

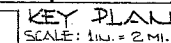
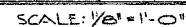
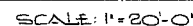
#66-F-211

W.P. #114-64

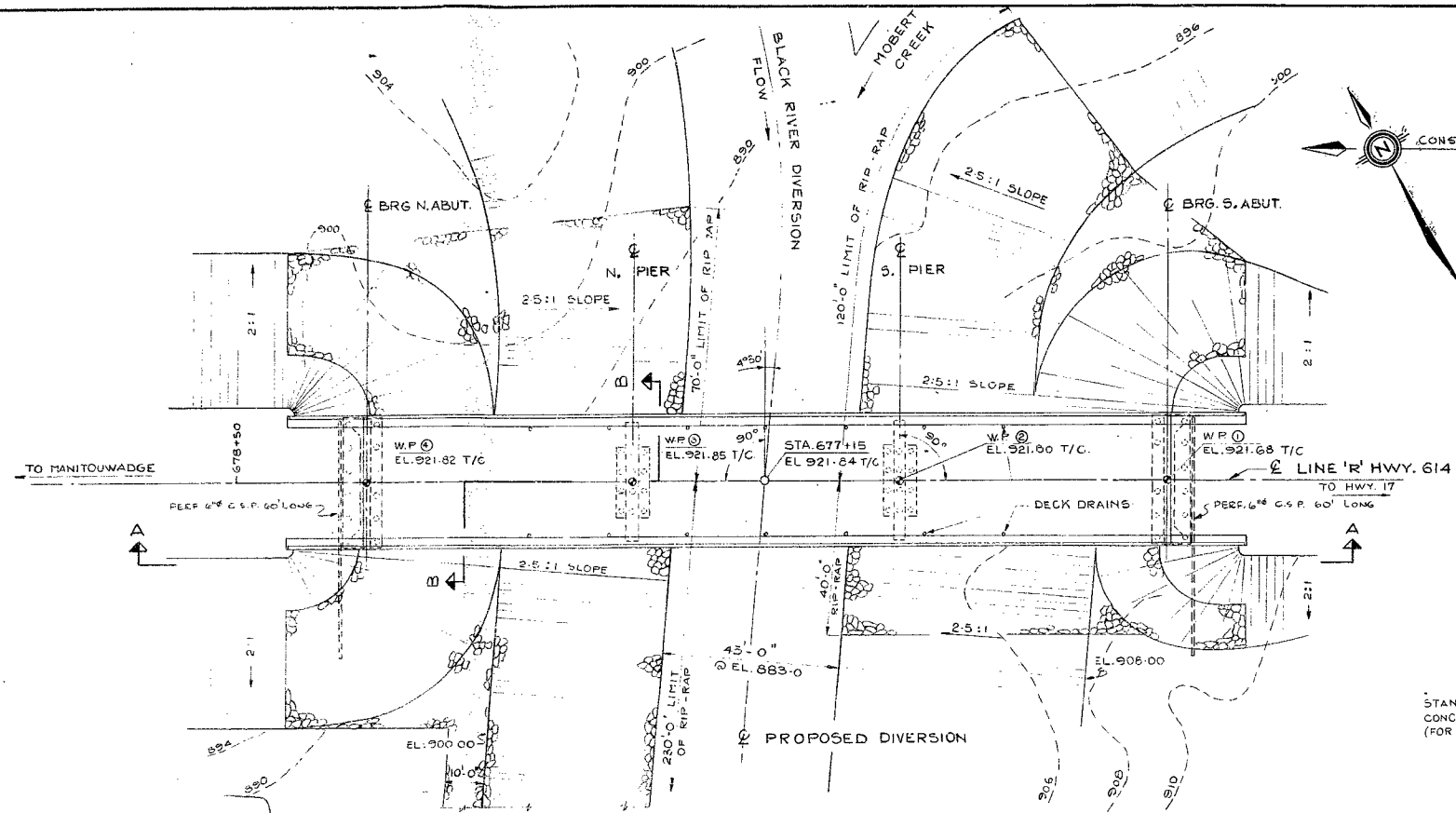
Hwy. #614

BLACK RIVER

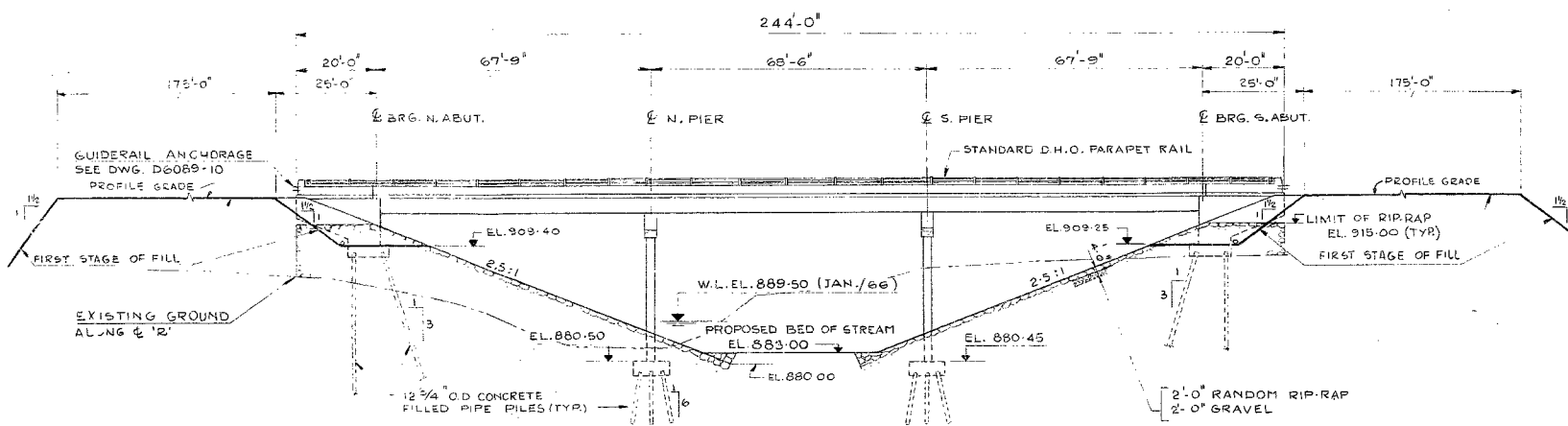
DIVERSION



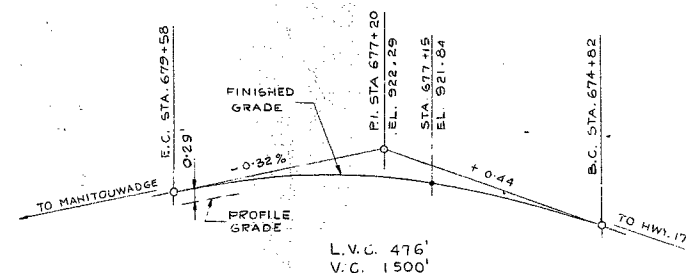
DATE		LOADING	H520-44	No.	D-6089-P
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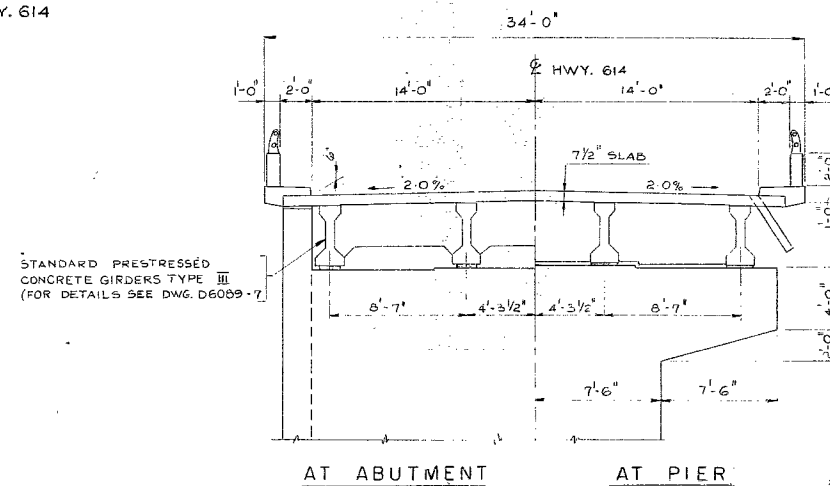
PLAN
SCALE - 1" = 20'-0"



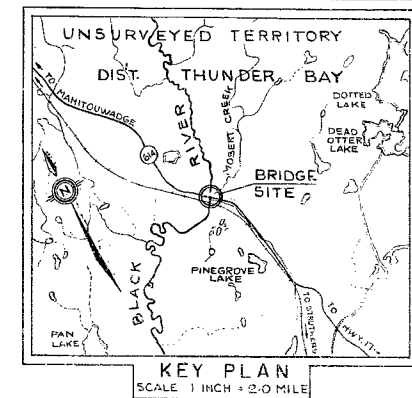
ELEVATION A-A
SCALE - 1" = 20'-0"



PROFILE OF HWY 614
N.T.S.



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



KEY PLAN
SCALE 1 INCH = 2.0 MILE

- LIST OF DRAWINGS**
- D6089-1 GENERAL ARRANGEMENT
 - D6089-2 BORE HOLE LOCATIONS & SOIL STRATA
 - D6089-3 BORE HOLE LOCATIONS & SOIL STRATA
 - D6089-4 FOUNDATION LAYOUT & REINFORCEMENT
 - D6089-5 ABUTMENT & WINGWALL REINFORCEMENT
 - D6089-6 PIER REINFORCEMENT
 - D6089-7 PRESTRESSED GIRDERS & BEARINGS
 - D6089-8 DECK LAYOUT & DETAILS
 - D6089-9 DECK REINFORCEMENT
 - D6089-10 PARAPET WALL DETAILS
 - D6089-11 STANDARD STEEL PARAPET RAIL
 - D6089-12 STANDARD BRIDGE DETAILS
 - D6089-13 STANDARD BRIDGE DETAILS

BENCH MARK

B.M. 904.16
N & W IN 0.4 SPRUCE STR.
125'-0" RT OF STA. 675+38

NOTES

CLASS OF CONCRETE:

PRETENSIONED GIRDERS	5000 PSI
DECK SLAB, DIAPHRAGMS AND PARAPET WALLS	4000 PSI
REMAINDER	3000 PSI

CLEAR COVER TO REINFORCING STEEL:

SURFACES IN CONTACT WITH EARTH & WATER	3"
TOP OF DECK SLAB	1 1/2"
BOTTOM OF DECK SLAB	1"
PRETENSIONED GIRDERS	1 1/2" EXCEPT AS NOTED
PARAPET WALLS	1 1/2"
PIER CAPS	2"
REMAINDER	2"

PILES: ALL PILES ARE CONCRETE FILLED STEEL TUBE. PILES 12 3/4" O.D., 0.25" WALL THICKNESS. DESIGN LOAD = 60 TONS/PILE.

CONSTRUCTION NOTES:

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING ALL BEARING SEATS TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF 1/8". OPEN EXCAVATION FOR PIER FOOTINGS IS NOT PERMITTED. THIS EXCAVATION SHALL BE CARRIED OUT WITHIN ADEQUATELY BRACED INTERLOCKING SHEETING. THE BRACING SHALL REMAIN UNTIL COMPLETION OF BACKFILLING. THE SHEETING SHALL THEN BE REMOVED.

SEQUENCE OF CONSTRUCTION:
REFER TO SPECIAL PROVISIONS



REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

Giffels ASSOCIATES LIMITED

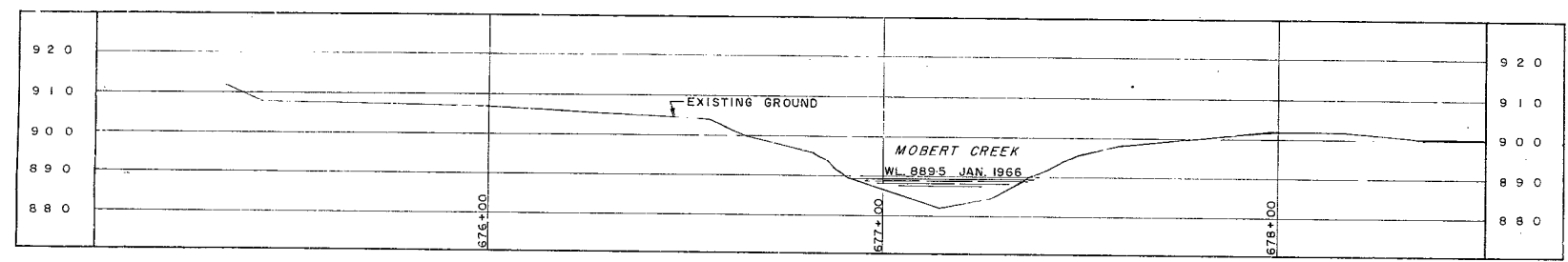
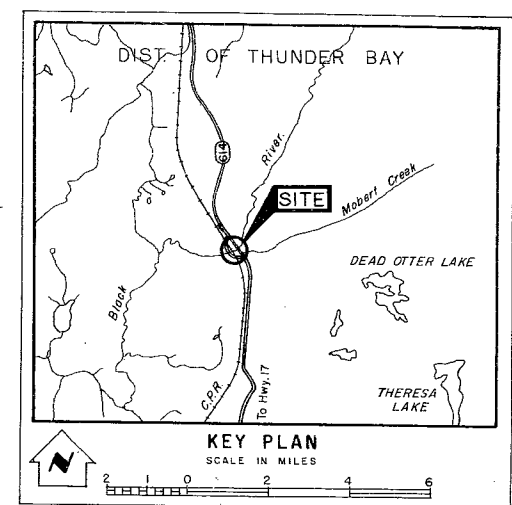
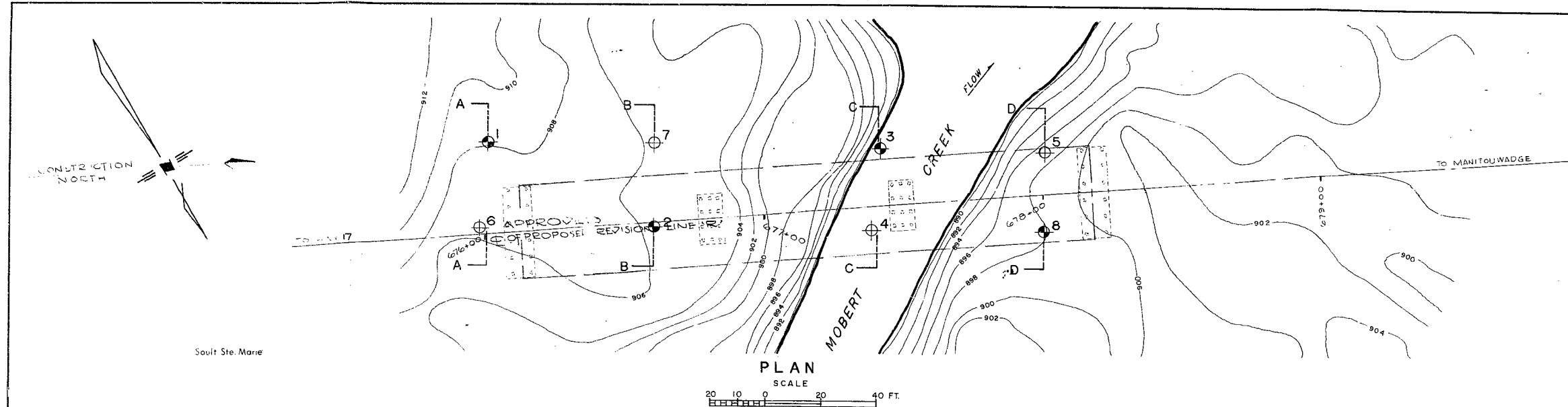
BLACK RIVER BRIDGE
19.5 MILES SOUTH OF MANITOWADGE

KING'S HIGHWAY No. 614 (SEC.) DIST. No. 18
DIST. THUNDER BAY
TWP. LOT CON.

GENERAL ARRANGEMENT

APPROVED	DATE	SITE No.	48E-37	W.P. No.	114-64
DESIGN	R.H.	CHECK	A.R.	CONTRACT	No.
DRAWING	I. H.	CHECK	A.R.	DRAWING	No.
DATE	APRIL, 1967	LOADING	H560-44		

D6089-1



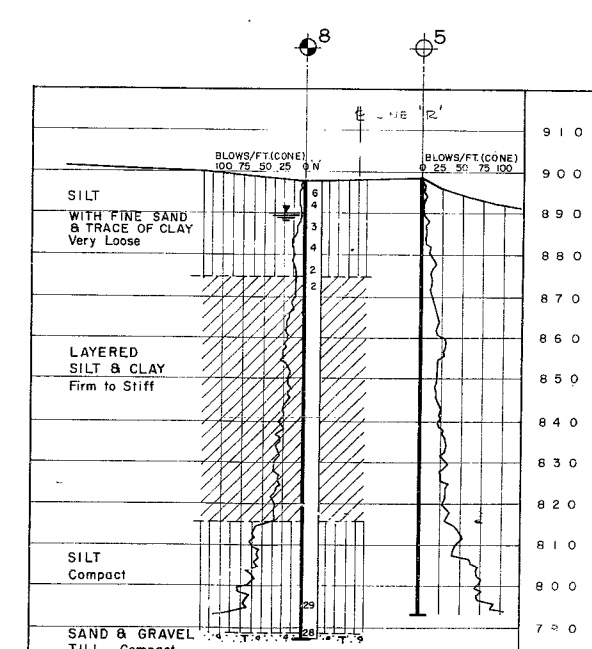
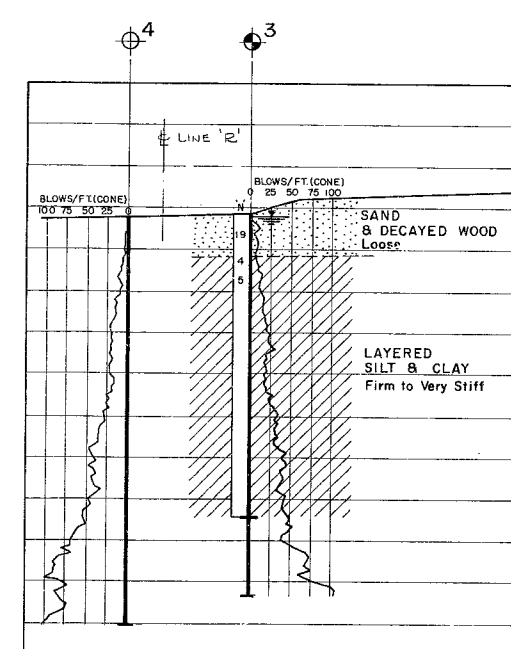
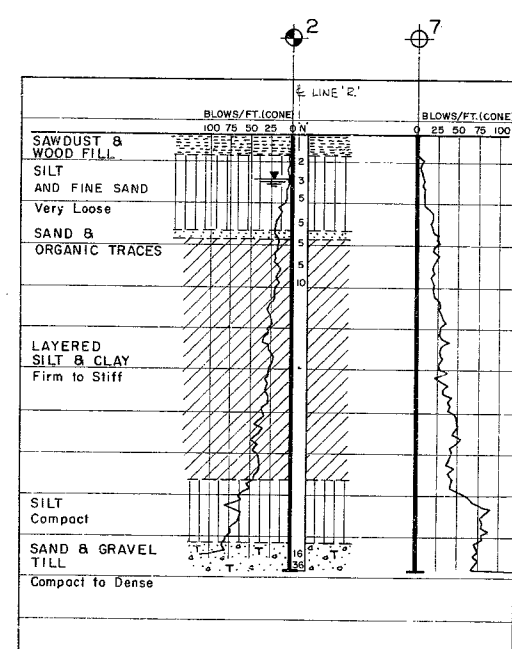
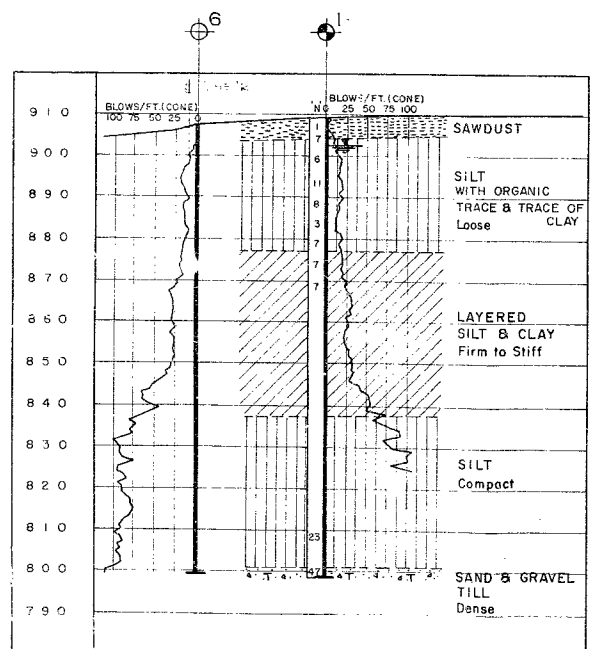
LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation. (Sept. 1966)		

NO.	ELEVATION	STATION	OFFSET
1	909.4	676+02	33' LT.
2	906.8	676+61	2' RT.
3	888.5	677+44	21' LT.
4	887.8	677+37	9' RT.
5	898.4	678+02	15' LT.
6	907.6	675+93	2' LT.
7	906.5	676+63	29' LT.
8	897.6	677+93	14' RT.

B.M. 904-16 GEODETIC DATUM
N.B.W. in 0-4' Spruce Stp.
150' Rt. of Sta. 675+18

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.

PRINT RECORD		
NO.	FOR	DATE
1	22	1966



REVISIONS	DATE	BY	DESCRIPTION

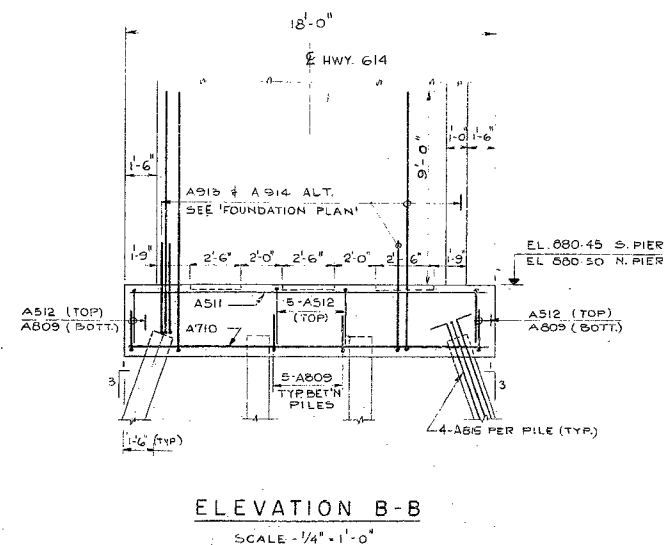
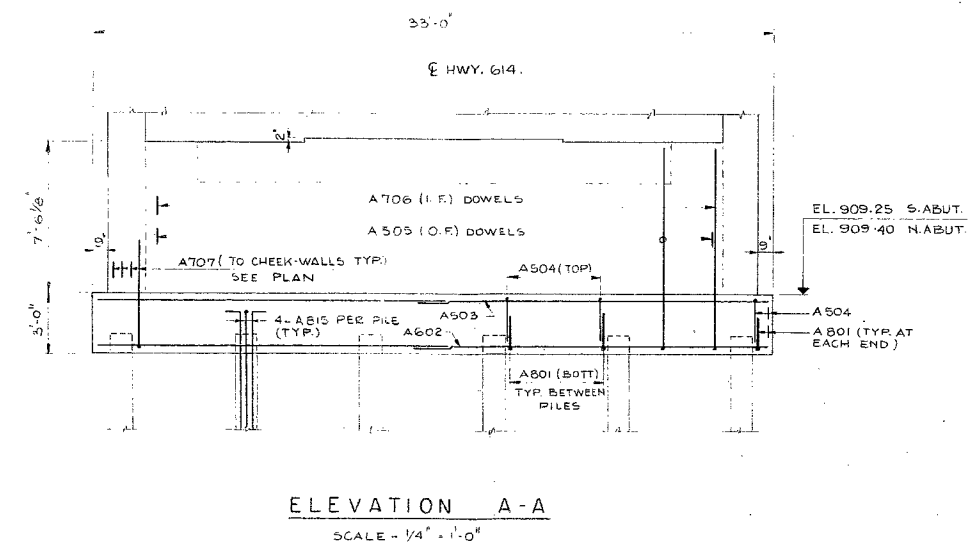
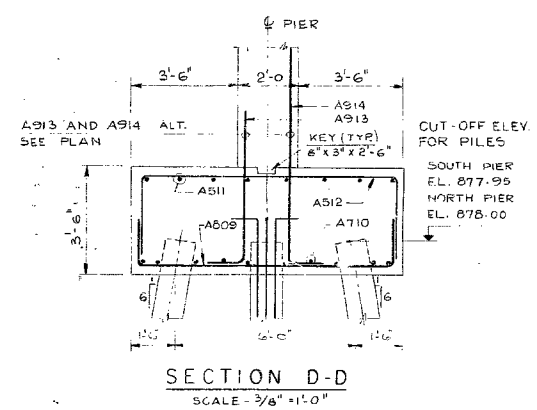
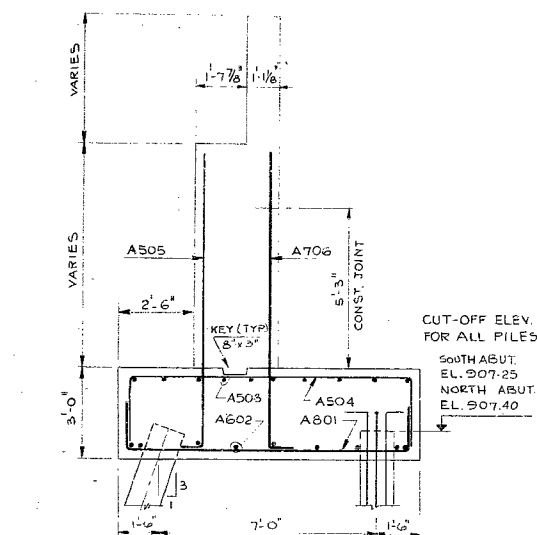
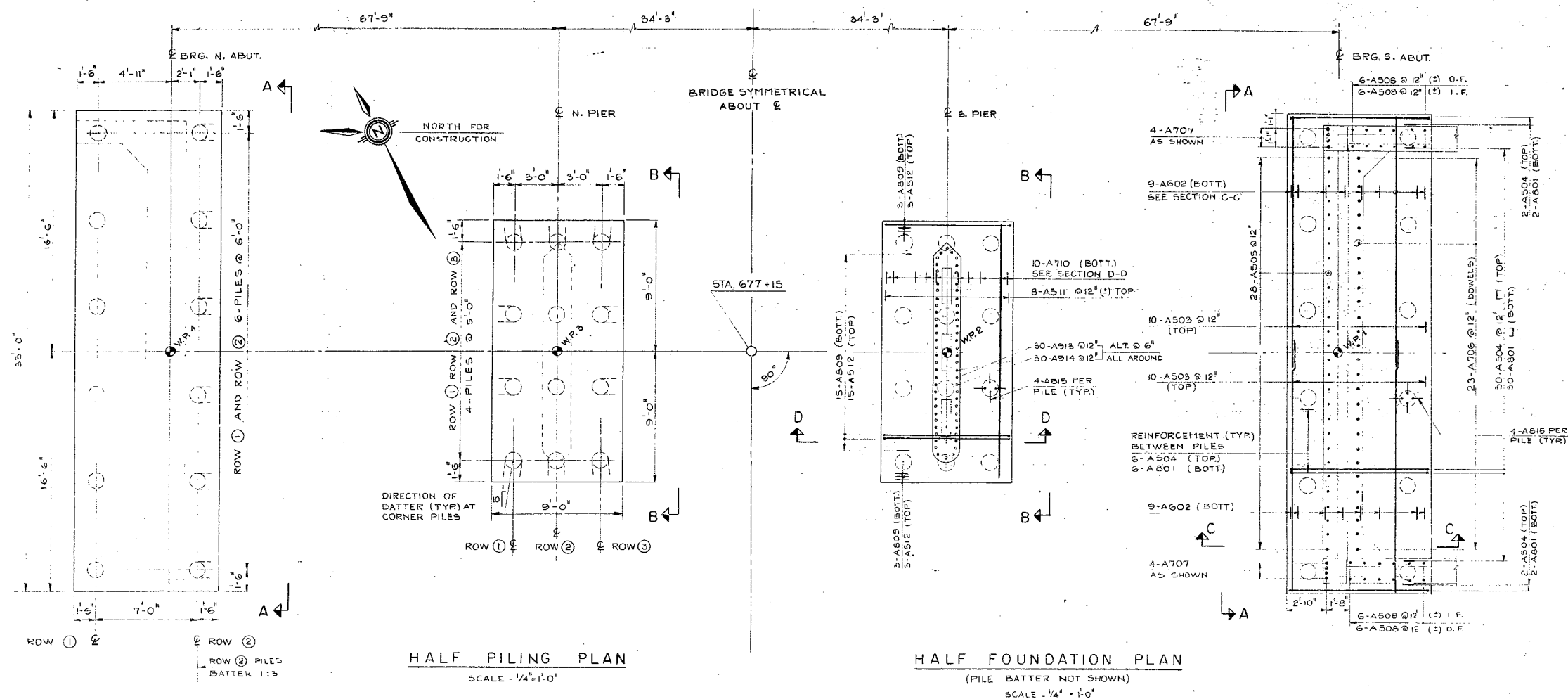
GEOCON LTD
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

BLACK RIVER DIVERSION

KING'S HIGHWAY NO. 614 (SEC.) DIST. NO. 18
DIST. THUNDER BAY
TWP. LOT CON.

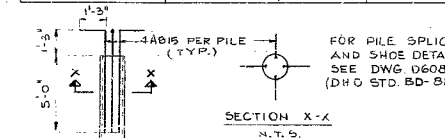
BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D H.L.M.	CHECKED D.B.O.	W.P. NO. 114-64	DRAWING NO.
DRAWN A.E.L.	CHECKED H.L.M.	JOB NO.	T 7923-1
DATE OCT. 7, 1966	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>D. Bates</i>	CONT. NO.	D6089-2	



PILE DATA
12 3/4" O.D. - .25" WALL THICKNESS

	APPROX. LENGTH	No. REQD.	BATTER
SOUTH ABUTMENT	106'	6	VERTICAL
	113'	6	1:3
SOUTH PIER	78'	2	VERTICAL
	79'	4	1:6
NORTH ABUTMENT	118'	6	VERTICAL
	124'	6	1:3
NORTH PIER	88'	2	VERTICAL
	90'	4	1:6
	95'	6	1:3



NOTE:

ABBREVIATIONS:

- O.F. = OUTSIDE FACE
- I.F. = INSIDE FACE
- W.P. = WORKING POINT
- (TYP) = TYPICAL
- (BOTT) = BOTTOM
- N. = NORTH
- S. = SOUTH
- ABUT. = ABUTMENT
- FTG. = FOOTING
- ALT. = ALTERNATE



REVISIONS

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

Giffels ASSOCIATES LIMITED

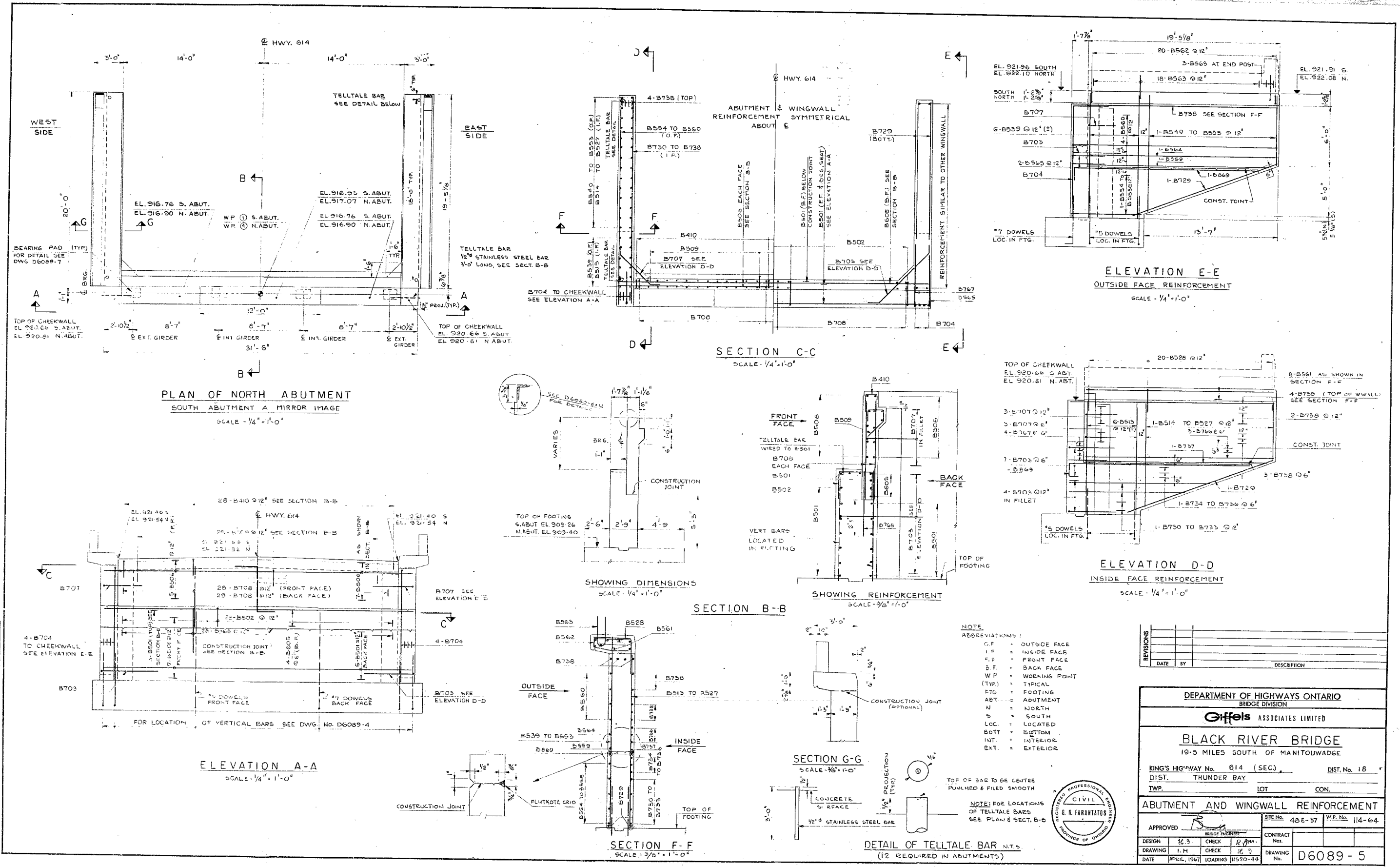
BLACK RIVER BRIDGE
19.5 MILES SOUTH OF MANITOUWADGE

KING'S HIGHWAY No. 614 (SEC.) DIST. No. 18
DIST. THUNDER BAY
TWP. LOT. CON.

