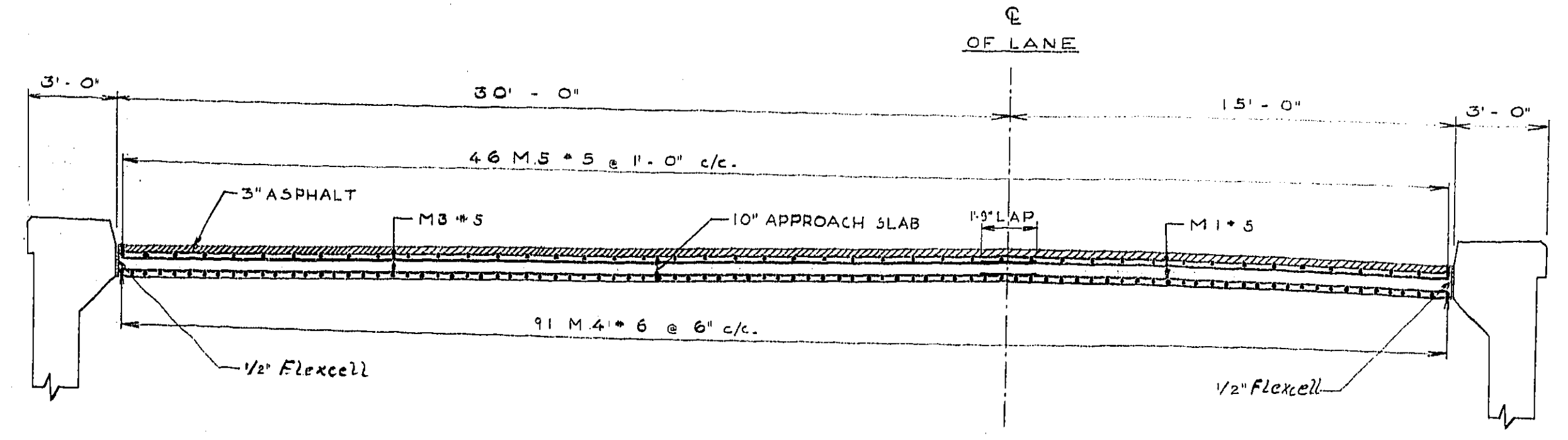
1959 Design Drawings

DETAIL OF WALL
SAME AS SOUTH
WALL

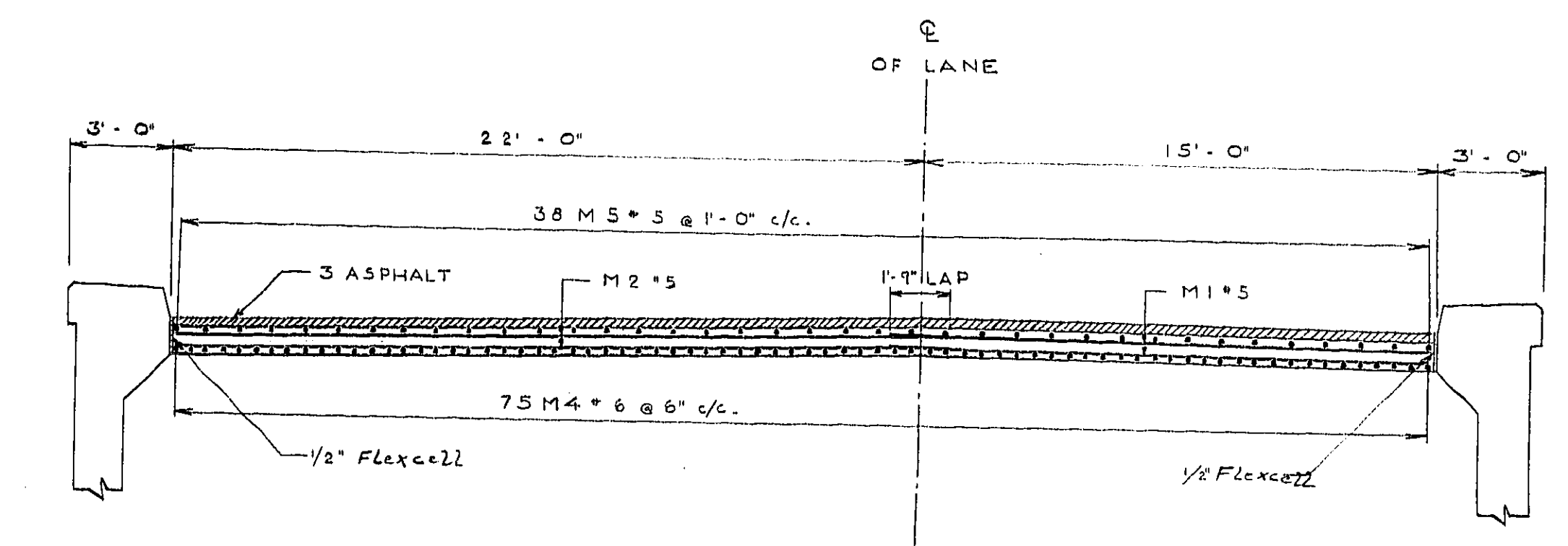
N.W.
WEST BOND LANES

N.E.

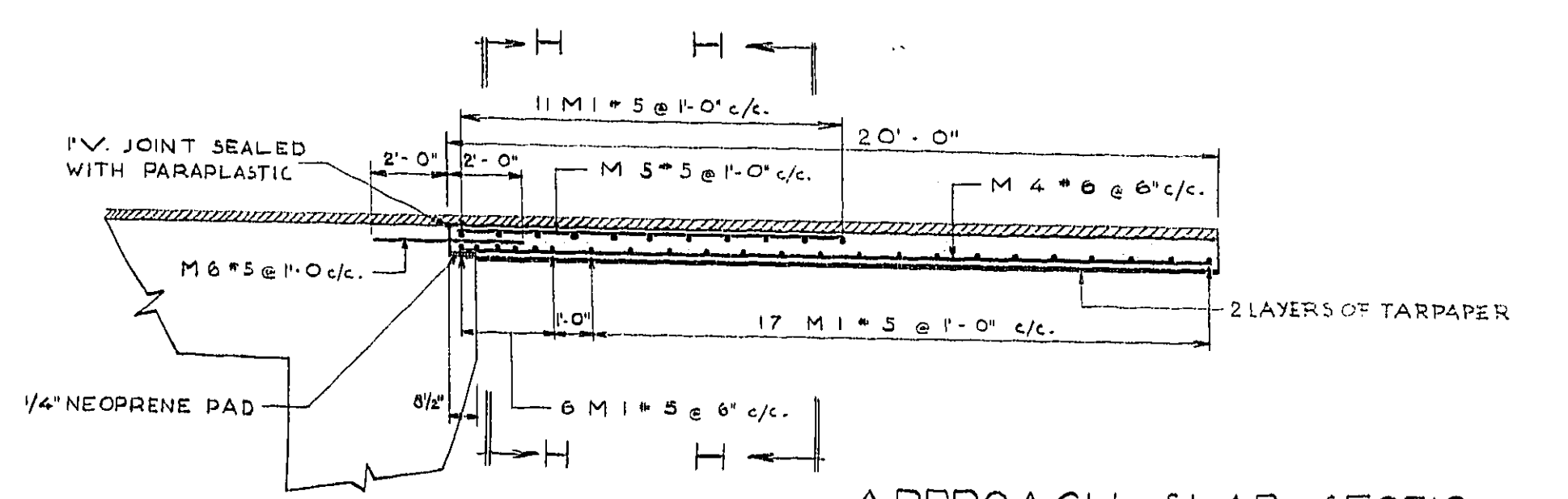
FOR CONSTRUCTION



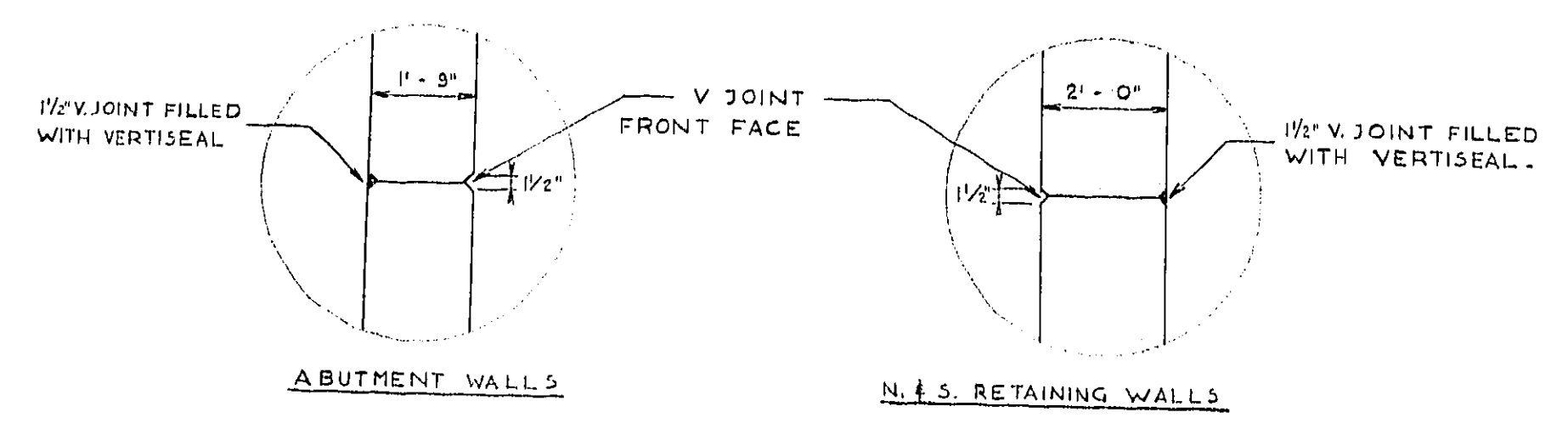
SECTION H-H (N. BRIDGE)



