



THURBER ENGINEERING LTD.

MEMORANDUM

To: Christopher Schueler, P.Eng.
AECOM

Date: December 16, 2015

From: Keli Shi, P.Eng.
Alastair Gorman, P.Eng.
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-4406-20

DETAIL FOUNDATION INVESTIGATION AND DESIGN SHELTER VALLEY CREEK CULVERT (SITE 21-272/C) REPLACEMENT OF RETAINING WALLS AT CULVERT INLET (NORTH END) GEOCRES NO. 31C-236

1. INTRODUCTION

A preliminary memorandum dated September 22, 2015 presented a brief summary of the factual findings from a foundation review carried out for the existing Highway 401 Crossing at Shelter Valley Creek in the geographical township of Haldimand, Northumberland County, Ontario. It also presents preliminary geotechnical recommendations for use in assessment of the existing foundations and for preliminary design at the site. Those recommendations were provided for planning, structure evaluation and preliminary design purposes only.

At the current detail design stage, information provided by AECOM indicated that a decision has been made to replace the retaining walls at the inlet (north end) of the culvert.

This memorandum presents the factual findings obtained from a field investigation conducted for the proposed replacement of retaining walls at the culvert inlet. The purpose of the current investigation was to explore the subsurface conditions at the retaining wall locations on either side of the culvert inlet, and to provide geotechnical recommendations for the design and construction of the replacement retaining structures.

The preliminary memo is included in Appendix D for reference and information purpose. Recommendations provided in the preliminary memo that are in conflict with the findings and recommendations presented in the current memo will be superseded.

The following reference numbers apply to this site:

- Current W.P. 4018-13-01
- Site No. 21-272/C
- Existing Geocres No. 31C00-045
- Historic W.P. 55-57

2. SITE DESCRIPTION

The site is located on Highway 401 northeast of Grafton and 3.5 km east of Regional Road 23. Based on the historic General Arrangement (GA) drawing, the existing stream crossing structure



is a single-span concrete arch culvert with a span of 15 m and a length of approximately 100 m across highway embankment. At the inlet/outlet of the 8 m high arch structure, the lower 3.6 m is vertical with wing retaining walls attached at a right angle and the upper 4.4 m is inclined at approximately 1.5H: 1V to meet the embankment fill slope above the top of arch. The maximum thickness of embankment fill above the top of arch is in the order of 11 to 12 m.

The site lies within the physiographical area of Iroquois Plain. Based on published information, this plain consists of sand and lacustrine clay deposited in glacial Lake Iroquois. The underlying bedrock consists of limestone of the Shadow Lake formation.

3. SITE INVESTIGATION AND LAB TESTING

The current site investigation and field testing were carried out during the period of May 5 to 9, 2015. A total of two boreholes, identified as 15-01 and 15-02, were advanced through the embankment fill behind the retaining walls to depths of 10.9 m and 11.1 m on east side and west side of the culvert inlet, respectively. Temporary wooden platform was built, where required on the steep embankment slope, to provide a level working surface for drilling operations. Standpipe piezometer was installed in borehole 15-01 upon completion of drilling.

The approximate locations of the two boreholes are shown on the Borehole Location Plan included in Appendix C. Borehole coordinates were obtained by a hand-held GPS unit during field investigation. Borehole elevations were estimated based on the borehole coordinates and a ground contour map provided by AECOM.

Both boreholes were advanced using portable tripod drill rig in combination with NW casing/wash boring methods. Soil samples were obtained from the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

The recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets included in Appendix A. Selected samples were also subjected to gradation analysis and the results of this testing program are summarized on the Record of Borehole sheets in Appendix A and shown on the figures included in Appendix B.

4. SUBSURFACE CONDITIONS

The subsurface stratigraphy below the existing embankment fill encountered at this site generally consists of a layer of compact sand and silt underlain by typically dense to very dense silty sand to sand deposit.

4.1. Existing Fill

Existing fill of the Highway 401 embankment encountered in boreholes consists of sand and silt to clayey silt. The thicknesses of fill were 3.8 m in both boreholes, with the base of the fill at Elev. 129.2 to 129.8 m.



SPT-N values ranged from 2 to 24 blows per 0.3 m penetration in the sand and silt fill, indicating a very loose to compact relative density. SPT-N values ranged from 19 to 35 blows per 0.3 m penetration in the clayey silt fill, indicating a very stiff to hard consistency and typically increasing with depth. Moisture contents measured in the fill ranged from 10 to 18%.

The results of two gradation analyses conducted on fill samples are provided on the Record of Borehole sheets included in Appendix A and illustrated in Figures B1a and B1b. The clayey silt fill contains 1% gravel, 14% sand, 66% silt and 19% clay, and the sand and silt fill contains 1% gravel, 41% sand, 49% silt and 9% clay.

4.2. Sand and Silt

A native sand and silt layer was encountered below the fill in both boreholes. The thicknesses of the sand and silt were about 1.8 m, with the base of the layer at Elev. 127.4 to 128.0 m.

SPT-N values recorded in the sand and silt ranged between 11 and 12 blows per 0.3 m penetration, indicating a compact relative density. Moisture contents measured on the sand and silt samples ranged between 12 and 18%.

The results of two gradation analyses conducted on the cohesionless samples are provided on the Record of Borehole sheets included in Appendix A and illustrated in Figure B2. The sand and silt contains 1 to 2% gravel, 44 to 58% sand, 34 to 45% silt and 6 to 10% clay.

4.3. Sand

A relatively thick layer of native sand was encountered below the sand and silt in both boreholes. The sand contains some silt to silty. Both boreholes were terminated within the sand layer at Elev. 122.1 to 122.5 m.

SPT-N values recorded in the sand ranged from 29 blows per 0.3 m penetration to 325 blows for 0.25 m penetration, indicating a relative density of compact to very dense. Moisture contents measured on the sand samples ranged from 18 to 20%.

The results of four gradation analyses conducted on the sand samples are provided on the Record of Borehole sheets included in Appendix A and illustrated in Figure B3. The sand contains 0 to 10% gravel, 71 to 84% sand, 12 to 23% silt and 3 to 4% clay.

4.4. Water Levels

A standpipe piezometer was installed in Borehole 15-01 to monitor the groundwater levels after drilling. The groundwater level measurements are summarized in the table below.

Borehole	Date	Water Level (m)		Remark
		Depth	Elevation	
15-01	May 7, 2015	2.1	130.9	In piezometer
	May 8, 2015	2.0	131.0	
	May 9, 2015	2.1	130.9	

Based on the historic information, water level in the creek was reported at Elevation 128.1 in February 1954. The estimated high water level was at Elevation 131.5 approximately.



The creek and groundwater levels are expected to fluctuate seasonally and subject to precipitation patterns, and may vary from the levels presented above.

5. FOUNDATION RECOMMENDATIONS

Based on the soil conditions encountered at the borehole locations, spread footings founded on native soil are considered a suitable foundation type for the replacement retaining walls. The following table summarizes the recommended highest founding elevations and geotechnical resistances assuming a minimum 2 m wide spread footing subjected to vertical concentric loading.

Retaining Wall Location at Culvert Inlet	Borehole	Recommended Highest Founding Elevation (m)	Bearing Stratum	SLS (kPa)	Factored ULS (kPa)
East Side	15-01	127.0	Very dense sand	300	450
West Side	15-02	128.0	Dense sand	300	450

The base of the footings for the replacement retaining walls must be at or above the footing base of the existing culvert so as not to undermine the existing culvert footing. The exposed native material at the subgrade level should be protected from disturbance such as construction traffic and weathering. Any topsoil and soft/loose fill or native material should be stripped from the footprint of the footing. A minimum 100 mm thick lean concrete mud slab or 150 mm of compacted granular 'A' material should be placed under the footing base to provide a level working surface.

Where eccentric or inclined loads are applied, the geotechnical resistances used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The geotechnical resistance at SLS is based on an estimated settlement not exceeding 25 mm. This settlement will be essentially complete by the end of construction.

The lateral resistance developed along the base of concrete footings founded on the above bearing stratum may be computed using an ultimate friction coefficient of 0.45.

Excavation and backfilling for the footings must be in accordance with OPSS 902.

The depth of frost penetration at this site is approximately 1.2 m. The base of footings must be provided with a minimum of 1.2 m of earth cover as protection against frost action.

Footing construction will be likely below the groundwater level at the recommended founding levels. Dewatering will be required during footing construction, and footings must be constructed in the dry.

It is understood that the existing retaining wall footings will be reused to support the replacement retaining walls. Design of the replacement walls founded on the existing footings should take into account the above recommendations where applicable.

6. LATERAL EARTH PRESSURES

Earth pressures acting on the retaining walls may be assumed to be distributed triangularly and to be governed by the characteristic of the backfill material. For a fully drained condition, the



lateral earth pressures acting on the wall should be computed in accordance with the CHBDC but generally given by the expression:

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

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

Earth pressure coefficients for backfill to the retaining wall are dependent on the material used as backfill. Typical values are given in the table below.

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$			OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		
	Horizontal Surface Behind Wall	Sloping Backfill		Horizontal Surface Behind Wall	Sloping Backfill	
		2H:1V	Existing 1.5H:1V Slope *		2H:1V	Existing 1.5H:1V Slope *
Active (K_a)	0.27	0.39	0.50	0.31	0.47	0.72
At-rest (K_0)	0.43	-	-	0.47	-	-
Passive (K_p)	3.7	-	-	3.3	-	-

* Inclination of existing embankment slope based on historic drawings.

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

The earth pressure coefficients in the above table are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the CHBDC.

The retaining wall design must be checked against sliding, overturning and global stability modes of failure. In addition, it should be noted that the fill above the retaining wall is at a steep inclination of 1.5H: 1V. For such steeply sloping backfill behind the wall, the earth pressure coefficients are considerably higher than horizontal backfill condition. The earth pressure coefficients for sloping backfill are provided in the above table.

It is understood that the existing wall footings will be reused to support the replacement retaining walls, and the embankment slope behind and the ground in front of the walls will be restored to the existing conditions following wall construction. In view of the typically very dense sand underlying the footing bases, the replacement structures were assessed to be stable in their final configuration from a global stability perspective. There was no indication of local or global instability of the existing embankment slope. This discussion is based on the final embankment



geometry matching the existing. If the embankment geometry changes, the geotechnical recommendations must be reassessed.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular 'B' Type I or III, or at a depth of 2.0 m for Granular 'A' or Granular 'B' Type II.

Proper drainage measures, such as subdrain and weep holes, should be provided behind the retaining walls to remove any hydrostatic groundwater pressure acting on the wall.

7. EXCAVATION AND DEWATERING

Excavation for footing construction for the replacement retaining walls must not undermine the existing culvert footings.

Excavation for the replacement of the retaining walls is expected to extend to approximate Elev. 126.8 and 127.8 m, and will be primarily within the existing fill and compact sand and silt. Based on the historic information, water level in the creek was reported at Elev. 128.1 in February 1954. The estimated high water level was at Elev. 131.5 approximately. The observed creek level during the current investigation was at approximately 0.5 m below the creek banks or Elev. 128 m. The excavation for footing construction is expected to extend below the water level in cohesionless soil.

Excavation of cohesionless soils below the water level at the retaining walls must be carried out with a properly designed cofferdam and dewatering system in place to lower groundwater table to a minimum of 0.5 m below the base of excavation and to prevent boiling of the footing base. An effective cofferdam and dewatering system may include watertight sheet pile enclosure and positive well points. The dewatering system must remain operational until the retaining wall is constructed and backfilled. The Contractor is responsible for design of the dewatering system and shall retain a dewatering specialist for design. An NSSP for dewatering is included in the Appendix E and the NSSP must be included in the tender document.

Groundwater table drawdown caused by dewatering operation is not expected to affect the existing arch footings and retaining wall footings from settlement perspective given the presence of dense to very dense sand below the existing footing bases. However, dewatering operation must avoid loss of soil due to seepage flows under the existing footing bases.

Concrete repair work proposed inside the arch culvert may require that the water level be depressed below the top of the footing. This can be achieved through the use of a cofferdam and pumping. The design of this system is also the responsibility of the Contractor but the contract documents must include constraints to guard against the loss of soil below the existing foundations.

It is recognized that there are constraints on the type of equipment that can be operated inside the culvert in order to construct a cofferdam. Consequently, it is possible that the Contractor will choose a system such as sandbags with an impermeable membrane in front to restrict the inflow of water from the stream, while pumping from inside the cofferdam. While a system like this may be made to work, the permeable nature of the soils means that there will be potentially large inflows of water from the base of the work area behind the cofferdam.



Whatever cofferdam/unwatering system is selected, it must be designed and operated so as to prevent the loss of soil from below the footing and to prevent heaving or boiling of the base of the enclosed work area. It is essential that this system be designed by professionals specializing in dewatering systems and the design must include, among other issues, the following:

- The unbalanced head of water acting upward on the base of the enclosed area, taking account of the design high water level, or such higher level as the designer considers to be prudent.
- The exit gradient of the groundwater and the potential for boiling, heaving or piping.

The measures to address these issues must be detailed by the Contractor's designer, but typically it may require the use of a filter on the base of the work area and/or a temporary concrete slab or granular blanket to protect and weigh down the filter. Under no circumstances may any excavation or other work be carried out that undermines, or may undermine, the existing foundations.

The volume of water that will be extracted during dewatering operation is expected to be high on a daily basis depending on the implemented dewatering scheme. A PTTW is recommended for this site.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill and native sand and silt may be classified as Type 3 soil above the water table and as Type 4 soil below the water table. Flatter slopes may be required at locations where water seepage affects surficial stability.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

8. STABILITY OF TEMPORARY EXCAVATION

Temporary support systems will be required in conjunction with an active dewatering system to allow construction of the retaining wall footings in the dry. Sheet piles or soldier pile and lagging walls are considered appropriate support systems. Design of the support systems should address potential base boiling for excavation in cohesionless soil below groundwater level.

The temporary excavation support systems for the existing embankment slope should be designed and constructed in accordance with OPSS 539. In general, the lateral movement of the temporary excavation support system should meet Performance Level 2 as specified in OPSS 539. Sheet piles or soldier piles and lagging wall are considered appropriate for slope protection. In view of the high lateral earth pressure associated with the existing steep embankment slope, soil anchors may be required to maintain slope stability and limit slope movements.

The design of the temporary support system is the responsibility of the Contractor and must be carried out by a professional engineer experienced in this design. The Contractor should select the type of temporary support system and design taking into account the soil conditions encountered in the boreholes and stability of the existing embankment slope behind the temporary support.



9. CLOSURE

Engineering analysis and preparation of the memorandum were carried out by Keli Shi, P.Eng. The memo was reviewed by Mr. Alastair Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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Geotechnical Engineer



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Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact





Appendix A
Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

RECORD OF BOREHOLE No 15-01

1 OF 2

METRIC

W.P. 4018-13-01 LOCATION Site 21-272/C, Hwy 401 N 4 875 774.2 E 185 569.6 ORIGINATED BY ME
 HWY 401 BOREHOLE TYPE Tripod/Casing COMPILED BY MFA
 DATUM DATE 2015.05.05 - 2015.05.06 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
133.0	GROUND SURFACE							20	40	60	80	100					
0.0	SAND and SILT , trace rootlets, trace gravel Loose to Compact Brown Moist (FILL)		1	SS	6												
132.2																	
0.8	Clayey SILT , some sand, trace gravel Very Stiff to Hard Brown Moist (FILL)		2	SS	19		132										
			3	SS	23												1 14 66 19
			4	SS	23		131										
			5	SS	35		130										
129.2																	
3.8	SAND and SILT , trace clay, trace gravel Compact Brown Moist		6	SS	11		129										1 44 45 10
							128										
127.4							127										3 81 12 4
5.6	SAND , some silt to silty, trace clay, trace gravel Very Dense Brown Moist		7	SS	105		126										
							125										
			8	SS	150												
							124										0 73 23 4
			9	SS	325/ 0.250												

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-01

2 OF 2

METRIC

W.P. 4018-13-01 LOCATION Site 21-272/C, Hwy 401 N 4 875 774.2 E 185 569.6 ORIGINATED BY ME
 HWY 401 BOREHOLE TYPE Tripod/Casing COMPILED BY MFA
 DATUM DATE 2015.05.05 - 2015.05.06 CHECKED BY KS




SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
122.1			10	SS	100/ .125												
10.9	END OF BOREHOLE AT 10.9m. Piezometer installation consists of 25.4mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2015.05.07 2.1 130.9 2015.05.08 2.0 131.0 2015.05.09 2.1 130.9																

RECORD OF BOREHOLE No 15-02

1 OF 2

METRIC

W.P. 4018-13-01 LOCATION Site 21-272/C, Hwy 401 N 4 875 771.0 E 185 549.3 ORIGINATED BY ME
 HWY 401 BOREHOLE TYPE Tripod/Casing COMPILED BY MFA
 DATUM DATE 2015.05.08 - 2015.05.09 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE					W P W W L								
								● QUICK TRIAXIAL × LAB VANE													
133.6	GROUND SURFACE						20	40	60	80	100	20	40	60							
0.0	SAND and SILT , trace clay, trace gravel Very Loose to Compact Brown Moist (FILL)		1	SS	2																
			2	SS	5								○								
			3	SS	13								○						1 41 49 9		
			4	SS	24								○								
			5	SS	5								○								
129.8	SAND and SILT , trace clay, trace gravel Compact Brown Wet		6	SS	12										○			2 58 34 6			
128.0	SAND , some silt, trace clay, trace gravel Compact to Very Dense Brown Wet to Moist		7	SS	29										○			10 71 15 4			
8			SS	74											○						
			9	SS	122										○						

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-02

2 OF 2

METRIC

W.P. 4018-13-01 LOCATION Site 21-272/C, Hwy 401 N 4 875 771.0 E 185 549.3 ORIGINATED BY ME
 HWY 401 BOREHOLE TYPE Tripod/Casing COMPILED BY MFA
 DATUM DATE 2015.05.08 - 2015.05.09 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W _p	W	W _L		
	Continued From Previous Page							20	40	60	80	100					
122.5			10	SS	168		123										0 84 13 3
11.1	END OF BOREHOLE AT 11.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG.																