FOUNDATION LAYOUT AND REINFORCEMENT

APPROVED: [Signature] BRIDGE ENGINEER
DESIGN: [Signature] CHECK: [Signature]
DRAWING: [Signature] CHECK: [Signature]
DATE: APRIL 1967 LOADING: HS20-44

SITE No. 48E-27 W.P. No. 114-64
CONTRACT No. [Blank]
DRAWING No. D6089-4



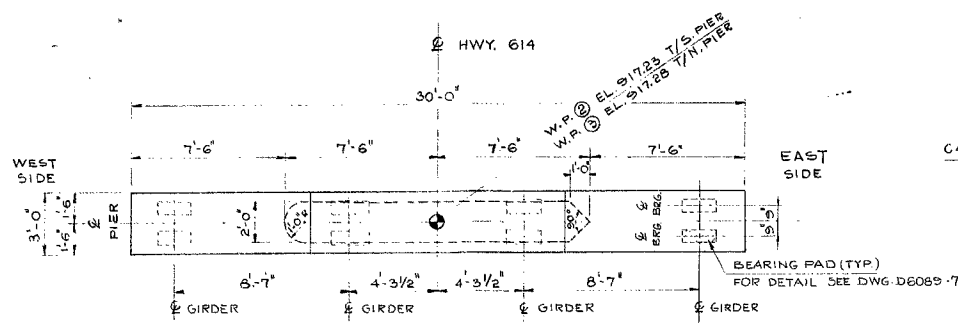
No.	FOR	DATE
1	01	10

NOTE: ABBREVIATIONS:

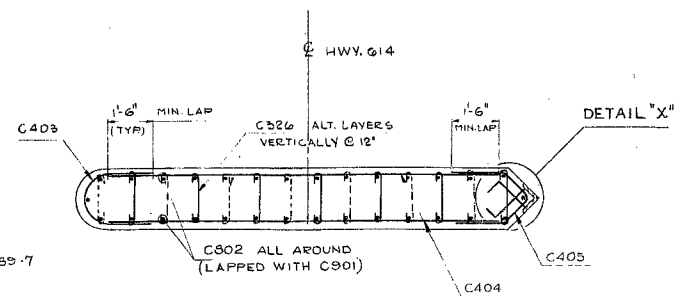
- G.F. = OUTSIDE FACE
- I.F. = INSIDE FACE
- F.F. = FRONT FACE
- B.F. = BACK FACE
- W.P. = WORKING POINT
- (TYP) = TYPICAL
- FTG. = FOOTING
- ABT. = ABUTMENT
- N. = NORTH
- S. = SOUTH
- LOC. = LOCATED
- INT. = INTERIOR
- EXT. = EXTERIOR

REVISIONS	
DATE	DESCRIPTION
DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION Giffels ASSOCIATES LIMITED BLACK RIVER BRIDGE 19.5 MILES SOUTH OF MANITOWADGE KING'S HIGHWAY No. 614 (SEC.) DIST. No. 18 DIST. THUNDER BAY TWP. LOT CON. ABUTMENT AND WINGWALL REINFORCEMENT APPROVED: <i>[Signature]</i> SITE No. 48E-37 W.P. No. 114-64 DESIGN: J.C.S. CHECK: R.P.M. CONTRACT No. DRAWING: I.H. CHECK: J.C.S. DRAWING No. D6089-5 DATE: APRIL, 1967 LOADING: 1520-44	

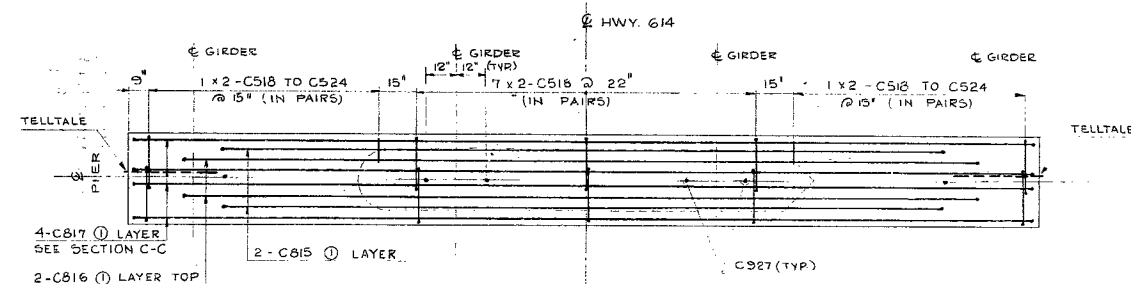




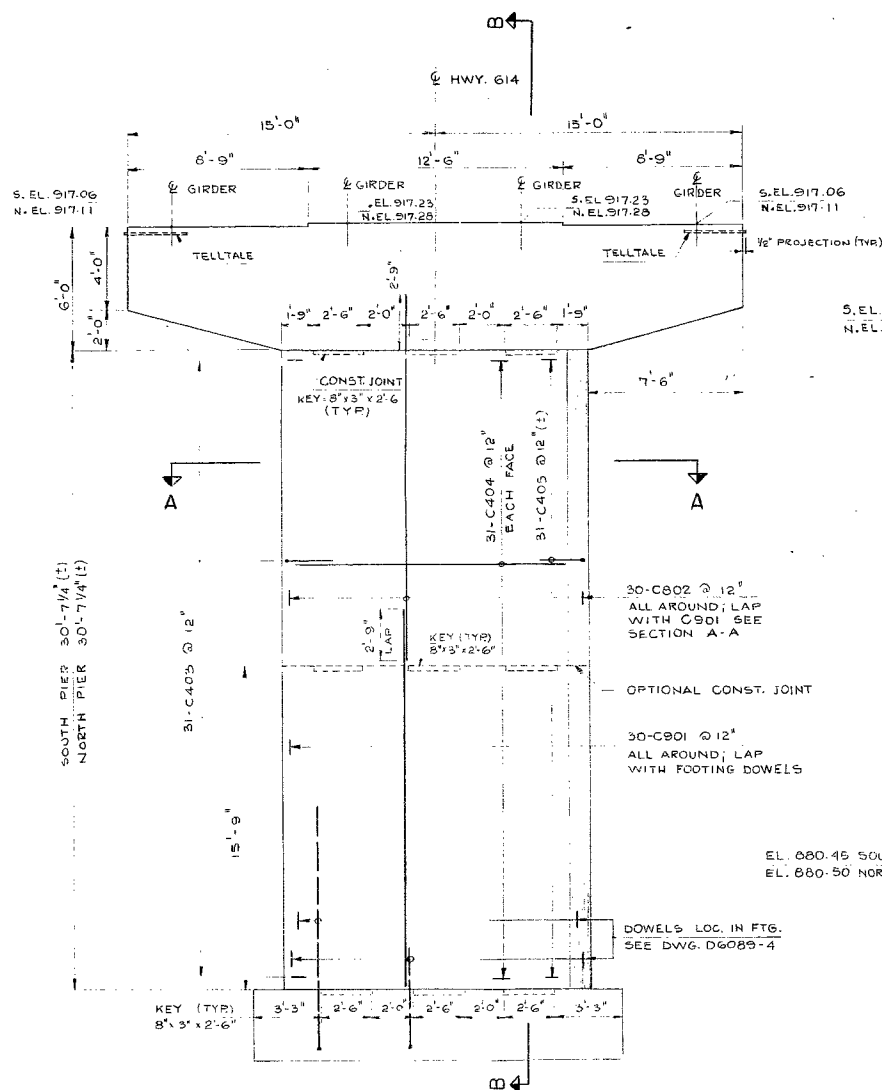
PLAN
SCALE - 1/4" = 1'-0"



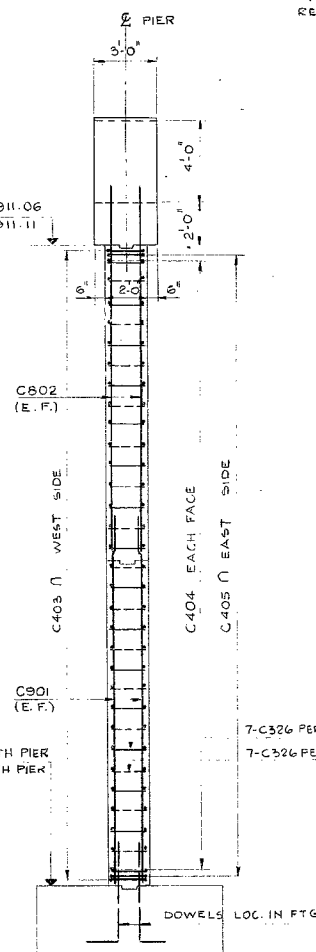
SECTION A-A
SCALE - 3/8" = 1'-0"



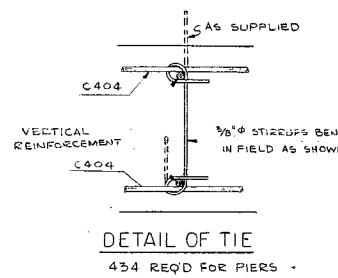
SECTION D-D
(ONLY LAYER ① SHOWN)
SCALE - 3/8" = 1'-0"



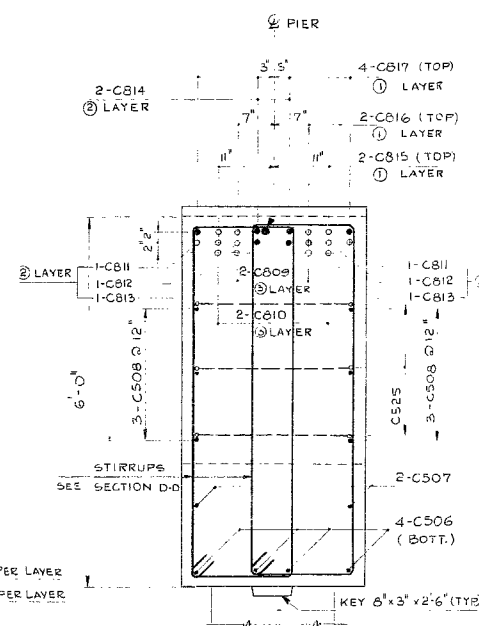
ELEVATION
SCALE - 1/4" = 1'-0"



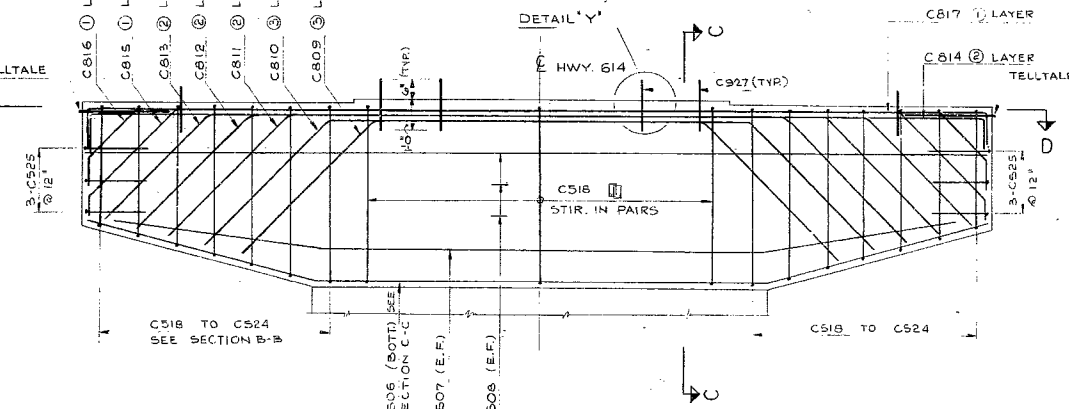
SECTION B-B
SCALE - 1/4" = 1'-0"



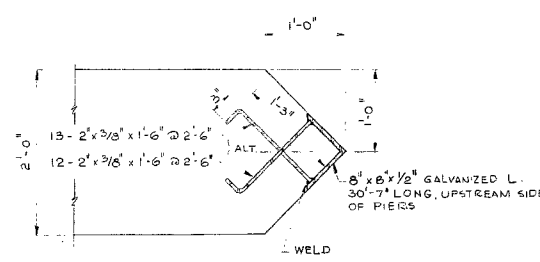
DETAIL OF TIE
434 REQ'D FOR PIERS



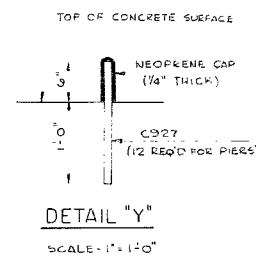
SECTION C-C
SCALE - 3/4" = 1'-0"



DETAIL OF PIER CAP REINFORCEMENT
SCALE - 3/8" = 1'-0"



DETAIL X
SCALE - 1" = 1'-0"



DETAIL Y
SCALE - 1" = 1'-0"

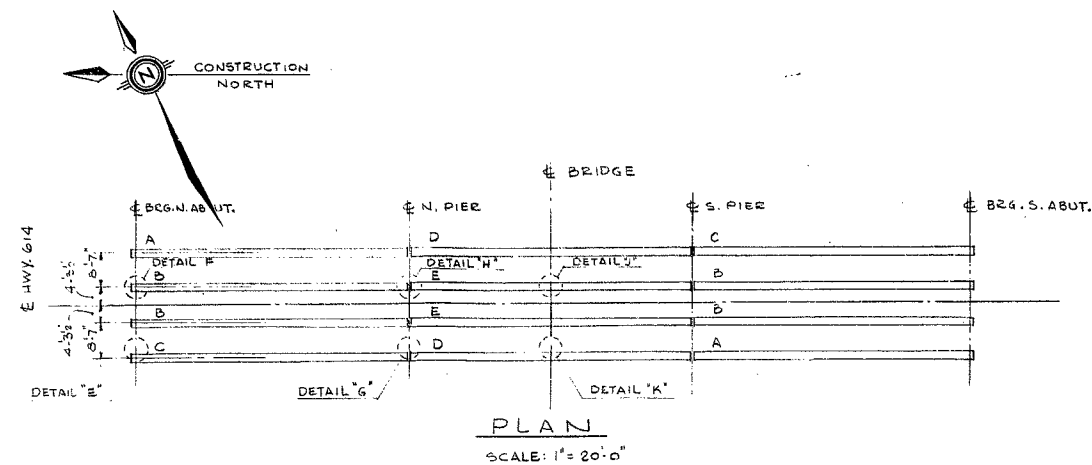
NOTES:
DIMENSIONS AND REINFORCEMENT SAME IN BOTH PIERS.
NUMBER OF BARS SHOWN ARE FOR ONE PIER ONLY.

ABBREVIATIONS: (E.F.) = EACH FACE
(TYP) = TYPICAL
LOC. = LOCATED
FTG. = FOOTING
W.P. = WORKING POINT
T/S = TOP OF SOUTH PIER
T/N = TOP OF NORTH PIER
ALT. = ALTERNATE
CONST. = CONSTRUCTION



REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
Giffels ASSOCIATES LIMITED			
BLACK RIVER BRIDGE 19.5 MILES SOUTH OF MANITOUWADGE			
KING'S HIGHWAY No. 614 (SEC.)		DIST. No. 18	
DIST. THUNDER BAY		LOT CON.	
TWP. LOT CON.			
PIER REINFORCEMENT			
APPROVED	BRIDGE ENGINEER	SITE No. 48 E-37	W.P. No. 114-64
DESIGN R. J. H.	CHECK T. C. Z.	CONTRACT No.	
DRAWING	DATE	DRAWING No.	D6089-6

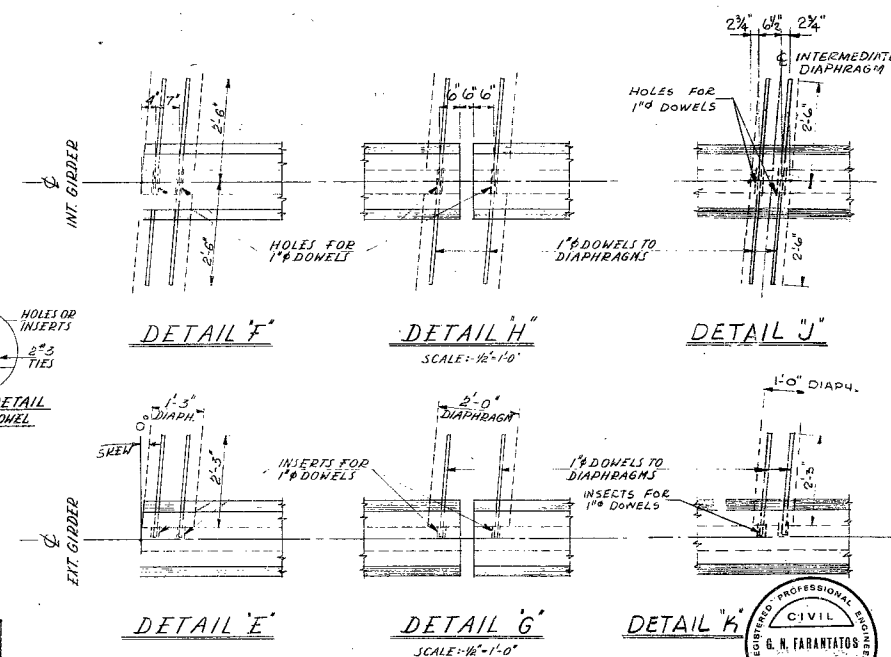
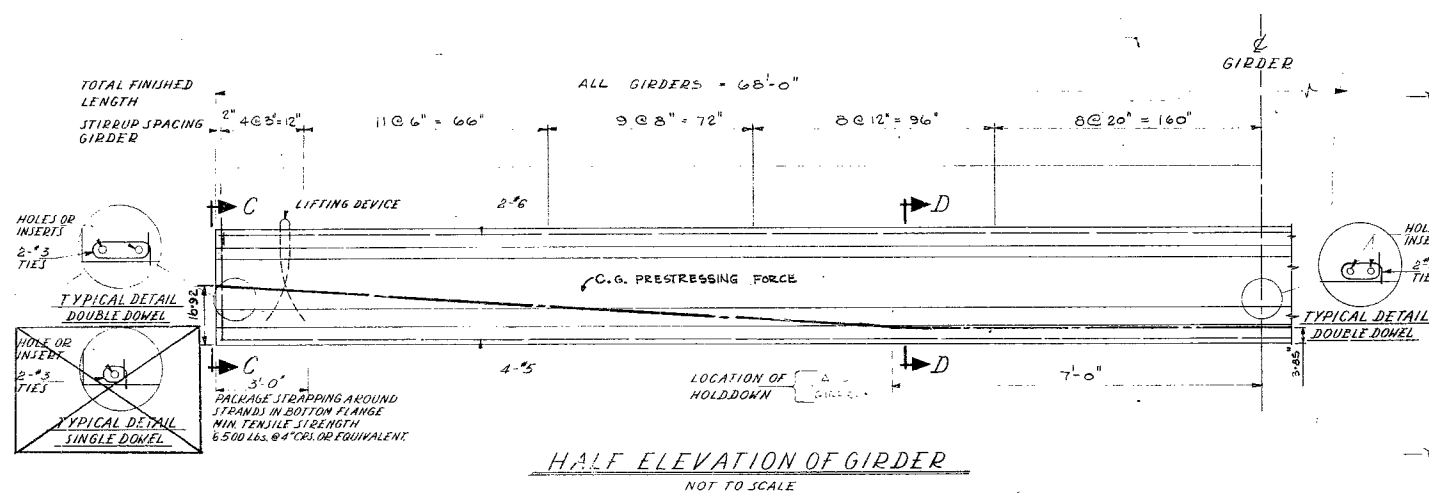
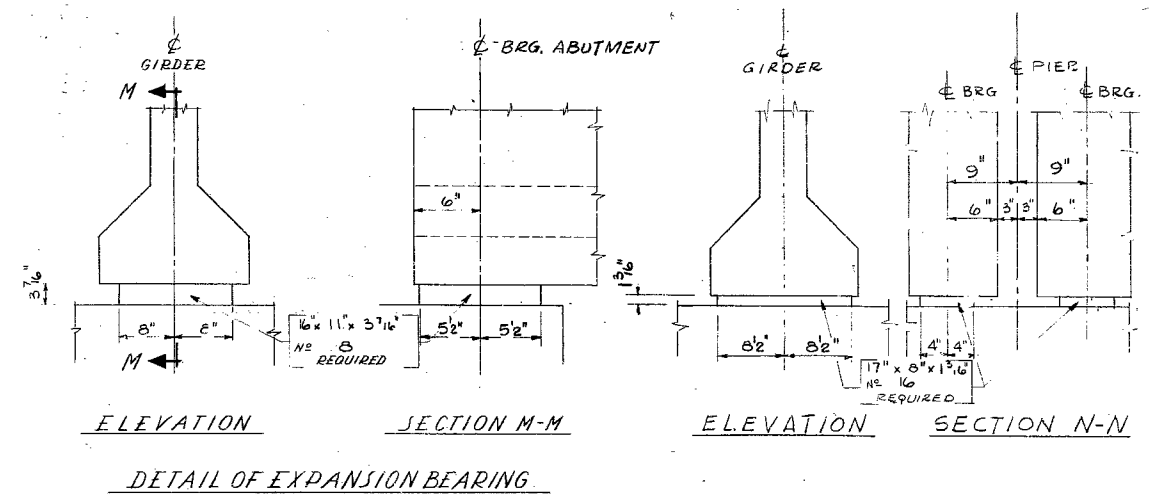


NOTE:
GIRDERS WITH THE SAME LETTERS
ARE INTERCHANGEABLE.
ALL GIRDERS SIMILAR EXCEPT
DETAILS OF INSERTS.

BEARINGS

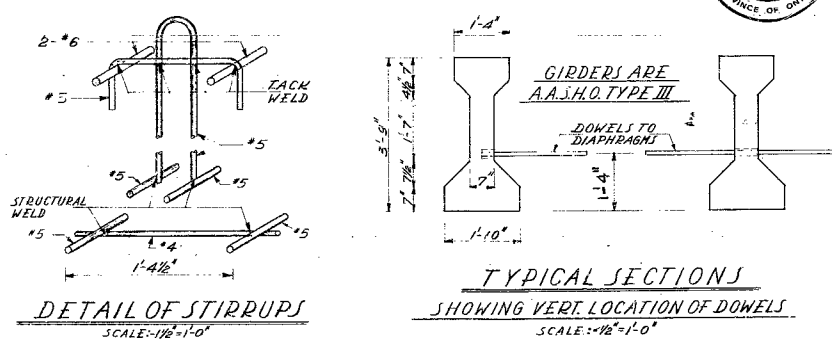
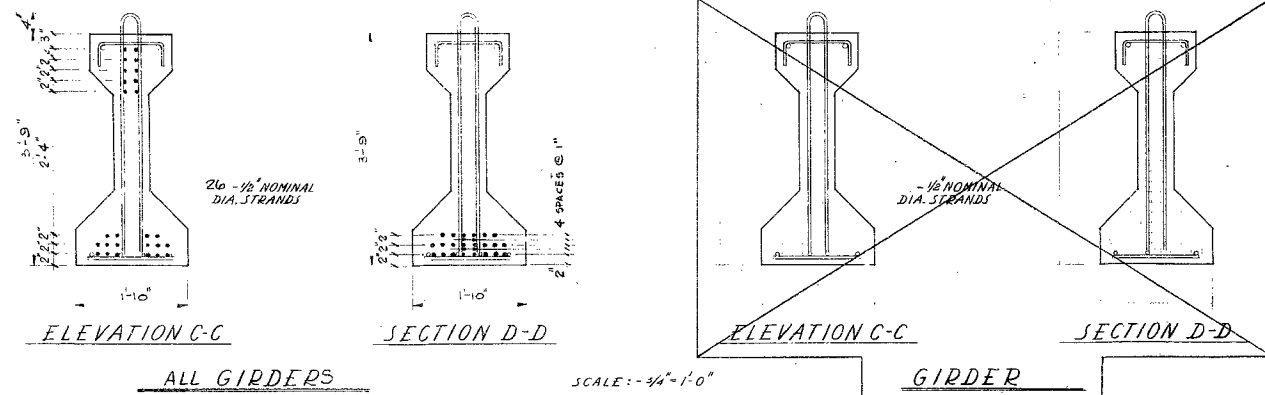
ALL BEARINGS TO BE NATURAL RUBBER/STEEL TYPE AS MADE BY ANDRE RUBBER CO. OF EQUAL.

	ABUTMENT	PIER
D. L.	119 KIPS	118 KIPS
D. L. + L. L.	176 KIPS	132 KIPS
MAX. MOVEMENT	± 1.12 INS.	± .25 INS.
MAX. ALLOWABLE SLEW RATE	7.6 K/INS.	18.6 K/INS.

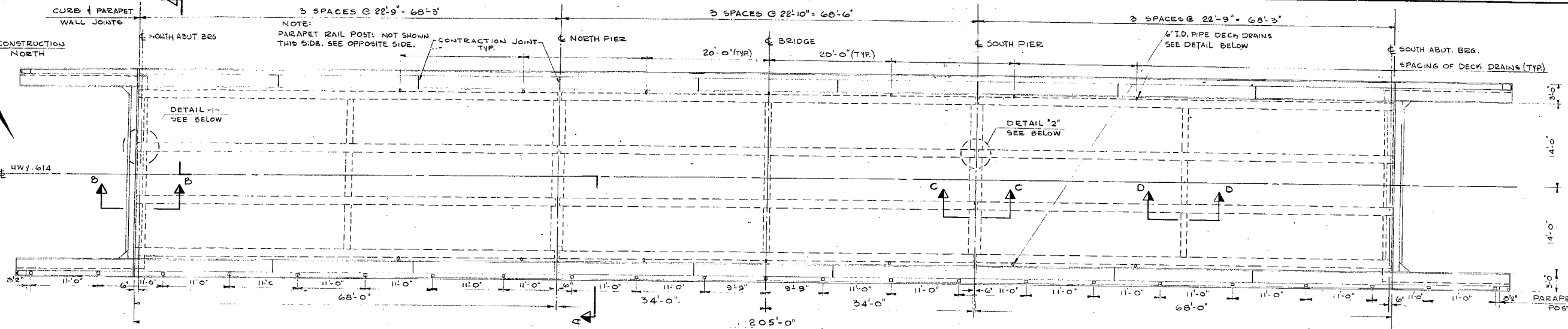


NOTES FOR PRE-TENSIONED GIRDERS

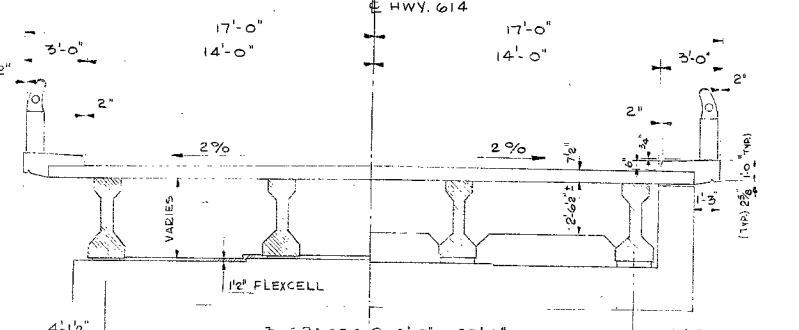
NOTES FOR DOWELS



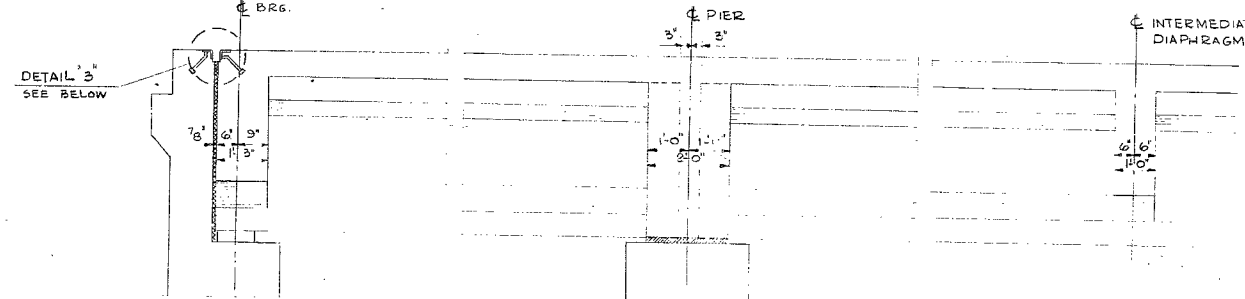
REVISIONS				
DATE	BY	DESCRIPTION		
Giffels ASSOCIATES LIMITED				
DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION				
BLACK RIVER BRIDGE 19-5 MILES SOUTH OF MANITOUWADGE				
KING'S HIGHWAY No.		614 (SEC.)		CIST. No. 18
DIST.		THUNDER BAY		
TWP.	LOT		CON.	
PRESTRESSED GIRDERS & BEARINGS				
APPROVED <i>[Signature]</i> BRIDGE ENGINEER		SITE No. 4B-E-37	W.P. No. 114-64	
DESIGN DRAWING R. J. MCZ	CHECK R. J. MCZ	NO. 3	CONTRACT No.	
DATE APRIL 1967	LOADING H520-44	DRAWING No.	D6089-7	



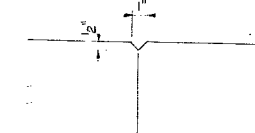
SCALE: 1/8" = 1'-0"



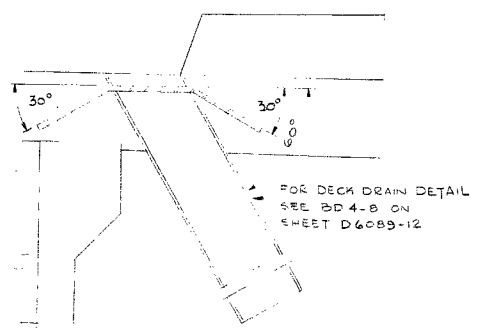
SECTION A-A
SCALE: 1/4" = 1'-0"



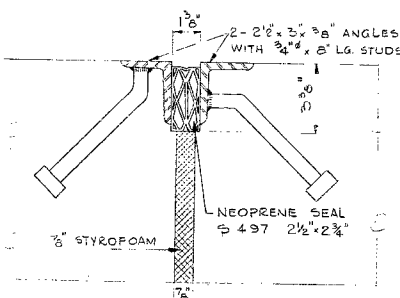
SECTION B-B
SCALE: 1/2" = 1'-0"



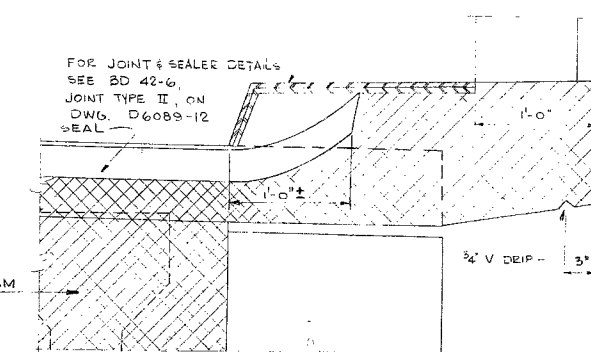
CURB
CONTRACTION JOINT
DETAIL (TYP.)
N.T.S.



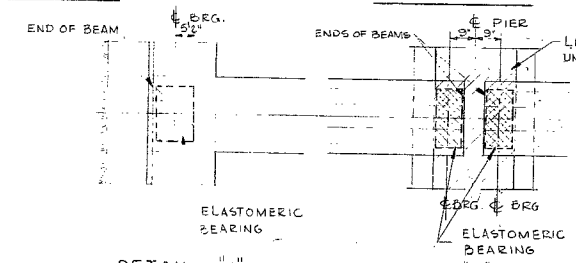
DECK DRAIN DETAIL
SCALE: 1/2" = 1'-0"



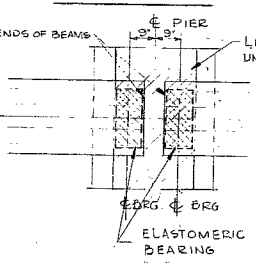
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SCALE: 3" = 1'-0"



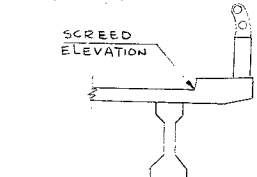
JOINTING MATERIAL AT ABUTMENT JOINT



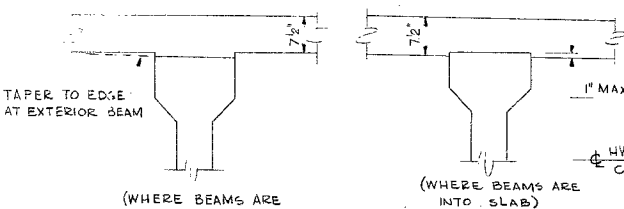
DETAIL "I"
BEAMS AT ABUTMENTS
SCALE: 1/2" = 1'-0"



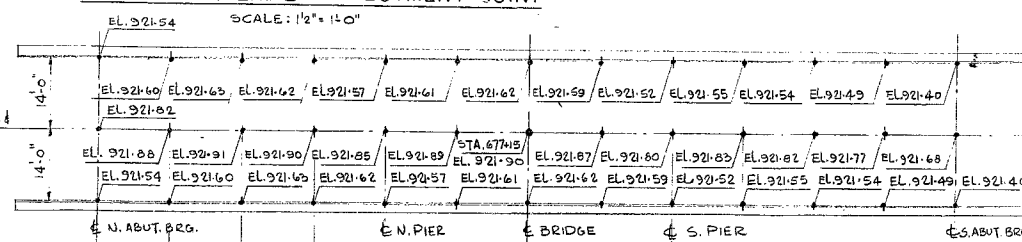
DETAIL 2
BEAMS AT
PIERS



SCREEN ELEVATION (TYPE)



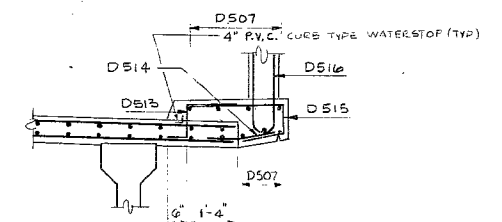
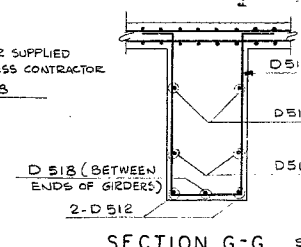
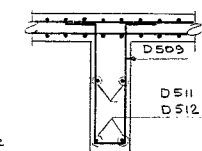
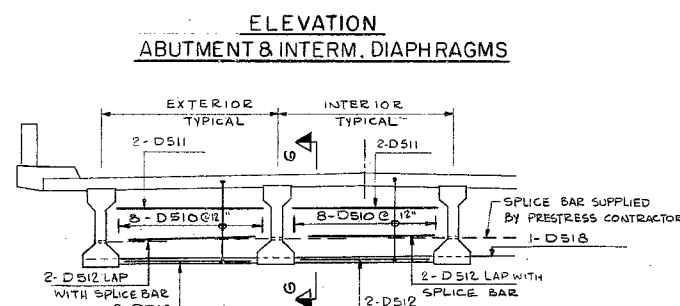
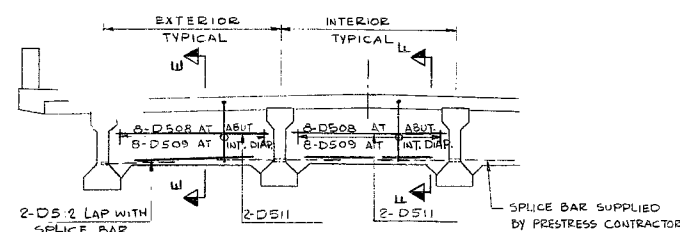
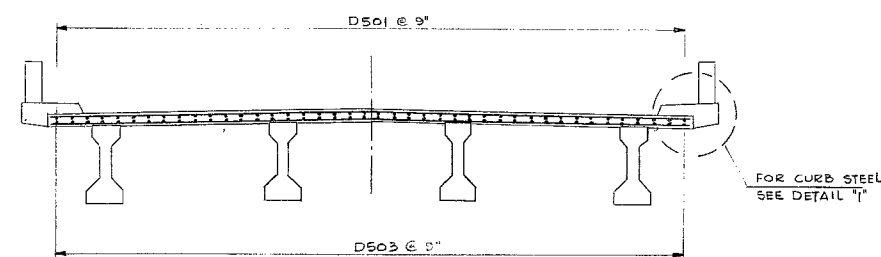
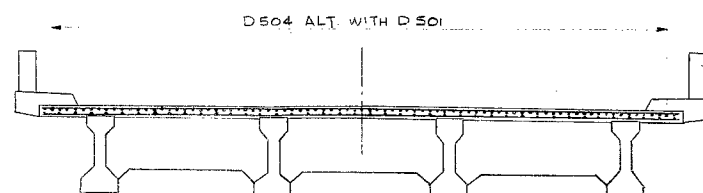
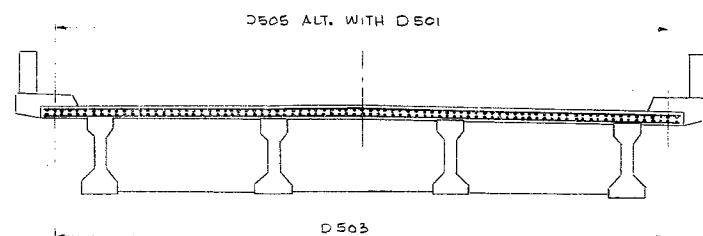
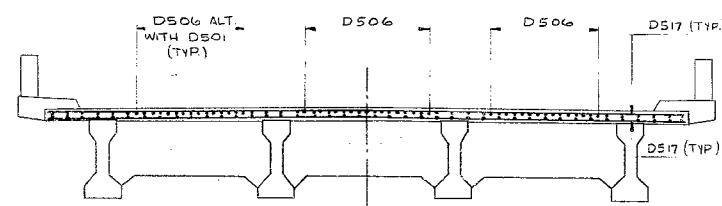
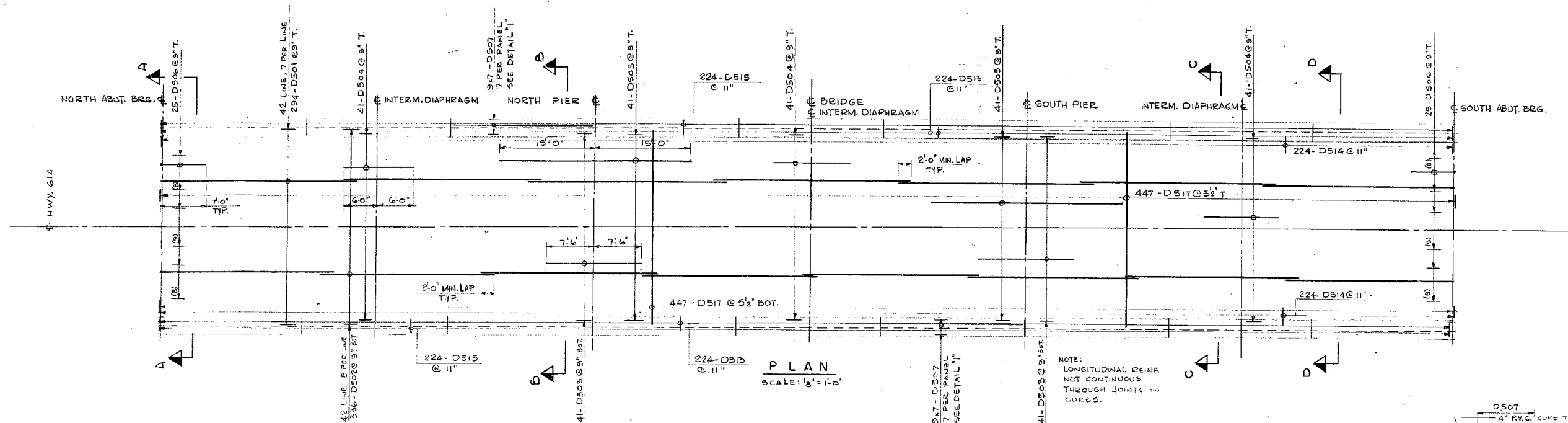
ADJUSTMENTS TO DECK SOFFIT
N.T.S.



SCREED ELEVATIONS N.T.S.

			SCREEN ELEVATION(TYP.)
REVISIONS			
	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION	
Giffels , ASSOCIATES LIMITED.	
BLACK RIVER BRIDGE 19.5 MILES SOUTH OF MANITOUWADGE	
KING'S HIGHWAY No. <u>614 (SEC.)</u>	DIST. No. <u>18</u>
DIST. <u>THUNDER BAY</u>	TWP. <u>LOT</u> <u>CON.</u>



DETAIL "1"
SCALE: 1/2" = 1'-0"

NOTE:

ABBREVIATIONS

ABUT. = ABUTMENT

BEG. = BEARING

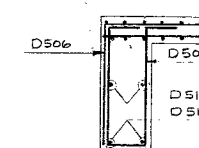
INTERM. = INTERMEDIATE

TYP. = TYPICAL

DIAP. = DIAPHRAGM

REINF. = REINFORCEMENT

P.V.C. = POLY VINYL CHLORIDE

[illegible]

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

Giffels ASSOCIATES LIMITED

BLACK RIVER BRIDGE

19.5 MILES SOUTH OF MANITOUWADGE

KING'S HIGHWAY No. 614 (SEC.) DIST. No. 18

DIST. THUNDER BAY

DECK REINFORCEMENT

SITE No.	48F-37	W.P. No.	114-64
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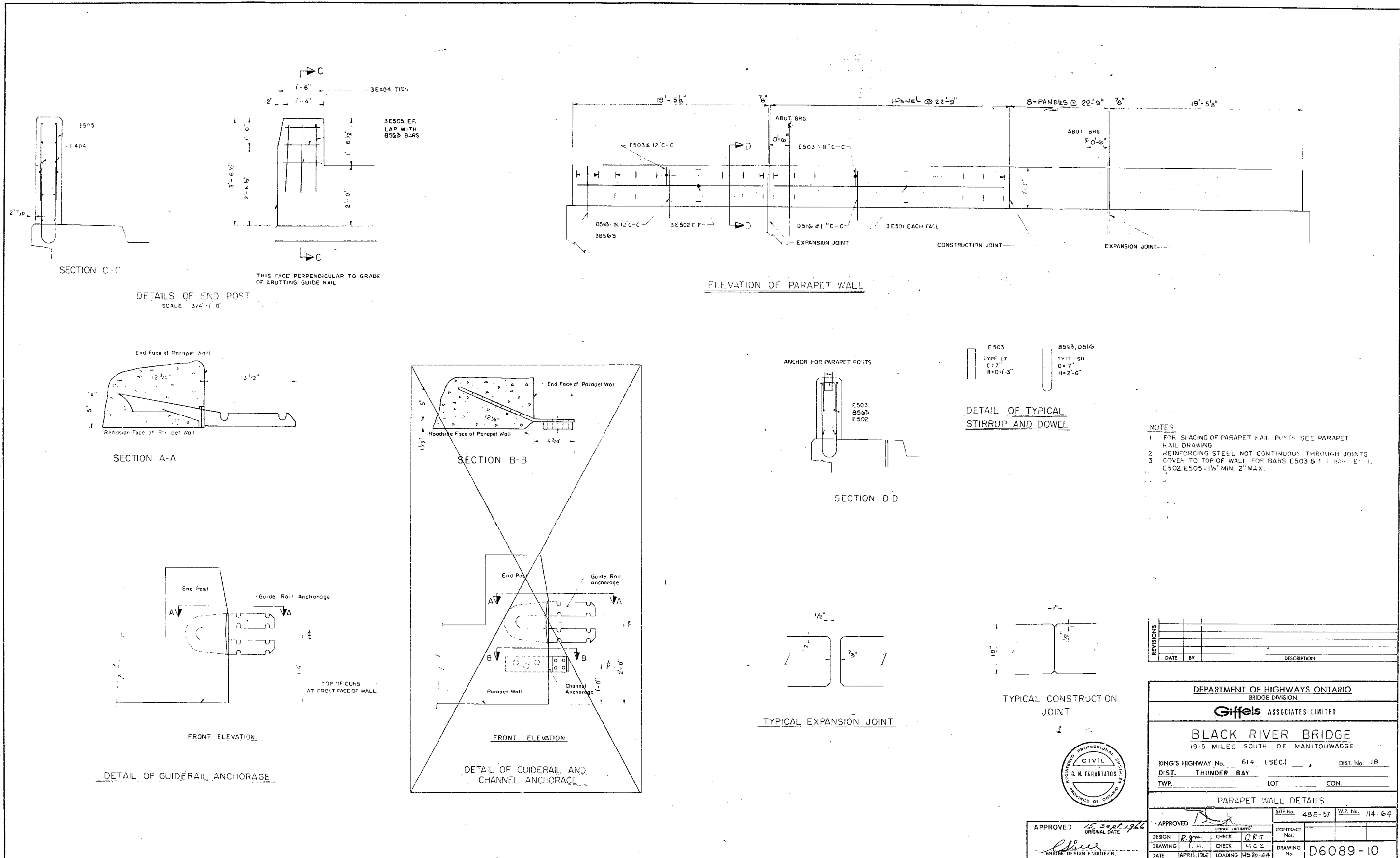
APPROVED 48E-37 114-64

BRIDGE ENGINEER				CONTRACT Nos.
DESIGN	D. Man	CHECK	11.9	

DESIGN	R. Ann.	CHECK	H. J.			
DRAWING	MCZ	CHECK	R. Ann.	DRAWING	DF080-0	

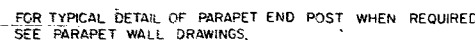
DATE	APRIL, 1967	LOADING	HS20-44
------	-------------	---------	---------

1000



BD 39-6 (REV. SEPT. 1966)

WHERE POST LAYOUT IS NOT SHOWN,
POSTS SHALL BE EQUALLY SPACED EXCEPT
THAT POSTS AT EXPANSION AND CONSTRUCTION
JOINTS SHALL BE MOVED AS REQUIRED TO
ATTAIN 9" CLEARANCE.
(SEE TABLE BELOW FOR POST SPACING LIMITS.)

