

68-F-58

W.P. 397-65

HWY. # 401

AIRPORT RD.

MEMORANDUM

To: Mr. B. E. Davis,
Bridge Engineer,
Bridge Division,
Admin. Bldg.

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

Attention: Mr. S. McCombie

DATE: November 12, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

PILE LOAD TESTS

On

Concrete Caissons at Hwy. #401
And Airport Road
District No. 6 (Toronto)

W.J. 68-P-58 -- W.P. 397-65

Enclosed, please find our detailed report on the
caisson load tests which were carried out at the above
mentioned site.

In addition to the factual information, the report
also contains recommendations for similar installations
which may be used in the foundations of Bridges #26 and #27.

Should additional information be required, or
should any points in the report require further clarification,
please feel free to contact this Office.

AGS/mieF

Attach.

cc: Messrs. B. E. Davis (2)
H. A. Tregaskes
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A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

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PILE LOAD TESTS
On
Concrete Caissons at Hwy. #401
And Airport Road
District No. 6 (Toronto)
W.J. 68-P-58 -- W.P. 397-65

1. INTRODUCTION:

In a memo dated June 24, 1968, Mr. C. S. Grebski, Bridge Design Engineer, suggested that load tests be implemented on concrete caissons at the location of the proposed bridges Nos. 26 and 27 of the Hwy. #401 and Airport Expressway interchange.

It was postulated that considerable savings could be realized by supporting the bridge footings on concrete caissons instead of the originally contemplated steel H-piles, due to the unusually large design loads. Moreover, the subsoil conditions were considered to be well suited for the installation of caissons since, beneath a shallow overburden, shale bedrock was found at a depth of 16 ft. below ground level. It was decided that caissons be installed some 3 - 6 ft. below the bedrock surface, and the frictional and end-bearing properties of the sound shale be tested. To this end, a scheme was devised by this Section involving the construction of three concrete caissons, one to test the frictional, one the end bearing, and the third the combination of both characteristics of the bedrock.

No attempt was made to test the adhesion between the overburden and the caisson shaft because the value of such adhesion is negligible as compared with the strength mobilized within the bedrock. The adhesion within the overburden might be considered, however, as an additional safety factor, when estimating the bearing capacity of the caisson. The portion of the shaft of each caisson embedded within the overburden was poured inside a 24" O.D. steel liner, which in turn, was lowered inside a liner of 36" O.D. The annular space was filled with

cont'd. /2 ...

1. INTRODUCTION: (cont'd.) ...

loose sand, so that practically no friction was mobilized along the wall of the liners. The portion of the shaft of the friction pile within the bedrock was poured without any liner, thus friction between bedrock and concrete could not be measured. To eliminate end bearing of this caisson, styrofoam pads were placed at the bottom of the hole prior to the pouring of the concrete. The styrofoam pad was 3" thick; it was cut oversize and 6 layers were forced into place. The end-bearing and friction caisson was similarly constructed, but no styrofoam was employed. The concrete was poured directly onto the prepared bedrock surface.

Both liners of the end-bearing caisson were lowered to the bottom of the hole, some 4 - 5 ft. below rock surface, consequently no friction developed between rock and concrete. The concrete again was poured onto the bottom of the hole, the annular space being filled with sand.

For each caisson the load was applied on the 24" liner and the top of the concrete shaft, which was carefully ground to a smooth horizontal surface. The outside 36" diameter liner was assumed to be unaffected by the loads.

A contract for the construction of the caissons, together with the anchorage and reaction beam system, was awarded to Western Caissons Ltd., Maple, Ontario, under W.O. No. 68-32106. Construction commenced on the 10th of September 1968, and the load tests were completed by the 23rd of October 1968. The construction was supervised and the load tests performed by personnel of this Section.

This report contains a description of soil conditions, the construction of the caissons, the results of the load tests, and recommendations as to the safe pressures on such caissons.

cont'd. /3 ...

2. DESCRIPTION OF SITE:

The site is situated immediately north of Hwy. #401 and west of the Airport Road ramp N.W. in the Twp. of Etobicoke. The area belongs to the Department of Transport, being the South-east corner of the Toronto International Airport. The test site is flat and, during the larger part of the year, the surface is wet and soggy due to poor drainage.

Geologically, the terrain belongs to the physiographic region known as the "South Slope" consisting of ground moraines with irregular knolls and hollows.

3. SUBSOIL CONDITIONS:

3.1) General:

Some four boreholes, numbered 1 to 4 inclusive, were carried out adjacent to the test site, prior to the construction of the caissons. The borings revealed an approx. 16-ft. deep glacial till overburden followed by shale bedrock. A detailed description of these strata follows:

3.2) Clayey Silt with some Sand and Gravel:

From ground elevation down to 16.0 - 16.8 ft. depth (El. 498.7' - 497.8') a heterogeneous layer of clayey silt with sand and gravel (glacial till) was found in each borehole. A stratified structure, consisting of 1/8" - 1/16" thick seams of sand intercepting the clayey silt, was noticed within the upper 8 ft. of the layer. Otherwise, the overburden was found to be unsorted, indicating that the material was deposited by glacial ice. Standard penetration 'N' values varied between 10 and over 100 blows/ft., corresponding to consistencies of stiff to hard. Atterberg limit tests, performed on each soil sample, resulted in plastic limits of 12 - 18% and liquid limits of 18 - 34%. The mean values of plastic and liquid limits may be taken to be 15% and 30%, respectively. The natural moisture contents were observed

cont'd. /4...

3. SUBSOIL CONDITIONS: (cont'd.) ...

3.2) Clayey Silt with some Sand and Gravel:

to be either below the plastic limit or just a small percentage above it, confirming the overconsolidated nature of this glacial drift. The results of a few density tests on 'undisturbed' samples indicated bulk densities between 146 PCF and 125 PCF. The average bulk density might be around 130 PCF.

3.3) Bedrock:

Bedrock was encountered in each borehole, as mentioned earlier. Laboratory unconfined compression tests were performed on cylindrical rock core samples of 2-3/32" diameter and 4" height. The tests resulted in compressive strength values, ranging from 1800 PSI to 2600 PSI. Representative stress-strain curves of the tests are appended (Fig. #8) to the end of this report.

A description of the bedrock by Mr. K. Ingham, D.H.O. Geologist, is as follows:

"The drill cores examined are shales typical of the Dundas formation of Ordovician age and most probably belong to the Credit-Member, although there is insufficient faunal evidence to establish this with certainty.

"The rock is medium to dark bluish gray shale which weathers light gray at the top of each section. In general the shale is only slightly calcareous but contains calcareous bands 0.1 to 0.2 feet in thickness which constitute approximately 10% of the upper 10 feet. Small calcareous pellets or concretions are present throughout. The upper 0.5 to 0.7 five feet is weathered and there is some incipient weathering down as far as five feet. The rock is platy bedded with the exception of the upper weathered portion which shows some fissility.

"Shale of this type is generally quite competent and I suspect that your values of 2000 psi for the unconfined compressive strength are certainly not high."

3. SUBSOIL CONDITIONS: (cont'd.) ...

3.4) Groundwater Conditions:

The water level in the borings was established around el. 509.5 ft., some 5 ft. below ground surface. Within a week or so, the water level in the holes rose to el. 513.5 ft., just about one ft. below ground level. Similar observations were made during the drilling of the caissons.

Laboratory and field test results are marked on the attached borelogs. The soil stratigraphy, based on the borelogs, may be seen on Drawing 68-F-58A.

4. INSTALLATION OF THE DRILLED CAISSONS:

4.1) Installation of the test and anchor piles commenced on the 10th of September and was completed on September 13, 1968. Sketches of the three test caissons and one of the anchor caissons, together with the sequence of installation, may be seen on Figures #1, 2, 3 and 4 at the end of the report. The general arrangement of the test and anchor caissons are shown on Drawing 68-F-58A.

The concrete, forming the caissons, was a high early strength type. The average 7-day compressive strength of Caisson #1 was 4450 PSI, Caisson #2 - 4760 PSI, and Caisson #3 - 4380 PSI.

Drilling of the holes was performed by a Hugh Williams earth and rock boring machine. The overburden was observed to be quite dry down to about 5 - 6 ft. depth, below which it became wet. Water seepage and some surface sloughing was apparent during the drilling operation. By installing the liners below the rock surface, a large portion of the seepage was eliminated. Rifling the bedrock wall was attempted by using a special tooth attached to the auger. Since the operation was somewhat cumbersome and the end result rather poor, the procedure was not further pursued.

cont'd. /6 ...

4. INSTALLATION OF THE DRILLED CAISSONS: (cont'd.) ...

4.1) - (cont'd.) ...

The inner liner, surrounding the concrete shaft, had an inside diameter of 23" and wall thickness of 1/2". The purpose of this liner was to eliminate skin friction along the perimeter of the caisson within the liner. The space between the inside liner and the 36" diameter outside liner was filled with sand, in order to prevent any lateral displacement of the caisson during load application.

4.2) Time Required for Installation of a Test Caisson:

For estimating purposes the approximate time spent for the construction of a caisson is given as follows:

- (a) Drilling 16 ft. of overburden -- approx. 10 - 15 minutes.
- (b) Drilling 4 - 5 ft. of bedrock, installation of 36" Ø liner and cleaning hole -- 2.5 - 3.0 hours.
- (c) Installation of 24" liner, placing reinforcing cage, pouring concrete -- approx. 2 hours.
- (d) Completion of one test caisson -- approx. 5 hours.

5. LOAD TESTING PROCEDURE:

The load testing commenced on the 7th of October 1968 on test Caisson #3, followed by testing Caisson #2, and finally Caisson #1. A heavy section reaction beam was welded to the two H-beams of the anchor piles with special stirrups. The reaction beam consisted of an H-beam 33" high with 16" wide flanges and 1" web thickness. The H-beam was reinforced all around with welded steel plates of 2" wall thickness, forming a closed box-like section. Loads were applied to the test caisson by means of a hydraulic jack, placed between the reaction beam and the caisson to be tested. The jack was electrically operated and had a maximum capacity of 7500 PSI, the equivalent of 500 tons on the contact area.

cont'd. /7 ...