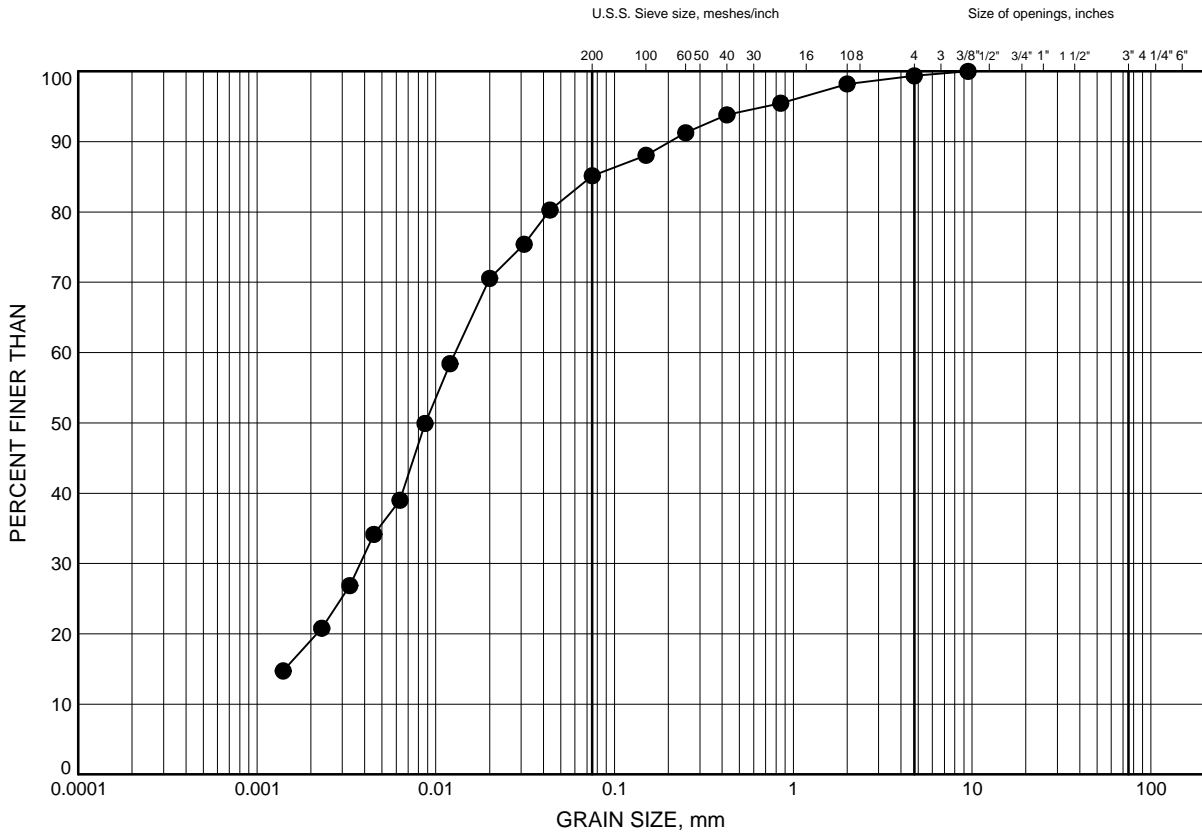


Appendix B
Lab Test Results

Eastern Rehabilitation 18 Structures
GRAIN SIZE DISTRIBUTION

FIGURE B1a

Clayey SILT (FILL)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	15-01	1.52	131.48

Date June 2015
W.P. 4018-13-01

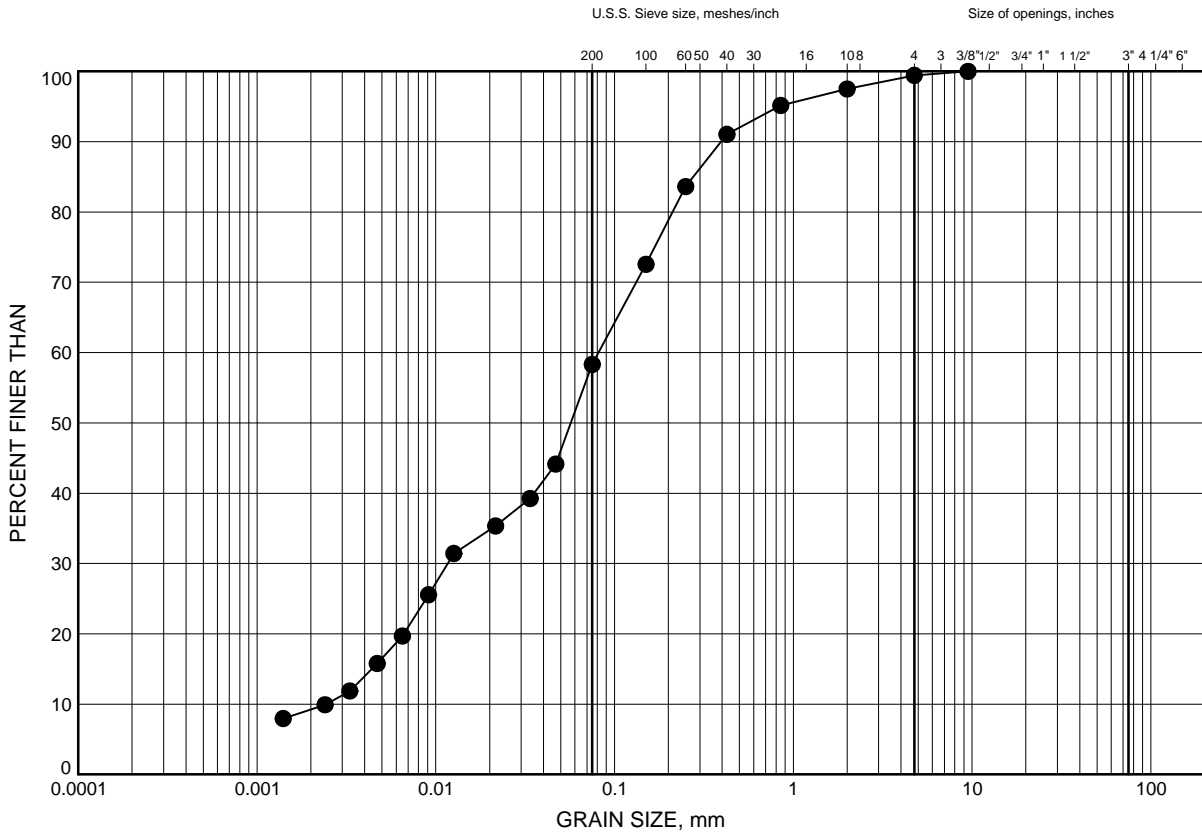


Prep'd AN
Chkd. KS

Eastern Rehabilitation 18 Structures
GRAIN SIZE DISTRIBUTION

FIGURE B1b

SAND & SILT (FILL)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	15-02	1.52	132.08

Date June 2015
W.P. 4018-13-01



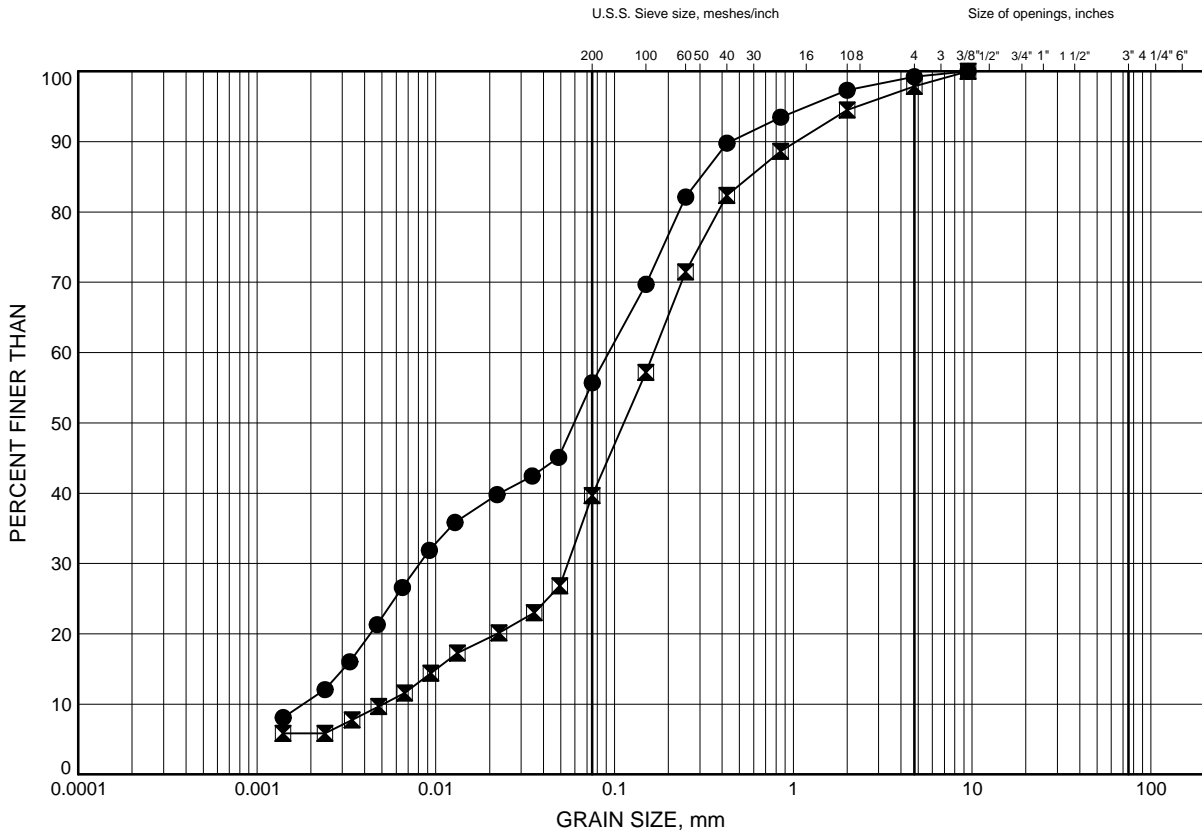
Prep'd AN
Chkd. KS

Eastern Rehabilitation 18 Structures

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	15-01	4.88	128.12
⊠	15-02	4.88	128.72

Date June 2015
W.P. 4018-13-01

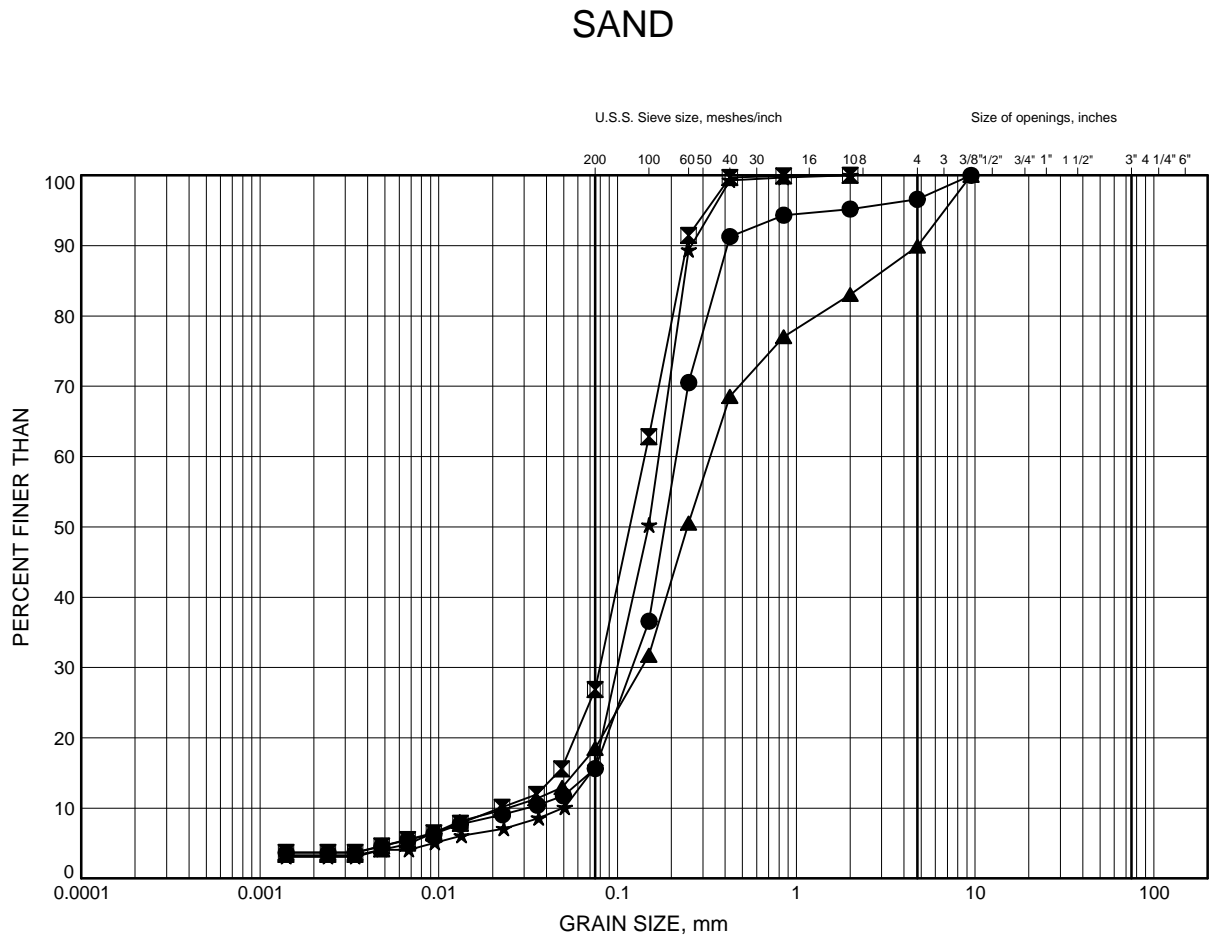


Prep'd AN
Chkd. KS

Eastern Rehabilitation 18 Structures

GRAIN SIZE DISTRIBUTION

FIGURE B3



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	15-01	6.40	126.60
⊠	15-01	9.35	123.65
▲	15-02	6.40	127.20
★	15-02	10.90	122.70