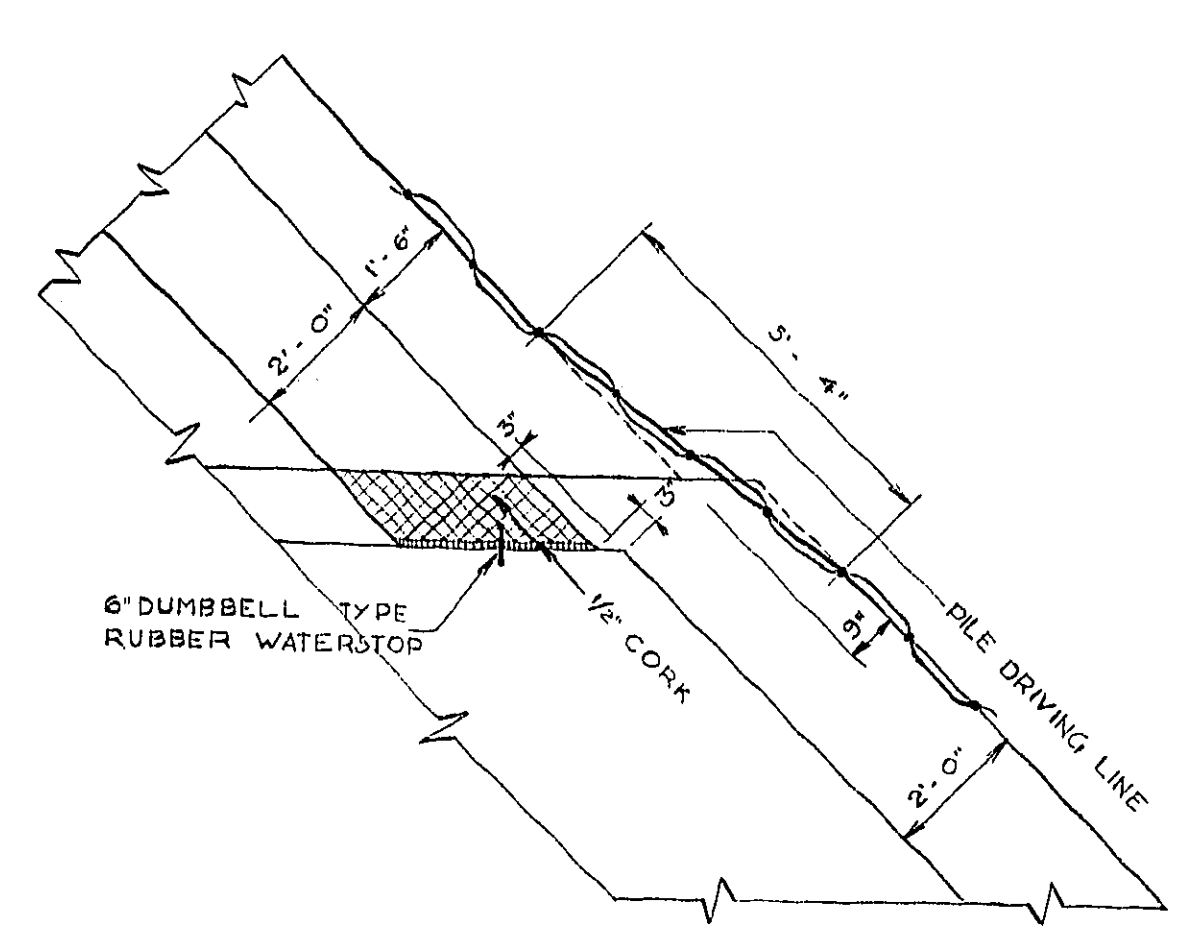
SECTION H-H (S. BRIDGE)



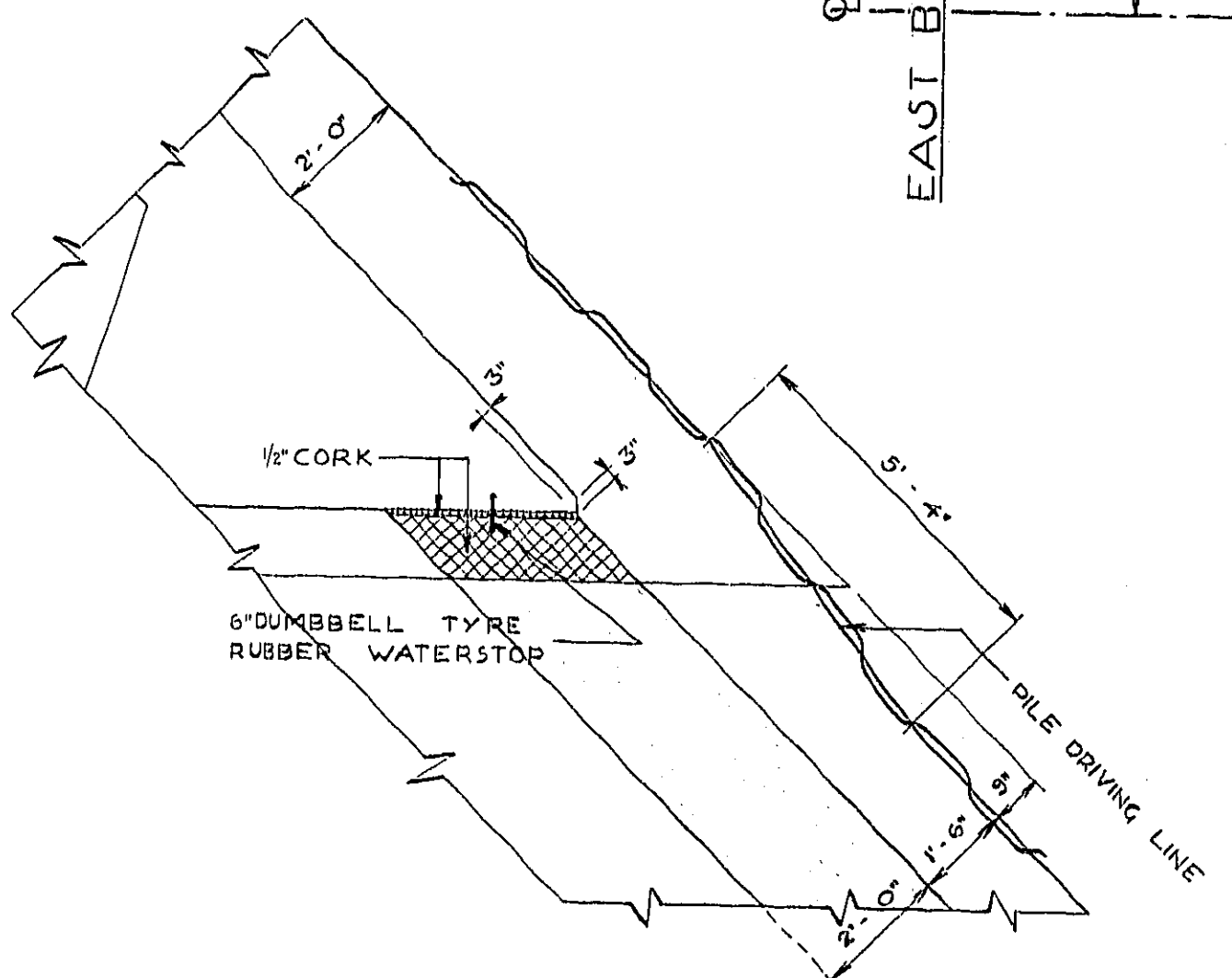
APPROACH SLAB SECTION
SCALE 1/2" = 1'-0"



DETAIL OF CONST. JOINT
SCALE 3/4" = 1'-0"



DETAIL 'A'



DETAIL 'B'

SHEET PILES:
ALGOMA FA-6 STEEL PILES
337 - 18'-0" LONG
CUT OFF ELEVATION 578.00

FOOTING PLAN
SCALE 1/2" = 1'-0"

| NO. | FOR | DATE |
|-----|-----|------|
| 1 | FOR | DATE |
| 2 | FOR | DATE |
| 3 | FOR | DATE |
| 4 | FOR | DATE |
| 5 | FOR | DATE |
| 6 | FOR | DATE |
| 7 | FOR | DATE |
| 8 | FOR | DATE |
| 9 | FOR | DATE |
| 10 | FOR | DATE |