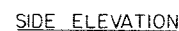


* POST LAYOUT SHALL BE SYMMETRICAL ABOUT C BRIDGE SPAN.

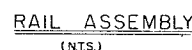
GALVANIZED
DIAMETER

GALVANIZED
DIAMETER

NOTE:
ALL UNSPECIFIED RADII 1/8"

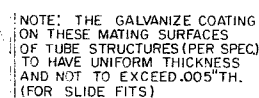


ALTERNATE
JOINT LOCATIONS



OUTSIDE DIA OF UNGALVANIZED
SPLICE TUBE TO BE $5/64" \pm 1/64"$
LESS THAN MEASURED I.D.
OF THE UNGALVANIZED OUTER
TUBE.

SPLICE TUBE



GENERAL NOTES:

- (1) RAIL ELEMENTS SHALL BE STRUCTURAL TUBING SUPPLIED IN ACCORDANCE WITH A.S.T.M. A36.
- (2) STEEL IN POSTS AND RAIL CAPS SHALL BE CAST STEEL SUPPLIED IN ACCORDANCE WITH A.S.T.M. A27-60 GR. 65-35.
- (3) RAIL TUBING SHALL BE SUPPLIED WITH SPlice IN LENGTHS OF 21'-11" (EXCLUDING "PLICE" EXCEPT AS NOTED).
- (4) POSTS, RAILS, WASHERS AND CAPS SHALL BE GALVANIZED IN ACCORDANCE WITH A.S.T.M. A123.
- (5) Q & P RAIL CLASS III. ALL GALVANIZING SHALL BE DONE AFTER FABRICATION.
- (6) ELECTRODES SHALL BE HYDROGEN SPECIFICATION E7018, E7018 OR E7018.
- (7) ANCHORAGE AND BOLTS SHALL BE RICHMOND TYPE D-GR1 OR EQUAL, MODIFIED AS SHOWN.
- (8) INSERTS AND ANCHOR BOLTS SHALL BE GALVANIZED IN ACCORDANCE WITH A.S.T.M. A153-61.
- (9) BOLTS SHALL BE GIVEN A LIBERAL COATING OF WHITE NON-STAINING GREASE.

ERECTION NOTES:

- (1) RAIL TUBING SHALL BE BENT TO FOLLOW CURVATURE OF ROAD.
- (2) RAIL POSTS SHALL BE SET PERPENDICULAR TO GRADE.
- (3) WHERE LAYOUT OF POSTS IS NOT KNOWN, POST LOCATION SHALL BE DETERMINED BY THE CONTRACTOR.
- (4) SET SCREWS SHALL NOT BE TIGHTENED ON POSTS ADJACENT TO EXPANSION JOINTS.
- (5) RAIL SHALL BE CUT AS REQUIRED WITH PIPE CUTTERS AND CUT SURFACE TREATED WITH ZINC RICH PAINT.
- (6) WHEN CONNECTING TO EXISTING RAIL, RAIL MUST BE MADE CONTINUOUS AND POST SPACING DETERMINED WITH REFERENCE TO EXISTING POST.

SUMMARY

SUMMARY			APPROX. OF RAIL	
ITEM	NO.	L.F.	LOCATION	
POST INCLUDING SET SCREWS & ANCHORAGE INCLUDING BOLTS WASHERS AND TEMPLATE	46			
END CAPS (INCLUDING SET SCREWS)	8			
3 1/2" Ø TUBE (21'-11" LENGTHS)	40	576-66		
3 1/2" Ø TUBE WITHOUT SPLICE (21'-6" LENGTHS)	4	820		

[illegible]

* NOTE:
SCALE: 1/2" = 1" (EXCEPT WHERE OTHERWISE INDICATED)

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

Giffels ASSOCIATES LIMITED

BLACK RIVER BRIDGE

19.5 MILES SOUTH OF MANITOUWADGE

KING'S HIGHWAY No. 614 (SEC.) DIST. No. 18
E. ST. THUNDER BAY
 TWP. _____ LOT _____ CON. _____

=STANDARD STEEL PARAPET RAIL=

APPROVED	SITE No.	48E-37	W.P. No.	114-64

APPROVED <i>[Signature]</i>		BRIDGE ENGINEER	CONTRACT		
REGION	CD	CLERK	No.		

DESIGN	R gm.	CHECK	G.R.T.	NO.			
DRAWING	MCZ	CHECK	I.H.	DRAWING No.	D6089-11		
DATE	M2211 1962	LOADING	Use 20-44				

3D 40-3 , REV. NOV. 1966

REV. MAY '66
BD 4-8

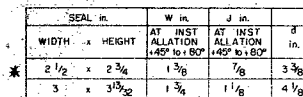


DETAILS OF MINIMUM GRANULAR BACKFILL REQUIREMENT

NOTE:
SECTION PERPENDICULAR TO ABUTMENT.
LATERAL LIMITS - INSIDE FACE TO INSIDE FACE OF WINGWALLS.

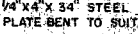
DIST. N°	APPROX. FROST PENETRATION.
1 TO 7	4 FT.
8 TO 11	5 FT.
13, 14, 17 AND 18	6 FT.
16, 19 AND 20	7 FT.

60-42-1
103-196



- JOINT SEALS TO BE PRESSED INTO JOINT USING MANUFACTURERS APPROVED LUBRICANT APPLIED TO BOTH WALLS OF GROOVE.
- JOINTS SHALL BE CLEAN, DRY AND FREE OF ALL FOREIGN MATERIALS.
- ALL MFTAL SHALL BE HOT DIPPED GALVANIZED IN ACCORDANCE WITH ASTM - A123.
- ANCHOR STUDS TO BE AT 18" CRS. EACH SIDE FOR CURB PLATES.

BD-82-1
REV. FEB. 1967



NOTE: SEE BD 100-2
FOR SIZE & SPACING
OF FIGURES.



LOCATION OF BRIDGE SITE & DATE FIGURES

[illegible][illegible]

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

Giffels ASSOCIATES LIMITED

BLACK RIVER BRIDGE

KING'S HIGHWAY No. 412 (CEG) DIST. No. 18

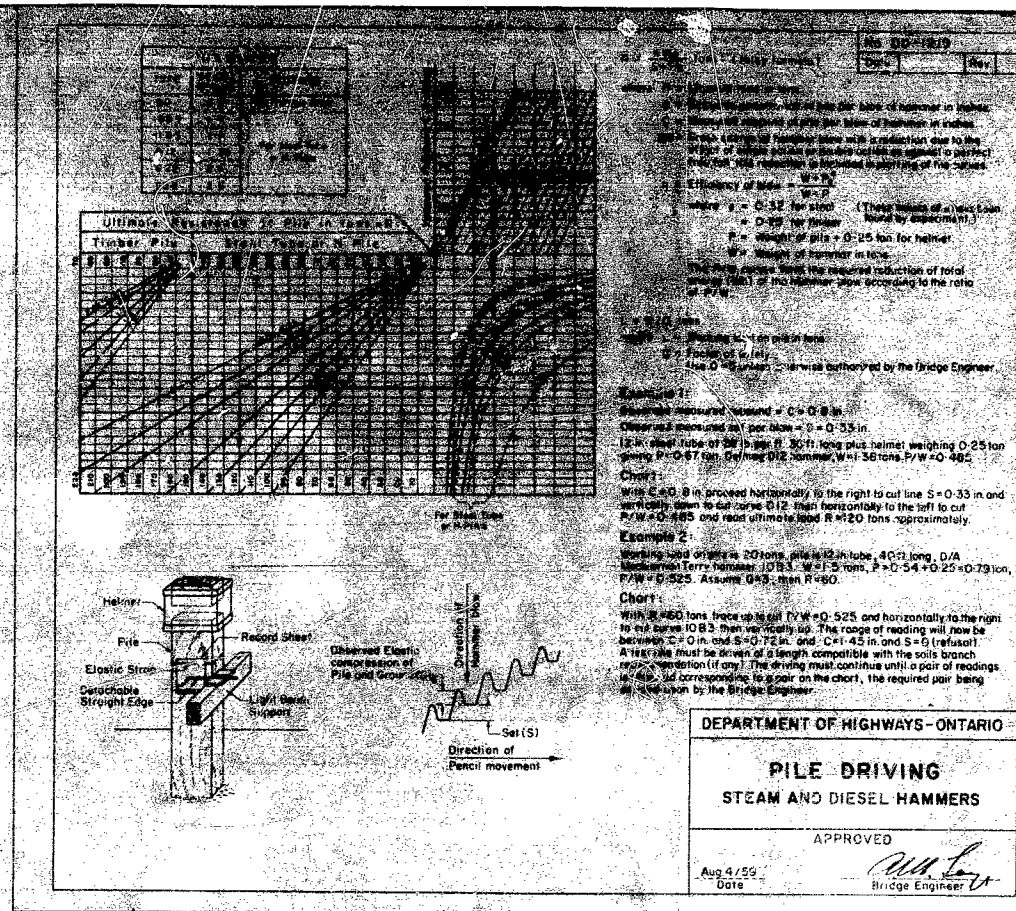
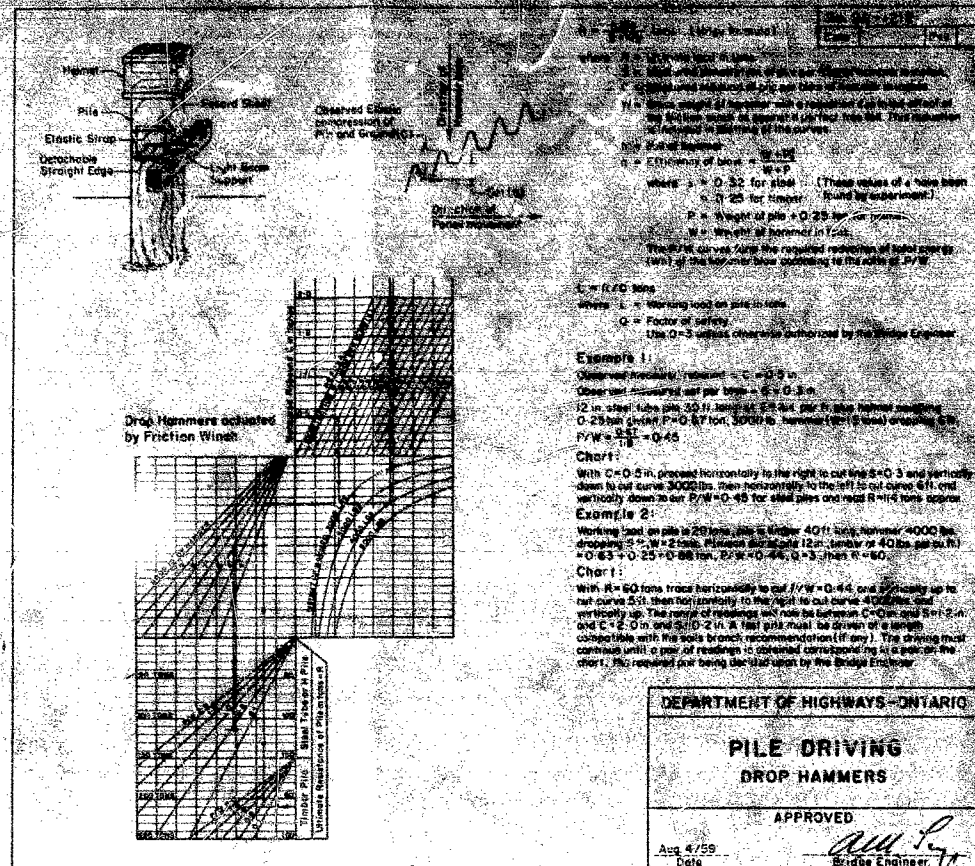
KING'S HIGHWAY NO. _____ DIST. NO. _____
 DIST. _____ TOWNSHIP DAY _____
 TWP _____ LOT _____ CON. _____

STANDARD BRIDGE DETAIL

APPROVED	SITE No.	495	W.P. No.	114-64

DESIGN		CHECK		CONTRACT Nos.			
DRAWING		CHECK					
DATE		LOADING		DRAWING No.	D6089-12		





REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE DIVISION

Giffels ASSOCIATES LIMITED

BLACK RIVER BRIDGE
2.5 MILES SOUTH OF MANTON, ONTARIO

KING'S HIGHWAY No. 6-1000 DIST. No. 1000
DIST. TUNNEL No. 1000
TWP. 1000 LOT 1000 CON. 1000

STANDARD BRIDGE DETAILS

APPROVED *[Signature]* BRIDGE ENGINEER
DESIGNING M.C.Z. CHECK
DRAWING M.C.Z. CHECK
DATE 11-11-67

SITE No. 448-31 14-24
CONTRACT No. 1000
DRAWING No. 1000



Drop Hammers actuated by Friction Winch

$R = \frac{nWh}{S+C}$ tons (Hiley formula)

where: R = Ultimate load in tons.
 S = Measured penetration of pile per blow of hammer in inches.
 C = Measured rebound of pile per blow of hammer in inches.
 W = Gross weight of hammer with a reduction due to the effect of the friction which acts against a perfect free fall. This reduction is included in plotting of the curves.
 h = Fall of hammer.
 n = Efficiency of blow = $\frac{W+P^2}{W+P}$
where e = 0.32 for steel (These values of e have been found by experiment.)
= 0.25 for timber
 P = Weight of pile + 0.25 ton for helmet.
 W = Weight of hammer in tons.

The P/W curves form the required reduction of total energy (Wh) of the hammer blow according to the ratio of P/W .

$L = R/Q$ tons
where L = Working load on pile in tons.
 Q = Factor of safety.
Use $Q=3$ unless otherwise authorized by the Bridge Engineer.

Example 1:
Observed measured rebound = $C=0.5$ in.
Observed measured set per blow = $S=0.3$ in.
12 in steel tube pile 30 ft long of 28 lbs. per ft. plus helmet weighing 0.25 ton giving $P=0.67$ ton, 3000 lb. hammer ($W=1.5$ tons) dropping 5 ft.
 $P/W=0.44$
 $P/W=0.45$

Chart:
With $C=0.5$ in. proceed horizontally to the right to cut line $S=0.3$ and vertically down to cut curve 3000 lbs. then horizontally to the left to cut curve 6 ft. and vertically down to cut $P/W=0.45$ for steel piles and read $R=114$ tons approx.

Example 2:
Working load on pile is 20 tons, pile is timber 40 ft long, hammer 4000 lbs. dropping 5 ft. $W=2$ tons. P (mean dia. of pile 12 in, timber of 40 lbs. per cu ft.) = 0.63 + 0.25 = 0.88 ton, $P/W=0.44$, $Q=3$, then $R=60$.

Chart:
With $R=60$ tons trace horizontally to cut $P/W=0.44$ and vertically up to cut curve 5 ft. then horizontally to the right to cut curve 4000 lbs. and vertically up. The range of readings will now be between $C=0$ in. and $S=1.2$ in. and $C=2.0$ in. and $S=0.2$ in. A test pile must be driven of a length compatible with the soils branch recommendation (if any). The driving must continue until a pair of readings is obtained corresponding to a pair on the chart, the required pair being decided upon by the Bridge Engineer.

DEPARTMENT OF HIGHWAYS-ONTARIO

PILE DRIVING
DROP HAMMERS

APPROVED

Aug 4/59
Date

Bridge Engineer

Steam and Diesel Hammers

$R = \frac{nWh}{S+C}$ tons (Hiley formula)

where: R = Ultimate load in tons.
 S = Measured penetration of pile per blow of hammer in inches.
 C = Measured rebound of pile per blow of hammer in inches.
 W = Gross energy of hammer blow with a reduction due to the effect of single action or double action as against a perfect free fall, this reduction is included in plotting of the curves.
 n = Efficiency of blow = $\frac{W+P^2}{W+P}$
where e = 0.32 for steel (These values of e have been found by experiment.)
= 0.25 for timber
 P = Weight of pile + 0.25 ton for helmet.
 W = Weight of hammer in tons.

The P/W curves form the required reduction of total energy (Wh) of the hammer blow according to the ratio of P/W .

$L = R/Q$ tons
where L = Working load on pile in tons.
 Q = Factor of safety.
Use $Q=3$ unless otherwise authorized by the Bridge Engineer.

Example 1:
Observed measured rebound = $C=0.8$ in.
Observed measured set per blow = $S=0.33$ in.
12 in steel tube of 28 lb. per ft. 30 ft long plus helmet weighing 0.25 ton giving $P=0.67$ ton, Delmag D12 hammer, $W=1.38$ tons, $P/W=0.485$.

Chart:
With $C=0.8$ in. proceed horizontally to the right to cut line $S=0.33$ in. and vertically down to cut curve D12 then horizontally to the left to cut $P/W=0.485$ and read ultimate load $R=120$ tons approximately.

Example 2:
Working load on pile is 20 tons, pile is 12 in. tube, 40 ft long, D/A Mackiernan Terry hammer 10B3, $W=1.5$ tons, $P=0.54$ + 0.25 = 0.79 ton, $P/W=0.525$. Assume $Q=3$, then $R=60$.

Chart:
With $R=60$ tons trace up to cut $P/W=0.525$ and horizontally to the right to cut curve 10B3 then vertically up. The range of reading will now be between $C=0$ in. and $S=0.72$ in. and $C=1.45$ in. and $S=0$ (refusal). A test pile must be driven of a length compatible with the soils branch recommendation (if any). The driving must continue until a pair of readings is obtained corresponding to a pair on the chart, the required pair being decided upon by the Bridge Engineer.

DEPARTMENT OF HIGHWAYS-ONTARIO

PILE DRIVING
STEAM AND DIESEL HAMMERS

APPROVED

Aug 4/59
Date

Bridge Engineer

DEPARTMENT OF HIGHWAYS-ONTARIO

Giffels ASSOCIATES LIMITED

PROJECT: HIGHWAY 7

TWP: LCT

APPROVED: [Signature]

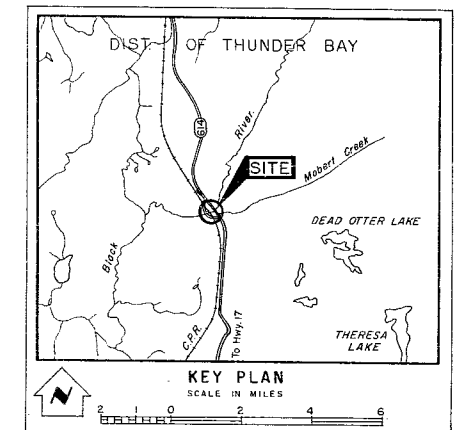
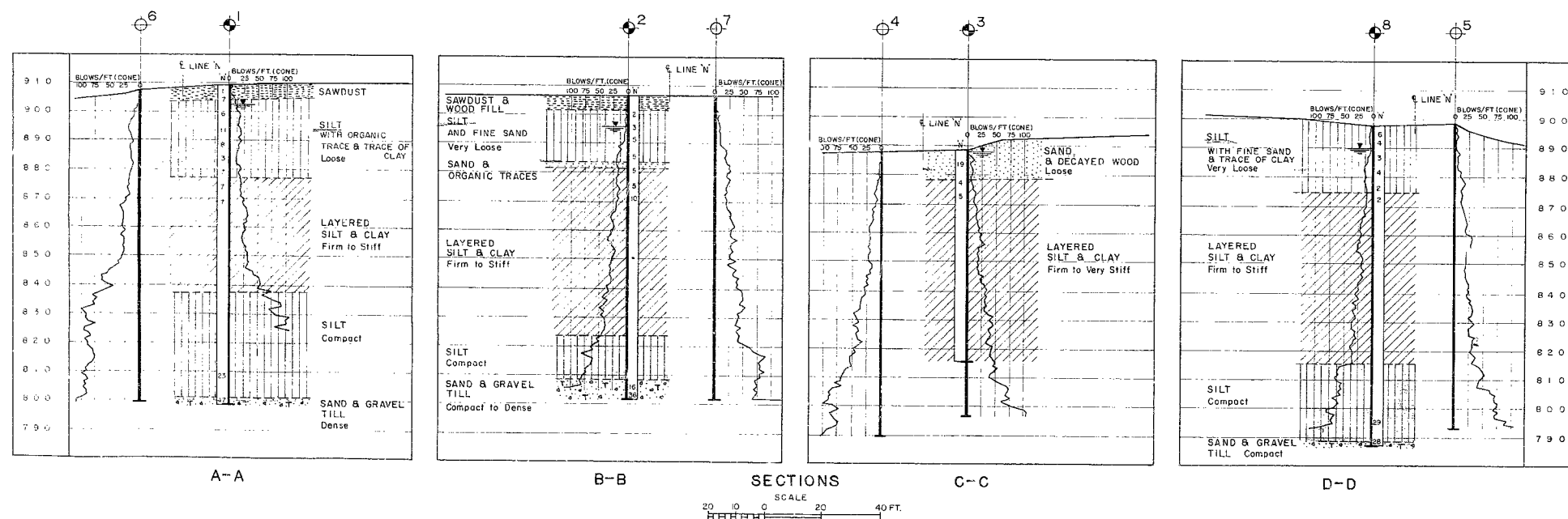
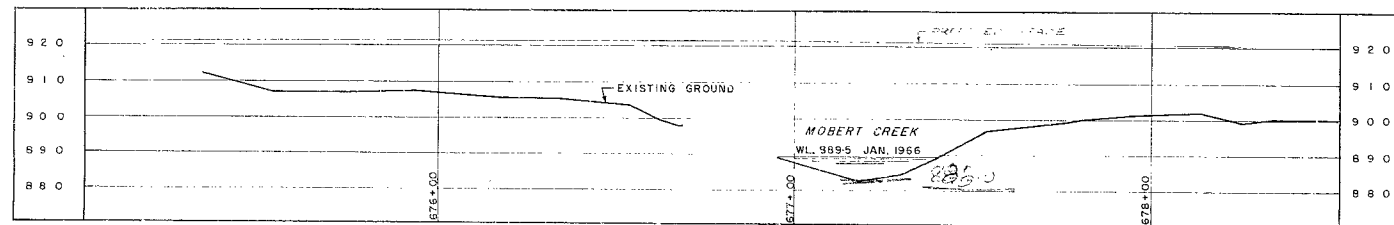
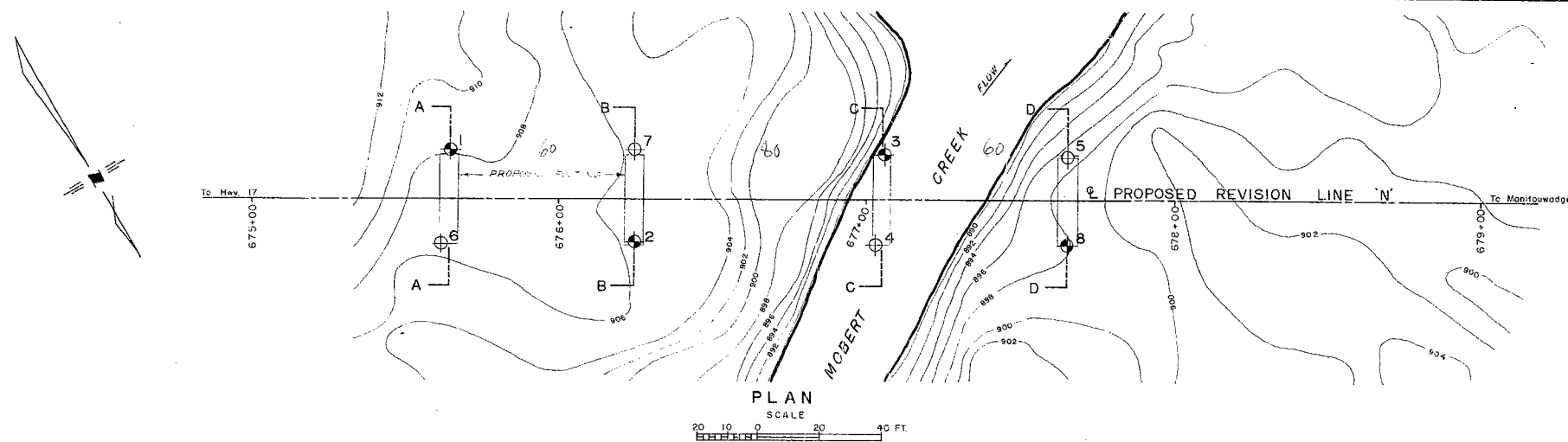
DATE: [Blank]

DESIGNED: [Blank]


CHECKED: [Blank]

DATE: [Blank]


DATE: [Blank]




LEGEND




Bore Hole



Cone Penetration Hole



Bore & Cone Penetration Hole



Water Levels established at time of field investigation. (Sept. 1966)

NO.	ELEVATION	STATION	OFFSET
1	909.4	675+65	16' LT.
2	906.8	676+25	14' RT.
3	889.5	677+06	15' LT.
4	887.8	677+03	15' RT.
5	838.4	677+65	14' LT.
6	907.5	675+62	15' RT.
7	906.5	676+25	16' LT.
8	897.6	677+65	15' RT.

BM 904-16 GEODETIC DATUM
N & W. in 0-4' Spruce Stg.
150' Rt. of Sta. 675+10

- 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.

REVISIONS			
DATE	BY	DESCRIPTION	

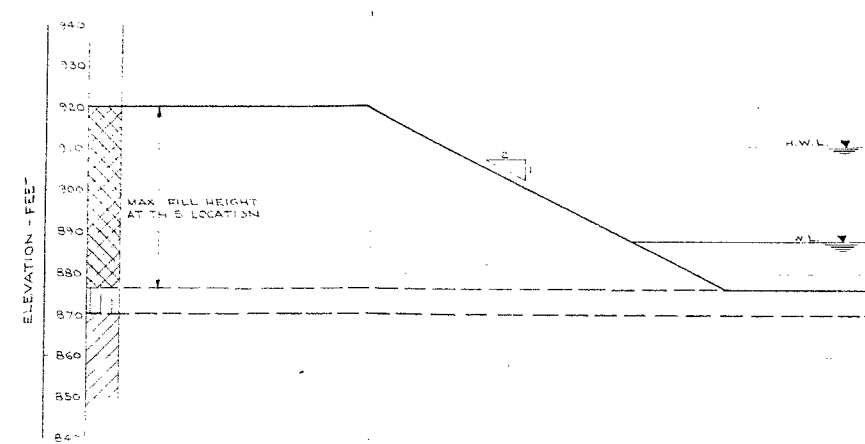
GEOCON LTD
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

BLACK RIVER DIVERSION

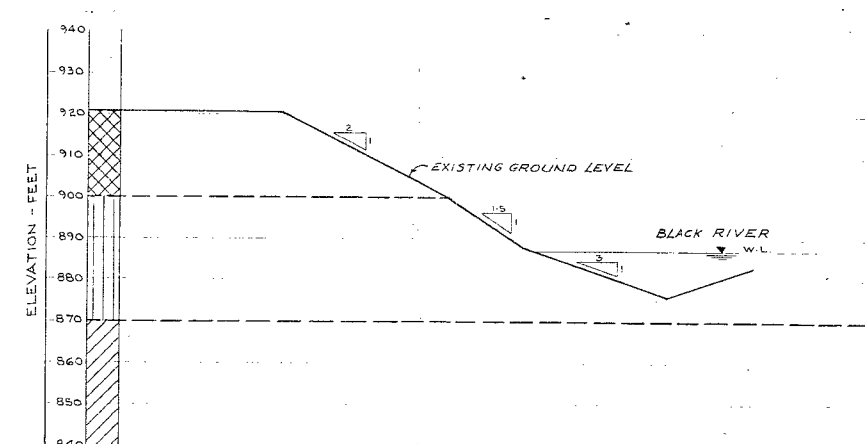
KING'S HIGHWAY NO. 514 (SEC.) DIST. NO. 18
DIST. THUNDER BAY
TWP. _____ LOT _____ CON. _____

BORE HOLE LOCATIONS & SOIL STRATA

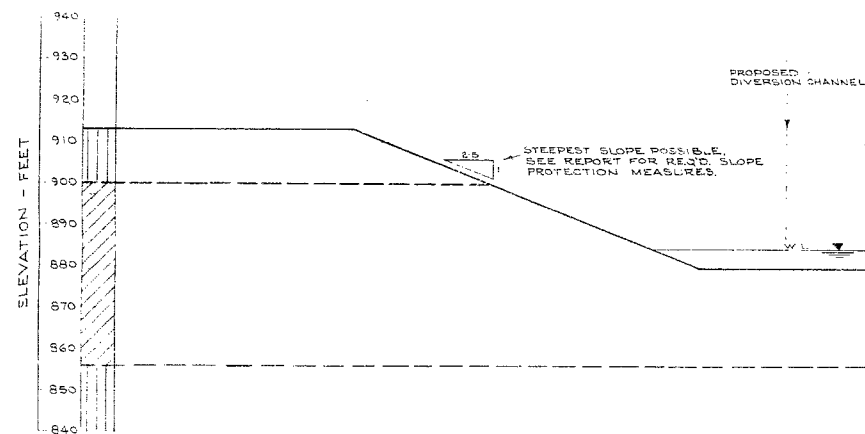
SUBM'D H.L.M.	CHECKED D.B.O.	WP NO. 114 - 64	DRAWING NO. T7923-1
DRAWN A.E.L.	CHECKED H.L.M.	NO	
DATE OCT. 7, 1966	SITE NO.		BRIDGE DRAWING NO.
APPROVED <i>D.B.O.</i>	CONT. NO.		



EMBANKMENT SLOPE AT BLACK RIVER CROSSING



EMBANKMENT SLOPE ADJACENT TO BLACK RIVER
(AT LOCATION WHERE NEW FILL PARALLELS EXISTING RIVER BANK)



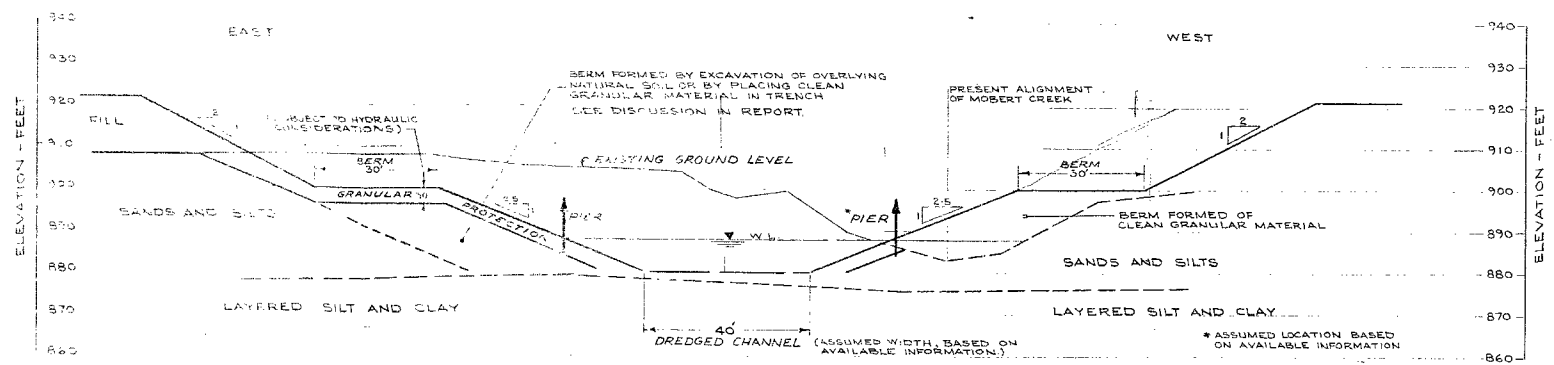
DIVERSION SLOPE

LEGEND

- FILL
- SILTS AND SANDS
- LAYERED SILT AND CLAY

NOTE:

1. THIS DRAWING IS INTENDED FOR ILLUSTRATION PURPOSES ONLY
2. IT SHOULD BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT



LONGITUDINAL SECTION AT PROPOSED BRIDGE SITE

REVISIONS			REFERENCE			REFERENCE			DEPARTMENT OF HIGHWAYS, ONTARIO			GEOCON LTD		
MARK	DATE	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION	DESCRIPTION	DESCRIPTION	DATE	SCALE	MADE	CHKD.	APPD.	NO.
			T7923-1	GEOCON LTD - BLACK RIVER DIVERSION BOREHOLE LOCATIONS AND SOIL STRATA DATED: OCT. 7 & 12, 1966	E-4546-1	DEPARTMENT OF HIGHWAYS, ONTARIO - BRIDGE SITE PROPOSED CROSSING AT BLACK RIVER DIVERSION AND SEC. HWY. 64, REV. LINE N-DATED: APR 1966	DEPARTMENT OF HIGHWAYS, ONTARIO - BRIDGE SITE PROPOSED CROSSING AT BLACK RIVER DIVERSION AND SEC. HWY. 64, REV. LINE N-DATED: APR 1966	DEPARTMENT OF HIGHWAYS, ONTARIO - BRIDGE SITE PROPOSED CROSSING AT BLACK RIVER DIVERSION AND SEC. HWY. 64, REV. LINE N-DATED: APR 1966	OCT. 21, 1966	1" = 20'-0"	AEL	WLD	JOE	T7923-3
			T7923-2		WR 114-64	GIFFELS ASSOCIATES, LIMITED - CONSULTING ENGINEERS - TORONTO - PLAN OF PROPOSED DIVERSION - DATED: SEPT. 21, 1966								

February 7, 1967

Proposed Black River Bridge & Diversion
Manitouwadge, Ontario
-- District #18 (Sault Ste. Marie) --

This is to confirm our discussion of Friday, February 3, 1967, regarding the necessity of stage construction on the above mentioned crossing.

In the covering memo to the foundation report, dated October 26, 1966, we have recommended that staging of the grading and bridge construction be implemented. This recommendation was made on the basis of findings regarding the subsoil conditions as presented in the soils report and a hypothetical bridge design.

At a later stage, the preliminary bridge design was prepared, and it became obvious that although desirable, it would be quite difficult to implement stage construction because of the following reasons:

(a) The two piers are located in the bottom of the diversion and the canal has to be excavated before pier construction can be commenced.

(b) On the west side of the diversion the present ground topography is such that filling operations are required to build the bank of the canal. This is also required in order to reach the elevation from which the piles for the west abutment are to be driven.

From the above, it becomes obvious that, unless some very special construction procedures are followed, the most logical construction sequence is as follows:

(1) Excavate the diversion canal to the required depth and shape it (by filling on the west side) to the required profile.

(2) Drive piles for piers and abutments and construct footings.

February 7, 1967

- (3) Build piers and abutments.
- (4) Build bridge.
- (5) Backfill and build approaches.

changes: To the above schedule, we would suggest the following

- (1) As above.
- (2) Build approach embankments.
- (3) Drive piles for piers and abutments and construct footings.
- (4) Build piers and abutments.
- (5) Backfill abutments.
- (6) Build bridge.

The change in sequence is suggested because it is felt that it is desirable to have the approach embankments built prior to pile driving. Once the approach embankments are built, the stability increases with time and the most critical phase has passed.

The pile driving may have some detrimental influence on the stability of the banks, but we are of the opinion that this will not be critical.

The requirement to build the bridge last, is desirable because it incorporates the safeguard that a stable condition prevails by the time the superstructure is being erected. Any failure that would have occurred earlier, would have involved the abutment and/or the pier only.

The decision as whether or not to stage this particular project is, to a certain extent, a matter of engineering judgment. It is our opinion that, although desirable, stage construction is not absolutely necessary. The advantages derived from such a procedure are outweighed by the difficulties connected with its

cont'd. /3 ...

February 7, 1967

implementation, and we would therefore recommend that the above project be carried out as one contract, and that the outlined sequence of work be maintained.

Altman

AGS/ndf

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. W. Wigle
S. McCombie
P. De Visser

Foundations Files ✓
Gen. Files

FEB. 2, 1967.

BLACK RIVER DIVERSION - SEC HWY 614 - MANITOUBIDGE
W.P. 114-64 ; DISTRICT 18.

INQUIRY BY MR. GEORGE TILLY (GIFFELE & ASSOC)

Q: IS 1 FT OF SAND CUSHION ENOUGH BETWEEN THE ORIG.
SUBSOIL AND THE RIP-RAP WITH MIN. SIZE 100 FT.²

A: NO. THERE SHOULD BE AT LEAST 1 FT LAYER OF
DIRTY GRAVEL ADDED ON TOP OF THE SAND CUSHION.

Q: CAN THE PILE DRIVING BE INCLUDED IN THE FIRST (GRADING)
CONTRACT?

A: YES. PILES SHOULD ALSO BE SURVEYED (AFTER DRIVING)
AND CHECKED AGAIN PRIOR TO FOOTING CONSTRUCTION.
IT IS ESSENTIAL TO PROVIDE A RELIABLE BENCH-MARK.

Q: SHOULD THE RIPRAPING BE INCLUDED IN THE FIRST CONTRACT?

A: IF THE DIVERSION CHANNEL IS IN USE THE RIP RAP SHOULD
INDEED BE IN PLACE.

Q: WHY NOT USE H-PILES?

A: THERE IS NO REAL OBJECTION TO THE USE OF H PILES. HOWEVER,
IT IS FELT THAT SOME ADDED COMPACTION OF THE UPPER LAYERS
WILL BE ACHIEVED WITH TUBE PILES. ALSO THE POSSIBILITY OF
ARTESIAN WATER COMING ALONG THE PILE IS LESS WITH
TUBES. A D-12 HAMMER (22,500 FT-LB) SHOULD BE USED
FOR DRIVING.

A.G.S.

JAN. 9. 1966.

BLACK RIVER DIVERSION - SEC. HWY. 614 - MANITOUWADGE
W.P. 114-64 ; DISTRICT 18.

INQUIRY BY GEORGE TILLY (GIFFELS & ASSOC.)

Q.: IN REPORT DRIVING OF PILES PRIOR TO EMBANKMENT
CONSTRUCTION IS RECOMMENDED. HOWEVER, IN THE REVIEW
STAGE CONSTRUCTION IS ASKED FOR. STAGING OF THE
GRADING CONTRACT AHEAD OF THE BRIDGE CONSTRUCTION
MAKES THE DRIVING OF PILES AS THE FIRST OPERATION
IMPOSSIBLE. WHAT SHOULD BE DONE?

A.: According to the Preliminary Bridge
Drawing N° D-6089-P1 the canal has to
be excavated prior to bridge construction
because of the location of the piers. The
east bank would also have to be built
up in order to be able to drive the piles
for the abutment. If staging is accepted
fill can be also built and when bridge
construction is to start only a very
small amount of the approaches would
have to be removed. The part that would
be removed will be replaced by the
abutments proper and the granular
fill behind them.

Also

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1966 NOV 7 AM 10:09

DOWN PTMR 3 NOV 7/66 955A

A STERNAC PRIN FOUNDATIONS ENGR

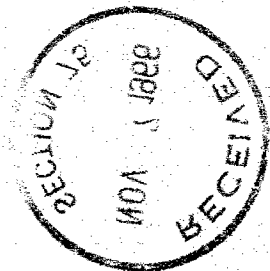
RE: WP 114-64 BLACK RIVER DIVERSION HIGHWAY 514

YOU RECENTLY DISTRIBUTED A FOUNDATION REPORT BY GEOPON LTD. FOR THE
ABOVE MENTIONED PROJECT. WOULD YOU PLEASE SEND A COPY OF THIS REPORT
TO GIFFELS ASSOCIATES LTD, 60 ADELAIDE STREET EAST, TORONTO 1, TO
THE ATTENTION OF G R TILLEY

F NORMAN REG MAT AND TEST

JO

*copy of report
sent to Giffels
Nov. 10/66*



Mr. E. E. Davis,
Bridge Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. E. MacCombie

October 26, 1966

OCT 27 1966

FOUNDATION INVESTIGATION REPORT FOR D.H.O.
BY: Gecon, Limited, Consulting Engineers -
Black River Diversion, Sec. Hwy. 614, -
Manitowadge, Ontario. Dist. 13 (Sault Ste. Marie)
-- S.P. 114-64 - Site 482-37 --

Attached, please find the above mentioned report prepared and submitted by the consultant, Gecon Limited.

