



THURBER ENGINEERING LTD.

**FOUNDATION DESKTOP STUDY REPORT
PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT
HIGHWAY 403 AT CNR SUBWAY
STRUCTURE REPLACEMENT ON NEW ALIGNMENT
HAMILTON, ONTARIO
W.O. #16-20004
SITE 36-31**

GEOCRES NO. 30M5-341

**Latitude: 43.288220°
Longitude: -79.897099°**

Report

to

WSP

Date: November 10, 2022
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PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT
HIGHWAY 403 AT CNR SUBWAY
STRUCTURE REPLACEMENT ON NEW ALIGNMENT
HIGHWAY 403 AND HIGHWAY 6 INTERCHANGE
HAMILTON, ONTARIO
W.O. #16-20004
SITE 36-31**

GEOCRES NO. 30M5-341

1.0 INTRODUCTION

This report presents the results of a foundation desktop study carried out by Thurber Engineering Ltd. (Thurber) for the preliminary design and environmental assessment for the replacement of the Highway 403 at CNR subway structure in Hamilton, Ontario.

This Phase 1 study is carried out for planning, structure evaluation and preliminary design purposes only. As part of the Phase 1 scope, a desktop study is to be carried out based on currently available subsurface and foundation information. Where this study determines that the existing information is insufficient to complete the preliminary design, additional foundation investigation and assessment will be recommended for completing Phase 1. It is understood that the budget for this additional investigation is to be drawn from the Phase 2 contingency upon approval by MTO.

Thurber was retained by AECOM to carry out this Phase 1 study under the Ministry of Transportation Ontario (MTO) Assignment Number 2016-E-0027. The CNR Subway structure replacement is to be designed by WSP to whom this report is addressed.

This site is a part of the overall Highway 403 and Highway 6 Interchange Improvements project where 14 bridges, 3 structural culverts and 15 retaining walls are planned to be replaced, reconstructed or rehabilitated.

It is a condition of this report that Thurber's performance of its professional services be subject to the attached Statement of Limitations and Conditions.

The following references and drawings are available in the general vicinity of this site.

- Department of Highways Ontario, Foundation Investigation Report, Chedoke Expressway at CNR and CPR Tracks, District 4, prepared by e.m. peto associated ltd., dated January 25, 1960, 60-F-308C, Geocres No. 30M05-013 (Reference 1).
- Department of Highways Ontario, Foundation Investigation Report, Chedoke Expressway at CNR and CPR Overhead, District 4, W.P. 99-60 and W.P. 100-60, dated June 9, 1960, 60-F-35C, Geocres No. 30M05-012. (Reference 2).
- Limited Condition Survey Report, CNR Subway, Hwy 403, Site # 36-31, prepared by Morrison Hershfield, Report No. 1130336-CS-02, dated January 2014. (Reference 3).
- CNR Subway Bridge over Hwy 403, Site No. 36-31, Structural Steel Inspection Report, prepared for Morrison Hershfield Ltd., prepared by Domson Engineering & Inspection Ltd., Report No. 27731-36-31, dated December 19, 2013. (Reference 4).
- Ontario Bridge Management System (OBMS), Ontario Structure Inspection Manual – Inspection Form, CNR Subway Hwy 403, Site number 36-31, Regular OSIM 10-24-2016 dated December 15, 2016. (Reference 5).
- Archive drawings, CC Parker & Parsons, Brinckerhoff Limited Consulting Engineers, Department of Highways Ontario, Subway at CN Railway, District 4, dated August 15, 1961. (Reference 6).
 - General Arrangement, TWP # 1336-31-2A
 - Abutments – Details and Reinforcing
 - Pier Details – Anchor bolt setting, Test boring data
- Archive drawings, Highway 403, CNR Subway, Structure Rehabilitation, General Arrangement, Site 36-31, Cont. No. 2019-2013, W.P. 2440-13-00, Sheet 21, prepared by Morrison Hershfield, dated January 2019. (Reference 7).

2.0 SITE AND PROJECT DESCRIPTION

The existing bridge is located at the crossing of Highway 403 and the CNR tracks, approximately 720 m southwest of the Highway 403 and Highway 6 interchange in Hamilton, Ontario. The subway structure currently carries two rail tracks over the Highway 403 EBL (eastbound lane) and WBL (westbound lane).

Highway 403 in the vicinity of the site generally runs in a north to south orientation along a relatively flat terrain. The immediate lands to the southwest of Highway 403 EBL are part of the Royal Botanical Gardens (RBG). At the site, a Canadian Pacific Railway (CPR) subway structure runs approximately 30 m south and approximately parallel to the CNR structure.



The existing CNR structure was built in 1962 and consists of four-span steel plate I-girders with a reinforced concrete deck. The structure is supported on two abutments and three piers. According to available information, the existing abutments and piers are supported on spread footings founded on bedrock. The span arrangement for the bridge is 14.8 m, 19.5 m, 19.5 m, and 13.0 m, resulting in a total curvilinear length of 66.8 m. The bridge deck width is approximately 10.0 m. The two middle spans extend over Highway 403, and the end spans extend over the approach slopes. It is estimated, from archive drawings, that the existing west and east approaches are approximately 7.5 m and 9.0 m high, respectively. There are also retaining walls extending beyond the wingwalls at the abutment corners.

Selected photographs of the site are included in Appendix C.

It is understood that the structure was rehabilitated in 2019 (Reference 7). The rehabilitation program included installation of new steel railing, repair and recondition of existing bearings at piers and abutments, localized patch repairs at abutments, wingwalls and piers, curb, fascia, sidewalk and deck soffit, re-coating and general repair of structural steel, repair of an impact damage caused to girder and cross-bracing, repair and modification of expansion joint at piers and abutments, installation of new concrete slope paving at the east abutment, and installation of new deck drains at piers and new drainage at the west abutment.

A preliminary GA drawing provided by AECOM and prepared by WSP dated January 2022 indicates that the existing structure will be replaced by a new bridge to be located approximately 15 m north of the centreline of the existing bridge. The new bridge will be a double span structure supported on two abutments and one pier founded on spread footings. According to the GA drawing, the base of the footings are proposed to be at approximate Elevation 89.5 at the West Abutment, at approximate Elevation 88.0 at the Pier and, at approximate Elevation 86.0 at the East Abutment. The structure will carry two rail tracks and will have a span length of 70 m (35 m + 35 m). The bridge will consist of two decks each of approximately 7.5 m wide, with a 1.0 m gap in between them. The proposed west and east approaches will be in the order of 7.5 m and 9.0 m high, respectively. Retaining walls are proposed at both approaches.

It is anticipated that track realignment will be carried out to accommodate the new structure alignment.

The project area is situated within the physiographic region known as the Niagara Escarpment, which forms a north-south trending strip, and is a major topographic break in the bedrock between the carbonate Amabel Formation to the west and the soft sediments of the Queenston



Formation to the northeast. At many locations, the Queenston Formation consists of up to 1.2 m of very weathered bedrock (red clay) which grades downward into typical brick-red shale and often with green mottling. Thin to medium beds of grey-green and reddish argillaceous, hard limestone are present in most sections. The Queenston shale is overlain by Halton Till in the area of the site. The Halton Till is a red clay to clayey silt till and is exposed in the form of a till plain extending from Lake Ontario southward to the Niagara escarpment.

3.0 SITE OBSERVATIONS

A site reconnaissance visit was conducted by a Thurber Senior Geotechnical Engineer in June 2021 and January 2022 to observe conditions related to the foundation performance of the existing bridge and approaches. The following observations for the CNR subway structure have been noted during our site visits:

- There was no visible sign of settlement or distress along the bridge alignment.
- The existing approaches are fully covered with heavy vegetation including tall grass, and appeared to be in stable condition. The side slopes did not exhibit obvious sign of instability or bulging.
- Mid-height berms were observed along the south and north side slopes of the west approach.
- Seepage was noticed at the three rectangular piers, which are wet stained along the concrete faces. Several narrow vertical and diagonal cracks were noted on the piers.
- Wet/seepage stains were noted at the abutment walls. Vertical and alligator cracks were noted at the concrete surfaces of the abutment and ballast walls.
- Concrete slope paving is present on the forward slope of the east abutment. Continuous and large seepage stains were noted on the concrete slope paving from top to bottom. Pipe drains, revealing seepage, were also observed at the top of the forward slope.
- The surface of the west forward slope appeared to consist of granular material.
- In general, the girder bottom flange cover plates and bracing exhibit signs of corrosion, including rust formation, scaling and flake formation. Corrosion was also noted on steel elements at the piers and abutments.

Selected photographs of the site taken during the site visits are presented in Appendix C.



4.0 SUBSURFACE CONDITIONS

Two foundation investigations were conducted near the site in 1960 (References 1 and 2) prior to construction of the existing bridge. Borehole location plans from References 1 and 2 appear to indicate that five (5) boreholes (numbered 4, 5, 6, 7 and 9) were located in the vicinities of the CNR tracks and Highway 403 (formerly Chedoke Expressway). A Standard Dutch Cone (Dynamic Cone Penetration Test - DCPT) was conducted in proximity of Boreholes 7 and 9. The dynamics cone tests were numbered P7 and P9. The actual locations of these boreholes in relation to the existing bridge cannot be confirmed since a co-ordinate system was not used at the time and there was no available record of the as-built locations of the bridge. In general, the boreholes were advanced through overburden soils using continuous flight augers to shale bedrock. According to the Record of Borehole Sheets, the boreholes were further advanced to obtain BX and AXT rock cores. Record of Borehole Sheets of Boreholes 4, 5, 6, 7 and 9, Standard Dutch Cones P7 and P9, and a borehole location plan are included in Appendix A.

The soil stratigraphy encountered at the site during the previous investigations consisted of a surficial layer of native silty clay underlain by shale bedrock. The silty clay was described as red to brown and contained some sand, some gravel to gravelly, shale fragments and roots. The thickness of the silty clay varied from 0.6 m to 1.5 m. An SPT 'N' value measured in the silty clay was 7 blows per 0.3 m of penetration, indicating a firm consistency.

Shale bedrock of the Queenston Formation was contacted below the silty clay. The shale was described as red to red-brown in colour with bands and specks of green to grey shale bedrock, ranging in thickness from 6 mm to 380 mm. Clay seams and mud seams, varying in thickness from 25 mm to 75 mm, were noted in the recovered rock cores. The upper 1.5 m of the shale was described as soft due to weathering, becoming medium hard with depth. SPT 'N' values measured in the shale were greater than 100 blows per 0.3 m of penetration. Total Core Recovery (TCR) ranged from 80 percent to 100 percent. The Standard Dutch Cone tests P7 and P9 were terminated at 0.7 m and 1.6 m depth, respectively.

For the purpose of reporting herein, the upper zone of the shale will be referred to as weathered shale and the underlying sound portion will be considered as sound shale bedrock. The depths and elevations, where weathered and sound shale bedrock was proven, are presented in Table 4.1 below.

Table 4.1 – Depth and Elevation of Shale Along Existing Bridge

Approx. Location Relative to Bridge⁽¹⁾	Borehole	Depth⁽³⁾ to Weathered Shale (m)	Weathered Shale Elevation⁽²⁾ (m)	Depth⁽³⁾ to Sound Shale (m)	Sound Shale Elevation⁽²⁾ (m)
West Abutment	5 ⁽⁴⁾	-	-	1.5	98.1
Pier 3	4 ⁽⁴⁾	0.6	105.6	1.5	104.7
Pier 2	6 ⁽⁵⁾	0.6	99.0	Below 1.7	Below 97.9
Pier 1	9 ⁽⁵⁾ / P9	1.5	97.6	Below 1.5	Below 97.6
East Abutment	7 ⁽⁵⁾ / P7	0.6	98.2	Below 0.6	Below 98.2

- (1) The actual locations of these boreholes cannot be confirmed due to incomplete information. The foundation elements referred to here were those used during design at that time and it is unclear if they are directly correlated to those of the existing bridge.
- (2) The elevations were reportedly referenced to CPR and/or CNR benchmarks at the time of the investigation. It is unknown how these benchmarks are related to the Canadian Geodetic Datum currently in use.
- (3) All depths were converted from Imperial Units and relative to the ground surface prior to construction of the existing bridge and approaches.
- (4) Sound bedrock proved by coring.
- (5) Bedrock cores recovered; depth to sound shale uncertain due to coring commencing at varying depths/elevations.

It is also noted that the soil and rock conditions, particularly within the upper portion, may have been modified by the original construction.

Groundwater levels were measured at ground surface and at 2.4 m depth (Elevations 99.7 and 103.8) during the previous investigations. The previous report indicated that a considerable amount of water was flowing in the north ditch along the CNR tracks during the field investigation. It is anticipated that the current drainage pattern at the site has been largely governed by the drainage measures along the existing highway.

5.0 EXISTING FOUNDATIONS

Based on archive design drawings (Reference 6) and foundation recommendations (Reference 1), the existing subway structure was designed to be supported on two abutments and three piers using spread footings founded on sound shale bedrock at the elevations presented in Table 5.1 below.

Table 5.1 – Founding Depths and Elevations of Foundation Elements

Approx. Location Relative to Bridge⁽¹⁾	Borehole	Design Underside Elevation of Footing	Estimated Founding Stratum
West Abutment	5	93.1	Sound shale bedrock
Pier 3	4	89.8	Sound shale bedrock
Pier 2	6	88.7	Sound shale bedrock
Pier 1	9/P9	88.8	Sound shale bedrock
East Abutment	7//P7	92.4	Sound shale bedrock

References 1 and 2 recommended that spread footings founded on shale bedrock be designed using a “conservative” allowable bearing capacity of approximately 645 kPa (6 tsf). The use of spread footings for the piers and abutments appears to be consistent with what is shown on the archive drawings (Reference 6), where the quoted allowable bearing pressure is 645 kPa (6 tsf) for shale bedrock.

Reference 6 indicated that the piers are about 10.4 m (34 ft.) long and 4.5 m (15 ft.) wide, and that the abutment strip footings are about 4.3 m (14 ft.) in width.

6.0 ASSESSMENT OF EXISTING FOUNDATIONS

The archive boreholes from References 1 and 2 were advanced at locations and elevations that cannot be confirmed. We understand that the existing bridge is to be replaced with a new bridge to be located approximately 15 m north (centreline to centreline) of the existing bridge, with new foundation elements at different locations. Since bridge replacement is involved and given the uncertainties regarding the archive boreholes and information on them, it is recommended that new boreholes be advanced at selected locations (Section 13.0 below) in order to obtain adequate information for preliminary foundation design of the replacement bridge.

