



THURBER ENGINEERING LTD.

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
RUSHING RIVER BRIDGE
HIGHWAY 71, KENORA DISTRICT
W.P. 6818-14-01, SITE 41S-061**

Geocres Number: 52E-067

Report to

McIntosh Perry

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PART 1: FACTUAL INFORMATION

1.0 INTRODUCTION

This report presents a description of the subsurface conditions anticipated at the location of the bridge carrying Highway 71 over Rushing River in the Township of Kirkup, District of Kenora. The description is based solely on information shown on/in the following documents:

Plan of Proposed Bridge over Rushing River, Drawing B1990-A, May 19, 1936.

Rushing River Structure Widening - General Plan, Drawing 5961-1; and Details, Drawing 5961-2. January 1967.

Foundation Investigation Report for Rushing River Bridge Replacement, W.P. 780-91-01, Site 41S-61, Highway 71, District 61, Thunder Bay. August 1996.
Geocres No. 52E-38.

The purpose of the report is to present existing subsurface data from MTO Geocres files, and a written description of the subsurface conditions at the site.

Thurber completed the report as a sub-consultant to McIntosh Perry who are completing the preliminary design of a replacement bridge under the Ministry of Transportation Ontario (MTO) Agreement Number 6017-E-0001.

2.0 SITE DESCRIPTION

The existing Rushing River Bridge on Highway 71 south of Highway 17 consists of a three span, three lane timber structure with a total length of 17.7 m and a width of 13.7 m. The span lengths are 8.2 m between the piers and 4.6 m at the abutments. The bridge is supported on concrete pedestals placed on bedrock. Based on the General Arrangement drawing for the existing



bridge, road grades on the bridge deck are at Elev. 339.5. Photographs of the bridge are presented in Appendix A.

Rushing River flows southwesterly at the bridge site, from Dogtooth Lake to Blindfold Lake. The river water passes approximately 3.5 m below the deck. Bedrock is exposed on the river banks, and rock fill has been placed to form the bridge approach embankments.

The site is located within the Rushing River Provincial Park, a recreational park located along a series of water rapids on Rushing River. The topography is generally one of subdued relief with exposed bedrock outcrops and small bogs developed within depressions in the bedrock surface. The surrounding area is treed with jackpine and aspen, with the exception of a cleared hydro corridor immediately to the west of the site.

The geology of the area generally consists of exposed Precambrian bedrock locally overlain by a veneer a sandy till ground moraine with cobbles and boulders.

3.0 DESCRIPTION OF SUBSURFACE CONDITIONS

A preliminary description of the subsurface conditions at the site, based on information shown on the historical structure drawings and contained in a previous Foundation Investigation Report, is presented below. No boreholes have been advanced at the site to confirm the stratigraphy and bedrock quality.

Reference is made to Drawings B1990-A (May 1936), D-5961-1 & 2 (January 1967), and 7809101-B (August 1996) reproduced in Appendices B to D for details of the soil stratigraphy and bedrock levels anticipated at the site. In general terms, the subsurface stratigraphy is expected to consist of rock fill overlying a layer of sandy till (under the approaches) and bedrock. Further details are as follows:

- The thickness of the rock fill placed in the approaches varies from approximately 1.0 to 5.0 m.
- Up to 1.5 m of till described as a “heterogeneous mixture of silt, sand and gravel” is present below the rock fill in the approaches beyond the abutments.
- Bedrock underlies the sandy till and rock fill, and is exposed along the river channel. The bedrock consists of migmatite (granite and gneiss) of Precambrian age. The bedrock level varies from a high of Elev. 338.5 at the east end of the south abutment to a low of Elev. 334.1 at the west end of the south pier.



The water level in Rushing River is shown at the following elevations:

January 1936	335.4
November 1993	335.0
Probable high water level	336.0

4.0 MISCELLANEOUS

The description of subsurface conditions at the site is based solely on existing information, and boreholes have not been drilled on site to confirm the conditions. Thurber provides no warranty and does not accept responsibility for the accuracy or reliability of the information prepared by others. Boreholes will be required during detailed design to confirm the subsurface conditions at the locations of the structure foundation units and bridge approaches.

Interpretation of the existing data and preparation of the report were performed by Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.



Murray R. Anderson, M.Eng., P.Eng.
Senior Geotechnical Engineer



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Review Principal



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Geocres Number: 52E-067

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

5.0 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents preliminary geotechnical recommendations to assist selection and preliminary design of the foundation system for the replacement bridge carrying Highway 71 over Rushing River.

Replacement of the existing three-span bridge structure is proposed. It is anticipated that the replacement bridge will be a single span structure with new abutments located behind the existing abutments. A deckless side-by-side precast concrete box girder structure as well as a steel plate girder structure are being considered.

The original timber bridge was 7.6 m wide and constructed in 1936. It was widened by 1.4 m on the east and 4.7 m on the west in 1967. The original structure is supported on 0.91 x 0.91 m concrete pedestals embedded in bedrock, and the widening sections are supported on 0.61 x 0.61 m concrete pedestals dowelled to rock.

The discussion and recommendations presented in this report are based on the information provided by McIntosh Perry and on the factual data obtained from the Geocres drawings.

The interpretation and recommendations are intended for the use of the design consultant and the Ministry of Transportation (MTO), and shall not be relied upon by any other parties including the construction contractor, or used for any purposes other than development of the project design. Comments on construction methodology and equipment, where presented, are provided only to highlight those aspects that could affect the design of the project. Contractors must make their own assessment of the factual information presented in Part 1 of the report, and the implications on equipment selection, construction methodology, and scheduling.



6.0 PRELIMINARY FOUNDATION DESIGN

In general terms, the subsurface stratigraphy at the new abutment locations is expected to consist of rock fill overlying a layer of sandy till underlain by bedrock. The bedrock surface elevations reported along the existing abutments vary as follows:

Table 6.1 – Bedrock Depths and Elevations along Existing Abutments

Foundation Unit	Approximate Bedrock Surface	
	Depth (m)	Elevation
South Abutment	1.0 to 5.4	338.5 to 334.1
North Abutment	2.3 to 3.6	337.2 to 336.2

Based on the subsurface conditions at the site, consideration was given to supporting the bridge using the following foundation types:

- Spread footings on native soil or bedrock
- Steel H-piles
- Drilled shafts (caissons)

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix E. The existing timber bridge structure is supported on concrete pedestals founded on bedrock. Considering the shallow depth to bedrock, the most practical foundation system for the replacement bridge also comprises spread footings constructed on bedrock. H-pile foundations could also be considered if integral abutment design is preferred, however it would be necessary to install the piles in sockets cored into the bedrock to achieve a minimum 5 to 6 m length of pile.

Preliminary recommendations for feasible foundation alternatives are presented in the following sections. A foundation scheme preferred from a foundations perspective is then recommended.

6.1 Spread Footings on Native Soil

Supporting the abutments on spread footings constructed on the native sandy till overlying the bedrock could be considered. However, no information is available on the relative density of the deposits overlying the bedrock, and therefore the support capability of these materials cannot be assessed at the current time. Further, the native soil layer is expected to be relatively thin (and locally absent), and much higher resistance values can be obtained by extending the



footings to bedrock. Therefore, recommendations have not been developed for spread footings on native soils.

6.2 Spread Footings on Bedrock

Constructing spread footings on bedrock, as per the existing bridge foundations, is considered to be the most practical means of supporting the new bridge. For evaluation of the bridge design concept, it is recommended that a geotechnical resistance of 5 MPa be assumed at factored ULS for preliminary design of footings on undisturbed bedrock. The resistance at SLS will not govern design of footings on bedrock. Coring of the bedrock will be required to assess the quality of the bedrock and confirm the geotechnical resistance.

The existing information indicates that the bedrock surface varies across the width of the roadway, from about 1.0 to 5.4 m. Boreholes and rock probes will be required during detailed design to define the bedrock surface profile within the foundation footprint and determine requirements for preparation of the bedrock surface to receive footing concrete. Where sloping bedrock is identified, it may be necessary to excavate the bedrock to provide a level founding surface and/or dowel the footing into bedrock to improve resistance to sliding.

The lateral resistance developed along the base of concrete footings founded on sound bedrock may be computed using an ultimate friction coefficient of 0.7.

