



THURBER ENGINEERING LTD.

**FOUNDATION DESKTOP STUDY REPORT
PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT
HIGHWAY 403 AT CPR SUBWAY
STRUCTURE REPLACEMENT ON NEW ALIGNMENT
HAMILTON, ONTARIO
W.O. #16-20004
SITE 36-32**

GEOCRES NO. 30M5-342

**Latitude: 43.287950°
Longitude: -79.897126°**

Report

to

WSP

Date: November 9, 2022
File: 25963



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**FOUNDATION DESKTOP STUDY REPORT
PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT
HIGHWAY 403 AT CPR SUBWAY
STRUCTURE REPLACEMENT ON NEW ALIGNMENT
HIGHWAY 403 AND HIGHWAY 6 INTERCHANGE
HAMILTON, ONTARIO
W.O. #16-20004
SITE 36-32**

GEOCRES NO. 30M5-342

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 CPR 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 CPR 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.

Client: WSP
File No. 25963

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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, CPR Subway, Hwy 403, Site # 36-32, prepared by Morrison Hershfield, Report No. 1130336-CS-03, dated January 2014. (Reference 3).
- CPR Subway Bridge over Hwy 403, Site No. 36-32, Structural Steel Inspection Report, prepared for Morrison Hershfield Ltd., prepared by Domson Engineering & Inspection Ltd., Report No. 27731-36-32, dated December 20, 2013. (Reference 4).
- Ontario Bridge Management System (OBMS), Ontario Structure Inspection Manual – Inspection Form, CPR Subway Hwy 403, Site number 36-32, 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 CP Railway, District 4, dated August 15, 1961. (Reference 6).
 - General Arrangement
 - Girder Bridge Details, Deck Expansion Joint
 - Abutments – Details and Reinforcing
 - Pier Details – Anchor bolt setting, Boring Log
- Archive drawings, Highway 403, CPR Subway, Structure Rehabilitation, General Arrangement, Site 36-32, Cont. No. 2019-2013, W.P. 2440-13-00, Sheet 29, 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 CPR track, approximately 750 m southwest of the Highway 403 and Highway 6 interchange in Hamilton, Ontario. The subway bridge carries a single rail track over the Highway 403 EBL (eastbound lanes) and WBL (westbound lanes).



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, the Canadian National Railway (CNR) subway structure is located approximately 30 m north and parallel to the CPR structure.

The existing CPR 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 22.1 m, 21.95 m, 21.95 m, and 22.1 m, resulting in a total curvilinear length of approximately 88.1 m. The bridge deck width is approximately 6.4 m. The two middle spans extend over Highway 403, and the end spans extend over the approaches. It is estimated, from archive drawings, that the existing west and east approaches are in the order of 13.5 m in height.

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 and modification of expansion joint at piers and abutments, installation of new concrete slope paving at both abutments, and installation of new deck drain downpipe.

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 south 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 88 at the West Abutment, at Elevation 87.5 at the Pier and, at Elevation 88.5 at the East Abutment. The structure will carry one rail track and will have a span length of 68 m (34 m + 34 m) between abutment bearings and a deck width of 7.6 m. The Highway 403 grade will slope from west to east, from about Elevations 91 to 88.5, at the proposed structure. The new track grade at the approaches will range from approximate Elevations 104 to 102.5, resulting in approach embankments up to order of 13 m to 14 m in height. A 6 m long retaining wall is proposed on the south side of the west abutment, perpendicular to the rail track alignment. An RSS retaining wall



is proposed on the south side of the east abutment. The wall will be parallel to the rail track and will be up to about 10 m in height.

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 CPR subway structure have been noted during our site visit:

- There was no visible sign of settlement or distress along the bridge alignment.
- The existing approach embankments are fully covered with heavy vegetation including tall grass, and appeared to be in good 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 embankment.
- Seepage was noticed at the three rectangular piers, which are wet stained along the concrete faces. Several narrow vertical and diagonal cracks were also noted on the piers.
- Wet/seepage stains were noted on 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 and west abutments. Occasional seepage stains were noted on the concrete slope paving. Pipe drains were also observed at the top of the forward slope.
- 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 to cover 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 four (4) boreholes (numbered 1, 2, 3 - from Reference 2, and numbered 2 - from Reference 1) were located in the vicinities of the CPR track crossing of Highway 403 (formerly Chedoke Expressway). A Standard Dutch Cone (Dynamic Cone Penetration Test - DCPT) was conducted in proximity of Borehole 2 from Reference 1. The Standard Dutch Cone was numbered P2. 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 1, 2, 3 (Reference 2) and 2, Standard Dutch Cone P2 (Reference 1), 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 clay underlain by shale bedrock. The clay was described as red to grey-brown in colour, and sandy with pebbles. Pockets of sand were noted in the clay layer. A 200mm thick layer of cinder was encountered surficially in Borehole 2 (Reference 1). The thickness of the clay varied from 1.5 m to 1.8 m. An SPT 'N' value measured in the clay was 17 blows per 0.3 m of penetration indicating a very stiff consistency.

Shale bedrock of the Queenston Formation was contacted below the clay. The shale was red in colour with layers of hard, green shale bedrock. Clay seams about 25 mm to 75 mm thick 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 varied from 58 to 200 blows per 0.3 m of penetration. Total Core Recovery (TCR) ranged from 88 percent to 100 percent. The Standard Dutch Cone P2 was terminated at 3.6 m depth.

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	3 ⁽⁴⁾ (Reference 2)	1.5	101.6	3.0	100.1
Pier 3	2 ⁽⁴⁾ (Reference 2)	1.8	100.7	2.9	99.6
Pier 2	1 ⁽⁴⁾ (Reference 2)	1.8	101.8	3.4	100.2
Pier 1, East Abutment	2 ⁽⁴⁾ / P2 (Reference 1)	1.5	102.0	3.0	100.5

- (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) Bedrock proved by coring.

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 0.9 m to 1.5 m depth (Elevations 102.2 and 101.0) during the previous investigations. The surface topography has since been altered and it is anticipated that the 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 CPR 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

Approximate Location Relative to Bridge	Borehole	Design Underside Elevation of Footing	Estimated Founding Stratum
West Abutment	3 (Reference 2)	97.0	Sound shale bedrock
Pier 3	2 (Reference 2)	87.3	Sound shale bedrock
Pier 2	1 (Reference 2)	86.9	Sound shale bedrock
Pier 1	2 / P2	86.5	Sound shale bedrock
East Abutment	(Reference 1)	95.3	Sound shale bedrock

References 1 and 2 recommended that spread footings founded on shale bedrock be designed using a “conservative” allowable bearing pressure 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 a design bearing pressure of 645 kPa (6 tsf) is indicated for shale bedrock.

Reference 6 indicated that the pier footings are about 7.9 m (26 ft.) long and 6.7 m (22 ft.) wide, and that the abutment strip footings are about 5.8 m (19 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 south of the existing bridge, with new foundation elements at different locations. Since bridge replacement and new foundations are involved and given the uncertainties regarding the information on the archive boreholes, it is recommended that a new borehole be advanced at a selected location (Section 14.0 below) in order to obtain adequate information for preliminary 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 walls of the new bridge be of the concrete cantilever type, and the abutment and pier footings be founded on sound shale bedrock.

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

7.0 RETAINING WALLS

A retaining wall is proposed on the south side of the west abutment, perpendicular to the CP rail track. The wall will be 6 m long and approximately 5.5 m high. It is anticipated that for any selected wall option, shale bedrock would likely be the founding stratum.

It is understood that RSS wall system is proposed on the south side of the east abutment, parallel to the CP track. The RSS wall will be about 10 m high. According to the preliminary GA drawing, the base of the wall will be founded near Elevation 92 within shale bedrock. The proprietary RSS system must meet the MTO's specifications for performance and appearance.



For retaining walls founded on shale bedrock, preliminary geotechnical resistances should be similar to the assessed values presented in the previous Section 6.0 of this report.

During Phase 2 of the investigation, global stability of the overall embankment slope with the proposed retaining walls, and settlement analysis, must be carried out.

8.0 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 up to the order of 4 m to 5 m in height will be required in order to accommodate the new west abutment. The cuts will extend into the existing embankment 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 permanent drains 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 12.0 of this report.

9.0 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 in good condition.

Based on the preliminary GA drawing, placement of up to about 8 m of new fill (mostly backfill behind the new wall) will be required at the west approach. Behind the wall and its backfill, a grade raise of up to 2 m is anticipated. The overall embankment height will be up to the order of 13 m near the new bridge location. It is recommended that the permanent earth slope remains at an inclination of 2H : 1V or flatter to maintain global stability.



The subgrade for the new fill is expected to be the existing approach fills, native clay or shale. No global stability issues are anticipated for the fills 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, 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 25 mm if the new fill is placed and compacted as outlined above.

10.0 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 10.1. 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 10.1 – 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	-

11.0 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 embankment 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 may also be involved at the abutment and pier 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.

12.0 RAIL TRACK PROTECTION AND SHORING

During bridge replacement operations where excavations may be required in the vicinity of the existing rail line, track protection should be supplied and designed 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.

13.0 ADJACENT STRUCTURES AND BURIED UTILITIES

It is recommended that the exact locations of 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 utility 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 may be required.



14.0 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 approaches.

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 advancing 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 below the highway grade and slope surface. The borehole depth is therefore anticipated not to exceed the order of 6 m at the pier.

The proposed borehole location is 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.

15.0 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
1	RC1	0' - 6'	Very stiff red clay with seams of green clay.	-	-	-	-	-	-	Ran BX casing.
	RC2	6' - 11'	Soft red shale with seams of soft green shale.	-	-	-	-	-	-	Ran AXT core. 100% core recovery.
	RC3	11' - 16'	Medium Hard red shale with seams of hard green shale.	-	-	-	-	-	-	AXT core. 93% core recovery.
	RC4	16' - 21'	Medium hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
	RC5	21' - 26'	Medium hard red shale with seams of hard green shale. 1" clay seam at 23'5".							AXT core. 97% core recovery.
	RC6	26' - 31'	Medium hard red shale with seams of hard green shale. 1" clay seam at 28'6".							AXT core. 97% core recovery.
	RC7	31' - 36'	Medium hard red shale with seams of hard green shale. 2" mud seam at 34.5.							AXT core. 97% core recovery.
	RC8	36' - 41'	Medium hard red shale with seams of hard green shale.							AXT core. 97% core recovery.
	RC9	41' - 46'	Medium hard red shale with seams of hard green shale.							AXT core. 98% core recovery.
	RC10	46' - 51'	Medium hard red shale with seams of hard green shale.							AXT core. 98% 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
1	RC11	51' - 56'	Medium hard red shale with seams of hard green shale.							AXT core. 92% core recovery.
	RC12	56' - 61'	Medium hard red shale with seams of hard green shale. 3" clay seam at 60'5".							AXT core 98% core recovery.
	RC13	61' - 66'	Medium hard red shale with seams of hard green shale.							AXT core. 85% core recovery.
2	RC1	0' - 6'	Stiff red clay	-	-	-	-	-	-	-
	RC2	6' - 9.5'	Soft red shale with seams of soft green shale.							AXT core. 100% core recovery.
	RC3	9'6"-14'6"	Medium hard red shale with seams of hard green shale. 3" clay seam 9'6"-9'9".							AXT core. 97% core recovery.
	RC4	14'6"-19'6"	Medium Hard red shale with seams of hard green shale.							AXT core. 100% core recovery.
3	RC1	0'-5'	Stiff red clay							
	RC2	5'-10'	Soft red shale with seams of green shale.							AXT core. 92% core recovery.
	RC3	10'-15'	Medium hard red shale with seams of hard green shale.							AXT core. 95% core recovery.
	RC4	15'-17'10"	Medium hard red shale with seams of hard green shale.							AXT core. 94% core recovery.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 100-60 & 99-60 BORE HOLE NO 1

JOB 60-F-35 STATION See drawing

DATUM Geodetic 339.9 COMPILED BY G.G.C. & B.K.