5. LOAD TESTING PROCEDURE: (cont'd.) ...

Steel bearing plates of 2" thickness were placed on the top of the caissons to ensure a smooth and hard bearing surface between jack and caisson. Vertical deflections of the test caissons were recorded by means of four dial gauges with divisions of 0.001". The stands of the gauges were welded on the four corners of the bearing plate, while the gauges sat on the independently supported reference beams. Each reference beam was comprised of 3" steel angles welded together to form a square section. The two reference beams were approx. 20 ft. long, and they were supported by spikes driven to a minimum depth of 2 ft. below ground level. Some 10 ft. distance was maintained between test caisson and reference beam supports so that no ground movements caused by the loading, could have any influence on the reference beams. As an additional precaution, frequent elevations were taken on the reference beams and the anchor piles during the tests. The true deflection of the caisson was taken to be the mean of the four deflections, recorded by the dial gauges. After the completion of one test the loading arrangement was dismantled and rebuilt for the next caisson to be tested.

The loading tests were performed by applying the loads in increments, beginning with 10 tons. Loads were maintained until the rate of settlement decreased to a value of 0.01" per hour, or for 2 hours - whichever was the shorter period. From the initial load, loads were increased to 25 Tons and 50 Tons, after which 50-Ton increments were added up to 500 Tons. The final load was kept on the caisson for an average of 24 hours, and then the load was decreased to 50%, 25%, 10% and 0% of the final load. Settlement readings were taken in such intervals as the rate of settlements warranted, ranging from 5 minutes to 1/2 hour.

The loading tests are shown graphically on Figures #5, 6 and 7. For each test three graphs were computed: a) Load versus Time, b) Time versus Settlement, and c) Load versus settlements.

cont'd. /8 ...

5. LOAD TESTING PROCEDURE: (cont'd.) ...

The following Table shows the dates of installation and load testing of the individual caissons:

No. of Caisson	Date of Installation	Date of Test
1	13 Sept./68	22 - 23 Oct./68
2	12 " "	15 - 17 " "
3	12 " "	7 - 8 " "

6. EVALUATION OF THE TESTS:

No failure was reached by any of the tests, indicating that both the end-bearing and frictional properties of the tested bedrock exceeded the maximum applied load of 500 Tons/caisson.

Total settlements of the purely frictional and the purely end-bearing caissons were measured to be approximately the same, averaging about 1 inch, under the maximum applied load. Roughly half of the total settlement was elastic, as shown by the rebound curves. The caisson for testing the combined resistances by friction and end bearing settled 0.55 inch, more than half of which was elastic.

The actual maximum unit load applied at the base of the end-bearing caisson, during the field loading test, was computed to be 159 TSF - i.e., 2200 PSI. The value of adhesion or frictional resistance mobilized between concrete shaft and rock surface under the 500-ton load was 23.8 TSF, around 330 PSI.

As was mentioned earlier, the caisson, testing the combined effect of end bearing and skin friction of the bedrock, settled approx. 1/2 inch, which was roughly half of the settlement of the purely end bearing or purely frictional caisson. This would imply

cont'd. /9 ...

6. EVALUATION OF THE TESTS: (cont'd.) ...

that both end bearing as well as the frictional resistances of the bedrock were mobilized under the applied load.

A summary of the applied loads and the induced settlements are tabulated below:

Description of Caisson	Duration of Loading Test	Duration of Final load of 500 Tons	Settlements (Inch)	
			Gross	Net
1) Friction Caisson	29 hours	15 hours	1.02	0.58
2) End-bearing "	49 "	36 "	1.09	0.43
3) End-bearing and Friction Caisson	33 "	24 "	0.54	0.21

For design purposes, based on the loading tests, it may be concluded that within the area investigated the sound shale bedrock will support safe pressures of 1100 PSI by end bearing, and will mobilize frictional resistances of 160 PSI around the perimeter of a drilled, reinforced concrete caisson shaft.

Settlements under the suggested safe loads are anticipated to be less than 0.5 inch.

7. MISCELLANEOUS:

The loading tests were performed by Messrs. P. Payer and A. Prakash, Project Foundation Engineers, under the supervision of Mr. A. K. Barsvary, Senior Foundation Engineer, who also wrote this report. Mr. K. G. Selby, Supervising Foundation Engineer, reviewed the report.

November 1968.

APPENDIX I.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 68-F-58 LOCATION 867,810 N; 976,313 E. ORIGINATED BY AP
 W. P. 396, 397-65 BORING DATE July 3-4, 1968 COMPILED BY AP
 DATUM Geodetic BOREHOLE TYPE Auger & Diamond Core CHECKED BY _____

SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— L PLASTIC LIMIT ——— P WATER CONTENT ——— W	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SHEAR STRENGTH P S F	WATER CONTENT % 10 20 30	P.C.F.	
514.7	Ground Level							
0.0	Clayey silt with sand and varying amount of gravel. Thin seams of sand within top 8 ft. (Glacial Till)		1	SS	24			513.8 July 10/68
			2	SS	16			
			3	SS	13			
			4	TW	PH			
			5	TW	PH			
			6	TW	PH			
			7	TW	PH			
			8	TW	PH			
			9	TW	PH			
			10	TW	PH			
			11	TW	PH			
			12	TW	PH			
	Stiff to hard		13	SS	45			509.5 July 3/68
			14	SS	76			
498.7			15	SS	167/8"			
16.0	Shale Bedrock		16	RC	90% NXL Rec.			
			17	RC	100% NXL Rec.			
484.7			18	RC	85% NXL Rec.			
30.0	End of Borehole							

[illegible]

FOUNDATION SECTION

CHECKED BY _____

SOIL PROFILE		STRAT. PLT	SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — W _L		REMARKS
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	PLASTIC LIMIT — W _P	WATER CONTENT — W	W _P	
514.5	Ground Level										
0.0	Clayey silt with sand and varying amount of gravel. Thin seams of sand within top 8 ft. (Glacial Till) Stiff to hard					510					
498.1			1	SS	100% Rec						
16.1			2	RC BXL	90% Rec						
	Shale Bedrock		3	RC BXL	70% Rec						
			4	RC BXL	100% Rec						
			5	RC BXL	90% Rec						
482.3											
32.2	End of Borehole					480					

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

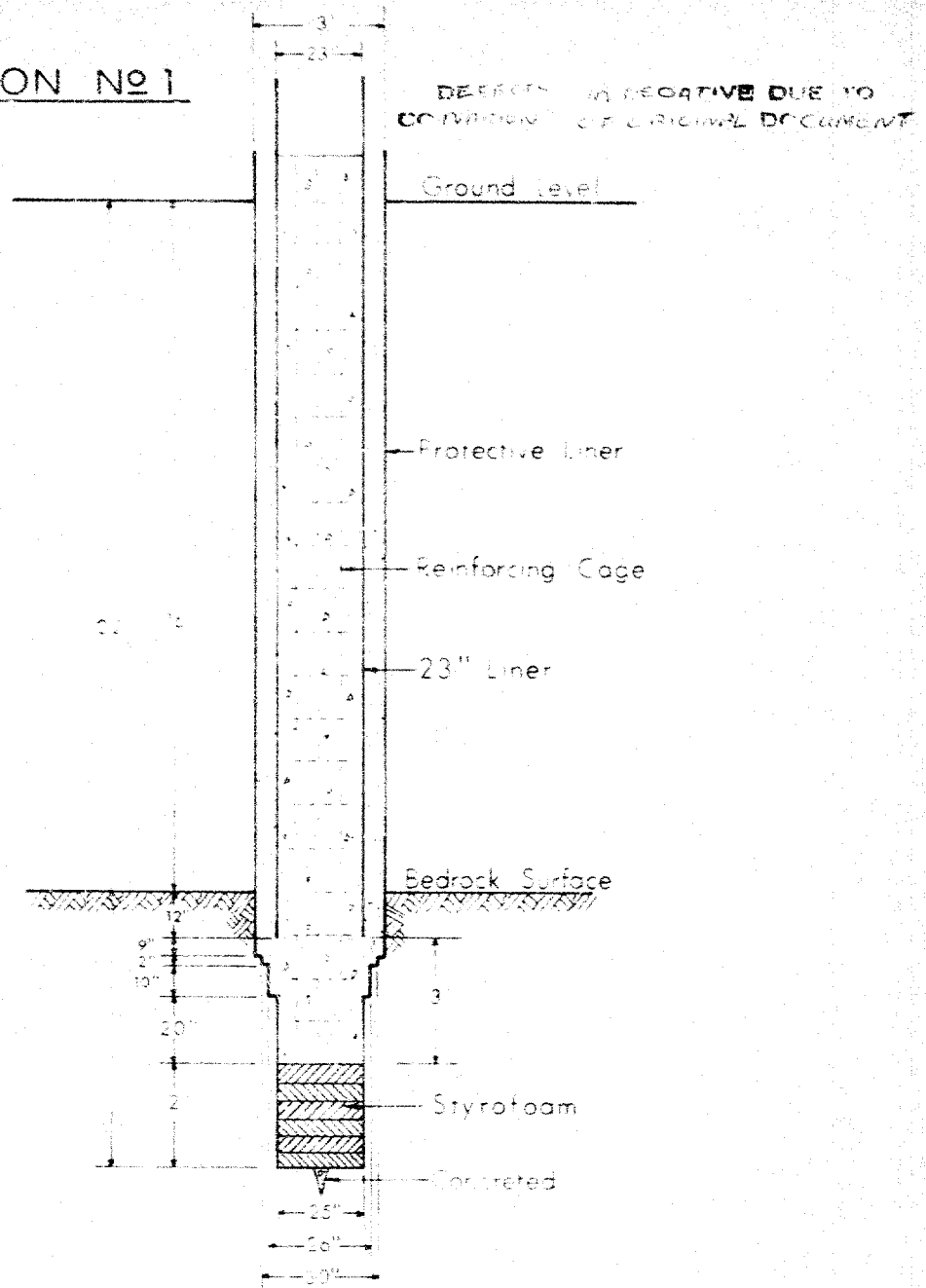
MATERIALS & TESTING DIVISION

JOB 68-F- 53 LOCATION 867.848 N; 976.314 E. ORIGINATED BY AP
W.P. 396, 397-65 BORING DATE July 8-10, 1968 COMPILED BY AP
DATUM Geodetic BOREHOLE TYPE Auger & Diamond Core CHECKED BY