Date June 2015
W.P. 4018-13-01

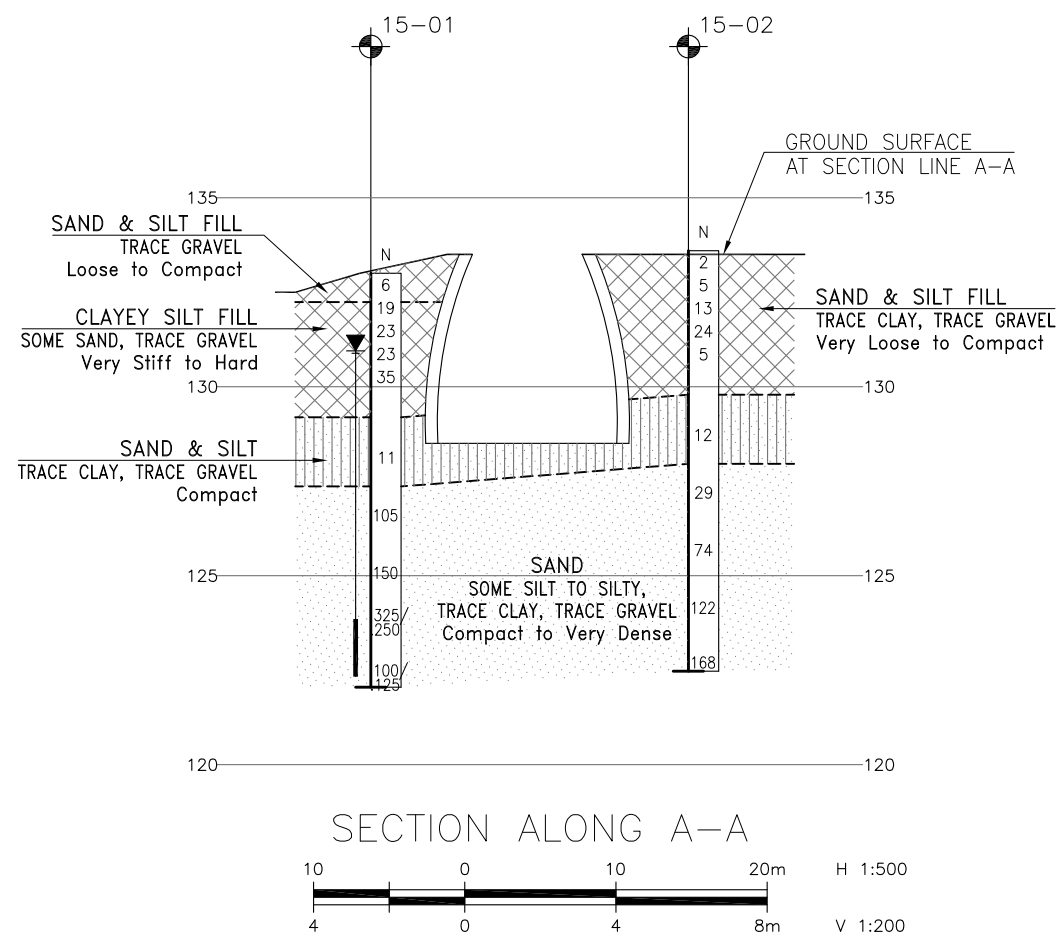
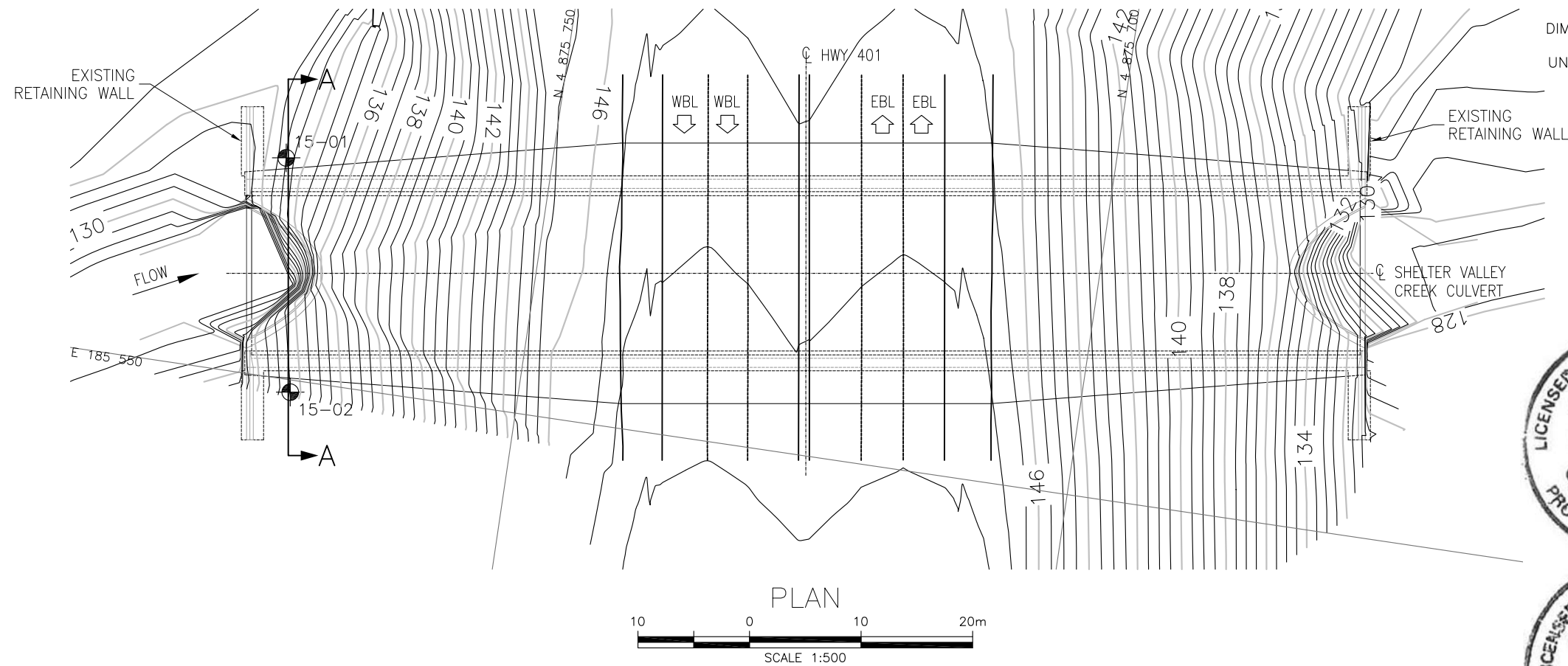


Prep'd AN
Chkd. KS



Appendix C

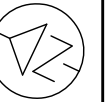
Borehole Locations and Soil Strata Drawing



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



HWY
CONT No
WP No 4018-13-01

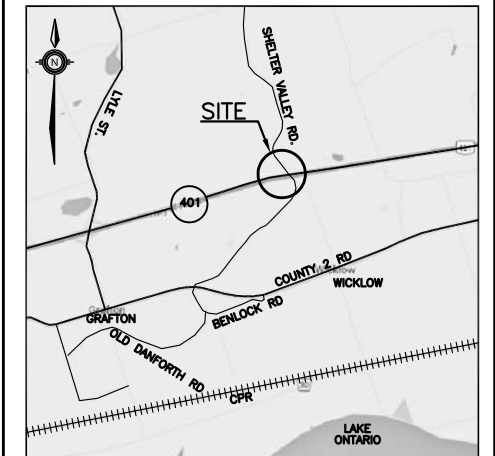


MEGA 4
SHELTER VALLEY CREEK CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
6






THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole
N	Blows /0.3m (Std Pen Test, 475J/blow)
	Water Level In Open Borehole
	Water Level In Standpipe Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
15-01	133.0	4 875 774.2	185 569.6
15-02	133.6	4 875 771.0	185 549.3

-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31C-236

[illegible]



Appendix D

Memo – Preliminary Foundation Investigation and Design



THURBER ENGINEERING LTD.

MEMORANDUM

To: Christopher Schueler, P.Eng.
AECOM

Date: September 22, 2015

From: Keli Shi, P.Eng.
Alastair Gorman, P.Eng.
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-4406-20

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN SHELTER VALLEY CREEK CULVERT (SITE 21-272/C) WP 4018-13-01, GEOCRES NO: 31C-229

1. INTRODUCTION

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Highway 401 Crossing at Shelter Valley Creek in the geographical township of Haldimand, Northumberland County, Ontario. It also presents preliminary geotechnical recommendations for use in assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives are not yet defined.

The recommendations provided in this memorandum are for planning, structure evaluation and preliminary design purposes only. Additional investigation and analysis may be required in any subsequent detail design phase of the project.

The following reference numbers apply to this site:

- Current W.P. 4018-13-01
- Site No. 21-272/C
- GEOCRES No. 31C00-045
- Historic W.P. 55-57

2. SITE DESCRIPTION

The site is located on Highway 401 northeast of Grafton and 3.5 km east of Regional Road 23. Based on the historic General Arrangement (GA) drawing, the existing stream crossing structure is a single-span concrete arch culvert with a span of 15 m and a length of approximately 100 m across highway embankment. At the ends of the 8 m high arch structure, the lower 3.6 m is vertical with retaining walls attached and the upper 4.4 m is inclined at approximately 1.5H: 1V to meet the embankment fill slope above the top of arch. The thickness of embankment fill above the top of arch is in the order of 11.6 m.



The site lies within the physiographical area of Iroquois Plain. Based on published information, this plain consists of sand and lacustrine clay deposited in glacial Lake Iroquois. The underlying bedrock consist of limestone of the Shadow Lake formation.

3. SUBSURFACE CONDITIONS

A foundation investigation report was completed by the Department of Highways Foundations Section in September 1957. Three boreholes (1, 2 and 3) were drilled in conjunction with Standard Penetration Tests (SPT) to depths of 6.7 to 15.7 m from the original ground surface. Dynamic cone penetration tests (DCPT) (4, 5 and 10) were advanced to practical DCPT refusal at depths ranging from 1.3 to 4.0 m. Borehole 1 and all DCPTs were completed in the vicinity of the creek crossing. The GEOCRE file is attached in Appendix A.

Topsoil was encountered only in Borehole 2 drilled approximately 0.8 km west of the creek. The thickness of the topsoil was 0.3 m.

A layer of dense to very dense gravelly sand to sand and gravel with occasional boulders was encountered from ground surface in Boreholes 1 and 3 and below the topsoil in Borehole 2, with SPT N-values ranging from 32 to 165 blows for 0.3 m of penetration. The layer was fully penetrated in Borehole 3 with the layer thickness of 2.7 m and the base of the layer at Elevation 129.3. Boreholes 1 and 2 were terminated within the layer at depths of 6.7 m and 15.7 m or Elevation 123.0 and 119.2. Natural moisture contents of the sand and gravel deposit ranged from 4 to 15% and typically from 13 to 15%.

The sand and gravel was underlain by a layer of dense to very dense sand in Borehole 3, with SPT N-values ranging from 38 to 110 blows for 0.3 m of penetration. The borehole was terminated in this layer at Elevation 119.4. Natural moisture contents of the sand samples ranged from 9 to 15% and typically from 12 to 15%.

Water level in the creek was reported at Elevation 128.1 in February 1954. The estimated high water level was at Elevation 131.5 approximately.

4. SITE OBSERVATIONS

Foundations engineering staff from Thurber visited the site to observe conditions related to the geotechnical performance.

There were no obvious signs of settlement or distress in the foundations.

The embankment slopes appeared to be stable, with no obvious signs of instability or bulging. There were multiple spots of minor water seepage on the interior face of the arch. Concrete spalling and exposed, corroded reinforcement were noted on the east retaining wall of the north end or inlet.

Photographs of the structure and the overlying embankment are attached in Appendix B.



5. EXISTING FOUNDATIONS

Based on the historic GA drawing for the structure, D-4111-1 dated May 1958, the arch structure and retaining walls are supported on spread footings.

The width and thickness of the arch footings vary along the length of the footing. The middle one third of the 100 m long footing or 33.5 m long middle section, has a constant base width of 4.7 m and a footing thickness of 2.4 m. The footing width decreases linearly from 4.7 m to 2.1 m at both ends, and the footing thickness decreases linearly from 2.4 m to 1.5 m at both ends. The top of footing is flat at Elevation 127.4 m.

The wing wall footings at the inlet and outlet are 2 m wide and 0.76 m thick with the top of footings located at the same elevation as the arch footings. Each retaining wall is 3.7 m high and 7.3 m long.

Based on the recommendations provided in the GEOCRETS report, the footings were designed for an allowable bearing pressure of 3 tons/sq. ft. (working stress design). This is approximately equivalent to an SLS geotechnical resistance of 300 kPa. The highest founding elevation of the footing base was recommended to be 125.0 m, i.e. approximately 3 m below the stream bed, to minimize scouring hazards.

6. ASSESSMENT OF EXISTING FOUNDATIONS

Based on the soil conditions shown to exist at this site and the information contained on the historical GA, Limit State Design geotechnical resistances have been calculated in accordance with the requirements of the CHBDC. These values can be used in carrying out an assessment of the existing structure and for preliminary design of any modifications that may be necessary.

Based on the founding elevations provided on the historic GA and the soil conditions reported in the borehole logs, the recommended bearing resistances for footing assessment are as follows:

Footing Width (m)	Footing Base Elevation (m)	SLS (kPa)	Factored ULS (kPa)
2.0	126.6	300	450
2.0	125.9	400	600
3.0	125.4	400	600
4.5	125.0	400	600

The above bearing resistance values are computed for vertical concentric loading with the top of footing at or below the ground surface. For eccentric or inclined loading, the bearing resistances must be adjusted as per the CHBDC.

7. EXCAVATION AND ROAD WAY PROTECTION

If the selected rehabilitation strategy requires excavation above or beside the culvert, it is recommended that site investigation and field testing be carried out in order to characterize the fill and to select parameters for the design of the rehabilitation, roadway protection, shoring and



dewatering. The number and depth of boreholes can be determined after the rehabilitation strategy has been selected.

8. UNWATERING/DEWATERING AND EROSION CONTROL

Any work carried out below the groundwater level or in the stream must be protected by an appropriate stream diversion and unwatering system. Any unwatering or dewatering system must protect the existing foundations and must prevent heaving, boiling or piping of the soils in the work zone and below the existing foundations.

Additional comments are provided in a supplementary memorandum prepared for this site.

9. CLOSURE

The factual subsurface information used in the preparation of this memorandum was taken from the report by the Department of Highways Foundations Section titled "Foundation Report on New Bridge at Highway 401 Crossing Shelter Valley Creek, about 2 miles North East of Grafton", W.J. F-57-27, W.P. 55-57 and dated September 30, 1957.

The memorandum was prepared by Keli Shi, P.Eng., and reviewed by Mr. Alastair Gorman, P.Eng., P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Keli Shi, P.Eng.
Geotechnical Engineer



Alastair Gorman, P.Eng.
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact



Attachments

Client: AECOM
File No.: 19-4406-20

E file: h:\19\4406\20 eastern rehab 18 structures\reports and memos\shelter vallery creek\final - original program\site 21-272c shelter valley creek culvert memo 1 sept 22 15 f.docx

Date: September 22, 2015
Page 4

Appendix A

GEOCRES Report and Historic Drawings

FOUNDATION REPORT

on

New Bridge at Highway 401
Crossing Shelter Valley Creek,
about 2 miles North East of Grafton.