FORING DATE April 29/60 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)	0
VANE TEST (C) AND SENSITIVITY (C)	15
NATURAL MOISTURE AND	
LIQUIDITY INDEX	20
LIQUID LIMIT	20
PLASTIC LIMIT	20

NO.	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE		CONSISTENCY		REMARKS
				P.S.F.	BLOW/FT.	MOIST. CONTENT - %	SHRINKAGE - %	
1	Groundlevel	339.9	0					
2	Stiff red clay with green clay seams.	333.9						
3	Seams of soft red & soft green shale	328.9	10					
	Medium hard red shale with seams of hard green shale.		20					
			30					
			40					
			50					
			60					
		273.9	70					
	End of borehole.		80					

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 100-60 & 99-60

BORE HOLE NO. 2

JOB 60-E-35

STATION See drawing

DATUM Geodetic 336.4

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

BORING DATE May 2/60.

CHECKED BY G.G.C.

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

2" SHELBY

CAS NO.

LEGEND


25% UNCONFINED COMPRESSION (Q_u)VANE TEST (G_v) AND SPINNING TEST

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE
	↓ Ground level	336.4	0	
	Stiff red clay	W.L. 331.4 330.4		
	Seams of soft red & green shale.	326.9	10	
	Medium hard red shale with seams of hard green shale.	316.9	20	
	End of borehole.		80	

UNDEVELOPED	NATURAL MOISTURE CONTENT	RECOVERY in %
		RC1 -
		RC2 100
		RC3 97
		RC4 100

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 100-60 & 99-60

BORE HOLE NO. 3

JOB 60-F-35

STATION See drawing

DATUM Geodetic 338.4

COMPILED BY C.R.C. & B.K.

BORING DATE May 3/60.

CHECKED BY G. G. C.

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

2" SHELBY

CASING

LEGEND

UNCONFINED COMPRESSION (Q)
VANE TESTS AND PENETRATION
NATURAL MOISTURE AND
LIQUID LIMIT
PLASTIC LIMIT

DEPTH FEET	DESCRIPTION	FEET FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	CONSIDERED	NATURAL MOISTURE CONTENT	RECOVERY in %
	↓ Groundlevel	338.4	0				
	Red Clay	WL 335.4					
	Soft red shale with seams of soft green shale.	333.4					
	Medium hard red shale with seams of hard green shale.	328.4	10				
	End of borehole.	320.5	20				
			30				
			40				
			50				
			60				
			70				
			80				

RC1

RC2

RC3

RC4

Recovery
in %

92

95

94

HOLE DISCONTINUED AT APPROXIMATELY 20 FT.

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

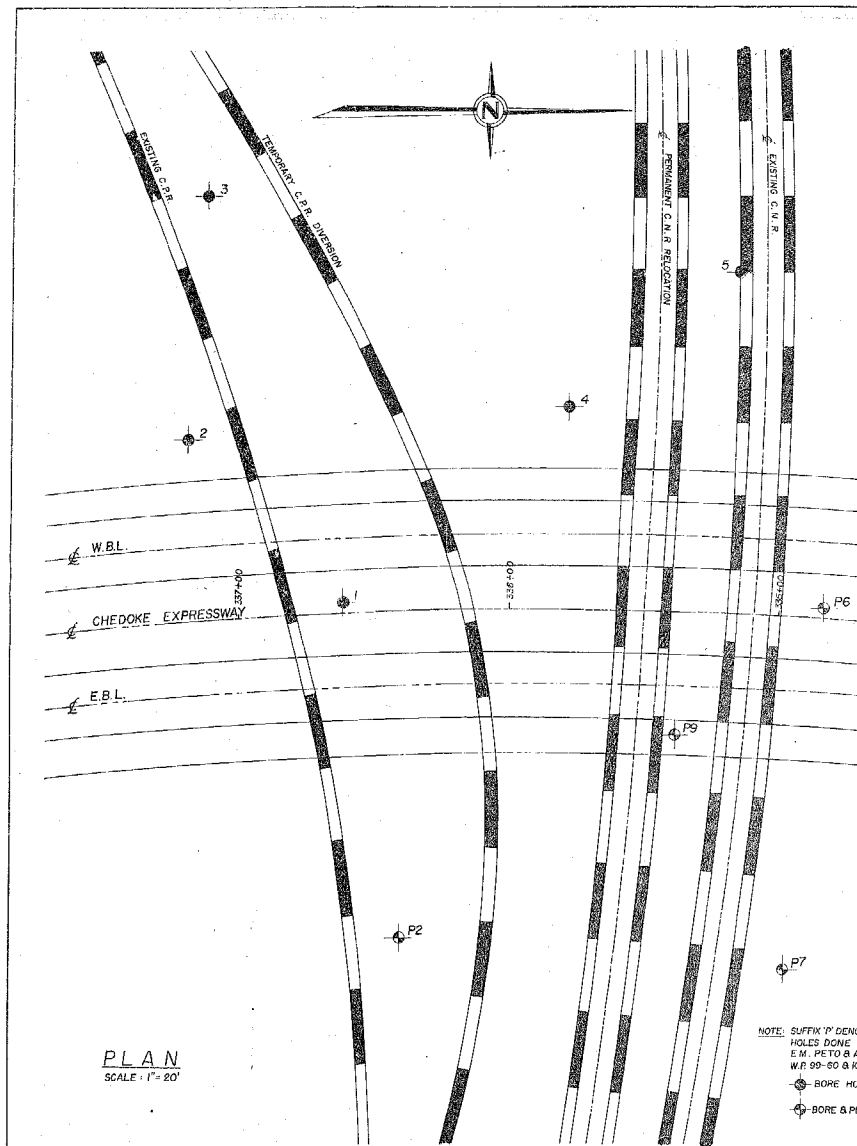
<u>Depth</u>	<u>Blows per foot</u>
0 - 1'	12
1 - 2'	6
2 - 3'	17
3 - 4'	26
4 - 5'	49
5 - 6'	60
6 - 7'	67
7 - 8'	130
8 - 9'	100
9 - 10'	133
10 - 11'	160
11' - 11'11"	240
11'11" - 12'	103

P-7

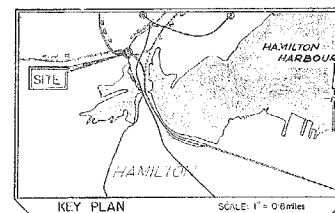
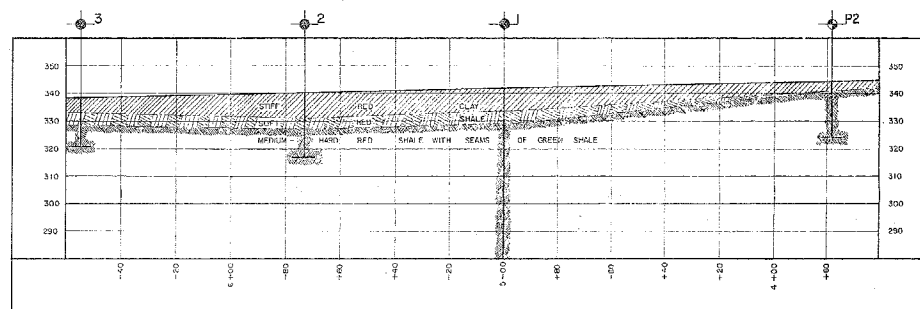
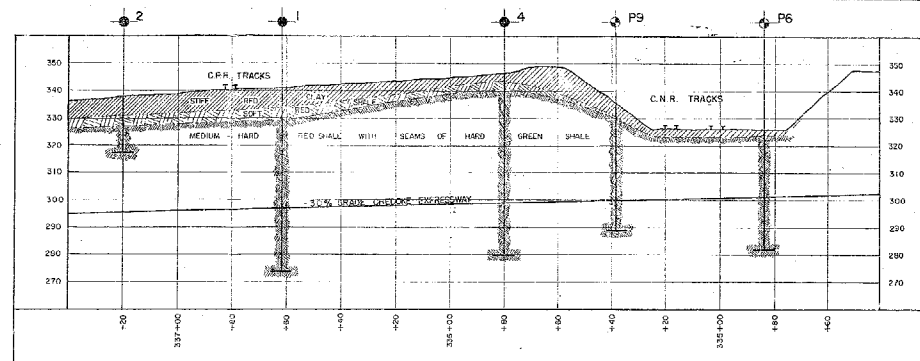
0 - 1'	1
1 - 2'	79
2' - 2'5"	100
2'5" - 2'5"	100

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



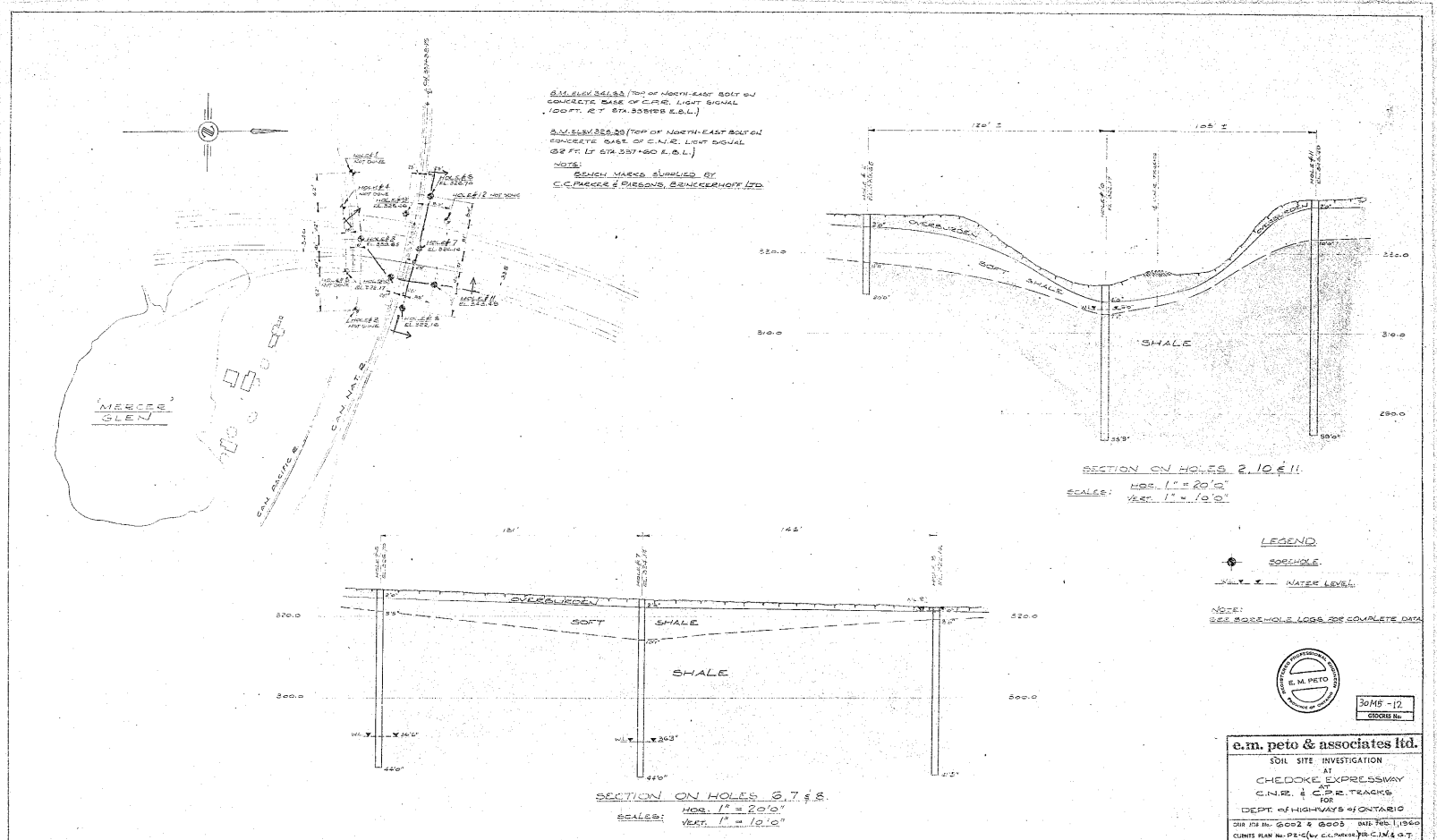
NOTE: SUFFIX "P" DENOTES
HOLES DONE BY
E.M. PETO & ASSOCIATES
W.P. 99-60 & 100-60
● BORE HOLE
● BORE & PENETRATION HOLE