We have reviewed the report and studied the various recommendations. In connection with certain proposals, we would like to make the following comments:

(a) Fill Across the Black River:

The material excavated for the diversion should and will be disposed of into the present Black River bed. Thus, the present river channel will be filled and no stability problem should be encountered with the new highway fill, either at the crossing site or in the stretch running parallel to the present river bed. This filling operation naturally also eliminates the necessity of making the road embankment watertight by placing a clay blanket on the upstream side.

(b) Diversion Channel:

From the ground contours and the presented cross sections, it appears that the average river bank slopes are 2:1. It is our opinion that the banks of the diversion channel should be built with the same slope and protected with a granular blanket and rip-rap only at the bridge site. The protection should extend to a minimum of 3 feet above the highest flood level.

Our suggestion to dispense with the expensive blanketing along the entire diversion channel is based on the performance of the present existing river banks, and also on the concept that no disaster should result from local sloughing, or even failure along the new river course. We would rather suggest that vegetation on the slopes be encouraged.

Mr. B. A. Davis,
Bridge Engineer,
Bridge Division.
Attn: Mr. B. McCombie

- 2 -

October 26, 1966

(c) New Bridge Across the Black River Diversion:

Thirty-foot berms are recommended in the report based on the rapid draw-down effective stress analysis. This is the most severe assumption resulting in the lowest factor of safety. It is our opinion that such a case does not fully apply to this crossing because of the following reasoning:

The material governing the stability of the approach embankment and the river banks is a layer of silt and fine sand. In places where, instead of this layer, there is sand and decayed wood, this material will be subexcavated and replaced with granular fill. Consequently, it will be either silt with fine sand, or just granular material (sand and/or gravel). Both of these materials can be considered as quite permeable. It is our contention, that in case of flooding and subsequent water level drop, these materials will readily drain, and there will be but a very small lag - i.e., difference in the ground water and river water elevations. It is our understanding that it takes less time for a flood to reach its peak than for the flood waters to recede. Consequently, there is more time for the banks to drain than there is for them to get saturated.

In case that the materials have a relatively low permeability, there shouldn't be enough time during peak flood conditions for the materials to become fully saturated, thus making the case of a rapid draw-down possible. Therefore, it would be our recommendation that the river banks be built with 3:1 slopes, properly protected against sloughing and scour, and the approach fills with 2:1 slopes. Staging of the grading and bridge construction contracts for at least six months, is highly recommended.

Regarding the bridge foundation, we would agree with the recommendation to use end-bearing steel tube piles driven to refusal within either the compact to very dense silt stratum or the underlying sandy gravelly till. We would not recommend the use of jetting because it would not, in our opinion, be essential to reach the till stratum should refusal to driving be met at a higher elevation. The fact that artesian conditions were encountered in B.B. No. 9, some 109 feet below ground level, makes the use of jetting prohibitive.

cont'd. /3 ...

Mr. B. E. Davis,
Bridge Engineer,
Bridge Division.
Attn: Mr. E. McCombie

- 3 -

October 26, 1966

We believe that the information contained in the report and the above comments will be sufficient for you to continue with your design work. Should you wish to discuss any aspects dealt with in this report or memo, please feel free to contact this Office.

AGS/ndef
Attach.

cc: Messrs. B. A. Davis (2)
R. A. Tregaskes
D. W. Farren
H. W. Murrell
J. A. Knowles
F. DeVisser
E. R. Saint
A. Watt

Altman
A. C. Sternac,
PRINCIPAL FOUNDATION ENGINEER

Foundations Office
Gen. Files

May, 401 & Keele St.
Downtown, Ontario.

August 15, 1966

Materials and Testing Division

Geecon, Limited,
14 Main Road,
Aurora, Ontario.

Attention: Mr. D. Gates, P. Eng.

Re: Authorization for Foundation Investigation -
A.P. 114-64 - Site 433-37, Black River Diversion,
19.5 Miles South of Hanitouwidge, Sec. 34, Twp. 614,
District No. 18 (Sault Ste. Marie).

Dear Sir:

This is to authorize you to carry out the foundation investigation at the above mentioned site.

The drawing showing the proposed footing location has been handed to your representative on August 15, 1966.

As discussed with your Mr. H. A. J. Ketch, you are requested to hire the drilling equipment from Canadian Longyear (Port William) who are carrying out an assignment for the Department south of Hornepayne, on Sec. 34, Twp. 631. You should be in touch with them and start your investigation upon their completion of the mentioned job.

We are advised that you can expect to encounter sandy subsoil conditions. It appears that drilling for the west pier will have to be carried out from a raft.

We would appreciate it if your representative at the site would contact Mr. F. De Visser, Regional Bridge Location Engineer, Port William - Telephone No. 345-2148 (Area Code 807) before the start of the drilling operations and be in touch with him before leaving the site. The details of the necessary communications should be arranged in due course, between your representative and Mr. De Visser.

We understand that accommodations are available in the town of Hanitouwidge.

cont'd. /2 ...

Cocoon, Limited,
Attn: Mr. D. Bates

- 2 -

August 15, 1966

In accordance with our terms of reference, you are to have a qualified soils engineer in charge of the field work at all times. Any deviation from this arrangement has to meet our prior approval.

You are requested to submit eleven (11) copies of the report to the Department as soon as possible. Previous arrangements as to preliminary borehole information and laboratory testing program, should be followed.

Since the drawing accompanying the foundation report, showing the location of borings, the inferred subsoil conditions, etc., is to become a contract drawing, you are requested to prepare it in accordance with the U.S.C. Standards. To enable you to do this, we are supplying you with a sample drawing with all the necessary explanations, together with linen sheet for your drawing. You are also requested to provide us with a Crossfield copy of the drawing.

Charges for the work will be in accordance with your Schedule of rates, dated July 8, 1966, and invoices to be addressed to the attention of the undersigned.

We are attaching Purchase Order J 34825, covering the purchase of any new material required for this work, in order that you may use this as a basis for exemption from the Federal Tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

AK

AGB/deep
Attn.

A. Kutta,
NATURAL & TESTING ENGINEER

cc: Messrs. S. McCombie
d. W. Murrell
J. A. Knowles
F. De Visser
E. A. Saint
Mrs. I. Steinberg
H. Rodings
A. Crowley
H. Brydowski (2)
Foundation Office
Gen. Files (2)

GEOCON LTD

HEAD OFFICE

420 MICHEL JASMIN, DORVAL, QUEBEC
TELEPHONE 631-9827

Rexdale, Ontario,
October 31, 1966.

DISTRICT OFFICES

14 HALLS ROAD
REXDALE, TORONTO, ONT.
TEL. 241-6476

295 EAST 11TH AVENUE
VANCOUVER 10, B.C.
TEL. 679-2620

Department of Highways, Ontario,
Materials and Testing Division,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Report No. T7923

Dear Sirs:

This will advise that there was an error on page 22 of the above report in that the reference to the cut slope in Conclusion number 3 should have read "1 vertical to 2.5 horizontal".

We are taking the liberty of enclosing herewith copies of the corrected page and would be grateful if you would have them inserted in your copies of the report. We would like to apologize for this inconvenience.

Yours very truly,

GEOCON LTD



D. B. Oates, P. Eng.,
District Engineer.

DBO:bm
T7923

*New Pages
distributed
Nov. 2*

CONTRACT CONTROL MEETING OCT. 20. 1967

It was decided that the only items to be scheduled would be

1. Excavation of the river diversion
2. Building of the approach embankments

Beyond this it will be the contractors choice how to proceed. Whether he chooses to drive the piles for the abutment first or the piers will be his decision.

The excavation for the pier footings is relatively shallow and will have to be shored because of the presence of water.

It is therefore felt that this operation will not endanger the stability of the bents.

It is also believed that the driving of abutment piles will have no detrimental effect on the stability either.

ALG.

Oct. 26. 1967.

Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division,
Admin. Bldg.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

December 8, 1967

Black River Bridge -
W.P. 114-64; Site 48#-37
Hwy. 614 - District No. 18

With reference to your memo of December 5, 1967, regarding the above structure, we wish to advise you that we have no comments. It would appear that all recommendations regarding foundations are incorporated on the final plans.

The construction sequence is given in Special Provisions, according to the note on Drawing D-6089-1.

AGS/MdeF

A. G. Sternac
A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. S. McCombie
F. De Visser

Foundations Files ✓
Gen. Files

MEMORANDUM

To: Mr. A.G. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Building

From: Bridge Division,
Downsview, Ontario

Date: December 5, 1967


Our File Ref.

In Reply To

Subject: Black River Bridge
W.P. 114-64, Site 48E-37
Highway 614, District 18

As requested during our discussion some time ago, enclosed is a final set of plans for the above structure. The limits of the First Stage of Fill have been revised as discussed, and this fill will be placed after the stream diversion has been completed and before the substructure piles are driven. A construction note calling for the excavation of pier footings to be carried out within adequately braced interlocking sheeting has also been added.

Would you please confirm if these changes are satisfactory.



K.G. Bassi,

Regional Bridge Project Engineer

KGB:rd

Encl.

c.c. C.S. Grebski
G.R. Tilly, Giffels Associates

1959 APR 2 PM 1:38

W.P. 114-64
4/2/66

MX SAUL APR 2/69 12:55

00202

FTWR I H W HURRELL REG ROAD DESIGN SUPVR

C.C. G M JORDAN ENG AUDIT SUPVR

C.C. F NORMAN REG MATERIALS ENGR

00203

ATT: D GUNTER

DOWN 3 X 6 SELBY FOUNDATION SECTION

00204

C.C. C CRIPPS BRIDGE CONST LIAISON ENGR

RE: CONTRACT 68-20 SLACK RIVER BRIDGE AND APPROACHES

00205

HWY 614

THIS IS TO CONFIRM THE FOLLOWING:

00207

- (1) THE CONTRACTOR, GEORGE ARMSTRONG, WILL BE AVAILABLE FOR A MEETING AT 10:30AM, APRIL 9/69 AT THE CONTRACT SITE.
- (2) ARRANGEMENTS HAVE BEEN MADE FOR THE CONTRACTOR, CONSULTANT AND ALL DEPARTMENT REPRESENTATIVES TO ATTEND THIS MEETING.
- (3) ARRANGEMENTS HAVE BEEN MADE FOR TRANSPORTATION FOR HURRELL, TILLEY, CRIPPS AND SELBY FROM THE SGO AIRPORT AND THE DISTRICT OFFICE IN THE AFTERNOON OF APRIL 8/69.
- (4) ARRANGEMENTS HAVE BEEN MADE FOR RESERVATIONS FOR SINGLE ROOMS FOR THE MEN MENTIONED IN (3) ABOVE, FOR THE NIGHT OF APR 8/69 AT THE MEL IRE MOTEL IN WAWA.
- (5) WE WILL MAKE ARRANGEMENTS DURING THE DAY OF APRIL 9, FOR ROOMS FOR THOSE REQUIRING THEM FOR THAT EVENING.
- (6) THE DISTRICT WILL PROVIDE TRANSPORTATION BACK TO THE SGO AND TO THE AIRPORT AS REQUIRED ON APRIL 9 AND 10, AS OUR PLANS DEVELOP.
- (7) WOULD YOU PLEASE ARRANGE FOR THIS NOTIFICATION TO MR TILLEY OF GRIFFELS ASSOCIATES LIMITED.

H G POTTS CONST ENGR

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MEMORANDUM

To: Mr. G. A. Wrong,
Principal Soils Engineer,
Downsview.

From: Mr. R. D. Gunter,
Project Soils Supervisor,
Port Arthur.

Date: October 23, 1967.

Our File Ref.

IN REPLY TO

SUBJECT:

W.P. 110-64 - 18.5 Miles South of Manitouwadge
Southerly 1.4 Miles, Highway 614

At your request, the following comments are made with regard to the proposed Black River diversion, included with W. P. 110-64.

Mr. F. DeVisser, Regional Bridge Location Engineer, has calculated the single structure plus the proposed river diversion will save approximately 90,000 dollars when compared to the cost of building structures over both the Black River and Mobart Creek. This saving, however, depends on the estimated earth excavation price on the diversion not exceeding one dollar per cubic yard and that future maintenance problems will not be costly.

As stated by Mr. A. Stermac, Principal Foundation Engineer, in his covering letter on Geocon Ltd's. foundation report, the borings and present bank slopes indicate that a $2\frac{1}{2}:1$ slope with no bank protection should be designed for the river diversion. It is my opinion that in order to obtain a stable slope without protection, something in the order of a $4:1$ slope will have to be obtained during construction. This opinion is based partially upon construction experience with similar materials and partially on the information obtained from Boreholes #10 and #11. These borings indicate a high ground water elevation (9030±) and a layered silt and clay material from an estimated elevation of 897.0 to 852.0. This material has a low disturbed shear strength even when both the silt and clay layers are considered. Thus the contractor will have to remove a material which will probably go into a quick condition when disturbed during excavating operations. With these conditions I cannot see how slopes of $2\frac{1}{2}:1$ will remain stable even during construction. A similar type of material encountered at the Big Pic River had to be excavated at $4:1$ and gravel sheeted. It is also my opinion that the contractor, after the initial 5 to 10 feet of the excavation has been removed will have to operate from this area with a dragline and even then with difficulty at the lower elevations of the diversion.

The majority of the material will have to be moved to the north if it is to be utilized as fill material in the old river channel. This operation must also include the stipulation that stockpiling of

excavated material within an estimated 100 feet of the top of the excavated slope cannot be tolerated as there is a possibility of massive circular arc failures due to the surcharge and lack of lateral support. The excavated material from the Mobert Creek portion of the diversion will have to be wasted as the proposed disposal area in the Black River channel is inaccessible from the south bank of the creek.

Thus a difficult and costly stream diversion will be encountered at this site. Cramped working conditions, double handling of materials, saturated materials, heavy slope sloughing with the possibility of circular arc failures are the main engineering problems that will have to be overcome.

The Regional Contract Review meeting solved none of these problems and the only conclusion drawn was that Head Office would deal with the construction sequence and that it was the contractor's obligation to evaluate all soils data available, including the foundation report, and bid accordingly.

My conclusions and recommendations are as follows:

- (1) A re-evaluation of the cost comparison between the present scheme of one structure plus a stream diversion and the two structure scheme should be made after the contract bids are submitted.
- (2) If the river diversion is to be built, strict control of the operations must be maintained by the Department of Highways.
- (3) A sequence of operations, including the seasonal scheduling, must be set up and followed. Seasonal scheduling is a necessity due to the high rapid spring runoff characteristics of the Black River.



R. D. GUNTER,
Project Soils Supervisor,

For:

RDG/bc
C.C.
Mr. A. Stermac

F. NORMAN,
REGIONAL MATERIALS ENGINEER.

MEMORANDUM

To: Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

From: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Date: November 6, 1967

Our File Ref

IN REPLY TO

Subject: W.P. 110-64 - 18.5 Miles South of Manitowadge
Southerly 1.4 Miles, Highway 614

With reference to the memorandum of October 23, 1967, from Mr. E. D. Gunter, Project Soils Supervisor to you, we wish to make the following comments:

Experience with either local materials or local construction procedures is certainly very valuable and its importance cannot be denied. Well documented case histories certainly provide by far the best parameters for future jobs where comparable conditions prevail.

Because of the above stated, we have been and always are anxious to hear any comments from persons or organizations who have been or still are active in engineering in certain areas.

Regarding the above mentioned river crossing, Mr. Gunter has, in his memorandum, disagreed with certain recommendations that were contained in our covering letter of October 26, 1966. Before going into the details of the disagreement, we would like to express our regret that Mr. Gunter's comments have been made one year after our letter was written. They were made after the Regional and Head Office contract review meetings had taken place. In this respect, we would question the purpose and usefulness of such comments.

In Mr. Gunter's opinion, the new river bank slopes will not be stable as $2\frac{1}{2}:1$, but will, rather, have to be in the order of $4:1$. Our recommendation for $2\frac{1}{2}:1$ slopes was based on two basic reasons:

(1) The slopes of the present river banks are standing up as $2:1$ or steeper. True, they show evidence of localized sloughing or small failures, but on the overall, are stable and are performing satisfactorily.

(2) The minimum factor of safety for $2\frac{1}{2}:1$ slopes computed by the consultant, and using the minimum soil parameters, was 1.3. Only surface sloughing was mentioned as a possible occurrence.

cont'd. /2 ...

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

2.

November 6, 1967

The foregoing reasons do appear to us, logical and sensible. Our further reasoning has been not to bother if some sloughing does occur in the future and if, in due course, the slopes become flatter. We have recommended that vegetation on the slopes be encouraged in order to minimize sloughing and erosion. In view of the satisfactory performance of the present natural river banks, we felt that the expenditure for providing a protective drainage blanket along the entire length of the new slopes would be unwarranted.

Regarding construction and Mr. Gunter's comment (based mainly on results of B.H.'s #10 and #11) that the soil when being excavated, may turn into liquid, we would like to comment as follows:

It would appear that most of the diversion channel excavation will be within the sand or silt layer (bottom of channel elev. 880.0). Whatever portion of the channel excavation is to be within the varved clay, it will be in a material that has an undrained shear strength of 1,000 p.s.f. or more and, therefore, has a stiff consistency.

Test results indicate the varved clay to be very sensitive. In spite of the high sensitivity, however, we feel that due to the stiff consistency, no major difficulties should be encountered during excavation, provided normal dragline or backhoe excavation procedures are used.

It should be borne in mind that the main body of the varved clay, which in certain cases may be the main controlling factor for the stability of the banks, will not be disturbed during the excavation. Only the material that is handled during the excavation operation will be partly disturbed - not the material which remains in its original position, with the possible exception of one or two inches at the surface.

Regarding Mr. Gunter's mention of the high water table (~903.0⁺), we feel that by applying the appropriate construction procedure, problems arising from such a condition can be overcome.

Under no circumstances are we trying to present this job as a simple or very easy one. However, we feel that enough factual data was assembled that enabled a reasonably thorough investigation and analysis of the whole problem. Some of the original recommendations have been modified in view of what appears to us as sound, valid reasons. Unless some more concrete arguments to the contrary are presented, we would suggest that these recommendations remain unchanged.

cont'd. /3 ...

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

3.

November 6, 1967

In closing, we would like to emphasize that the wide distribution of the foundation reports is intended to keep informed, as many persons as practical, and to provide them with the opportunity to voice their opinion on all or any part or aspect of the report. It is our firm belief that constructive criticism and discussions are indeed necessary, and should be encouraged because they are challenging and stimulating. The ultimate outcome is bound to be a better, a more thorough, and a more competent job.

ACS/MdeP

A. G. Sternac
A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

cc: Mr. R. D. Gunter

Foundations Files
Gen. Files

GEOCON LTD

HEAD OFFICE

Rexdale, Ontario,

October 24, 1966.

420 MICHEL JASMIN, DORVAL, QUEBEC

TELEPHONE 631-9827

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

295 EAST 11TH AVENUE
VANCOUVER 10, B.C.
TEL. 679-2680

Department of Highways, Ontario,
Materials and Testing Division,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

W.P. 114-64-

Re: Soil Conditions and Foundations,
Proposed Black River Bridge and Diversion,
Manitouwadge, Ontario.

Dear Sirs:

This letter accompanies our detailed report on the
above investigation.

We find that the site is underlain by a stratum of loose
silts and sands which overlies in turn, layered silt and clay, silt and
till strata. The encountered ground water level was usually close
to the ground surface but artesian pressure was encountered within
the basal till stratum. The actual soil and ground water conditions en-
countered are given in detail in the report.

The stability of the proposed Diversion Channel and
Embankment slopes have been considered in detail herein, on a location
by location basis, and from a soil mechanics standpoint only. The final
slope configurations will, however, depend on a number of factors such
as considerations of hydraulics, the form of surface protection measures

utilised, and economics. The relevance of such factors is discussed in the report.

It is considered that the most suitable type of foundation solution would be the use of friction or end bearing piles, as discussed. Because of the sensitive nature of the loose silt and sand stratum, recommendations are given regarding order of construction of the structure foundations and earthworks involved.

We believe that this report covers the information required from this investigation at this time. Since however, the evolution of final design of the structure foundations and earthworks for this project will involve concurrent consideration of soil mechanics together with hydraulics and structural design, the interpretation given herein is necessarily general. We would appreciate the opportunity to maintain liasson with you and provide more specific interpretive comments where these are required in the course of final design. Should you have any questions in the interval please give us a call.

Yours very truly,

GEOCON LTD



M. A. J. Matich, P. Eng.,
President.

MAJM:bm

T7923
REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND FOUNDATIONS
PROPOSED BLACK RIVER BRIDGE AND DIVERSION
MANITOUWADGE **ONTARIO**

Distribution:

11 copies - Department of Highways, Ontario,
Downsview.
3 copies - Geocon Ltd

GEOCON

INDEX

INTRODUCTION	Page 1
SUMMARIZED SOIL CONDITIONS	1
DISCUSSION	
General	2
Diversion Channel	3
Embankments	8
Foundations	18
Construction	20
CONCLUSIONS AND RECOMMENDATIONS	22
PERSONNEL	24

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL CONDITIONS

Sawdust

Topsoil

Very Loose to Loose Brown and Grey Sand

Very Loose to Loose Grey Silt

Firm to Very Stiff Grey Layered Silt and Clay
(Varved Clay)

Compact Grey Silt

Compact to Dense Grey Sand and Gravel Fill

WATER CONDITIONS

GEOCON

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter and by Purchase Order No. J 34825 both dated August 15th, 1966 to carry out a soil investigation for the proposed Bridge structure and Diversion scheme involved where Secondary Highway 614 crosses the Moberg Creek and Black River. The above crossing is located about 20 miles south of Manitowadge, Ontario.

The purpose of this investigation was to determine and interpret the soil and ground water conditions as they affect the design and construction of the proposed Bridge structure and Diversion scheme.

SUMMARIZED SOIL CONDITIONS

With the exception of the river bed the site is generally overlain by a layer of black topsoil about 6 inches thick and by about 5 feet of sawdust and wood fill on the east side of Moberg Creek. Underlying the topsoil, wood fill and the river bed is a deposit of generally loose silts and sands up to 27 feet thick containing organic and decayed wood traces in some locations. The sands and silts are underlain by a stratum of generally firm to stiff grey layered silt and clay up to 63 feet thick which in turn is generally underlain by compact grey silt. On the east side of Moberg Creek the silt stratum

GEOCON

was found to be underlain by a stratum of compact to dense grey sand and gravel till at a depth of about 100 feet below ground level.

At the time of investigation river level was at elevation 887.7. An artesian pressure was observed in the till in borehole 8 equivalent to elevation 906.

DISCUSSION

General

It is understood that it is proposed to cross adjacent water courses, Mobert Creek and Black River, with Secondary Highway 614. Mobert Creek will be crossed with a Bridge structure and Black River will be crossed with an Embankment. In this respect the flow of Black River will be diverted to Mobert Creek before it reaches the proposed crossing. The locations of the proposed Bridge and the proposed Diversion are shown on Drawings T7923-1 and 2 respectively, located at the rear of this report.

Proposed profile grade elevation of Secondary Highway 614 at the crossing is about 921. With grade as proposed Approach Embankments to the Bridge structure will be as high as 20 feet and the Embankment crossing the Black River will be built about 45 feet above river bottom. As presently proposed the bridge structure

General (continued)

consists of 3 spans with a mid span of 80 feet and end spans of 60 feet. The proposed Diversion will involve a cut about 500 feet long and up to about 30 feet deep since the elevation of the bottom of the Diversion channel will be about 880.

The discussion which follows, deals with the soil mechanics aspects of foundations and earth works involved in the proposed construction.

Diversion Channel

The proposed alignment of the Diversion channel is such that in order to divert Black River into Mobert Creek upstream of the proposed Bridge a considerable amount of excavation will be involved, necessitating cut slopes up to about 30 to 35 feet high above channel bottom.

The results of the two boreholes put down along the alignment of the channel indicate that typical soil conditions consist of about 12 feet of loose sand and silt overlying layered silt and clay. In the stability computations carried out, the soil properties given in Appendix I have been used with the exception that an effective cohesion value, c' , of 300 pounds per square foot has been used

Diversion Channel (continued)

instead of the value of 800 pounds per square foot obtained in one of the laboratory tests. Based on available precedent with varved clays, it was considered that the value of c' of 300 pounds per square foot was the maximum value that could be used in conjunction with an effective angle of internal friction of 25 degrees, as given below. In addition, the slopes in the layered clay and silt have been analysed for the "all clay" and "all silt" conditions using respective parameters, ϕ' and c' , of 25 degrees and 300 pounds per square foot, and 35 degrees and zero.

Computations have been carried out to investigate the end of construction and long term conditions of stability. For long term stability analyses, a normal river level has been assumed in conjunction with a stabilised ground water level in the cut slope, with a maximum elevation of 900. A more severe situation is obtained for the case of rapid drawdown from the reported flood level of 913 in conjunction with horizontal seepage flow. This is discussed further, later.

Since the majority of the Diversion cut will be located in land away from the proposed structures, the required slope for a minimum computed factor of safety of 1.3 under normal conditions of

Diversion Channel (continued)

ground water level has been determined. The minimum permissible computed factor of safety in the area of the Bridge structure has been similarly taken as 1.5, but with more severe assumed conditions of seepage as discussed later. The net effect results in a more conservative cut slope design in the vicinity of the Bridge structure.

Stability computations have been carried out to determine the factor of safety of the cut slope using a side slope of 1 vertical to 2.5 horizontal with an overall maximum cut height of 33 feet. Using values of undrained shear strength within the layered silt and clay stratum increasing from 900 pounds per square foot at elevation 885, to 1330 pounds per square foot at elevation 860, stability computations give a minimum factor of safety of about 2.0 under conditions of total stress. Using parameters of $c' = 300$ pounds per square foot and $\phi' = 25$ degrees, computations give a minimum factor of safety of 1.6 under conditions of effective stress. However, using shear strength parameters of $c' = 0$ and $\phi' = 35$ degrees, a minimum factor of safety of 1.3 is computed under conditions of effective stress with the drawdown conditions given above. It is therefore concluded that Diversion channel cut slopes of 1 vertical to 2.5 horizontal are the steepest permissible slopes

Diversion Channel (continued)

from the standpoint of slope stability, other than shallow surficial sloughing. Experience indicates that, unless suitable protection is provided, surficial sloughing of cut slopes in varved clay will occur due to frost effects, wherever the slope is steeper than about 1 vertical to 3 horizontal.

Some retrogression of the 1 vertical to 2.5 horizontal cut slopes, in the form of shallow sloughing, would therefore occur until natural vegetation took hold on unprotected slopes, or unless a suitable granular cover was provided. In this connection, the protection applied to the cut slope surfaces to guard against shallow seated instability should be reviewed in conjunction with scour protection measures as discussed later.

Due to the layered nature of the stratum to be exposed along the cut-slopes for a majority of the Channel length, and the high incidence of thick silt laminae, general erosion or undercutting of channel banks by river action, and sloughing of the cut-slopes due to ground-water seepage from rapid flood drawdown is likely to occur. For the conditions of rapid drawdown from maximum reported flood level of 913, horizontal flow, and full saturation of the slope, an unprotected cut slope of 1 vertical to 4 horizontal would be required to maintain a

Diversion Channel (continued)

factor of safety of about 1.0 against sloughing. The existing banks show evidence of localized failures but are generally sloping at 1 vertical to 2 horizontal or steeper, due probably to the presence of vegetation.

From the standpoint of protection of slopes steeper than 1 vertical to 4 horizontal against sloughing due to rapid draw-down effects, suitable protection would also have to be applied to the slope surface.

In addition to effects of frost action and drawdown effects, unprotected cut slopes of 1 vertical to 2.5 horizontal in varved materials such as exist at this site, would be subject to erosion due to surface run off, although this would be minimized if a terraced rather than a continuous uniform slope were used. Such slopes as mentioned earlier, also are susceptible to scour due to river action particularly during flood stages. The extent to which scour would affect design of unprotected slopes would however have to be determined from a detailed study of the hydraulic aspects of the Diversion channel design, which are beyond the scope of this report. It appears evident however that if cut slopes of as steep as 1 vertical to 2.5 horizontal are used, surface protection will be

Diversion Channel (continued)

required for reasons of frost, drawdown, erosion from run-off, and river scour. It is recommended therefore that the final slope design be selected after the hydraulic study mentioned above, on the basis of the various pertinent considerations affecting slope stability. In this connection it might be mentioned that a surface protection of granular material, filter graded with respect to the natural soil and sufficiently coarse to prevent scour, could be designed to give simultaneous protection against all the factors mentioned. The final slope design is therefore dependent on the extent of the granular surface cover provided. Economic considerations will naturally also have a bearing on design.

Embankments

a) The surficial topsoil over the site and the sawdust located east of the proposed Bridge site are not considered suitable as a foundation material for either the embankments or the structures. It is recommended that initial stripping operations remove all vegetation, peat, topsoil, sawdust, pockets of alluvium and other unsuitable materials. This would involve excavation at certain locations to a depth of 5 feet or more. In addition, a certain amount of removal of

Embankments (continued)

organic material from below river level, either by excavation or displacement may be expected at the location where the approach embankment crosses the Black River.

It is recommended that the embankments be constructed of clean, granular material, suitably protected on the slopes where necessary by rip-rap to avoid erosion by wave action, through seepage, or water flow. The embankment above river level should be constructed in horizontal lifts, compacted to at least 90 percent Modified AASHO dry density with the top 4 feet of the embankment compacted to at least 95 percent Modified AASHO dry density.

With complete removal of surficial organic material as mentioned above, the overall height of fill above natural strata within the section covered by this report, and assuming a crest elevation of 921, would be a maximum of 45 feet at the crossing of Black River. Borehole 9, at the crossing of the Black River indicates about 6 feet of loose sand and silt overlying layered silt and clay. Stability computations have been carried out to determine the factor of safety of the embankment at this section using the measured undrained shear strength values in borehole 9 which increase from 1120 pounds per square foot at elevation 865 to 1400 pounds per square foot at elevation 850 for the

Embankments (continued)

layered silt and clay stratum. Assuming side slopes of 1 vertical to 2 horizontal for the embankment the computed factor of safety is about 1.3 under conditions of total stress. Stability computations give a factor of safety of about 1.4 under conditions of effective stress.

In examining the stability of the embankment under the conditions of both total and effective stress, it has been assumed that rapid drawdown from maximum reported flood level could occur through the embankment at some time in the future. This of course does not account for the possible improvement in the area ground-water regime which might result from the diversion works. Nor does it account for possible imbalance of head across the embankment which might result, depending on the method used to isolate Black River at its intersections with the Diversion Channel.

If it is likely that the embankment at the crossing of Black River is required to act as an earth dam, a possibility which depends on the design of the diversion channel, it would be desirable to provide a seal of impermeable material on the upstream slope of the embankment, together with suitable toe protection to prevent erosion by seepage through the embankment.

Embankments (continued)

Since a large quantity of silt and clay will be available from the Diversion Channel excavation, consideration might be given to using the isolated portion of the Black River as a spoil area. The clay and silt could then be used as an impermeable seal on the embankment and also improve the embankment stability by acting as a berm.

Because of the sensitive nature of the clay and silt, the material on end dumping would adopt very flat slopes, possibly of the order of 1 vertical to 8 horizontal. This would have to be considered in the selection and design of the structures used to form the banks of the diversion at the intersections with the Black River. At the time of writing, it is not known what form these structures might take, although the use of sheeting has been mentioned as a possibility. Sand and gravel retaining dykes could also be used to retain the silt and clay.

The consolidation test carried out on a representative sample of clay from the layered silt and clay stratum, and shown on Figure 8 of Appendix II, indicates that the deposit has been overconsolidated by a pressure of 4 tons per square foot in excess of existing overburden pressure. Allowing for the fact that the layered silt and clay stratum consists of approximately 60 percent thickness of silt and 40

Embankments (continued)

percent thickness of clay, it is computed that the centre of the embankment at the crossing of Black River will undergo a maximum settlement of about 6 inches.

b) Between the crossing of Black River and the proposed Bridge site the proposed embankment runs parallel to the Black River for a distance of about 200 feet. Due to the proximity of the river bank to the toe of the proposed embankment, stability computations have been carried out to determine the factor of safety against failure. Undrained shear strength values increasing from 860 pounds per square foot at elevation 870 to 1150 pounds per square foot at elevation 850 for the layered silt and clay stratum were used in computations since these represented the measured values at the Bridge structure. The minimum computed factor of safety under conditions of total stress was 1.2. Interpolating the soil conditions between boreholes 8 and 9 to the area where the embankment will be parallel to the Black River, the embankment fill will overlie about 30 feet of loose sands and silts, then layered silt and clay. Approximately 22 feet of sand and silt will be above river bottom elevation. For this soil stratigraphy and for rapid drawdown the factor of safety on effective stress is about 1.0.

In view of the low computed factor of safety for the end of construction condition, and also long term stability, some measures

Embankments (continued)

are necessary to improve the stability in this section of embankment, including the existing River bank. This could be done using available clay and silt from the Diversion excavation, or other suitable borrow as a berm within the Black River in the same manner as described above for the crossing of Black River. The most practical scheme depends also on factors involved in the Diversion Channel design, particularly at the downstream intersection with the Black River.

As discussed later, berms will be required to provide adequate stability of the approach embankments at the Bridge structure. A berm of scour-resistant granular material, rather than the spoil from the Diversion Channel, should be used for a sufficient distance along the adjacent north bank of the Black River to ensure that the berm is not removed by river action.

c) For approach embankments with a grade elevation of 921, the overall height of fill above the natural strata would be about 13 feet at the site of the east abutment to the Bridge, and about 21 feet at the site of the west abutment to the Bridge. Due to the location of the west pier of the main Bridge span being within the present bed of Moberg Creek, additional fill will be necessary to form the final west abutment slope by filling, as shown on Drawing T7923-3. This will be

Embankments (continued)

placed over the present western bank to the Creek, which has an existing slope of about 1 vertical to 2 horizontal. For the proposed Diversion Channel alignment, the east abutment slope will however be formed primarily by excavation within the loose silts and sands in contrast to the west slope where fill will be required.

The significant soil stratum at both abutment locations is loose silt with traces of sand, overlying the layered silt and clay which was encountered at both locations at elevations 876 to 878.

Stability computations have been carried out to determine the factors of safety of the eastern and western approach embankments in the longitudinal direction, i. e. at the Bridge abutments, under varying configurations of drawdown, angle of slope, and with and without toe berms. The most critical embankment conditions studied was found to be that of an embankment and cut slope of 1 vertical to 2 horizontal without a toe berm and under rapid drawdown. This gave a computed factor of safety of less than 1.0 for the east abutment slope under conditions of effective stress, for a slip circle confined within the loose silt stratum. Possible aggravating effects of pile driving, which might further reduce the factor of safety during construction, are discussed later.

Embankments (continued)

It was found that to provide a minimum computed factor of safety of 1.5 under rapid drawdown and full saturation of the loose silt stratum, it would be necessary to incorporate a berm at elevation 900 having a top width of 30 feet. The abutment slope configuration would involve a 1 vertical to 2 horizontal slope in the granular fill and 1 vertical to 2.5 slope in the silt. Suitable surface granular protection would be required for the slope in the silt for reasons already discussed.

Since a stable scour resistant fill will be necessary to form the west abutment slope, in contrast to excavation at the east abutment slope, it would be necessary to construct the berm at this location out of suitable sized and graded granular fill. Though it is not essential for slope stability purposes, that the berm at the toe of the east abutment be similarly constructed entirely out of suitable granular fill, it may be desirable from the point of view of scour protection to excavate the Diversion Channel wide at this location and form the berm of granular fill to the depth shown by the dotted line on the longitudinal section, Drawing T7923-3. The length of this granular berm would be approximately the same as the length of the berm on the opposite side of the River.

Embankments (continued)

Unless this is done, it will be necessary to protect the cut slopes in the silt using rip-rap and a suitable intermediate filter of granular material. It is recommended that the protective granular layer be at least 3 feet in thickness, subject to hydraulics requirements.

At the east abutment, existing ground contours and embankment height are such that it does not appear necessary to provide berms along the sides of the approach embankment. On the other hand, at the west abutment the berm at elevation 900 will have to be extended around the adjacent bank of the Black River, as already discussed. It will also have to be carried around the north side of this abutment to tie in with the existing elevation 900 ground contour and to form the transition section into the new cut slope. In effect therefore the berm under the bridge and to the north of it would also form the side of the Diversion Channel. Depending on the scheme used to form the side of the Diversion Channel to the south of the Bridge, the berm might be carried through to the C. P. R. Crossing. Alternatively, it could be following the outline of the north bank of the Black River, either maintaining a width of at least 30 feet at elevation 900 all the way to the embankment crossing of the Black River or merging at some intermediate point with other fill, such as the excavated silt and clay, as required for

Embankments (continued)

stability in this section of the Black River as mentioned earlier. The final configuration of the granular berm to the west abutment of the Bridge will therefore develop from concurrent consideration of a number of pertinent factors. We would be pleased to assist further in the application of the findings of this report to finalizing the details of the earthworks associated with the above.

The consolidation test carried out on a representative clay sample from the layered silt and clay stratum, and shown on Figure 8 of Appendix II, indicates that the deposit has been overconsolidated by a pressure of about 4 tons per square foot in excess of existing overburden pressure. It is computed therefore, allowing for a 40 percent thickness of clay, the remainder being silt, that the embankment at the location of the west abutment will undergo a maximum consolidation settlement of about 3.5 inches under the crest. Some additional settlement will occur due to consolidation of the loose silt stratum. For all practical purposes this will occur as load is applied during construction of the embankments.

Foundations

With regard to the possible use of spread foundations for the Bridge substructure, the significant soil stratum is the firm to stiff grey layered silt and clay; the overlying sand and silt stratum being loose and therefore unsuitable. The use of spread footings would necessitate a foundation level lower than elevation 877, this being the approximate surface elevation of the layered silt and clay stratum at this location. Such construction would also entail excavation beneath Diversion Channel water level. Based on the shear strength properties of the layered silt and clay stratum the allowable net increase in pressure for a spread footing solution would be approximately 0.75 tons per square foot. This value is probably too low for practical use, and with possible construction difficulties in dealing with the inflow of water into excavations, this type of foundation solution is believed to be impractical and will not be considered further in this report.

It would probably be most practical to found the proposed structure on piles, and a number of pile types, such as treated timber, steel tube or H piles, or precast concrete piles would be suitable for this purpose. One possibility would be the use of piles, such as steel tube or H piles, driven to the required resistance in end-bearing within the dense grey sand and gravel till. This would require piles in the order

Foundations (continued)

of 110 feet long. Some jetting may be required to facilitate penetration of displacement piles to the till, and to minimise drift of this type. In selecting the working load per pile, the effect of negative skin friction due to the consolidation of the silt and clay strata, under the weight of the approach embankments where applicable, should be taken into account using soil properties in the attached Appendix I.

The bridge structure might also be carried on friction piles. In such a case, the length of piles required would be shorter than those for use as end-bearing piles. Such piles could be founded almost entirely within the layered silt and clay stratum. For preliminary design of a friction pile foundation, design could be based on a side shear value of 1000 pounds per square foot and a unit resistance in end-bearing of 9 times this value. On this basis and incorporating a factor of safety of 3.0, the computed working load for a 12 inch diameter pile, 40 feet long is approximately 20 tons. Due to the presence of artesian pressure within the till stratum, and undoubtedly in the silt immediately overlying the till, it is considered important that any type of friction pile be terminated within the layered clay and silt without penetration into the silt. As a margin of safety it is recommended that for preliminary design of friction piles, at least 15 feet of layered silt and clay be maintained between

Foundations (continued)

the pile tip elevation and the surface of the silt stratum which directly overlies the till. Irrespective of the pile type selected, it is recommended that the pile working load be verified by a program of pile driving and loading tests.

The use of a non-displacement steel H pile would have the advantage of minimising pore pressures in the layered silt and clay during pile driving operations. In addition, the loose silt and sand at this site is in the range of grain sizes which are susceptible to liquefaction from vibration effects such as those involved in pile driving operations. Previous experience at the Little Pic River in the same area of Ontario has demonstrated the effect of pile driving and resulting slides in silt of the same gradation characteristics as that at this site. In this regard, therefore, the sequence of construction is of particular importance, as discussed later.