W.P. 14-59
DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

RALEIGH T.W.P.
BRIDGE N° 6

THE KING'S HIGHWAY No. 401 DIST. No. 1
CO. KENT
TWP. RALEIGH LOT 18 CON. VII

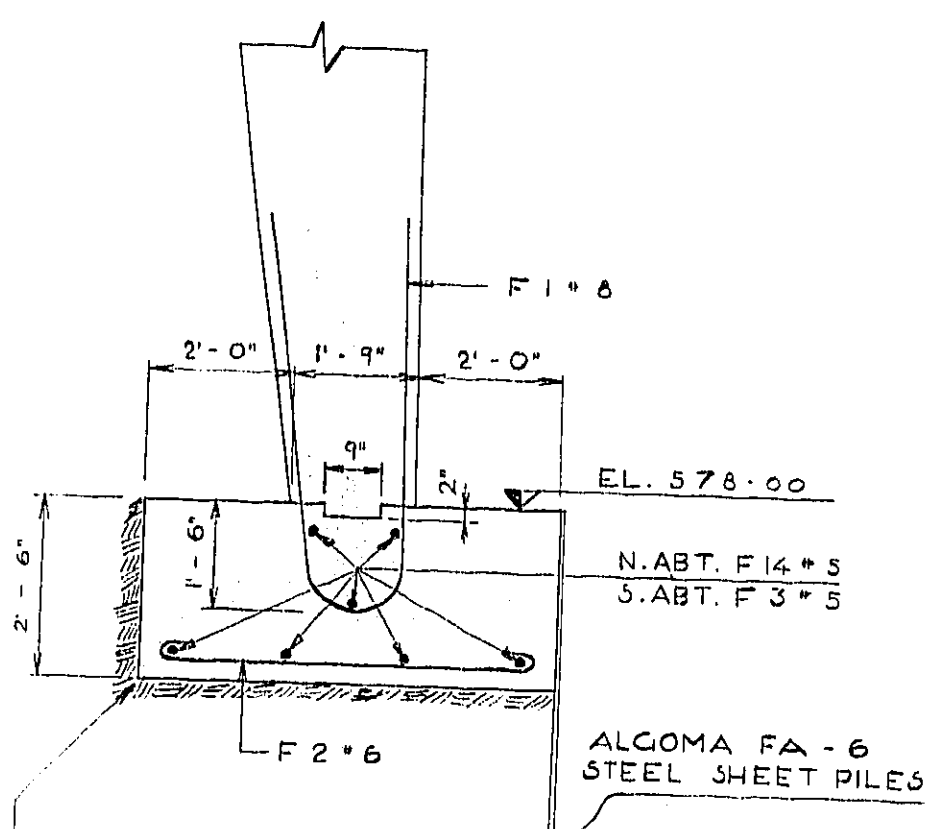
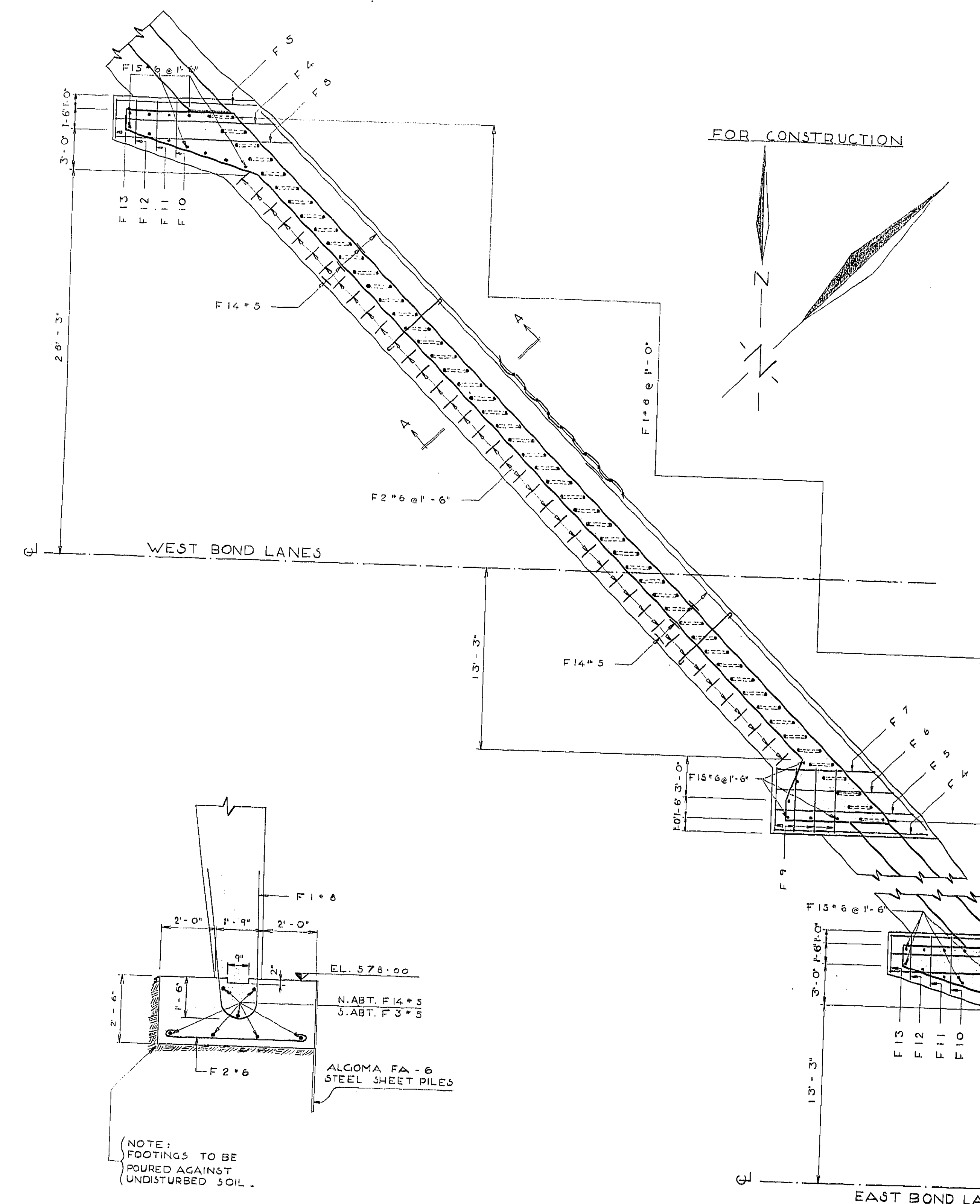
FOOTING PLAN & APPROACH SLAB

APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER

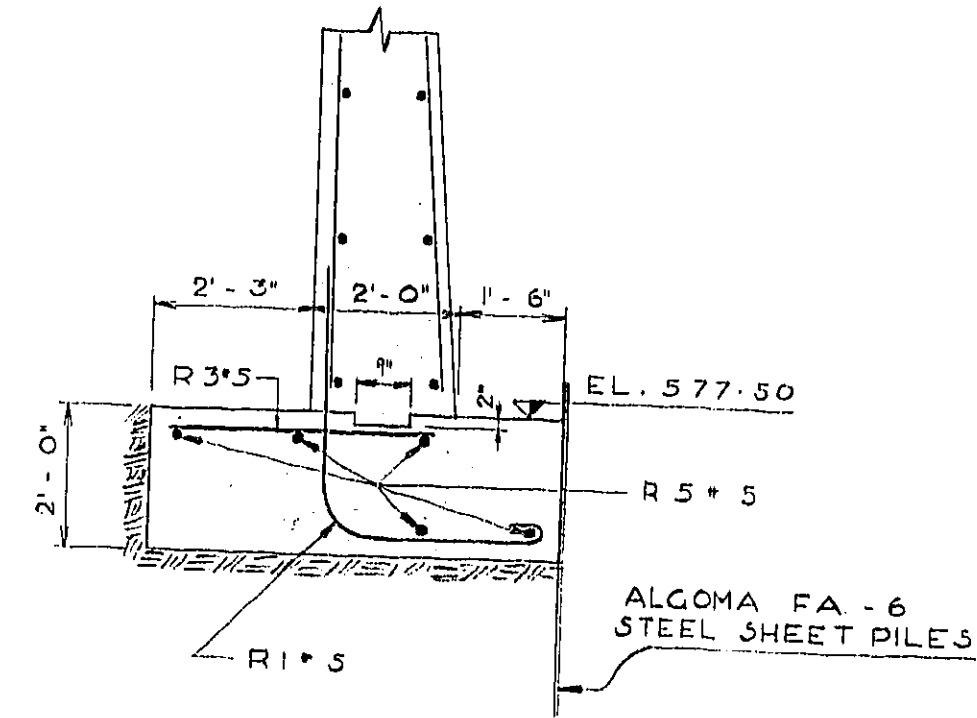
61299
DATE NOVEMBER 1959

CONTRACT NO. 67-243
LOADING NUMBER 420316
D 4376-2

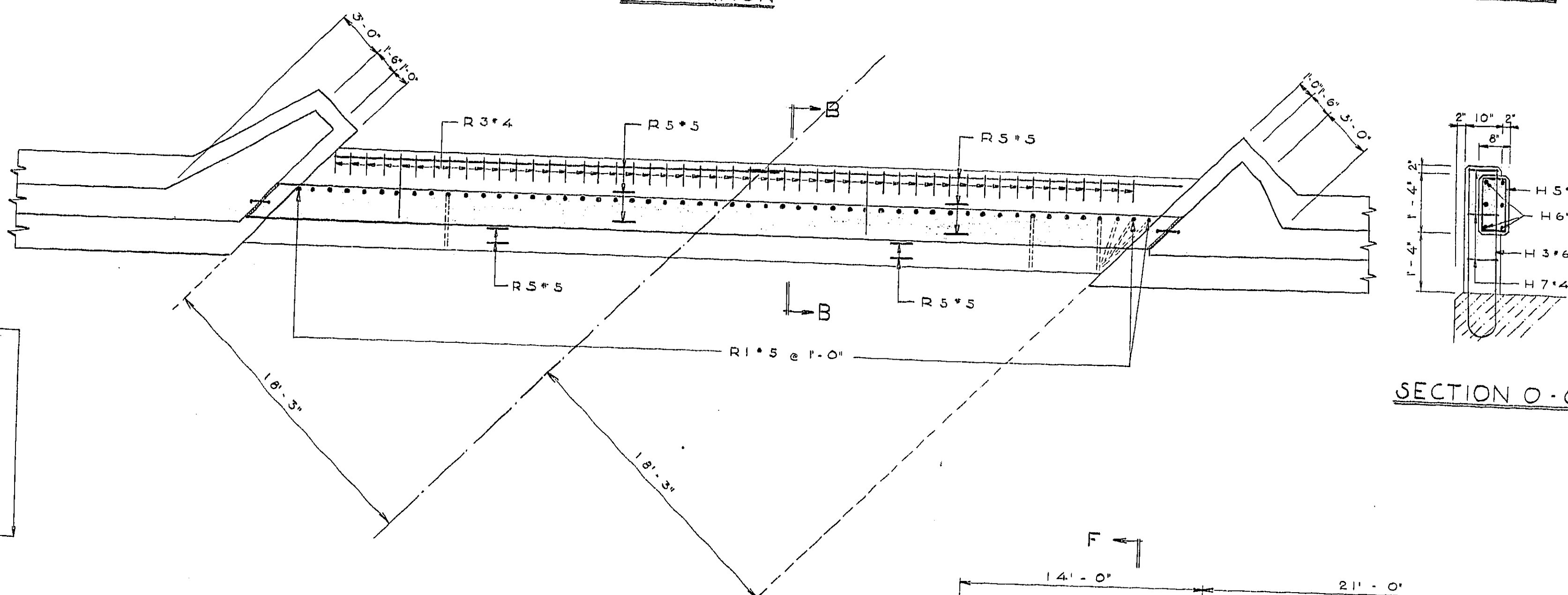
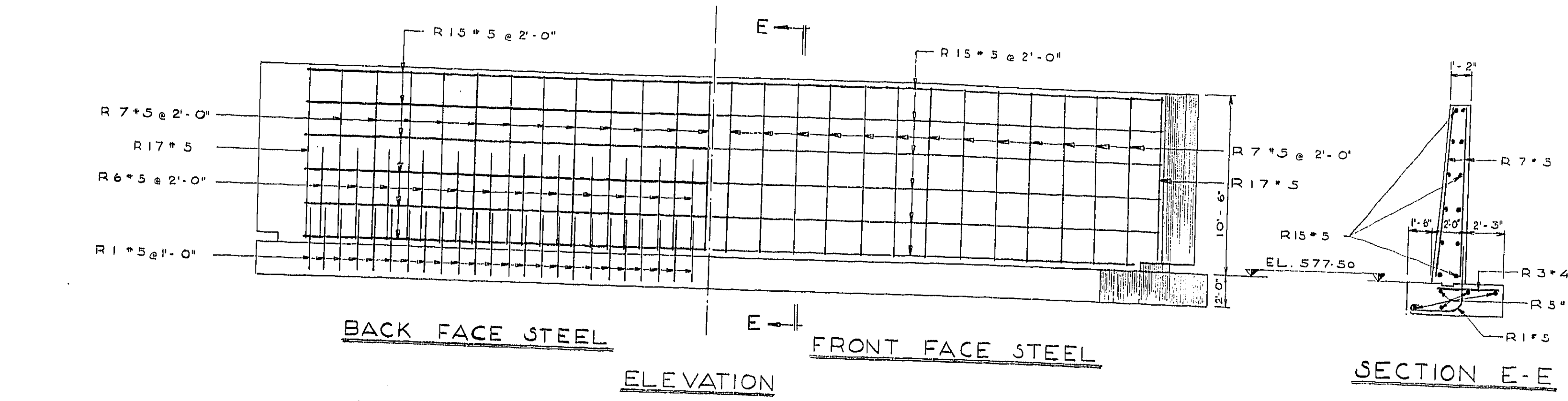
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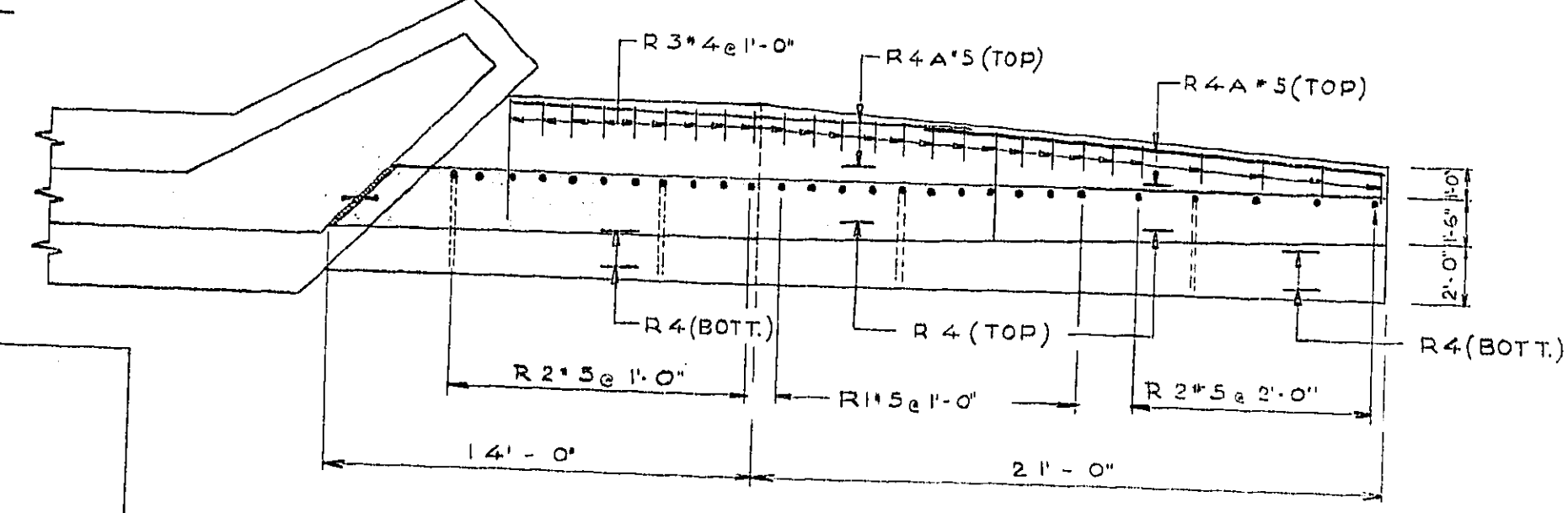
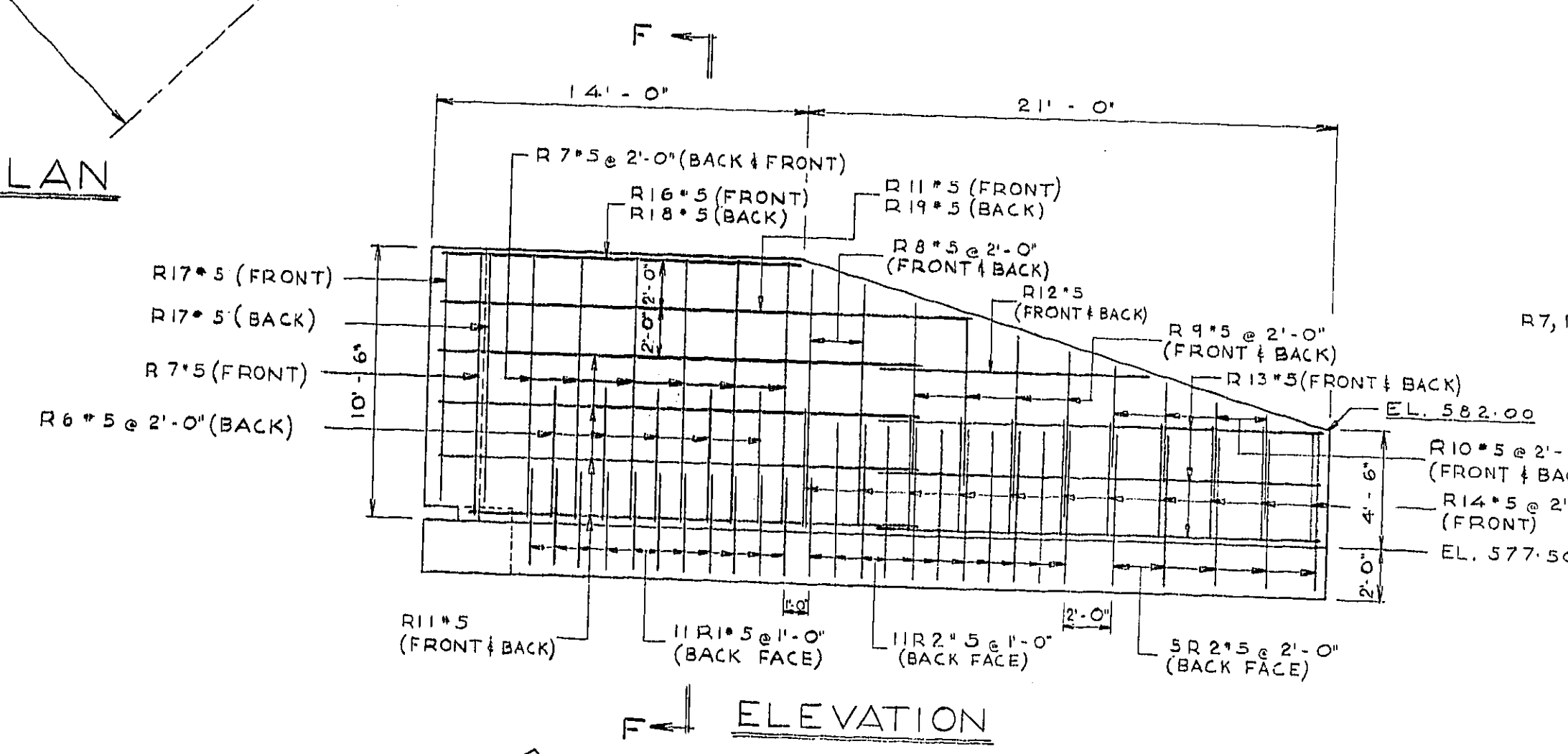
SECTION A-A
SCALE 3/8" = 1'-0"



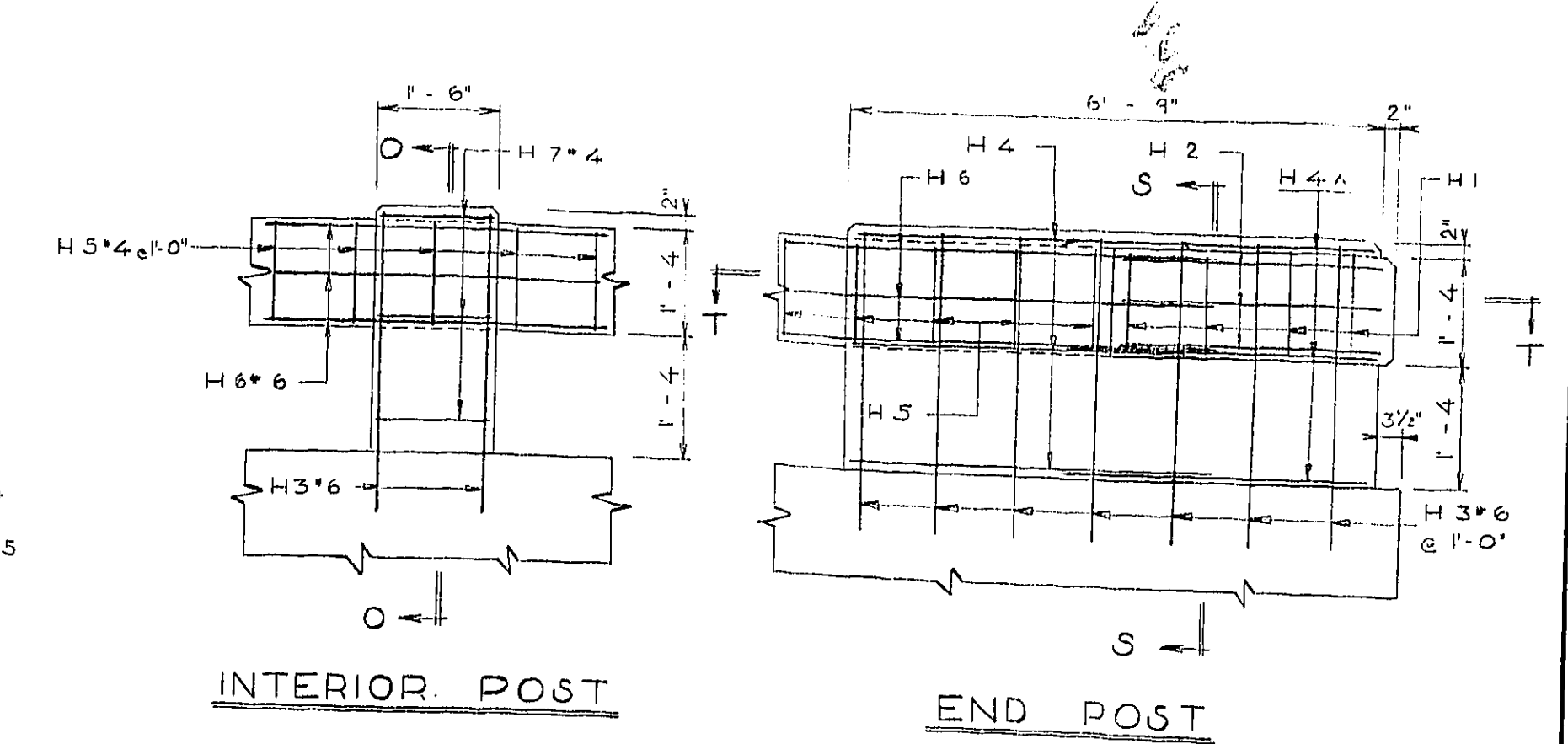
SECTION B-B
SCALE 3/8" = 1'-0"