Construction of footings on rock will require excavation through the existing rock fill embankment and native sandy till overlying bedrock to a maximum depth of about 5.4 m. For preliminary evaluation of staging requirements, temporary excavation slopes of 1H:1V in the rock fill and 1.5H:1V in the native till may be assumed. If sloped excavations cannot be accommodated due to spatial restrictions, roadway protection involving soldier pile and lagging walls will be required. Soldier piles will need to be socketed into bedrock. Driving of soldier piles or steel sheeting is not feasible due to the presence of rock fill and shallow bedrock.

6.3 Steel H-Pile Foundations

Consideration may be given to socketing steel H-piles into bedrock to support the structure, particularly if an integral abutment design is preferred. Driving of H-piles is not feasible due to the shallow depth to bedrock.

Installation of the piles would involve either excavation or coring through the rock fill and sandy till, then coring an adequate depth into bedrock to form a rock socket, inserting the pile, and grouting the annular space in the socket with concrete. The actual length of socket will need to



be determined by additional investigation and coring. A socket diameter approximately 200 mm larger than the largest dimension (corner to corner) of the pile will be required.

For preliminary design purposes, a factored geotechnical resistance of 2,000 kN per pile is recommended for steel HP 310x110 piles socketed at least 1.5 m into bedrock. The SLS resistance will not govern design. Downdrag on the piles is not an issue at this site.

Boreholes and rock probes will be required during detailed design to define the bedrock surface profile along the abutment alignment. Coring will be required to assess the quality of the bedrock and confirm the geotechnical resistance.

6.4 Drilled Shafts (Caissons)

Considering the shallow depth to bedrock for support of spread footings, the increased cost of constructing caissons does not appear to be warranted at this site. Excavation or augering/coring would be required to penetrate the rock fill, and coring into the bedrock would be required to form a rock socket. On this basis, the use of caissons is not recommended, and recommendations for caisson design have not been developed.

6.5 Recommended Foundation

From a geotechnical perspective, the preferred foundation option to support the replacement bridge comprises spread footings constructed on bedrock. However, socketed H-piles could be employed if an integral abutment bridge design is preferred.

7.0 FROST COVER

The depth of frost penetration at this site is 2.4 m. The base of pile caps embedded in soil must be provided with a minimum of 2.4 m of earth cover as protection against frost action. Frost protection is not required for footings on bedrock.

8.0 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutments should consist of rock backfill or free-draining granular material conforming to OPS Granular A or B Type II specifications. Rock backfill must be restricted to a maximum dimension of 250 mm. The rock backfill and granular material should be placed to the extents shown in OPSD 3101.200 or 3101.150. 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 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 8.1. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for any wingwalls or unrestrained walls.

Table 8.1 – Lateral Earth Pressure Coefficients

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		Rock Backfill $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.20	0.25*
At-rest (Restrained Wall)	0.43	-	0.33	-
Passive	3.7	-	5.0	-

* For wing walls.

The parameters in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC 2014.

In accordance with Clause 6.12.3 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 2.0 m for Rock Backfill or 1.7 m for Granular A or Granular B Type II.



The design of the abutment walls must incorporate measures such as weep holes and/or subdrains to permit drainage of the backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

9.0 EMBANKMENT SLOPES

The existing bridge approaches reach a maximum height in the order of 5.5 m and generally comprise rock fill placed at an inclination in the order of 1H:1V, overlying bedrock. The rock fragments are boulder-sized on the lower part of the slope exposed to river flow. An asphalt cover has been placed at the crest of the slope behind the abutments, in an apparent effort to mitigate washing of pavement granular materials into the rock fill by surface water runoff.

If a grade raise or widening of the highway on the new bridge approaches is planned, widening of the embankments will be required. Widening of the existing rock fill embankment should be carried out using rock fill. The embankment slopes in rock fill are expected to be stable with side slopes inclined no steeper than 1.25H:1V. Settlement of the embankments due to compression of the foundation subgrade is expected to be negligible, provided any loose or organic materials are removed from the embankment footprint.

Embankment slopes comprising exposed earth fill or native soils must be provided with erosion protection in accordance with OPSS.PROV 804. Typically, rock protection should be provided over all surfaces with which river flow is likely to be in contact. Rock fill or a vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion.

10.0 SEISMIC CONSIDERATIONS

In accordance with the CHBDC, the selection of the seismic site class is based on the average soil conditions encountered in the upper 30 m of the ground profile. The stratigraphy at this site generally consists of rock fill overlying a layer of sandy till underlain by shallow bedrock. It is anticipated that the bridge will be supported on spread footings constructed on bedrock. As per Table 4.1 of the CHBDC, the site may be classified as Seismic Site Class B.

Based on the National Building Code of Canada (NBCC 2015), the peak horizontal ground acceleration (PGA), corresponding to a design earthquake having a 2 percent probability of being exceeded in 50 years (i.e. 2,475 year return period) is 0.039 g at the site.

Seismically-induced liquefaction of foundation soils is not an issue for shallow bedrock.



11.0 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with OPSS 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope requirements in compliance with the OHSA, the sandy till and any river channel deposits are classified as Type 3 soils above the water level and Type 4 below. Temporary excavations in rock fill should be stable at 1H:1V.

Where temporary excavations cannot be constructed with inclined slopes due to space limitations, roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

In general, excavation for footing construction at the abutment locations is expected to remain above the river level, and groundwater control is not expected to be an issue. Locally at the west end of the south abutment, the bedrock surface falls to about 0.9 m below the river water level observed in November 1993, and dewatering measures such as pumping from within a sandbag enclosure may be required to construct footings in the dry. If socketed H-piles are employed, water may enter the sockets through fractures in the bedrock if present. The flow volumes to be removed from the excavations and rock sockets will be highly dependent on the variations in the bedrock levels and the rock conditions encountered at individual locations.

12.0 DETAILED INVESTIGATION REQUIREMENTS

Further subsurface investigation, analysis and design must be carried out during detailed design to confirm the soil and bedrock conditions at the location of the structure foundation elements and approaches. This investigation should include:

- Drilling of boreholes within the footprint of the new abutment foundations to delineate the bedrock surface. Typically this should comprise five boreholes within each foundation element (centre and four corners of the footing). As rock fill is present, all boreholes should be cored to confirm bedrock.

The foundation boreholes should include determination of the thickness and relative density of the native sandy till layer underlying the rock fill, to assess the practicality of constructing footings on the undisturbed soil in order to raise the founding level. In addition, the bedrock coring should determine the rock strength and quality, the geotechnical resistance of the bedrock, and rock conditions to be anticipated during socket construction, if employed.



- Drilling of a borehole to bedrock at each approach embankment within 20 m of the abutment.
- Drilling of boreholes to bedrock along potential roadway protection systems, to determine the thickness and composition of the rock fill and native soil, evaluate the difficulty of excavation and/or augering, assess permissible excavation slope inclinations, and assist development of staging operations.

13.0 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Murray R. Anderson, P.Eng.
Senior Geotechnical Engineer