Piles subject to frost action should be provided with at least 6 feet of earth cover for frost protection purposes.

Construction

As discussed earlier, to provide adequate stability at the Bridge structure it will be necessary to incorporate a 30 foot berm having a top elevation at 900. For the berm width and required slopes in the silts and sand the overall Bridge structure between abutments will increase by about 80 feet.

Construction (continued)

With centre line of the Diversion Channel as presently proposed the west pier will be located within the present river channel, whereas the east pier and the two abutments will be located greater than 50 feet back from the present river bank.

As mentioned earlier the grain size characteristics of the sands and silts and previous experience on the Little Pic River, indicates that slopes in the loose silt and sand stratum are susceptible to slides due to the pore pressures and vibrations caused by pile driving.

It is recommended therefore that pile driving operations for the pier and abutment foundations be carried out ahead of excavation for the Diversion Channel in the area of the bridge structure. If this is done, the possibility of failure of prepared slopes in the loose silt and sand strata would be avoided, and at three locations the topography would be relatively flat lying.

Construction of pier pile caps would require excavation below water level in the silt and sand, and layered clay and silt. Excavation should be carried out using close sheeting with de-watering by the procedure of pumping from filter equipped sumps maintained ahead of excavation level at least equal to the hydrostatic differential head expected.

Construction (continued)

Some disturbance of the base of excavations can be expected because of excess pressure in the silt layers within the layered silt and clay. Since a piled foundation is involved, this disturbance would probably not be of any consequence. To improve trafficability of the base of excavation during placement of the formwork for pile caps, it is suggested that a layer of sand and gravel be placed on the silt exposed at base of excavation level.

CONCLUSIONS AND RECOMMENDATIONS

1. With the exception of the area in the vicinity of the river bed the site is generally underlain by up to 27 feet of loose silts and sands which in turn overlie a stratum of firm to stiff grey layered silt and clay up to 63 feet thick. Beneath the layered silt and clay is a stratum of compact silt which in turn overlies dense till at a depth of about 100 feet i.e. elevation 800.
2. The groundwater level over the site was usually close to ground surface though evidence was obtained indicating the presence of artesian pressure within the basal till stratum.
3. It is considered that along the alignment of the proposed Diversion Channel, cut slopes of 1 vertical to 2.5 horizontal could be utilized, provided that a suitable covering of granular material is placed,

CONCLUSIONS AND RECOMMENDATIONS (continued)

as discussed in the report, to act as protection against erosion, frost, and seepage effects. Without such suitable protective measures, local repeated sloughing is likely to occur which would eventually establish stabilized slopes in the cut possibly as flat as 1 vertical to 4 horizontal.

4. West of the Bridge structure and at the location of the crossing of Black River where the embankment fill attains a maximum height of about 45 feet, side slopes of 1 vertical to 2 horizontal may be used for the embankment if constructed of clean sand and gravel. The need for stabilizing berms constructed of either granular material or excavated material for the Diversion Channel excavation is discussed in the report.
5. At the site of the Bridge structure, berms 30 feet wide should be incorporated into the cut slopes of the Diversion channel as discussed. Protective measures to safeguard against local sloughing due to erosion, seepage and frost action are essential at this location.
6. The most suitable foundation solution for the Bridge Piers and Abutments would appear to be the use of piles. A discussion is given relative to the use of end-bearing or friction piles.

CONCLUSIONS AND RECOMMENDATIONS (continued)

7. Due to the sensitive nature of the loose silts and sands overlying the layered silt and clay, consideration should be given to phasing construction procedures such that pile-driving is carried out prior to excavation of adjacent cut slopes, as discussed in the report.

PERSONNEL

The field work for this investigation was carried out by Mr. W. Burwash. The report was written by Mr. H. L. MacPhie, P. Eng., and Mr. G. M. Harris, checked by Mr. D. B. Oates, P. Eng., and reviewed by Mr. M. A. J. Matich, P. Eng.

D. B. Oates

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G. M. Harris per DBC

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G. M. Harris

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL CONDITIONS

WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

GEOCON

PROCEDURE

The field work for this investigation was carried out between August 30th and September 28th, 1966. A total of 7 boreholes with accompanying dynamic penetration tests and an additional 5 dynamic penetration tests were put down at the site. Boreholes 1, 2, 3 and 8 and dynamic penetration tests 4, 5, 6 and 7 were put down at the proposed foundation locations for the bridge structure crossing Mobert Creek. Borehole 9 was put down at the proposed crossing of Black River. Borehole 9 was put down in the Black River where it intersects proposed revision line "N". Boreholes 10 and 11 and dynamic penetration test 12 were put down in the area of the proposed Diversion of the Black River.

The boreholes and dynamic penetration tests were put down to depths ranging from about 60 to 110 feet using a standard skid-mounted machine drill rig. The use of a drum raft was required for borehole 9 put down in the Black River and for access across Mobert Creek. Sampling of the overburden was carried out using both a 2 inch O. D. split spoon sampler and 2 inch thin walled tubes. A number of in-situ field vane shear strength tests were also carried out in some of the boreholes.

A complete log of each borehole and dynamic penetration test is given in the Office Reports on Soil Exploration in this

Appendix. The locations of the boreholes and dynamic penetration tests together with the inferred soil stratigraphy are shown on Drawings T7923-1 and 2 located in the pocket at the rear of this report.

Laboratory testing of selected soil samples was carried out in the Toronto Soil Mechanics Laboratory of Geocon Ltd. The results of laboratory testing are shown on the Figures in Appendix II and on the Office Reports on Soil Exploration. All samples remaining after testing will be stored until November 1st, 1967 at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic datum. The bench mark used is located 150 feet right of station 675+18 of proposed revision line "N", Secondary Highway 614. The bench mark consists of a nail and washer in a 0.4 foot diameter spruce stump and has an elevation of 904.16.

SITE AND GEOLOGY

The site is located about 20 miles south of Manitouwadge, Ontario and north east of the existing junction of Secondary Highway 614 and the Black River.

In this area the Mobert Creek joins the Black River where both are contained by river banks generally about 10 feet above

water level at the time of investigation. The existing slopes of the river banks are as steep as 1 vertical to 1 horizontal and shallow slope failures are evident along the river banks. In some areas of the site adjacent to the crest of the river banks a recently deposited layer of silt about 2 inches thick was found to overlie topsoil.

River level at the time of investigation was at elevation 887.7. Information taken from Department of Highways, Ontario Drawing No. B-6000-90 dated April, 1966 indicates that high water level is at elevation 913. It is understood that further hydrological data is being collected to determine frequency and magnitude of flooding in this area.

Available geological information in the area indicates that the site lies between glacial lakes Borlow-Ojibway and Algonquin. However, in addition to the varved deposits associated with the larger glacial lakes such as those mentioned above, isolated deposits of varved sediments occur in depressions well removed from the larger basins. It is believed that the varved deposit at the site is of this nature.

SOIL CONDITIONS

The principal soil strata encountered in the boreholes are as follows:

Sawdust

A surficial layer of sawdust and wood fill was encountered in boreholes 1 and 2 put down on the east side of Moberg Creek in the area of an abandoned sawmill. The encountered thickness of the sawdust fill was 5 feet at the location of both boreholes 1 and 2.

Topsoil

Underlying the sawdust fill at the location of boreholes 1 and 2, a layer of topsoil 4 inches thick was encountered. A surficial layer of topsoil was also encountered in boreholes 10 and 11. The topsoil contained roots and decayed vegetation and was generally silty. Apart from the river bed the site is generally covered with topsoil. However, as pointed out earlier the topsoil was covered by about 2 inches of recently deposited silt in some areas adjacent to the crest of river banks.

Very Loose to Loose Brown and Grey Sand

A deposit of sand was encountered in boreholes 2, 3, 9, 10 and 11. In boreholes 2, 3 and 9 the sand was grey and generally contained organic traces. In borehole 3 evidence of decayed wood was encountered in the sand deposit. The grey sand deposit consisted generally of fine sand sizes with the exception of the sand at borehole 2 where 2 feet of coarse grey sand was encountered underlying silt.

Very Loose to Loose Brown and Grey Sand (continued)

At borehole 3 a 10 foot thick surficial layer of grey sand was encountered and at borehole 9, four inches of grey sand was encountered underlying river bottom.

In boreholes 10 and 11, the sand was brown in colour with observed thicknesses of about 9 and 5 feet, respectively. The sand generally consisted of fine to medium sand sizes, however, the bottom portion of the brown sand stratum at borehole 11 graded to a sandy silt.

One mechanical analysis was carried out on a representative sample of the brown sand encountered in borehole 10. The results of the mechanical analysis indicate that the sample tested contains 82 percent sand sizes and 18 percent silt sizes. The results of above test are shown on Figure 2 of Appendix II.

Four standard penetration resistances determined in the grey and brown sand deposit gave "N" values ranging from 1 to 5 blows per foot with an average of 3 blows per foot. Based on the above "N" values and the results of dynamic penetration tests, the relative density of the sand deposit is believed to range from very loose to loose, and is generally very loose.

Very Loose to Loose Grey Silt

Underlying the topsoil in boreholes 1 and 2, the sand in boreholes 9 and 10 and surficially at borehole 8 is a stratum of generally grey silt. The encountered thickness of this stratum ranged from about 5 to 32 feet and the greater thickness was encountered in the area of the proposed bridge structure. Generally, the top part of this stratum contained organic traces and the lower part close to the underlying layered clay and silt stratum contained some clay; however, no organic traces were encountered in boreholes 9 and 10. Some fine sand sizes occurred in this stratum as well as some traces of decayed wood. In boreholes 2 and 8 this stratum was brown to grey in colour.

A number of mechanical analysis tests were carried out on samples recovered from the silt stratum and the resulting grain size distribution curves are shown on Figures 3 and 4, in Appendix II. The results of the above tests indicate that the samples tested contained up to 60 percent sand sizes, from 32 to 88 percent silt sizes and from 6 to 30 percent clay sizes.

Standard penetration resistances determined in this stratum gave "N" values ranging from 2 to 7 blows per foot with an average of 4 blows per foot. Based on the results of the above

Very Loose to Loose Grey Silt (continued)

"N" values and the results of dynamic penetration tests, the relative density of this stratum ranges from very loose to loose and is believed to be generally loose.

For design purposes the silt has been assigned an effective angle of friction of 30 degrees and a wet unit of 115 pounds per cubic foot.

Firm to Very Stiff Grey Layered Silt
and Clay (Varved Clay)

Underlying the surficial strata of silts and sands described above is an extensive deposit of layered silt and clay. The encountered thickness of this stratum ranged from 29 to 63 feet and was generally from 50 to 60 feet thick. This stratum was fully penetrated in all boreholes with the exception of boreholes 8 and 9 which terminated in this stratum and the maximum encountered thickness of 63 feet was observed in borehole 9. This stratum consists essentially of alternate layers of grey silt and dark grey clay of varying thicknesses.

The layering of this stratum is irregular; however, there is a general trend for the silt layers to increase in thickness with depth and the clay layers to decrease in thickness with depth.

Firm to Very Stiff Grey Layered Silt
and Clay (Varved Clay) (continued)

The bottom portion of this stratum is therefore predominantly silt and the boundary between the layered silt and clay and the underlying silt is indistinct. The layering of the thin walled tube samples recovered from borehole 3 was examined in detail and a sketch of each sample showing relative layer thicknesses was drawn. The results of the visual examination of the above samples are shown pictorially on Figure 11 of Appendix II and the nature of the layering indicated on this Figure is generally representative of the layered silt and clay stratum as a whole. The observed thickness of the silt layers ranged from about 0.2 to 3.5 inches and the clay layers ranged from 0.2 to 2.5 inches. The clay constitutes approximately 40 percent and the silt 60 percent of the total stratum thickness.

A number of wet unit weights determined on undisturbed samples recovered from this stratum gave values ranging from 103 to 128 pounds per cubic foot with an average of 118 pounds per cubic foot. Corresponding natural water contents carried out on the bulk samples used for unit weight determination gave values ranging from 24.2 to 37.2 percent.

Natural water content determinations carried out on individual layers of this stratum indicated the natural water content

Firm to Very Stiff Grey Layered Silt
and Clay (Varved Clay) (continued)

of the clay layers to range from 37.4 to 74.9 percent and for the silt layers to range from 17.4 to 26.7 percent. Atterberg limit tests were carried out on the clay layers giving liquid limits ranging from 29 to 89 and generally greater than 50 and plastic limits ranging from 16 to 29. The corresponding range in natural water content is that given above for the clay layers. Atterberg limit tests were attempted on the silt layers, however, these tests could not be carried out successfully due to the lack of cohesion in the silt. The lowest liquid limit of 29 indicated above probably reflects some contamination by silt during testing.

A total of five mechanical analyses were carried out on material from individual layers of the layered silt and clay stratum. Three mechanical analyses carried out on samples from the clay layers indicated the samples tested to consist of 1 to 16 percent silt sizes and 84 to 99 percent clay sizes. Also two mechanical analyses carried out on samples from the silt layers indicated the samples tested to consist of 83 and 87 percent silt sizes and 17 and 13 percent clay sizes respectively. The results of the above mechanical analysis testing are shown on Figure 5 of Appendix II. One test was also carried out on a composite sample of silt and clay

**Firm to Very Stiff Grey Layered Silt
and Clay (Varved Clay) (continued)**

and the result plotted on Figure 5.

A number of unconfined compression tests carried out on undisturbed samples of the layered silt and clay stratum gave shear strengths ranging from 704 to 2500 pounds per square foot, with an overall average value of 1100 pounds per square foot.

Laboratory vane shear strengths were determined on tube samples from this stratum and values of shear strengths ranging from 490 to 1950 pounds per square foot were observed. Based on remoulded laboratory vane shear strengths the sensitivity of this stratum ranges from about 2.7 to 13.9 and is generally 7.8.

In-situ field vane shear strengths determined in this stratum gave shear strengths ranging from 800 to 2560 pounds per square foot. Based on remoulded field vane shear strengths the sensitivity of this stratum was found to range from 1.2 to 4.0 and be generally 2.3. The discrepancy between the lower sensitivities of the stratum as determined in the field vane test compared with the sensitivities as determined from the laboratory vane test is considered due to the effect of the silt layers on the field vane results.

Firm to Very Stiff Grey Layered Silt
and Clay (Varved Clay) (continued)

All the results of shear strength testing are shown as a composite plot of all shear strength values obtained versus elevation on Figure 1 of Appendix II.

A consolidated undrained triaxial test with pore pressure measurements was carried out on a sample taken from a silt layer using the technique of stage loading. The results of this test are plotted on Figure 9 of Appendix II. The effective stress parameters, ϕ' and c' , for the silt as determined from the above test are 36 degrees and 0 pounds per square foot, respectively. A similar test was carried out on a sample prepared from undisturbed clay taken from three separate clay layers and the results of this test are plotted on Figure 10 of Appendix II. The effective stress parameters, ϕ' and c' , as determined from the above test are 25 degrees and 800 pounds per square foot respectively.

One consolidation test was carried out on a sample selected from a relatively thick layer of dark grey clay and the resulting void ratio pressure curve is shown on Figure 8 of Appendix II. The results of this test indicate that the deposit is preconsolidated by about 4 tons per square foot in excess of existing overburden pressure. The compression index (C_c) and the rebound index (C_r) as determined from the above test were 0.90 and 0.075 respectively.

Firm to Very Stiff Grey Layered Silt
and Clay (Varved Clay) (continued)

A number of standard penetration resistances determined in this stratum gave "N" values ranging from 2 to 21 blows per foot with an average of about 8 blows per foot. It is believed that the higher "N" values reflect the predominance of silt layers.

Compact Grey Silt

Underlying the layered silt and clay stratum at the location of all boreholes with the exception of boreholes 8 and 9 is a stratum of grey silt. The encountered thickness of this stratum ranged from about 6 to 37 feet and this stratum was fully penetrated in boreholes 1 and 2.

A number of mechanical analyses carried out on samples from this stratum indicated the samples tested to consist of from 84 to 96 percent silt sizes and from 4 to 16 percent clay sizes. The results of the above mechanical analyses are shown on Figure 6 of Appendix II.

Standard penetration resistances determined in this stratum gave "N" values ranging from 14 to 29 blows per foot with an average of 22 blows per foot. Based on the above "N" values and the results of dynamic penetration tests the relative density of this stratum is found to be compact.

Compact to Dense Grey Sand and Gravel Fill

Underlying the silt stratum in boreholes 1 and 2 is a stratum of sand and gravel till. Both boreholes 1 and 2 terminated in this stratum where encountered thicknesses were about 2 and 7 feet respectively. It appears that the top of this stratum is contaminated with silt from the overlying silt stratum.

One mechanical analysis carried out on a sample from this stratum indicated the sample tested to consist of 48 percent gravel sizes, 22 percent sand sizes, 24 percent silt sizes and 6 percent clay sizes. The results of the above test are shown on Figure 7 of Appendix II.

Three standard penetration resistances determined in this stratum gave "N" values of 47, 16 and 36 indicating the relative density to range from compact to dense.

WATER CONDITIONS

Ground water levels were observed in the boreholes over the period of investigation. In boreholes 1 and 2 water levels were observed in the open uncased holes. Also water conditions were observed in the hole made by dynamic penetration test 4.

River level at the time of investigation was at elevation 887.7 that is, about 2 feet lower than ice level in January, 1966. The elevation of observed water level in the boreholes generally increased with increasing distance from Mobert Creek and was observed to be at elevation 903.1 in borehole 11 where the highest static water level was encountered.

Artesian conditions were encountered in borehole 8 where an artesian head of 8.3 feet above ground level or elevation 906 was observed when the borehole was at a depth of 109 feet. However, after the casing was removed the water level in borehole 8 was observed to be at a depth of 7.7 feet below ground level, that is, at elevation 889.9. During the field work for this borehole, measurements were taken in the casing within the layered deposit at depths of 35 and 82 feet on two occasions over-night. The water level was at about elevation 891. Artesian conditions were also encountered at the location of dynamic penetration test 4. Penetration test 4 was advanced to a depth of 98 feet below ground level and when the drill rods were removed, water carrying clay, silt and sand flowed out of the hole made by the penetration test. This penetration test was put down on September 15th, 1966 and at the end of this investigation on September 28th, 1966 water was still flowing at this elevation.

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

<u>Consistency</u>	<u>U-Strength Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration Resistance Blows/ft.</u>
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

OTHER TESTS

In this column are shown by symbol the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

GLOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT Y7923 SERIES 1 DATUM THEORETIC CASING NX 2 BK
 BORING DATE Aug 30 SEPT 2 REPORT DATE SEPT 21, 1946 COMPLETED BY AEI CHECKED BY AEI
 SAMPLER MARKED WT 142 LBS DEEP 30 FATHOMS (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4000 IN. LBS ENERGY)

SAMPLE CONDITION


 021-5232 2222
 021-5232 2222
 021-5232 2222

A-2 - ALLIANCE SAMPLE
 B-7 - CLAYTON'S FLAME
 W-4 - MAGNUSON SAMPLE
 C-4 - WHITE-IRON
 C-7 - WHITE-IRON VALVE
 C-8 - COLLIER SAMPLE

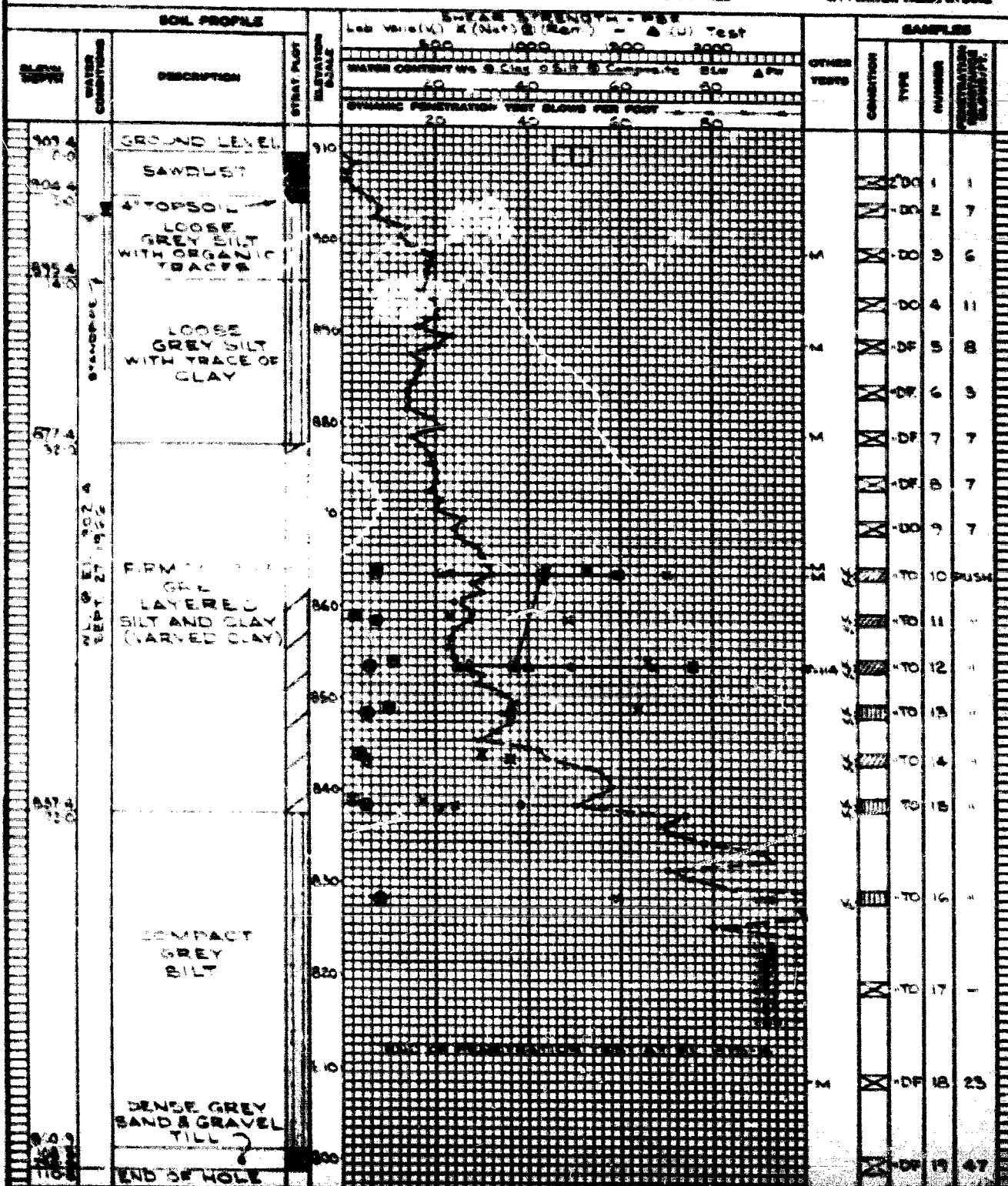
SAMPLE TYPES

4.2 - FILL SAMPLE
4.3 - SLEEVE OPEN
4.4 - SLEEVE-CUT VALVE
4.5 - PUMP HALL OPEN
4.6 - ROCK CORE

ABSTRACT

ABBREVIATIONS

U	IN-SITU WARE TEST	T	TEST UNIT WEIGHT
M	Mechanical Analysis	K	PERMEABILITY
CC	UNCONFINED COMPRESSION	C	CONSOLIDATION
CS	TRIAXIAL CONSOLIDATED UNDRAINED		
U	TRIAXIAL UNDRAINED	WL	WATER LEVEL IN CARRIER
S	TRIAXIAL SWAGED	WT	WATER TABLE IN GROUND



CONTRACT T7323 BORING # 2 DATUM GEODETIC CASING NK 1 BX
BORING DATE SEPT 5/1966 REPORT DATE SEPT 21, 1966 COMPILED BY AEI CHECKED BY J. H. H.
SAMPLER MANNER WT. 140 LBS. DEEP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4300 IN. LBS. ENERGY)

SAMPLE TYPES



**DISORDERLY
FAIR
GODS
LOST**

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHURN SAMPLE

F.S. - POIL SAMPLE
S.O. - SLEEVE OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

- Y • IN-SITU VACUUM TEST
- M • MECHANICAL ANALYSIS
- U • UNCONFINED COMPRESSION
- CS • TRIAXIAL CONSOLIDATED UN
- C • TRIAXIAL UNDRAINED
- S • TRIAXIAL DRAINAGE

ABBREVIATIONS

7. WET UNIT WEIGHT
8. PERMEABILITY
C. CONSOLIDATION
LABOR
9. WATER LEVEL IN CASING
WT. WATER TABLE IN SOIL

[illegible]

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7923 BORING 3 DATUM GEODETIC CASING NX & BX
 BORING DATE SEPT. 10-14/66 REPORT DATE SEPT. 20, 1966 COMPILED BY AEL CHECKED BY SL
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4500 IN. LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOSE

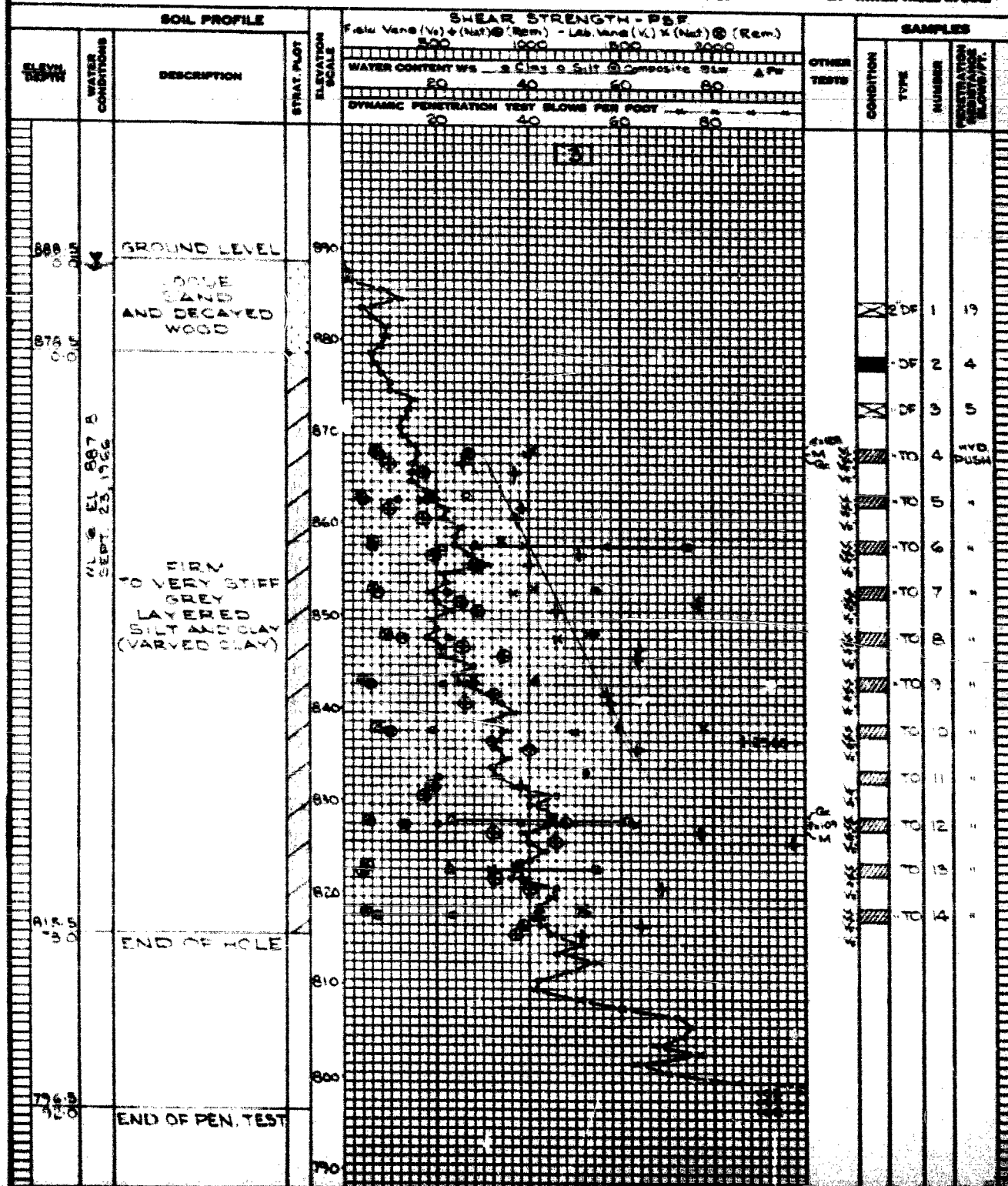
SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 S.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

F.S. - POIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCOMPACTED COMPRESSION
 U - TRIAXIAL CONSOLIDATED UNDEGRADED
 U - TRIAXIAL UNDEGRADED
 S - TRIAXIAL DRAINED
 W - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7923 PEN. TEST 4 DATUM GEODETIC CASING ---
 BORING DATE SEPT. 15/66 REPORT DATE SEPT. 21, 1966 COMPILED BY AEI CHECKED BY ---
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4000 IN - LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 S.O. - DRIVE-OPEN
 S.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

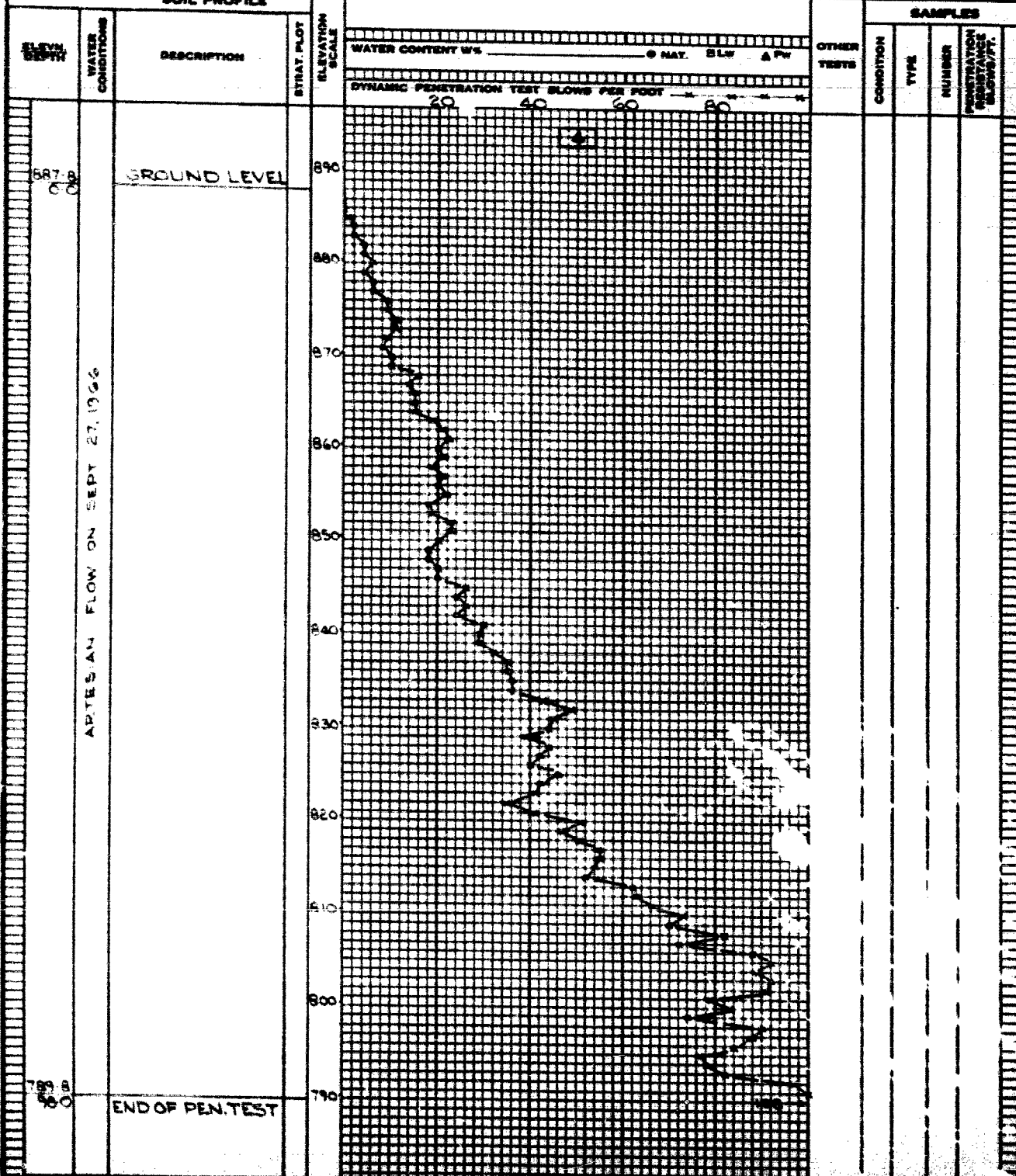
SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED PEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 CC - TRIAXIAL CONSOLIDATED UNDRAINED
 CU - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINAGE
 W - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7923 PEN TEST 5 DATUM GEODETIC CASING 7
 BORING DATE SEPT 19, 1966 REPORT DATE SEPT 26, 1966 COMPILED BY AEL CHECKED BY 211H
 SAMPLER HAMMER WT. 40 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4000 IN - LBS. ENERGY)

SAMPLE CONDITION

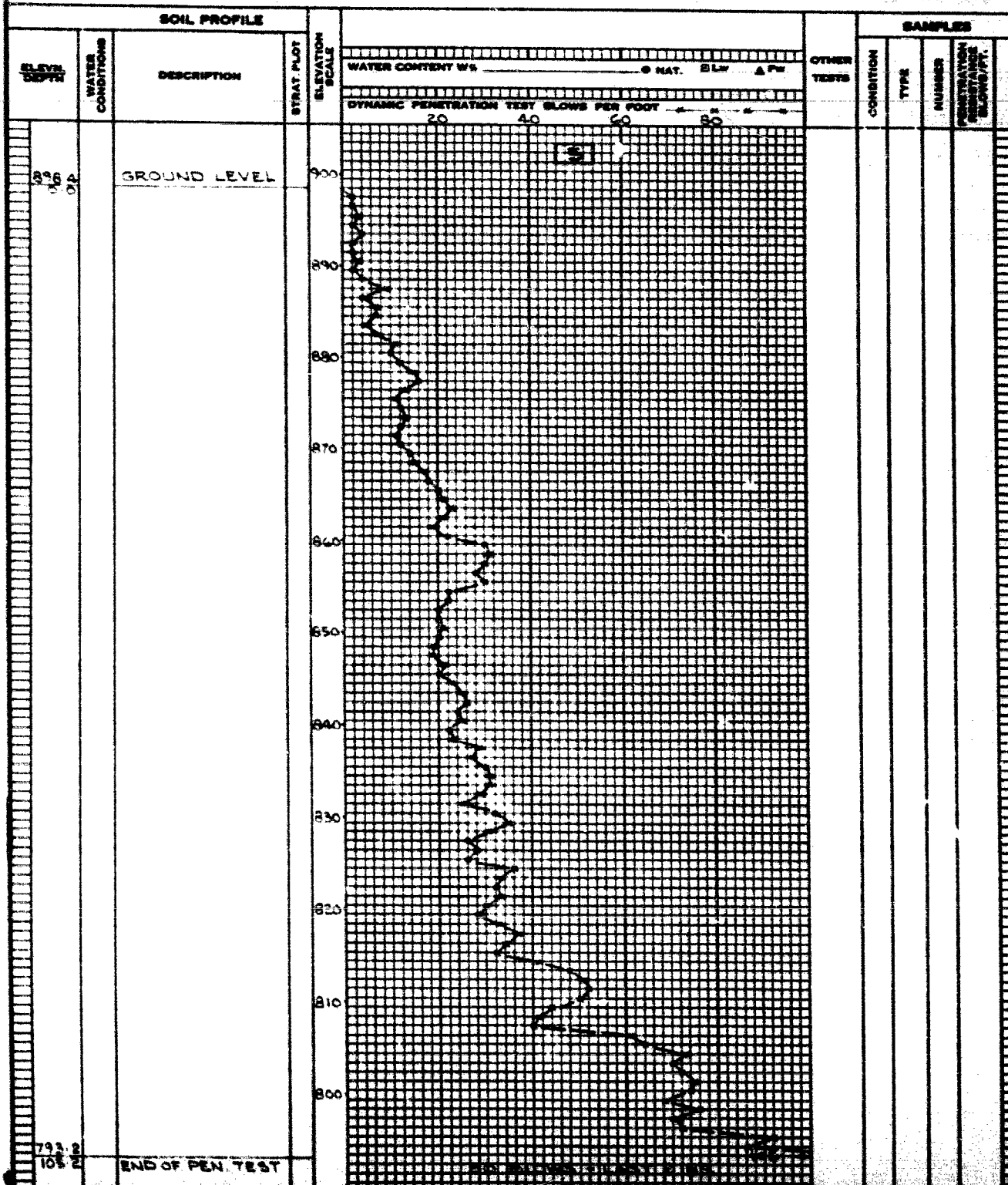
☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 GC - TRIAXIAL CONSOLIDATED UNDRAINED
 S - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINED
 T - WET UNIT WEIGHT
 E - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7923 PEN TEST 6 DATUM GEODETYC CASING —
 BORING DATE SEPT. 2, 1966 REPORT DATE SEPT. 20, 1966 COMPILED BY A. E. L. CHECKED BY G. H. E.
 SAMPLER HAMMER WT. 60 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO SLOWS OF 4800 IN. LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

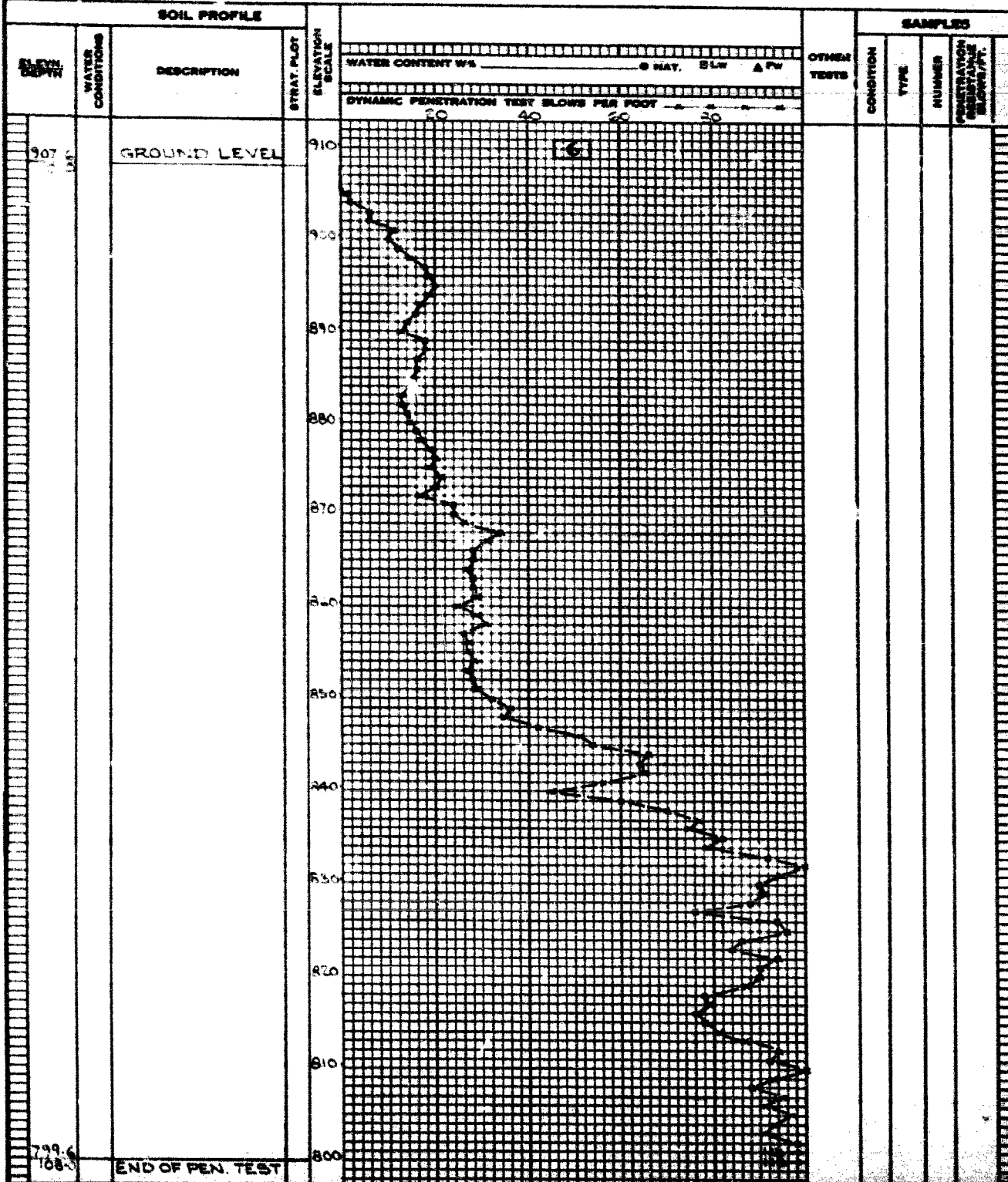
A.S. AUGER SAMPLE
 S.T. SLOTTED TUBE
 W.S. WASHED SAMPLE
 D.O. DRIVE-OPEN
 D.F. DRIVE-FOOT VALVE
 C.S. CHUNK SAMPLE