A foundation assessment of the existing structure, based on current information, has been carried out to provide some information to the designers regarding the feasibility of the proposed foundations.

There is insufficient subsurface information for assessing the strength and deformation characteristics of the shale bedrock. There is very limited to no data on unconfined compressive strength, rock quality and fracture index on which the geotechnical resistance is based. For the purpose of this assessment, the Hoek and Brown rock characterization criteria and typical range



of unconfined compressive strengths for Queenston shale have been used. Reference has also been made to geotechnical resistances found in published information and past projects in the general area of the site.

For spread footings founded on undisturbed weathered, fair quality Queenston shale bedrock, it is assessed that the factored geotechnical resistance would be in the order of 1,000 kPa at Ultimate Limit States (ULS). For sound, slightly weathered to fresh intact shale bedrock, it is assessed that the factored geotechnical resistance at ULS could be up to the order of 1,500 kPa or greater depending on fracture pattern and rock strength etc. These values apply to vertical and concentric loads. The SLS condition does not apply to footings founded on unyielding bedrock.

According to the archive drawings, the footings founded on sound Queenston shale (below the weathered zone) were designed as per the recommendations in Reference 1 discussed above using an allowable bearing resistance of about 645 kPa, which is lower than the assessed value of fair quality shale. There is, however, no documentation on the conditions of the founding shale for confirmation.

Based on the above assessment, it is considered feasible that the abutment and pier foundations of the new bridge be supported on spread footings founded on sound shale bedrock.

The design of the bridge must be carried out in accordance with the CNR design manuals, American Railway Engineering and Maintenance-of-Way Association (AREMA) guidelines, CHBDC 2019, MTO guidelines and all other applicable codes and standards having jurisdiction over the project.

7.0 EARTH AND ROCK CUTS

The archive design information and site observations suggest that this section of Highway 403 has been constructed in a rock cut.

According to the preliminary GA drawing, cuts will be required in order to accommodate the new east and west abutments. Cuts up to the order of 5 to 6 m deep are anticipated at the west abutment area, while cuts up to the order of 10 to 11 m are anticipated at the east abutment area. The cuts will extend into the existing fill, native soils and shale. It is anticipated that the existing highway drainage systems would be sufficient to maintain relatively dry excavations during construction, although accumulation of surface runoff and precipitation should be expected.



Temporary drainage of the cuts should be provided, where required, to maintain relatively dry and stable excavations. Surface runoff and precipitation should be diverted away from the excavations at all stages during construction. Permanent drainage will be required along the realigned highway and new bridge. It is recommended that the water be controlled by means of ditches incorporated within the highway design.

For temporary slopes, plastic sheetings or tarps may be used for covering where required.

Temporary protection (shoring) may be required at some locations for the earth cut operations. Preliminary comments on temporary protection (shoring) are presented in Section 11.0 of this report.

8.0 APPROACH SLOPES

Reference 6 indicates that the design approach and side slopes be at an inclination of 2H : 1V. Our site observations indicate that the existing approaches are generally in stable condition.

Based on the preliminary GA drawing, the proposed west and east approaches will be in the order of 5 m to 11 m in height. It is proposed to place approximately 1 m of new fill on both sides to raise the grade. It is recommended that the permanent earth and shale slopes be designed for an inclination of 2H : 1V or flatter to maintain global stability. Mid-height benches would be required for exposed earth cut slopes greater than 6 m in depth.

The subgrade for the new fill is expected to be existing fill, native silty clay and shale. No global stability issues are anticipated for approaches at an inclination of 2H : 1V or flatter, provided the approved new fill is placed and compacted in accordance with OPSS.PROV 206, OPSS.PROV 501 and AREMA, and provided that all surficial vegetation, organics and topsoil, soft/loosened or wet soils and debris are removed from the proposed embankment footprints prior to fill placement.

It is recommended that all exposed slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Erosion protection measures must be provided for the slopes.

Foundation settlement of the soil subgrade is expected to take place as the fill is placed and be completed by the end of construction. Settlement of the underlying shale may be considered negligible. The magnitude of post construction settlement due to compression of the embankment fill itself depends on the type of materials to be used, but it is not anticipated to exceed 10 mm if the new fill is placed and compacted as outlined above.

9.0 ABUTMENT WALL BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutment should consist of free-draining granular material conforming to OPSS.PROV 1010 Granular A or B Type II specifications. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

Earth pressures acting on the structure may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC 2019 but generally are given by the expression:

$$p = K (\gamma h + q)$$

Where:	p	= horizontal earth pressure on the wall at depth h (kPa)
	K	= earth pressure coefficient (see table below)
	γ	= unit weight of retained soil (see table below)
	h	= depth below top of fill where pressure is computed (m)
	q	= value of any surcharge (kPa)

The earth pressure coefficients are dependent on the material used as backfill. Recommended unfactored values are shown in Table 9.1. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for any wingwalls or unrestrained walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is generally preferred as it results in lower earth pressures acting on the wall.

Table 9.1 – Lateral Earth Pressure Coefficients

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H : 1V)	Horizontal Backfill	Sloping Backfill (2H : 1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At-rest (Restrained Wall)	0.43	0.62	0.47	0.70
Passive	3.7	-	3.2	-



10.0 EXCAVATION AND GROUNDWATER CONTROL

All excavations must be carried out in accordance with OPSS.PROV 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope and temporary support requirements in compliance with the OHSA, the fills above water level and native cohesive soils are classified as Type 3 soils. Cohesionless soils and fill below water level are classified as Type 4 soils.

It is anticipated that excavation of existing fills and native cohesive soils would be required throughout the site. Shale excavation will also be involved at the foundation locations. Where required, shale excavation should be carried out using methods that will avoid disturbing the intact bedrock below the founding elevation. It is possible that rock excavation may extend into relatively sound shale with hard limestone interbeds at some locations. Heavy excavating equipment, ripping machinery and rock breakers/splitters may be required to break up strong limestone slabs.

It is anticipated that the existing highway drainage systems would be sufficient to maintain relatively dry excavations during construction. However, seepage or perched water from the water-bearing interlayers within the soils as well as accumulation of surface runoff and precipitation are to be expected. Also, concentrated seepage may be experienced from seams or fractures in the shale. All surface runoff should be diverted away from excavations.

The Contractor should be prepared to pump from properly filtered sumps to remove any seepage water or surface water collecting in an excavation. Unwatering must remain operational and effective until the excavation is backfilled.

The design of any dewatering or unwatering systems that may be required is the responsibility of the Contractor.

Where required, construction will need to be carried out in conjunction with temporary protection.

11.0 RAIL TRACK PROTECTION AND SHORING

During bridge replacement operations where excavations may be required in the vicinity of the existing rail tracks, track protection should be designed and provided in accordance with AREMA, Chapter 8. Discussions with the railway authorities should be carried out to determine the required performance level of protection.



Due to shallow shale bedrock, sheetpiles and driven H-piles do not appear to be suitable for use as temporary protection. An augered soldier pile and lagging system with H-piles socketted into the shale should be feasible.

The design of railway and other temporary protection (shoring) is the responsibility of the Contractor. All rail track protection should be designed by a Professional Engineer experienced in such designs.

12.0 ADJACENT STRUCTURES AND BURIED UTILITIES

Based on information provided in Reference 3, a steel conduit containing CNR cables was found protruding from the ground at the end of the southeast and southwest wingwalls. It is recommended that the exact locations of this conduit and any existing utilities that are present in the vicinity of the work areas be established by the designer, and compared with the extent of the potential work zones related to the proposed construction.

No utilities or conduits should be undermined or damaged during construction of the new bridge and approaches. Relocation of, and/or special protective measures for, some or all of these affected utilities and conduits may be required.

13.0 INVESTIGATION FOR PRELIMINARY DESIGN

References 1 and 2 are available from the GEOCREST library for this site. As discussed previously, these reports were prepared in the early 1960's prior to construction of the existing bridge and approaches. The locations and elevations of the boreholes cannot be confirmed. It is also known that the site topography had been altered as part of the original construction. Given that the proposed replacement bridge will be along a new alignment and the archive boreholes do not provide much information on the shale including unconfined compressive strength, rock quality and fracture pattern to facilitate a more detail assessment of rock geotechnical resistance that is critical for the replacement bridge foundation design, it will be necessary to carry out additional site investigation and field testing to support the preparation of foundation design recommendations for preliminary design of the new bridge and its approach fills.

In consideration of the currently available design information, a preliminary investigation for preliminary design is proposed as follows:



- One (1) borehole near the pier location and one (1) borehole near a selected abutment location, each to be advanced to core a minimum 3 m of shale.
- The depth to shale varies across the site, but based on the archive information, should not exceed about 2 m to 4 m below the highway grade. Depending on where the boreholes are going to be drilled, the borehole depths are anticipated not to exceed the order of 8 m.
- The proposed abutments are to be located part way up the slope and in close proximity to the rail tracks. Advancing a borehole at the abutments will require access preparation and specialized portable equipment which should better be carried out once the abutment locations are finalized during detail design. For preliminary design, it is considered appropriate to advance a borehole from highway grade near the toe of the existing cut slope just below the proposed abutment location.

The proposed boreholes are schematically shown on a plan in Appendix D for illustrative purposes.

For detail design, the full requirements of the MTO (2022) guideline will need to be satisfied.

14.0 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Rocio Reyna, P,Eng. The report was reviewed by Sydney Pang, P.Eng. and P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



THURBER ENGINEERING LTD.



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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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5. INTERPRETATION OF THE REPORT

- a) **Nature and Exactness of Soil and Contaminant Description:** Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b) **Reliance on Provided Information:** The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) **Design Services:** The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) **Construction Services:** During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.



Appendix A

Record of Borehole Sheets and Borehole Plan (Geocres)

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-35

W.P. 100=60 & 99-60.

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
4	RC1	0'-2'	Stiff red clay with presence of organic materials.							
	RC2	2'-5'	Soft red shale							Ran BX casing - no core.
	RC3	5'-10'	Medium hard red shale with seams of hard green shale. Presence of mud seams in shale.							AXT core. 80% core recovery.
	RC4	10'-15'	Medium hard red shale with seams of hard green shale. 9" seam of soft shale from 13'3" to 14'0".							AXT core. 90% core recovery.
	RC5	15'-20'	Medium hard red shale with seams of hard green shale. 15" seam of soft shale from 15-16'3.							AXT core. 85% core recovery.
	RC6	20'-25'	Medium hard red shale with seams of hard green shale.							AXT core. 92% core recovery.
	RC7	25'-30'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
	RC8	30'-35'	Medium hard red shale with seams of hard green shale.							AXT core. 97% core recovery.
	RC9	35'-40'	Medium hard red shale with seams of hard green shale.							AXT Core. 95% core recovery.
	RC10	40'-45'	Medium hard red shale with seams of hard green shale.							AXT core. 90% core recovery.
	RC11	45'-50'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-35
W.P. 100-60 & 99-60.

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
4	RC12	50'-55'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
	RC13	55'-60'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
	RC14	60'-65'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
	RC15	65'-68'6"	Medium hard red shale with seams of hard green shale.							AXT core. 93% core recovery.
5	RC1	0'-5'	Stiff red clay with fine to coarse sand and gravel.							BX casing shoe
	RC2	5'-6'	Medium hard green shale.							BX casing shoe
	RC3	6'-11.0'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
	RC4	11'-17'7"	Medium hard red shale with seams of hard green shale.							AXT core. 97% core recovery.
			RC denotes rock core samples.							

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W + 190-60 & 99-60

BORE HOLE NO. 4

JOB 60-F-35

STATION See drawing






DATUM Geodetic 348.4

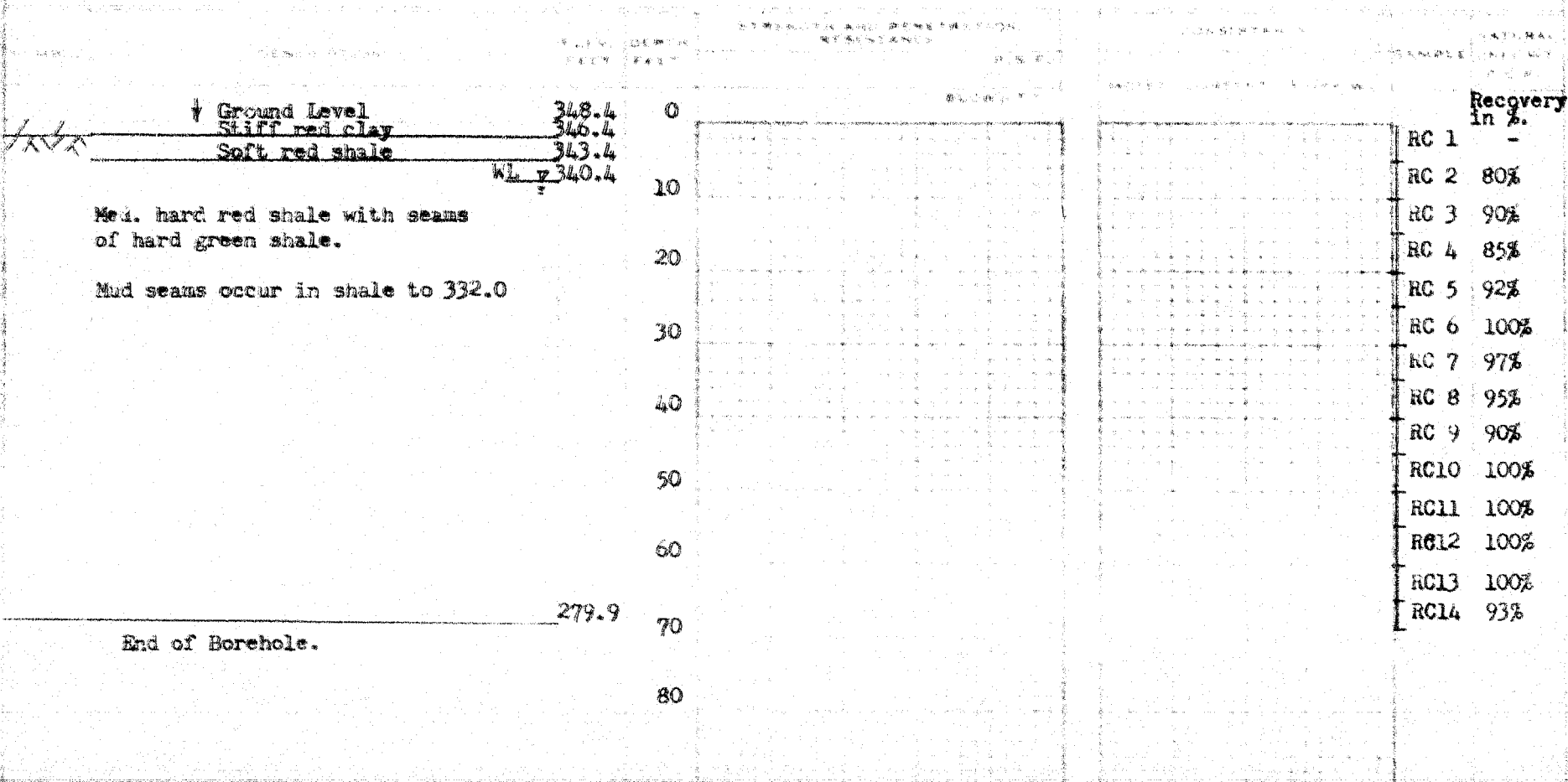
COMPILED BY G. G. C. & B.K.