Dr. P.K. Chatterji, P.Eng.
Review Principal





Appendix A
Site Photographs



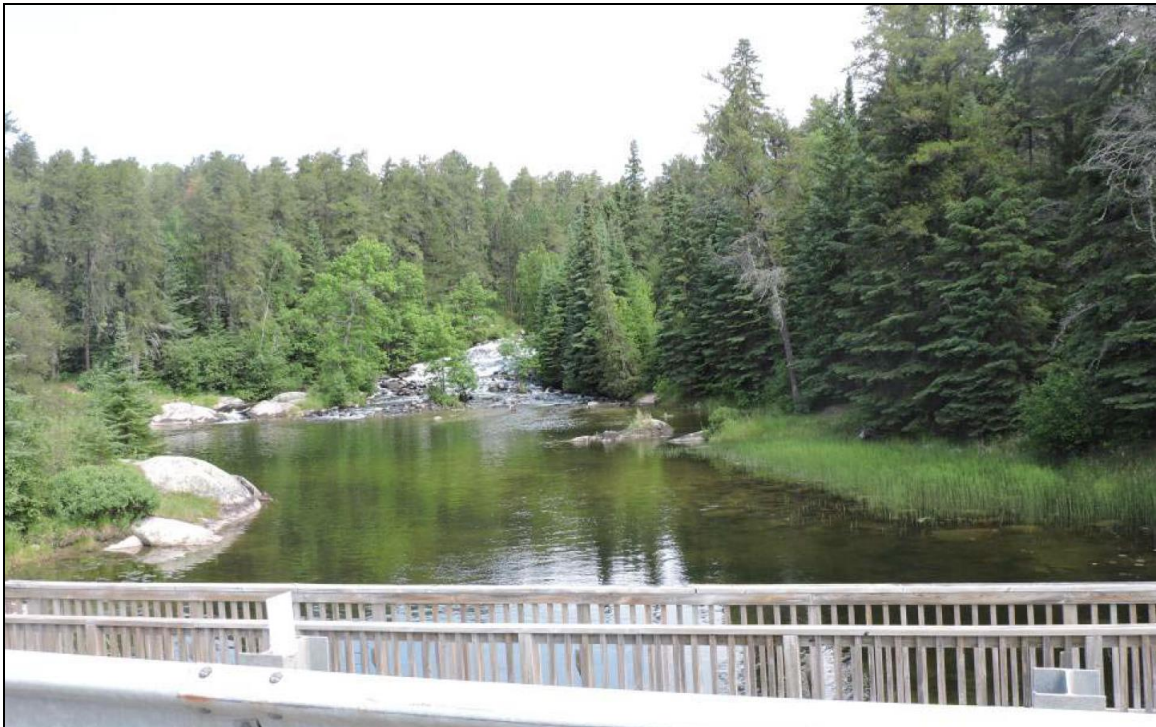
Photograph 1 – East side of bridge looking north (June 2015)



Photograph 2 – West side of bridge looking south (June 2015)



Photograph 3 – West side of bridge looking north (June 2015)

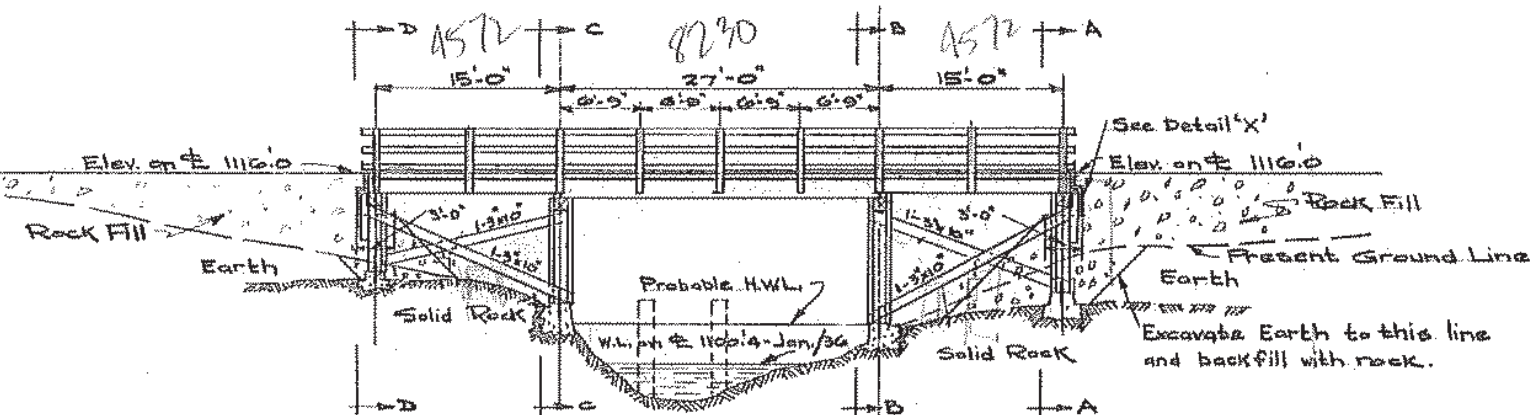
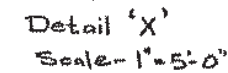
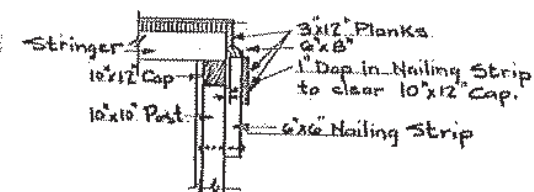
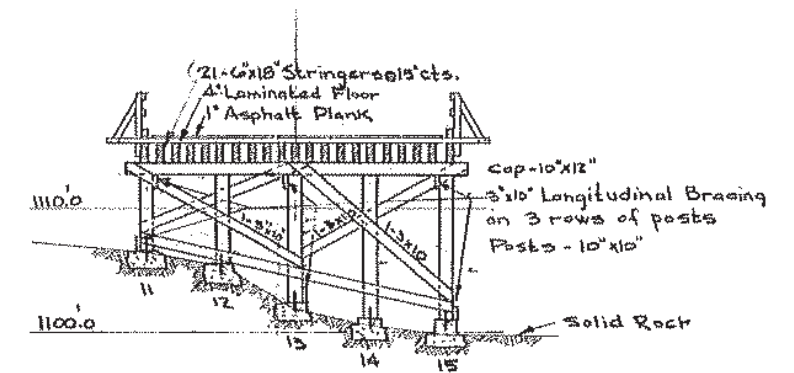
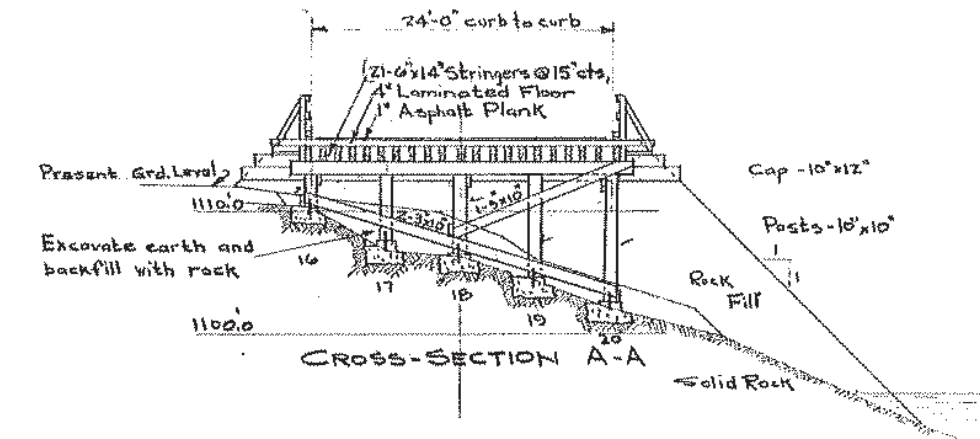
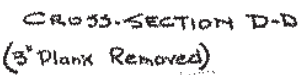
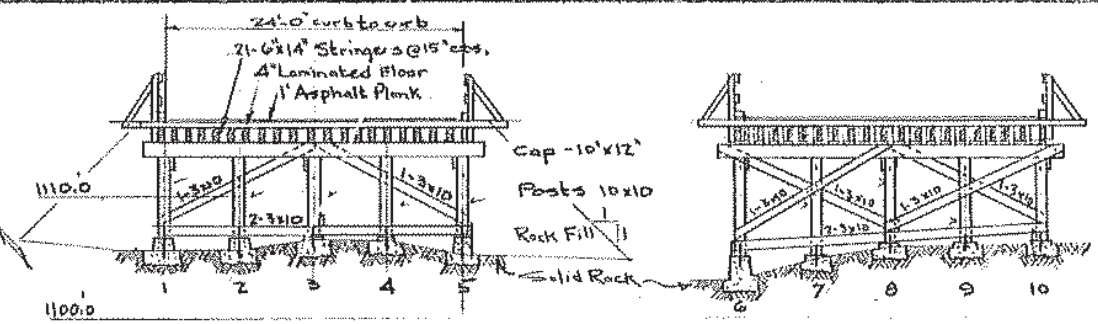
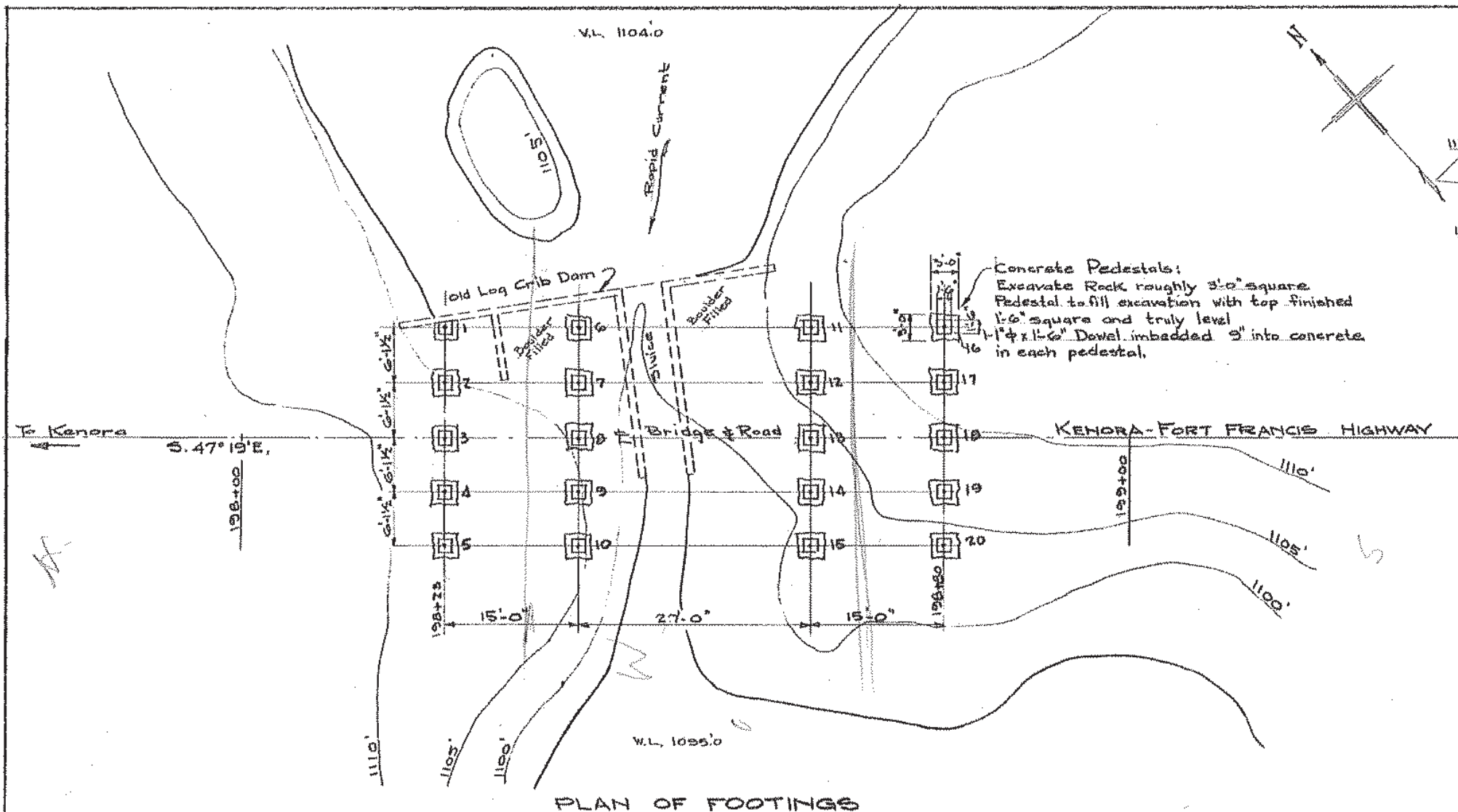