SOIL PROFILE		STRAT. PLLOT	SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT ——— % PLASTIC LIMIT ——— % WATER CONTENT ——— %		BULK DENSITY pcf	REMARKS
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	W.P.	W.L.		
514.6	Ground Level										
0.0	Clayey silt with sand and varying amount of gravel. Thin seams of sand within top 3 ft. (Glacial Till) Stiff to hard.										
497.8			1	SS							
16.0			2	SS 100% BXL Rec							
	Shale Bedrock		3	RC 100% BXL Rec							
			4	RC 95% BXL Rec							
			5	RC 90% BXL Rec							
482.4			6	RC 95% BXL Rec							
32.2	End of Borehole										

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

TEST CAISSON No 1

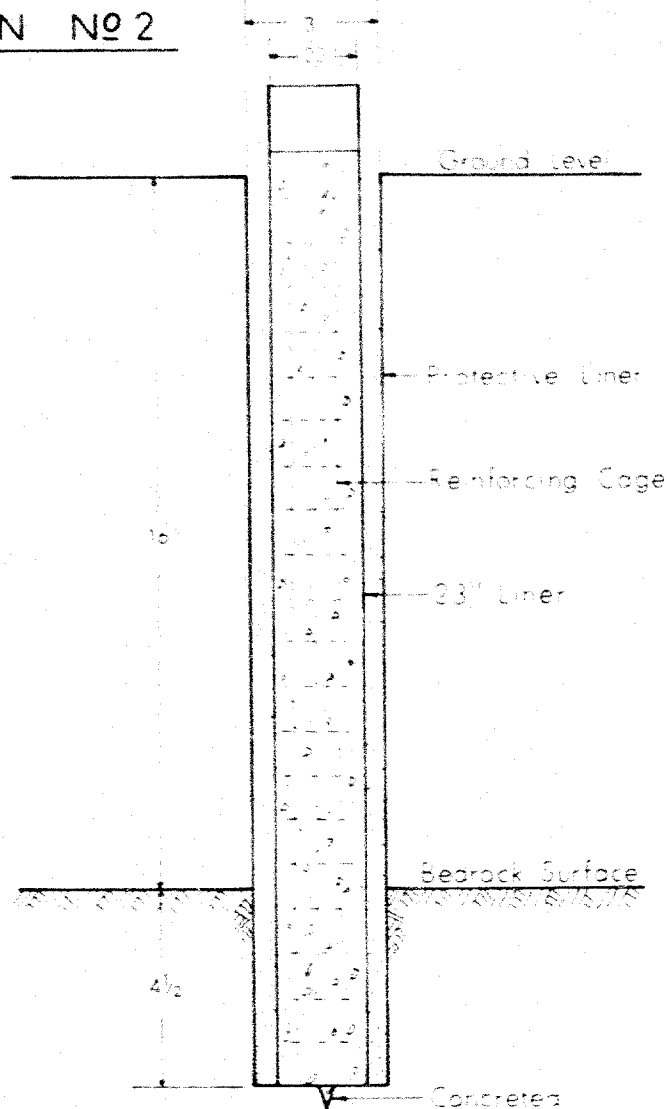


SEQUENCE OF INSTALLATION

- 1) Drilled 36" dia hole to a depth of 17.5', some 1.5' below bedrock surface.
- 2) Installed 36" dia protective liner.
- 3) Cleared hole with 30" dia auger.
- 4) Drilled down to 22' below ground level, 6' within bedrock with 25" dia auger.
- 5) Cleaned hole manually & placed 6 layers of styrofoam pad to bottom of hole.
- 6) Filled bottom 3' of hole, above styrofoam with concrete.
- 7) Installed 23" I.D. liner on top of concrete.
- 8) Pushed reinforcing steel cage into concrete.
- 9) Placed concrete & sand backfill between liners simultaneously, ensuring that the 23" I.D. liner remained in place.

FIG. 1

TEST CAISSON No 2



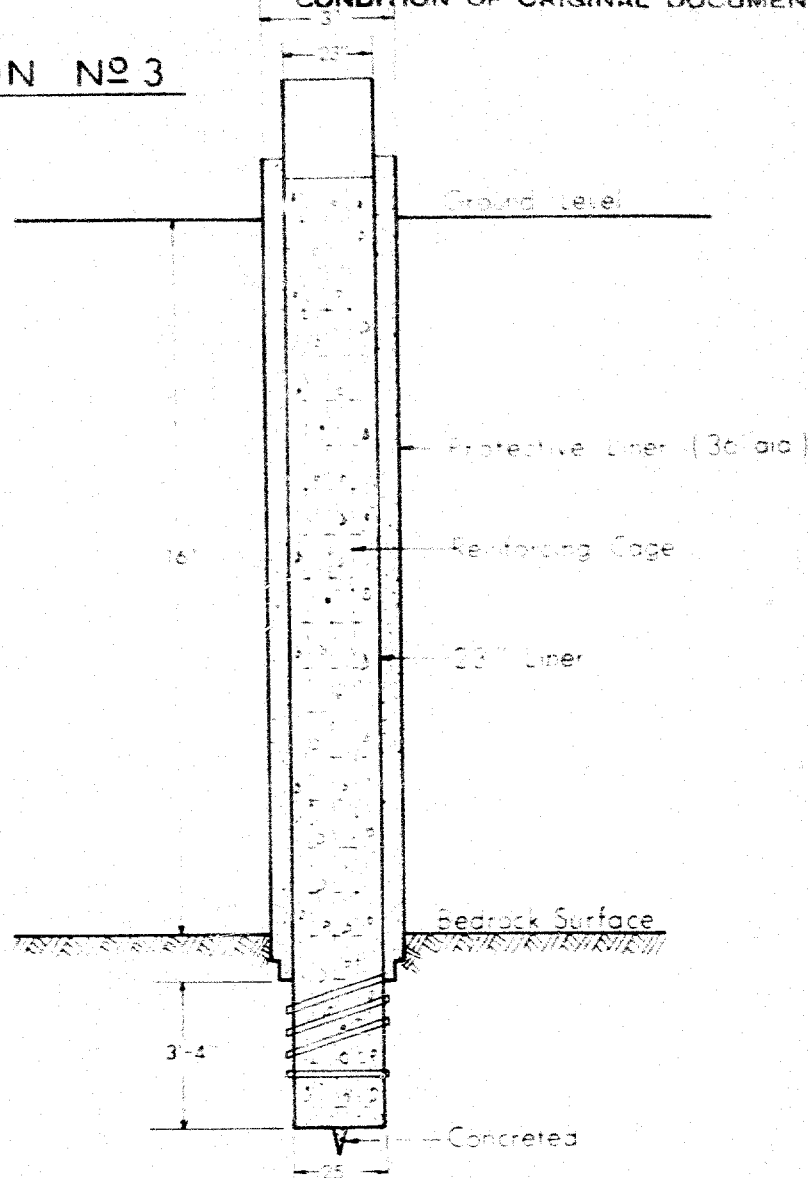
SEQUENCE OF INSTALLATION:

- 1) Drilled 36" dia hole down to 33' 4" some 4' below bedrock surface
- 2) Installed 36" dia protective liner
- 3) Inspected & cleaned bottom of hole
- 4) Pumped out collected water from bottom
- 5) Installed 23" I.D. liner & dropped a few bucketfulls of concrete
- 6) Installed reinforcing steel cage
- 7) Placed concrete & sand backfill between liners simultaneously

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FIG. 2

TEST CAISSON N° 3

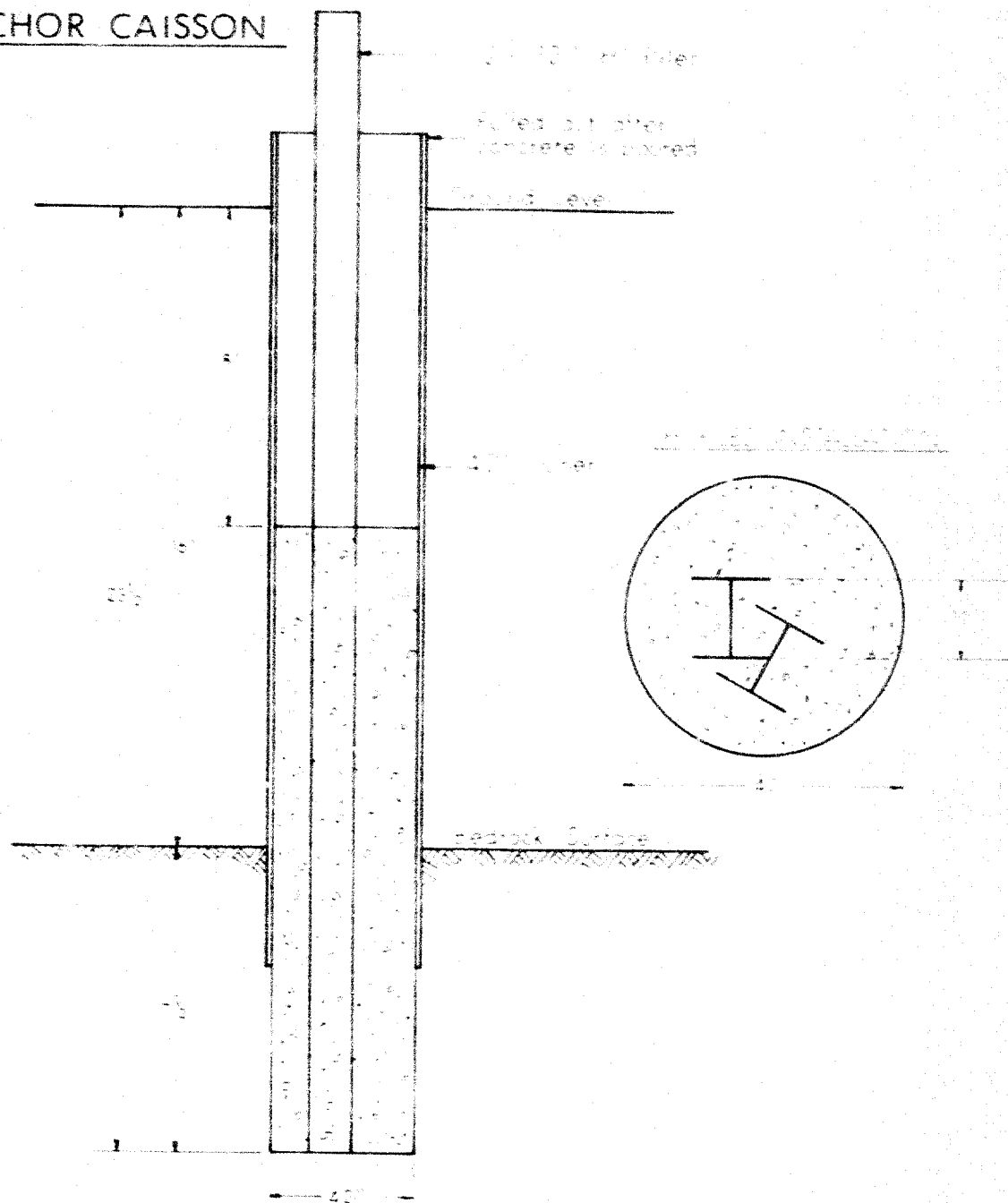


SEQUENCE OF INSTALLATION

- 1) Drilled 36" dia hole down to 16.5', some 0.5' below rock surface
- 2) Installed 36" dia liner, ensuring good seal at bottom by applying a few blows on top
- 3) Cleared hole with 30" dia auger approx. 5" below bottom of 36" dia liner
- 4) Drilled rock with 25" dia auger to a depth of 20'-4"
- 5) Tried to rifle rock wall with limited success
- 6) Inspected & cleaned hole manually
- 7) Filled bottom 3' of hole with concrete
- 8) Lowered 23" ID liner down to top of concrete & installed reinforcing steel cage
- 9) Placed concrete & sand backfill between liners simultaneously

