

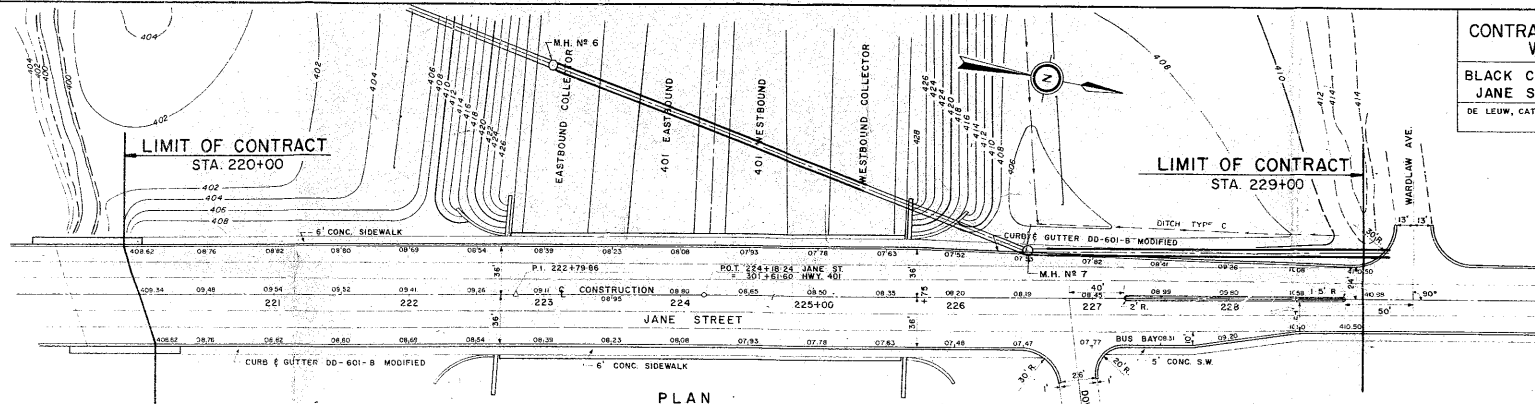
62-F-51

W.P.# 232-60

Hwy. # 401 ;

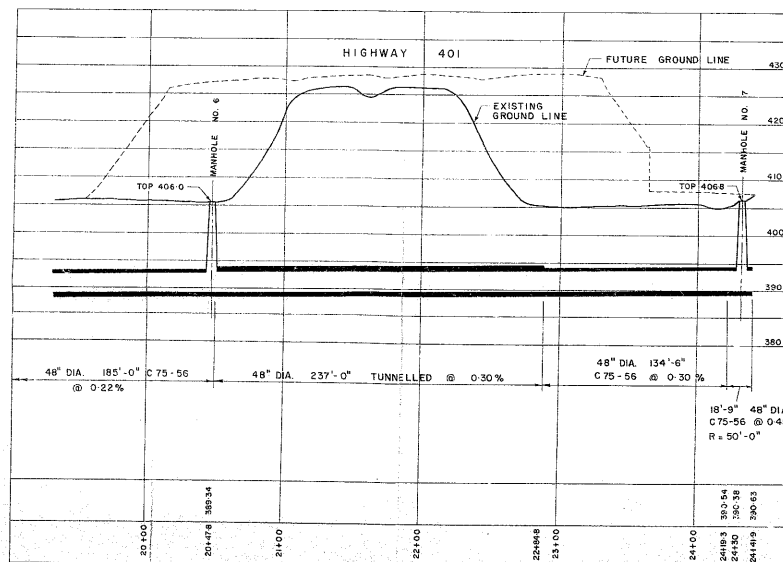
JANE STREET

CONTRACT No. **W.P. No.**
BLACK CREEK SANITARY SEWER
JANE STREET AT HWY. 401
 DE LEUW, CATHEN & COMPANY OF CANADA LIMITED
 CONSULTING ENGINEERS