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

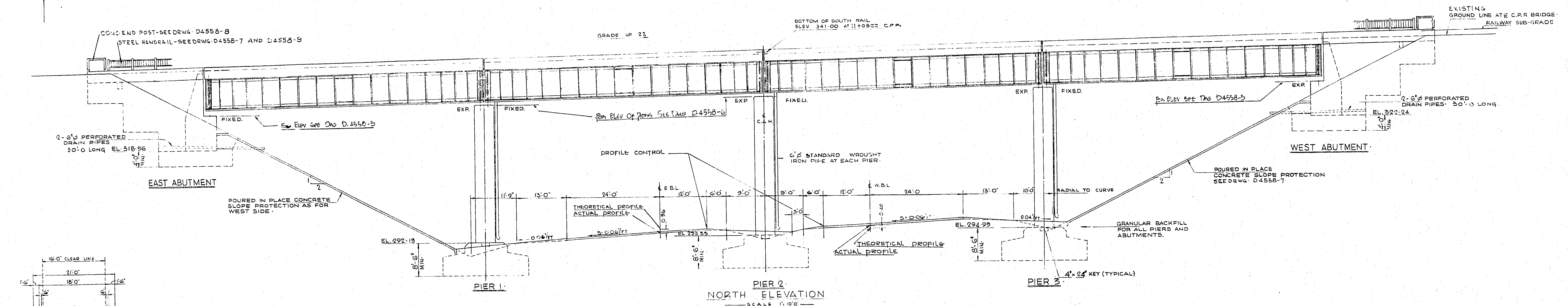
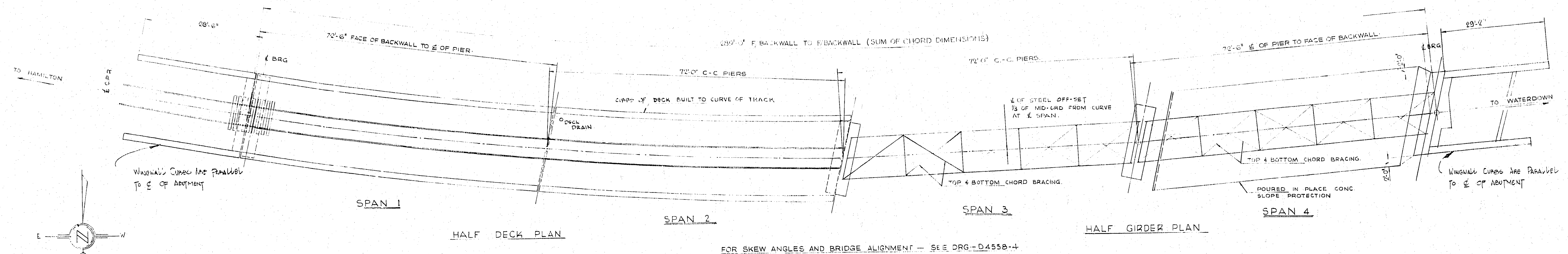
DEPARTMENT OF HIGHWAYS — ONTARIO			
MATERIALS & RESEARCH SECTION			
C.N.R. & C.P.R. CROSSINGS			
SHOWING POSITIONS & ELEVATIONS OF HOLES			
Highway 403	District 4	County WENTWORTH	
LOCATION HAMILTON			
Drawn by J. J. [Signature]	Checked by [Signature]	W.P. 99-60 & 100-60	
DATE 8 June 1960	APPROVED BY [Signature]	SCALE 1 inch = 20 feet	
			60-F-35A





Appendix B

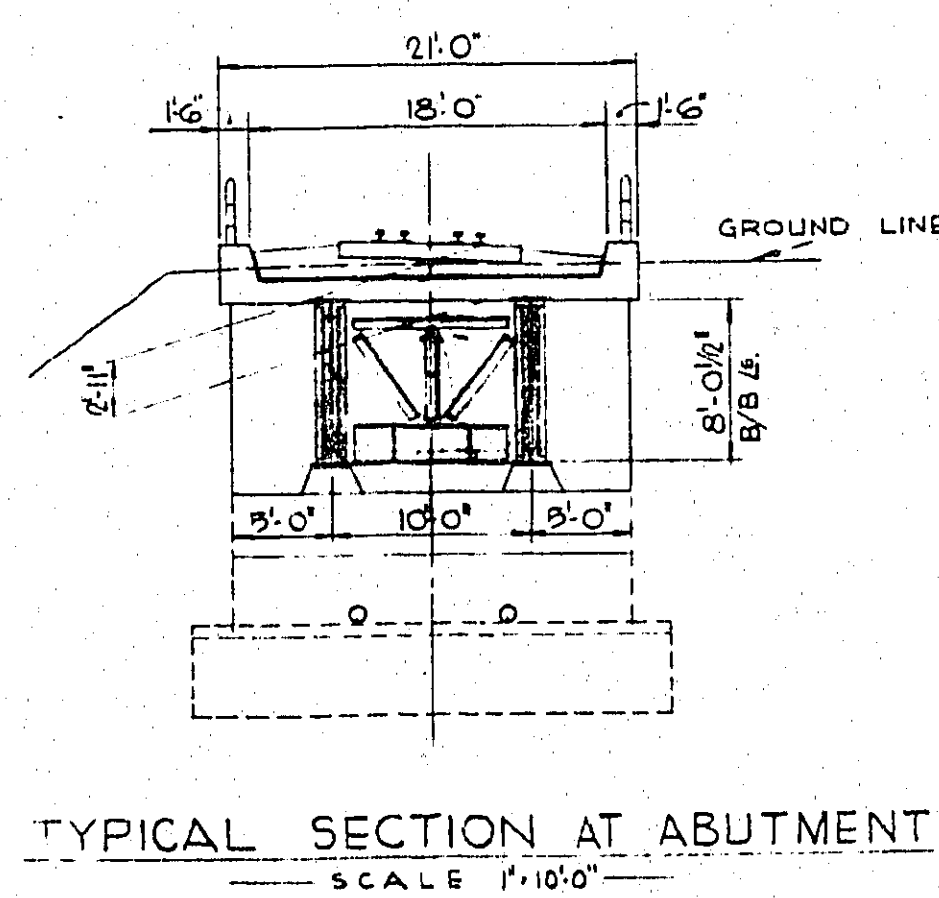
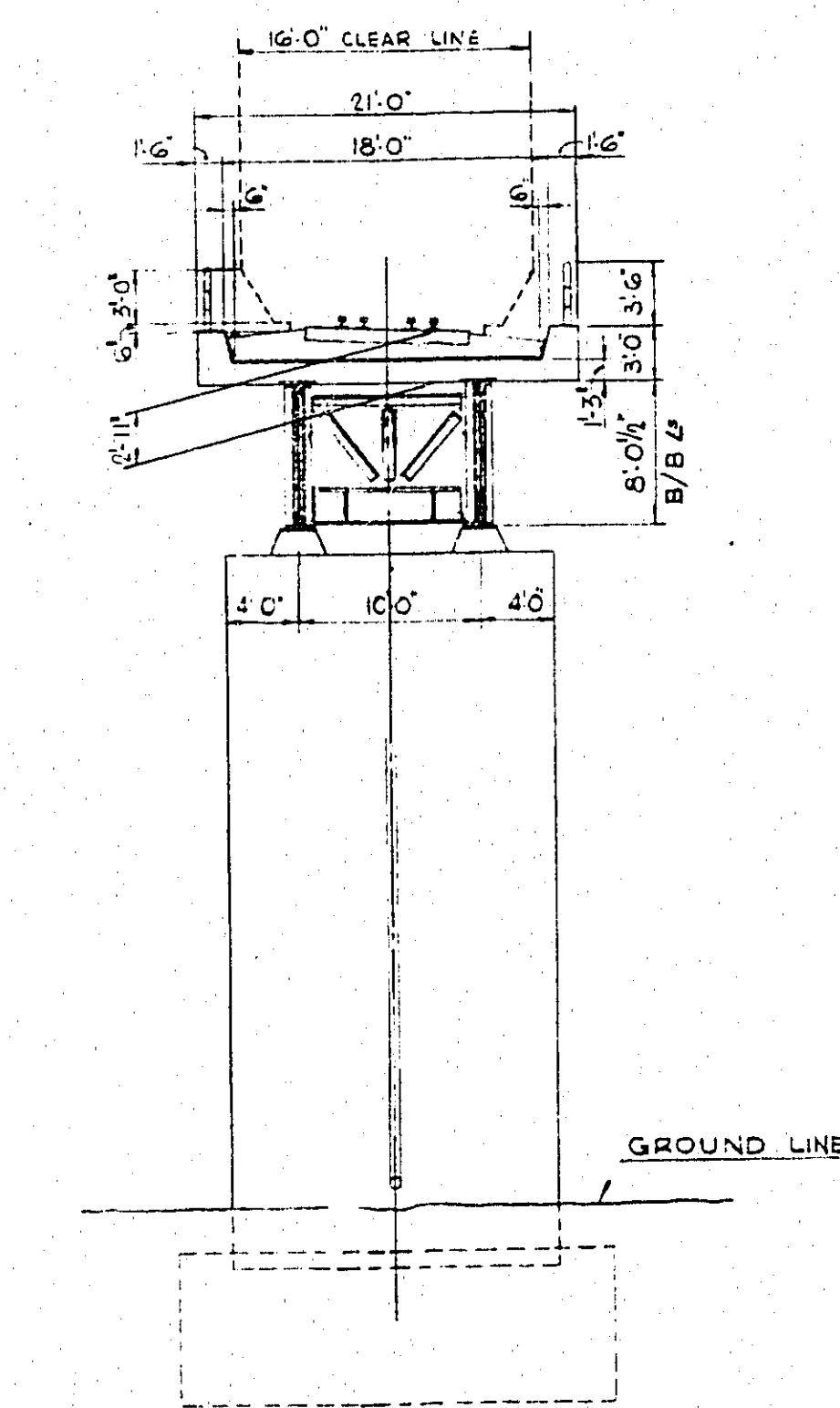
Archive Drawings of Existing Bridge (Construction and Rehabilitation)