FIG 3

ANCHOR CAISSON



SEQUENCE OF INSTALLATION

1. Drilled hole with 42" dia auger down to 23.5' (some 7.5' below rock surface)
2. Lowered 42" liner down to 2' below rock surface
3. Inspected & cleaned hole manually
4. Poured a few buckets of concrete to bottom of hole
5. Installed 3-12" steel "H" piles in the hole as shown on sketch
6. Filled hole with concrete up to about 8' beneath ground level
7. Withdrew 42" dia liner

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FIG 4

CAISSON No 1

RESULTS OF LOAD TEST

HWY. 401 & AIRPORT ROAD

W.O. 68-32106 W.P. 397-65 JOB. 68-F-58.

PILE TYPE : 24" dia cast in place reinforced
concrete caisson

MODE OF SUPPORT : Friction in bedrock contact only.

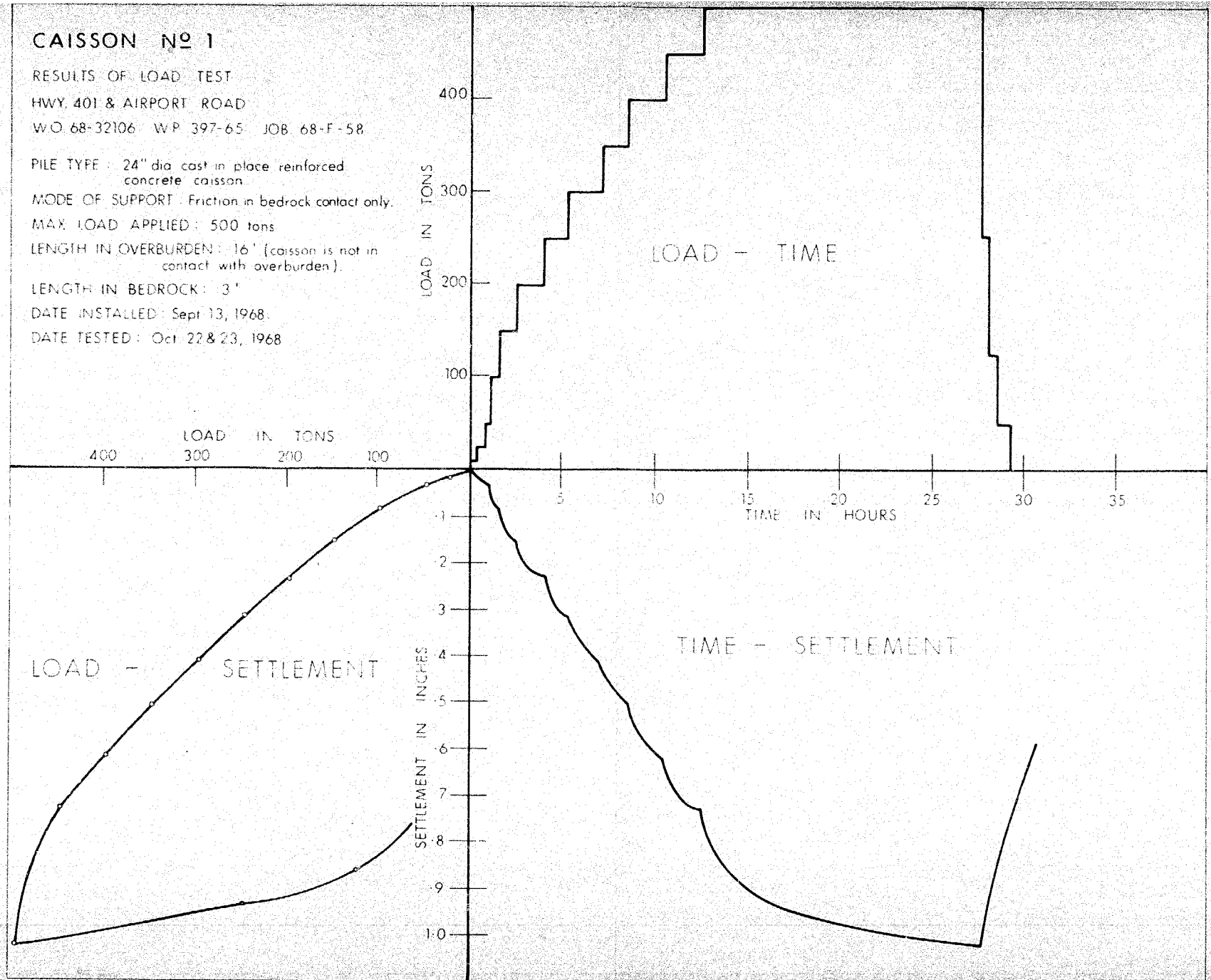
MAX. LOAD APPLIED : 500 tons

LENGTH IN OVERBURDEN : 16' (caisson is not in
contact with overburden).

LENGTH IN BEDROCK : 3'

DATE INSTALLED : Sept 13, 1968.

DATE TESTED : Oct 22 & 23, 1968



CAISSON NO 2

RESULTS OF LOAD TEST

HWY. 401 & AIRPORT ROAD

W.O. 68-32706 W.P. 397-65 JOB 68-F-58

PILE TYPE: 24" dia. cast in place reinforced concrete caisson

MODE OF SUPPORT: End bearing in bedrock

MAX. LOAD APPLIED: 500 tons

LENGTH IN OVERBURDEN: 16' (caisson is not in contact with overburden)

LENGTH IN BEDROCK: 4.5' (caisson is not in contact with bedrock wall)

DATE INSTALLED: Sept. 12, 1968

DATE TESTED: Oct. 15-17, 1968



CAISSON No 3

RESULTS OF LOAD TEST

HWY 401 & AIRPORT ROAD

W.D. 68 32106 W.P. 397-65 JOB 68-F-58

PILE TYPE : 24" dia cast in place reinforced concrete caisson.

MODE OF SUPPORT : End bearing & friction in bedrock contact

MAX LOAD APPLIED : 500 tons.

LENGTH IN OVERBURDEN : 16' (caisson is not in contact with overburden).

LENGTH IN BEDROCK : 3-4'