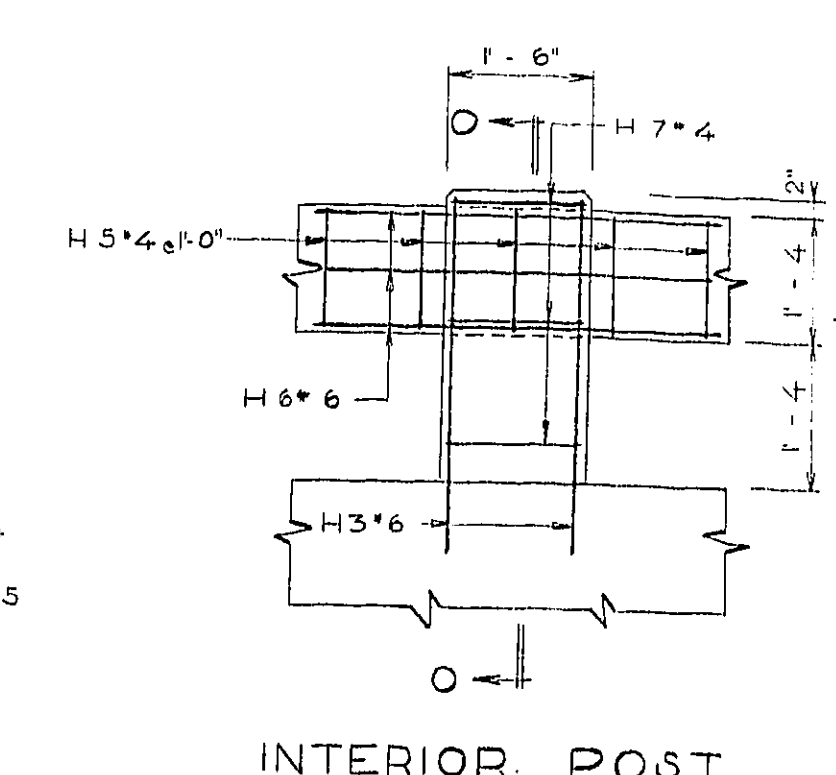
RETAINING WALL PLAN
SCALE 3/8" = 1'-0"



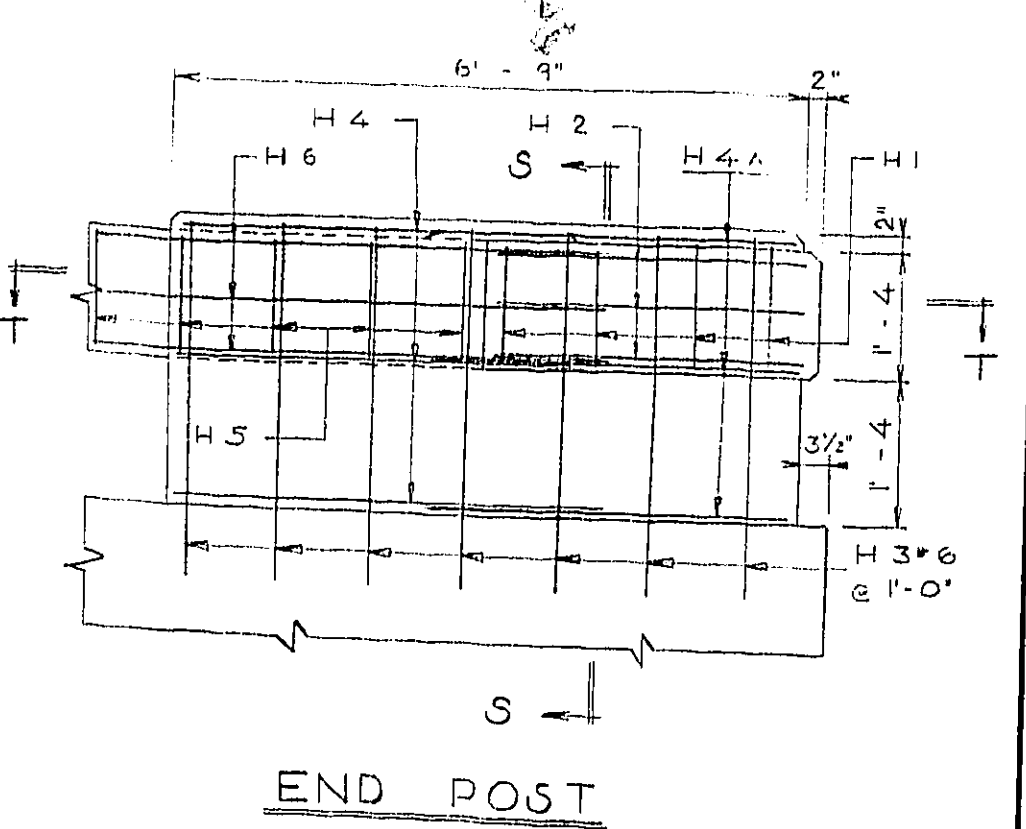
END RETAINING WALL
SCALE 3/8" = 1'-0"