GENERAL NOTES

- NOTE TO DISTRICT ENGINEER: CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DISTRICT ENGINEER.
- NOTE TO STRUCTURAL STEEL CONTRACTOR: FOR NOTES CONCERNING STRUCTURAL STEEL SEE DRG NO D-4558-3
- NOTE TO GENERAL CONTRACTOR: STRUCTURE TO BE BUILT IN ACCORDANCE WITH SPECIFICATION FOR STRUCTURES D.H.O. FORM NO 9 (LATEST REVISION) AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER.
- LOCATION: FOR LOCATION PLAN AND PROFILE SEE DRG NO D-4558-1.
- BORING DATA: THE COMPLETE SOIL INVESTIGATION REPORTS BA 994 AND BA 994 A MAY BE EXAMINED AT THE BRIDGE OFFICE, DOWNSVIEW, ONT. THE DEPARTMENT DOES NOT GUARANTEE THE ACCURACY OF THESE REPORTS OR THE BORING LOG DATA SHOWN ON THESE DRGS.
- CONCRETE MIX: ALL CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 4000 P.S.I. AT 28 DAYS. APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR MUST BE ADDED TO ALL CONCRETE AS SPECIFIED BY MATERIALS & RESEARCH SECTION D.H.O.
- CLEAR COVER ON REINFORCING STEEL:
 - ALL STEEL IN FOOTINGS-----3"
 - PIERS AND ABUTMENTS-----2"
 - DECK AND HANDRAIL POSTS-----1 1/2"
- CONSTRUCTION NOTES: ALL EXPOSED CONCRETE EDGES TO HAVE 1"x1" CHAMFER UNLESS OTHERWISE NOTED.
 - CONSTRUCTION JOINTS TO BE MADE ONLY WHERE LOCATED ON DRGS. UNLESS OTHERWISE APPROVED BY THE ENGINEER.
 - NO HORIZONTAL CONSTRUCTION JOINTS WILL BE PERMITTED IN PIERS AND ABUTMENTS.
 - NO CONCRETE SHALL BE PLACED BEFORE MATERIALS HAVE BEEN APPROVED AND A MIX ESTABLISHED TO THE SATISFACTION OF THE ENGINEER.
 - NO CONCRETE SHALL BE PLACED BEFORE FORMWORK, FALSEWORK, AND REINFORCING STEEL HAVE BEEN CHECKED AND APPROVED BY THE ENGINEER.
 - ALL REINFORCING BAR SPLICES TO BE LAPPED 35 DIAMETERS OF BAR (1.1"N.) UNLESS OTHERWISE NOTED.

DATE	BY	REVISION
1961	W.P.	1
1961	W.P.	2
1961	W.P.	3
1961	W.P.	4
1961	W.P.	5
1961	W.P.	6
1961	W.P.	7
1961	W.P.	8
1961	W.P.	9
1961	W.P.	10



WP 88-60

C.C. PARKER & PARSONS, BRINCKERHOFF LIMITED

HAMILTON CONSULTING ENGINEERS ONTARIO

DEPARTMENT OF HIGHWAYS-ONTARIO

BRIDGE OFFICE-TORONTO

SUBWAY AT C.P. RAILWAY

THE KING'S HIGHWAY No. 403 DIST. No. 4

CO. OF WENTWORTH CHEDOKE BRIDGE NO. 19

TWP. WEST FLAMBOURGH LOT 28 CON. 1

GENERAL ARRANGEMENT

APPROVED

BRIDGE ENGINEER

DESIGN ENGINEER

REVISIONS

DATE	BY	DESCRIPTION
1961/4/6	A.E.S.	REVISED AS-CONSTRUCTED.

PERFORMANCE PLANS

DESIGN	R.K.C.C.	CHECK	I.M.W.	CONTRACT
DRAWING <td>R.M.T.<td>CHECK<td>D.C.C.<td>NUMBERS</td></td></td></td>	R.M.T. <td>CHECK<td>D.C.C.<td>NUMBERS</td></td></td>	CHECK <td>D.C.C.<td>NUMBERS</td></td>	D.C.C. <td>NUMBERS</td>	NUMBERS
TRACING <td></td> <td>CHECK<td></td><td>LOADING</td></td>		CHECK <td></td> <td>LOADING</td>		LOADING