F.S. FOIL SAMPLE
 S.C. SLEEVE-OPEN
 S.V. SLEEVE-FOOT VALVE
 T.O. THIN WALLED OPEN
 R.C. ROCK CORE

ABBREVIATIONS

V. IN-SITU VANE TEST
 M. MECHANICAL ANALYSIS
 U. UNCONSOLIDATED COMPRESSION
 O. TRIAXIAL CONSOLIDATED UNDRAINED
 S. TRIAXIAL UNDRAINED
 W. WET UNIT WEIGHT
 K. PERMEABILITY
 C. CONSOLIDATION
 W.L. WATER LEVEL IN CASING
 W.T. WATER TABLE IN SOIL

SOIL PROFILE



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 179.23 PEN TEST 7 DATUM GEODETIC CASING
 BORING DATE SEPT. 8, 1966 REPORT DATE SEPT. 21, 1966 COMPILED BY A.E.L. CHECKED BY
 SAMPLED HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4500 IN. LBS. ENERGY)

SAMPLE CONDITION



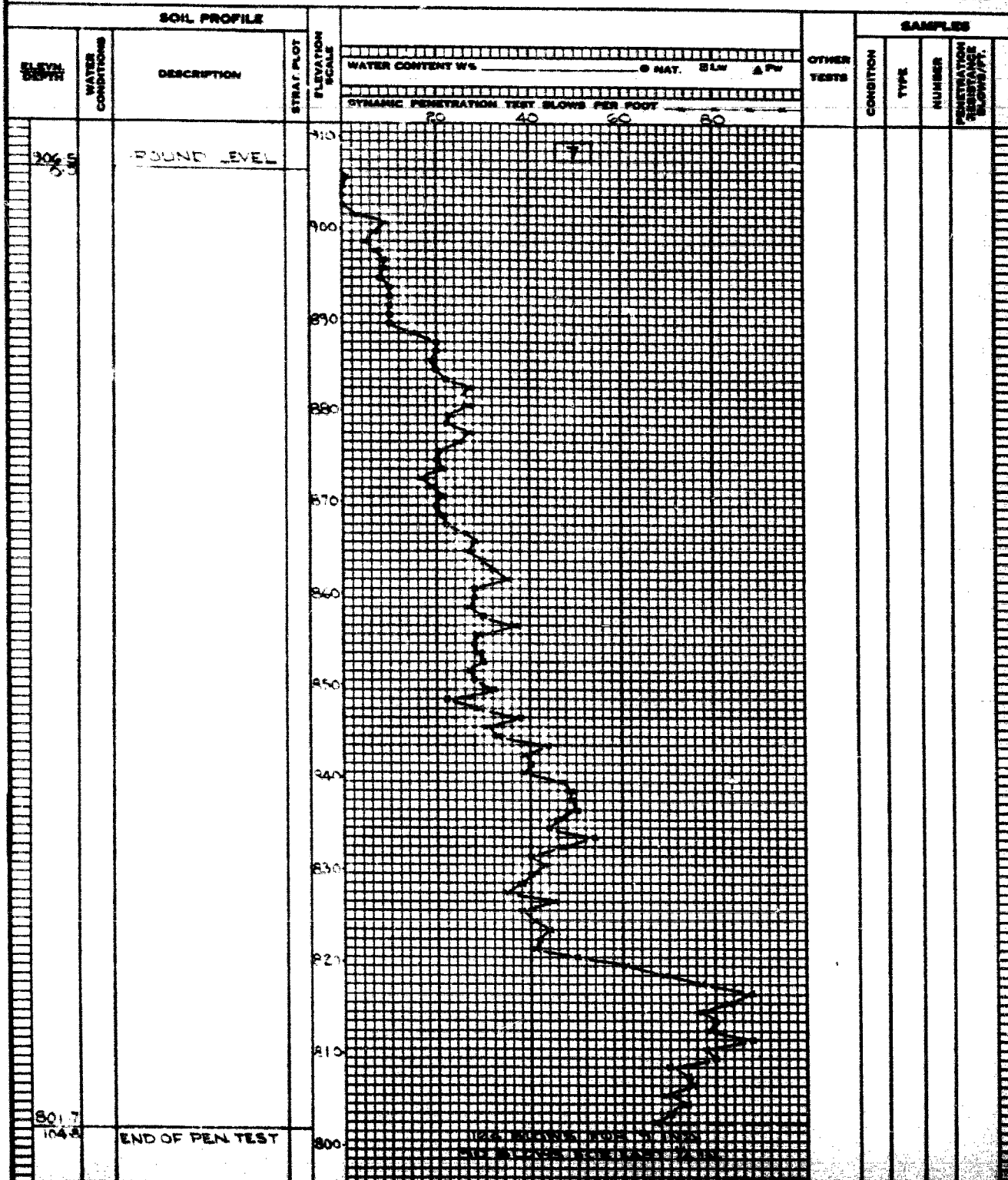
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOUL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 CC - TRIAXIAL CONSOLIDATED UNGRAINED
 CU - TRIAXIAL UNGRAINED
 C - TRIAXIAL GRAINED
 W - WET UNIT WEIGHT
 N - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7223 BORING # 8 DATUM GEODETIC CASING NX & BK
 BORING DATE SEPT 16, 1966 REPORT DATE SEPT 26, 1966 COMPILED BY AEL CHECKED BY GTH
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

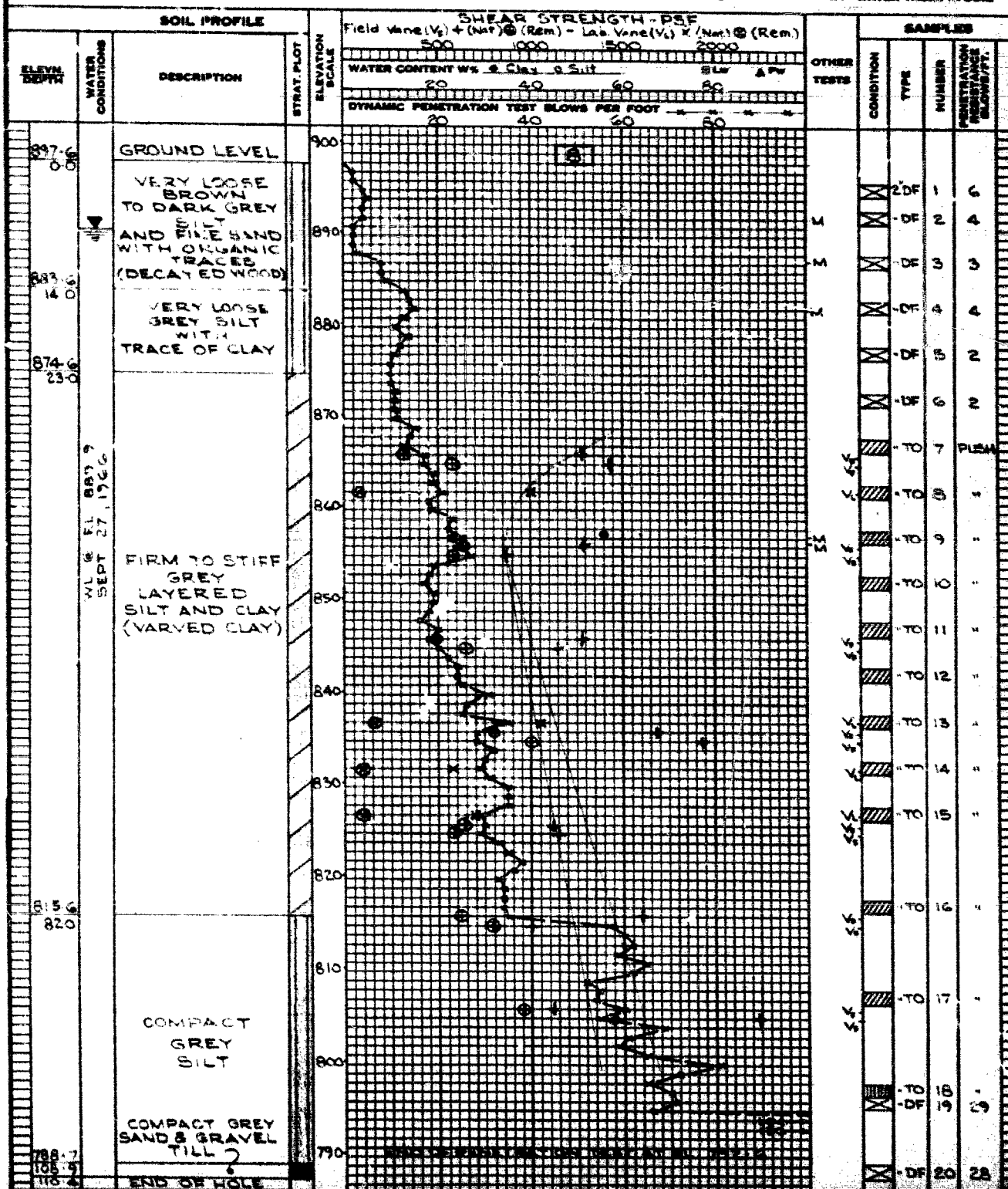
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
S.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V. - IN-SITU VANE TEST
M. - MECHANICAL ANALYSIS
U. - UNCONFINED COMPRESSION
OC. - TRIAXIAL CONSOLIDATED UNDRAINED
O. - TRIAXIAL UNDRAINED
S. - TRIAXIAL DRAINED

ABBREVIATIONS

1. - TEST UNIT WEIGHT
K. - PERMEABILITY
C. - CONSOLIDATION
WL. - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7923 BORING # 9 DATUM GEODETIC CASING BL
 BORING DATE SEPT. 21-22/66 REPORT DATE SEPT. 23, 1966 COMPILED BY ARL CHECKED BY G.M.H.
 SAMPLER HAMMER WT. 160 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO FLOWS OF 4800 IN. - LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOSE

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 S.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

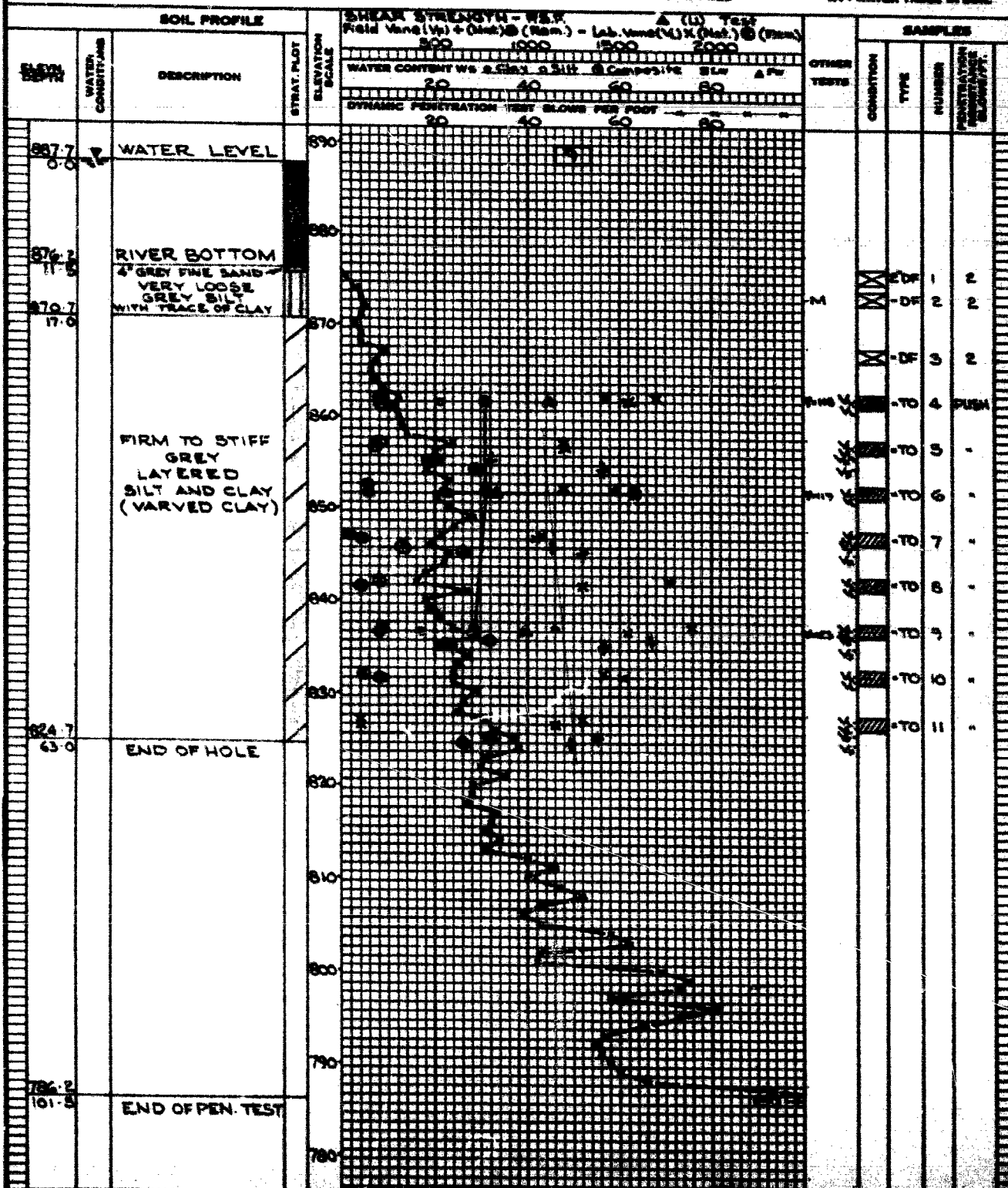
SAMPLE TYPES

F.S. - FILL SAMPLE
 S.S. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - VIBROT VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 CC - TRIAXIAL CONSOLIDATED UNDRAINED
 CU - TRIAXIAL UNCONSOLIDATED
 S - TRIAXIAL, DRAINAGE

Y - TEST UNIT WEIGHT
 S - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7923 BORING # 10 DATUM GEODETIC CASING Bx
 BORING DATE SEPT. 26/66 REPORT DATE SEPT. 30, 1966 COMPILED BY AEL CHECKED BY S
 SAMPLER MANNER W 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4500 IN. LBS. ENERGY)

SAMPLE CONDITION



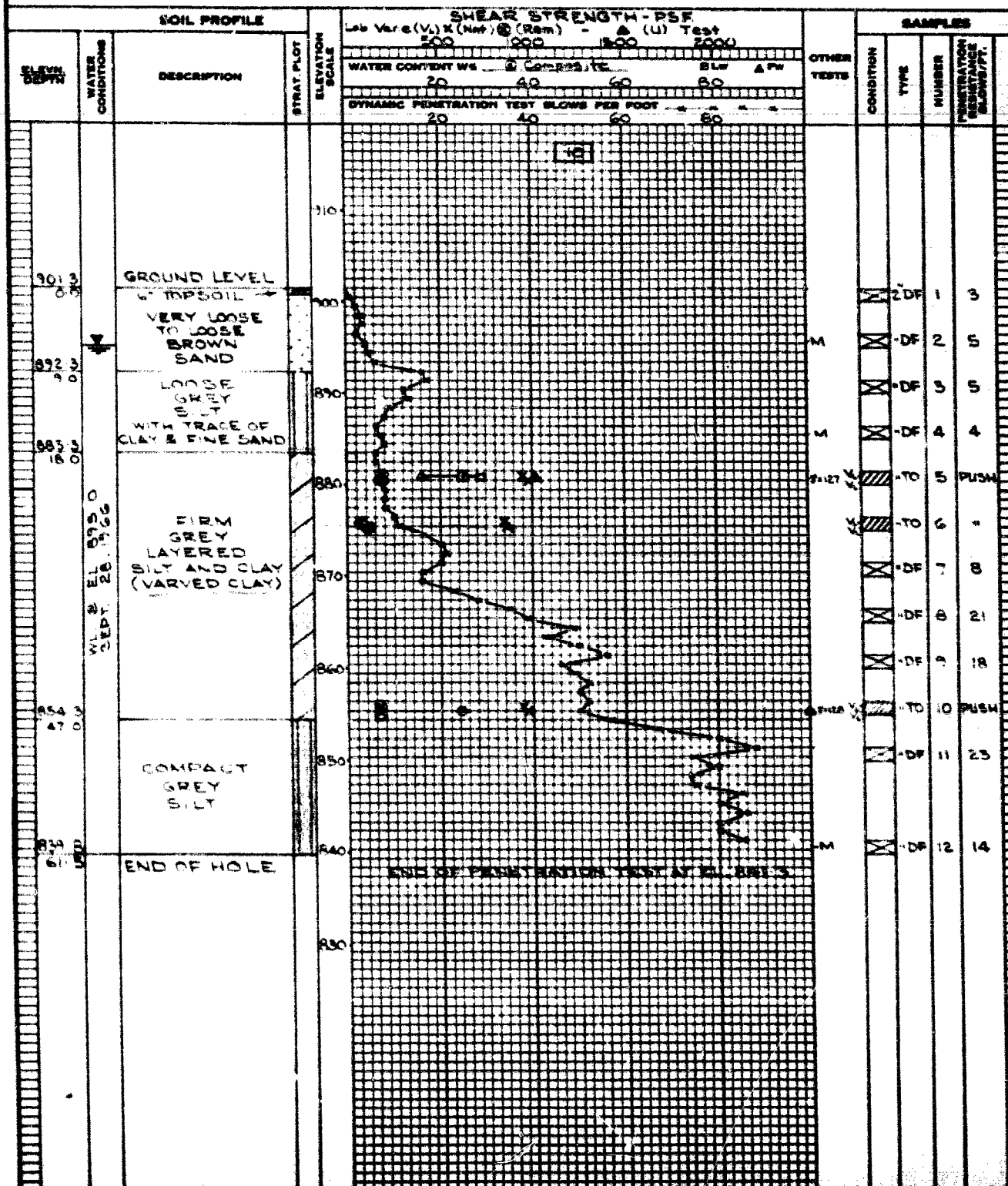
S.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUCK SAMPLE

SAMPLE TYPES

V.S. - SOIL SAMPLE
 S.D. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 V.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 UC - TRIAXIAL CONSOLIDATED UNDRAINED
 UG - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINED
 W - WET UNIT WEIGHT
 X - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T3323 BORING 11 DATUM GEODETIC CASING EX
 BORING DATE SEPT. 27-28, 1966 REPORT DATE SEPT. 30, 1966 COMPILED BY AEL CHECKED BY G.H.
 SAMPLER HAMMER WT 140 LBS. SHIP 20 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4000 IN. LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

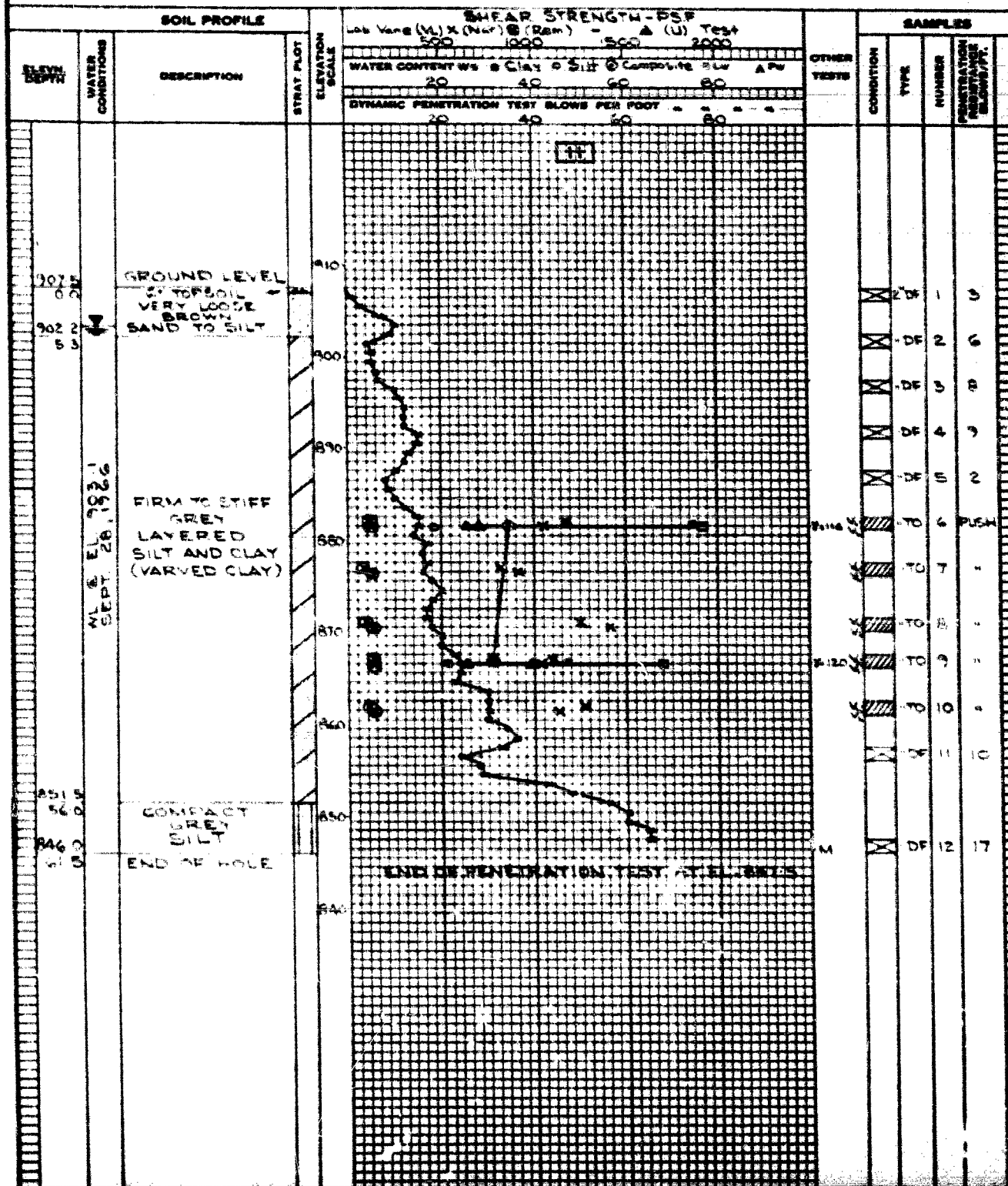
A.S. AUGER SAMPLE
 S.T. SLOTTED TUBE
 W.S. WASHED SAMPLE
 D.O. DRIVE-OPEN
 D.F. DRIVE-FOOT VALVE
 C.S. CHUNK SAMPLE

SAMPLE TYPES

F.S. FOIL SAMPLE
 S.O. SLEEVE-OPEN
 S.F. SLEEVE-FOOT VALVE
 T.O. THIN WALLED OPEN
 R.C. ROCK CORE

ABBREVIATIONS

V. IN-SITU VANE TEST
 M. MECHANICAL ANALYSIS
 U. UNCONFINED COMPRESSION
 C. TRIAXIAL CONSOLIDATED UNCONFINED
 S. TRIAXIAL UNCONFINED
 S. TRIAXIAL DRAINED
 W. WET UNIT WEIGHT
 K. PERMEABILITY
 C. CONSOLIDATION
 WL. WATER LEVEL IN CASING
 WT. WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7923 PEN TEST 12 DATUM GEODETIC CASING 11H
 BORING DATE SEPT 17/66 REPORT DATE SEPT 30, 1966 COMPILED BY AEL CHECKED BY JTH
 SAMPLE DR HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4000 IN. LBS. ENERGY)

SAMPLE CONDITION

☒ DISTURBED
☐ PASS
☐ GOOD
☐ LOOSY

SAMPLE TYPES

A.S. AUGER SAMPLE
 S.T. SLOTTED TUBE
 W.S. WASHED SAMPLE
 S.O. SLEEVE-OPEN
 S.F. SLEEVE-FOOT VALVE
 T.O. THIN WALLED OPEN
 R.C. ROCK CORE

F.S. FOL SAMPLE
 S.O. SLEEVE-OPEN
 S.F. SLEEVE-FOOT VALVE
 T.O. THIN WALLED OPEN
 R.C. ROCK CORE

ABBREVIATIONS

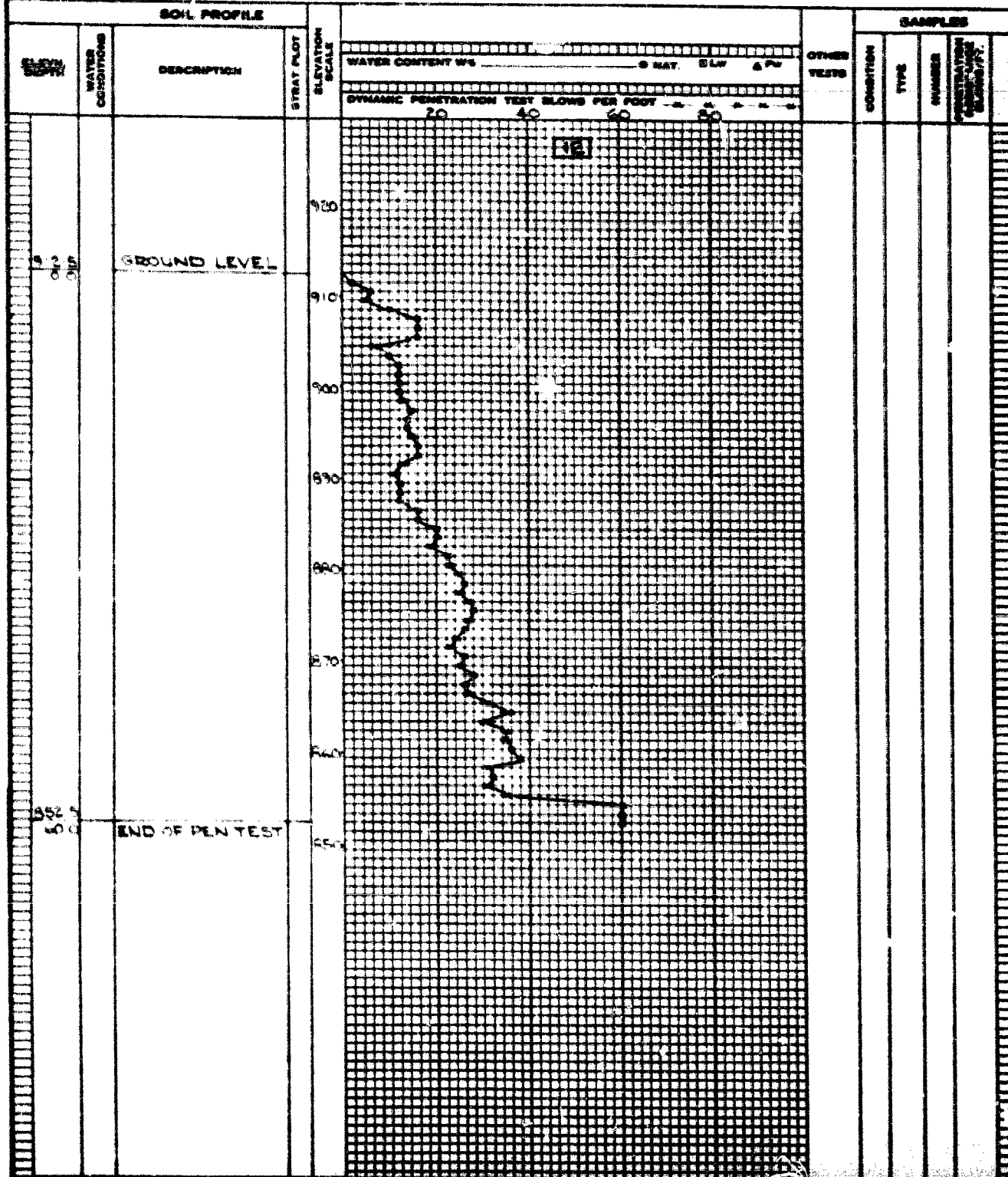
V. IN-SITU VANE TEST
 M. MECHANICAL ANALYSIS
 U. UNCOMPIRED COMPRESSION
 C. TRIAXIAL CONSOLIDATED UNDRAINED
 U. TRIAXIAL UNDRAINED
 S. TRIAXIAL DRAINED

W. WET UNIT WEIGHT
 P. PERMEABILITY
 C. CONSOLIDATION

WL. WATER LEVEL IN CASING
 WT. WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



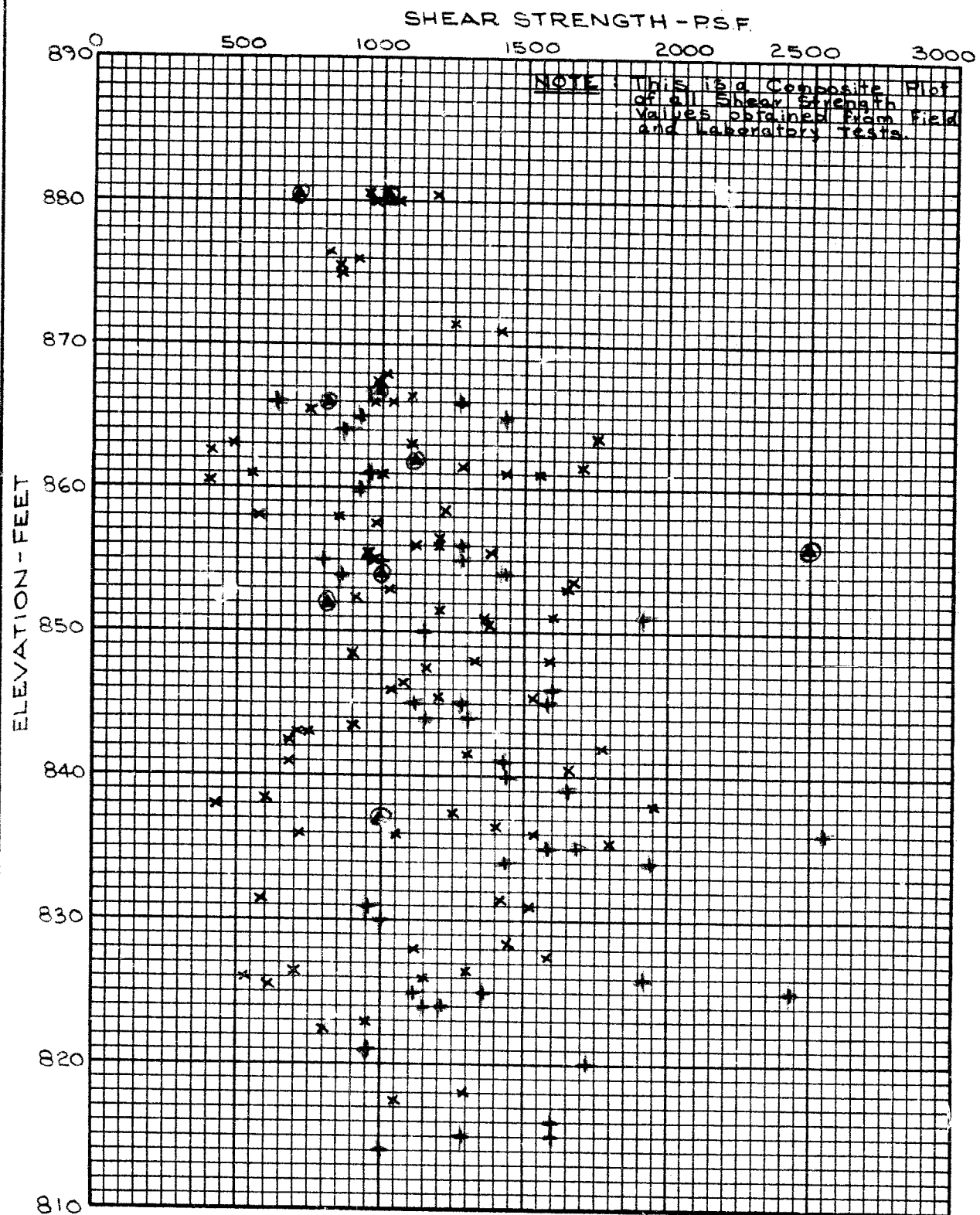
APPENDIX II

FIGURES - LABORATORY TESTING

GEOCON

SHEAR STRENGTH vs. ELEVATION

APPENDIX II
FIGURE 1
PROJECT T7923



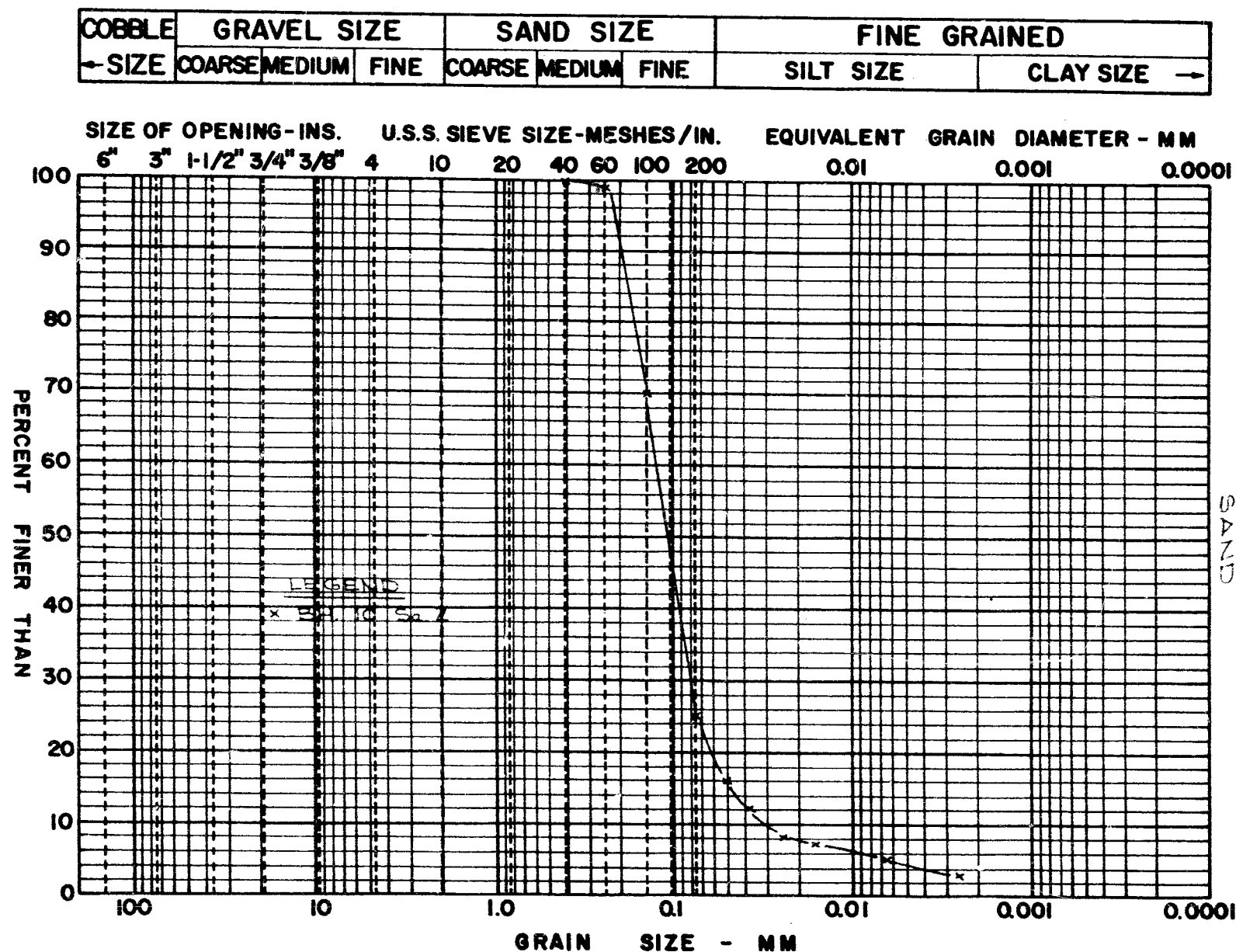
LEGEND

- + Field Vane
- x Laboratory Vane
- ▲ Unconfined Compression

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GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 2
PROJECT T7923



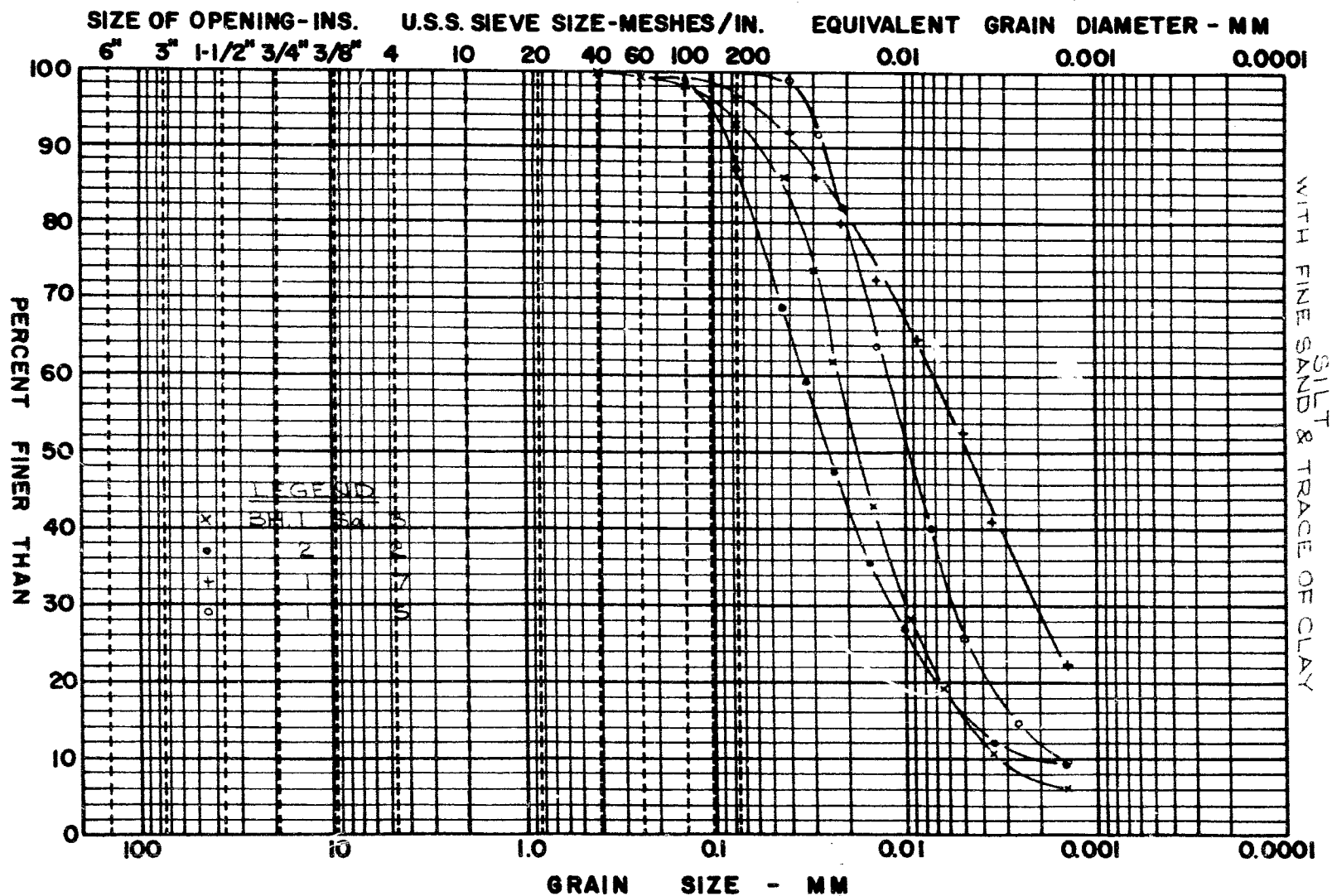
M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 3
PROJECT T7923