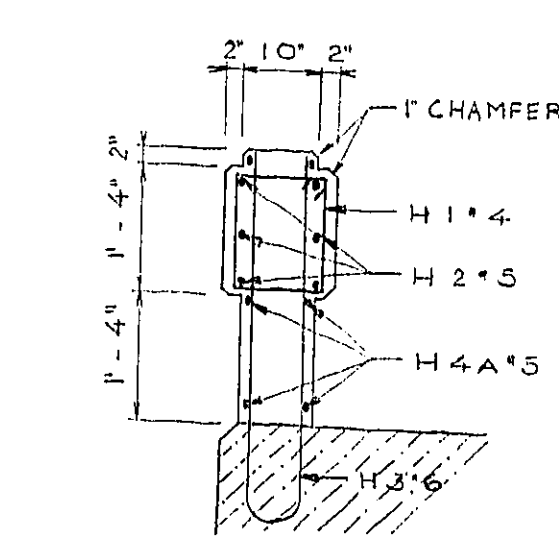
SECTION E-E



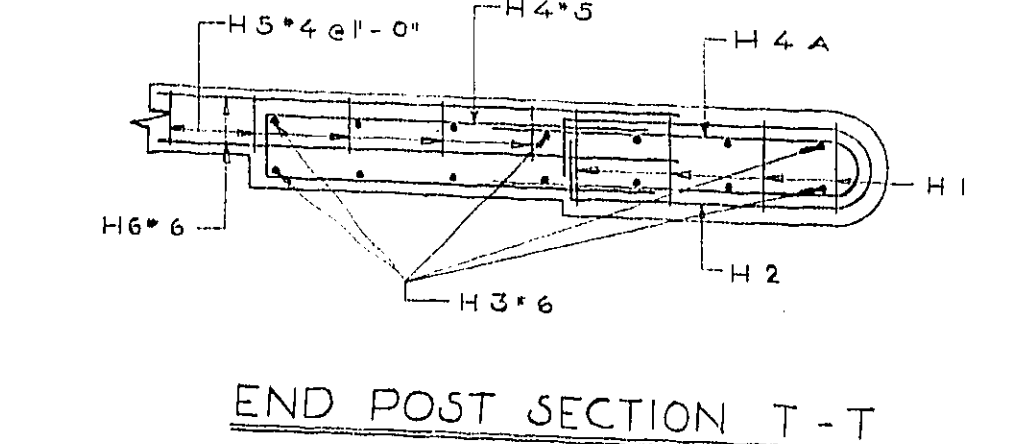
INTERIOR POST



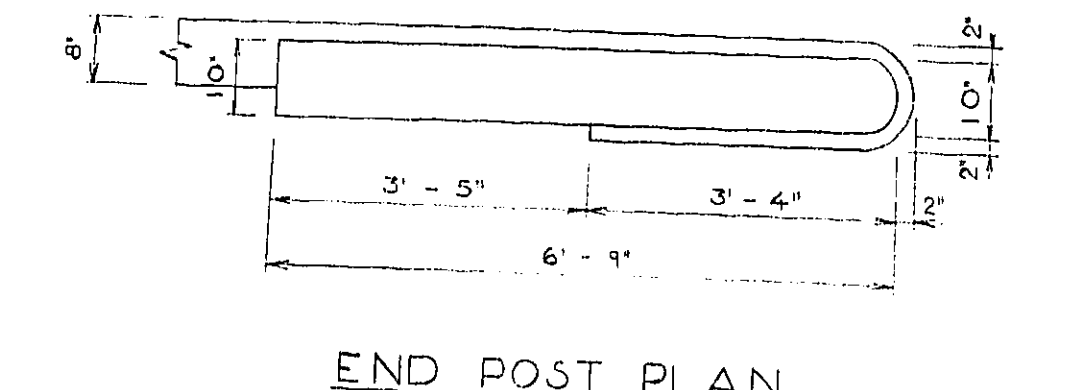
END POST



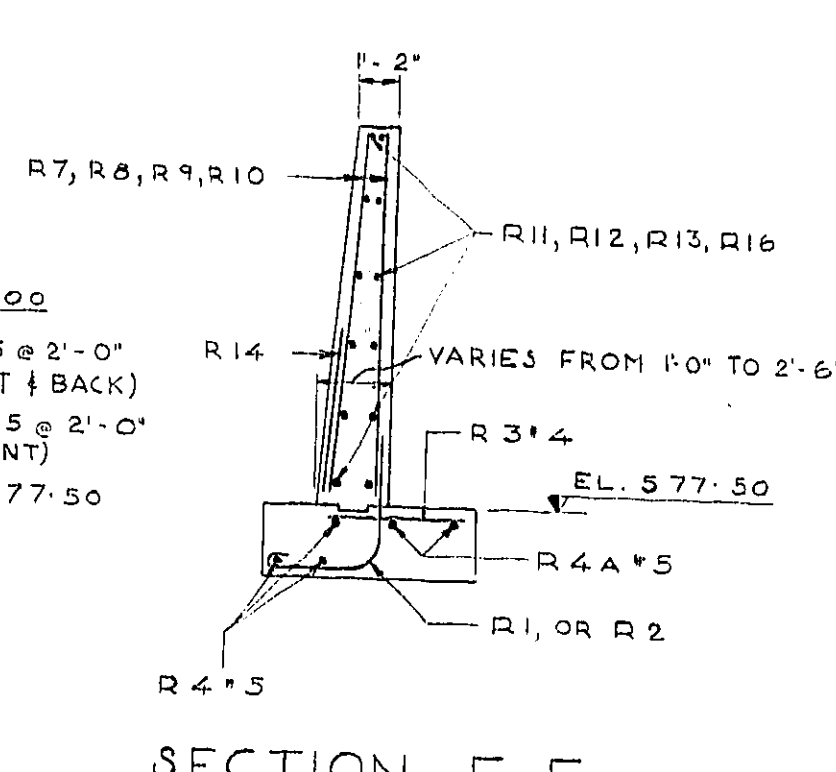
SECTION S-S



END POST SECTION T-T



END POST PLAN



SECTION F-F

| PRINT RECORD | | |
|--------------|------|---------|
| NO. | REV. | DATE |
| 10 | 1 | 12/1/59 |
| 11 | 1 | 12/1/59 |
| 12 | 1 | 12/1/59 |
| 13 | 1 | 12/1/59 |
| 14 | 1 | 12/1/59 |
| 15 | 1 | 12/1/59 |
| 16 | 1 | 12/1/59 |
| 17 | 1 | 12/1/59 |
| 18 | 1 | 12/1/59 |
| 19 | 1 | 12/1/59 |
| 20 | 1 | 12/1/59 |

W.P. 14-59
DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

RALEIGH TWP.
BRIDGE N° 6

THE KING'S HIGHWAY No. 401
CO. KENT
TWP. RALEIGH
LOT 16
CON.