Plan No: F-3133-4

Station No: 307/00

Distribution:

Mr. A. Toye
Bridge Engineer (2)

Mr. H. Tregaskes
Construction Engineer (1)

Mr. D. G. Ramsay
Design Engineer (1)

Mr. H. D. Duff
Dist.Eng. PORT HOPE (1)

Foundation Section (1)

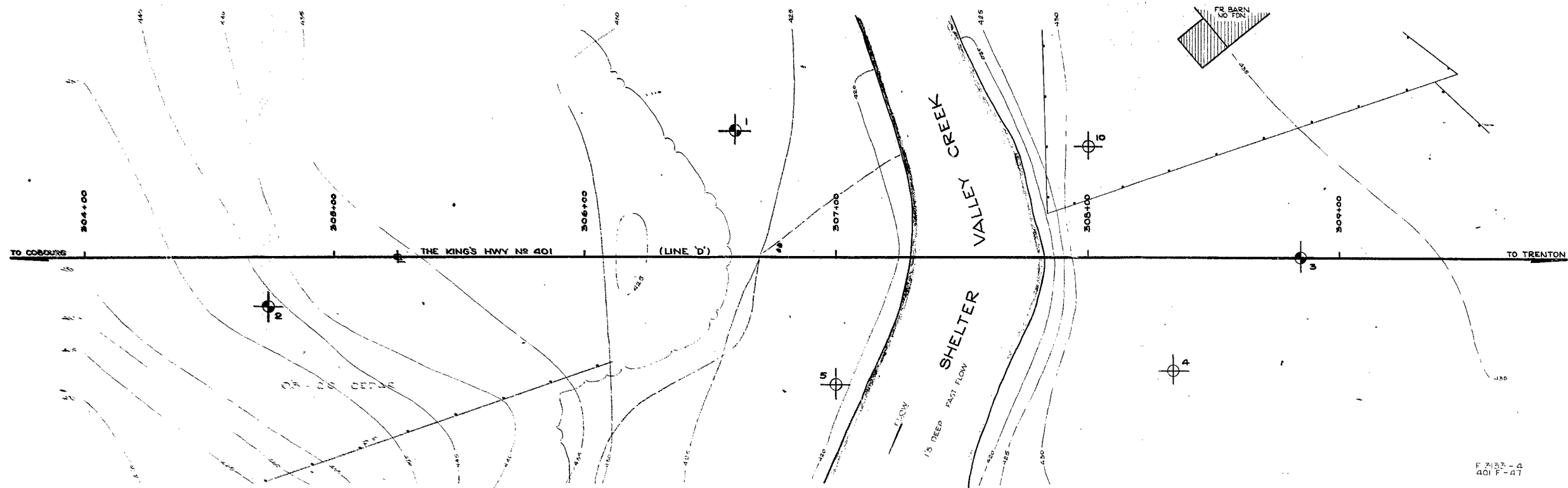
File (1)

W.P. 55-57

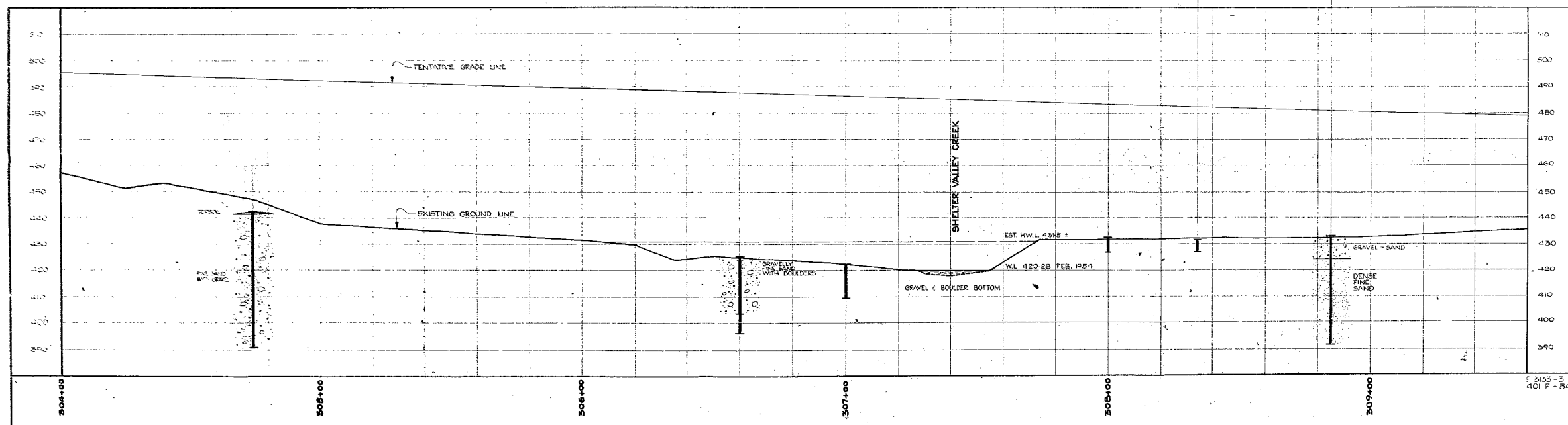
W.J. F-57-27

57-F-27
W.P. # 55-57
Hwy. # 401
CROSSING
SHELTER VALLEY
CR. - 2 MI. N.E. OF
GRAFTON

EDITED
FOR MICROFILMING
BY *LB* DATE *1/1/72*



PLAN SCALE 1 IN = 20 FT



PROFILE SCALE HOR VER 1 IN = 20 FT

LEGEND			
BORE HOLES			
PENETRATION HOLE			
BORE & PENETRATION HOLE			
HOLE NO.	ELEVATION	STATION	DISTANCE FROM &
1	425.65	306+60'	50' LT
2	442.5	304+74'	20' RT
3	437.25	308+65'	4
4	431.4	308+34'	45 RT
5	422.5	307+00'	51' RT
10	432.6	308+00'	44' LT

NOTE

THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS - ONTARIO		
MATERIALS & RESEARCH SECTION - DOWNSVIEW		
SHELTER VALLEY CREEK PROPOSED CROSSING 2 MILES N.E. OF GRAFTON SHOWING POSITION & ELEVATION OF HOLES		
HWY. NO. 401 (LINE 'D')	W.P. 55-57	DIV. NO. 7
CO. NORTHUMBERLAND		
TWP. HALDIMAND	LOT. 14	CON. 1
SCALE AS SHOWN	SUBMITTED BY	DATE 30 SEPT. 57
DRAWN BY R.E.F.	APPROVED BY	DRAWING NO. F-57-27A

Introduction:

A subsoil investigation was carried out to determine the bearing values of the layers for supporting the foundations of a proposed bridge and approach fills to the structure.

The location is where the new highway No. 401 crosses the M. J. Valley Creek about 2 miles North East of Grafton, Haldimand Township. (Station 307+00, Profile F-3133-4).

The job started on July 23, 1957 and was completed on August 2, 1957.

Procedure:

The subsoil investigations were carried out by means of a skid mounted coredrill machine. In the course of investigations one borehole with dynamic cone penetration (No. 1), and three dynamic cone penetrations (Nos. 4, 5, 10) were made separately to investigate for supporting the foundations. Also, two boreholes (Nos. 2 & 3) were made to investigate for approach fill stability.

The location of the boreholes is shown on drawing F-57-27A, and their elevations on log sheets under Appendix I.

Subsoil Findings and Analysis:

In this area the topography is characterized by large drumlins, some with steep slopes, cut by deep stream valleys.

The terrain is spillway deposit, filled with sand and gravel and large boulders. The investigations all across the valley revealed the same subsoil stratigraphy.

Boreholes No. 1 & 2 on the west side of the creek were made by driving the casing by means of BX casing shoe. While borehole No. 3 on the eastern side of the creek was driven down by hammering. From the holes, samples were extracted and tested in the laboratory. During sampling standard Penetration tests were also registered.

In all the samples the soil was identified as gravelly sand. The standard penetration tests indicated the very dense nature of the layer. The natural moisture content of the layer was found to be 14%.

The eroded banks and the washed down large boulders in and around the streambed picture the impressive scouring action taking place along the valley during flood water times.

CONCLUSIONS AND RECOMMENDATIONS:

From the above discussion it will follow that:

1. The terrain is spillway deposit. The subsoil is uniform layer of gravelly sand with large boulders down to elevation 391 ft.
2. From the standard penetration tests the layer starting at elevation about 420 ft. and down can provide a bearing value of 2.5-3 T.s.f. to support the spread footing foundations.
3. However, the scouring hazards at this site constitute an important problem. From calculations developed by the Connecticut Highway Department, it is found that the footings should be placed some 10 ft. below the bottom of stream elevation. This gives a safety factor of 4 against scouring hazards at this crossing.

4. It will be convenient to support the structure on spread footing type foundations. These footings, while they may be placed at elevation 420 ft., should not be placed higher than elevation about 410 ft., due to scouring hazards mentioned in (3) above. At this elevation the layer can provide a conservative 3 T.s.f. bearing value.
5. The subsoil layer can provide sufficient bearing value to support the amount of fill anticipated for the approaches to the structure.

V. Korlu,
Foundation Engineer.

APPENDIX I.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORING & PENIT'N JOB F-57-27 WP 55-57 BORING 1 STA. 306+60 (50' LT.)
CASING BX (standard samplers to fit unless noted) DATUM GEODITIC DATE REPORT SEPT 1957
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 23 JULY 1957

ABBREVIATIONS

- V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
Q_c - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

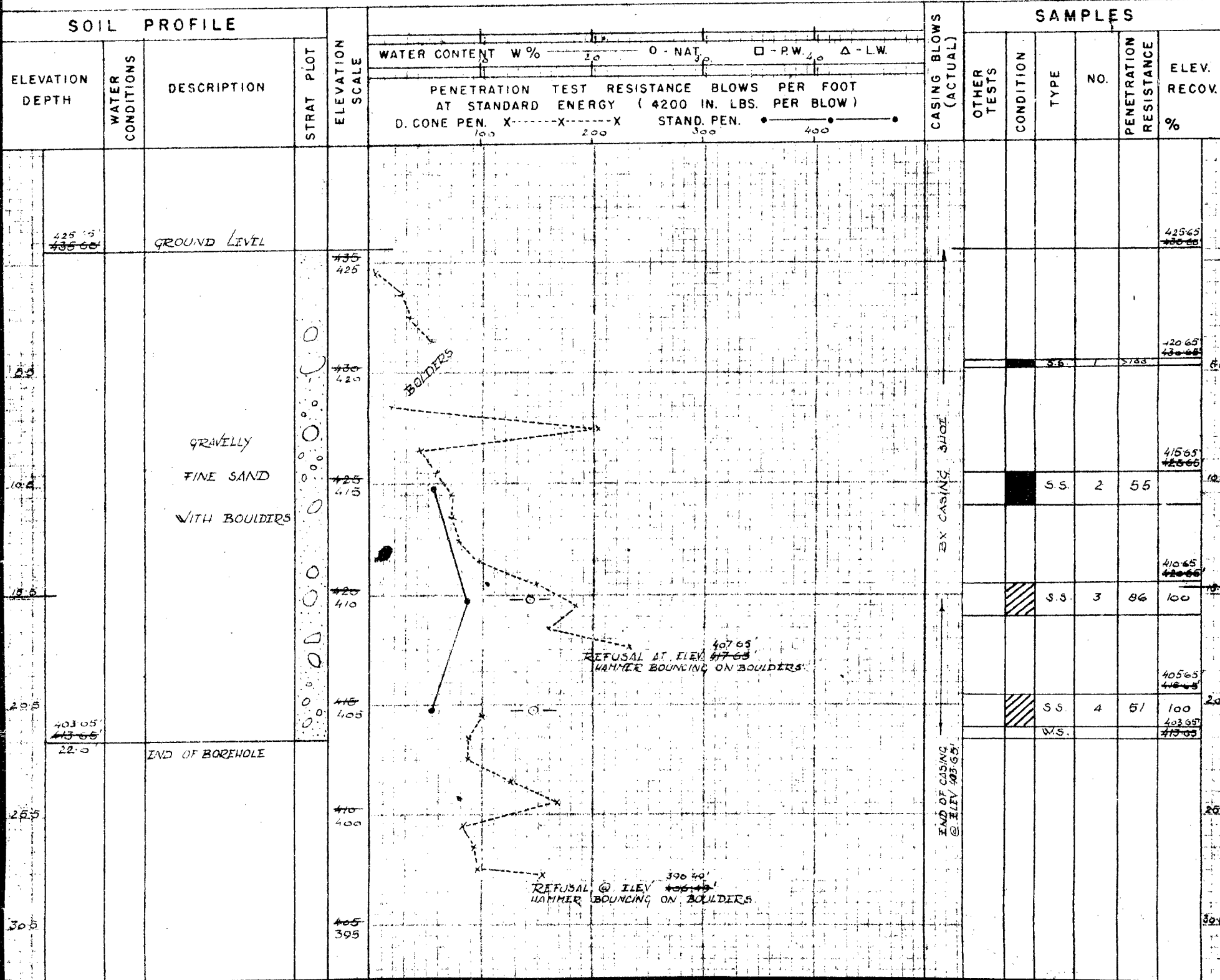
- C.S. - CHUNK S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN R.C. - ROCK CORE

SAMPLE CONDITION

-  - DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

SAMPLES



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORING & PENETRATION JOB F-57-27 W.P. 55-57 BORING 2 STA. 304+74 (20' BT)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT SEPT. 1957
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY AL CHECKED BY AL DATE BORING 25 JULY 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

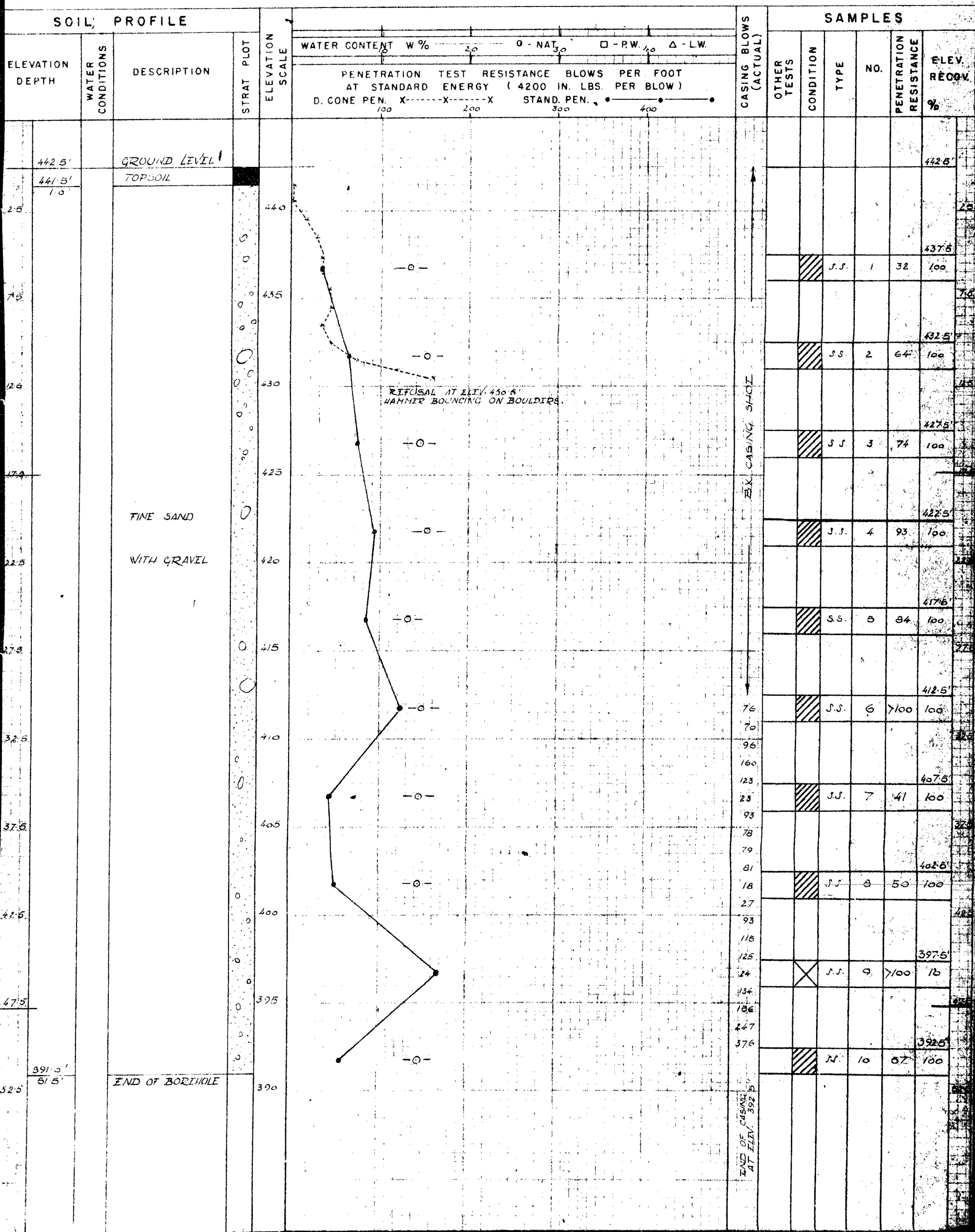
SAMPLE TYPES

C.S. - CHUNK S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN R.C. - ROCK CORE

SAMPLE CONDITION



- DISTURBED
- FAIR
- GOOD
- LOST



DRILL RIG 541 OPERATION BORE & PENETIN JOB F 57-27 W.P. 55 57 BORING 3 STA. 308 +85 ON 4
CASING BA (standard samplers to fit unless noted) DATUM GEODINIC DATE REPORT SEPT. 1957
SAMPLER HAMMER WT. 350 LBS. DROP 40 INCHES COMPILED BY H.S. CHECKED BY M.L. DATE BORING 31 JULY 1957