Photograph 4 – Looking east from bridge (June 2015)



Appendix B

**Drawing B1190-A
Plan of Proposed Bridge over Rushing River
May 1936**



Pedestal	Elevation of Rock	Elevation of Top of Pedestal
1	1105.6	1106.5
2	1105.6	1106.5
3	1106.4	1106.5
4	1106.4	1106.5
5	1105.2	1106.5
6	1103.1	1105.0
7	1104.8	1105.0
8	1105.8	1106.0
9	1105.8	1106.0
10	1105.9	1106.0
11	1106.4	1106.5
12	1105.3	1105.5
13	1101.9	1102.5
14	1100.5	1101.0
15	1099.8	1101.0
16	1110.6	1101.5
17	1106.8	1107.5
18	1106.4	1106.5
19	1104.3	1104.5
20	1102.1	1102.5

Designed for H2O Loading
For Details not shown see S.T.B. 18
Stringers, caps & posts to be Struct Gr. B.C. Fr
All timber to be creosote treated,
except handrail, wheel guard and handrail posts.
All concrete to be 2500 ψ at 28 days.

REVISIONS	May 20	Elevations Added	CCF
	May 22	Sections C-C and D-D Added	CCF
	May 23	Table for Elevations Added	CCF
	May 20	Centre span increased from 26'-0" to 27'-0"	CCF
	DATE	REMARKS	REV BY
DEPARTMENT OF NORTHERN DEVELOPMENT DISTRICT OF KENORA			
PLAN OF PROPOSED BRIDGE OVER RUSHING RIVER MI. 3.76 KENORA-FORT FRANCIS H'WY			
Scale: 1" = 10'-0"	Approved: <i>[Signature]</i> Engineer of Construction Toronto, Ont. May 19, 1936		
Dwg. CCF Dr. CCF	Dwg. - A 1990-A 1		

DEPARTMENT OF NORTHERN DEVELOPMENT
DISTRICT OF KENORA

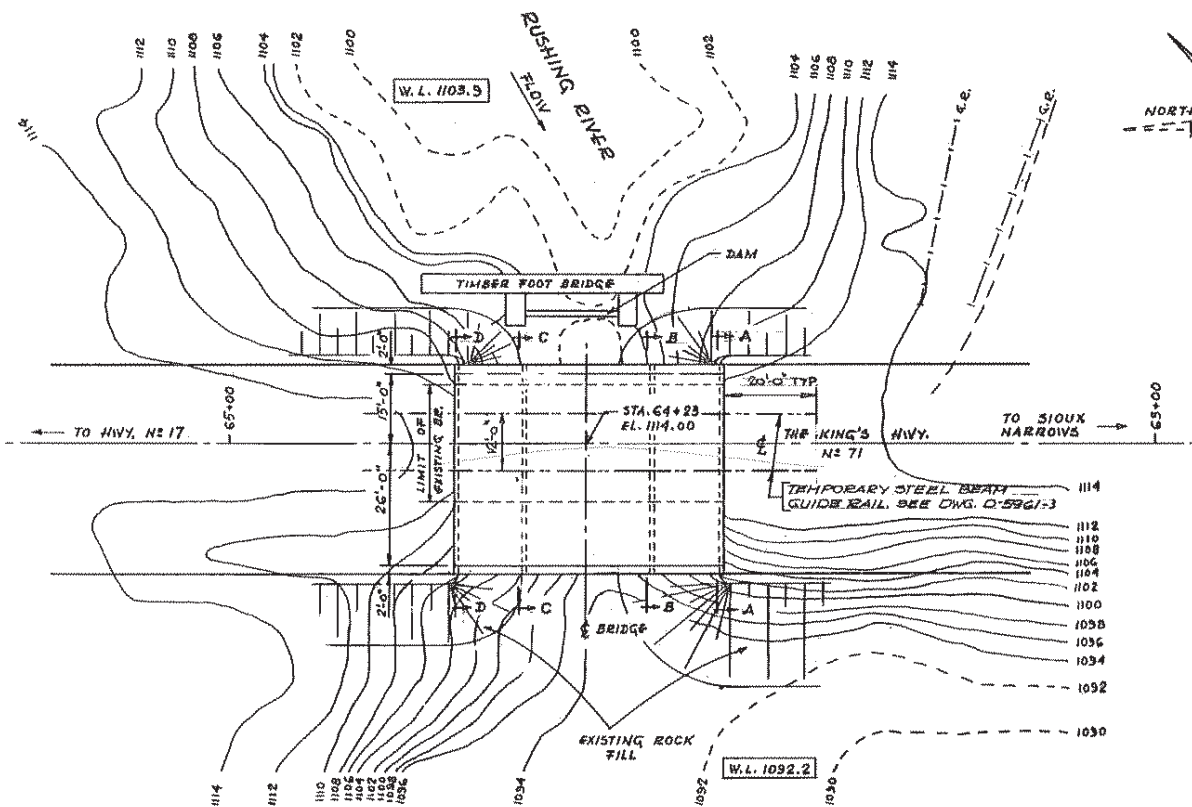
PLAN OF PROPOSED BRIDGE
OVER RUSHING RIVER
MI. 3.76 KENORA-FORT FRANCIS H'W'Y.