BORING DATE May 5/60

CHECKED BY G.G.C.

LEGEND

UNCONFINED COMPRESSION (QU) 
VANE TEST (C) AND SENSITIVITY (S) 
NATURAL MOISTURE AND LIQUIDITY INDEX 
Liquidity Limit 
PLASTIC LIMIT 



DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 100-60 & 99-60 BORE HOLE NO. 5

JOB 60-F-35 STATION See drawing

DATUM Geodetic 327.0 COMPILED BY G.G.C. & B.K.

BORING DATE May 10/60. CHECKED BY G.G.C.

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (QU)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT





SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P.S.F.	BLOWS/FT.
	↓ Ground level	W.L. 327.0	0		
	Stiff red clay	322.0			
	Medium hard red shale with seams of green shale.		10		
		309.4	20		
	End of borehole.		30		
			40		
			50		
			60		
			70		
			80		

CONSISTENCY		NATURAL MOIST. WT. %	RECOVERY IN %.
MOIST. CONTENT - % DRY WT.	LIQUIDITY INDEX		
		RC1	-
		RC2	-
		RC3	100%
		RC4	97%

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Name Chedoke Expressway, C.N.R. Job No. 6003 Borehole No. 6
ient Dept. of Highways, Ontario Casing BX (AX core) Boring Date Jan. 9 & 11 1960
atum Client's Compiled By P.M. & A.M. Checked By E.M.P.

SAMPLE CONDITION

-  UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

- A.S. AUGER SAMPLE
C.S. CASING SAMPLE
S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
C. SOIL SHEAR STRENGTH LBS/SQ.FT.
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL
W.T.P.L. WETTER THAN PLASTIC LIMIT
D.T.P.L. DRIER THAN PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
Ground surface			0'0"						
Gravelly silty clay, roots	Dk red-brown		320.70			C.S.			Sat.
Clay shale	Red-brn & grn-grey		2'0"			C.S.			Dry
Shale	Reddish-brn.					S.S.	127/1"	5.4	Dry
Transitional shale	Reddish-brn & band of grn-grey		5'5"			S.S.	100/8"	8.2	
Reamed hole from 5'5" to 25'0" with concave bit									
(No attempt to recover core)									
Core recovered below 25'0"			25'0"						
Red -brown shale with 3 bands of grey shale 1/4" to 3" thick						R.C.			Longest piece 6-3/4" 38 fractures
			30'0"						4'8 1/2" recovery 94%
Red-brown shale with 6 bands of grey shale 1/4" to 1 1/2" thick						R.C.			Longest piece 3-1/4" 36 fractures 100% recovery
			34'0"						Core v. moist to wet from 34' to 35'8"
Red-brown shale with 3 bands of grey shale 1/2" to 7" thick						R.C.			Longest piece 2-3/4" Very badly fractured 100% recovery
			39'0"						
Red-brown shale with 3 bands of grey shale 1/4" to 1 1/2" thick									(Wet seam 40'9" to 41'3" approx.) Longest piece 3-3/4" 47 fractures 4'9" Recovery 45%
			44'0"						
Borehole terminated at 44'0"									Wash water returned during diamond drilling but W.L. dropped to 34' on completion.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Chedoke Expressway, C.N.R. Job No. 6003

Borehole No. 2

Client Dept. of Highways, Ontario Casing BX

Boring Date Jan. 7, - 9th, 1960

..... Datum Client's Compiled By P.M. & A.M.

Checked By E.M.P.

SAMPLE CONDITION

 UNDISTURBED

 FAIR

 DISTURBED

LOST

SAMPLE TYPE

A.S. AUGER SAMPLE

C.S. CASING SAMPLE

2.5. 2" STANDARD SPLIT TUBE SAMPLE

S.L. SPLIT BARREL WITH LINERS

S.T. THIN-WALLED SHELBY TUBE SAMPLE

W.S. WASH SAMPLE

R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST

C SOIL SHEAR STRENGTH LBS/SQ.FT.

W.L. WATER LEVEL IN CASING

W.T. GROUND WATER TABLE IN SOIL

W.T.P.L. BETTER THAN PLASTIC LIN

D.T.P.L. DRIER THAN PLASTIC LIMIT

[illegible]

OBSERVATIONS AND CONCLUSIONS (continued)

take up moisture readily and will roll and heave during compaction effort if it becomes too wet.

(d) Since we understand that this site is to be abandoned as the crossing for the expressway, we have not carried out any strength tests on the rock core. In the absence of such tests we would suggest an allowable bearing value for this shale of 6.0 tons per square foot. This figure is, in our opinion, quite conservative.

(e) As stated previously, Dutch cones were driven at boreholes 2, 7, 9 and 11. The following table gives the results:

P-2

<u>Depth</u>	<u>Blows per foot</u>
0 - 1'	12
1 - 2'	6
2 - 3'	17
3 - 4'	26
4 - 5'	49
5 - 6'	60
6 - 7'	67
7 - 8'	130
8 - 9'	100
9 - 10'	133
10 - 11'	160
11' - 11'11"	240
11'11" - 12'	103

P-7

0 - 1'	1
1 - 2'	79
2' - 2'5"	100
2'5" - 2'5"	100

P - 9

0 - 1'	1
1 - 2'	1
2 - 3'	1
3 - 4'	23
4 - 5'	140
5' - 5'2"	100
5'2" - 5'3"	300

OBSERVATIONS AND CONCLUSIONS (continued)

P - 11

<u>Depth</u>	<u>Blows per foot</u>
0 - 1'	3
1 - 2'	1
2 - 3'	21
3 - 4'	41
4 - 5'	47
5 - 6'	50
6 - 7'	80
7 - 8'	309

We trust that the information supplied briefly herewith may be useful in review of conditions existing at any adjacent site now under examination. Should you require any supplementary information concerning this investigation, we shall be pleased to be of further service.

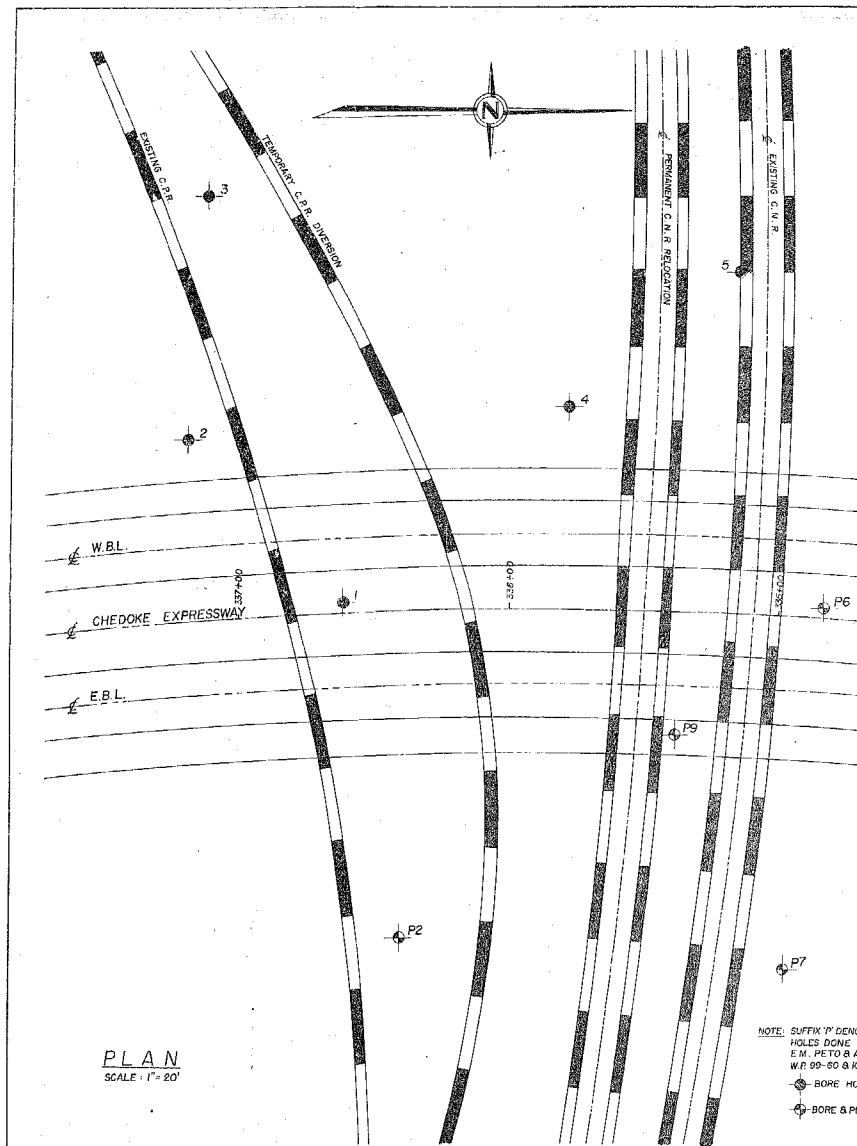
Yours very truly,

E. M. PETO ASSOCIATES LTD.

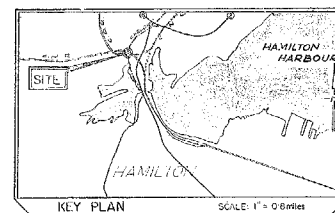
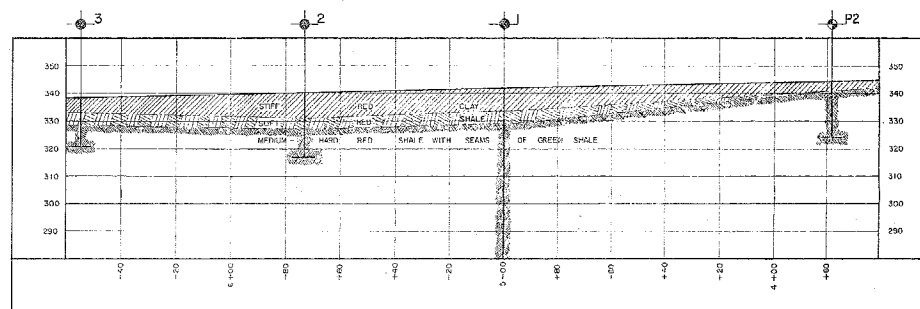
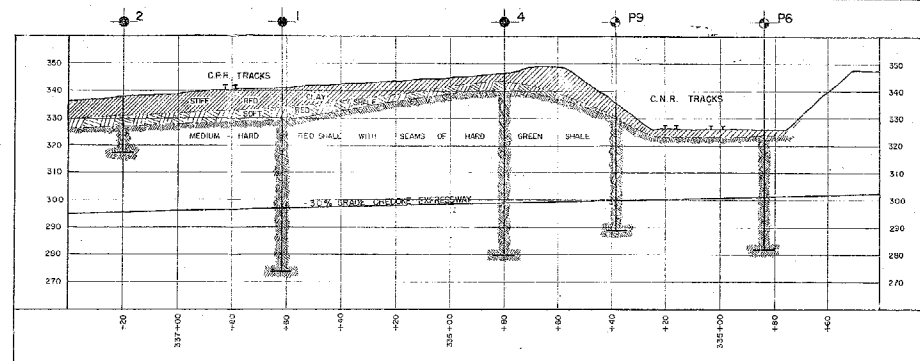


C. F. Freeman, P. Eng.
Chief Engineer

UJV/jn



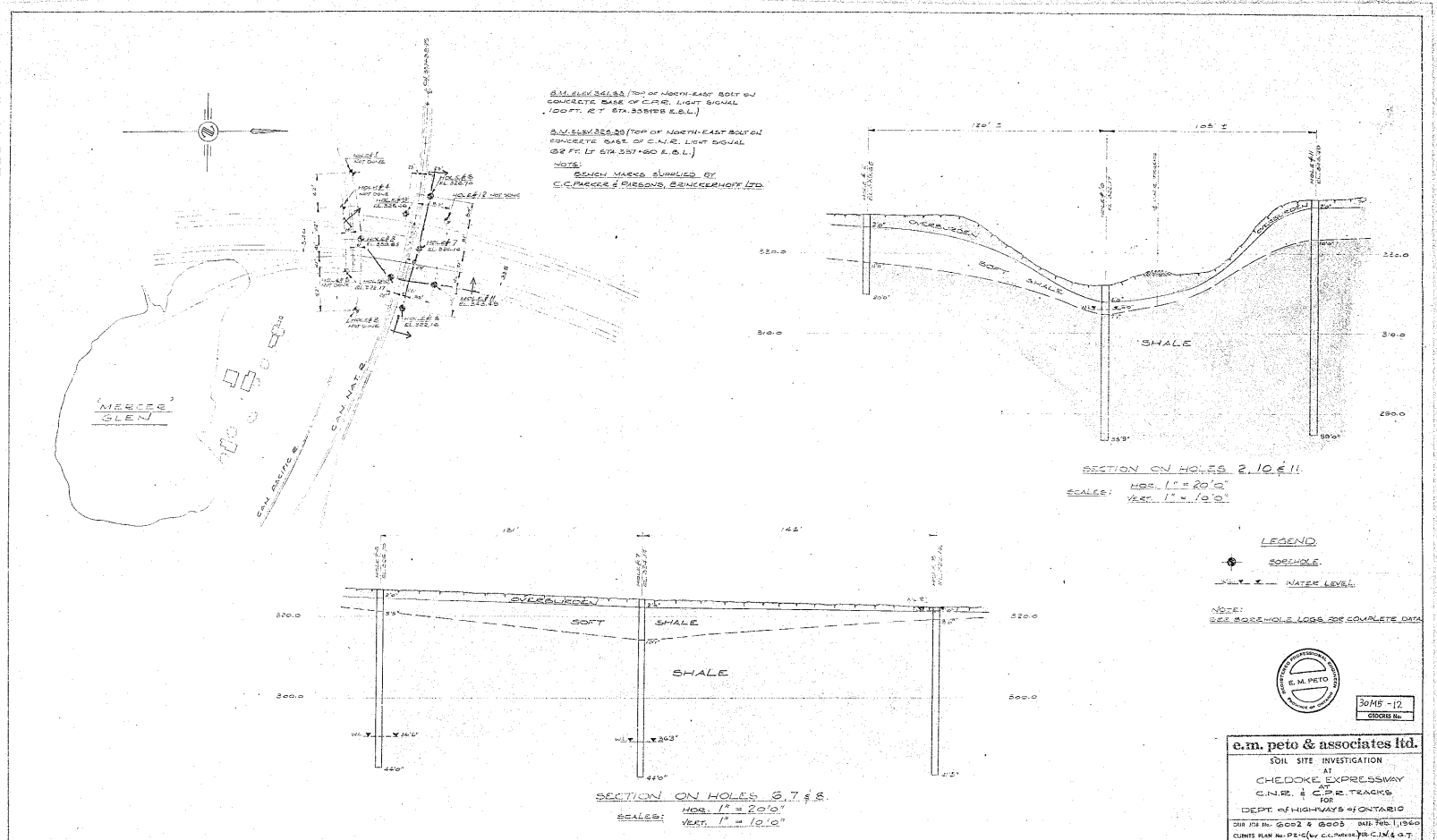
NOTE: SUFFIX "P" DENOTES
HOLES DONE BY
E.M. PETO & ASSOCIATES
W.P. 99-60 & 100-60
● BORE HOLE
● BORE & PENETRATION HOLE