COBBLE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	
← SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE →

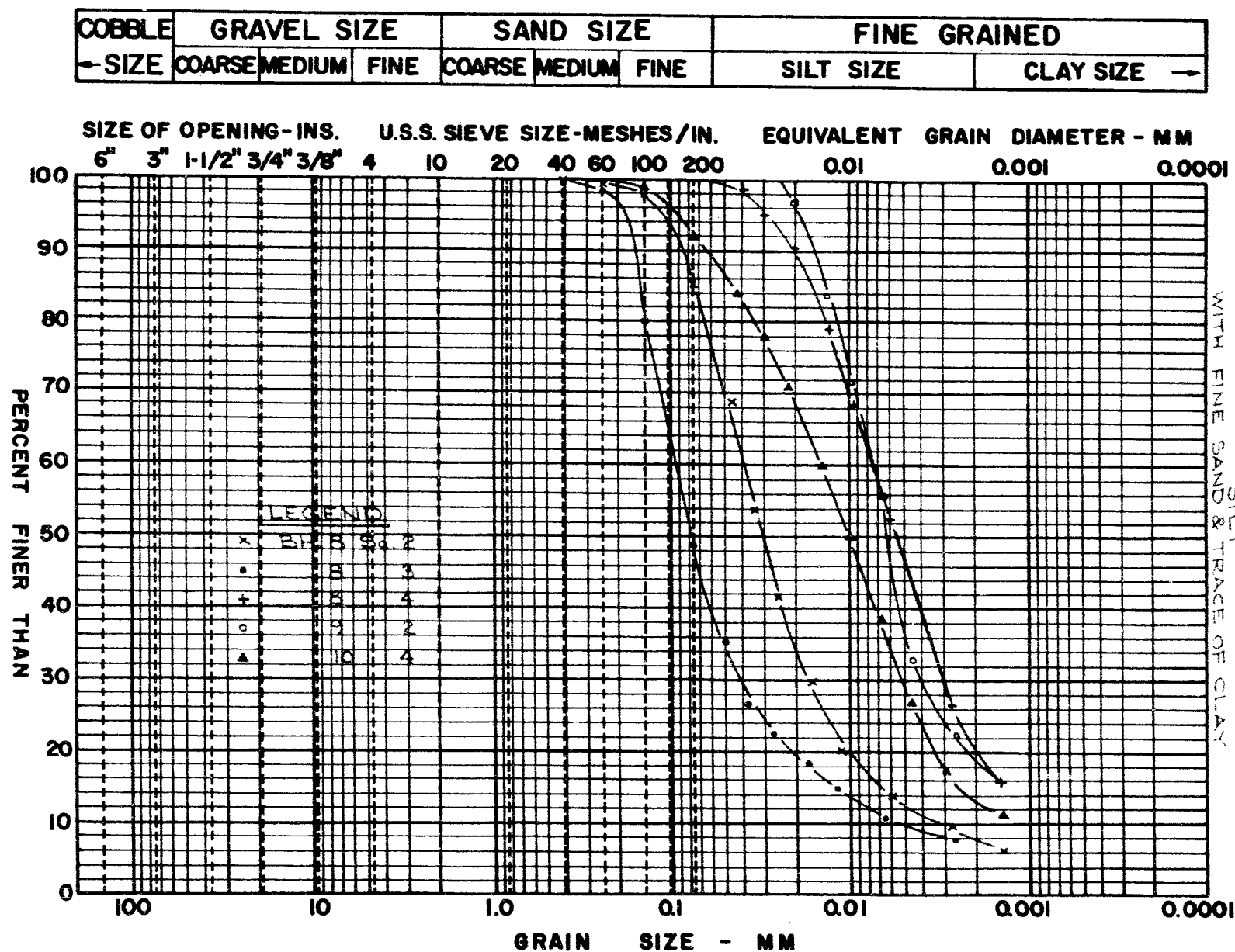


M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 4
PROJECT T7923

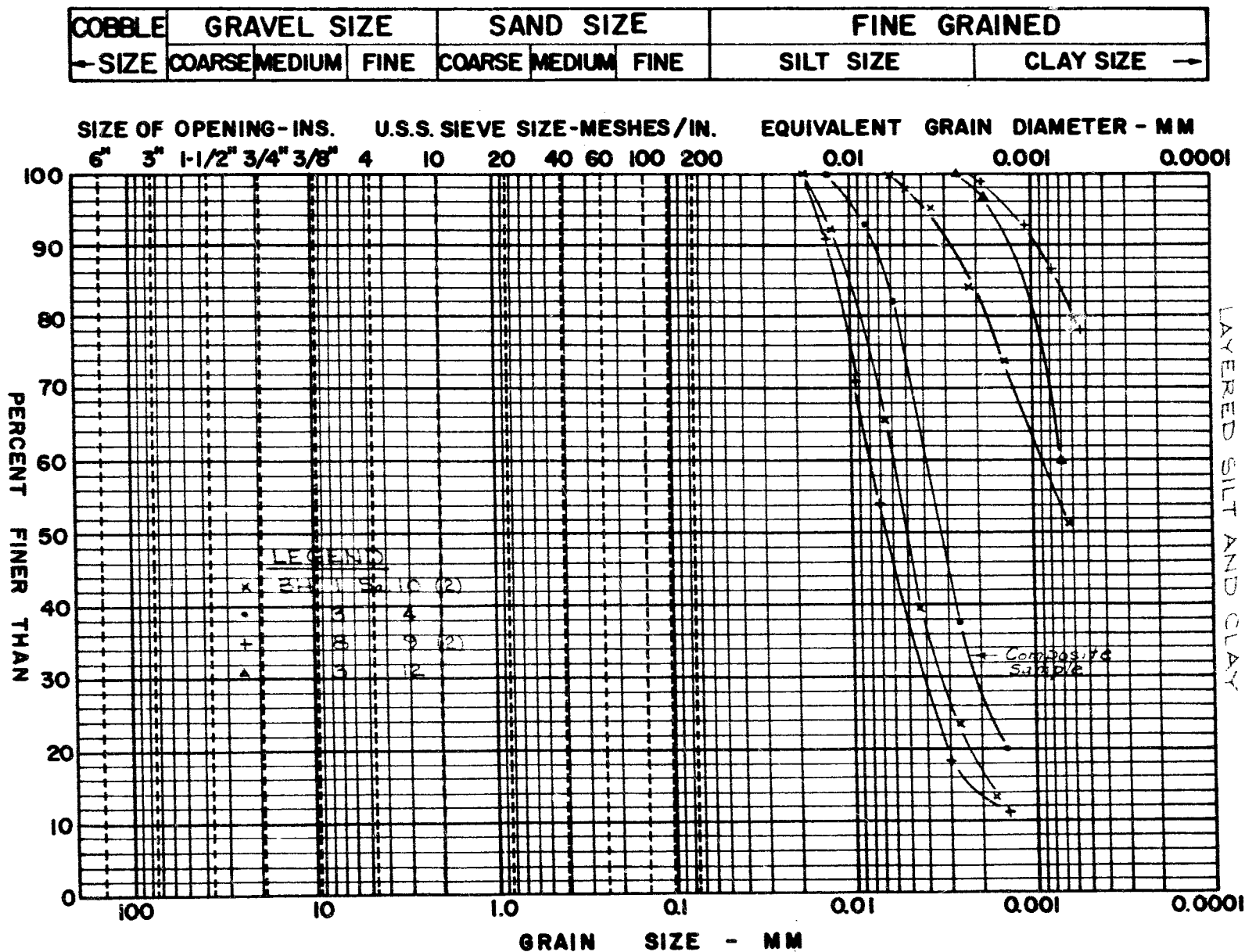


M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 5
PROJECT T7923

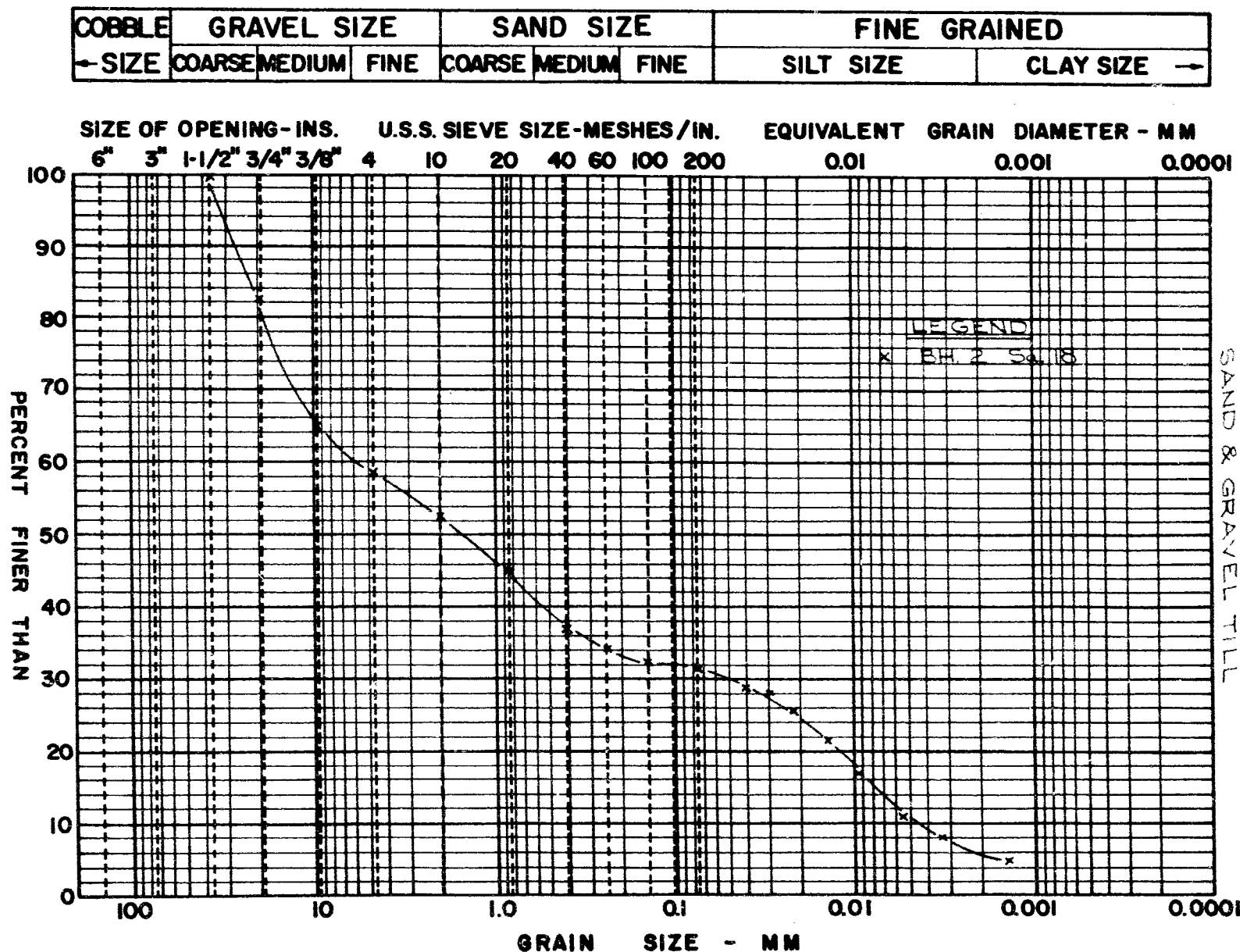


M.I.T. GRAIN SIZE SCALE

GEOCON

GRAIN SIZE DISTRIBUTION

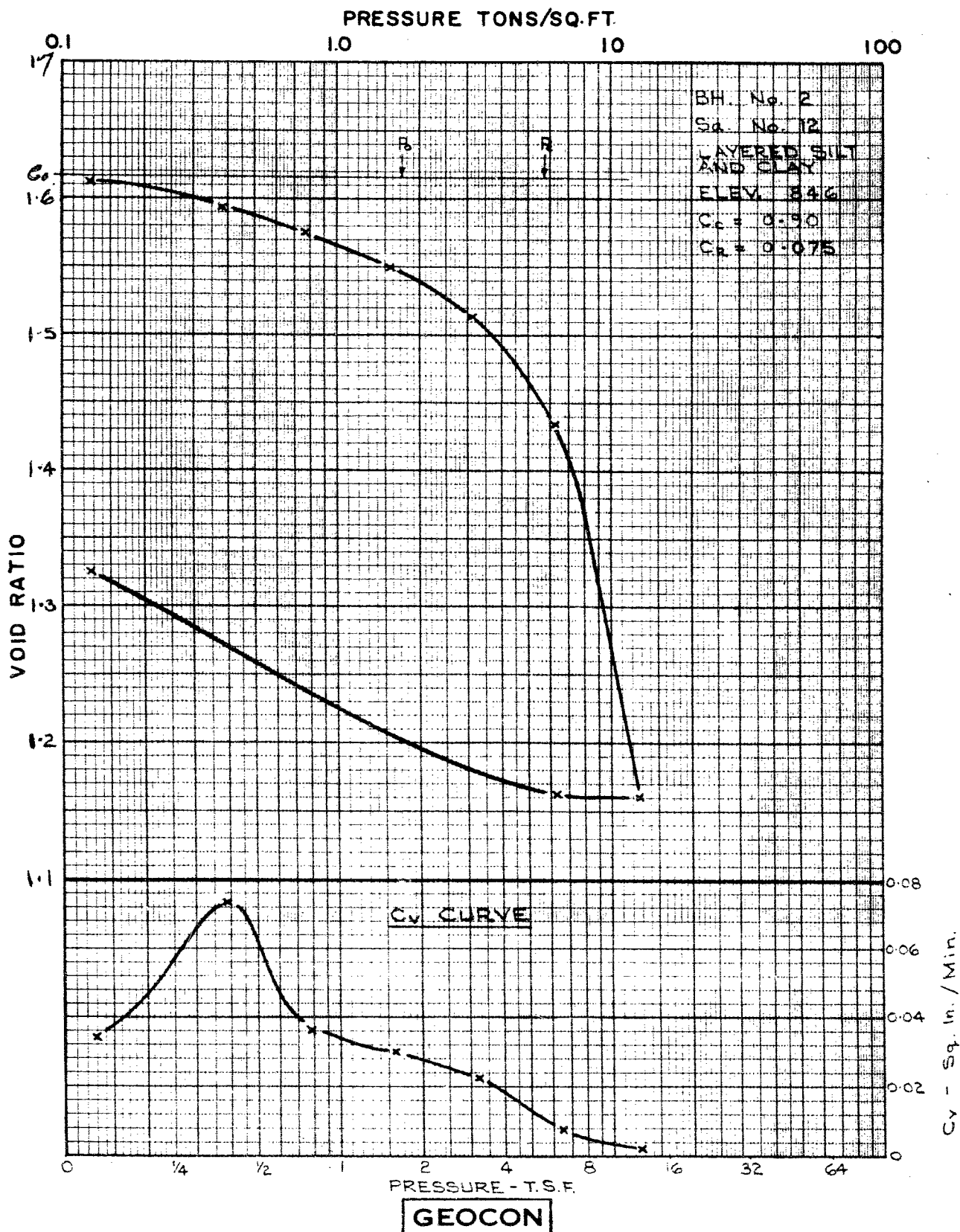
APPENDIX 11
FIGURE 7
PROJECT T7923



M.I.T. GRAIN SIZE SCALE

VOID RATIO-PRESSURE DIAGRAM CONSOLIDATION TEST

APPENDIX II
FIGURE 8
PROJECT T7923

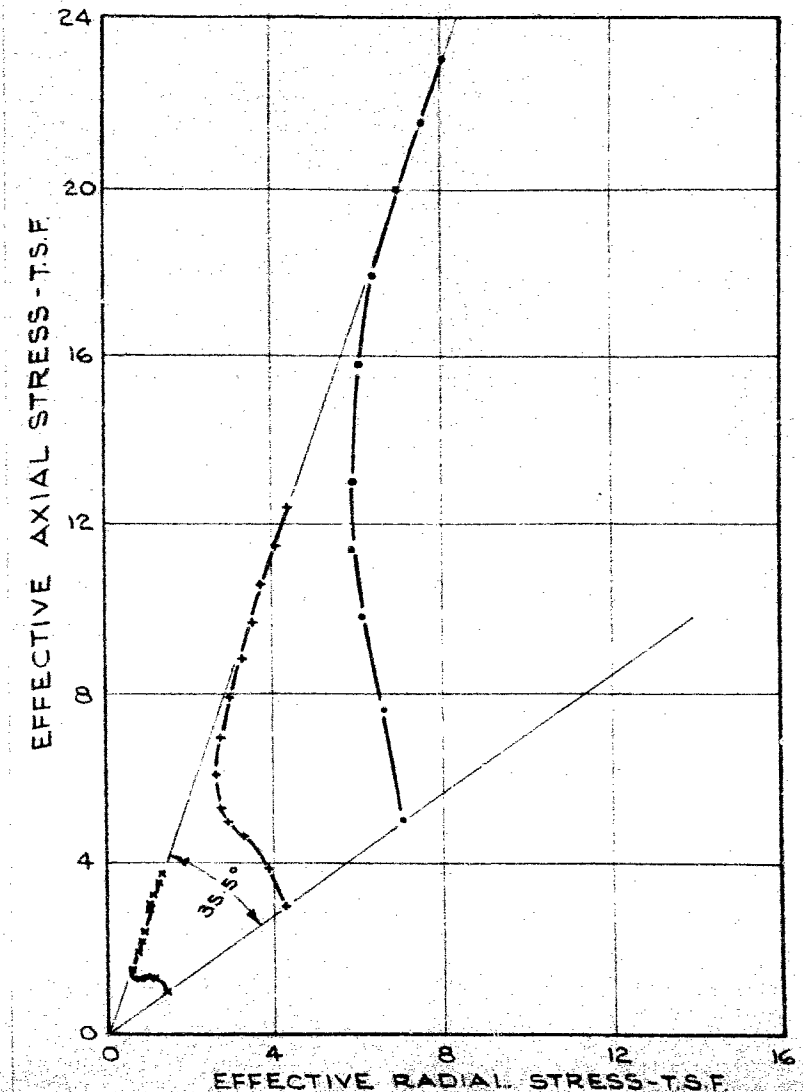
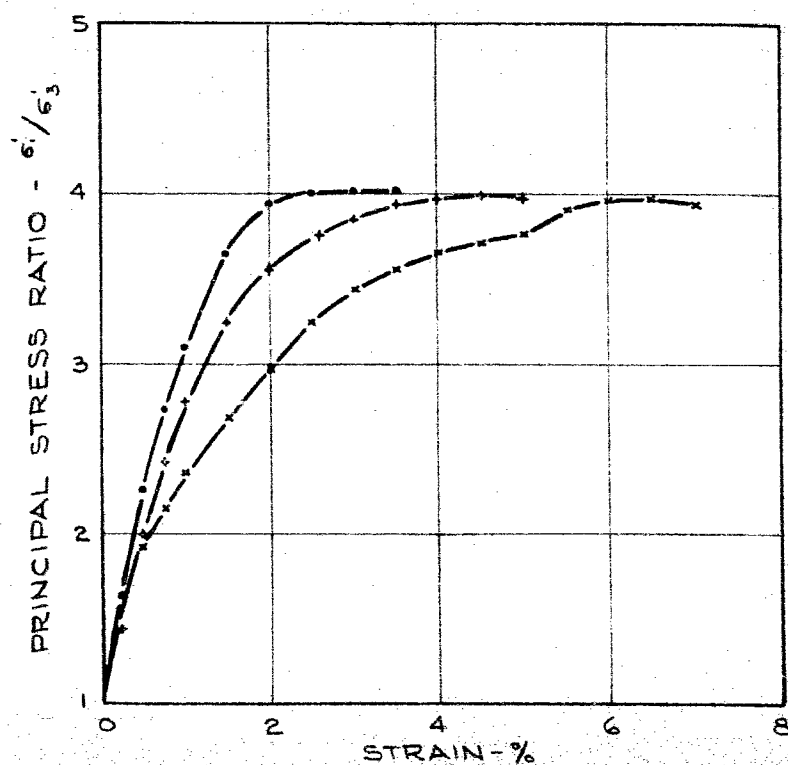
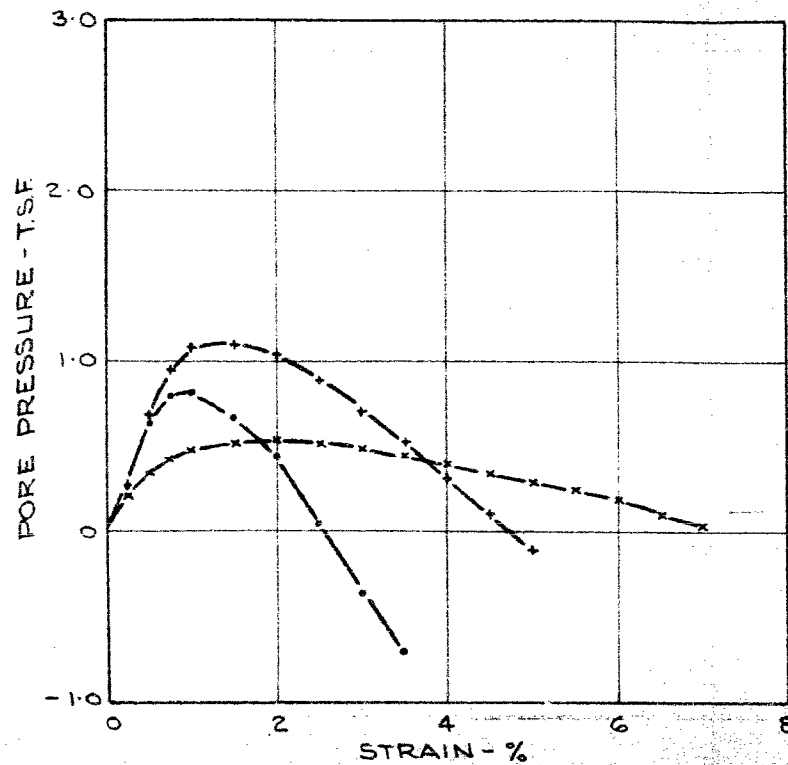
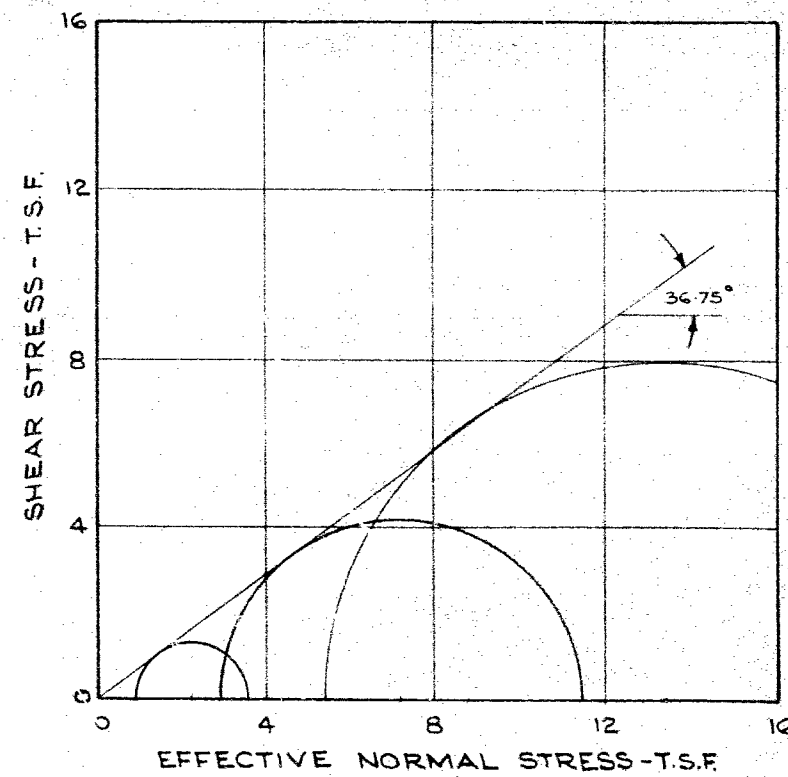
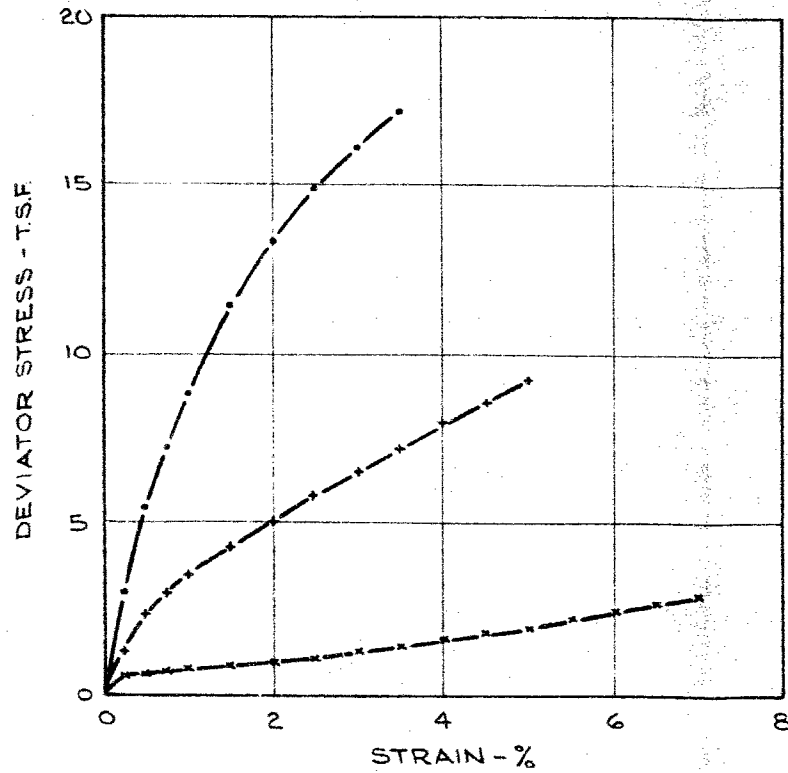


CONSOLIDATED UNDRAINED TRIAXIAL TESTS LAYERED SILT AND CLAY BOREHOLE 3 SAMPLE 4

APPENDIX 1.
FIGURE 9
PROJECT T7923

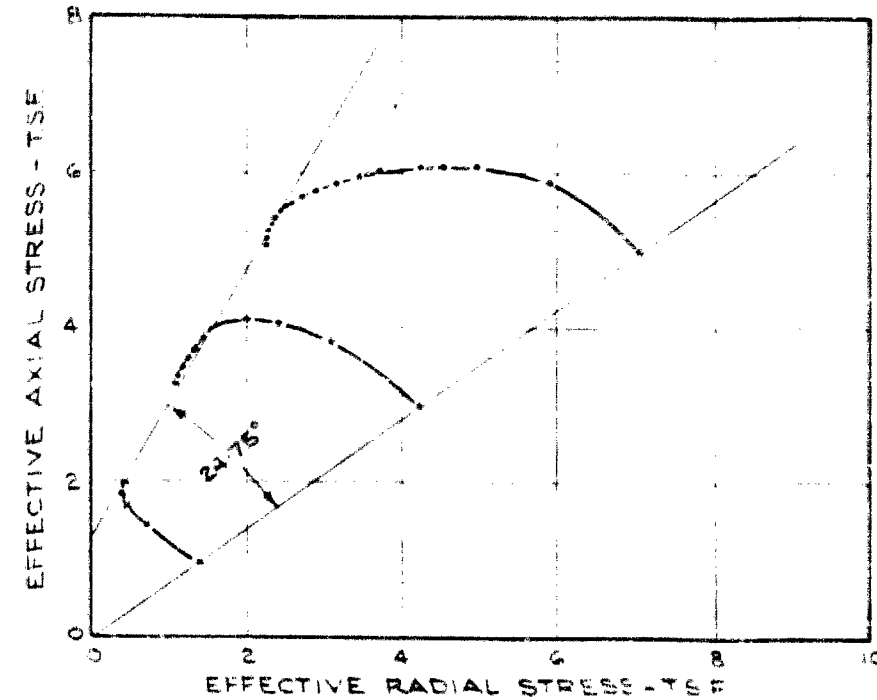
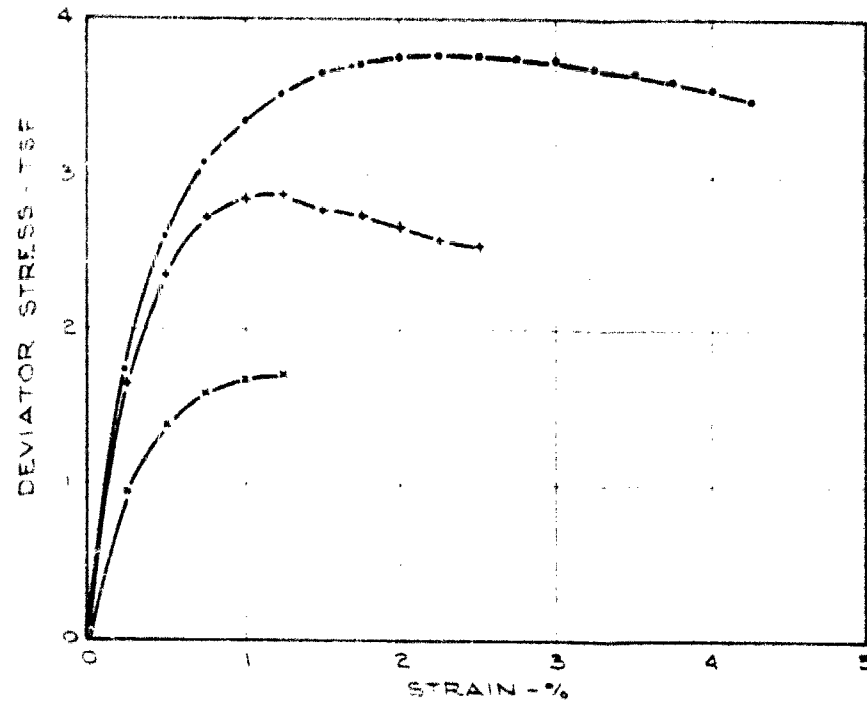
SYMBOL	TEST No.	σ_3 (T.S.F.)	γ (P.C.F.)	WATER CONTENT
x	1	1.0		
+	2	3.0	127.8	26.5
•	3	5.0		

1. FILTER DRAINS - TOP, SIDE & BOTTOM
2. RATE OF STRAIN - 2% PER HOUR
3. BACK PRESSURE - 2.0 T.S.F. (1.0 T.S.F. Test No.3)



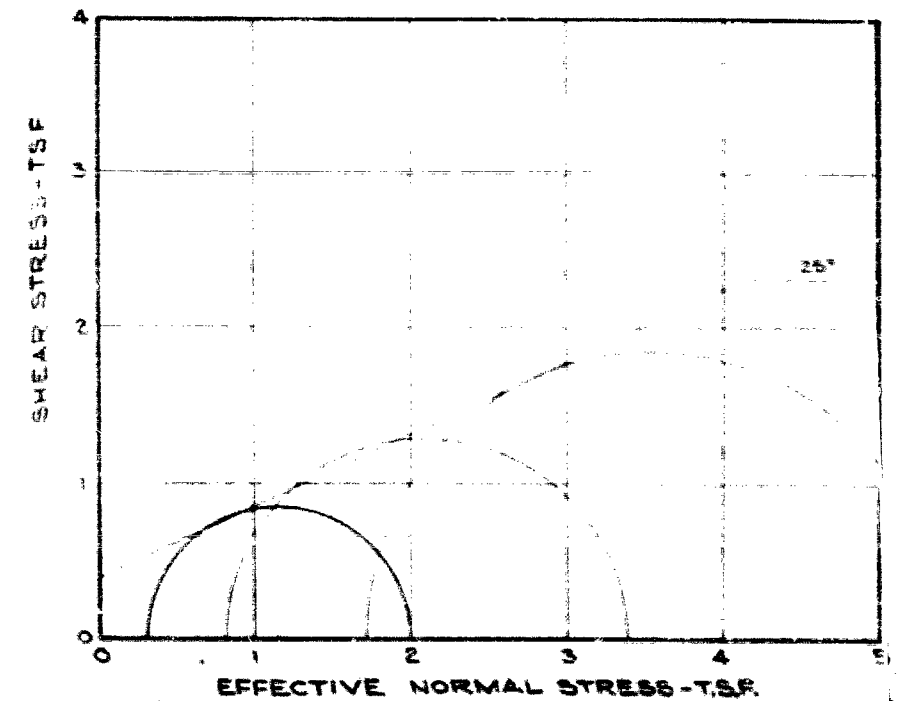
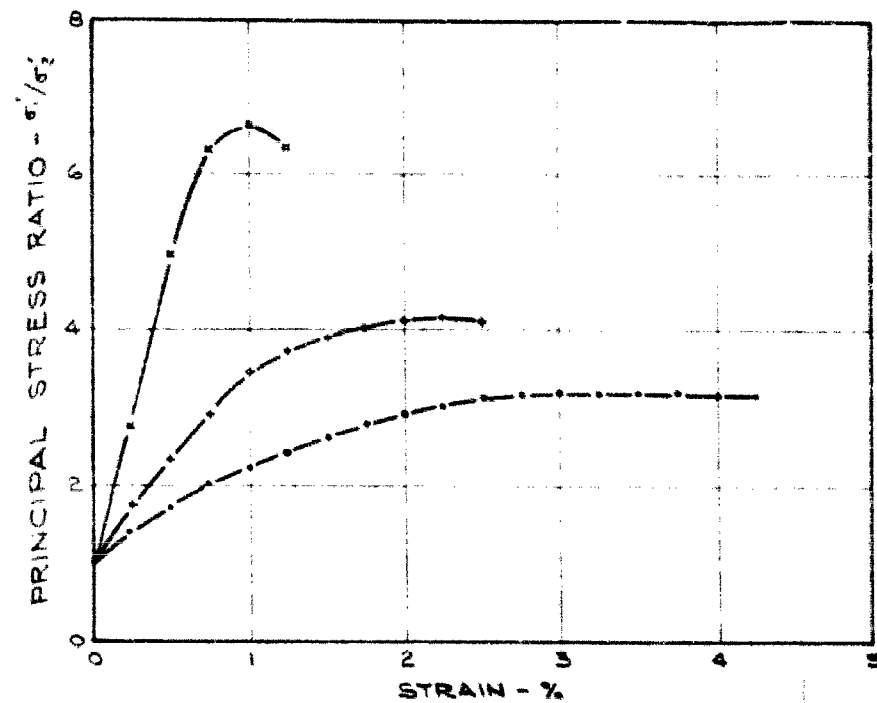
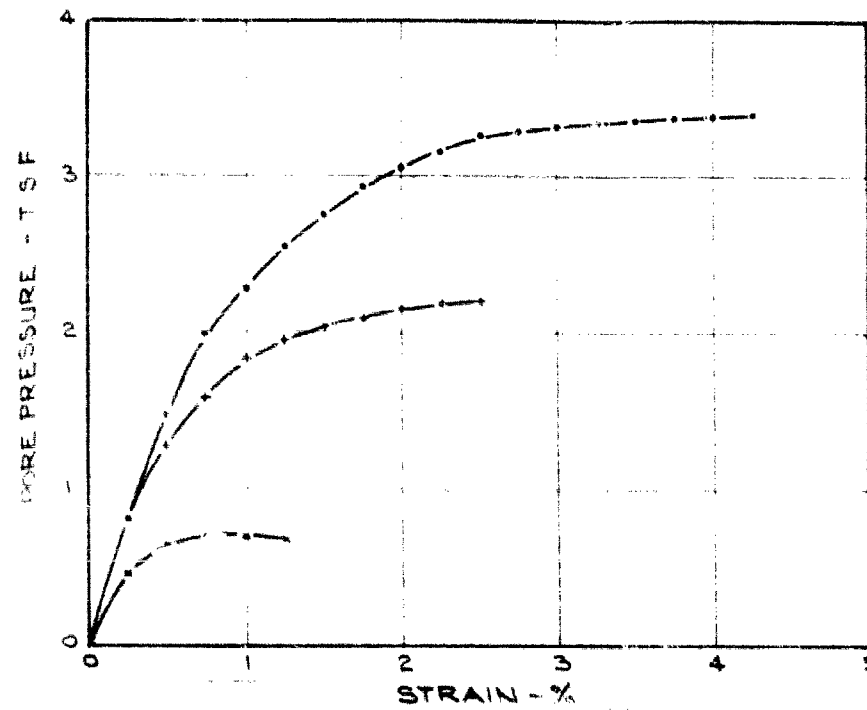
CONSOLIDATED UNDRAINED TRIAXIAL TESTS LAYERED SILT AND CLAY BOREHOLE 3 SAMPLE 12

APPENDIX II
 FIGURE 10
 PROJECT T7923



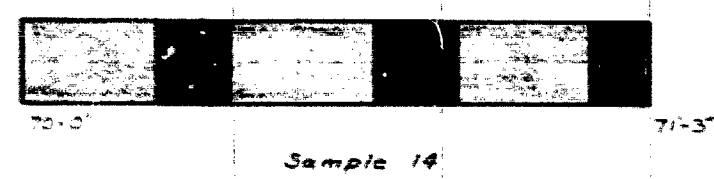
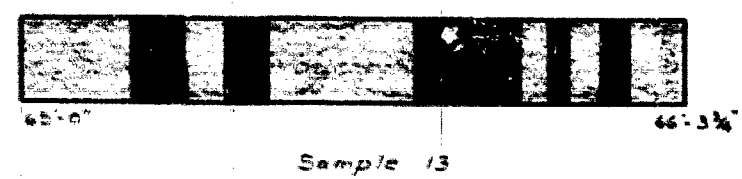
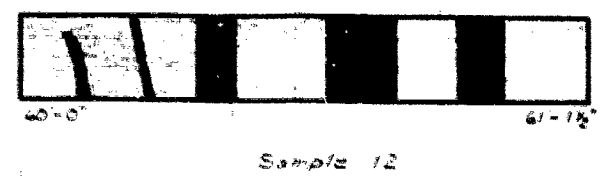
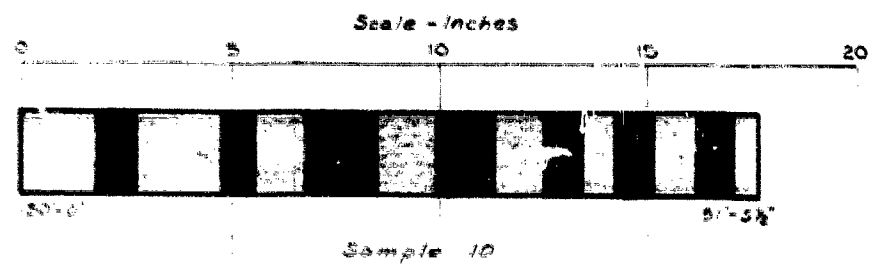
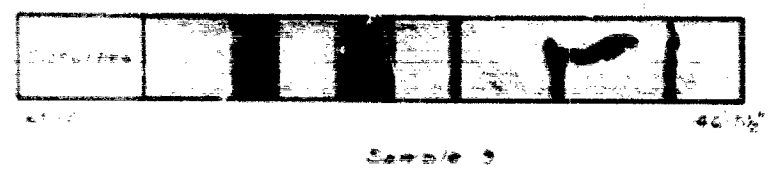
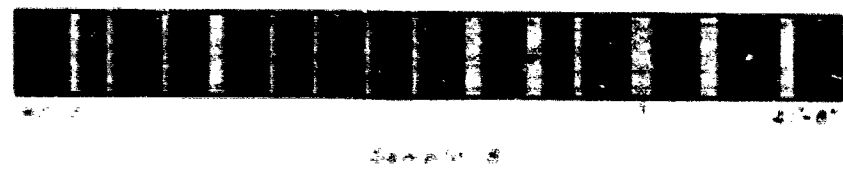
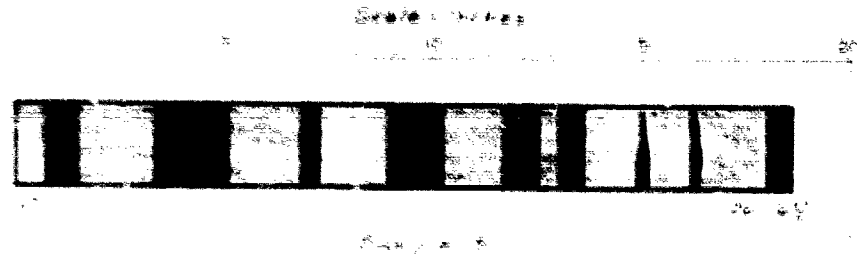
SYMBOL	TEST No.	σ_3 (TSF)	γ (PCF)	WATER CONTENT
x	1	1.0		
+	2	3.0	109.3	42.4
•	3	5.0		

1. FILTER DRAINS - TOP, SIDE & BOTTOM
2. RATE OF STRAIN - 2% PER HOUR
3. BACK PRESSURE - 2.0 TSF



SAMPLE SECTIONS
OF LAYERED SILT & CLAY
BOREHOLE 3

APPENDIX II
FIGURE II
PROJECT T7923



LIGHT GREY SILT



DARK GREY CLAY

GEOCON

MEMORANDUM

To: Mr. J. H. Blevins,
District Engineer,
District #18 - Sault Ste. Marie.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. H. G. Potts,
District Construction Engr.