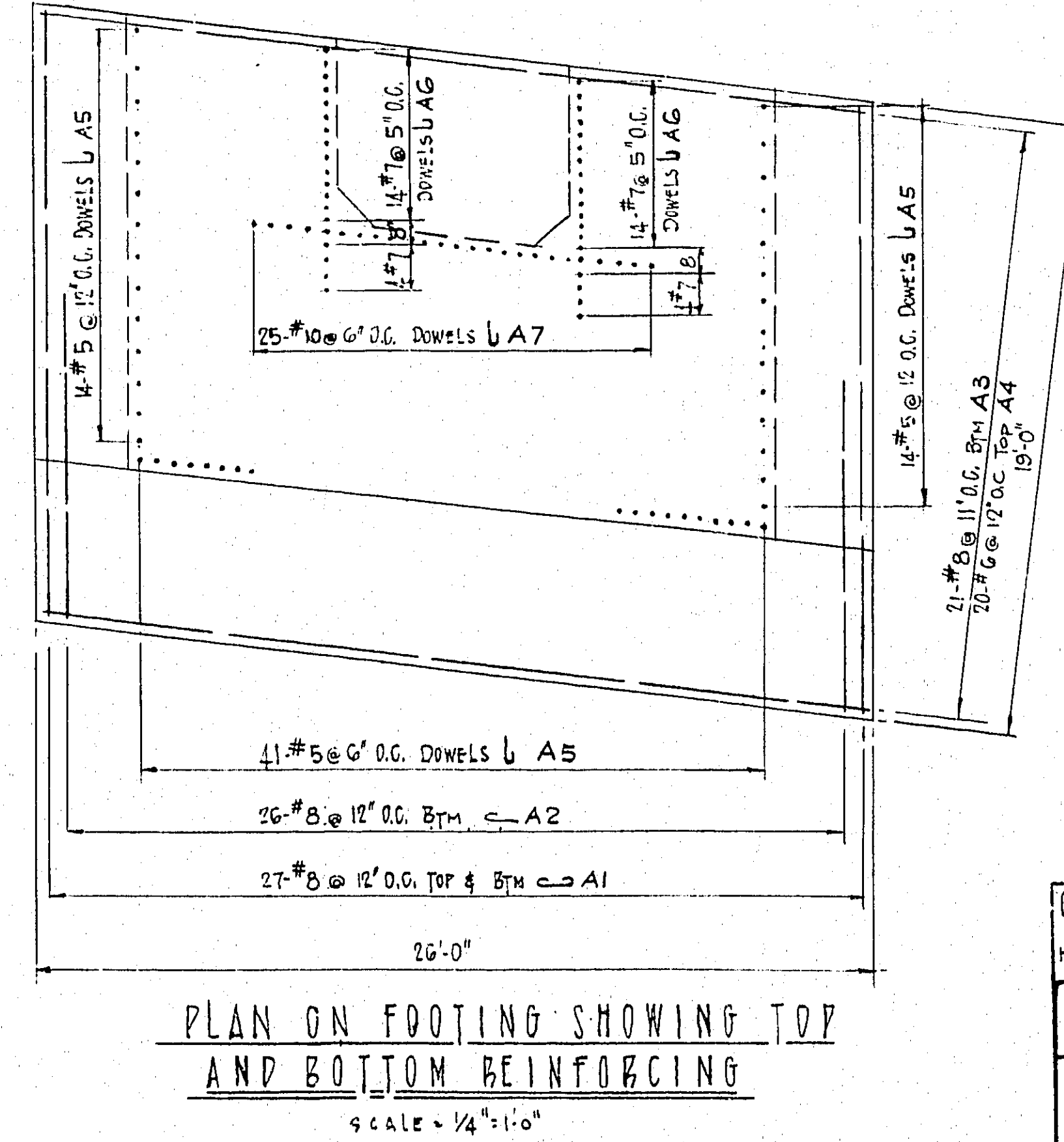
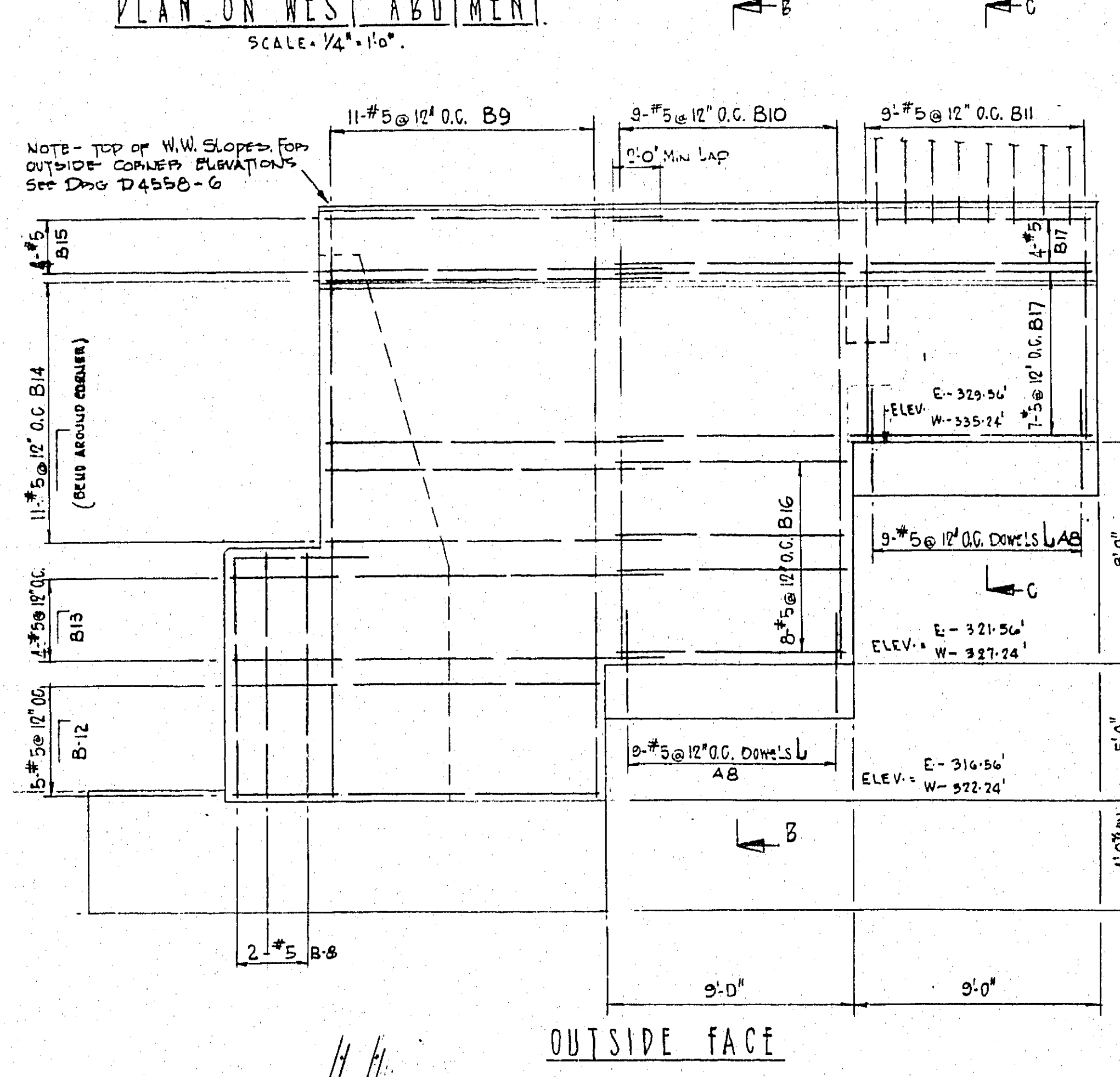
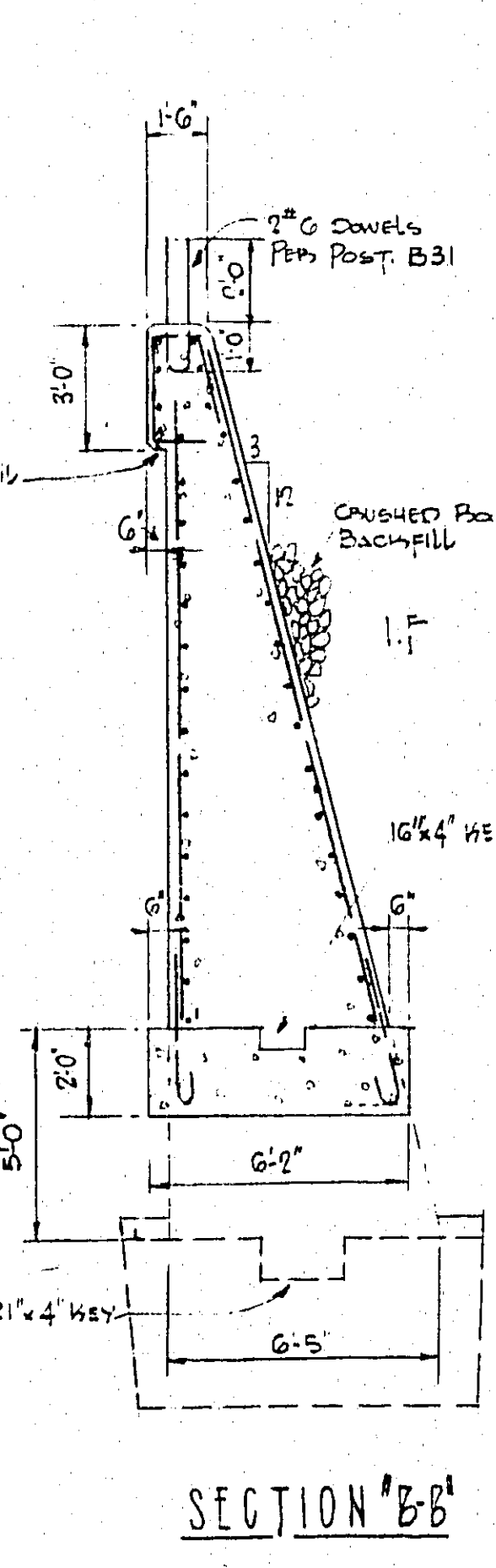
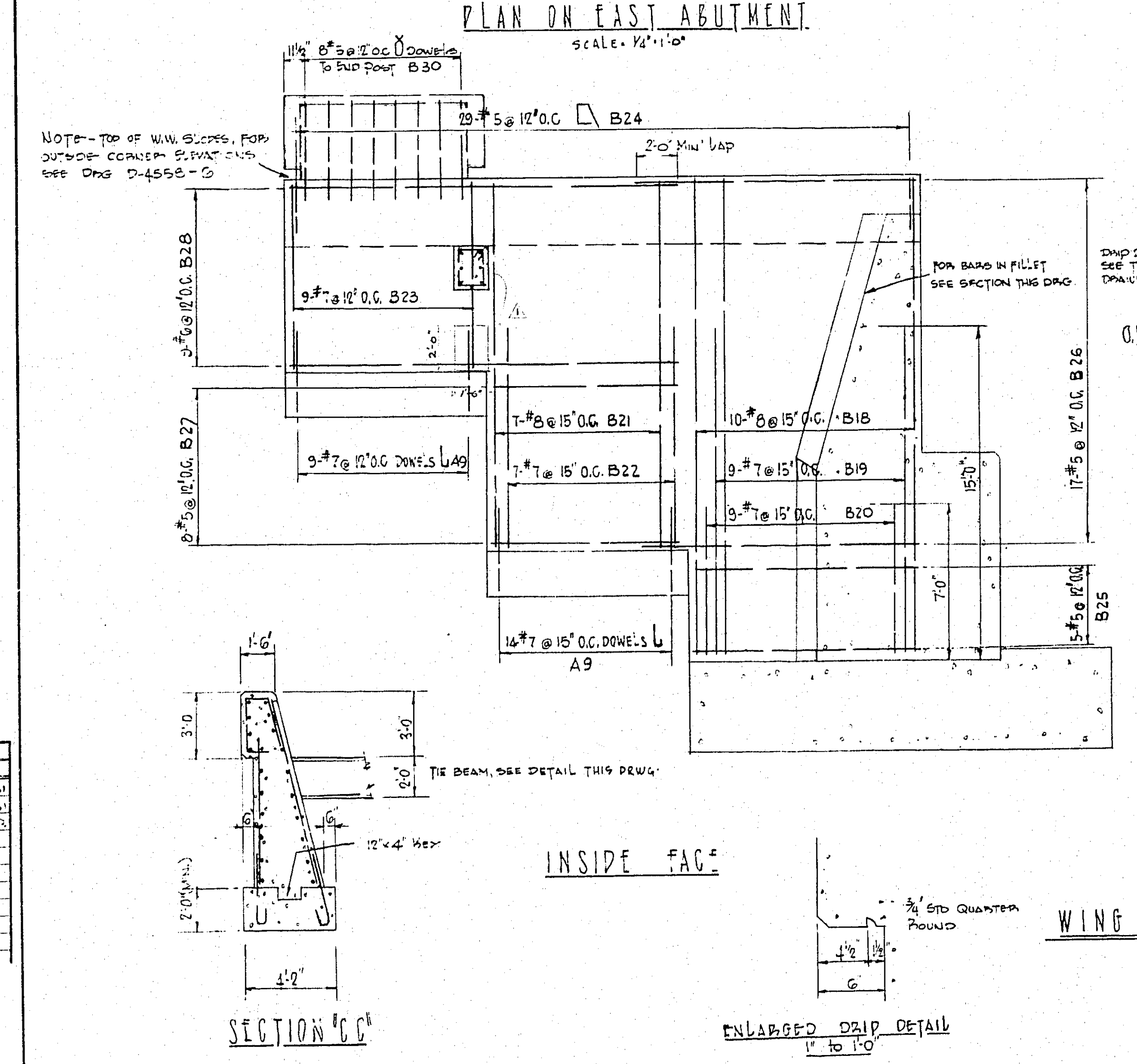