SAMPLE TYPES

SAMPLE CONDITION

V - INSITU VANE SHEAR TEST	Q - TRIAXIAL QUICK	K - PERMIABILITY
M - MECHANICAL ANALYSIS	S - TRIAXIAL SLOW	C - CONSOLIDATION
U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	CA - CASING
Q _c - TRIAXIAL CONSOLIDATED QUICK	WT - WATER TABLE IN SOIL	γ - UNIT WEIGHT

C.S. - CHUNK	S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN	P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE



- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLES

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W % 0 - NAT 30 □ - PW 40 △ - LW	PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND PEN.	CASING BLOW (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV.
												%	
433.25 437.26		GROUND LEVEL											433.25 437.26
3.5		GRAVEL-SAND		130 130	-O-								428.25 430.26
9.5	424.25 431.26 9.3'			425 430	-O-				X	DO	1	72	6.7
13.5				420 430	-O-								423.25 432.26
17.5		DENSE FINE SAND		415 425	-O-				/	J.S.	2	49	100
23.5				410 420	-O-								418.25 420.26
28.5				405 415	-O-				/	J.S.	3	38	33
33.5				400 410	-O-								413.25 423.26
38.5				395 405	-O-				/	J.S.	4	67	78
43.5	391.75 401.26 41.5'	END OF BOREHOLE		390 400	-O-				/	J.S.	5	98	100
													408.25 410.26
													403.25 413.26
									/	J.S.	6	>100	100
													398.25 408.26
									/	J.S.	7	90	100
													393.25 403.26
									/	J.S.	8	>100	100
													433.25 437.26

DRILL RIG 5- OPERATION PENETRATION JOB F-37-27 WP 55-57 BORING 4 STA. 308+34 (45' RT.)
CASING 31 (standard samplers to fit unless noted) DATUM GLIODETIC DATE REPORT SEPT 1957
SAMPLER HAMMER WT. 25 LBS. DROP 19 INCHES COMPILED BY 45 CHECKED BY 46 DATE BORING 30 JULY 1957

SAMPLE TYPES

V - INSITU VANE SHEAR TEST	Q - TRIAXIAL QUICK	K - PERMIABILITY
M - MECHANICAL ANALYSIS	S - TRIAXIAL SLOW	C - CONSOLIDATION
U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	CA - CASING
Q _c - TRIAXIAL CONSOLIDATED QUICK	WT - WATER TABLE IN SOIL	γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK	S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN	P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE

SAMPLE CONDITION



- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLES

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W %			CASING BLOW (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV.
					0 - NAT.	□ - P.W.	△ - L.W.							
					PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. ●-----●-----●									
					100 200 300 400									
42.0		GROUND LEVEL		430										
				426	REFUSAL AT ELEV. 427 HAMMER BOUNCING ON BOULDERS.									

DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION PENETRATION JOB F-57-27 W.P. 53-57 BORING 5 STA. 307+00 (51' BT.)
 CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT SEPT. 1957
 SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY A.I. DATE BORING 29 JULY 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
 QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

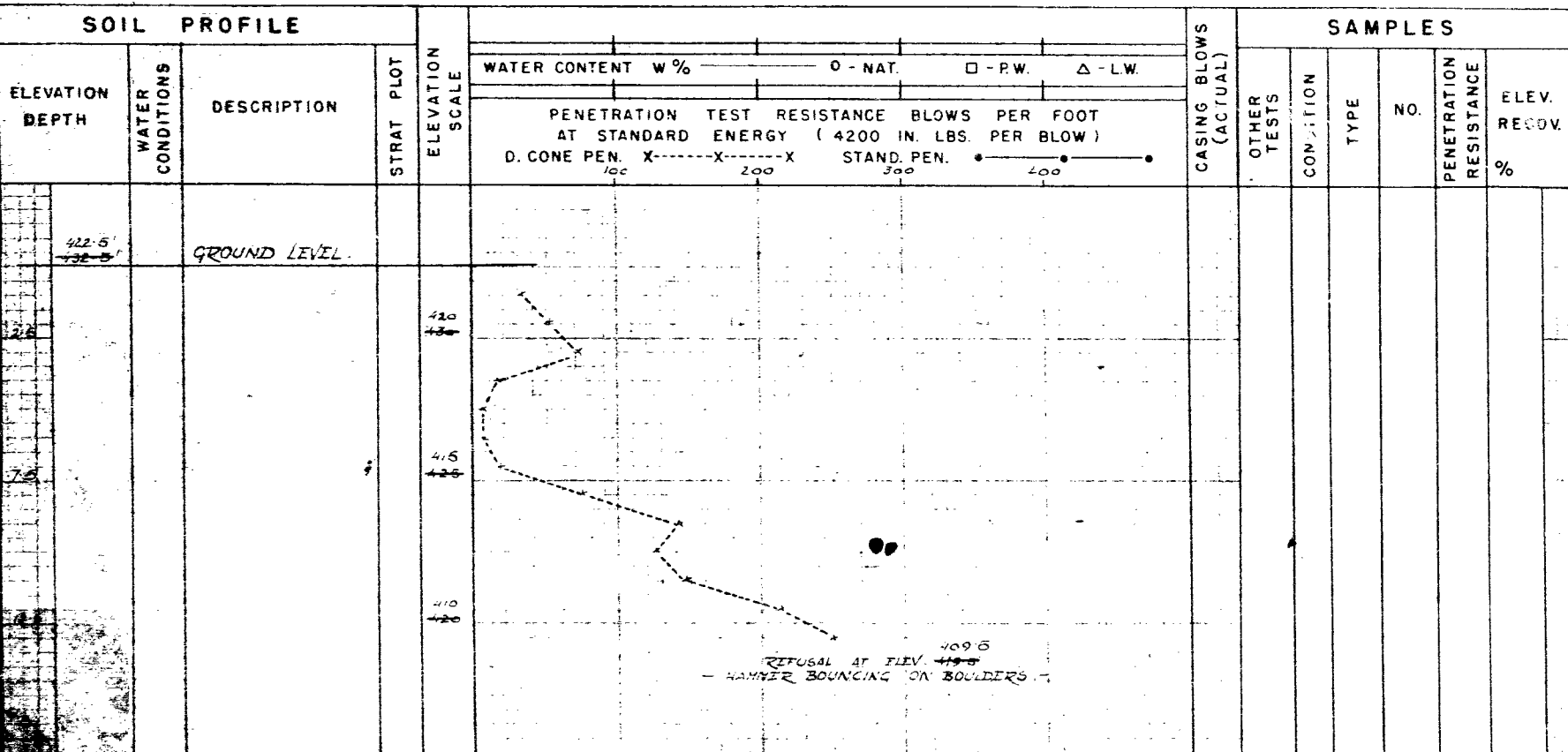
C.S. - CHUNK SS - SLEEVE SAMPLE
 D.O. - DRIVE OPEN PS - PISTON SAMPLE
 D.F. - DRIVE FOOT VALVE WS - WASHED SAMPLE
 T.O. - THIN WALLED OPEN R.C. - ROCK CORE

SAMPLE CONDITION



- DISTURBED
 - FAIR
 - GOOD
 - LOST

SOIL PROFILE



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL NO. 54-1 OPERATION PENETRATION
CASING BK (standard samplers to fit unless noted)
SAMPLER HAMMER WT. 350 LBS. DROP 19 INCHES

JOB F-57-27 W.P. 55-57
 DATUM GEODETIC
 COMPILED BY H.S. CHECKED BY A.L.

BORING 10 STA. 308+00 (44' LT.)
DATE REPORT SEPT. 1957
DATE BORING 14 AUG. 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
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SAMPLE TYPES

C.S. - CHUNK	S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN	P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE

SAMPLE CONDITION

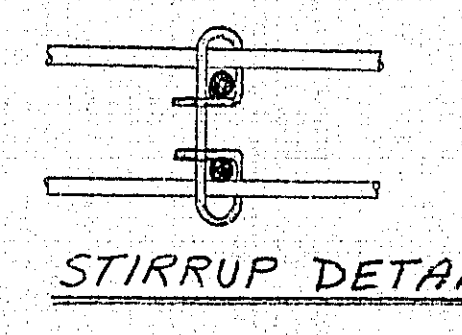
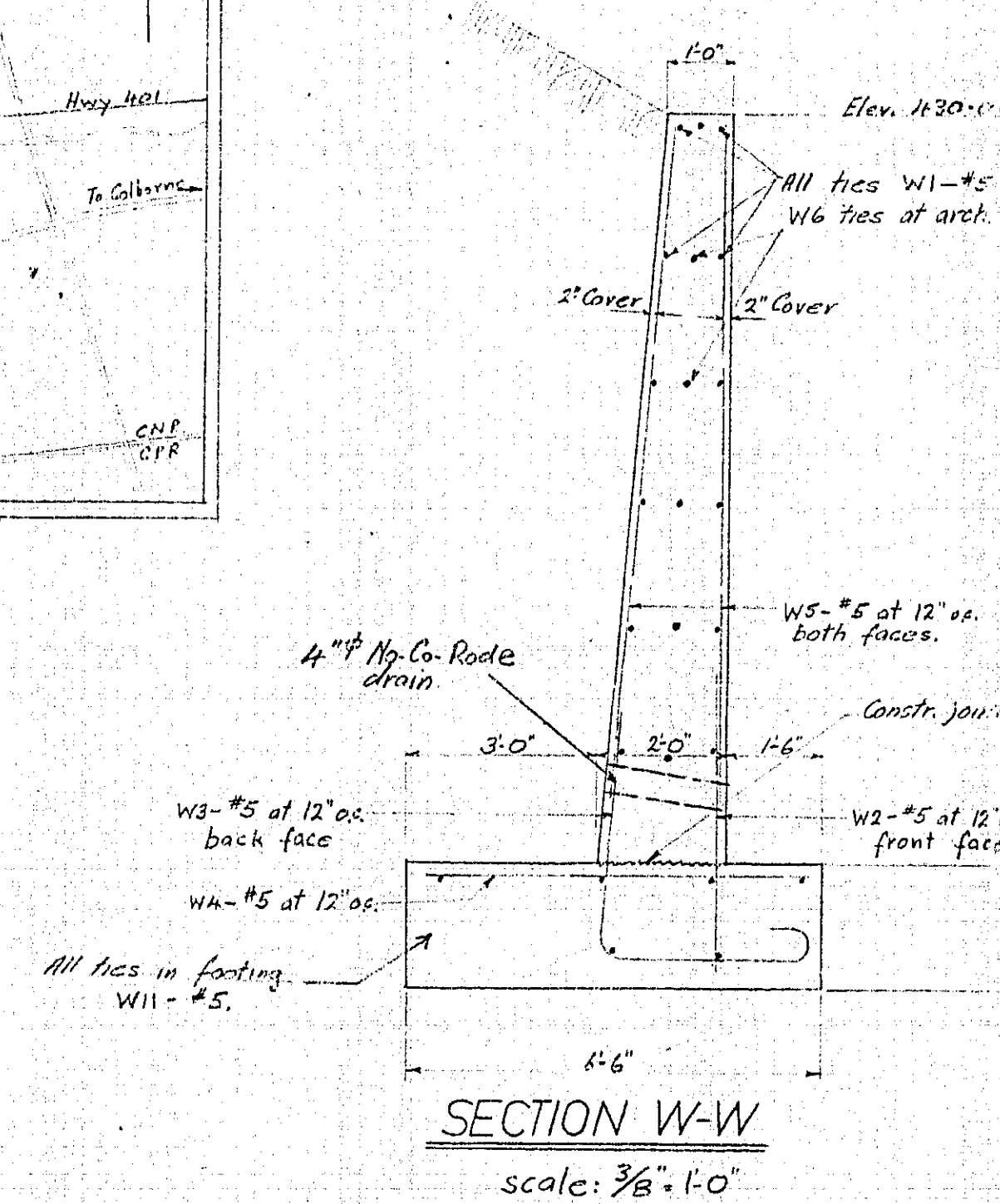
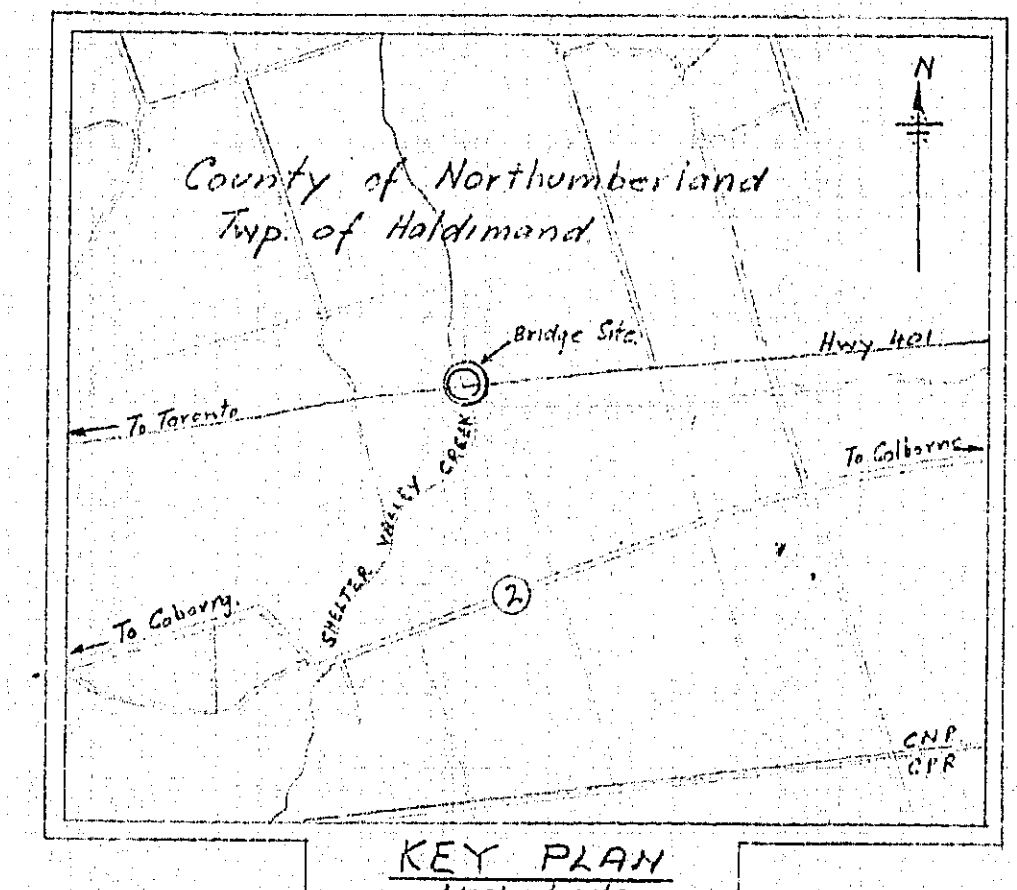
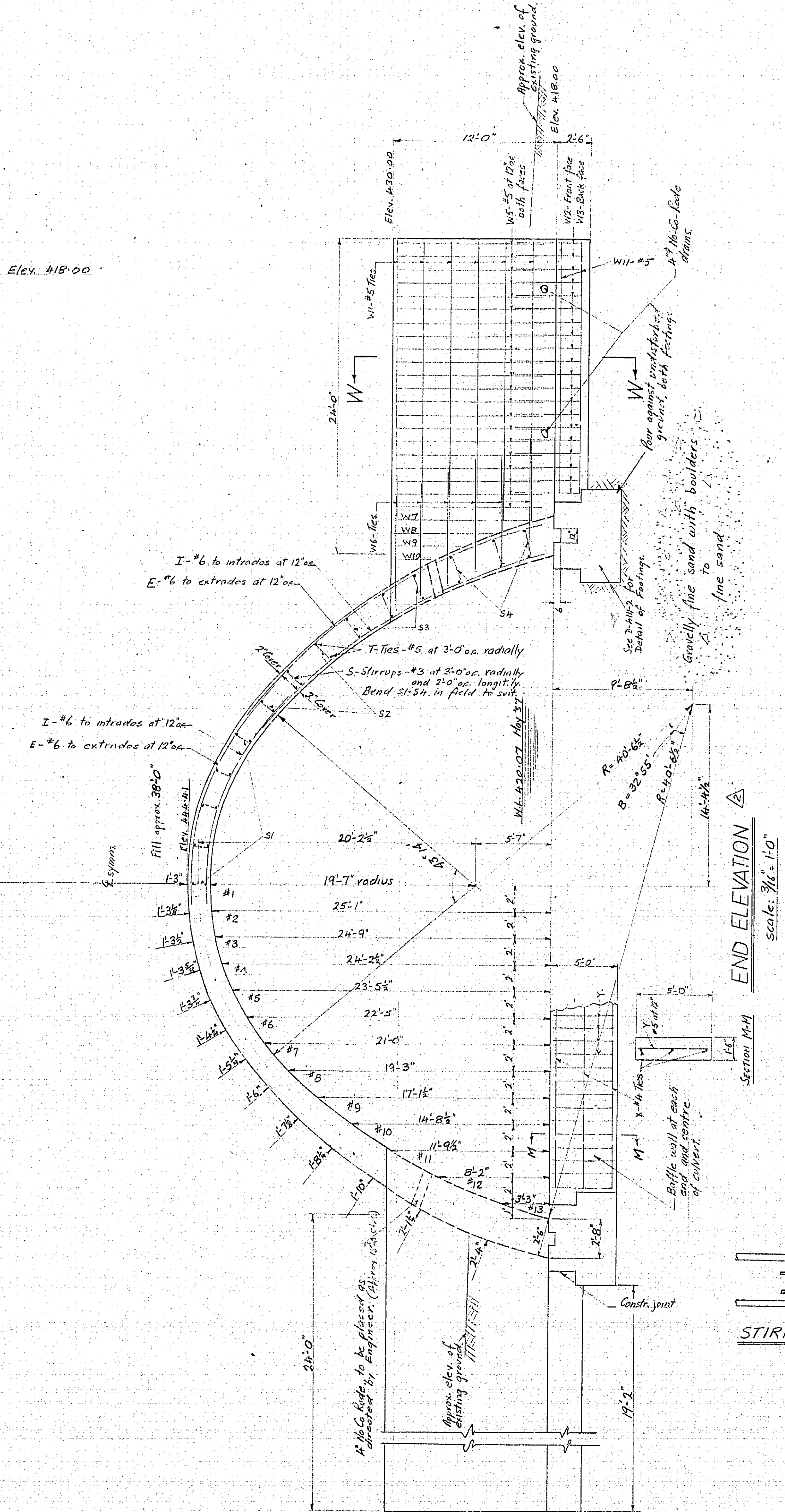
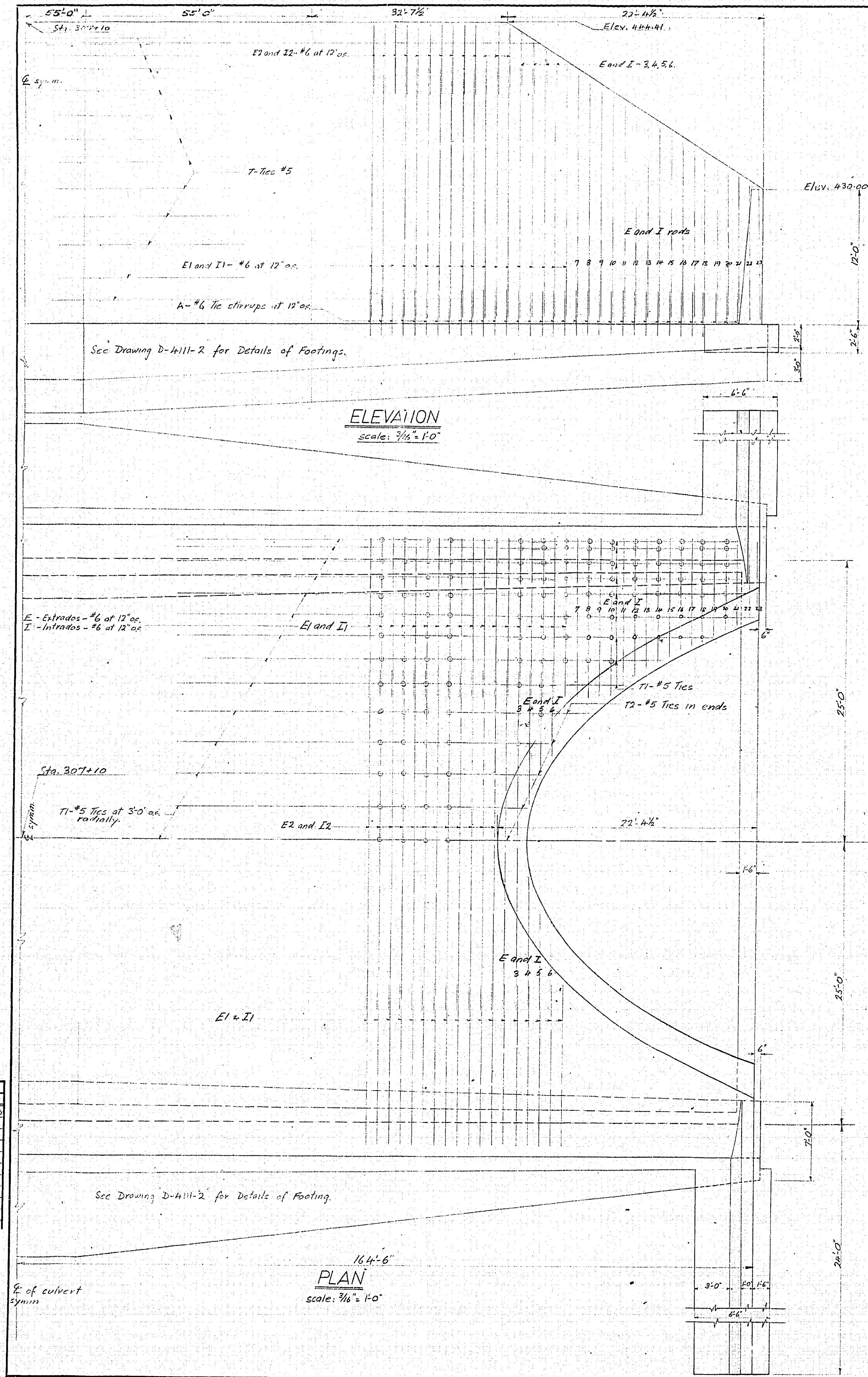


- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

SOIL PROFILE						SAMPLES						
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W% ———— O - NAT. □ - P.W. Δ - L.W. PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. ●————●————● 100 200 300 400	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
		GROUND LEVEL		432.6								
25				430								
75				426	REFUSAL AT ELEV. 437.1 HAMMER BOUNCING ON BOULDERS							

PRINT RECORD		
NO.	FOR	DATE
9	F.S.D.	8-5-58
11	F.S.D.	2-6-59
30	REVISION	8-10-58
31	REVISION	4-23-59



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 CONTRACTOR
Structure to be built in accordance with Form No. 2, revised March 1957 and the Special Provisions, extra copies of which may be obtained from the District Engineer. All construction joints must be approved by the Bridge Engineer. Particular attention is to be given to the expansion joints, see D-4111-2.

CONCRETE MIX
All concrete in footings to be 2500 p.s.i. at 28 days. Arch and wing wall concrete to be 3000 p.s.i. at 28 days. Maximum size of aggregate to be 3/4".

BORING DATA
The complete soil report BA 673 may be examined at the D.H.O. Bridge Design Office, 380 Davenport Road, Toronto. The Dept. does not guarantee the accuracy of this report or the abridged version shown on these plans.

REINFORCING STEEL
Clear cover in footings to be 3" and is noted. Clear cover elsewhere to be 2". All exposed edges of concrete to be chamfered 1".

WP 55-57

DEPARTMENT OF HIGHWAYS, ONTARIO
BRIDGE OFFICE - TORONTO

SHELTER VALLEY CREEK
HALDIMAND TWP. BRIDGE NO. 14

THE KING'S HIGHWAY NO. 401 DIST. NO. 7
CO. Northumberland Sta. 307+10
TWP. Haldimand LOT CON.