DATE INSTALLED : Sept 12, 1968

DATE TESTED : Oct. 7 & 8, 1968

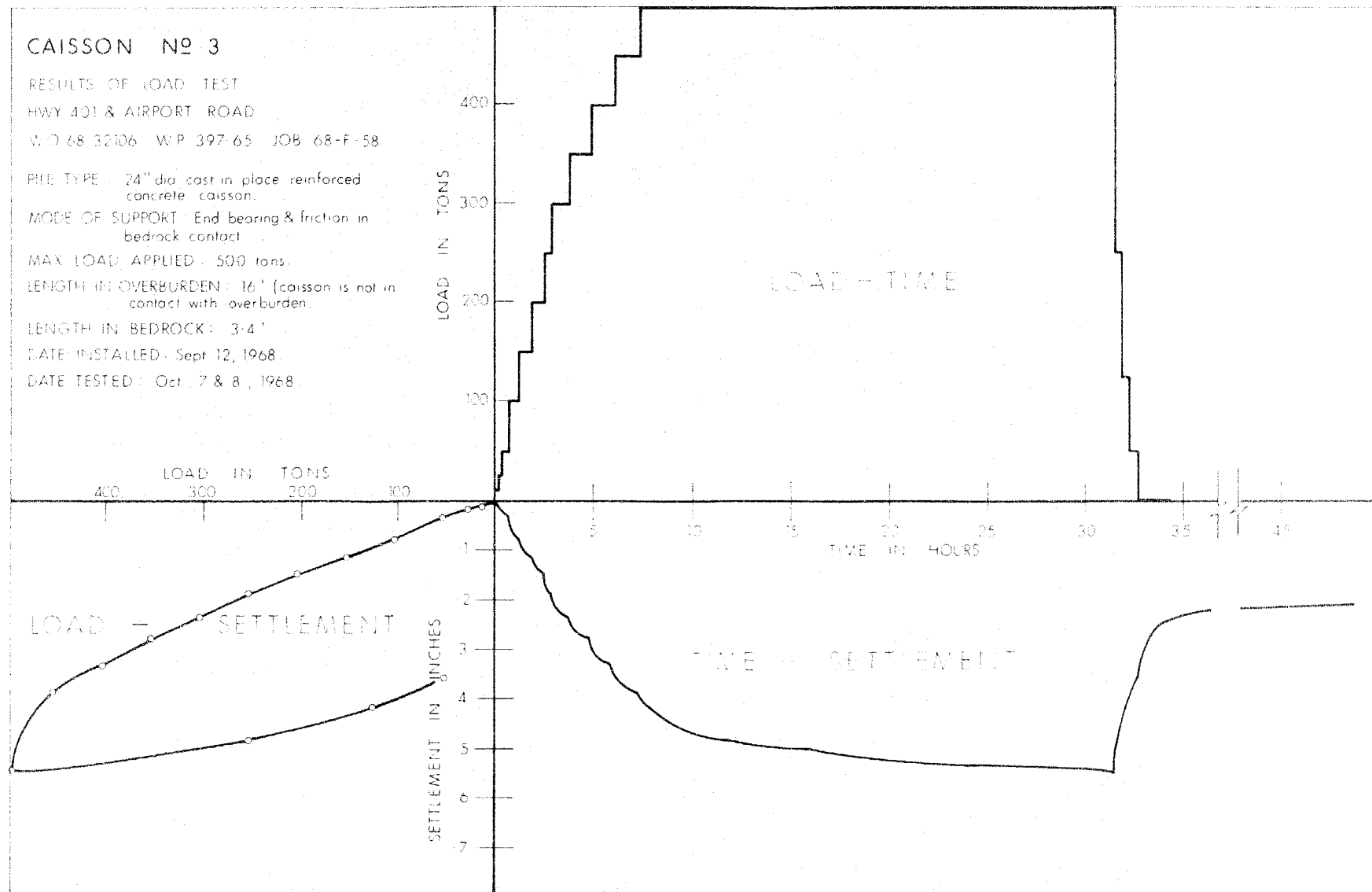
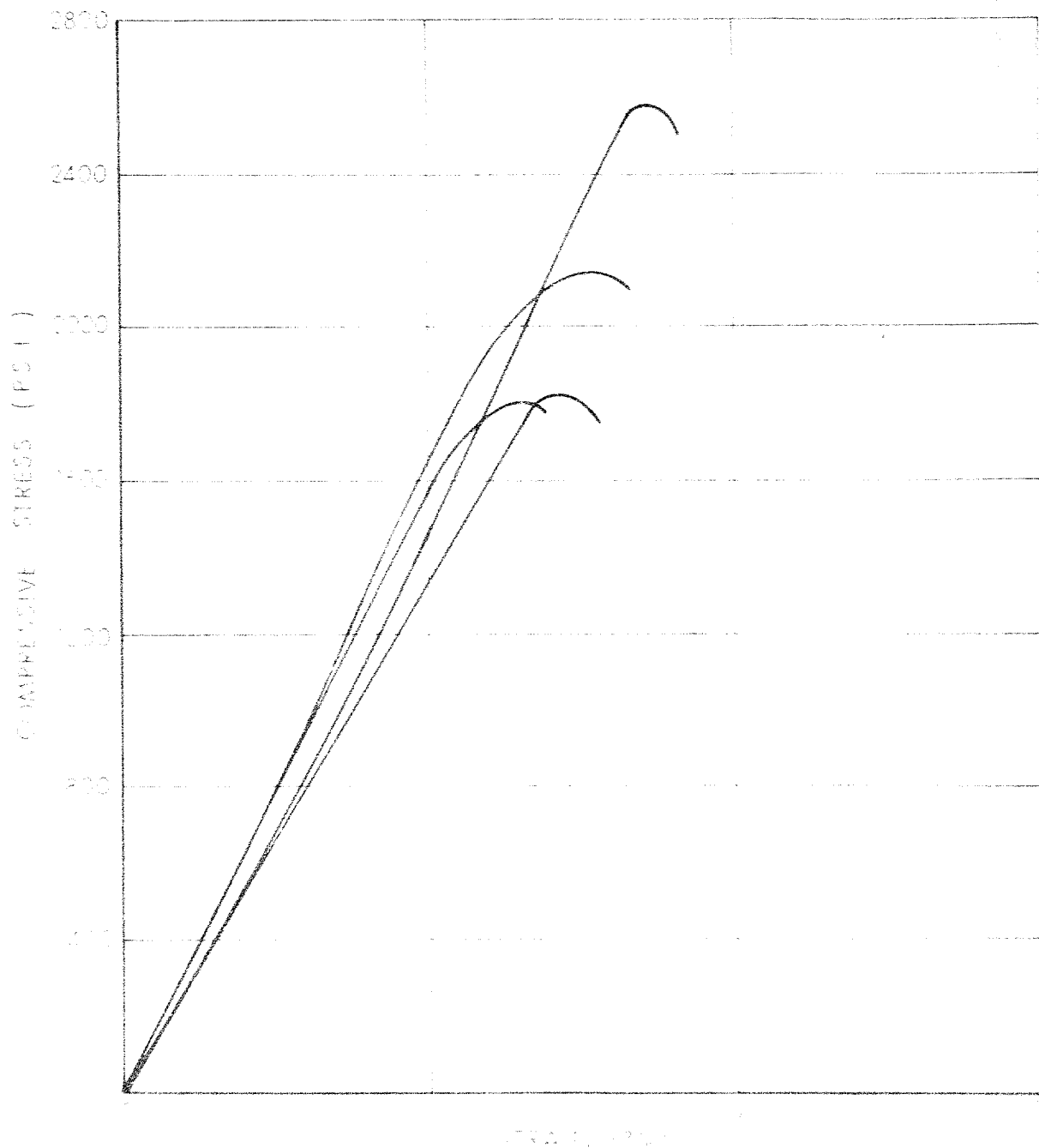


FIG. 7

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

STRESS STRAIN CURVES OF THE UNCONFINED COMPRESSION TESTS ON ROCK SAMPLES



REPRODUCED IN NEGATIVE FROM XERO
COPYING OF ORIGINAL DOCUMENT

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE ('N') - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES

DYNAMIC PENETRATION RESISTANCE (f) - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

SS	SPLIT SPOON	TW	THINWALL OPEN
WS	WASHED SAMPLE	TP	THINWALL PISTON
SB	SCRAPER BUCKET SAMPLE	OS	OSTERBERG SAMPLE
AS	AUGER SAMPLE	FS	FOIL SAMPLE
CS	CHUNK SAMPLE	RC	ROCK CORE
ST	SLOTTED TUBE SAMPLE		
	PH		SAMPLE ADVANCED HYDRAULICALLY
	PM		SAMPLE ADVANCED MANUALLY

SOIL TESTS

QU	UNCONFINED COMPRESSION	LV	LABORATORY VANE
U	UNDRAINED TRIAXIAL	FV	FIELD VANE
QCU	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
QU	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
Q	RATE OF DISCHARGE
V	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
C_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{C_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ	SHEAR STRENGTH
c'	EFFECTIVE COHESION
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE OR FRICTION
μ	COEFFICIENT OF FRICTION
S	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

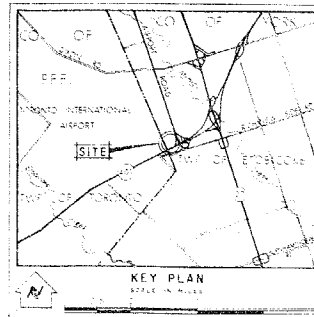
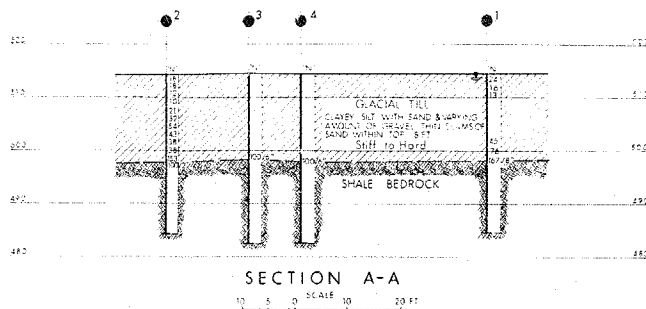
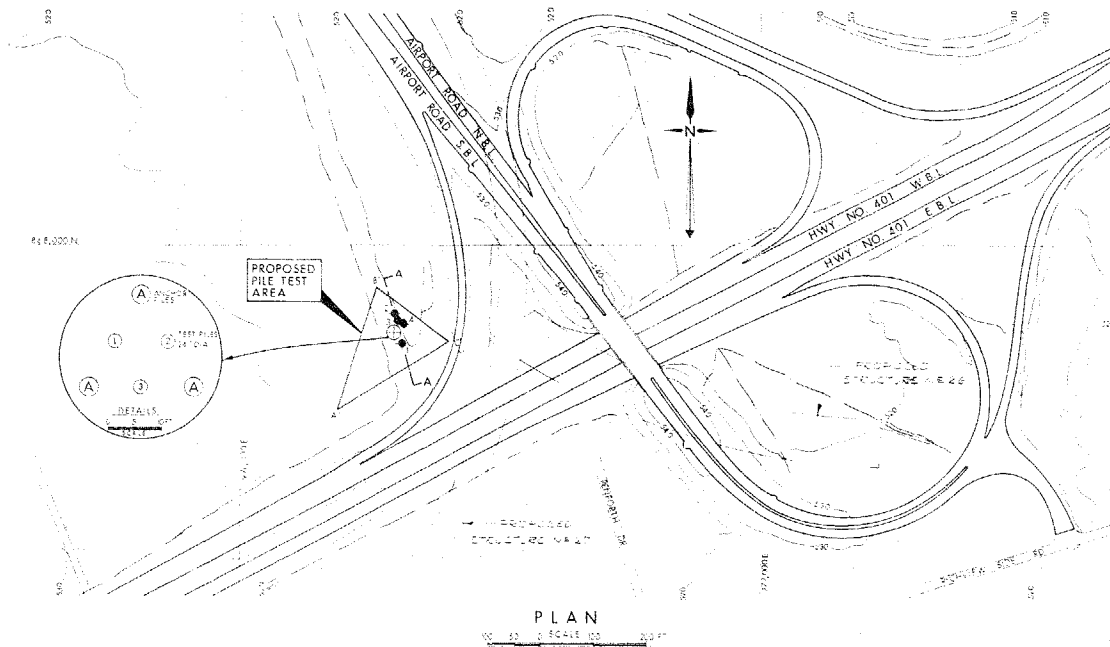
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



LEGEND			
	Bore hole		
	Core Penetration Hole		
	Bore & Core Penetration hole		
	Water Levels established at time of field investigation		

NO.	ELEVATION	TEST PILE	TEST PILE
1	514.7	8' x 8" DIA.	1' x 1' DIA.
2	514.4	8' x 8" DIA.	1' x 1' DIA.
3	514.2	8' x 8" DIA.	1' x 1' DIA.
4	513.6	8' x 8" DIA.	1' x 1' DIA.