Scale: 1"=10'-0"	Approved: <i>[Signature]</i> Engineer of Construction
Des. CCF In. CCF	Toronto, Ont. Dwg. - Sheet May 19, 1936 B 1990-A 1

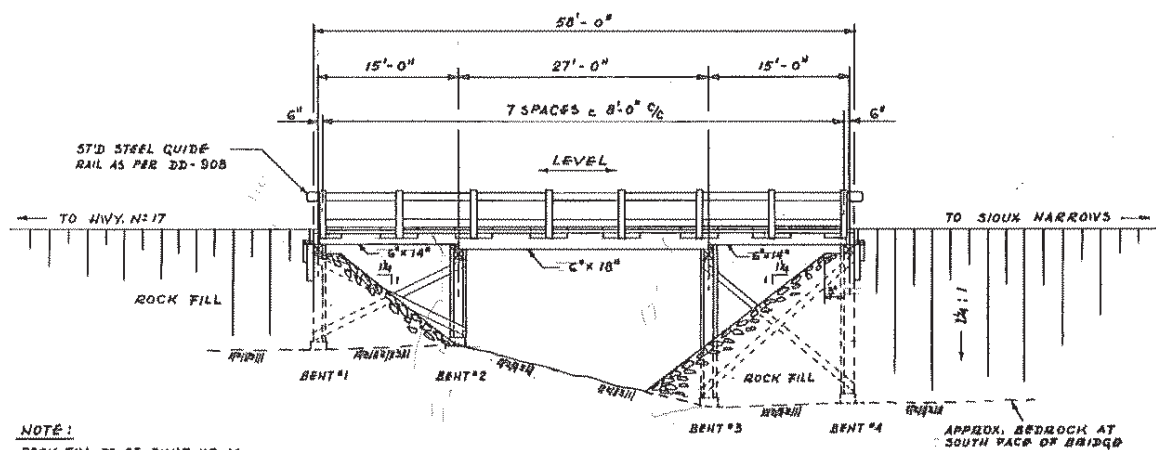


Appendix C

**Drawings D-5961-1 and 2
Rushing River Structure Widening, General Plan and Details
January 1967**



PLAN
SCALE: 1" = 20'-0"

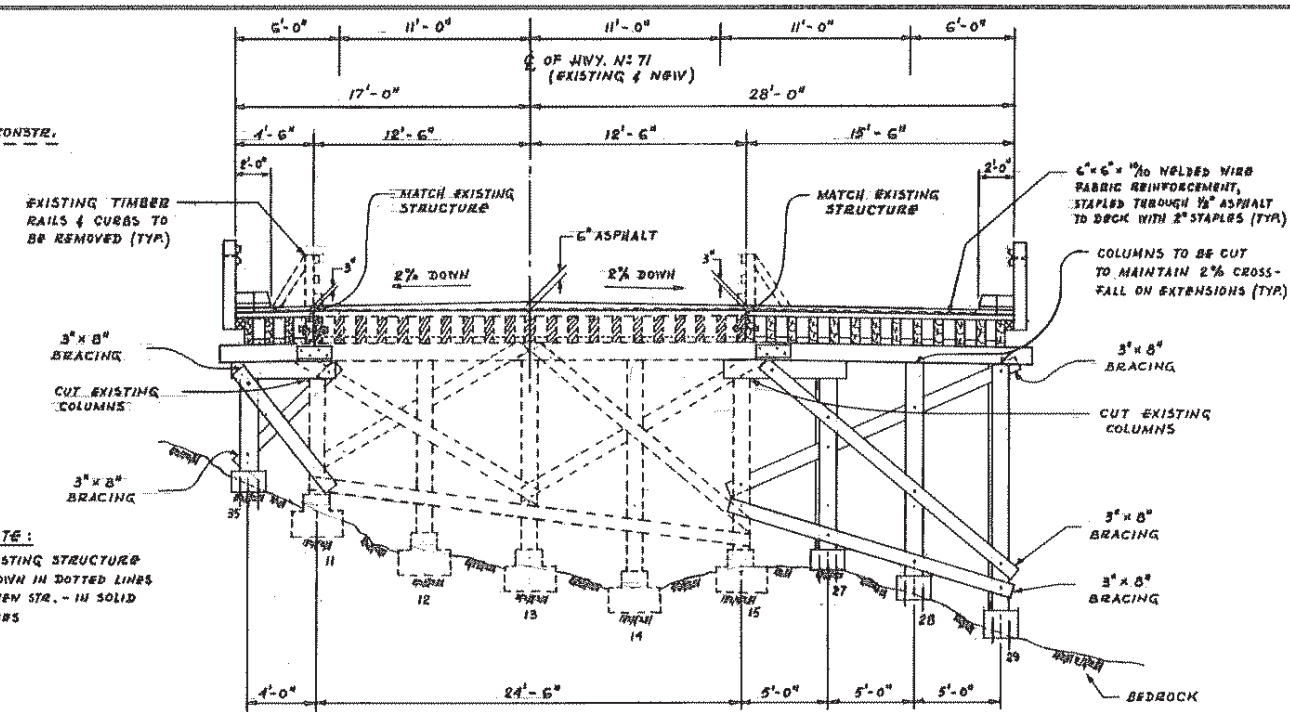


NOTE:
ROCK FILL TO BE BUILT UP AS
EQUALLY AS PRACTICABLE
AROUND COLUMN BENTS

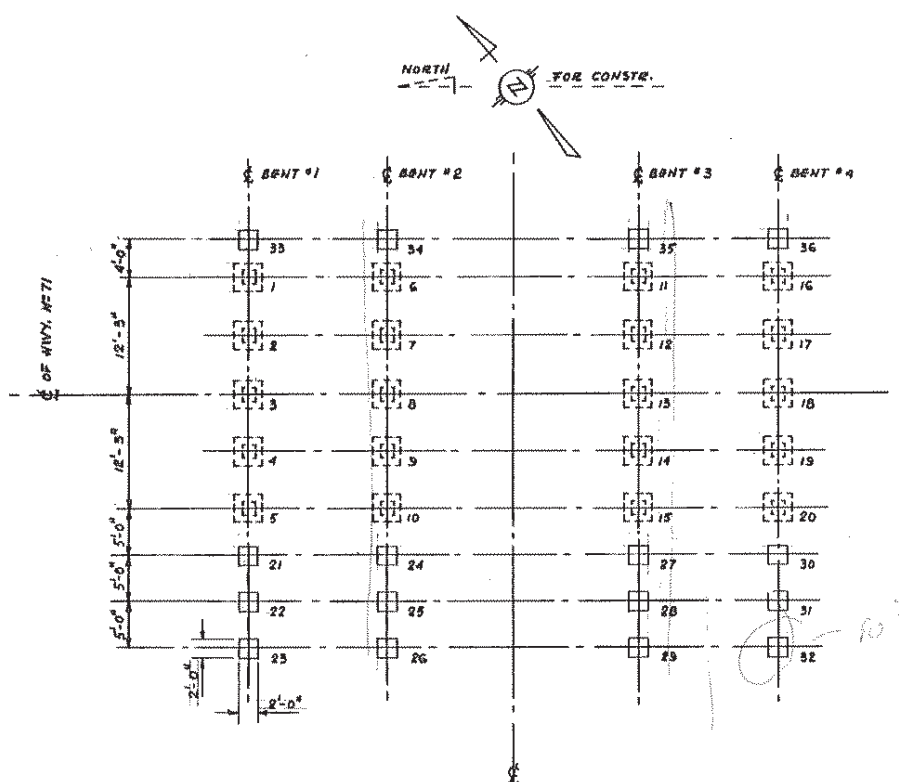
ELEVATION
SCALE: 1" = 10'-0"