GENERAL PLAN

APPROVED *Alan L. ...* DESIGN ENGINEER

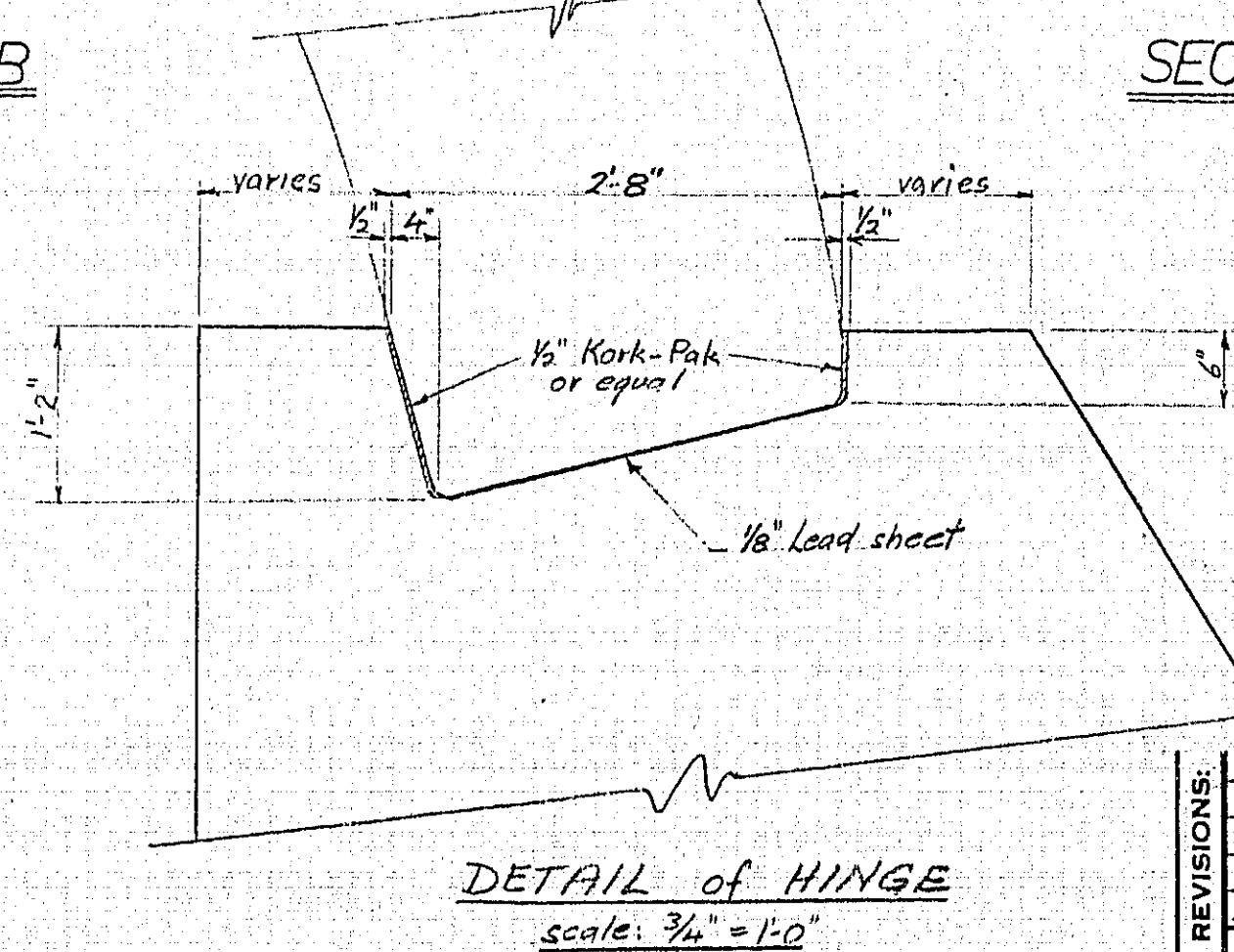
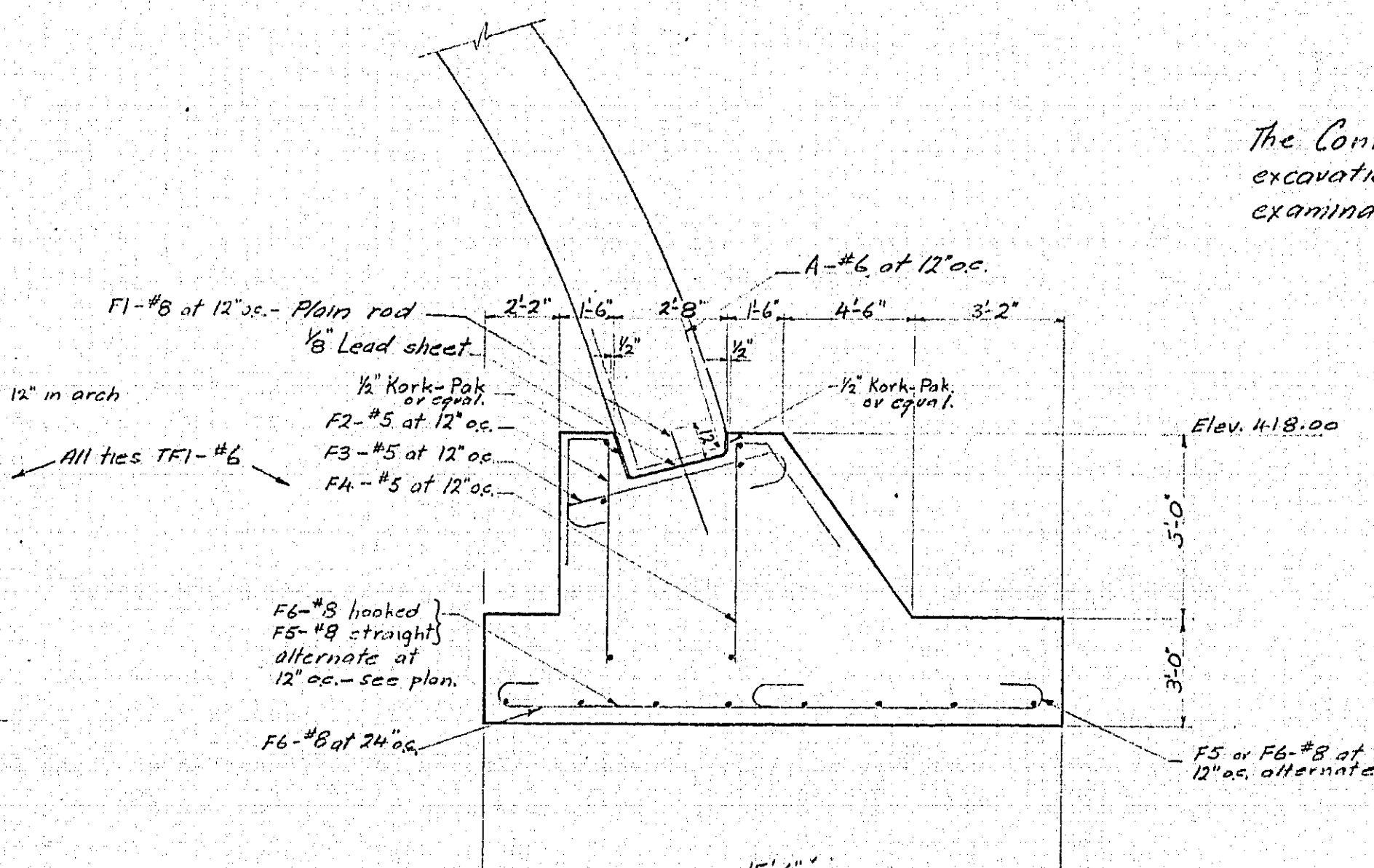
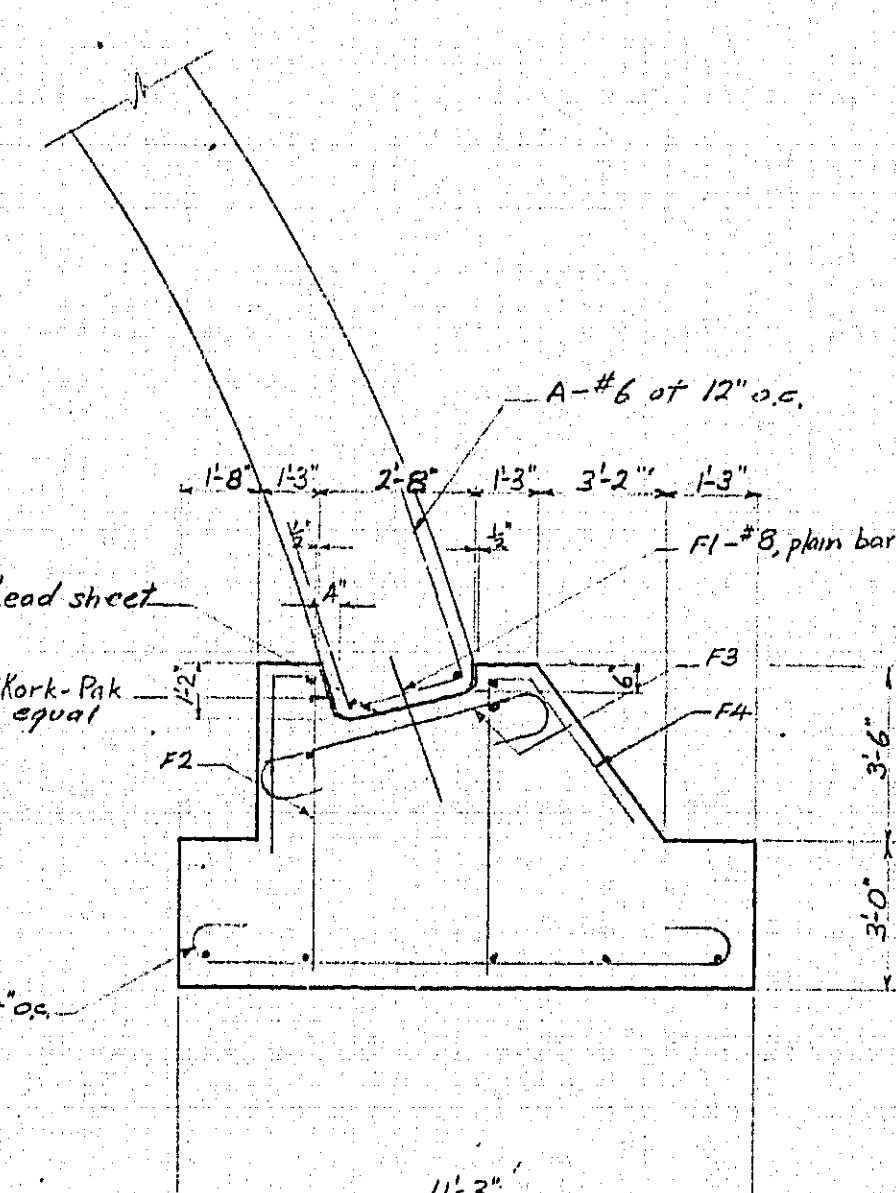
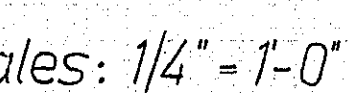
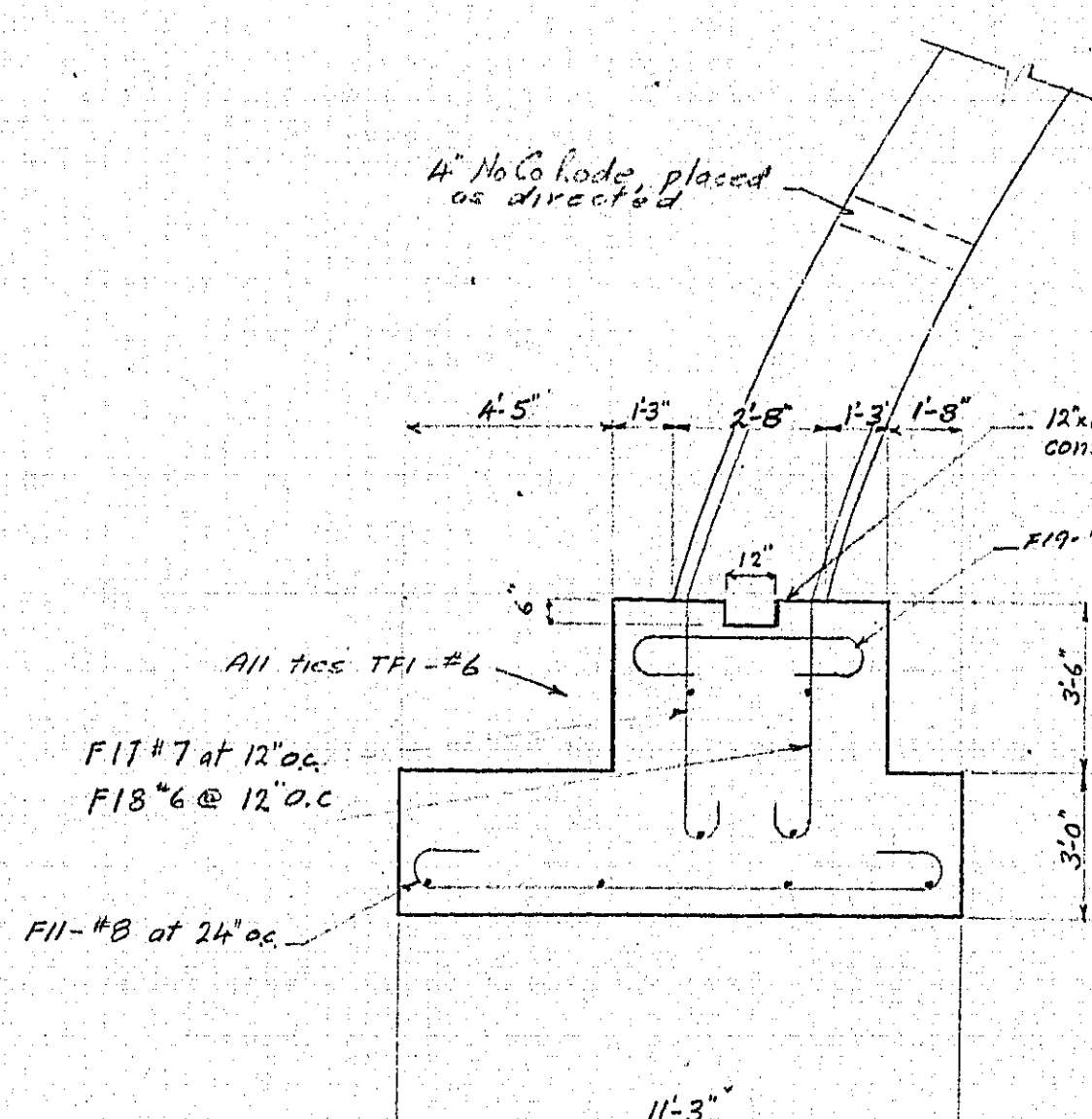
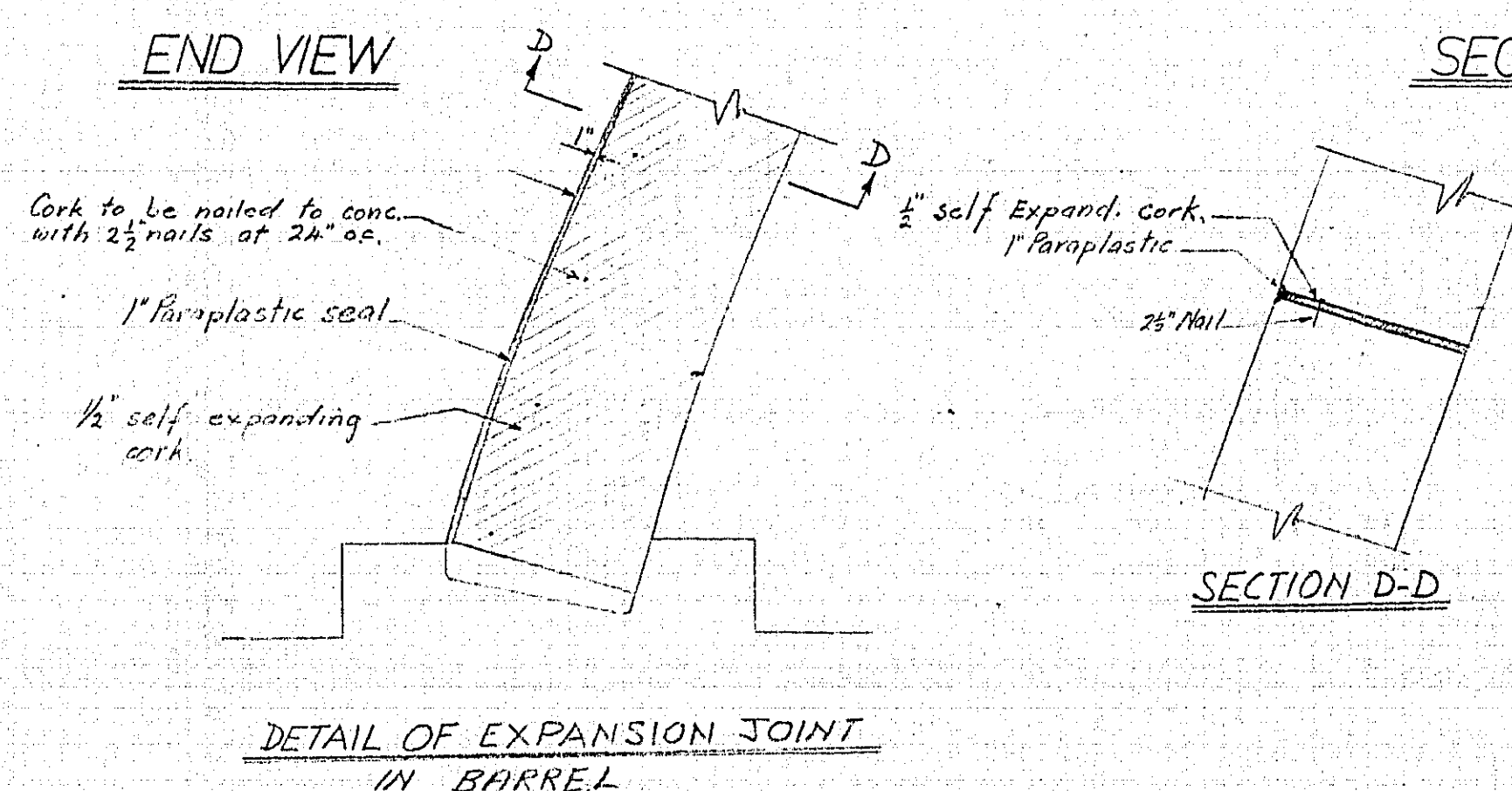
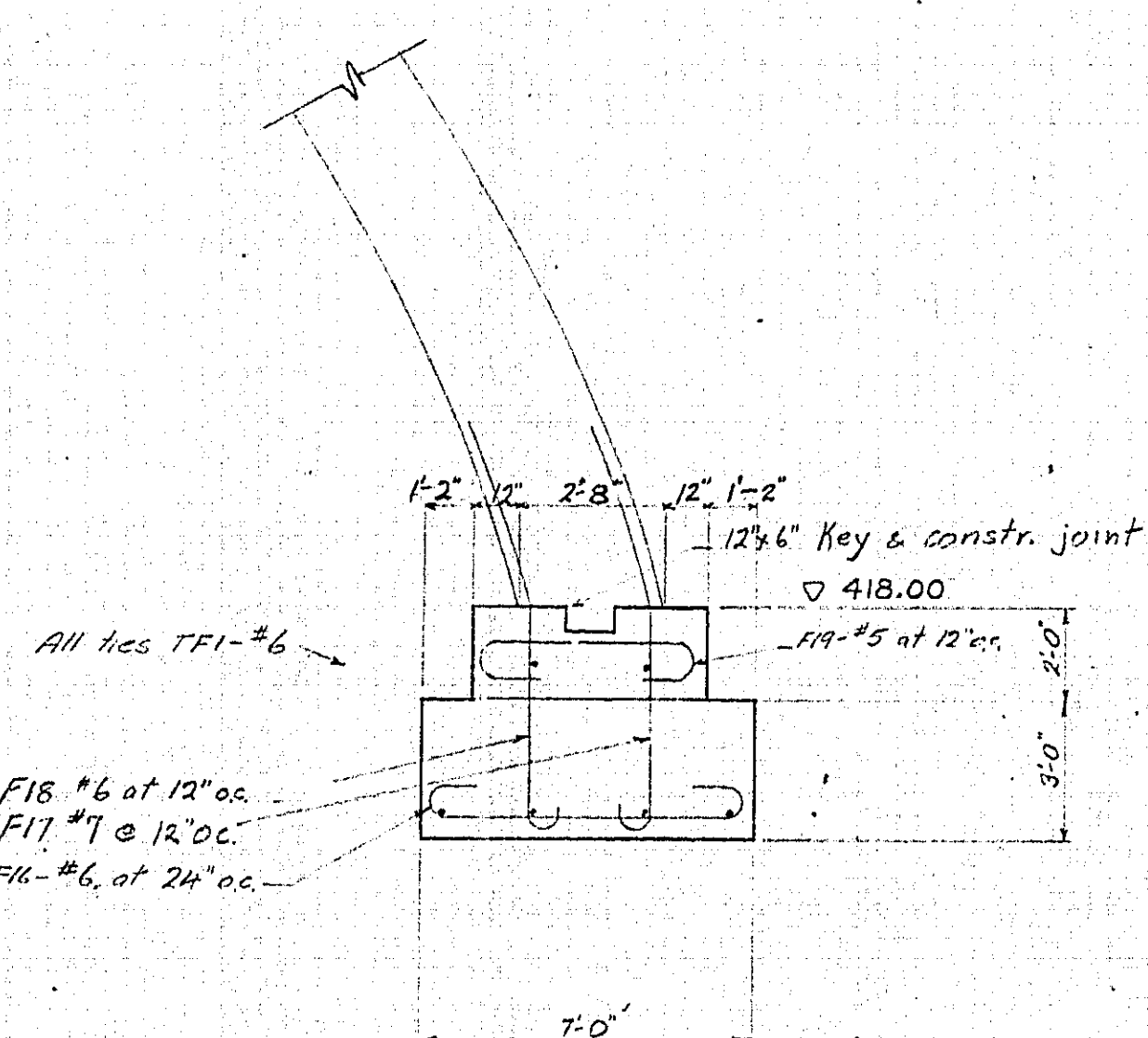
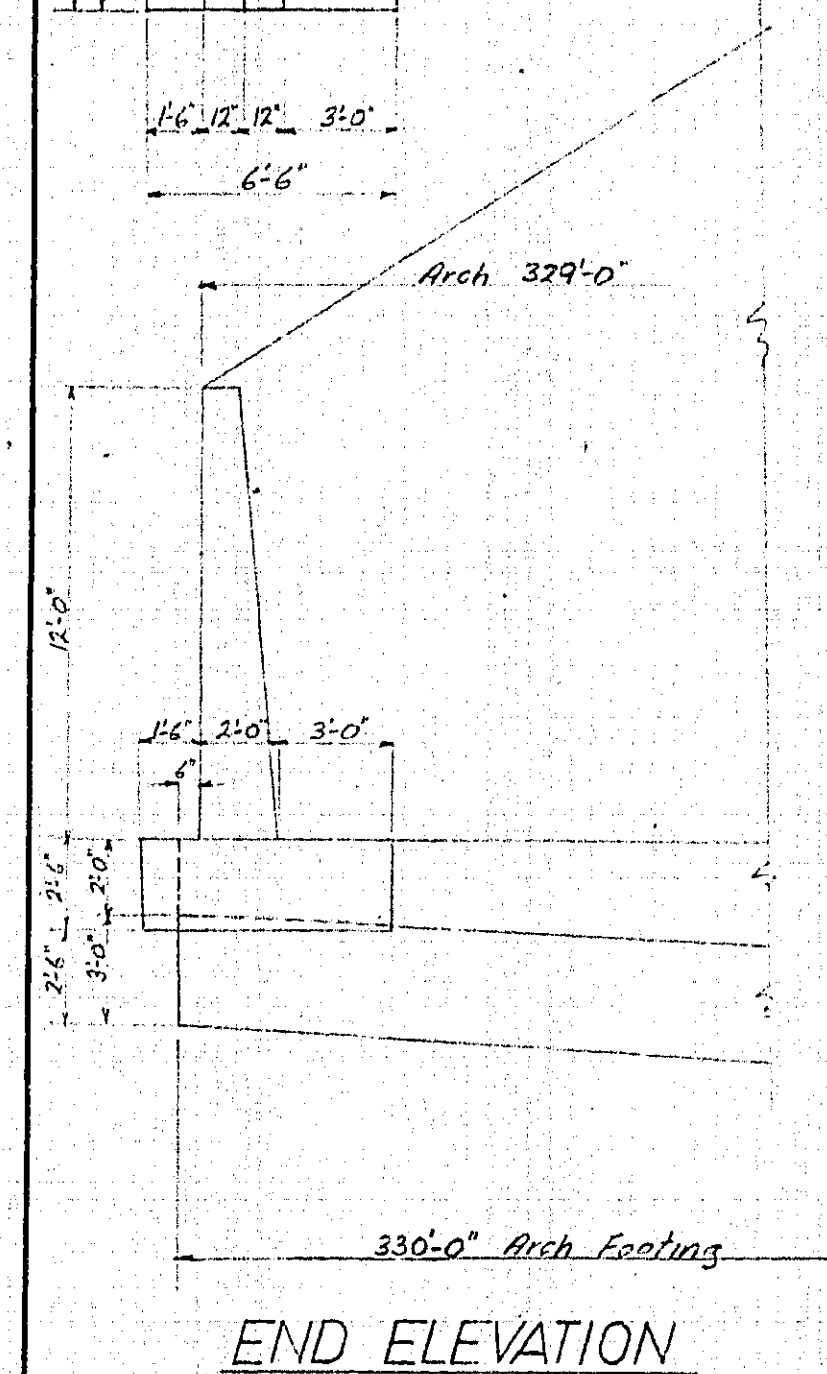
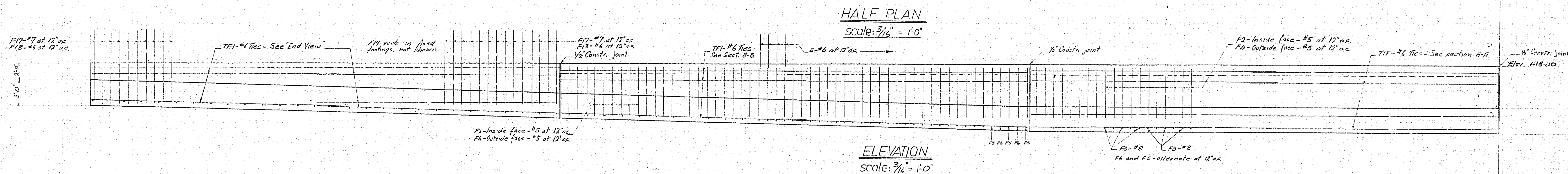
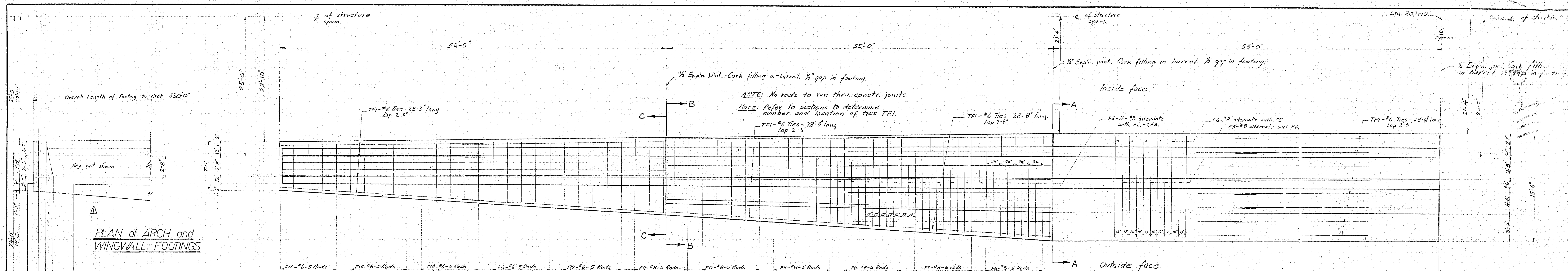
BRIDGE ENGINEER