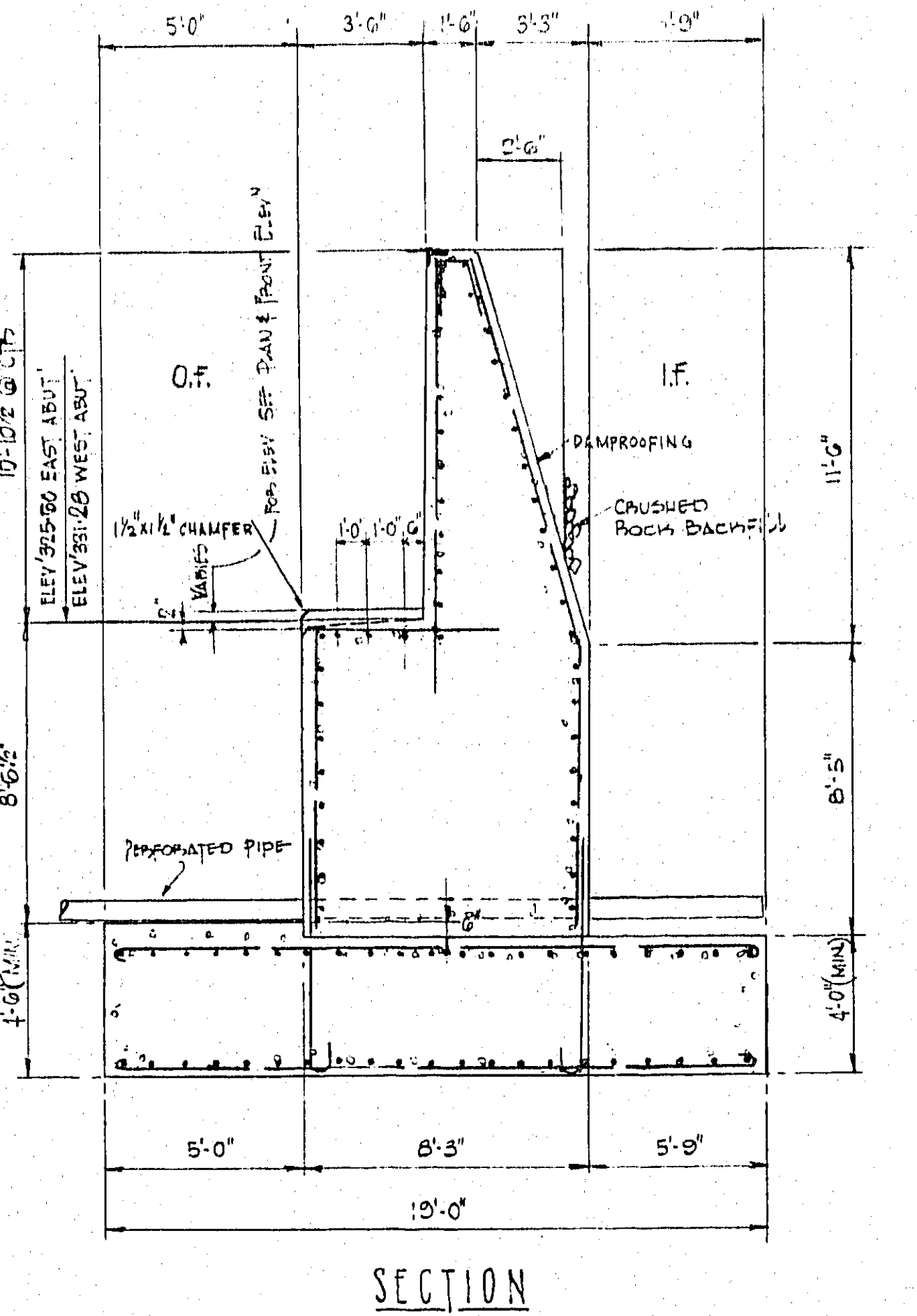
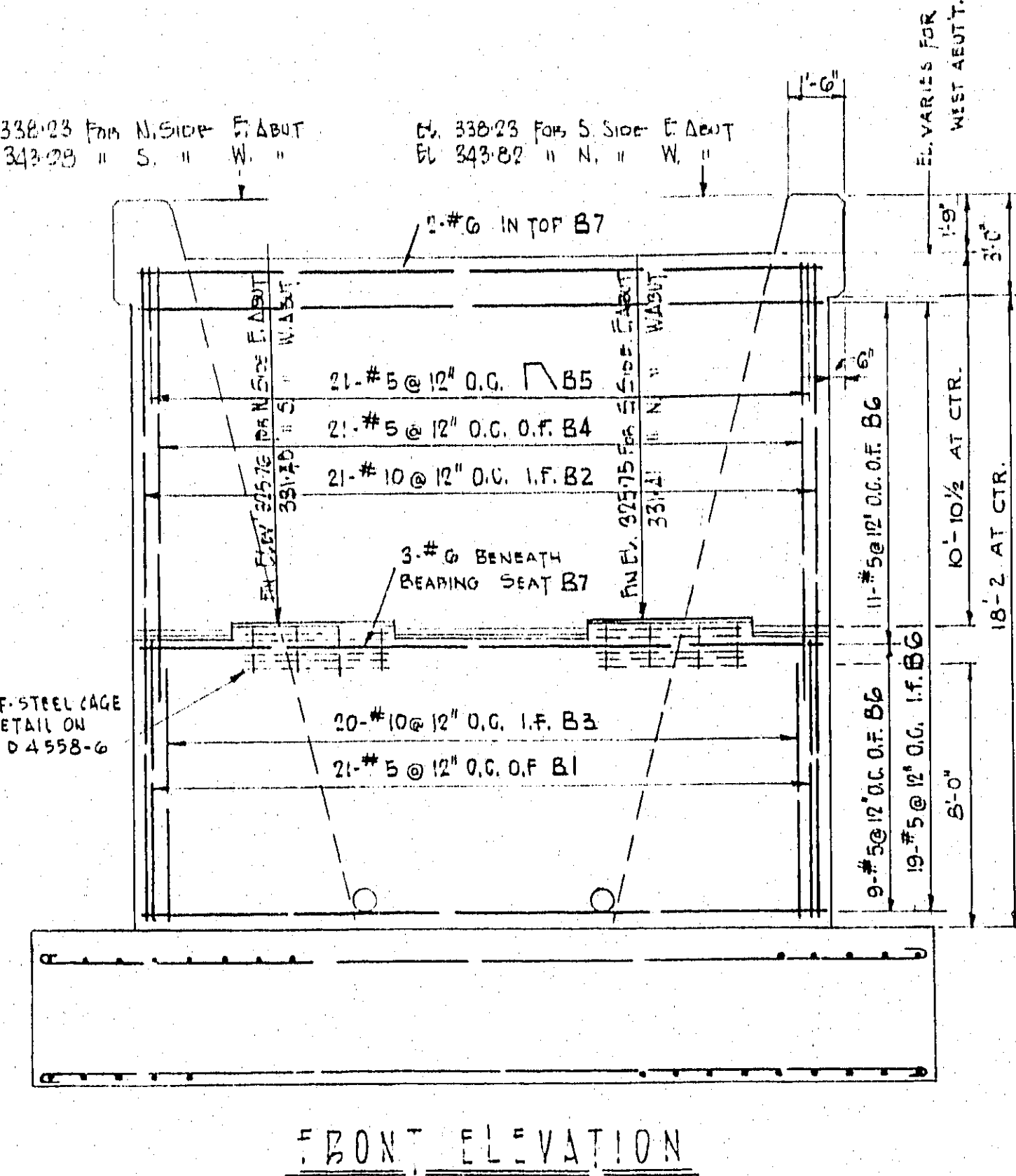
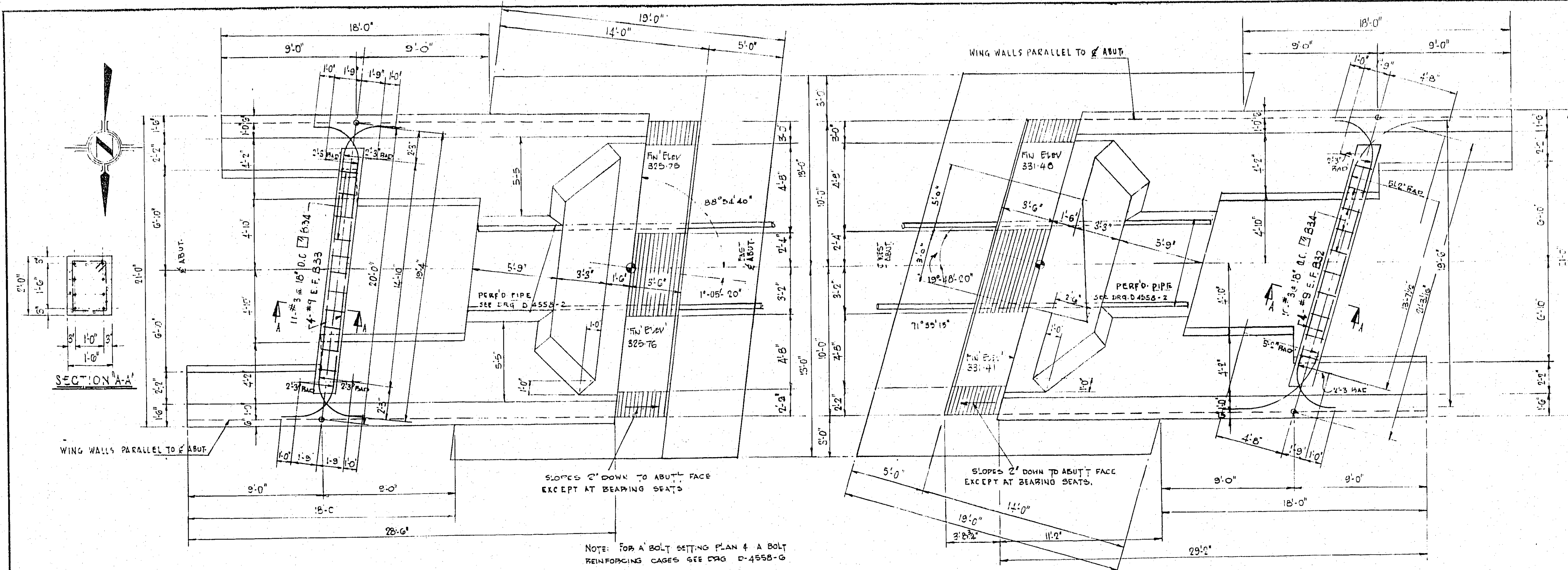
DATE AUGUST 15, 1961