PEDESTAL	ELEV. OF TOP OF PEDESTAL
21	1104.0
22	1104.0
23	1104.0
24	1103.50
25	1100.0
26	1100.0
27	1100.0
28	1099.0
29	1096.0
30	1100.0
31	1093.0
32	1037.0
33	1103.0
34	1100.50
35	1106.50
36	1110.0

LIST OF DRAWINGS
D-5961-1 GENERAL PLAN
-2 DETAILS
-3 BILL OF MATERIAL

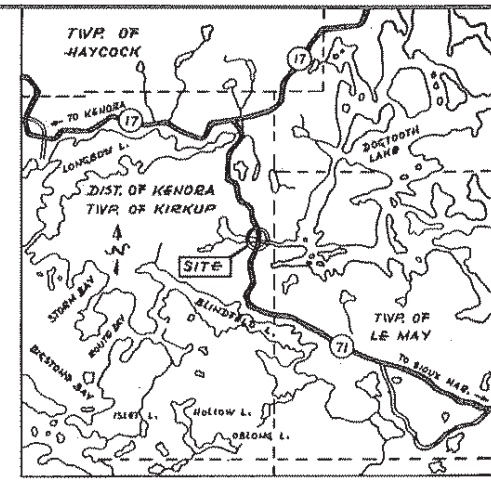


SECTION B-B
SCALE: 3/16" = 1'-0"



FOUNDATION PLAN
SCALE: 1" = 10'-0"

NOTE:
NEW ASPHALT & WEAR REINFORCEMENT
TO BE PROVIDED TO ENTIRE DECK OF
FINISHED BRIDGE



KEY PLAN
SCALE: 1" = 2 MI.

NOTES:
Q. B. M. N° 544 K. ELEV. 1116.383
LOW ROCK OUTCROP ON SOUTH SIDE OF KENORA - FORT
FRANCES HWY, 155' WEST OF WEST END OF RUSHING R.
BRIDGE & 20' SOUTH OF E. OF HWY. TABLET IN NORTH FACE OF
ROCK 2' ABOVE GRADE LEVEL.
PUBLICATION N° 20, PAGE 36, KENORA
19' LT. OF STA. 65+88