NOTE:
THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN
ESTABLISHED ONLY AT BORE HOLE LOCATIONS BETWEEN
BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEO-
LOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR

NOTE: ALL ELEV. REFER TO GEODETIC

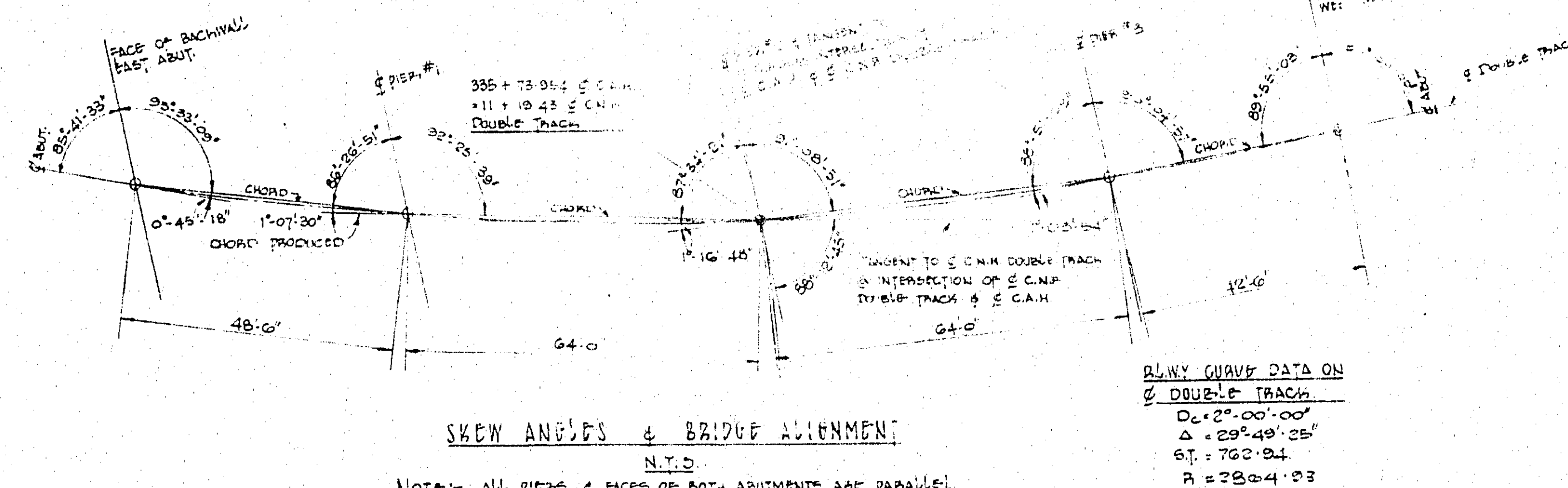
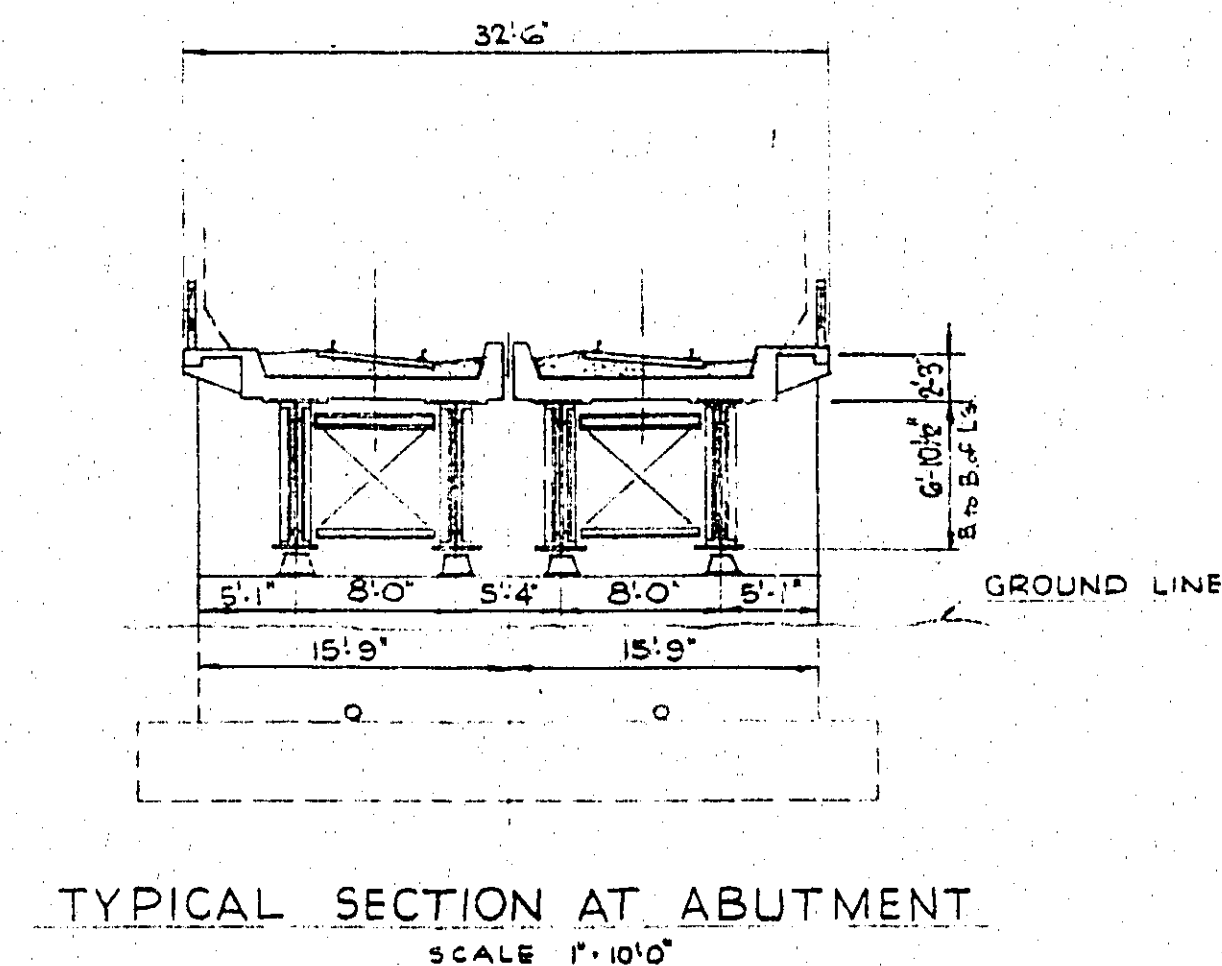
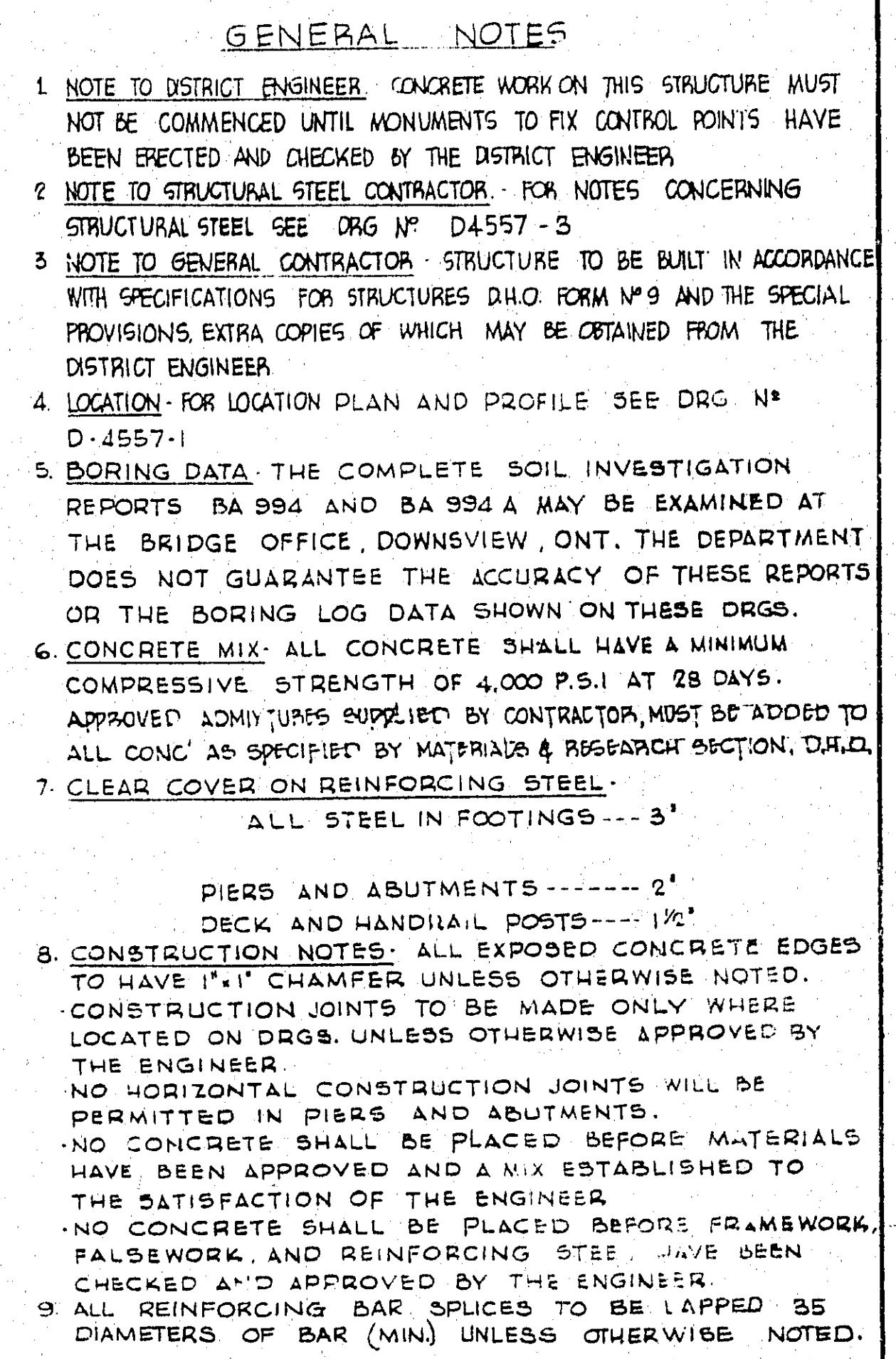
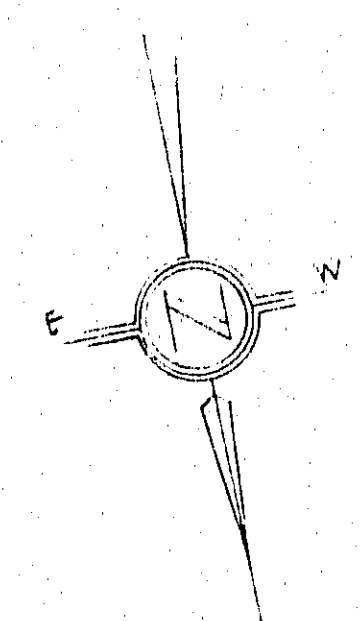
DEPARTMENT OF HIGHWAYS — ONTARIO			
MATERIALS & RESEARCH SECTION			
C.N.R. & C.P.R. CROSSINGS			
SHOWING POSITIONS & ELEVATIONS OF HOLES			
HIGHWAY 403 DISTRICT 4		COUNTY WENTWORTH	
LOCATION HAMILTON			
DRAWN BY J. J. [Signature]	CHECKED BY [Signature]	W.P. 99-60 & 100-60	
DATE 8 JUNE 1960	APPROVED BY [Signature]	SCALE 1" = 20'	
			60-F-35A