E-70

CONTRACT NUMBER

D-4558-2

TWP 1334-1336-81-2



NOTES -
1. FOR GENERAL NOTES SEE DRWG. D-4558-2.
2. FOR ADITL. NOTES SEE DRWG. D-4558-6.
3. FOR SUBSTRUCTURE LAYOUT SEE DRWG. D-4558-4.

MAX. SOIL PRESSURE -
D.L. + L.L. + I + C.F. = 6.78 K/ft²

NO.	FOR	DATE
1	C.A. P.	3-8-61
2	C.A. P.	3-8-61
3	C.A. P.	3-8-61
4	C.A. P.	3-8-61
5	C.A. P.	3-8-61

WP-99-60

C. C. PARKER & PARSONS, BRINCKERHOFF LIMITED.
HAMILTON CONSULTING ENGINEERS ONTARIO

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

SUBWAY AT C. P. RAILWAY

THE KING'S HIGHWAY No. 403 DIST. No. 4
CO. OF WENTWORTH. CHEDOKO BRIDGE #3
TWP. WEST FLAMBOURGH. LOT 28 CON. I

ABUTMENTS-DETAILS & REINFORCING.

APPROVED

DESIGN ENGINEER

DATE AUGUST 15, 1961

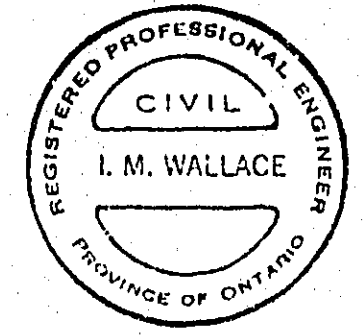
DESIGNER

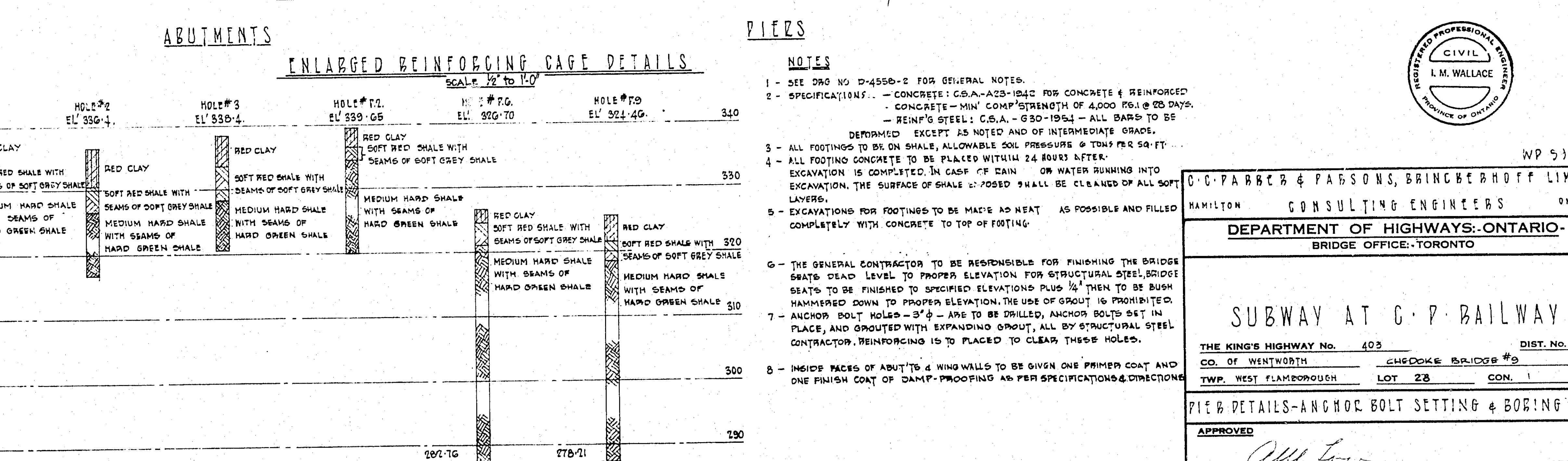
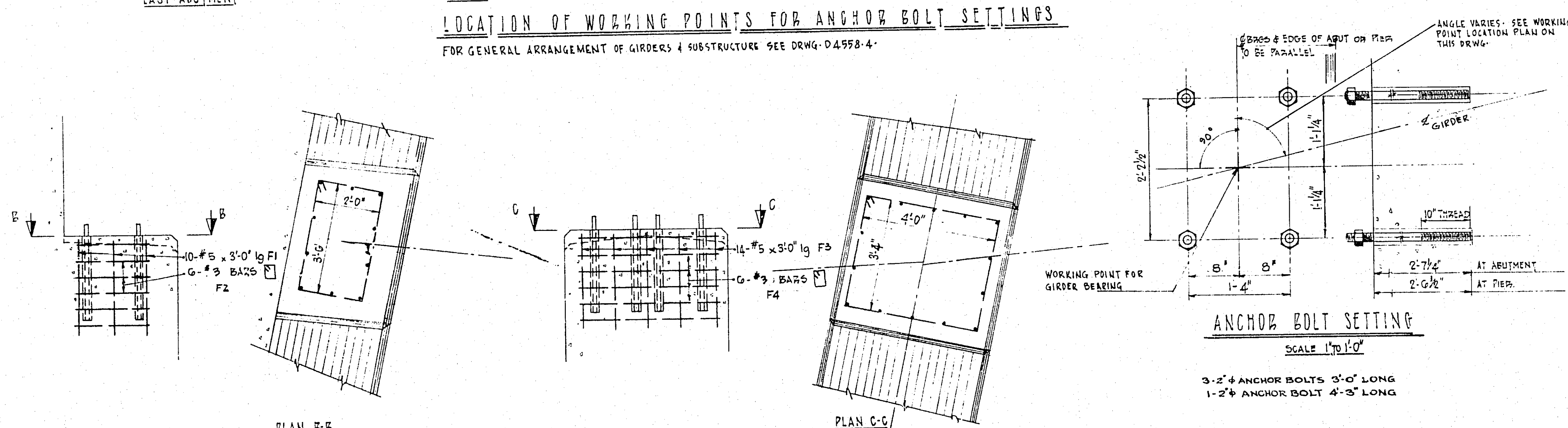
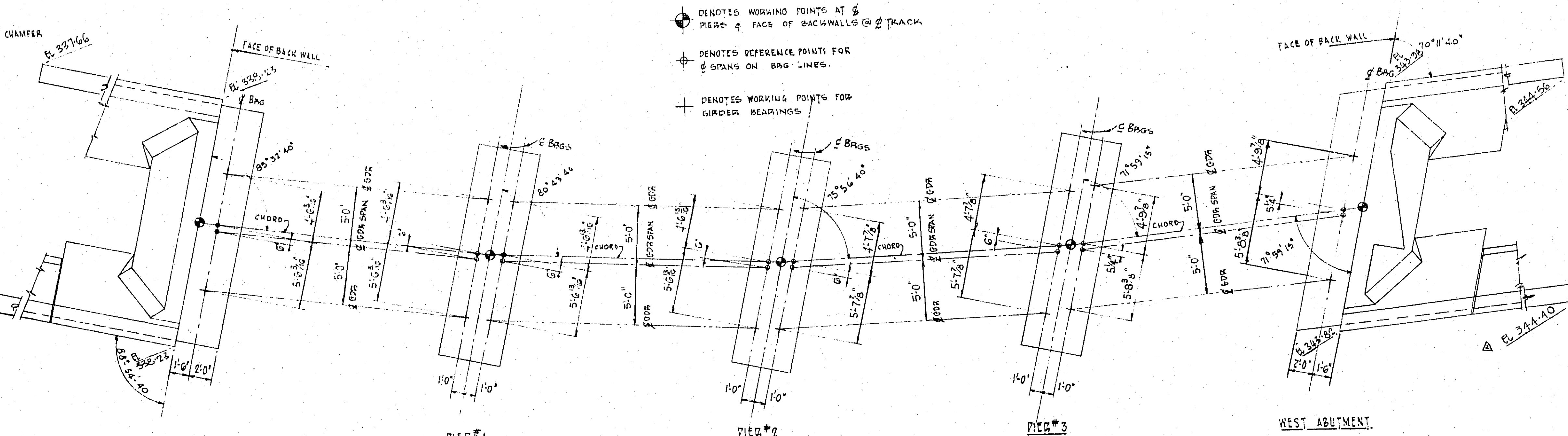
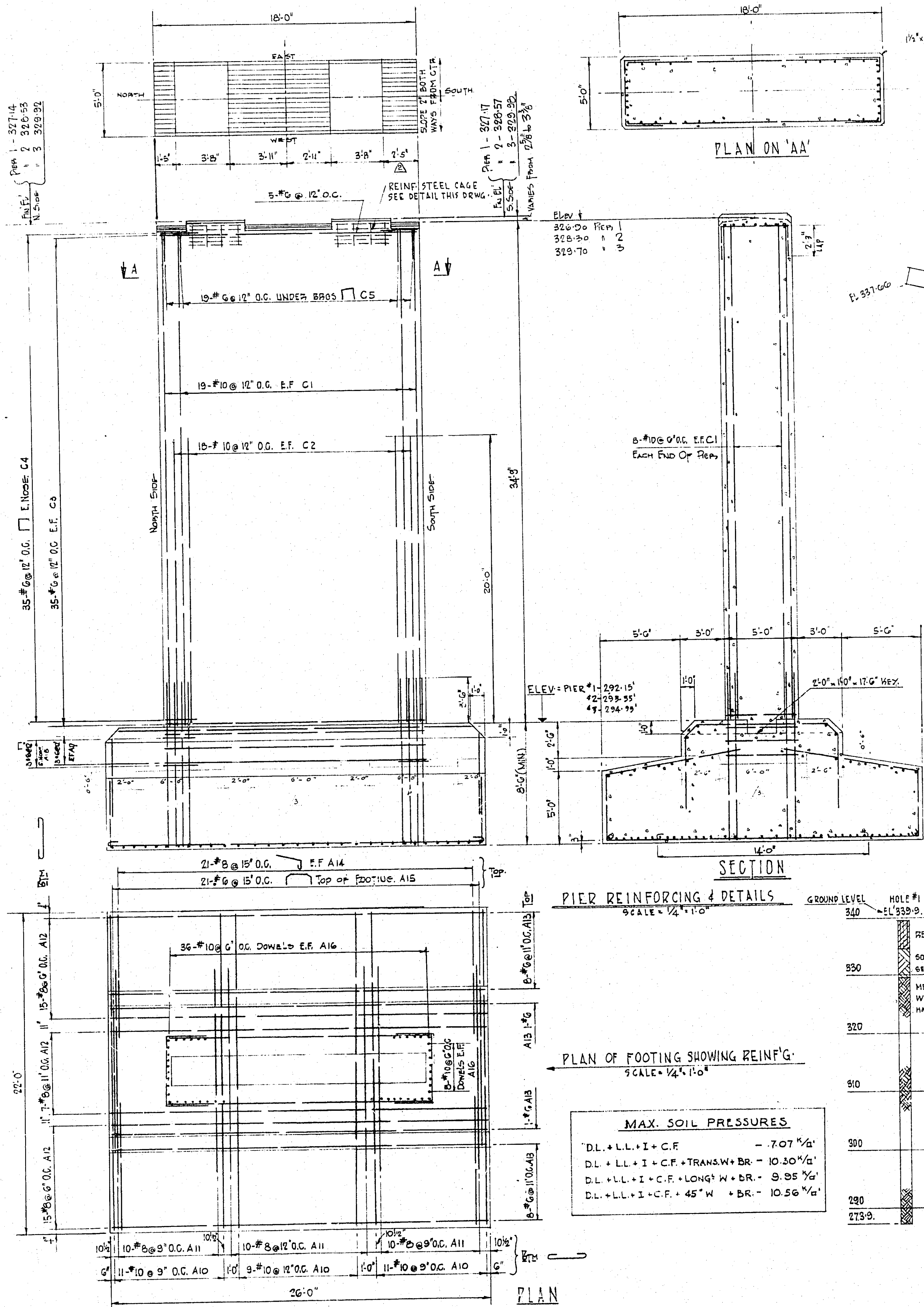
CONTRACT NUMBER

LOADING

DATE

DESCRIPTION





Plan 1-327.14
7-328.55
3-329.92

35-#6 @ 12" O.C. E. Note: C4
35-#6 @ 12" O.C. E.F. C6

22'-0"

10'-0"

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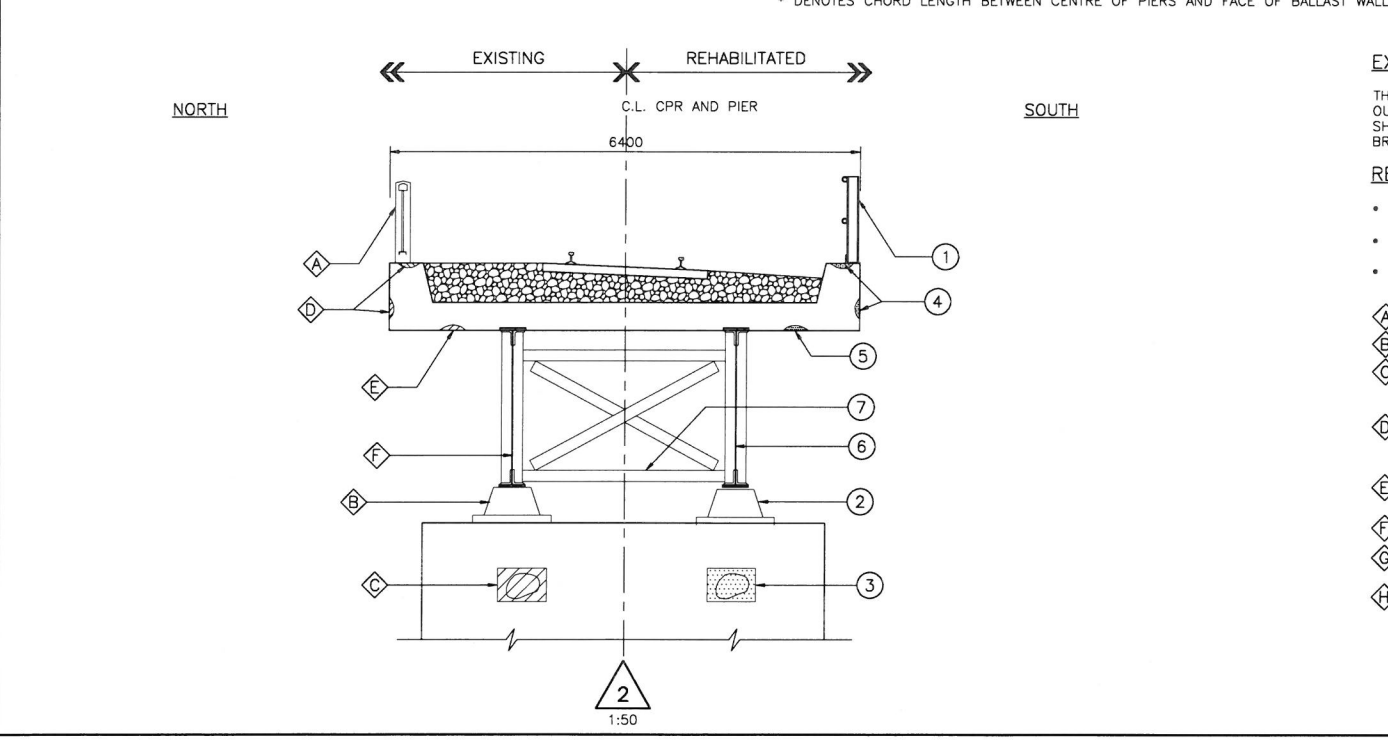
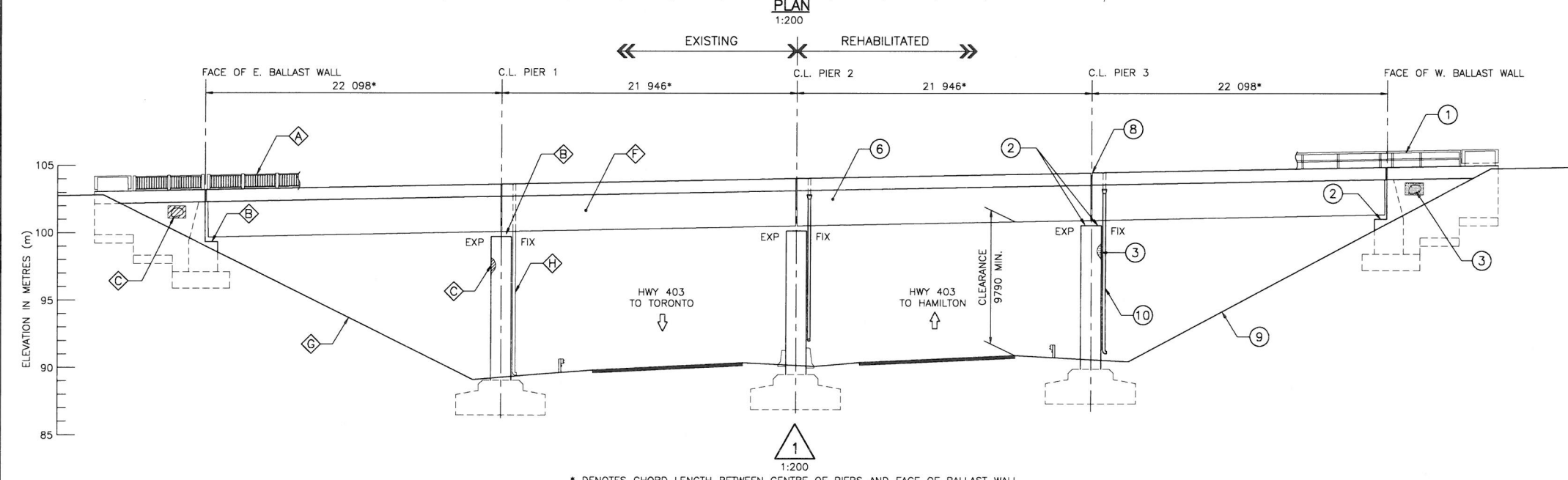
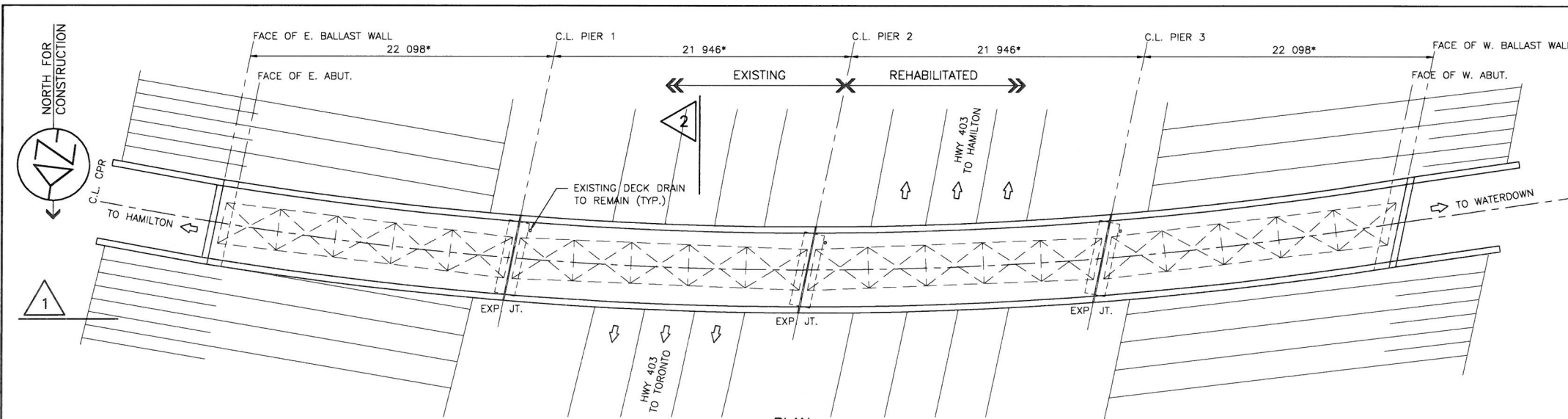
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EXTENT OF REHABILITATION WORK