NOTE
The boundaries between soil strata have been established only on Bore hole locations. Between Bore holes the soil strata are assumed from geological evidence and may be subject to considerable error.

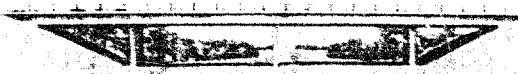
DATE	BY	REVISION

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & TESTING DIVISION - ROYALTON, ONTARIO			
PILE LOADING TEST ON			
CAST IN PLACE CONCRETE CAISSON'S			
KING'S HIGHWAY NO. 401 & AIRPORT ROAD		DIST. NO. 0	
CO. YORK		LOT	
TWP. ETOBICOKE		CON.	
BORE HOLE & TEST PILE LOCATIONS			
SUBMIT A-B CHECKED	W.P. NO. 307-25	W.P. NO. 307-25	
DRAWN 3-C CHECKED	JOB NO. 08-F-58	68-F-58A	
DATE 31 OCT 1968	31-1-1	31-1-1	
APPROVED <i>[Signature]</i>	PROJECT NO.	PROJECT NO.	

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



*file
also*



Hwy. 401 & Keele St.,
Downsview, Ontario.

Tel. 243-3282

(Area Code 416)

DEPARTMENT OF HIGHWAYS
Materials and Testing Division

July 15, 1968

Frankl (Canada) Ltd.,
105 Nantucket Blvd.,
Scarborough, Ontario.

Attention: Mr. A. Prior

68-F-58

Dear Sirs:

Please supply us with a firm lump sum quotation for carrying out the work described on the attached sheets, together with a time schedule showing the date when you would be in a position to start the work. A sketch outlining your proposals for a suitable reaction beam should also be included.

It is to be understood that the work will be performed under the technical supervision of the D.H.O. Foundation Section and must conform to the provisions contained on the attached sheets and to the relevant D.H.O. Standards and Specifications.

It should be noted that no materials or equipment of any kind will be supplied by the Department.

Your quotation must be submitted to this Office on Wednesday, July 24, 1968, before 12:00 Noon.

Yours very truly,

K. G. Selby

KGS/MdeF
Attach.

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



Hwy. 401 & Keele St.,
Downsview, Ontario.

Tel. 248-3282

(Area Code 416)

DEPARTMENT OF HIGHWAYS
Materials and Testing Division

July 15, 1968

Western Caissons Ltd.,
46 Creditstone Rd.,
Maple, Ontario.

Attention: Mr. P. Kozicki

Dear Sirs:

Please supply us with a firm lump sum quotation for carrying out the work described on the attached sheets, together with a time schedule showing the date when you would be in a position to start the work. A sketch outlining your proposals for a suitable reaction beam should also be included.

It is to be understood that the work will be performed under the technical supervision of the D.H.O. Foundation Section and must conform to the provisions contained on the attached sheets and to the relevant D.H.O. Standards and Specifications.

It should be noted that no materials or equipment of any kind will be supplied by the Department.

Your quotation must be submitted to this Office on

Yours very truly,

K. G. Selby

KGS/MdeF
Attach.

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

-- LOAD TESTS ON CONCRETE CAISSONS --
Highways 401 & 27 Interchange, District #6 (Toronto)
W.P. 397-65 -- Site No. 37-821 -- W.J. 68-F-58

1. GENERAL DESCRIPTION OF WORK:

The work to be performed by the Contractor, consists of installing three 24-inch diameter concrete caissons and subsequently carrying out loading tests up to a maximum of 500 tons on each caisson. All materials, equipment and personnel necessary for the work must be provided by the Contractor, and the work will be performed as directed by the D.H.O. Foundation Section.

2. CONCRETE CAISSONS:

The test caissons will be of nominal diameter 24 inches, and may be formed with their bases up to 6 ft. below the shale bedrock surface as directed by the Department. The concrete in the caissons must have a 7-day compressive strength of 5000 p.s.i. The details of, and the installation procedures for each caisson, are shown on Drawings 68-F-58C, 68-F-58D, and 68-F-58E. It will be the complete responsibility of the Contractor to ensure that the test caissons are installed in such a way as to permit no lateral movements during the load tests: if such lateral movements do occur, the Contractor must carry out all necessary remedial measures at his own expense.

3. LOAD TESTS:

The load tests will be carried out, in general, according to the National Building Code of Canada and, specifically, as directed by the D.H.O. Foundation Section. The Contractor must provide all necessary personnel, equipment and materials to make adjustments during the load tests, and must have at least one skilled workman present for the complete duration of each test.

4. WORKING AREA:

The Contractor must provide a level, dry working area at the site of the test caissons and must provide, also, an enclosure sufficient to afford complete protection from adverse weather conditions.

Load Tests on Concrete Caissons

July 15, 1968

5. DRAWINGS:

The following drawings are provided for the Contractor's information:

- 68-F-58 A Site Location and Layout of Test Caissons.
 B Subsoil Stratigraphy
 C Test Caisson #1 - Installation Details.
 D " " #2 - " "
 E " " #3 - " "

On award of the Contract and prior to commencement of the work, the Contractor must submit a complete set of working Drawings to the Department, together with a time schedule.

6. MATERIALS:

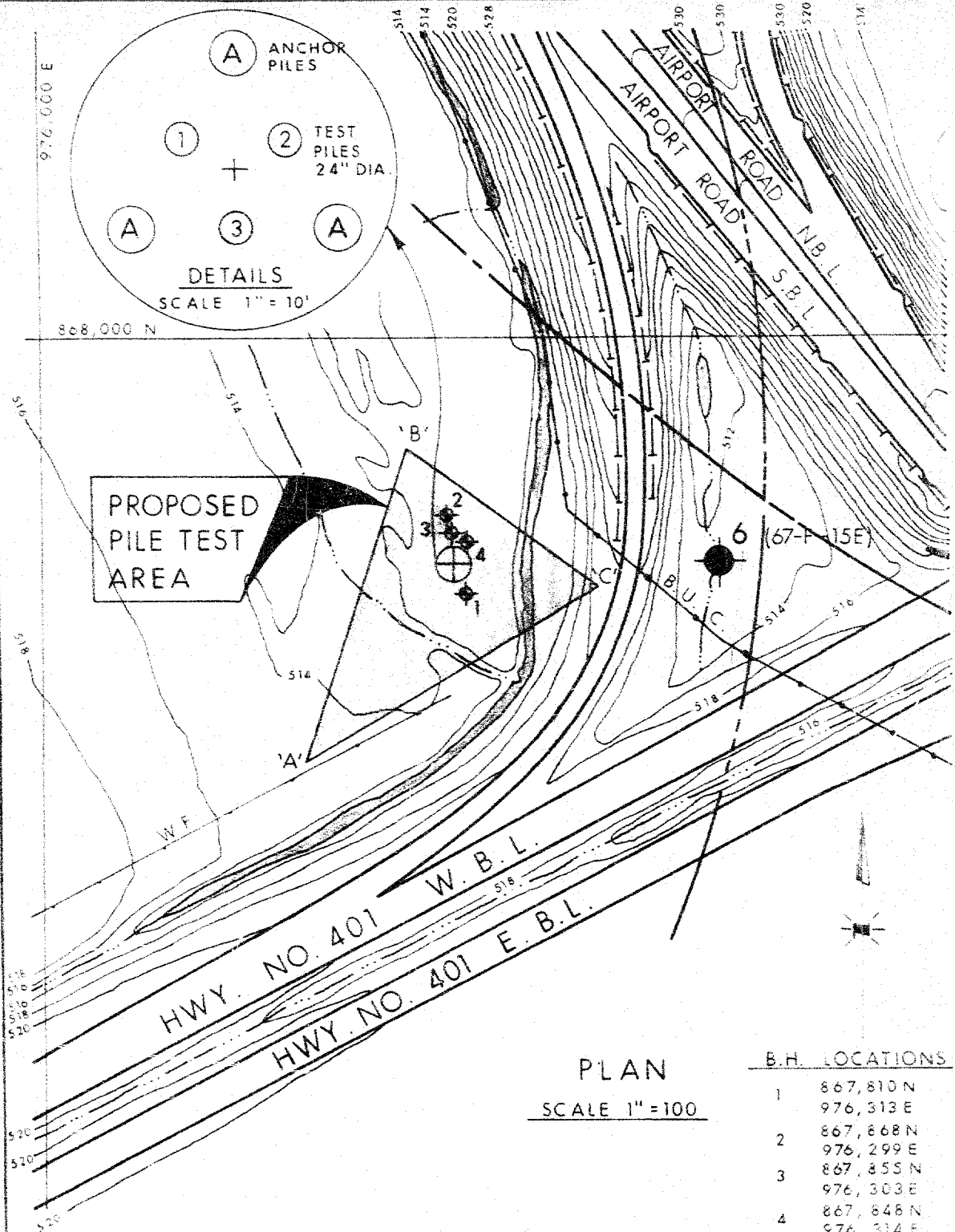
No materials or equipment will be supplied by the Department. All materials and equipment supplied by the Contractor must conform to the relevant D.H.O. Standards and Specifications.

7. ACCESS TO SITE:

The Contractor must carry out all temporary work necessary to obtain access to the site for his materials, equipment and personnel.

8. CLEARING THE SITE:

At completion of the work, the Contractor must clear the site to the satisfaction of the D.H.O. and restore it to the original condition insofar as grades are concerned. All test caissons and anchor caissons must be cut off at elev. 508 and the resulting voids backfilled with suitable fill material.



ONTARIO

DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

HIGHWAY 401 & AIRPORT ROAD

PILE TEST LOCATION

W.P.