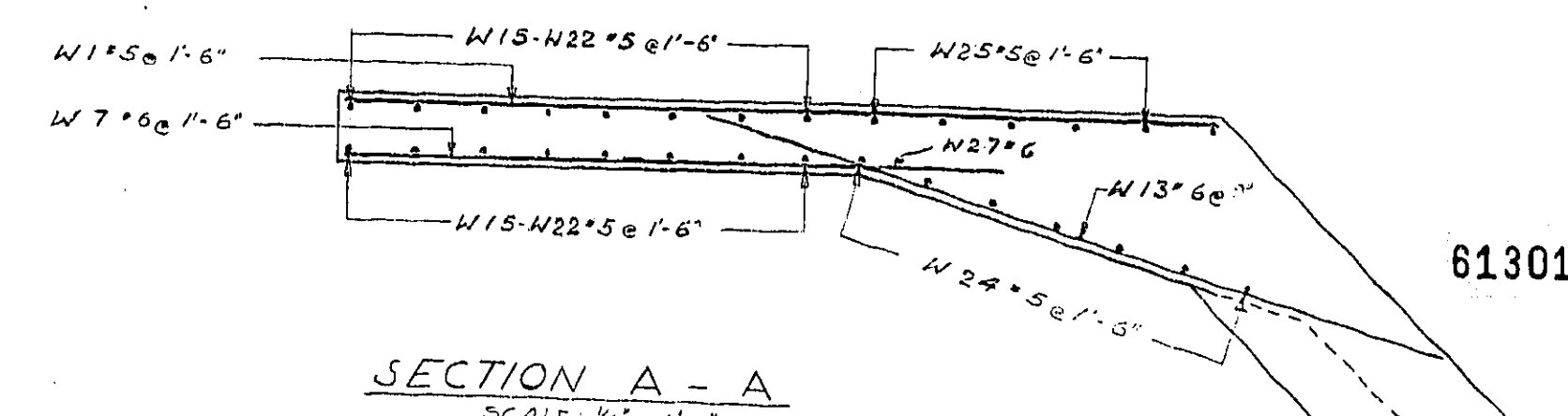
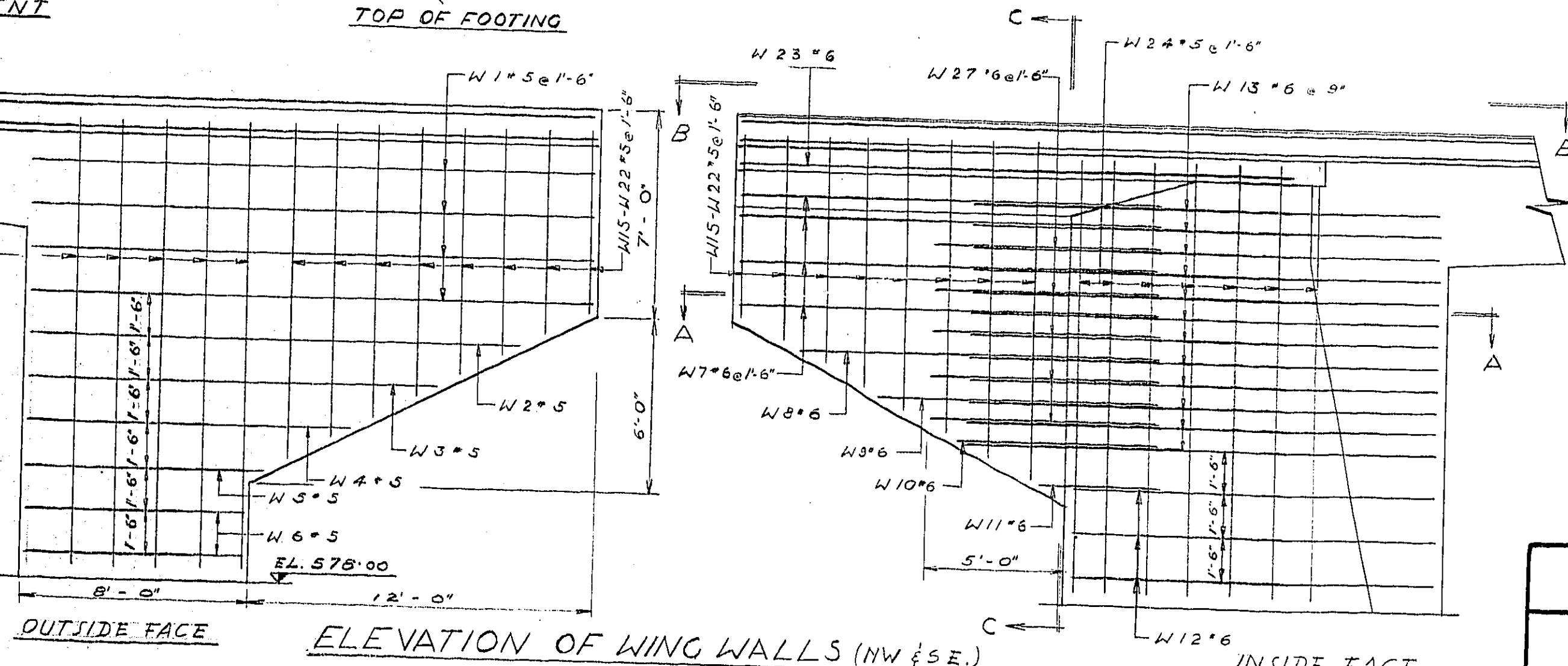
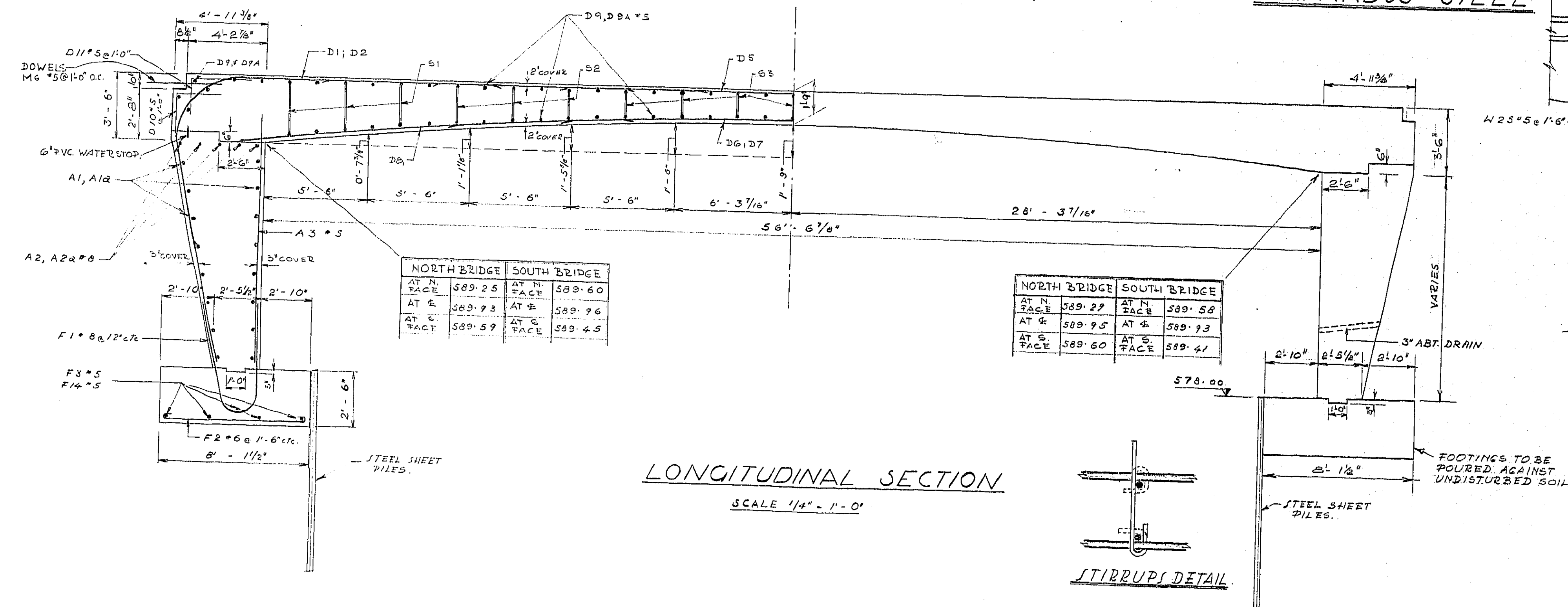
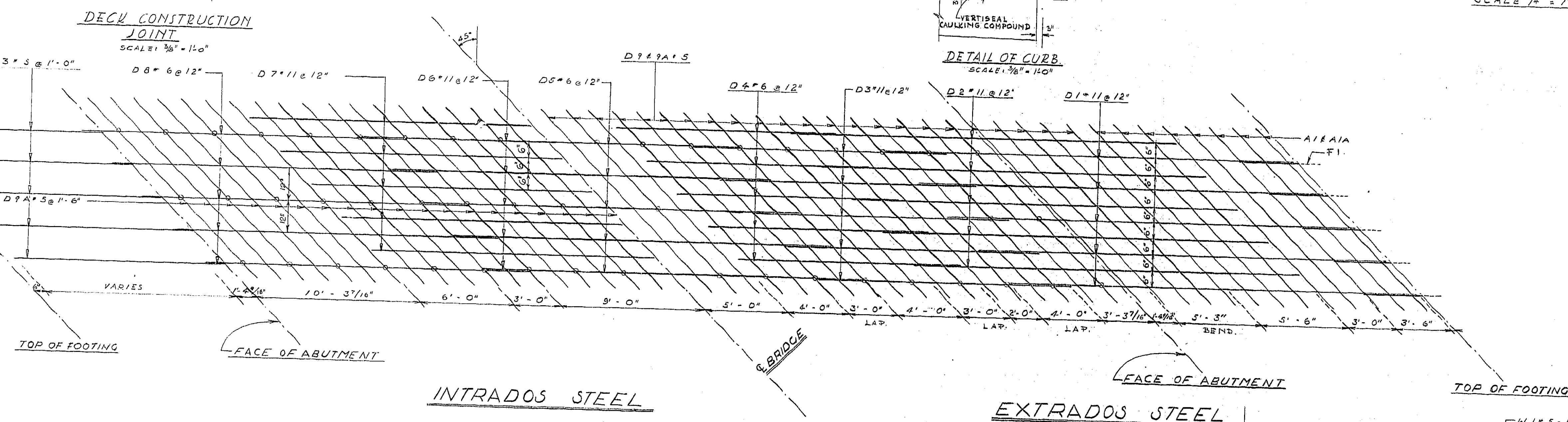
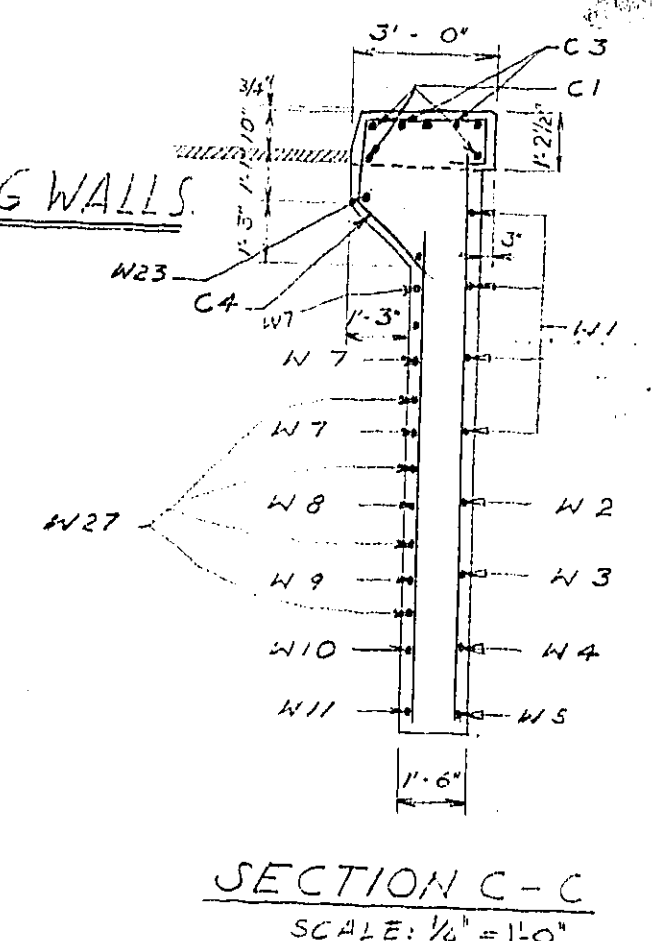
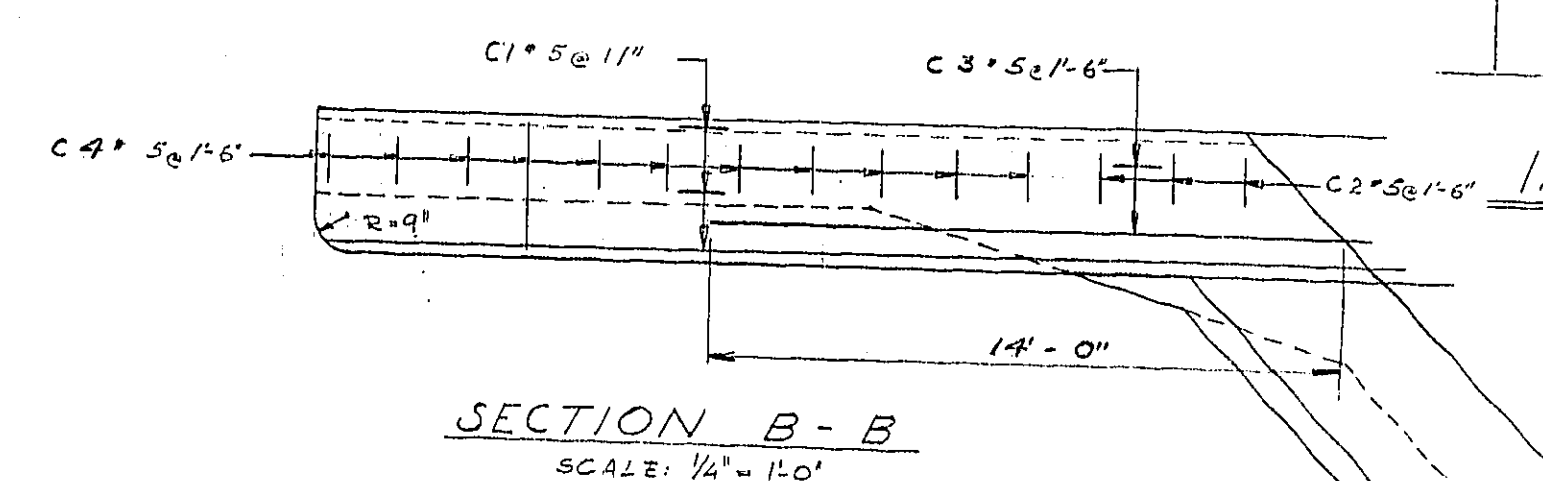
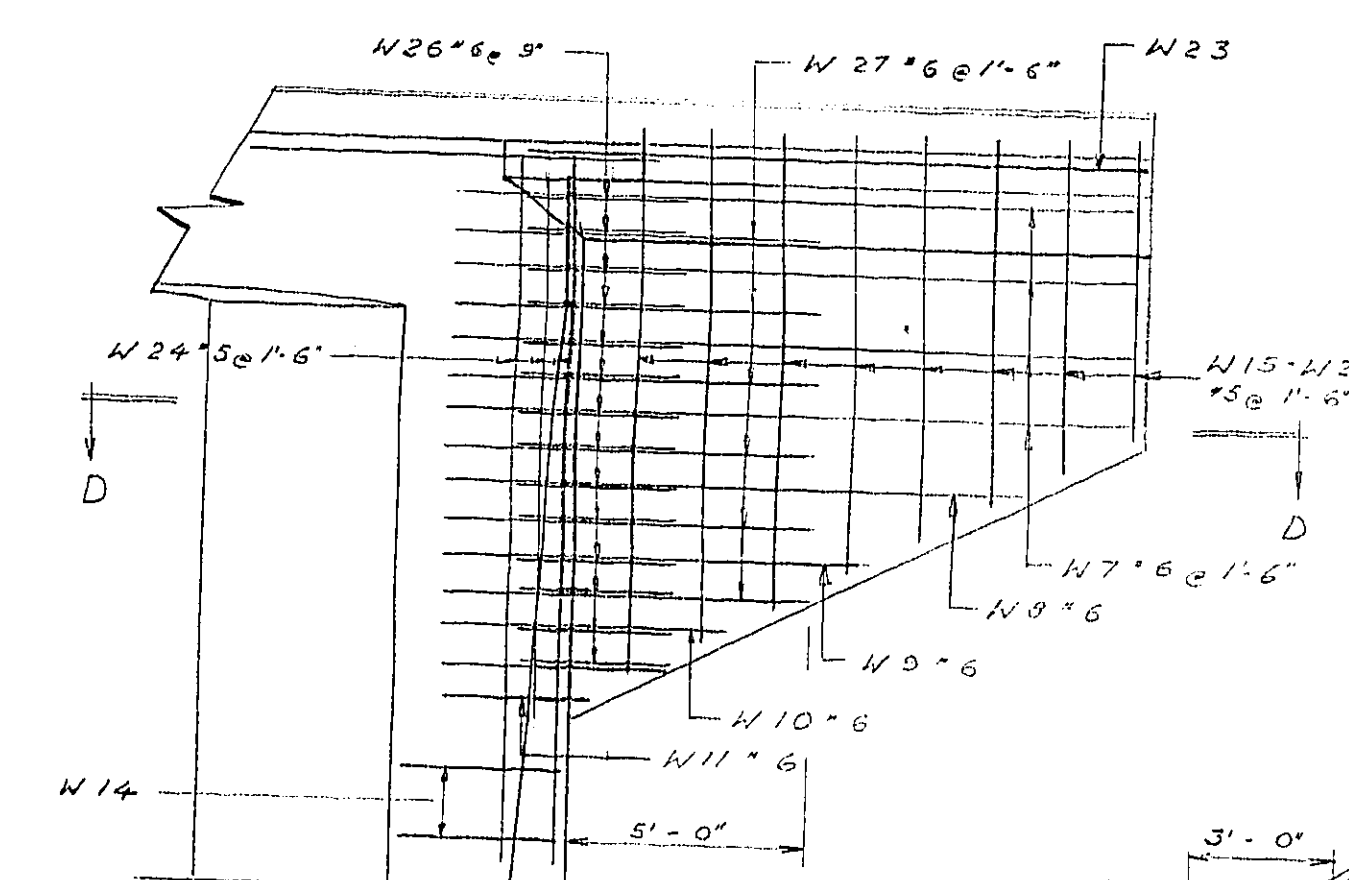
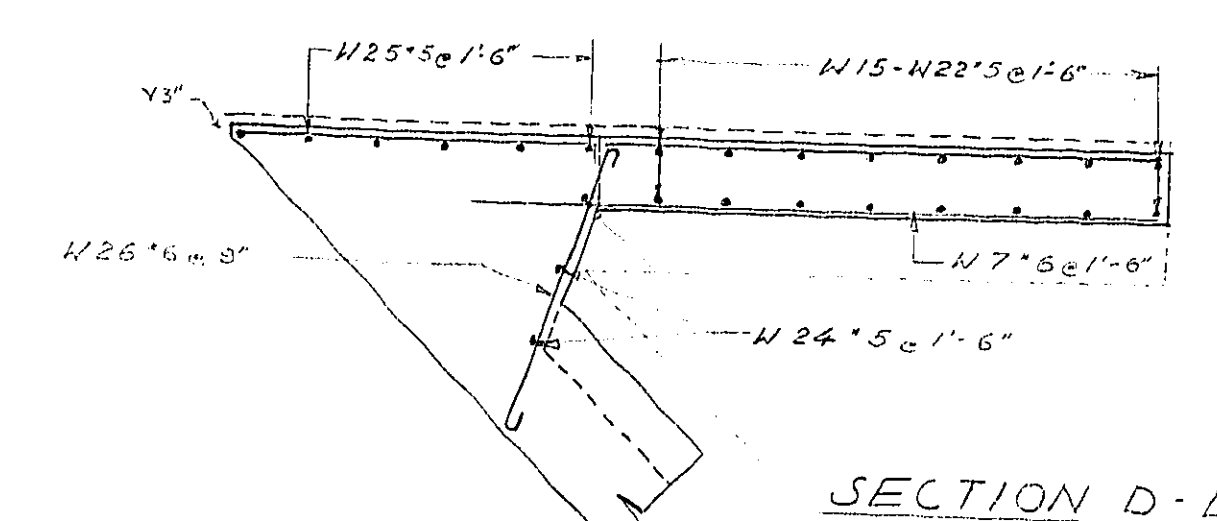
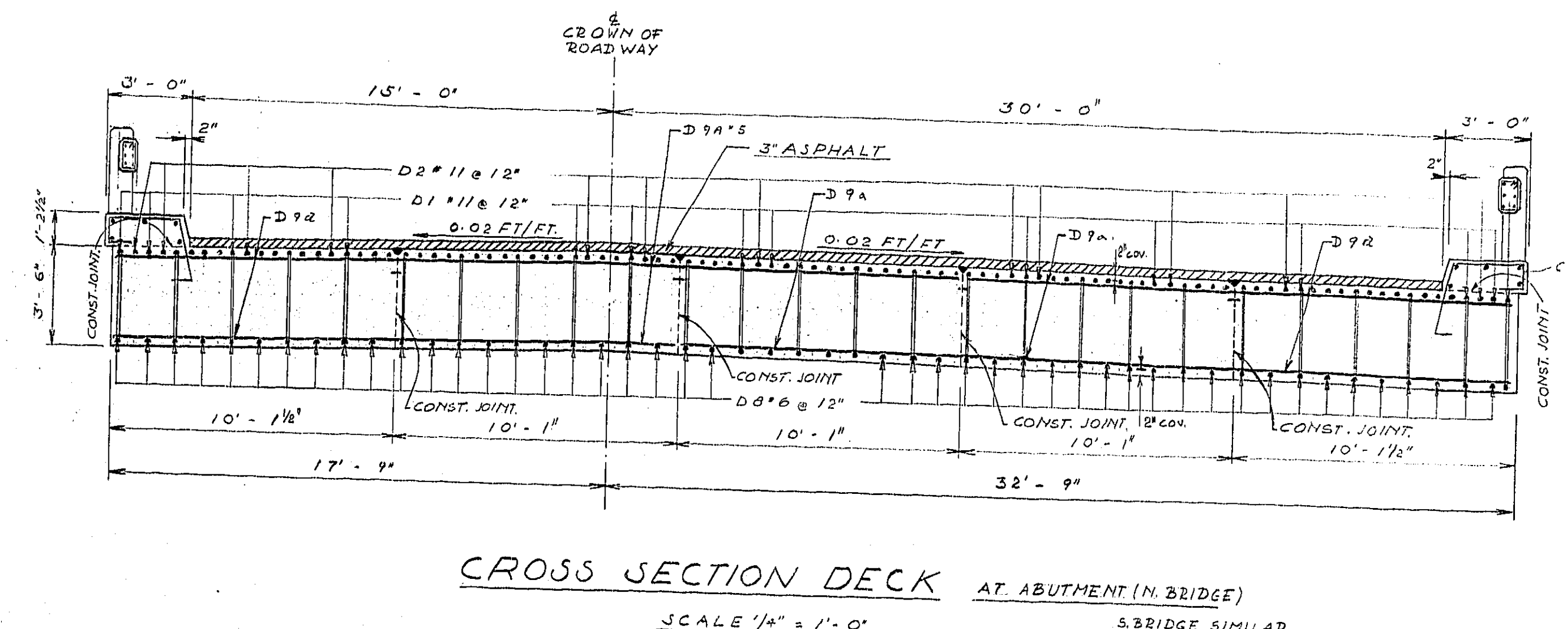
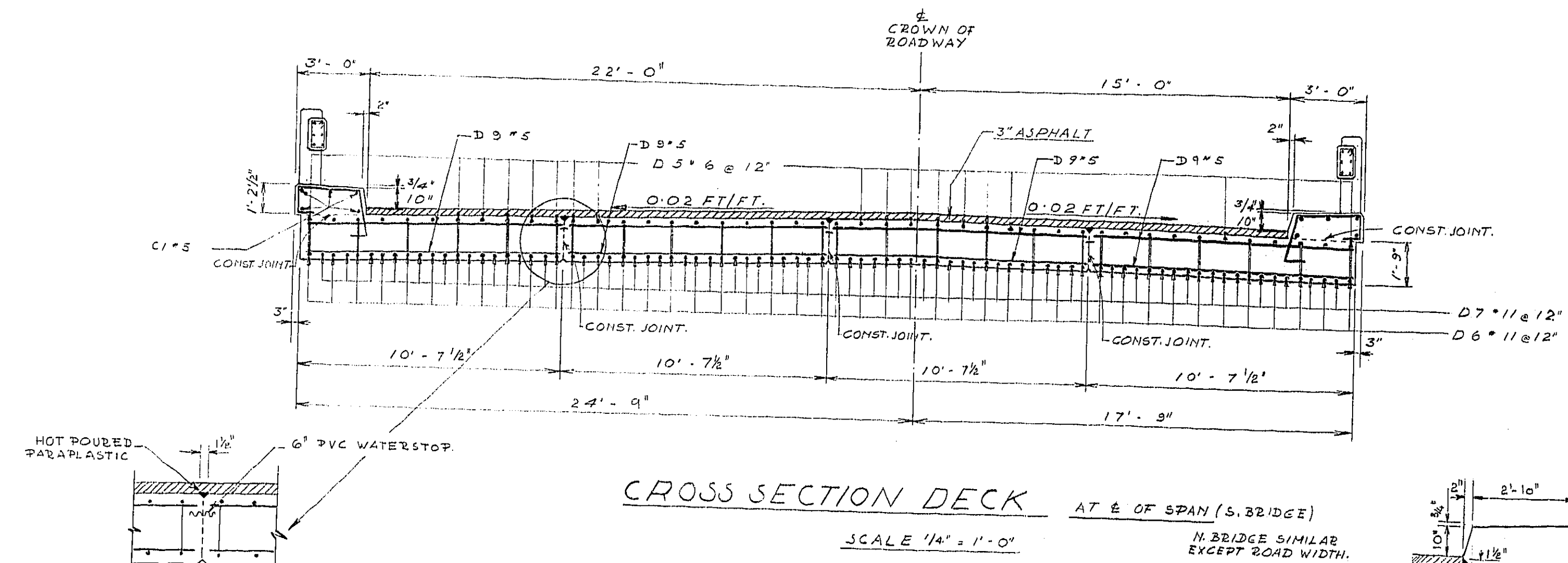
FOOTINGS, RETAINING WALLS, END
INTERM. POSTS & HANDRAILS REINFORC
APPROVED