DESIGN	CHECK	WITH	CONTRACT NUMBER	5-3-291
DRAWING	CHECK	WITH	LOADING	
DATE	CHECK	WITH	DRAWING NUMBER	D-4111-1

DATE MAY 1958

REVISIONS	DATE	BY	DESCRIPTION
12-7-63	J.G.G.		ADDITIONAL INFORMATION TO END ELEVATION.
2-6-68	H.G.		Design Revision (REV. NO. OF CORR.)

TWP# 9-272-1-A 1 to 4



The Contractor to notify the Design Engineer of this structure when excavation of footings is to be completed, in order that an examination may be made before any concrete is poured.

[illegible]

DEPARTMENT OF HIGHWAYS: ONTARIO

BRIDGE OFFICE: TORONTO

SHELTER VALLEY CREEK

HALDIMAND TWP BRIDGE NO. 14.

THE KING'S HIGHWAY No. 401

DIST. NO. 7

Co. Northumberland

Sta. 307+10

TWP. Haldimand

LOT

CON.

FOOTING DETAILS

APPROVED

BRIDGE ENGINEER

DESIGN	<u>SKP</u>	CHECK	<u>MS</u>
DRAWING	<u>HB</u>	CHECK	<u>SKP</u>
TRACING		CHECK	
DATE	<u>MAY 1958</u>		

LOADING

#20

Twp # 9-252-2-A

25.1.1

MARK	NO. BARS	SIZE	LENGTH	TYPE	A	B	C	D	E	F	G	H	J	K	O	R	SHAPE	LOCATION
E2	290	#6	33'-8"	9			30'-5"					6'-4"					21'-1 1/2"	Extrados of arch at 12' o.c.
E3	4		10'-10"				10'-8 1/2"					8"						ditto
E4	4		8'-9"				8'-5"					5 1/2"						ditto
E5	4		6'-6"				6'-8 1/2"					3 1/2"						ditto
E6	4	#6	5'-0"	9			4'-11 1/2"					2"					21'-1 1/2"	ditto
I2	290	#6	32'-4"	9			28'-10"					6'-3"					19'-9 1/2"	Intrados of arch at 12' o.c.
I3	4		32'-4"				28'-10"					5"						ditto
I4	4		8'-3"				8'-2"					3"						ditto
I5	4		6'-0"				5'-11 1/2"					1 1/2"						ditto
I6	4	#6	4'-6"	9			4'-5 1/2"					1 1/2"					19'-9 1/2"	ditto
E1	600	#6	26'-3"	9			25'-9"					1'-10"					42'-6"	Extrados of arch at 12' o.c.
E7	4		26'-0"				25'-6 1/2"					1'-11 1/2"						ditto
E8	4		25'-6"				25'-0 1/2"					1'-10 1/2"						ditto
E9	4		24'-1"				23'-8 1/2"					1'-8 1/2"						ditto
E10	4		23'-3"				22'-10 1/2"					1'-7"						ditto
E11	4		22'-1"				21'-9"					1'-5"						ditto
E12	4		21'-5"				21'-1 1/2"					1'-4"						ditto
E13	4		20'-4"				20'-0 1/2"					1'-2 1/2"						ditto
E14	4		19'-9"				19'-5"					1'-1 1/2"						ditto
E15	4		18'-8"				18'-5 1/2"					1'-0 1/2"						ditto
E16	4		17'-10"				17'-7 1/2"					11"						ditto
E17	4		16'-11"				16'-8 1/2"					10 1/2"						ditto
E18	4		16'-2"				16'-0"					9 1/2"						ditto
E19	4		15'-3"				15'-1 1/2"					8 1/2"						ditto
E20	4		14'-6"				14'-4 1/2"					7 1/2"						ditto
E21	4		13'-10"				13'-9"					6 3/4"						ditto
E22	4		13'-0"				12'-11"					6"						ditto
E23	4	#6	12'-6"	9			12'-5"					5 1/2"					42'-6"	Intrados of arch at 12' o.c.
I1	600	#6	24'-9"	9			24'-5"					1'-10"					40'-9"	Intrados of arch at 12' o.c.
I7	4		26'-6"				25'-11 1/2"					2'-2"						ditto
I8	4		26'-0"				25'-6"					2'-1"						ditto
I9	4		24'-9"				24'-3 1/2"					1'-10 1/2"						ditto
I10	4		23'-9"				23'-4 1/2"					1'-8 1/2"						ditto
I11	4		22'-9"				22'-5 1/2"					1'-7"						ditto
I12	4		22'-0"				21'-8 1/2"					1'-5 1/2"						ditto
I13	4		21'-0"				20'-8 1/2"					1'-4 1/2"						ditto
I14	4		20'-3"				20'-0"					1'-3 1/2"						ditto
I15	4		19'-3"				19'-0 1/2"					1'-1 1/2"						ditto
I16	4		18'-6"				18'-3 1/2"					1'-0 1/2"						ditto
I17	4		17'-6"				17'-3 1/2"					11 1/2"						ditto
I18	4		16'-9"				16'-7"					10 1/2"						ditto
I19	4		15'-9"				15'-7 1/2"					9 1/2"						ditto
I20	4		15'-0"				14'-10 1/2"					8 1/2"						ditto
I21	4		14'-6"				14'-4 1/2"					8"						ditto
I22	4		14'-0"				13'-10 1/2"					7 1/2"						ditto
I23	4	#6	13'-6"	9			13'-5"					6 3/4"					40'-9"	ditto
X	18	#4	23'-6"	Str.													Lap 1'-6" Min.	Ties in buffer wall, lap 1'-6" in buffer wall or 12' o.c.
Y	135	#5	4'-6"	Str.														
A	444	#6	10'-2"	4		4'-0"	2'-2"					4'-0"						Bottom of arch at footing of 12' o.c.

MARK	NO. BARS	SIZE	LENGTH	TYPE	A	B	C	D	E	F	G	H	J	K	O	R	SHAPE	LOCATION
T1	262	#6	28'-8"	Str.														Ties in footing lap 2'-0"
F1	444	#5	3'-0"	Str.														In hinge at footing of 12' o.c. as shown
F2	444	#5	10'-3"	5	3'-6"	9"	6'-0"											In footing at 12' o.c. as shown
F3	444	#5	6'-6"	1	7'	5'-4"												ditto (hooked)
F4	444	#5	10'-6"	6	6'-0"	1'-0"	3'-6"											ditto
F5	176	#8	9'-0"	1	1'-1"	6'-10"												In bottom of footing all with P.E. at 12' o.c.
F6	132	#8	16'-8"	1	1'-1"	14'-6"												In bottom of footing all with P.E. at 12' o.c.
F7	20	#8	15'-8"	1	1'-1"	13'-6"												In bottom of footing all with P.E. at 12' o.c.
F8	20	#8	14'-8"	1	1'-1"	12'-6"												In bottom of footing all with P.E. at 12' o.c.
F9	20	#8	14'-0"	1	1'-1"	11'-0"												In bottom of footing all with P.E. at 12' o.c.
F10	20	#8	13'-2"	1	1'-1"	11'-0"												In bottom of footing all with P.E. at 12' o.c.
F11	20	#8	12'-5"	1	1'-1"	10'-3"												ditto
F12	20	#6	10'-10"	1	8"	9'-6"												ditto
F13	20	#6	10'-1"	1	8"	8'-9"												ditto
F14	20	#6	9'-4"	1	8"	8'-0"												ditto
F15	20	#6	8'-7"	1	8"	7'-3"												ditto
F16	20	#6	7'-10"	1	8"	6'-6"												ditto
F17	220	#7	9'-4"	11	4'-0"	4'-6"												In footing & arch extrados at 12' o.c.
F18	220	#6	9'-2"	12	4'-0"	4'-6"												In footing & arch intrados at 12' o.c.
F19	220	#5	6'-2"	1	7"	5'-0"												In footing at 12' o.c.
W1	48	#5	21'-0"	Str.														Ties in wing-wall as shown
W2	80		4'-9"	Str.														In wing-wall at 12' o.c. front face
W3	80		8'-4 1/2"	14	4'-3"	2'-9"	9 1/2"											In wing-wall at 12' o.c. back face
W4	80		6'-0"	Str.														In wing-wall at 12' o.c.
W5	168	#5	11'-6"	Str.														In wing-wall at 12' o.c. front & back faces
W6	24	#6	9'-0"	2	6"	8'-6"												Ties in wing-wall as shown
W7	8	#5	10'-3"	Str.														In wing-wall near arch, as shown
W8	8		7'-4"	Str.														ditto
W9	8		5'-0"	Str.														ditto
W10	8	#5	2'-9"	Str.														ditto
W11	24	#5	19'-3"	Str.														6" IN EACH FTNG. AS SHOWN.
T1	960	#5	19'-8"	Str.														Ties in barrel as shown
T2	40	#5	12'-0"	Str.														Ties in ends lap to suit with T1, minimum 2'-0"

ALL STEEL PLACED IN ARCH.
ALEX. M. CRAE

1. All dimensions are cut to out of bar.

2. "J" dimension on 180° hooks to be shown only where necessary to restrict hook size, otherwise standard hooks to be used.

3. Where "J" is not shown, "J" will be kept equal to or less than "H", where "J" is shown, "H" should be shown.

4. "K" dimension on stirrups to be shown where necessary to restrict hooks.

5. Where bars are to be bent more accurately than standard bending tolerances, bending dimensions which require close working should have limits indicated.

6. Figures in circles show types.

ENLARGED VIEW SHOWING BAR BENDING DETAILS

11. 12. 13. 14.

ALL BARS TO BE DETAILED AS PER A.C.I. SPECIFICATIONS

ALL STEEL TO BE HARD GRADE & HIGH BOND EXCEPT AS NOTED

WP 55-57

DEPARTMENT OF HIGHWAYS - ONTARIO

BRIDGE - OFFICE - TORONTO

SHELTER VALLEY CREEK

HALDIMAND TWP. BR. NO. 14

THE SING. HWY. NO. 221 DIV. NO. 1

CO. Northumberland Sta. 307+10

TWP. Haldimand LOT CON.

REINFORCING STEEL SCHEDULE

WEIGHT OF STEEL 163.456 LBS

CONTRACT NUMBER R 35127

DRAWING NUMBER D-4111-3

DATE MAY 1958

TWP # 9-272-3-A

Appendix B
Site Photographs



Photo 1 North End of Shelter Valley Creek Culvert



Photo 2 South End of Shelter Valley Creek Culvert



Photo 3 South End of Shelter Valley Creek Culvert showing Hwy 401 Embankment



Photo 4 Interior of Shelter Valley Creek Culvert



Appendix E

List of Standard Specifications and Special Provisions



1) The following Standard Specifications and Special Provisions are referenced in this memo:

- OPSS 539
- OPSS 902

2) Recommended wording for “NSSP – Dewatering”

Excavation of cohesionless soils below the water level must be carried out with a properly designed cofferdam and dewatering system in place to lower groundwater table to a minimum of 0.5 m below the base of excavation and to prevent boiling of the footing base. An effective cofferdam and dewatering system may include watertight sheet pile enclosure and positive well points. The dewatering system must remain operational until the retaining wall is constructed and backfilled. The Contractor is responsible for design of the dewatering system and shall retain a dewatering specialist for design.

Concrete repair work proposed inside the arch culvert may require that the water level be depressed below the top of the footing. This can be achieved through the use of a cofferdam and pumping. The design of this system is also the responsibility of the Contractor but the contract documents must include constraints to guard against the loss of soil below the existing foundations.

It is recognized that there are constraints on the type of equipment that can be operated inside the culvert in order to construct a cofferdam. Consequently, it is possible that the Contractor will choose a system such as sand bags with an impermeable membrane in front to restrict the inflow of water from the stream, while pumping from inside the cofferdam. While a system like this may be made to work, the permeable nature of the soils means that there will be potentially large inflows of water from the base of the work area behind the cofferdam.

Whatever cofferdam/unwatering system is selected, it must be designed and operated so as to prevent the loss of soil from below the footing and to prevent heaving or boiling of the base of the enclosed work area. It is essential that this system be designed by professionals specializing in dewatering systems and the design must include, among other issues, the following:

- The unbalanced head of water acting upward on the base of the enclosed area, taking account of the design high water level, or such higher level as the designer considers to be prudent.
- The exit gradient of the groundwater and the potential for boiling, heaving or piping.

The measures to address these issues must be detailed by the Contractor's designer, but typically it may require the use of a filter on the base of the work area and/or a temporary concrete slab or granular blanket to protect and weigh down the filter. Under no circumstances may any



THURBER

excavation or other work be carried out that undermines, or may undermine, the existing foundations.