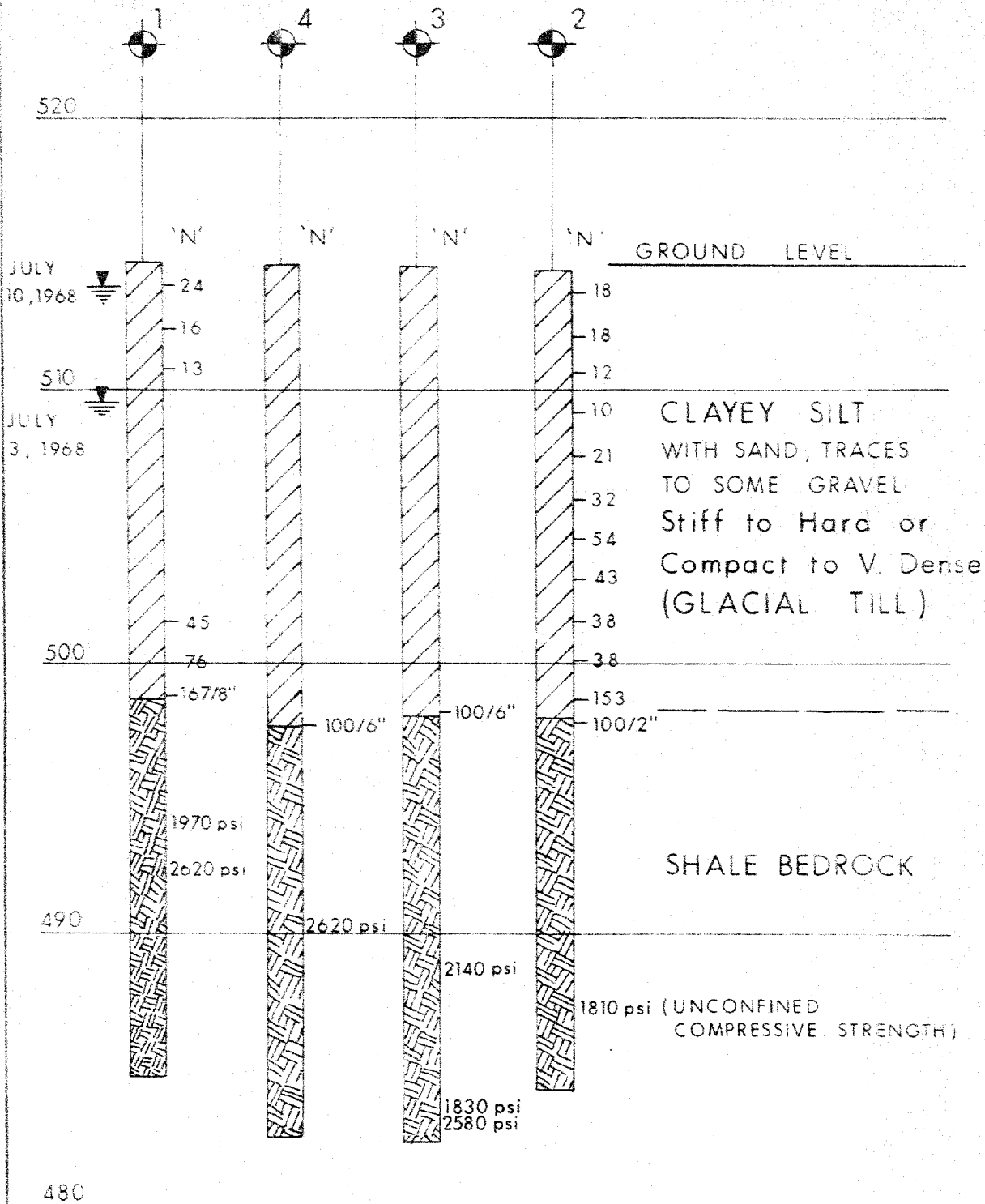
DISTRICT 6

JOB 68-F-58

DATE JULY 12, 1968

APPROVED

DRAWING NO. 68-F-58A



ONTARIO

DEPARTMENT OF HIGHWAYS
**MATERIALS and
 TESTING
 DIVISION**

HIGHWAY 401 & AIRPORT ROAD SUB-SOIL STRATIGRAPHY

W.P.

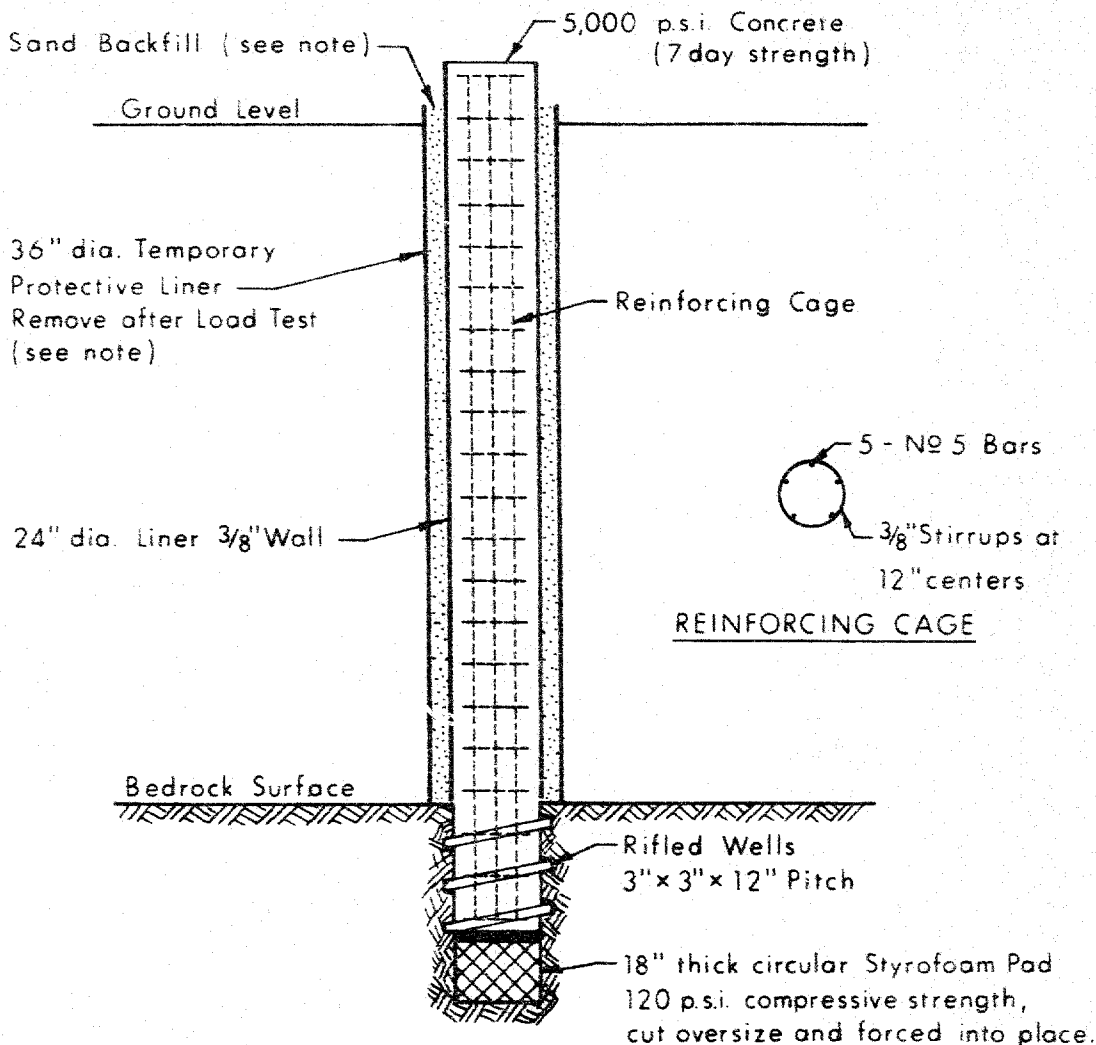
DISTRICT 6

JOB 68-F-58

DATE JULY 15, 1968

APPROVED

DRAWING NO. 68-F-58B



INSTALLATION PROCEDURES

- 1 Drill 36" hole to rock
- 2 Install protective 36" liner
- 3 Drill 24" hole in rock
- 4 Rifle walls of 24" hole
- 5 Inspect and clean base and walls
- 6 Install styrofoam pad
- 7 Install 24" liner
- 8 Install reinforcing steel
- 9 Place concrete and sand backfill simultaneously

NOTE - The 36" and the 24" liner must be installed so as to permit no lateral movements during the load tests.



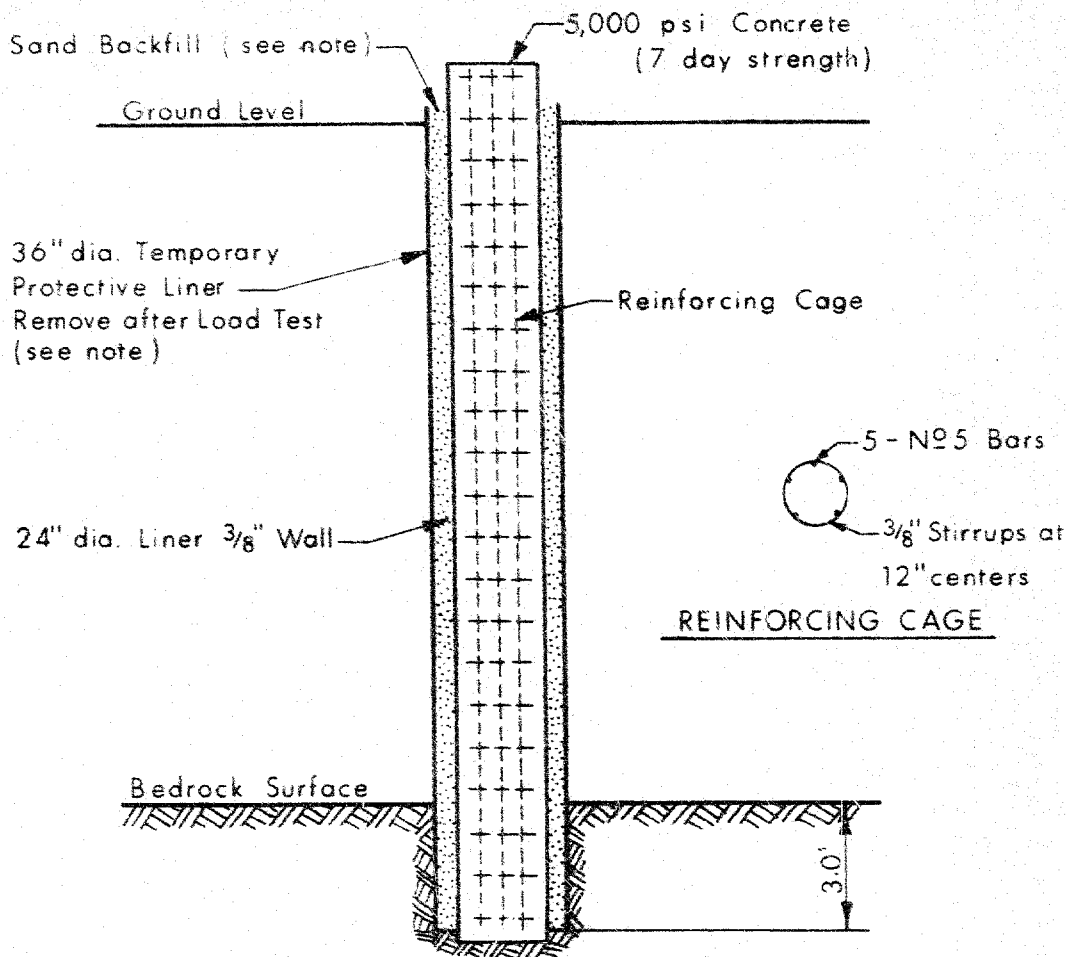
DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

TEST CAISSON No 1

DATE 15 July 1968

APPROVED

DRAWING NO. 68-F-58C



INSTALLATION PROCEDURES

- 1 Drill 36" hole to 3' below rock surface
- 2 Install 36" protective liner
- 3 Inspect and clean base of hole to ensure rock is sound
- 4 Install 24" liner
- 5 Install reinforcing steel cage
- 6 Place concrete and sand backfill simultaneously

NOTE-The 36" and the 24" liner must be installed so as to permit no lateral movements during the load tests.