61300

DESIGN W.L.L. CHECK M.G. CONTRACT 61-55
DRAWING V.D.P. CHECK M.G.
TRACING CHECK M.G.
DATE NOVEMBER 1959
DRAWING NUMBER H20-516

TWP# 103-229-3-A

| REVISIONS | DATE | BY | DESCRIPTION |
|-----------|------|----|-------------|
| | | | |
| | | | |
| | | | |
| | | | |

[illegible]

| NORTH BRIDGE | | SOUTH BRIDGE | |
|--------------|--------|--------------|--------|
| AT N. FACE | 589.25 | AT N. FACE | 589.60 |
| AT E | 589.73 | AT E | 589.96 |
| AT S. FACE | 589.59 | AT S. FACE | 589.45 |

| NORTH BRIDGE | | SOUTH BRIDGE | |
|--------------|--------|--------------|--------|
| AT N. FACE | 589.29 | AT N. FACE | 589.58 |
| AT 2 | 589.95 | AT 4 | 589.93 |
| AT 6 | 589.60 | AT 8 | 589.11 |

[illegible]

W.P. 14-59.

DEPARTMENT OF HIGHWAYS: ONTARIO.
BRIDGE OFFICE: TORONTO

RALEIGH TWP.
BRIDGE N° 6

THE KING'S HIGHWAY No. 401 _____ DIST. No. 1
CO. KENT
TWP. RALEIGH LOT 18 CON. III

RICID FRAME AND WING WALLS

APPROVED
John L.

BRIDGE ENGINEER _____ DESIGN ENGINEER _____

| | | | | | | |
|---------|----------------------|-------|-------------|--------------|----------------|--------------|
| DESIGN | <u>K.L.L.</u> | CHECK | <u>M.G.</u> | CONTRACT NO. | <u>61-55</u> | REF. DRAWING |
| DRAWING | <u>K.D.D.</u> | CHECK | <u>M.G.</u> | NO. | | |
| DATE | | CHECK | | LOADING | | |
| TRACED | <u>NOVEMBER 1959</u> | | | NO. 0-516 | CHECKED NUMBER | <u>61-36</u> |



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