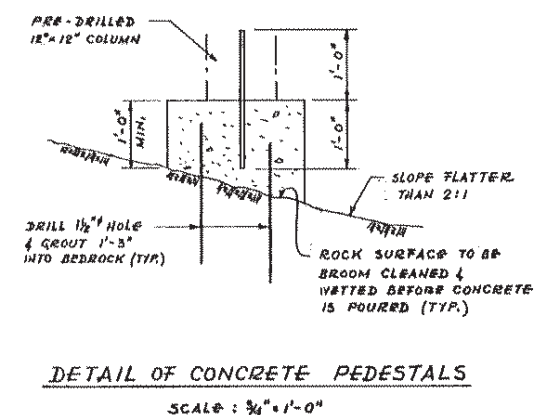
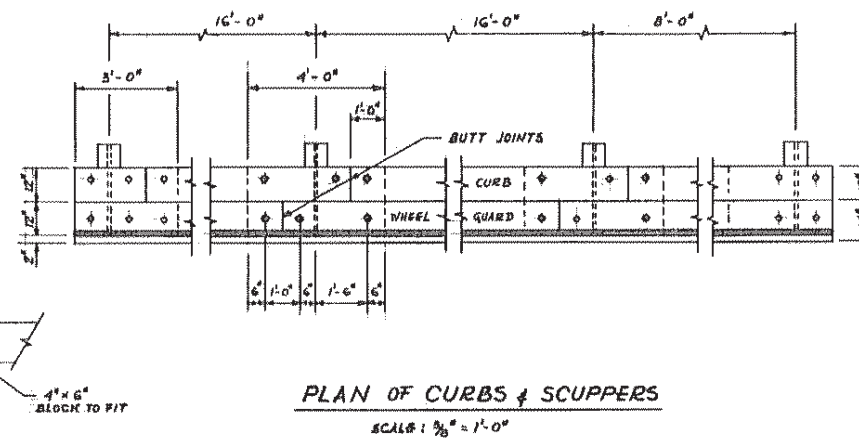
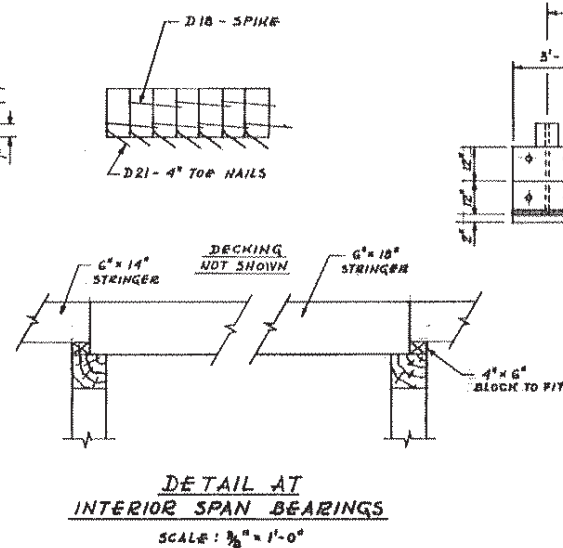
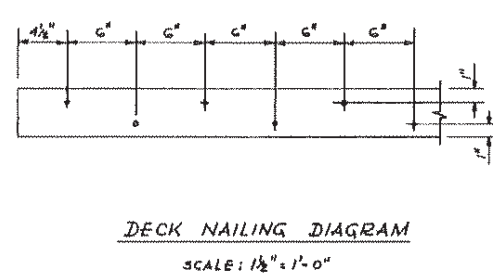
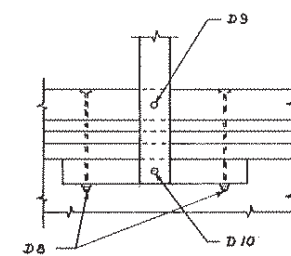
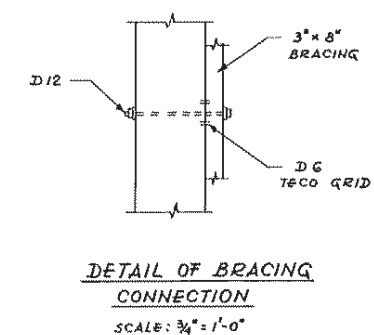
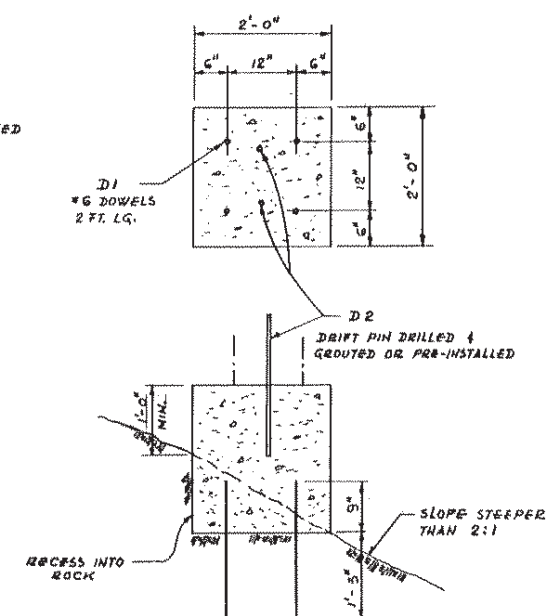
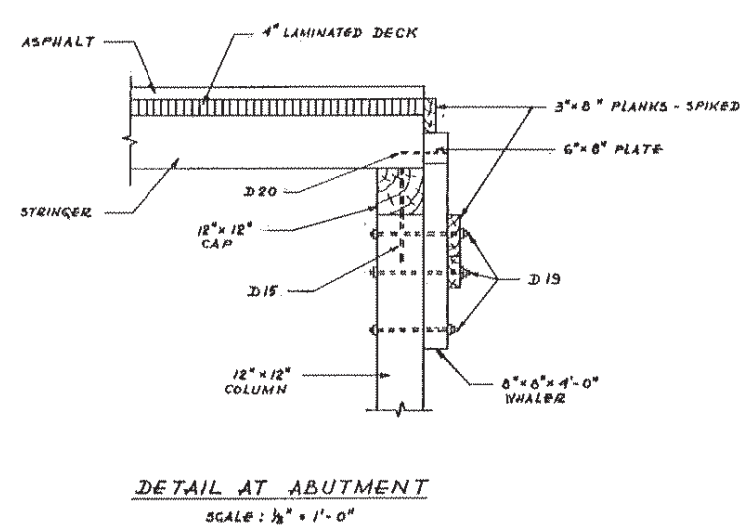
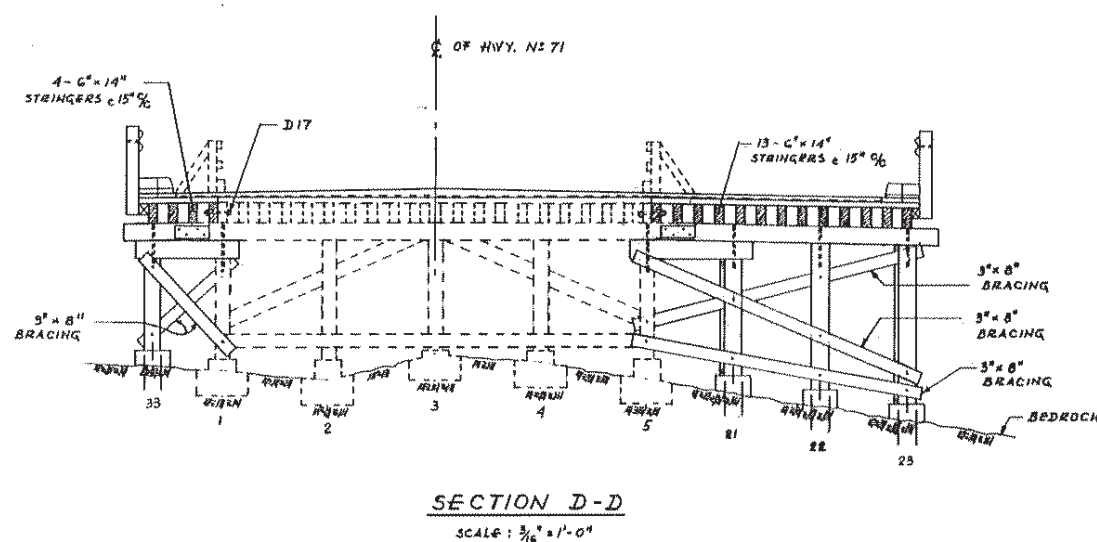
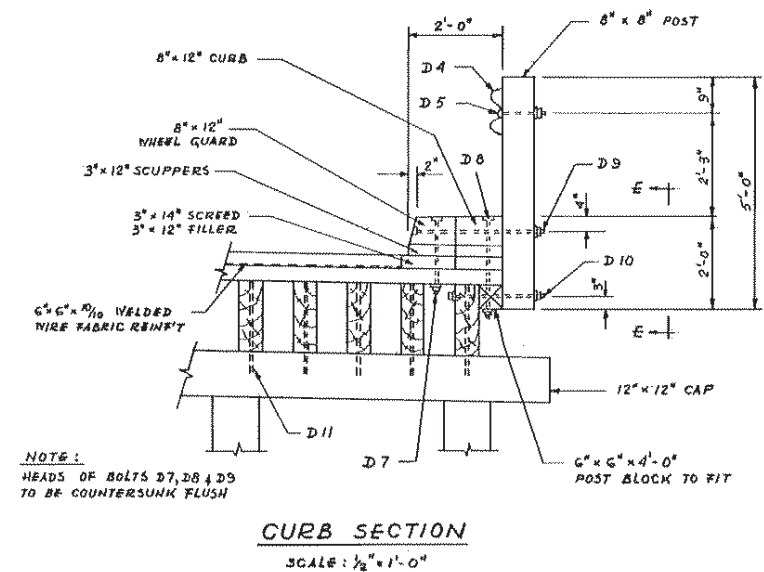
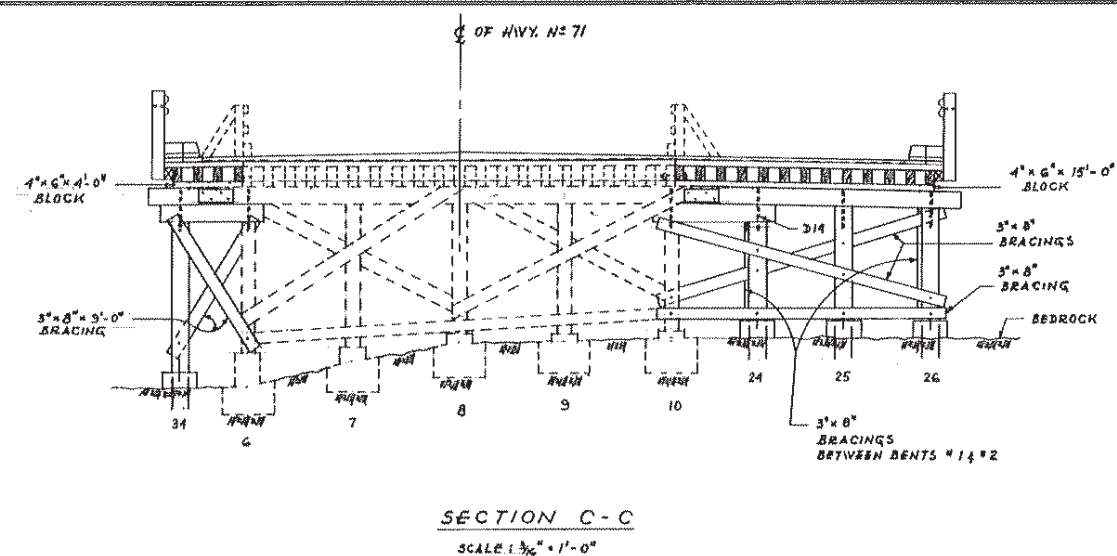
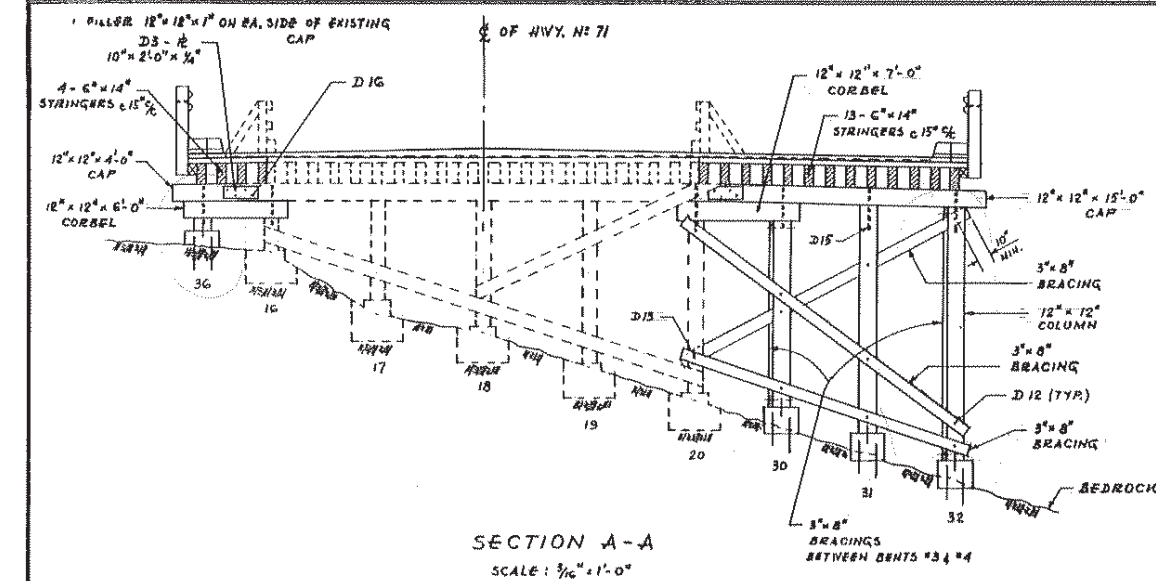
ERECTION NOTES
ALL HOLES DRILLED IN THE FIELD (FOR BOLTS, DRIFT PINS,
ETC.) MUST BE TREATED WITH HOT CREOSOTED OIL APPLIED
WITH A BOLT TREATER. FRESH WOOD SURFACES EXPOSED BY
FIELD CUTTING, MUST BE TREATED WITH 3 BRUSH COATS
OF HOT CREOSOTE OIL.
ALL DIFFERENT MEMBERS & LENGTHS SHOULD BE
STACKED SEPARATELY. BEFORE CUTTING OFF COLUMNS, THE
CUT OFF ELEVATIONS WILL BE CHECKED BY THE ENGINEER
IN THE FIELD, TO ENSURE A TRUE GRADE ACROSS THE STRUCTURE.
ALL HOLES FOR DRIFT PINS SHALL BE 3/4" DIA.
ALL HOLES FOR BOLTS SHALL BE 1/8" LARGER THAN THE
BOLT SIZE.