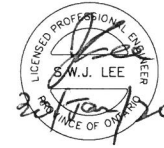
THE GENERAL EXTENT OF THE REHABILITATION WORK OUTLINED BELOW AND DESIGNATED ON THIS DRAWING SHALL BE CONSIDERED SIMILAR ON EACH SIDE OF THE BRIDGE UNLESS NOTED OTHERWISE.

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.
- ① REMOVE EXISTING STEEL RAILING.
- ② BLAST CLEAN EXISTING BEARINGS AT PIERS AND ABUTMENTS.
- ③ REMOVE DETERIORATED CONCRETE IN ABUTMENTS, WINGWALLS AND PIERS LOCALLY AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
- ④ REMOVE DETERIORATED CONCRETE IN EXISTING CURB, FASCIA AND SIDEWALK LOCALLY AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
- ⑤ REMOVE DETERIORATED CONCRETE IN DECK SOFFIT LOCALLY AS DIRECTED BY THE CONTRACT ADMINISTRATOR.
- ⑥ REMOVE EXISTING COATING ON STRUCTURAL STEEL.
- ⑦ REMOVE EXISTING CONCRETE SLOPE PAVING AT BOTH ABUTMENTS.
- ⑧ REMOVE EXISTING DECK DRAIN DOWNPIPE.

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.
- ① INSTALL NEW STEEL RAILING.
- ② REPAIR/RECONDITION EXISTING BEARINGS AT PIERS AND ABUTMENTS.
- ③ PATCH REPAIR ABUTMENTS, WINGWALLS AND PIERS LOCALLY.
- ④ PATCH REPAIR CURB, FASCIA AND SIDEWALK LOCALLY.
- ⑤ PATCH REPAIR DECK SOFFIT LOCALLY.
- ⑥ RE-COAT EXISTING STRUCTURAL STEEL.
- ⑦ GENERAL REPAIR OF STRUCTURAL STEEL.
- ⑧ REPAIR AND MODIFY EXISTING EXPANSION JOINT AT PIERS AND ABUTMENTS.
- ⑨ INSTALL NEW CONCRETE SLOPE PAVING AT BOTH ABUTMENTS.
- ⑩ INSTALL NEW DECK DRAIN DOWNPIPE.



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

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

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

SHEET
29

HWY 403 - CPR SUBWAY
STRUCTURE REHABILITATION

GENERAL ARRANGEMENT

MORRISON HERSHFIELD

GENERAL NOTES:

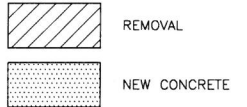
1. CLASS OF CONCRETE:
ALL ----- 30 MPa
2. CLEAR COVER TO REINFORCING STEEL:
DECK --- TOP ----- 70 +/- 20 mm
 BOTTOM ----- 40 +/- 10 mm
REMAINDER ----- 70 +/- 20 mm
UNLESS NOTED OTHERWISE.
3. REINFORCING STEEL:
 - REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
 - BAR MARKS WITH PREFIX '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:

1. 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.
2. THE CONTRACTOR SHALL REFER TO THE REFERENCE DRAWINGS (AS LISTED IN CONTRACT DOCUMENTS) FOR DETAILS OF THE EXISTING STRUCTURE.
3. 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



LIST OF DRAWING:

1. GENERAL ARRANGEMENT
2. GENERAL REPAIR
3. STRUCTURAL STEEL REPAIR - 1
4. STRUCTURAL STEEL REPAIR - 2
5. STEEL RAILING
6. DETAIL OF CONCRETE SLOPE PAVING
7. DRAINAGE DETAIL
8. STANDARD DETAILS

REVISIONS		DATE		BY		DESCRIPTION	
DESIGN	ST	CHK.	JL	CODE	CHBDC 2006	LOAD CL-625-ONT	DATE JAN. 2019
DRAWN	TLC	CHK.	ST	SITE	36-32		DWG. 1

DRAWING PATH AND NAME: O:\Toronto\Proj\130336\Structure\Drawings\Struct\09-Contract_A\36-32_01.dwg
DRAWING LAYOUT: Layout1
REVISED: Jan 30, 2019-18:57
LAST UPDATED: Jan 30, 2019-18:57



Appendix C
Selected Site Photographs



Photo 1- CPR Overpass, south side
Photo taken from Highway 403 EBL on December 1, 2021



Photo 2- CPR Overpass
West Abutment, north side
Photo taken from Highway 403 WBL in June 2021



Photo 3- CPR Overpass
West Abutment, north side
Photo taken from Highway 403 WBL on January 3, 2022



Photo 4- CPR Overpass
West Abutment, south side
Photo taken from Highway 403 WBL on January 3, 2022



Photo 5- CPR Overpass
West Abutment, south side
Photo taken from Highway 403 WBL on January 3, 2022



Photo 6- CPR Overpass
West Abutment, south side
Photo taken from Highway 403 EBL on January 29, 2022



Photo 7- CPR Overpass
West Abutment, south side
Photo taken from Highway 403 EBL on January 29, 2022



Photo 8- CPR Overpass
East Abutment, north side
Photo taken from Highway 403 WBL in June 2021

CN Rail Tracks

CP Rail Tracks



Photo 9- CPR Overpass
East Abutment, north side
Photo taken from Highway 403 EBL on December 1, 2021



Photo 10- CPR Overpass
East Abutment, south side
Photo taken from Highway 403 EBL on December 1, 2021

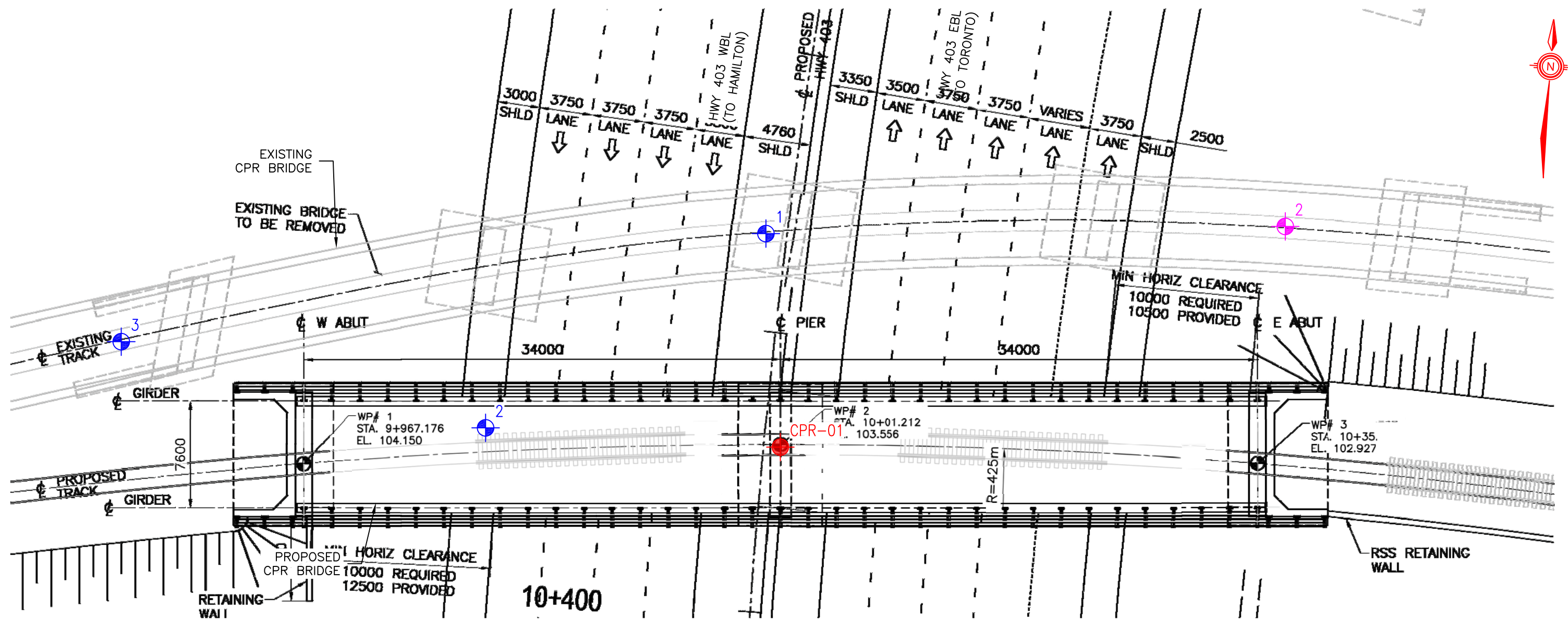


Photo 11- CPR Overpass
Pier 3
Photo taken from Highway 403 EBL on December 1, 2021



Appendix D

Plan of Proposed Borehole



PLAN

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

- PROPOSED BOREHOLES
- APPROX. BOREHOLE LOCATIONS (PREVIOUS INVESTIGATION)
(BY ONTARIO DEPARTMENT OF HIGHWAY, APRIL-MAY, 1960)
- APPROX. BOREHOLE LOCATIONS (PREVIOUS INVESTIGATION)
(BY E.M. PETO ASSOCIATES LTD., JANUARY, 1960)