DATE: April 17, 1969

OUR FILE REF:

IN REPLY TO

SUBJECT:

Minutes of Job Site Meeting for
Contract 68-20 - Black River Bridge
and Approaches - Hwy. No. 614.

WP's 44-64
114-64
Gleason/66

Further to your memo of April 11, 1969, I enclose suggested amendments to Points 2, 6, and 9 of your draft copy of the minutes:

2. Mr. Selby carefully explained the history of the design of the structure from the foundation and stability view. It was emphasized that the sequence of construction, as indicated in the Special Provisions, was the result of a thorough investigation and judgment, and that no reasons for changing this sequence could be foreseen at this time. It was also stated by Mr. Selby that a necessary requirement for stability was that the present Black River channel adjacent to the proposed embankment, must be filled to el. 900 prior to any embankment construction taking place in this area.
6. Mr. Selby questioned the Contractor on how he was going to excavate the stream diversion. He advised the Contractor that excavating in layers down to the groundwater level, starting from the downstream end, would have the effect of gradually depressing the groundwater as the work proceeded, thus rendering the silty soil more workable. Mr. Armstrong indicated that he would likely be removing the soil in this manner. Mr. Fancy pointed out that some of the material will likely have to be moved or hauled away.
9. The Contractor indicated his desire to sheet in the piers and pump the water outside of the sheeting as well as inside the sheeting. This would enable him to use a shorter length of sheeting, since it would have to project above the level of the water in the diversion. Mr. Selby stated that it would be most unwise to lower the water outside of the sheeting since, at this stage, the approach embankments would be built and quite probably, boiling would occur at the embankment toes, thus

Mr. J. H. Blevins, District Engr.,
Dist. #18 - Sault Ste. Marie,
Attn: Mr. H. G. Potts, Construction Engr.

2

April 17, 1969

Contract 68-20 - Black River Bridge and Approaches - (cont'd.) ...

9. (cont'd.) ...

creating a dangerous stability problem. In connection with pumping inside the sheeting, no problems would occur, provided that the sheeting was advanced far enough into the subsoil and was adequately braced. This latter problem is, however, the specific responsibility of the Contractor, and he would also have to ensure that his method of construction precluded the possibility of ground heave inside the cofferdam. At a later stage, advice on this aspect could be obtained from the Foundation Section.

With regard to the question raised in Point No. 5, I feel that the spoil material will be so wet that it will not be possible to pile it to a sufficient height to cause a base failure of the subsoil. The spoil itself, might possibly slide back into the stream diversion, however. For that reason, I suggest that the toe of the spoil heap be kept back about 10 feet from the edge, and that the material be dumped so as to form a slope not steeper than the sides of the new channel. It should, of course, be removed as soon as possible.

K. G. Selby

KGS/MdeF

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

cc: Foundations Files
Gen. Files

MEMORANDUM

To: Mr. J. H. Blevins,
District Engineer,
District #18 - Sault Ste. Marie.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. H. G. Potts,
District Construction Engr.

DATE: April 17, 1969

OUR FILE REF:

IN REPLY TO

SUBJECT:

Minutes of Job Site Meeting for
Contract 68-20 - Black River Bridge
and Approaches - Hwy. No. 614.

W.P. 114-64
Gecoon/66

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Mr. J. H. Blevins, District Engr.,
Dist. #18 - Sault Ste. Marie,
Attn: Mr. H. G. Potts, Construction Engr.

2

April 17, 1969

Contract 68-20 - Black River Bridge and Approaches - (cont'd.) ...

9. (cont'd.) ...

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K. G. Selby

KGS/MdeF

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

cc: Foundations Files
Gen. Files

Department of Highways Ontario

Copy for the information of

Mr. K. Selby, ✓

FOUNDATION SECTION,

Downsview.

Encl.

P. O. Box 500,
Sault Ste. Marie, Ontario,
June 23, 1969.

George Armstrong Co. Limited,
P. O. Box 818,
FORT FRANCES, Ontario.

Gentlemen:

WP - 7-10-64
114-64 Geocon 166

Re: Contract 68-20 - Black River Bridge
and Approaches - George Armstrong
Co., Limited.

We are writing you in relation to Crossland Construct-
ion Limited's letter of June 9, 1969, and our telephone conver-
sation of this morning.

The intent of the Contract, as you admit, is that the
gravel sheeting plus the rip-rap, be placed along the new stream
diversion along with the completion of the approach fills as in-
dicated.

We recognize that some contamination of the back slope
in the area of the piers may take place. It is agreed, there-
fore, that the rip-rap and the gravel sheeting will not be re-
quired around the piers location (about 10' away) and back to the
abutments. This work, of course, will have to be completed as
the pier work eventually permits.

Please bear in mind that there is a considerable amount
of work to be done on the approach fills and the application of
gravel sheeting and rip-rap to the stream diversion, prior to
work commencing on the piers.

Yours truly,


H. G. Potts,
District Construction Engineer,

for J. H. Blevins,
District Engineer.

HGP/ma

c.c. - Mr. H. A. Tregaskes
Mr. K. Selby
Crossland Construction Co.
Mr. F. Fancy
Mr. B. D. MacKinnon

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

November 6, 1967

W.P. 110-64 - 13.5 Miles South of Manitowadge
Southerly 1.4 Miles, Highway 614

With reference to the memorandum of October 23, 1967, from Mr. R. D. Gunter, Project Soils Supervisor to you, we wish to make the following comments:

Experience with either local materials or local construction procedures is certainly very valuable and its importance cannot be denied. Well documented case histories certainly provide by far the best parameters for future jobs where comparable conditions prevail.

Because of the above stated, we have been and always are anxious to hear any comments from persons or organizations who have been or still are active in engineering in certain areas.

Regarding the above mentioned river crossing, Mr. Gunter has, in his memorandum, disagreed with certain recommendations that were contained in our covering letter of October 26, 1966. Before going into the details of the disagreement, we would like to express our regret that Mr. Gunter's comments have been made one year after our letter was written. They were made after the Regional and Head Office contract review meetings had taken place. In this respect, we would question the purpose and usefulness of such comments.

In Mr. Gunter's opinion, the new river bank slopes will not be stable as $2\frac{1}{2}:1$, but will, rather, have to be in the order of $4:1$. Our recommendation for $2\frac{1}{2}:1$ slopes was based on two basic reasons:

(1) The slopes of the present river banks are standing up as $2:1$ or steeper. True, they show evidence of localized sloughing or small failures, but on the overall, are stable and are performing satisfactorily.

(2) The minimum factor of safety for $2\frac{1}{2}:1$ slopes computed by the consultant, and using the minimum soil parameters, was 1.3. Only surface sloughing was mentioned as a possible occurrence.

cont'd. /2 ...

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

2.

November 6, 1967

The foregoing reasons do appear to us, logical and sensible. Our further reasoning has been not to bother if some sloughing does occur in the future and if, in due course, the slopes become flatter. We have recommended that vegetation on the slopes be encouraged in order to minimize sloughing and erosion. In view of the satisfactory performance of the present natural river banks, we felt that the expenditure for providing a protective drainage blanket along the entire length of the new slopes would be unwarranted.

Regarding construction and Mr. Gunter's comment (based mainly on results of B.H.'s #10 and #11) that the soil when being excavated, may turn into liquid, we would like to comment as follows:

It would appear that most of the diversion channel excavation will be within the sand or silt layer (bottom of channel elev. 880.0). Whatever portion of the channel excavation is to be within the varved clay, it will be in a material that has an undrained shear strength of 1,000 p.s.f. or more and, therefore, has a stiff consistency.

Test results indicate the varved clay to be very sensitive. In spite of the high sensitivity, however, we feel that due to the stiff consistency, no major difficulties should be encountered during excavation, provided normal dragline or backhoe excavation procedures are used.

It should be borne in mind that the main body of the varved clay, which in certain cases may be the main controlling factor for the stability of the banks, will not be disturbed during the excavation. Only the material that is handled during the excavation operation will be partly disturbed - not the material which remains in its original position, with the possible exception of one or two inches at the surface.

Regarding Mr. Gunter's mention of the high water table (903.01), we feel that by applying the appropriate construction procedure, problems arising from such a condition can be overcome.


Under no circumstances are we trying to present this job as a simple or very easy one. However, we feel that enough factual data was assembled that enabled a reasonably thorough investigation and analysis of the whole problem. Some of the original recommendations have been modified in view of what appears to us as sound, valid reasons. Unless some more concrete arguments to the contrary are presented, we would suggest that these recommendations remain unchanged.

cont'd. /3 ...

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

November 6, 1967

In closing, we would like to emphasize that the wide distribution of the foundation reports is intended to keep informed, as many persons as practical, and to provide them with the opportunity to voice their opinion on all or any part or aspect of the report. It is our firm belief that constructive criticism and discussions are indeed necessary, and should be encouraged because they are challenging and stimulating. The ultimate outcome is bound to be a better, a more thorough, and a more competent job.



A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

ACS/KdeF

cc: Mr. R. D. Gunter

Foundations Files ✓
Gen. Files

MEMORANDUM

To: Mr. G. A. Wronz,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

From: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

DATE: November 6, 1967

Our File Ref.

IN REPLY TO

SUBJECT: W.P. 110-64 - 18.5 Miles South of Manitowadge
Southerly 1.4 Miles, Highway 614

With reference to the memorandum of October 23, 1967, from Mr. R. D. Gunter, Project Soils Supervisor to you, we wish to make the following comments:

Experience with either local materials or local construction procedures is certainly very valuable and its importance cannot be denied. Well documented case histories certainly provide by far the best parameters for future jobs where comparable conditions prevail.

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cont'd. /2 ...

April 1/69

2:35 p.m.

To: Ken

Message from K. Bassi

(Local 3516)

You are booked to fly up north -
April 8 - 1:10 p.m. from Toronto.
District personnel will pick you
up and drive to Wawa where you will
spend the night - next day you will
go to the bridge site to attend
meeting - drive back to Wawa, stay
overnight - On April 10 you will
drive to the Soo - catch 3:20 p.m.
flight there back to Toronto.

MdeF

Mr. G. A. Wreng,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

3.

November 6, 1967

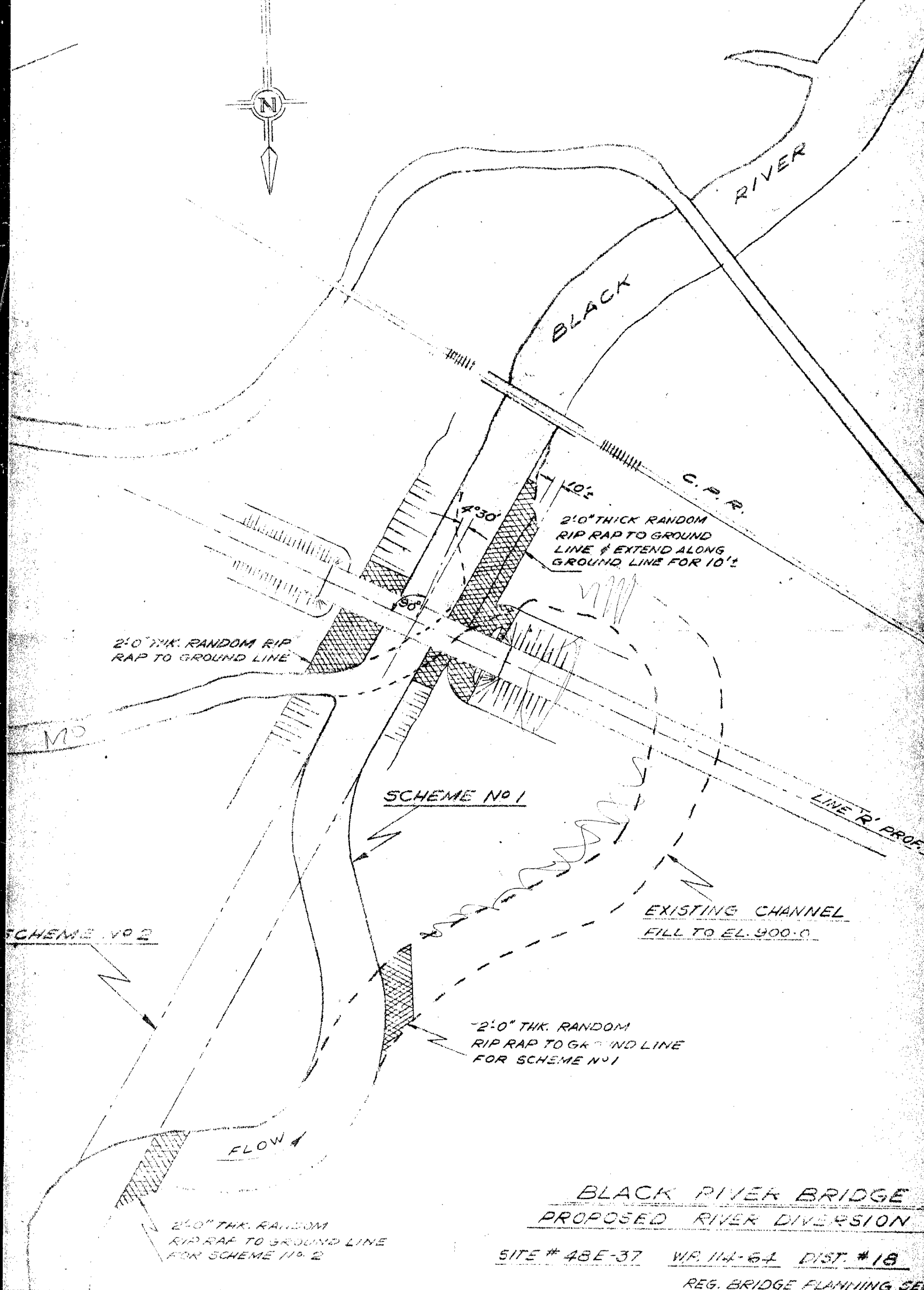
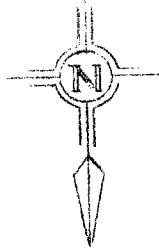
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AGS/MdeF

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

cc: Mr. R. D. Gunter

Foundations Files
Gen. Files



BLACK RIVER BRIDGE
PROPOSED RIVER DIVERSION

SITE # 48E-37 WP. 114-64 DIST. # 18

REG. BRIDGE PLANNING SEC.

MEMORANDUM

TO: Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

FROM: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

DATE: November 6, 1967

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 110-64 - 18.5 Miles South of Manitowadge
Southerly 1.4 Miles, Highway 614

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cont'd. /2 ...

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

2.

November 6, 1967

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It should be borne in mind that the main body of the varved clay, which in certain cases may be the main controlling factor for the stability of the banks, will not be disturbed during the excavation. Only the material that is handled during the excavation operation will be partly disturbed - not the material which remains in its original position, with the possible exception of one or two inches at the surface.

Regarding Mr. Gunter's mention of the high water table (~903.0⁺), we feel that by applying the appropriate construction procedure, problems arising from such a condition can be overcome.

Under no circumstances are we trying to present this job as a simple or very easy one. However, we feel that enough factual data was assembled that enabled a reasonably thorough investigation and analysis of the whole problem. Some of the original recommendations have been modified in view of what appears to us as sound, valid reasons. Unless some more concrete arguments to the contrary are presented, we would suggest that these recommendations remain unchanged.

cont'd. /3 ...

Mr. G. A. Wrong,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

2.

November 6, 1967

The foregoing reasons do appear to us, logical and sensible. Our further reasoning has been not to bother if some sloughing does occur in the future and if, in due course, the slopes become flatter. We have recommended that vegetation on the slopes be encouraged in order to minimize sloughing and erosion. In view of the satisfactory performance of the present natural river banks, we felt that the expenditure for providing a protective drainage blanket along the entire length of the new slopes would be unwarranted.

Regarding construction and Mr. Gunter's comment (based mainly on results of B.H.'s #10 and #11) that the soil when being excavated, may turn into liquid, we would like to comment as follows:

It would appear that most of the diversion channel excavation will be within the sand or silt layer (bottom of channel elev. 880.0). Whatever portion of the channel excavation is to be within the varved clay, it will be in a material that has an undrained shear strength of 1,000 p.s.f. or more and, therefore, has a stiff consistency.

Test results indicate the varved clay to be very sensitive. In spite of the high sensitivity, however, we feel that due to the stiff consistency, no major difficulties should be encountered during excavation, provided normal dragline or backhoe excavation procedures are used.

It should be borne in mind that the main body of the varved clay, which in certain cases may be the main controlling factor for the stability of the banks, will not be disturbed during the excavation. Only the material that is handled during the excavation operation will be partly disturbed - not the material which remains in its original position, with the possible exception of one or two inches at the surface.

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cont'd. /3 ...

Mr. G. A. Wreng,
Principal Soils Engineer,
Soils Section,
Rm. 134-A, Lab. Bldg.

3.

November 6, 1967

In closing, we would like to emphasize that the wide distribution of the foundation reports is intended to keep informed, as many persons as practical, and to provide them with the opportunity to voice their opinion on all or any part or aspect of the report. It is our firm belief that constructive criticism and discussions are indeed necessary, and should be encouraged because they are challenging and stimulating. The ultimate outcome is bound to be a better, a more through, and a more competent job.

AGS/MdeP

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

cc: Mr. R. D. Gunter

Foundations Files
Gen. Files

1200 blow

Francis Tancy

W.P. 114-64
CONTR. NO

Black River Division Maintenance

$$\frac{180T/pile}{3} = 60T$$

~~705~~

Sault Ste Marie 254 79 73

15 ft part

2nd vertical pile - river pier

80
110

890
110
780

1969 APR 22 AM 11:09

00130

B

SAUL DOWN 3 APR 22/69 10.15A VR

J H ELEVINS DIST ENGR

*4/22/69
W.P. 114-64*

ATTN H G POTTS DIST CONST ENGR

RE MINUTES OF JOBSITE MEETING FOR CONTRACT 68-20 BLACK RIVER BRIDGE AND APPROACHES HWY NO. 614 RE OUR TELEPHONE CONVERSATION THIS DATE FURTHER TO YOUR MEMO OF APR 11/69 I ENCLISE SUGGESTED AMENDMENTS TO POINTS 2, 6 AND 9 OF YOUR DRAFT COPY OF THE MINUTES

POINT NO. 2 MR SELBY CAREFULLY EXPLAINED THE HISTORY OF THE DESIGN OF THE STRUCTURE FROM A FOUNDATION AND STABILITY VIEW IT WAS EMPHASIZED THAT THE SEQUENCE OF CONSTRUCTION AS INDICATED IN THE SPECIAL PROVISIONS WAS THE RESULT OF A THOROUGH INVESTIGATION AND JUDGEMENT AND THAT NO REASONS FOR CHANGING THIS SEQUENCE COULD BE FORESEEN AT THIS TIME IT WAS ALSO STATED BY MR SELBY THAT A NECESSARY REQUIREMENT FOR STABILITY WAS THAT THE PRESENT BLACK RIVER CHANNEL ADJACENT TO THE THE PROPOSED EMBANKMENT MUST BE FILLED TO ELEVATION 900 PRIOR TO ANY EMBANKMENTS CONSTRUCTION TAKING PLACE IN THIS AREA

POINT NO. 6 MR SELBY QUESTIONED THE CONTRACTOR ON HOW HE WAS GOING TO EXCAVATE THE STREAM DIVERSION HE ADVISED THE CONTRACTOR THAT EXCAVATING IN LAYERS DOWN TO THE GROUND WATER LEVEL STARTING FROM THE DOWN STREAM END WOULD HAVE THE EFFECT OF OF GRADUALLY DEPRESSING THE GROUND WATER AS THE WORK PROCEEDED THUS RENDERING THE SILTY SOIL ^{MORE} WORKABLE

MR ARMSTRONG INDICATED THAT HE WOULD LIKELY BE REMOVING THE SOIL IN THIS MANNER MR FANCY POINTED OUT THAT SOME OF THE MATERIAL WILL LIKELY HAVE TO BE MOVED OR HAULED AWAY

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POINT NO.9 THE CONTRACTOR INDICATED HIS DESIRE TO SHEET IN THE PIERS
AND PUMP THE WATER OUTSIDE THE SHEETING AS WELL AS INSIDE
THE SHEETING THIS WOULD ENABLE HIM TO USE A SHORTER
LENGTH OF SHEETING SINCE IT WOULD HAVE TO PROJECT ABOVE
THE LEVEL OF THE WATER IN THE DIVERSION. MR SELBY STATED
THAT IT WOULD BE MOST UNWISE TO LOWER THE WATER OUTSIDE OF
THE SHEETING SINCE AT THIS STAGE THE APPROACH EMBANKMENT
WOULD BE BUILT AND ~~QUITE~~ PROBABLY BOILING WOULD OCCUR AT
THE EMBANKMENT TOES THUS CREATING A DANGEROUS STABILITY
PROBLEM. IN CONNECTION WITH PUMPING INSIDE THE SHEETING NO
PROBLEMS WOULD OCCUR PROVIDED THAT THE SHEETING WAS ADVANCED
FAR ENOUGH INTO THE SUBSOIL AND WAS ADEQUATELY BRACED. THIS
LATTER PROBLEM IS HOWEVER THE SPECIFIC RESPONSIBILITY OF THE
CONTRACTOR AND HE WOULD ALSO HAVE TO ENSURE THAT HIS METHOD
OF CONSTRUCTION PRECLUDED THE POSSIBILITY OF GROUND ~~HEAVE~~ ^{HEAVE}
INSIDE THE COFER DAM. AT A LATER STAGE ADVICE ON THIS ASPECT
COULD BE OBTAINED FROM THE FOUNDATION SECTION. WITH REGARD TO
THE QUESTION RAISED IN POINT NO.5 I FEEL THAT THE SPOIL
MATERIAL WILL BE SO WET THAT IT WILL NOT BE POSSIBLE TO
~~PUMP~~ ^{PUMP} IT TO A SUFFICIENT HIGHT TO CAUSE A EASE FAILURE
OF THE SUBSOIL. THE SPOIL ITSELF MIGHT POSSIBLY SLIDE
BACK INTO THE STREAM DIVERSION HOWEVER FOR THAT REASON I
SUGGEST THAT THE TOE OF THE SPOIL HEAP BE KEPT BACK ABOUT
10 FT FROM THE EDGE AND THAT THE MATERIAL BE DUMPED SO AS TO
FORM A SLOPE NOT STEEPER THAN THE ^{SIDES} SIZE OF THE NEW CHANNEL
IT SHOULD OFCOURSE BE REMOVED AS SOON POSIBLE

M G SELBY SUPVNG FOUNDATION ENGR FOR

A G STERMAC PRINCIPAL FOUNDATION ENGR

Mr. K. G. Selby,
Foundation Engineer,
DOWNSVIEW.



ONTARIO

DEPARTMENT OF HIGHWAYS

P. O. Box 500,
Sault Ste. Marie, Ontario,
April 22, 1969.

Mr. H. A. Tregaskes,
Construction Engineer,
DOWNSVIEW, Ontario.

Minutes of Job Site Meeting for
Contract 68-20 - Black River Bridge
and Approaches - Hwy. No. 614.

This job site meeting was held, as is normally done, to familiarize the Contractor and his staff with the Contract requirements and also to meet the Regional and District Staff with whom he may be dealing. It is basically a meeting to be an exchange of ideas along with questions and answers.

ATTENDANCE

Mr. George Armstrong	Contractor - Geo. Armstrong Co., Ltd.
Mr. Sid. Ross	Supervisor - Geo. Armstrong Co., Ltd.
Mr. S. S. Seydegare	Crossland Construction Ltd.
Mr. Lyle Armstrong	George Armstrong Co., Ltd.
Mr. George R. Tilly	Giffels Associates
Mr. A. C. Cripps	Bridge Construction Liaison Engineer
Mr. K. G. Selby	Foundation Engineer - Downsview
Mr. D. Gunter	Regional Materials Supervisor - Ft. Wm.
Mr. H. W. Hurrell	Reg. Road Design Superintendent - Ft. Wm.
Mr. G. Jordan	Engineering Audit Supervisor - Ft. Wm.
Mr. H. G. Potts	District Construction Engineer
Mr. J. E. Hunter	Project Supervisor
Mr. B. D. MacKinnon	Project Supervisor
Mr. F. Fancy	Construction Supervisor
Mr. H. Beemer	Hwy. Construction Inspector

The meeting was convened at 11:30 A.M., April 9, 1969. There was a short prior job site review for those individuals not recently familiar with the construction site. The following points are the ones discussed in the main, although others were of minor interest and were not noted:

1. The Contractor indicated that he would prefer not to follow the sequence of construction as is indicated in the Special Provisions but would prefer to partially excavate the stream diversion at the structure to enable work on the pier footings and piers.
2. Mr. Selby carefully explained the history of the design of the

Continued-----

structure from the foundation and stability view. It was emphasized that the sequence of construction as indicated in the Special Provisions was the result of a thorough investigation and judgment and that no reasons for changing this sequence could be foreseen at this time. It was also stated by Mr. Selby that a necessary requirement for stability was that the present Black River channel adjacent to the proposed embankment must be filled to elevation 900, prior to any embankments construction taking place in this area.

3. The Contractor, but mainly Crossland Construction (proposed Sub-Contractor) indicated a concern about being able to complete the structure this year. He posed the question that the completion of the structure would depend upon the Department's desire to see the Contract completed this year. Various points of the work were discussed, however, Mr. Armstrong indicated that the fabrication of the concrete beams was beginning Monday, April 14, 1969, and the piling operations would be relatively continuous. The remaining work and order of completion was in some doubt, however, it appeared that the deck might not be completed this year.

I indicated that the Contract was binding and that 125 Working Days were chargeable from May 15, 1969. This might or might not permit carryover of work, depending on weather and temperature according to Specifications. This would relate mainly to concrete work. I could not see any importance other than adhering to the Contract.

4. There was thorough discussion on the use of excavated materials. The material from the road cut areas will likely be wasted and earth borrow will be required for fill purposes including the earth fill in the existing channel of the Black River.

5. Mr. Gunter was questioning the distance that waste material could be safely piled back from the new stream diversion without causing a slide or failure of the embankment.

Mr. Selby indicated that he would investigate this point and give a reply to the District for further action. (See No. 9 for reply.)

6. Mr. Selby questioned the Contractor on how he was going to excavate the stream diversion. He advised the Contractor that excavating in layers down to the ground water level starting from the down stream end would have the effect of gradually depressing the ground water as the work proceeded thus rendering the silty soil more workable. Mr. Armstrong indicated that he would likely be removing the soil in this manner. Mr. Fancy pointed out that some of the material will likely have to be moved or hauled away.

7. The Contractor indicated that he will need access to the area between the Black River and Mobert Creek. He realized that this would be at his own cost and would likely cross Mobert Creek by means of culverts or timber crossing. This would also include the cleanup and removal of the temporary structure, including silt, waste material, etc.

Continued-----

8. It was pointed out by Mr. Selby and Mr. Gunter, as well as the Department men present, that the rip rap around the approach fills and the gravel sheeting in the stream diversion would need to be completed as soon as the stream diversion was carrying water.

9. The Contractor indicated his desire to sheet in the piers and pump the water outside the sheeting as well as inside the sheeting. This would enable him to use a shorter length of sheeting. Since it would be most unwise to lower the water outside of the sheeting since at this stage the approach embankment would be built and quite probably boiling would occur at the embankment toes, thus creating a dangerous stability problem in connection with pumping inside the sheeting. No problems would occur provided that the sheeting was advanced far enough into the subsoil and was adequately braced. This latter problem is, however, the specific responsibility of the Contractor and he would also have to ensure that his method of construction precluded the possibility of ground heave inside the cofferdam. At a later stage advice on this aspect could be obtained from the Foundation Section with regard to the question raised in Point No. 5.

Mr. Selby feels that the spoil material will be so wet that it will not be possible to pile it to a sufficient height to cause a base failure of the subsoil. The spoil itself might possibly slide back into the stream diversion, however, for that reason Mr. Selby suggests that the toe of the spoil heap be kept back about 10 feet from the edge and that the material be dumped so as to form a slope not steeper than the sides of the new channel; it should, of course, be removed as soon as possible.

10. The point was brought up once more about the importance to the Department that this work be completed this year and reference was made to the Contract conditions and working days as noted in Item #3, above.

The District did point out that the Contract was awarded last fall and that the Contractor could have carried out his Clearing and some of the stream diversion prior to winter setting in very heavily.

11. Mr. Selby asked Mr. Armstrong how long he thought it would take to excavate the stream diversion. The reply was approximately 6 weeks from about May 1, 1969.

12. Mr. Armstrong and Mr. Seydegare enquired about payment for the fabrication of the concrete beams. It was indicated that the beams would be fabricated in the very near future, however, he did not wish to have them shipped until required. The beams are being fabricated by B & B Stone Limited of Port Arthur, Ontario, who is not an approved Sub-Contractor at this date nor is there a request for this. Mr. Armstrong wished to leave the beams at the point of fabrication, however, receive a partial payment in the meantime. He has to submit this request in writing and he indicated that one other District had obtained permission for this.

13. Pits, Aggregates Sources, Etc. A short discussion took place regarding possible sources of concrete aggregates, however, we do not

have detailed knowledge at this time. It was understood that Materials and Testing Section had looked into this subject last fall and it was suggested to the Contractor that he do further investigation with the District Staff and the Regional office to obtain more up to date information.

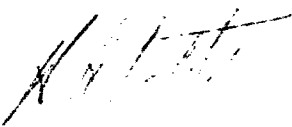
14. The Contractor's representative on the job will be Mr. Sid. Ross, at the beginning, and when any change takes place the authorized representative will be named in writing.

15. Black River Stream Bed. More questions came up regarding the need for excavation or displacement of some soft organic material in this stream bed. It is noted for displacement on the profile; however, Mr. Gunter agreed to check to see that there is, in actual fact, muck or any material to be excavated or displaced.

16. Mr. Fancy reminded the Contractor that licensed vehicles only could be used on the existing Black River Bridge and that no overloading would be permitted.

The Meeting adjourned at 2:00 P.M.

HGP/ma



H. G. Potts,
District Construction Engineer,
18 - Sault Ste. Marie.

c.c. - Mr. George Armstrong (4)
Mr. George R. Tilly
Mr. A. C. Cripps
Mr. K. G. Selby
Mr. D. Gunter
Mr. H. W. Hurrell
Mr. G. Jordan
Mr. J. E. Hunter
Mr. B. D. MacKinnon
Mr. F. Fancy
Mr. H. Beemer

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. J. H. Blevins,
District Engineer,
District #18 - Sault Ste. Marie.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. H. G. Potts,
District Construction Engr.

DATE: April 17, 1969

OUR FILE REF:

IN REPLY TO

SUBJECT:

Minutes of Job Site Meeting for
Contract 68-20 - Black River Bridge
and Approaches - Hwy. No. 614.

Further to your memo of April 11, 1969, I enclose suggested amendments to Points 2, 6, and 9 of your draft copy of the minutes:

2. Mr. Selby carefully explained the history of the design of the structure from the foundation and stability view. It was emphasized that the sequence of construction, as indicated in the Special Provisions, was the result of a thorough investigation and judgment, and that no reasons for changing this sequence could be foreseen at this time. It was also stated by Mr. Selby that a necessary requirement for stability was that the present Black River channel adjacent to the proposed embankment, must be filled to el. 900 prior to any embankment construction taking place in this area.
6. Mr. Selby questioned the Contractor on how he was going to excavate the stream diversion. He advised the Contractor that excavating in layers down to the groundwater level, starting from the downstream end, would have the effect of gradually depressing the groundwater as the work proceeded, thus rendering the silty soil more workable. Mr. Armstrong indicated that he would likely be removing the soil in this manner. Mr. Fancy pointed out that some of the material will likely have to be moved or hauled away.
9. The Contractor indicated his desire to sheet in the piers and pump the water outside of the sheeting as well as inside the sheeting. This would enable him to use a shorter length of sheeting, since it would have to project above the level of the water in the diversion. Mr. Selby stated that it would be most unwise to lower the water outside of the sheeting since, at this stage, the approach embankments would be built and quite probably, boiling would occur at the embankment toes, thus

Mr. J. E. Blevins, District Engr.,
Dist. #18 - Sault Ste. Marie,
Attn: Mr. H. G. Potts, Construction Engr.

2

April 17, 1969

Contract 68-20 - Black River Bridge and Approaches - (cont'd.) ...

9. (cont'd.) ...

creating a dangerous stability problem. In connection with pumping inside the sheeting, no problems would occur, provided that the sheeting was advanced far enough into the subsoil and was adequately braced. This latter problem is, however, the specific responsibility of the Contractor, and he would also have to ensure that his method of construction precluded the possibility of ground heave inside the cofferdam. At a later stage, advice on this aspect could be obtained from the Foundation Section.

With regard to the question raised in Point No. 5, I feel that the spoil material will be so wet that it will not be possible to pile it to a sufficient height to cause a base failure of the subsoil. The spoil itself, might possibly slide back into the stream diversion, however. For that reason, I suggest that the toe of the spoil heap be kept back about 10 feet from the edge, and that the material be dumped so as to form a slope not steeper than the sides of the new channel. It should, of course, be removed as soon as possible.

KGB/MdeF

cc: Foundations Files ✓
Gen. Files

K. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
DOWNSVIEW.

FROM: Mr. H. G. Potts,
District Construction Engineer,
18 - Sault Ste. Marie.

ATTENTION: Mr. K. G. Selby,
Foundation Engineer.

DATE: April 11, 1969.

OUR FILE REF.

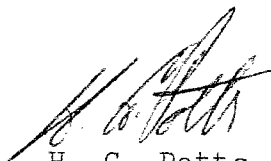
IN REPLY TO

SUBJECT:

Minutes of Job Site Meeting for
Contract 68-20 - Black River Bridge
and Approaches - Hwy. No. 614.

Further to our meeting of April 9, 1969, I enclose one draft copy of the minutes of this Job Site meeting.

As agreed with yourself and Mr. A. C. Cripps, would you please review these notes and suggest and make any clarifications and corrections which you find necessary. Upon receipt of your comments I will then prepare the completed set of minutes for distribution for all people who were in attendance at the meeting.



H. G. Potts,
District Construction Engineer,

for J. H. Blevins,
District Engineer.

HGP/ma

Encl.

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18 - Sault Ste. Marie,
April 11, 1969.

Minutes of Job Site Meeting for
Contract 68-20 - Black River Bridge
and Approaches - Hwy. No. 614.

This job site meeting was held, as is normally done, to familiarize the Contractor and his staff with the Contract requirements and also to meet the Regional and District Staff with whom he may be dealing. It is basically a meeting to be an exchange of ideas along with questions and answers.

ATTENDANCE

Mr. George Armstrong	Contractor - Geo. Armstrong Co., Ltd.
Mr. Sid. Ross	Supervisor - Geo. Armstrong Co., Ltd.
Mr. S. Seydegare	Crossland Construction Ltd.
Mr. Lyle Armstrong	George Armstrong Co., Ltd.
Mr. George R. Tilly	Giffels Associates
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Mr. K. G. Selby	Foundation Engineer - Downsview
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Mr. G. Jordan	Engineering Audit Supervisor - Ft. Wm.
Mr. H. G. Potts	District Construction Engineer
Mr. J. E. Hunter	Project Supervisor
Mr. B. D. MacKinnon	Project Supervisor
Mr. F. Fancy	Construction Supervisor
Mr. H. Beemer	Hwy. Construction Inspector

The meeting was convened at 11:30 A.M., April 9, 1969. There was a short prior job site review for those individuals not recently familiar with the construction site. The following points are the ones discussed in the main although others were of minor interest and were not noted:

1. The Contractor indicated that he would prefer not to follow the sequence of construction as is indicated in the Special Provisions but would prefer to partially excavate the stream diversion at the structure to enable work on the pier footings and piers.
2. Mr. Selby carefully explained the history of the design of the structure from the foundation and stability view. It was emphasized that the sequence of construction as indicated in the Special Provisions was the result of a thorough investigation and judgment.
3. The Contractor, but mainly Crossland Construction (Proposed Sub-Contractor) indicated a concern about being able to complete the structure this year. He posed the question that the completion of the structure would depend upon the Department's desire to see the Contract completed this year. Various points of the work were discussed, however, Mr. Armstrong indicated that the fabrication of the concrete beams was beginning Monday, April 14, 1969, and the piling operations would be relatively continuous. The remaining work and order of completion was in some doubt, however, it appeared that the deck might not be completed this year.

I indicated that the Contract was binding and that 125 Working Days were chargeable from May 15, 1969. This might or might not

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Continued-----

Re: Contract 68-20.

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Mr. Selby indicated that he would investigate this point and give a reply to the District for further action.

6. Mr. Selby questioned the Contractor on how he was going to excavate the stream diversion and Mr. Armstrong indicated that he would likely be removing it in layers down to water level. Mr. Fancy pointed out that some of the material will likely have to be moved or hauled away.

7. The Contractor indicated that he will need access to the area between the Black River and Mobert Creek. He realized that this would be at his own cost and would likely cross Mobert Creek by means of culverts or timber crossing. This would also include the cleanup and removal of the temporary structure, including silt, waste material, etc.

8. It was pointed out by Mr. Selby and Mr. Gunter, as well as the Department men present, that the rip rap around the approach fills and the gravel sheeting in the stream diversion would need to be completed as soon as the stream diversion was carrying water.

9. The Contractor indicated his desire to sheet in the piers and pump the water out to enable an earlier start and ease of construction. The question came up about the possible effect that the pumping of the water might have on the soil conditions. Careful analysis of the soil data available indicated that there would not likely be a problem because the clay at the elevation of the footing appeared to be stable. It was stressed that this was an opinion based on our best knowledge available and if soil conditions were disturbed around the pier area the Contractor would have to resort to other means of control of the water level and construction.

10. The point was brought up once more about the importance to the Department that this work be completed this year and reference was made to the Contract conditions and working days as noted in Item #3, above.

The District did point out that the Contract was awarded last fall and that the Contractor could have carried out his Clearing and some of the stream diversion prior to winter setting in very heavily.

11. Mr. Selby asked Mr. Armstrong how long he thought it would take to excavate the stream diversion. The reply was approximately 6 weeks from about May 1, 1969.

Continued-----

Continued-----

Re: Contract 68-20.

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13. Pits, Aggregates Sources, Etc. A short discussion took place regarding possible sources of concrete aggregates, however, we do not have detailed knowledge at this time. It was understood that Materials and Testing Section had looked into this subject last fall and it was suggested to the Contractor that he do further investigation with the District staff and the Regional office to obtain more up to date information.

14. The Contractor's representative on the job will be Mr. Sid Ross, at the beginning, and when any change takes place the authorized representative will be named in writing.

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16. Mr. Fancy reminded the Contractor that licensed vehicles only could be used on the existing Black River Bridge and that no overloading would be permitted.

The meeting adjourned at 2:00 P.M.

HGP/ma



H. G. Potts,
District Construction Engineer.