CLASS OF CONCRETE
3000 P.S.I.

REVISIONS	DATE	BY	NOTES ADDED	DESCRIPTION
1	17.7.67	R.K.	NOTES ADDED	

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
RUSHING RIVER STRUCTURE WIDENING 3.3 MI. SOUTH OF LONGBOW CORNERS			
KING'S HIGHWAY No. 71		DIST. No. 20	
DIST. OF KENORA		TWP. KIRKUP LOT CON.	
GENERAL PLAN			
APPROVED	DESIGN	CHECK	W.M.
DATE	JAN. 1967	LOADING	HS20-44
SITE No. 415-61		W.P. No. 21-66	
CONTRACT No.		DRAWING No. D-5961-1	

NOTE:
CONCRETE PEDESTALS MARKED
N° 1 TO 20 ARE EXISTING STRUCTURE
& N° 21 TO 36 ARE NEW STRUCT.



REVISIONS			
	17, 7, '67	P.K.	LOCATION TO THE TITLE# ADDED
	DATE	BY	DESCRIPTION


DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

RUSHING RIVER STRUCTURE WIDENING

3.3 MI. SOUTH OF LONGBOW CORNERS

KING'S HIGHWAY No. 71 DIST. No. 20
~~DIST. OF KENORA~~
 TWP. KIRKPUR LOT CON.

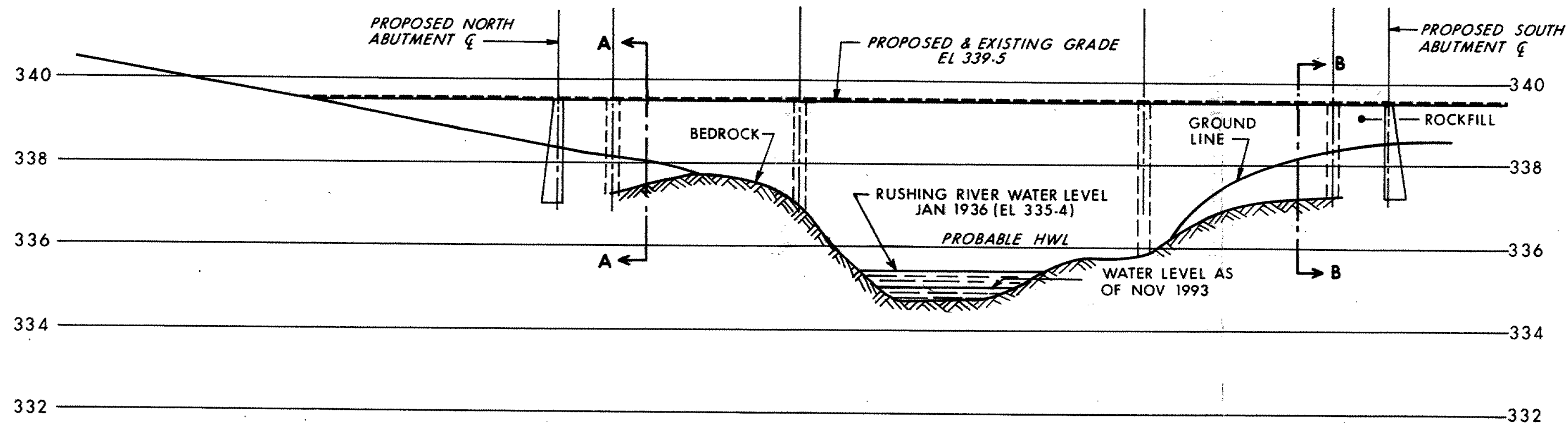
DETAILS

APPROVED 				SITE No. 415-61		W.P. No. 21-66	
REGION ENGINEER				CONTRACT No.		94-93	
DESIGN	DSM	CHECK	WM				
DRAWING	P.K.	CHECK	MB				
DATE	JAN. 1967		LOADING	4520-44			
				DRAWING No.		D-5961-2	

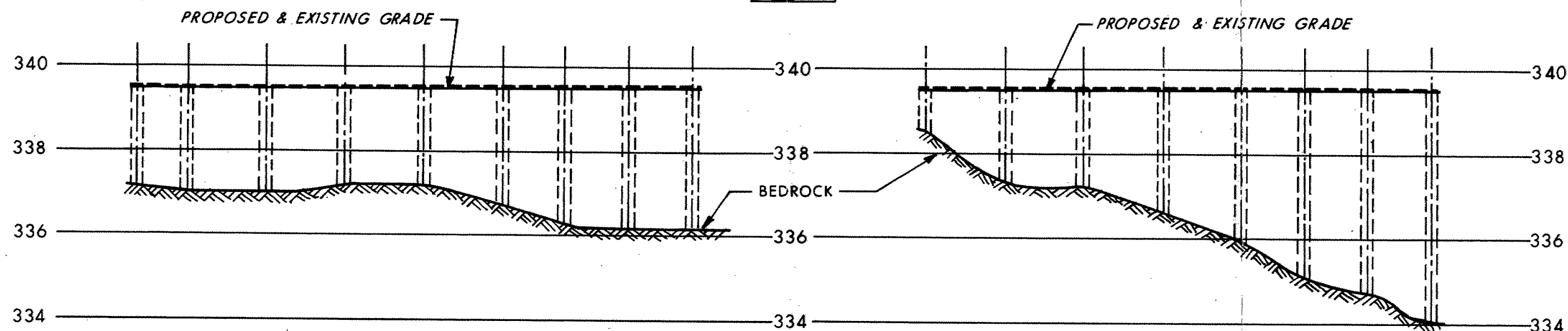
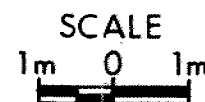


Appendix D

**Drawing 7809101-B
Profile and Sections from previous Foundation Investigation Report
Geocres 52E-38**



ζ PROFILE HWY 71



SECTION A-A
PROBABLE BEDROCK ELEVATION AT
NORTH ABUTMENT

SECTION B-B
PROBABLE BEDROCK ELEVATION AT
SOUTH ABUTMENT



NOTE
FOR PLAN REFER TO Dwg No
7809101-A

WP 780-91-01
Dwg No 7809101-B



Appendix E

Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Bedrock	Socketed Piles	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Potentially less excavation than footings on bedrock. iii. Scour of bedrock of minimal concern. iv. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Does not allow use of integral abutment design. ii. Excavation of existing rock fill required. iii. Higher resistance values can be obtained on bedrock at shallow depth. iv. Scour protection required. v. Roadway protection required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. High resistance values are available on bedrock at shallow depth. iii. Scour of bedrock of minimal concern. iv. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Does not allow use of integral abutment design. ii. Excavation of existing rock fill and sandy till required. iii. Bedrock elevation may vary. iv. Roadway protection including socketed soldier piles required to maintain traffic. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance in bedrock. ii. May require less excavation than footing construction, reducing roadway protection requirements. iii. Possibly more adaptable to staging alignments / support of temporary bridge. iv. Pile installation may continue in freezing weather. v. Allows use of integral abutments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than footings. ii. Need to socket piles into very strong bedrock. iii. Difficult augering through existing rock fill. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded on bedrock. ii. May require less excavation than footing construction, reducing roadway protection requirements. iii. Construction could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Much higher cost than shallow footings. ii. Temporary steel liners may be required to install caissons through cohesionless fill and native soils. iii. Augering through existing rock fill required. iv. Difficulty in cleaning and inspecting bases.
NOT RECOMMENDED	RECOMMENDED	FEASIBLE	NOT RECOMMENDED