Appendix B

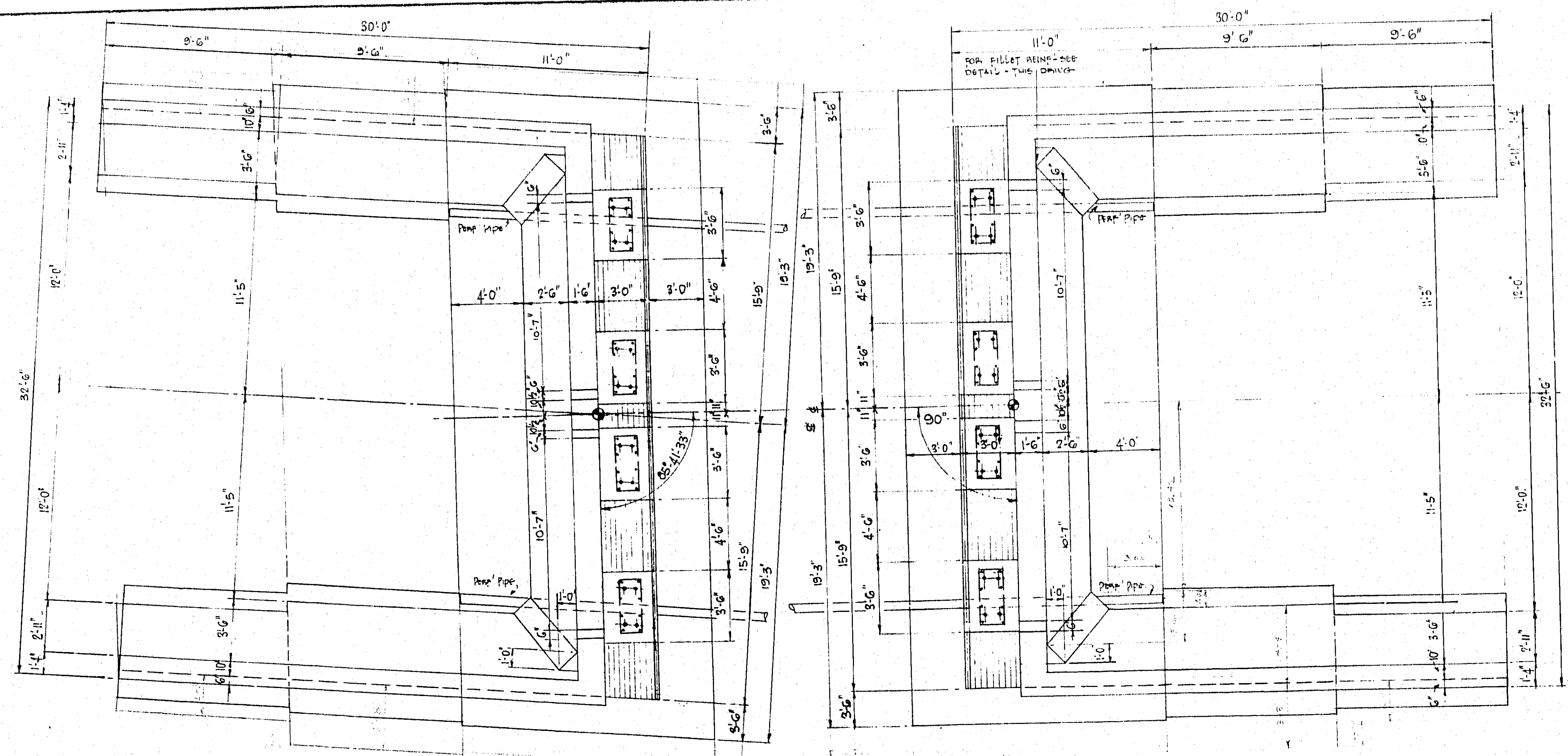
Archive Drawings of Existing Bridge (Construction and Rehabilitation)

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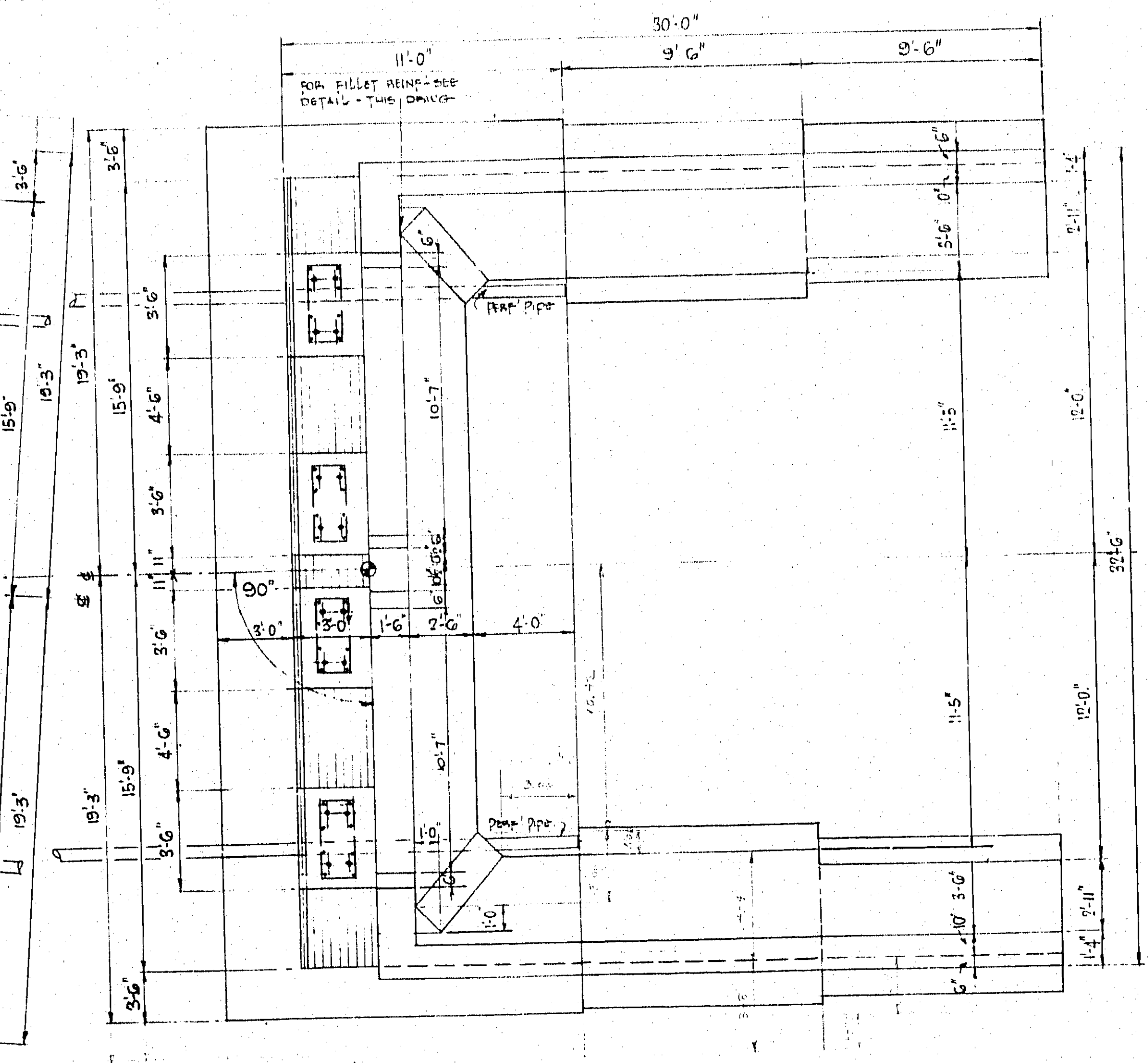
REVISIONS:		DESCRIPTION
DATE	BY	REVISD CARBONALUMINUM RICH AT-CONSTRUCTIO, PER

C C PARKER & PARSONS, BRINCKERHOFF LIMITED									
HAMILTON			CONSULTING ENGINEERS				ONTARIO		
DEPARTMENT OF HIGHWAYS-ONTARIO-									
BRIDGE OFFICE-TORONTO									
SUBWAY AT C.N. RAILWAY									
THE KING'S HIGHWAY No. 403						DIST. No. 4			
CO. OF WENTWORTH			CHEDOKE		BRIDGE NO. 8.				
TWP. WEST FLAMBOROUGH			LOT 28		CON. 1				
GENERAL ARRANGEMENT.									
APPROVED									
<i>Wm. L. ...</i>									
BRIDGE ENGINEER									
DESIGN ENGINEER									
DESIGN		I.M.W.		CHECK		R.K.C.		CONTRACT NUMBERS	
DRAWING		R.M.T.		CHECK		D.C.C.		6-5557-1 6-5557-2	
DATE				CHECK				6-1-236 6-1-174	
DATE		AUG 15 1921				LOADING		DRAWING NUMBER	
						F-60		D 4557-2	