ONTARIO

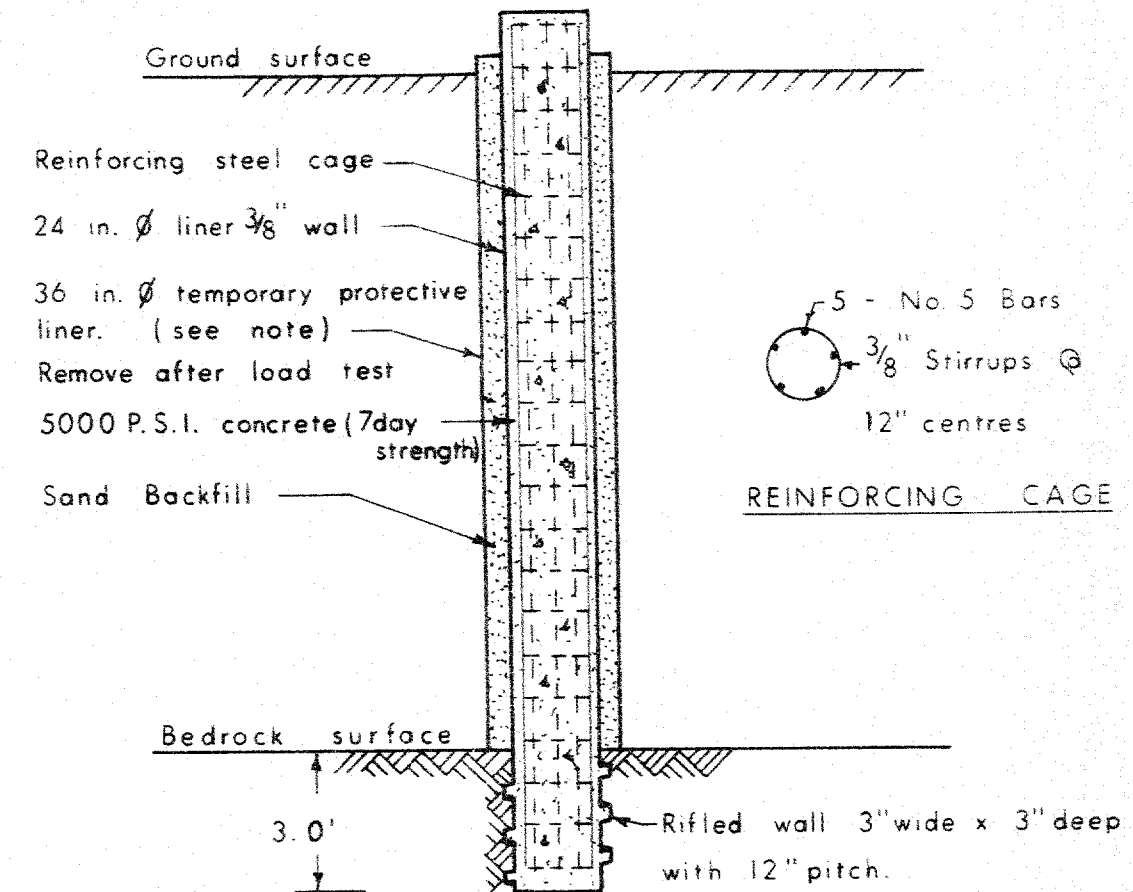
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

TEST CAISSON No 2

DATE JULY 16, 1968

APPROVED

DRAWING NO. 68-F-58 D



INSTALLATION PROCEDURES

- 1 Drill 36 inch Ø hole to rock surface
- 2 Install 36 inch Ø protective liner.
- 3 Drill 24 inch Ø hole in rock.
- 4 Rifle walls of 24 inch Ø hole.
- 5 Inspect and clean walls and base of 24 inch Ø hole to ensure contact with ground rock.
- 6 Install 24 inch Ø liner.
- 7 Install steel reinforcing cage.
- 8 Place concrete and sand simultaneously.

NOTE: The 36 inch liner and the 24 inch liner must be installed so as to permit no lateral movements during the load tests



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

TEST CAISSON No. 3

DATE 16 July 1968

APPROVED

DRAWING NO. 68-F-58 E

401 E. Appleton Street
Toronto, Ontario

July 16, 1964

Master Fed. Investigation
151 Market Drive
Toronto, Ontario

Dear Sirs:

This is to confirm our request of July 2, 1964 for the supply of
a Diamond Drill together with all necessary equipment, as specified
under the terms of our Contract Agreement, of Niagara Falls, Ontario,
Toronto, Ontario, on July 2, 1964.

This request bears Job Number 47-1-14.

Yours truly,

K. A. [Signature]

W/est

E. G. Kelly
Surrey Engineering Engineer
1001 A. G. Street
Principal Construction Engineer

cc: G. H. [Name]
Foundation File
General File

To: Mr. C. A. Grubbs,
Bridge Design Engineer,
Bridge Division, Admin. Bldg.

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

ATTENTION: K. G. Bressi,
Regional Bridge Project
Eng'r.

DATE: November 6, 1968

OUR FILE REF:

BGR.

IN REPLY TO

SUBJECT:

Load Tests on Concrete Caissons, Bridges 26 and 27 -
Hwy. #401 and Airport Road, District No. 6 (Toronto)
-- W.J.P. 397-65 -- W.J. 68-F-58 --

We have recently completed a series of load tests on three 24-inch nominal diameter concrete caissons at the above mentioned site. The caissons were installed in such a way as to eliminate the effect of the overburden and derive all of their supporting capacity from the underlying shale bedrock.

Caisson #1 was socketted 3 feet into sound shale bedrock and the end-bearing effect was eliminated by placing a compressible styrofoam pad under the base prior to pouring concrete.

Caisson #2 was founded 3 feet below the surface of the sound bedrock and the skin friction effect was eliminated by casing the entire shaft and surrounding it with loose sand.

Caisson #3 was socketted 3 feet into sound bedrock so as to derive support from end bearing and skin friction within the rock. The maximum load applied on each caisson was 500 tons; however, in no case was the failure point reached. For your information, the test results are summarized as follows:

No.	Caisson Support	Duration of Test	Duration of 500 Ton Load	Gross Settlement	Net Settlement
1	Skin Friction in Shale only.	29 Hrs.	15 Hrs.	1.02 in.	0.58 in.
2	End bearing on Shale only.	49 Hrs.	36 Hrs.	1.09 in.	0.43 in.
3	End bearing & Skin Friction.	33 Hrs.	24 Hrs.	0.54 in.	0.21 in.

cont'd. /2 ...

Mr. C. E. Selby,
Bridge Design Engineer,
Bridge Div., Admin. Bldg.
Attn: Mr. K. G. Bassi.

November 6, 1968

Based on these test results, the maximum design capacities of concrete caissons installed at this site can be calculated in the following way, subject of course, to the allowable stresses in the concrete:

- $Q = 75 A_b + 10 A_s$ -- where
- Q = Safe capacity in tons.
- A_b = Area of base in sq. ft.
- A_s = Area of uncased shaft embedded in sound rock in sq. ft.

With regard to the proposed structures in this area, it appears that Piers 1 - 5, inclusive, Bridge 27, and Piers 3 and 4, Bridge 26, could be supported on concrete caissons since the subsoil conditions are favourable for this type of foundation. We have made inquiries regarding the cost of installing 106 34-inch to 60-inch diameter caissons on the nearby Carling Breweries site and find that this was about \$50.00 per cu. yd. This price includes installation of, but not supply of, reinforcing steel. Estimates obtained from Western Caissons, Anchor Shoring Ltd., and Franki Canada Ltd., indicate that the cost would probably range from \$45.00 to \$55.00. It seems, therefore, that a considerable saving could be effected by using this type of foundation.

We will be sending you our complete report on the load tests in the near future.

K. G. Selby

KGS/aceP

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

cc: Messrs. B. McDonnell
W. J. Kelly-Smyth

Foundations Files
Gen. Files

MEMORANDUM

To: Mr. Ken Selby
Foundation Section

FROM: K. Ingham
Materials & Testing Division

DATE: July 11, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

Foundation at the 401 and 27 Interchange

The drill cores examined are shales typical of the Dundas formation of Ordovician age and most probably belong to the Credit-Member, although there is insufficient faunal evidence to establish this with certainty.

The rock is medium to dark bluish gray shale which weathers light gray at the top of each section. In general the shale is only slightly calcareous but contains calcareous bands 0.1 to 0.2 feet in thickness which constitute approximately 10% of the upper 10 feet. Small calcareous pellets or concretions are present throughout. The upper 0.5 to 0.7 five feet is weathered and there is some incipient weathering down as far as five feet. The rock is platy bedded with the exception of the upper weathered portion which shows some fissility.

Shale of this type is generally quite competent and I suspect that your values of 2000 psi for the unconfined compressive strength are certainly not high.

K. Ingham per dds

K. Ingham
Geologist

KI:nm