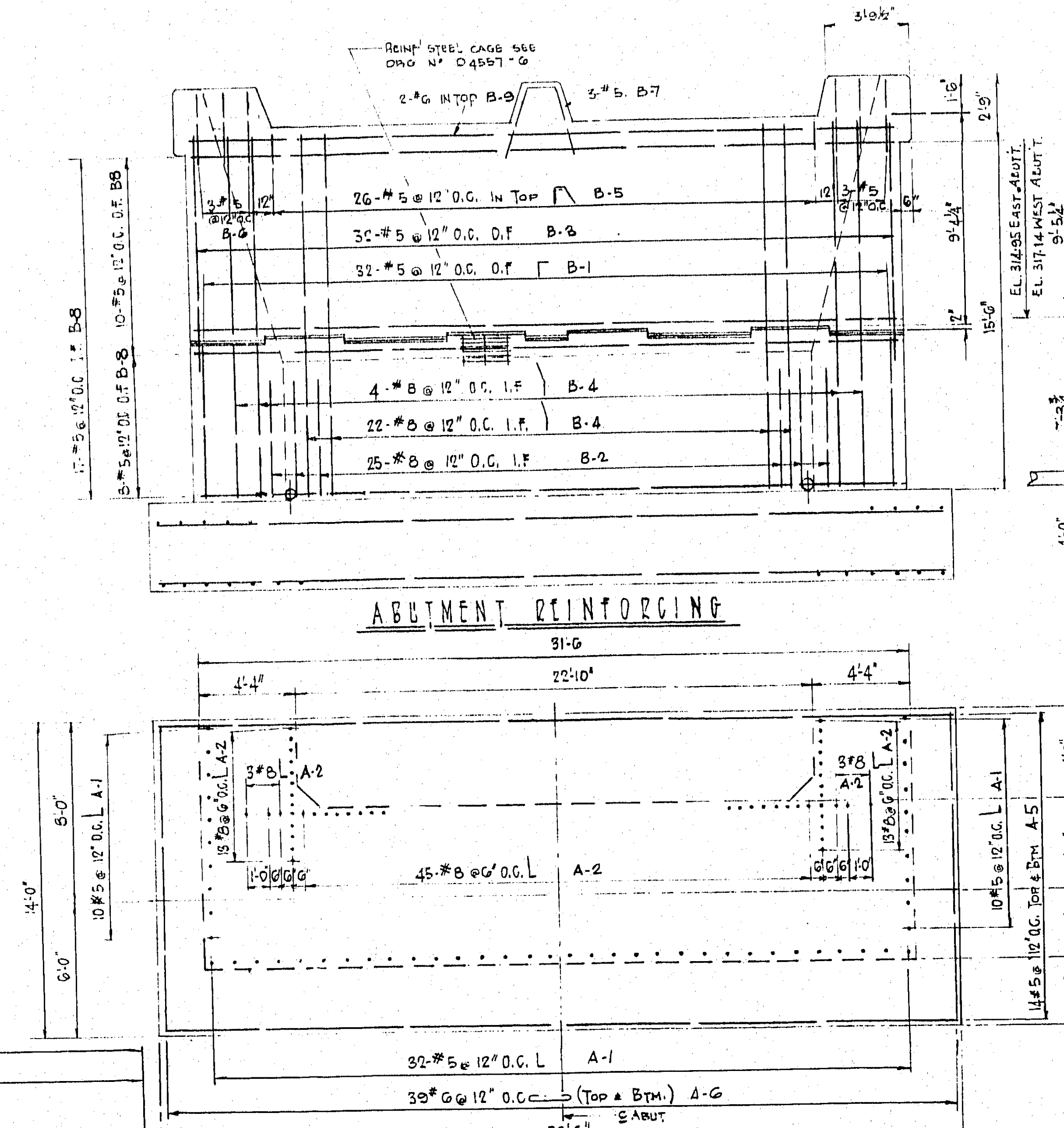
Tw# 1336-31-2-A Tw# 1336-31-1336-78-2



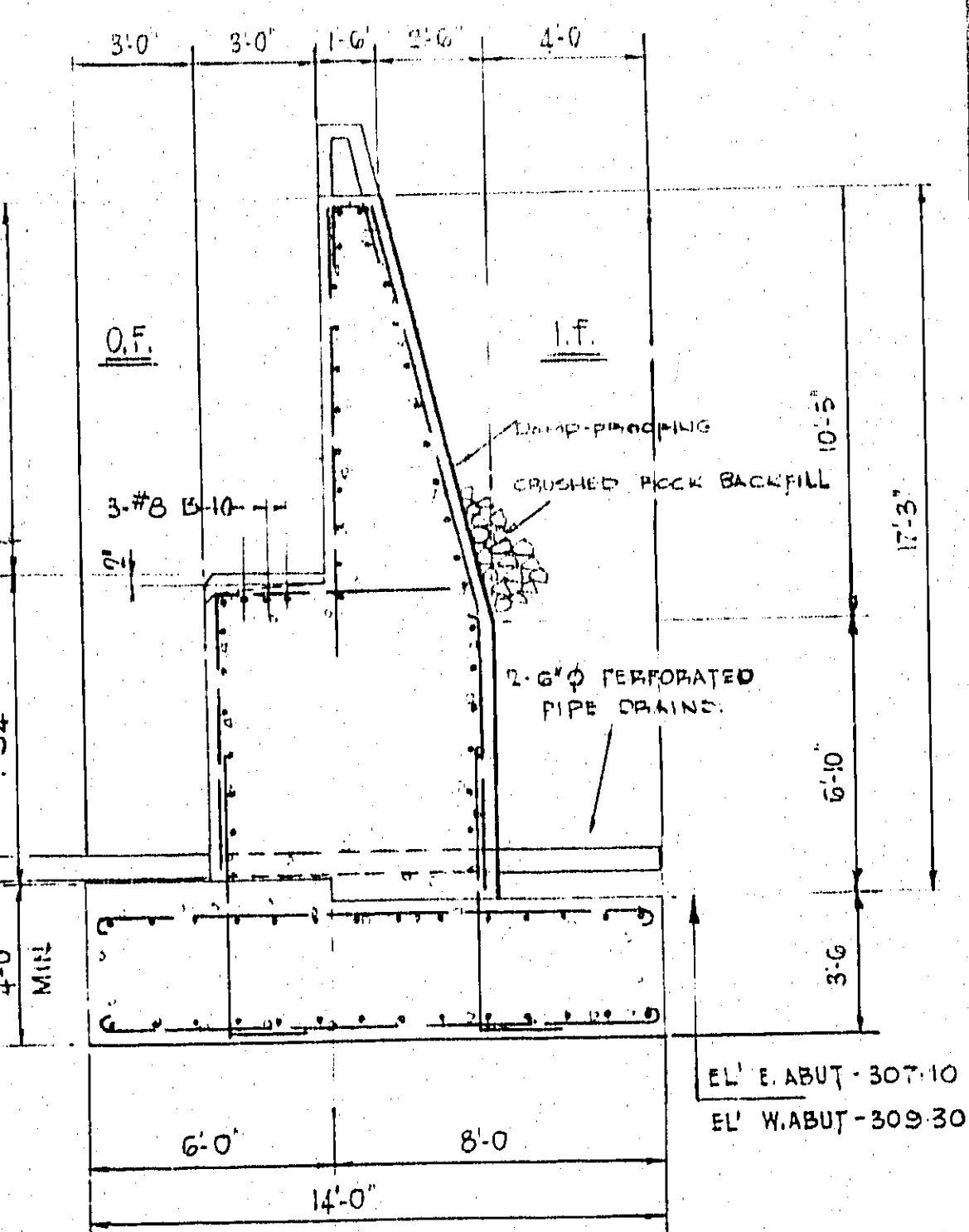
PLAN ON EAST ABUTMENT



PLAN ON WEST ABUTMENT

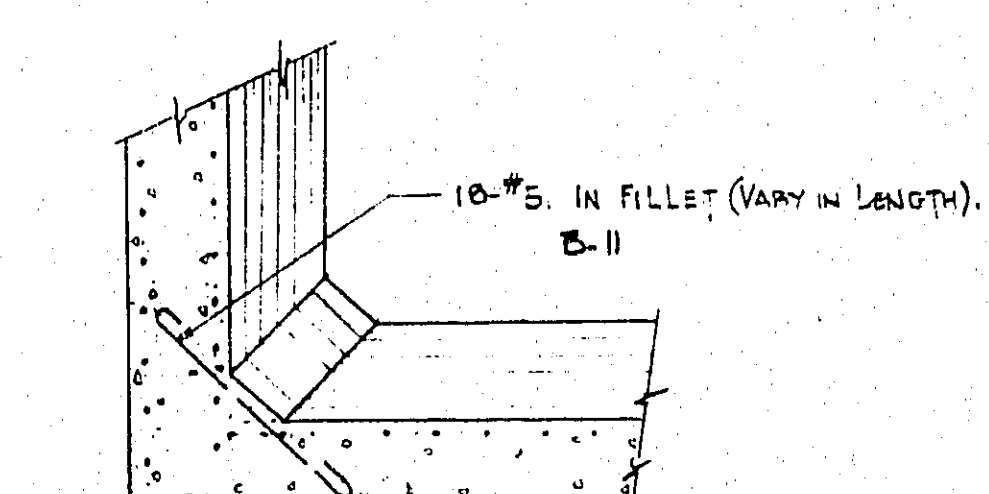


ABUTMENT REINFORCING



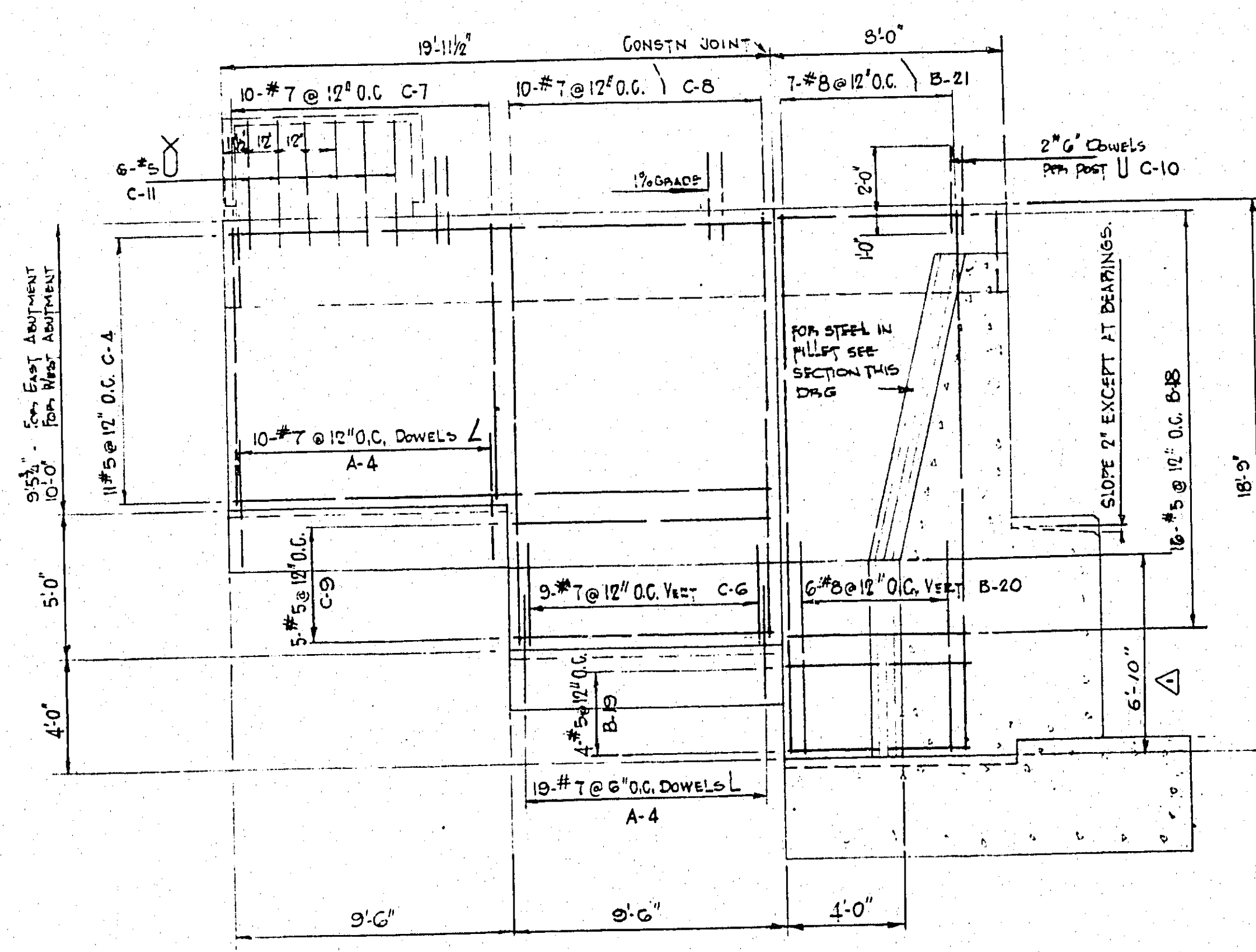
SECTION

- NOTES:
1. E. ABUT. SAME AS W. ABUT. EXCEPT WING WALLS & FOOTING SKEWED.
 2. ALL REINFO. IN E. ABUT. SAME AS IN W. ABUT.

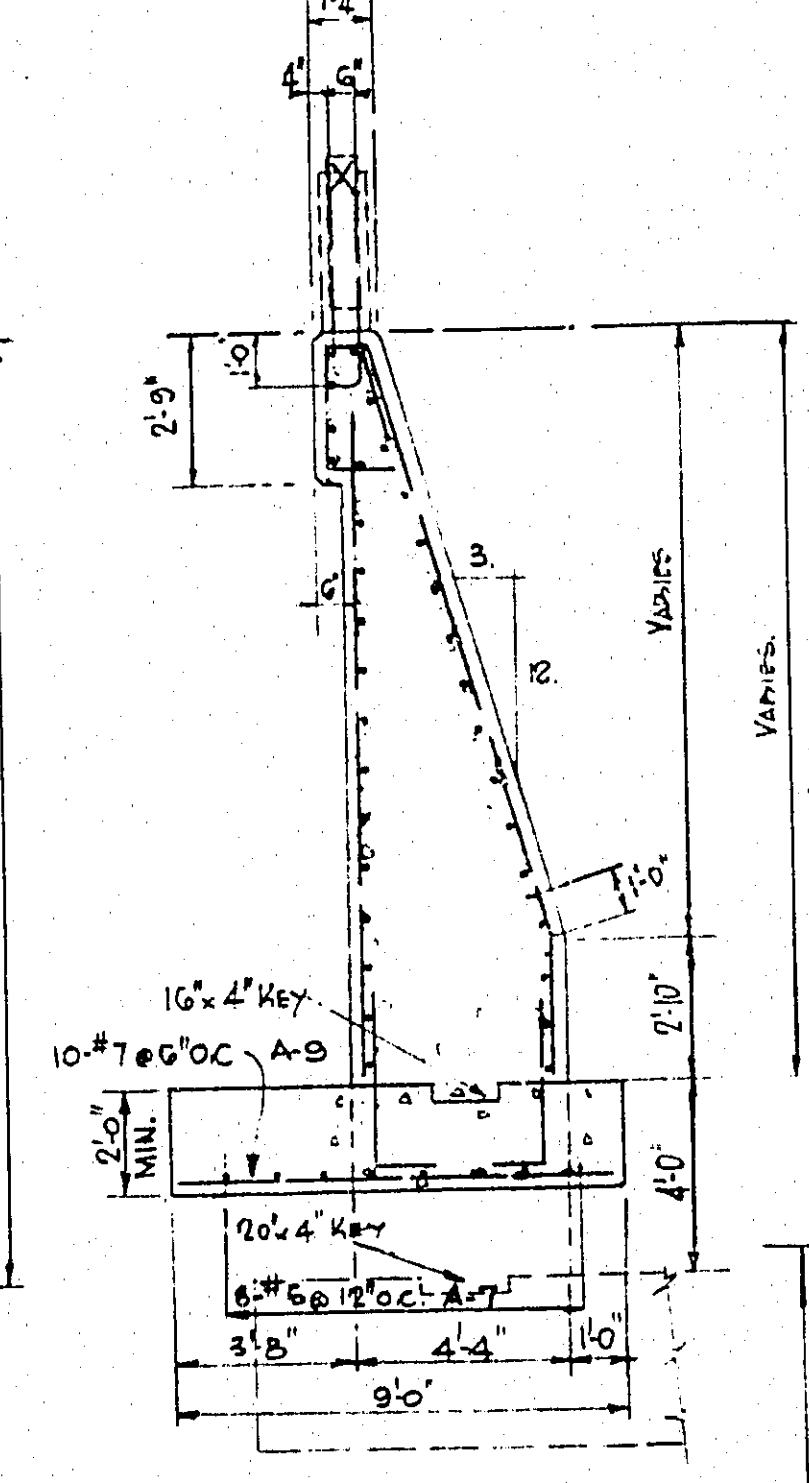


PLAN SHOWING FILLET STEEL [TYPICAL]

- NOTES:
1. FOR GENERAL NOTES - SEE DRG. NO. D-4557-2.
 2. FOR NOTES CONCERNING STRUCTURAL STEEL SEE DRG. NO. D-4557-3.
 3. FOR NOTES CONCERNING FOUNDATION CONSTN. SEE DRG. NO. D-4557-6.
 4. EAST ABUT. SAME AS WEST ABUT. EXCEPT FOR SKEW AND SLOPE OF TOP OF WING WALLS.
 5. ALL REINFORCING IN EAST ABUT. SAME AS IN WEST ABUT.
 6. FOR SUBSTRUCTURE LAYOUT SEE DRG. NO. D-4557-6.

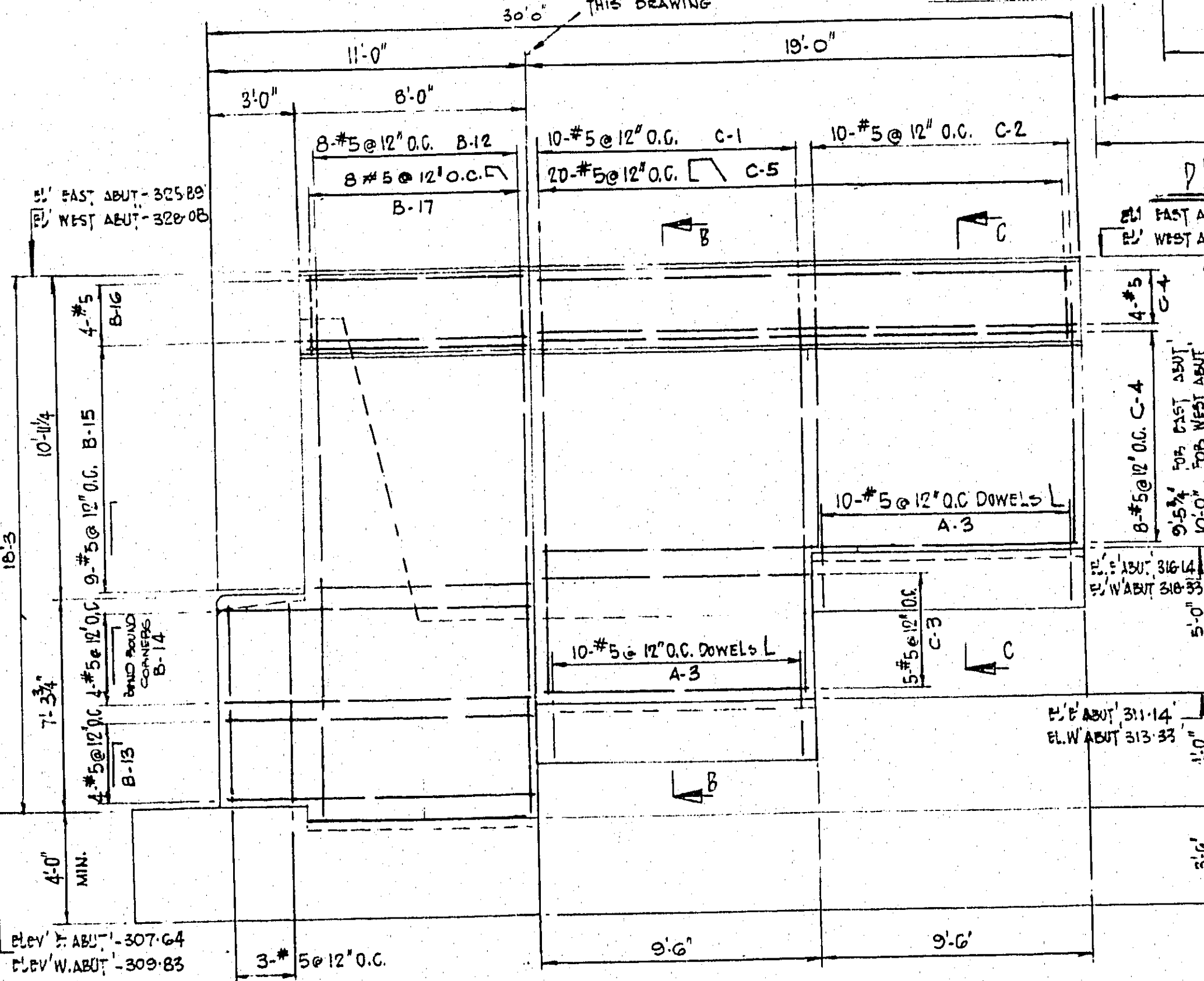


INSIDE FACE

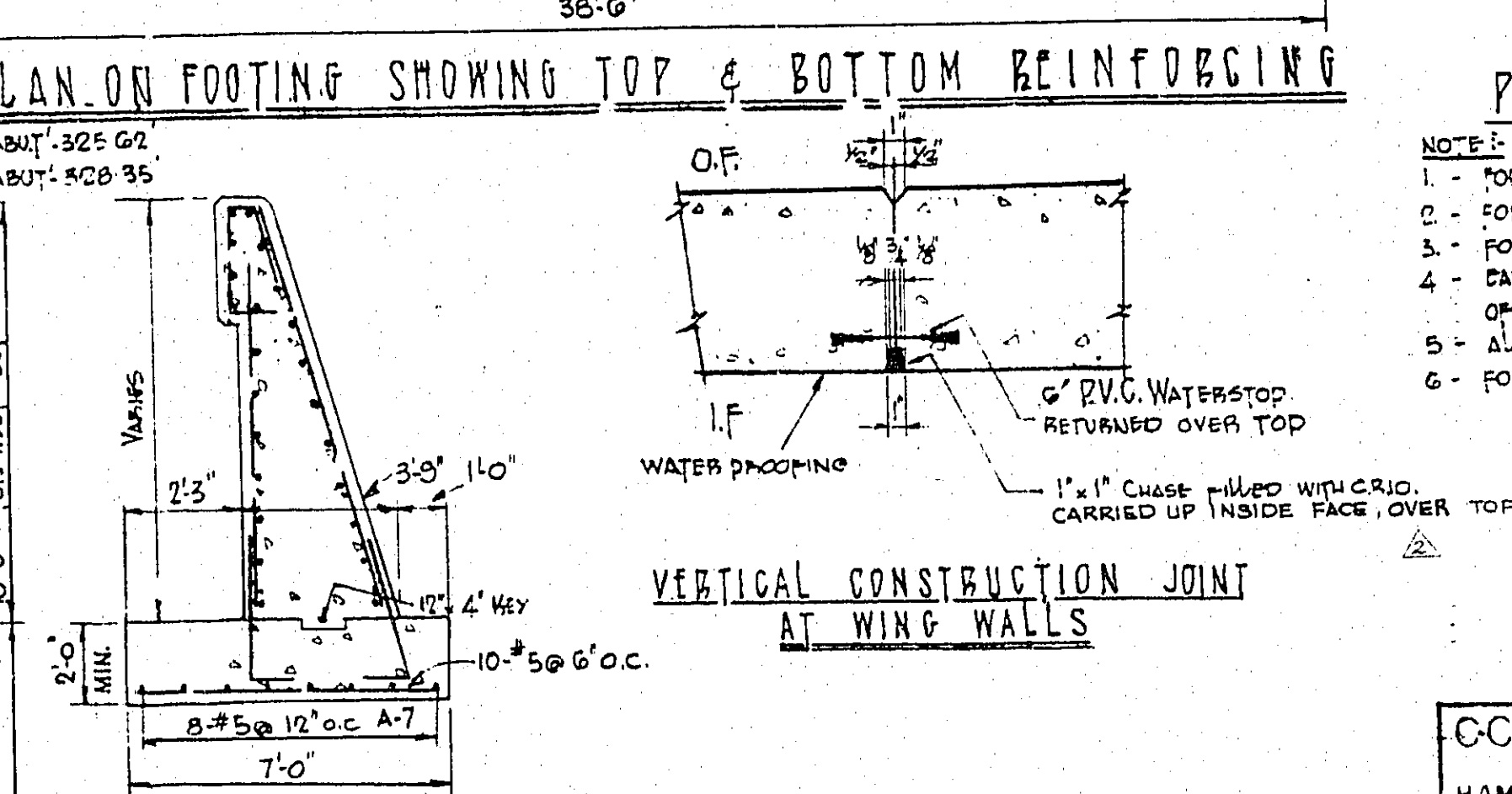


SECTION BB
WING WALL REINFORCING [TYPICAL]

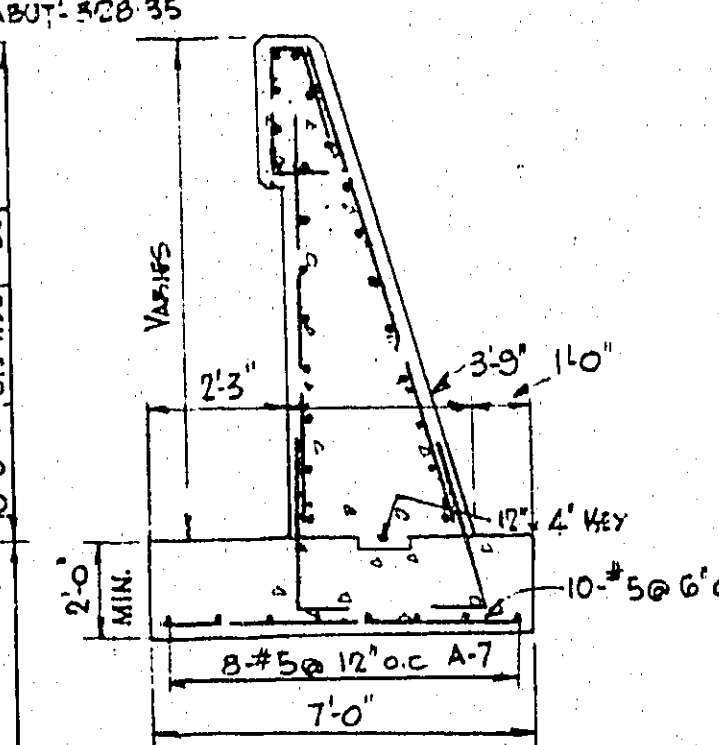
SCALE 1/4" = 1'-0"



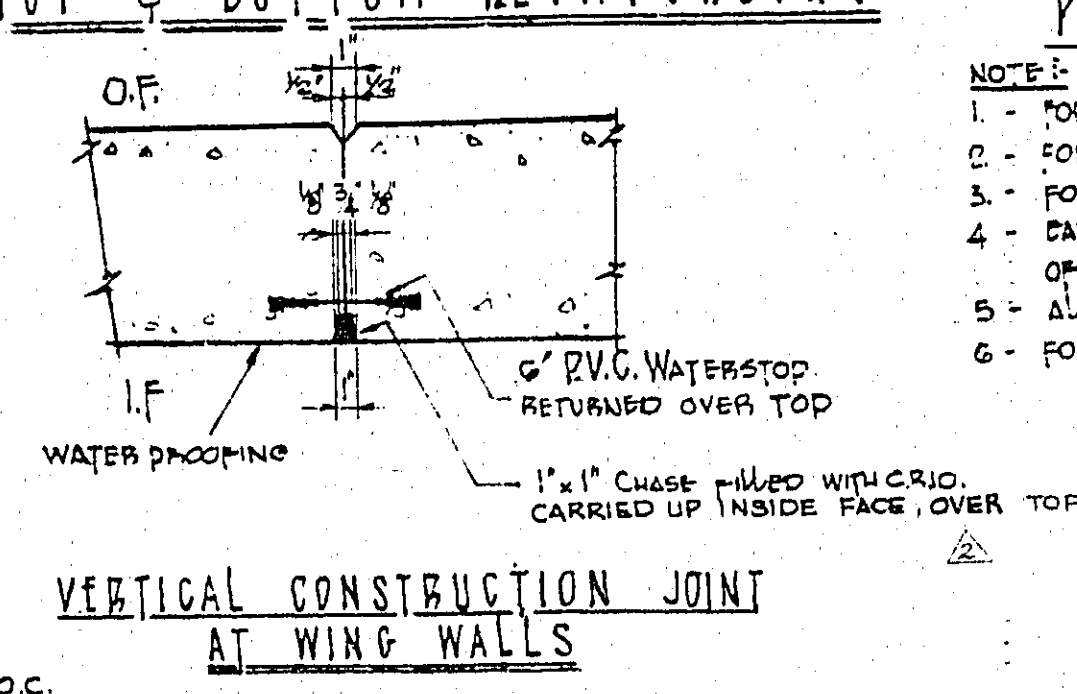
OUTSIDE FACE



PLAN ON FOOTING SHOWING TOP & BOTTOM REINFORCING



SECTION C-C



VERTICAL CONSTRUCTION JOINT AT WING WALLS

NO.	FOR	DATE
1	C.A. C.	2-23-88
2	REVISION	6-13-91
3	REVISION	12-1-92

REVISION	DATE	BY	DESCRIPTION
1	10/1/88	J.C.G.	PLAN ON EAST & WEST ABUTMENT REVISED AS-CONSTRUCTED
2	2/2/92	J.C.G.	NOTE REGARDING CHASE REVISED
3	10/9/91	J.C.G.	ADD REV.

W. 00-60.

CC-PARKER & PARSONS, BRINCKERHOFF LIMITED
HAMILTON CONSULTING ENGINEERS ONTARIO

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

SUBWAY AT C.N. RAILWAY

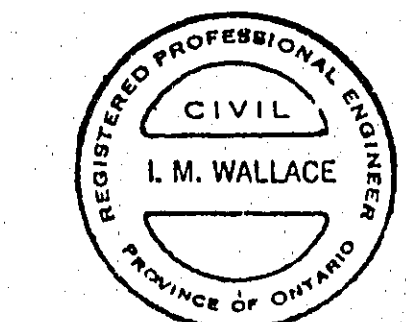
THE KING'S HIGHWAY No. 403 DIST. No. 4
CO. OF WENTWORTH CHEDOKE BRIDGE No. B
TWP. WEST FLAMBOROUGH LOT 28 CON. 1

ABUTMENTS - DETAILS & REINFORCING.

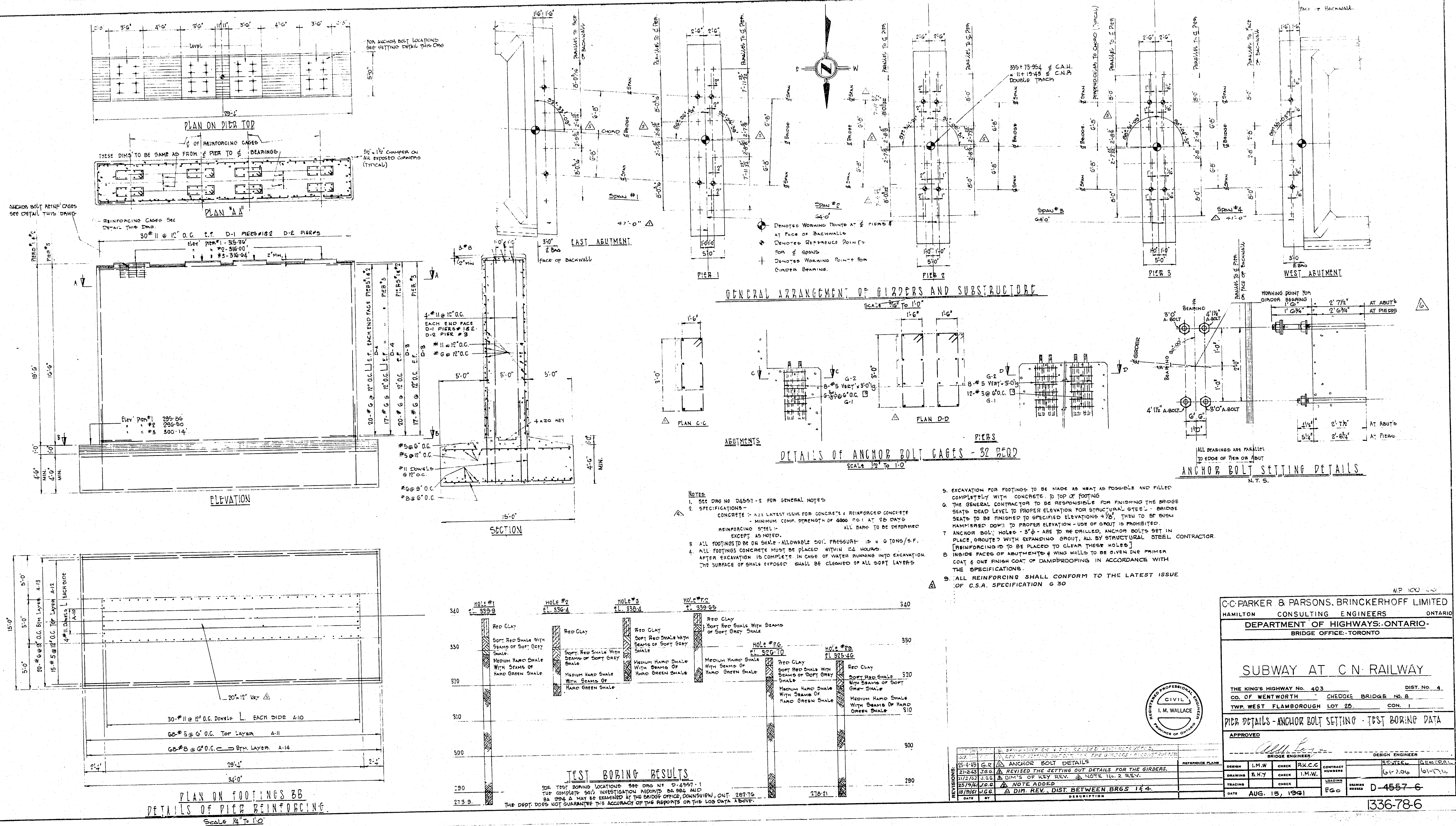
APPROVED: *[Signature]* BRIDGE ENGINEER

DESIGN ENGINEER: *[Signature]*

DESIGN	I.M.W.	CHECK	A.H.C.C.	CONTRACT	NO.	61-174
DRAWING	A.H.V.	CHECK	I.M.W.	LOADING		
TRACING		CHECK				
DATE	AUG. 15, 1991			EGG	DRAWING	D-4557-5



1336-78-5



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

DIST. No.
CONT. No.2019-2013
WP. No.2440-13-00



HWY 403 - CNR SUBWAY
STRUCTURE REHABILITATION
GENERAL ARRANGEMENT

SHEET
21

MH MORRISON HERSHFIELD

EXTENT OF REHABILITATION WORK

THE GENERAL EXTENT OF THE REHABILITATION WORK OUTLINED BELOW AND DESIGNATED ON THIS DRAWING SHALL BE CONSIDERED SIMILAR ON EACH SIDE OF THE BRIDGE UNLESS NOTED OTHERWISE.

REMOVALS:

- WORK IS NOT LIMITED TO ITEMS LISTED BELOW. OTHERS AS SHOWN AND/OR SPECIFIED IN CONTRACT.
- NO INTENTIONAL ORDER OF SEQUENCE OF ITEMS LISTED BELOW.
- DESIGN BASED ON ORIGINAL DRAWINGS. SEE CONSTRUCTION NOTES ALSO.

- REMOVE EXISTING STEEL RAILING.
- BLAST CLEAN EXISTING BEARINGS AT PIERS AND ABUTMENTS.
- REMOVE DETERIORATED CONCRETE IN ABUTMENTS, WINGWALLS AND PIERS LOCALLY AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
- REMOVE DETERIORATED CONCRETE IN EXISTING CURB, FASCIA AND SIDEWALK LOCALLY AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
- REMOVE DETERIORATED CONCRETE IN DECK SOFFIT LOCALLY AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
- REMOVE EXISTING COATING ON STRUCTURAL STEEL.
- REMOVE EXISTING CONCRETE SLOPE PAVING AT EAST ABUTMENT.

NEW CONSTRUCTION:

- WORK IS NOT LIMITED TO ITEMS LISTED BELOW. OTHERS AS SHOWN AND/OR SPECIFIED IN CONTRACT.
- NO INTENTIONAL ORDER OF SEQUENCE OF ITEMS LISTED BELOW.
- DESIGN BASED ON ORIGINAL DRAWINGS. SEE CONSTRUCTION NOTES ALSO.

- INSTALL NEW STEEL RAILING.
- REPAIR/RECONDITION EXISTING BEARINGS AT PIERS AND ABUTMENTS.
- PATCH REPAIR ABUTMENTS, WINGWALLS AND PIERS LOCALLY.
- PATCH REPAIR CURB, FASCIA AND SIDEWALK LOCALLY.
- PATCH REPAIR DECK SOFFIT LOCALLY.
- RE-COAT EXISTING STRUCTURAL STEEL.
- GENERAL REPAIR OF STRUCTURAL STEEL.
- REPAIR IMPACT DAMAGE TO EXISTING GRIDER AND CROSS-BRACING.
- REPAIR AND MODIFY EXISTING EXPANSION JOINT AT PIERS AND ABUTMENTS.
- INSTALL NEW CONCRETE SLOPE PAVING AT EAST ABUTMENT.
- INSTALL NEW DECK DRAINS AT PIERS.
- INSTALL NEW DRAINAGE OUTLETS AT WEST ABUTMENT.

GENERAL NOTES:

- CLASS OF CONCRETE:
ALL ----- 30 MPa
- CLEAR COVER TO REINFORCING STEEL:
DECK --- TOP ----- 70 +/- 20 mm
 BOTTOM ----- 40 +/- 10 mm
REMAINDER ----- 70 +/- 20 mm
UNLESS NOTED OTHERWISE.
- REINFORCING STEEL:
 - REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
 - STAINLESS REINFORCING STEEL SHALL BE TYPE 316 LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa UNLESS OTHERWISE SPECIFIED.
 - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
 - UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
 - BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND ALL DETAILS ON SITE AGAINST THE PROPOSED WORK AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
- THE CONTRACTOR SHALL REFER TO THE REFERENCE DRAWINGS (AS LISTED IN CONTRACT DOCUMENTS) FOR DETAILS OF THE EXISTING STRUCTURE.
- THE CONTRACTOR SHALL CHECK AND IDENTIFY ALL EXISTING UTILITIES WITHIN THE WORK AREA. PRIOR TO THE CONSTRUCTION WORK, THE CONTRACTOR SHALL CARRY OUT ALL NECESSARY PROTECTION AND PRECAUTIONARY MEASURES FOR OR ARRANGE TO DIVERT EXISTING UTILITIES AS MAY BE REQUIRED BY RELEVANT AUTHORITIES.

LEGEND:

T/P - TOP OF PAVEMENT
T/C - TOP OF CONCRETE
T/F - TOP OF FOOTING
WP - WORKING POINT
ALT - ALTERNATE
I.F. - INSIDE FACE
O.F. - OUTSIDE FACE
E.F. - EACH FACE
C.L. - CENTRE LINE
C.J. - CONSTRUCTION JOINT
SBGR - STEEL BEAM GUIDE RAIL
S.S. - STAINLESS STEEL
U.N.O. - UNLESS NOTED OTHERWISE
N.T.S. - NOT TO SCALE

REMOVAL

NEW CONCRETE

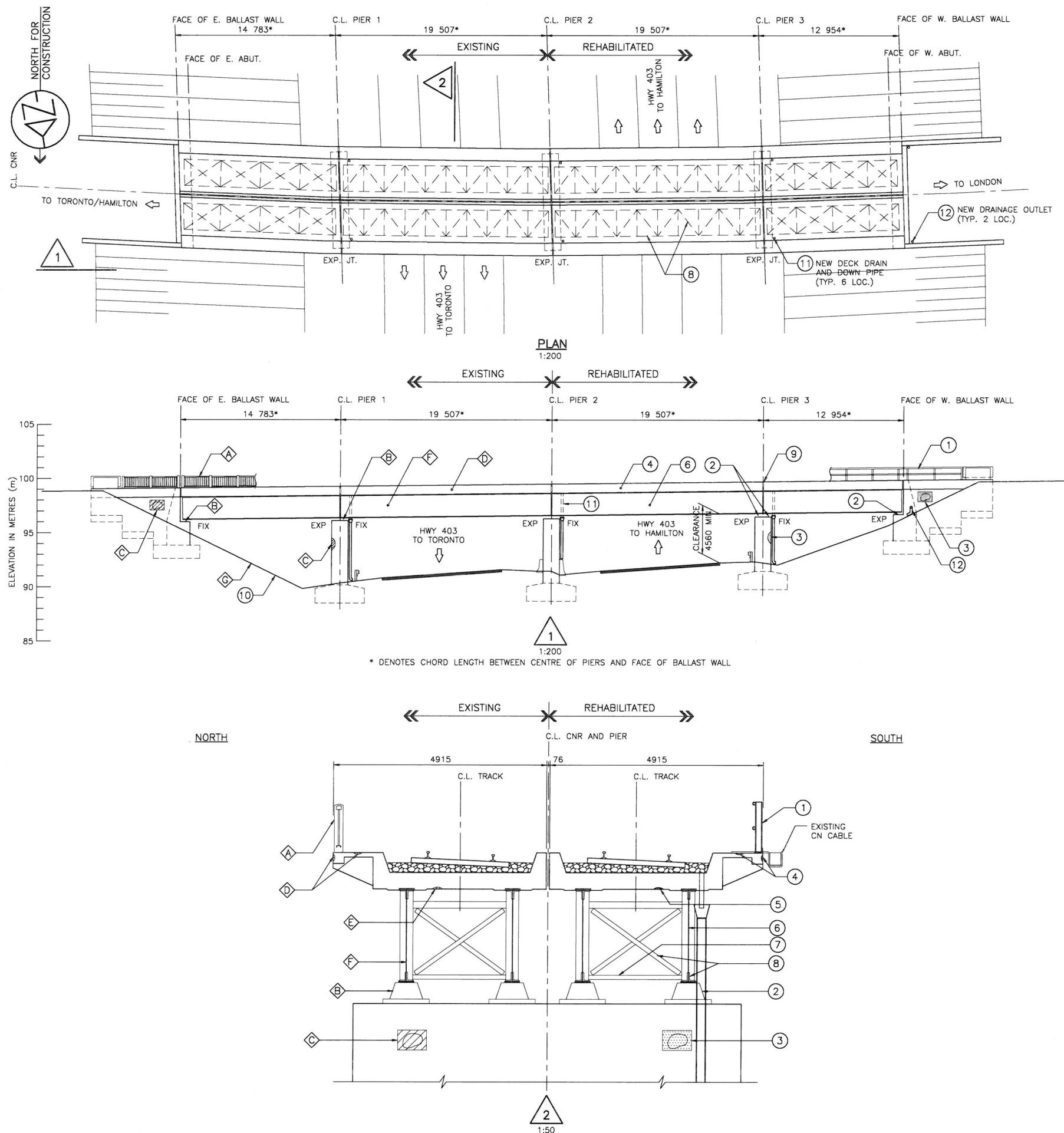
LIST OF DRAWING:

- GENERAL ARRANGEMENT
- GENERAL REPAIR
- STRUCTURAL STEEL REPAIR - 1
- STRUCTURAL STEEL REPAIR - 2
- STEEL RAILING
- DETAIL OF CONCRETE SLOPE PAVING
- DRAINAGE DETAIL
- STANDARD DETAILS



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DATE	BY	DESCRIPTION
DESIGN	ST	CHK. JL	CODE CHBDC 2006	LOAD CL-625-ONT
DRAWN	TLC	CHK. ST	SITE 36-31	DWG. 1





Appendix C

Selected Site Photographs



Photo 1- CNR Subway, north side
Photo taken from Highway 403 WBL on December 1, 2021



Photo 2- CNR Subway, north side
Photo taken from Highway 403 WBL on December 1, 2021



Photo 3- CNR Subway, north side

West Abutment and Piers
Photo taken from Highway 403 WBL on December 1, 2021



Photo 4- CNR Subway
West Abutment, north side
Photo taken from Highway 403 WBL on December 20, 2021



Photo 5- CNR Subway
West Abutment, north side
Photo taken from Highway 403 WBL on December 20, 2021



Photo 6- CNR Subway
West abutment, south side
Photo taken from Highway 403 WBL on December 20, 2021



Photo 7- CNR Subway
West abutment, south side
Photo taken from Highway 403 WBL on December 20, 2021



Photo 8- CNR Subway
East abutment, south side
Photo taken from Highway 403 EBL on December 20, 2021



Photo 9- CNR Subway
East abutment, south side
Photo taken from Highway 403 EBL on December 20, 2021



Photo 10- CNR Subway
East abutment, north side
Photo taken from Highway 403 EBL on December 20, 2021

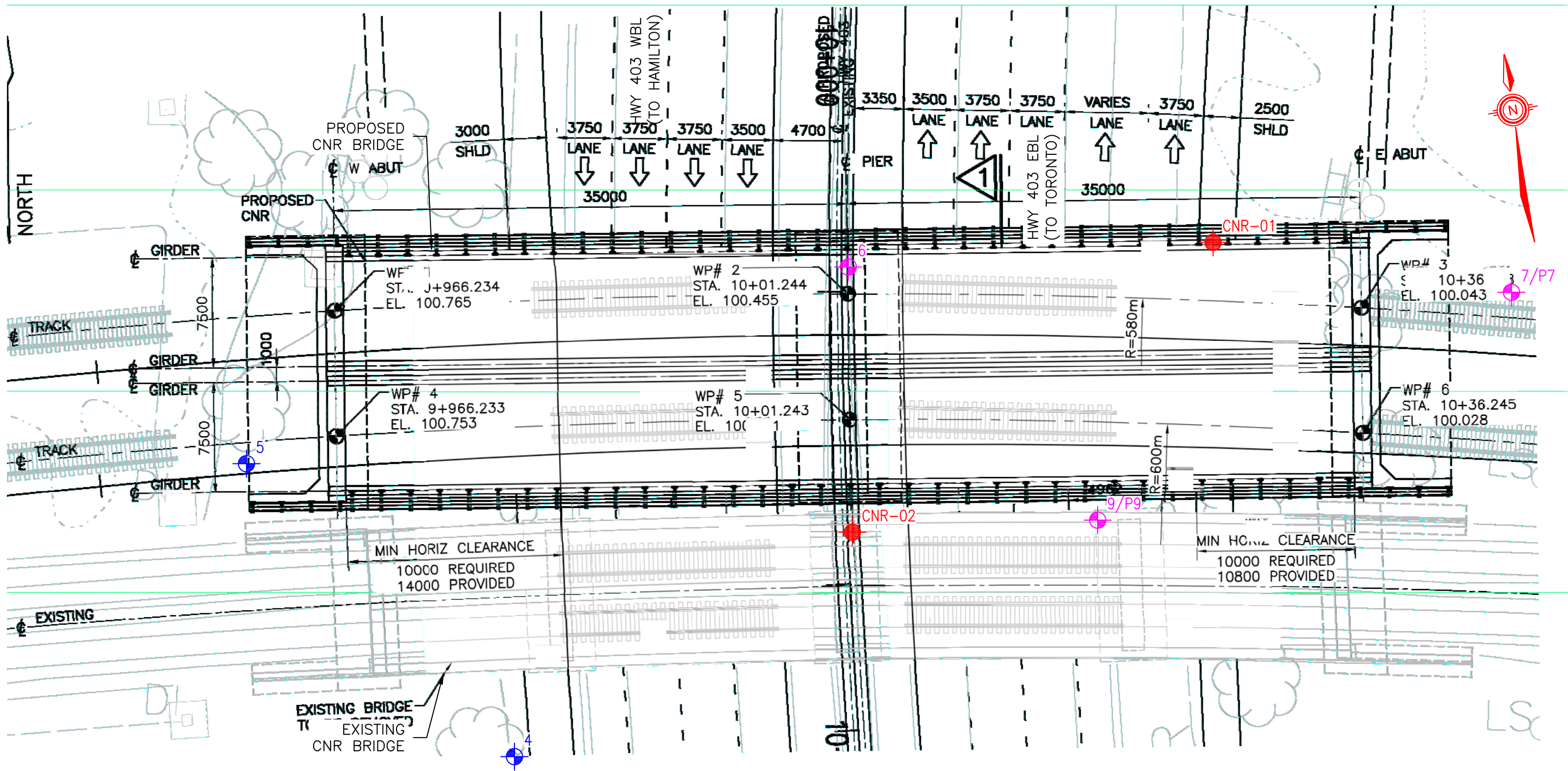


Photo 11- CNR Subway
East abutment, north side
Photo taken from Highway 403 EBL on December 20, 2021



Appendix D

Plan of Proposed Boreholes



PLAN

HIGHWAY 403 CNR SUBWAY PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT PROPOSED BOREHOLE LOCATIONS (N.T.S. SCHEMATIC ONLY)

- PROPOSED BOREHOLES
- APPROX. BOREHOLE LOCATIONS (PREVIOUS INVESTIGATION)
PREVIOUS INVESTIGATION (BY ONTARIO DEPARTMENT OF HWY, MAY 1960)
- APPROX. BOREHOLE & DUTCH CONES (DYNAMIC CONE PENETRATION TEST) LOCATIONS
PREVIOUS INVESTIGATION (BY E.M. PETO ASSOCIATES LTD., JANUARY 1960)