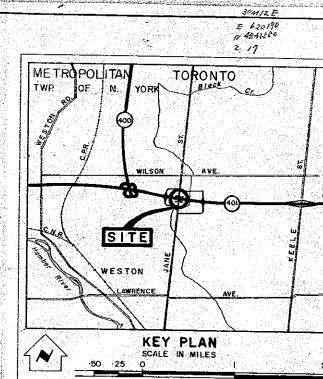
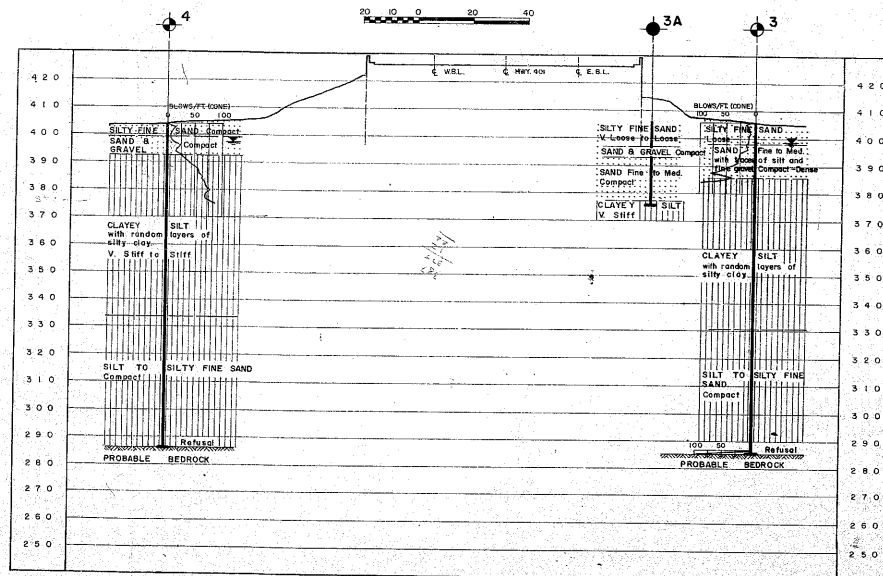
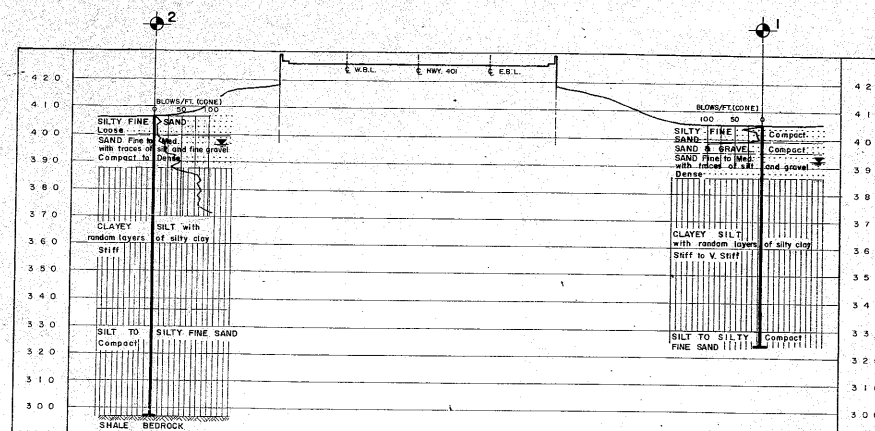
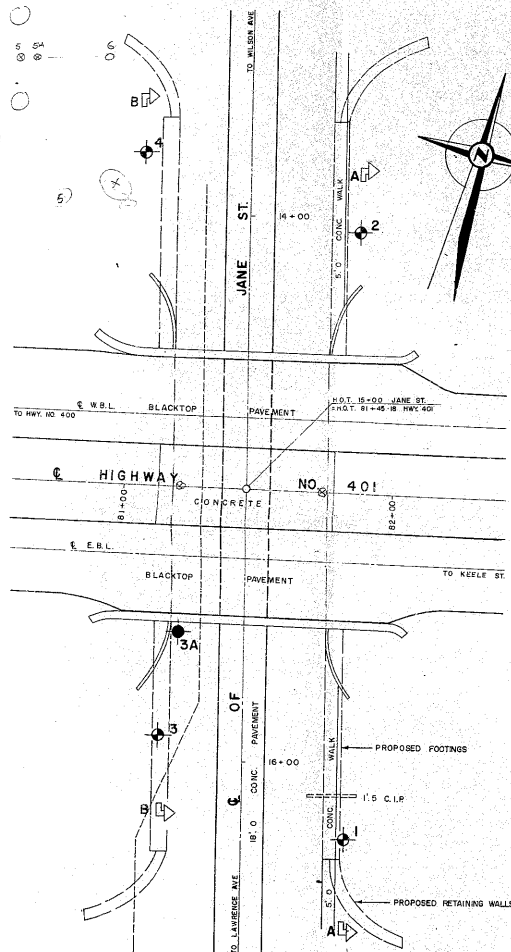
PLAN
 SCALE: 1" = 40'-0"

PROFILE
 SCALE: HORIZ. 1" = 40'-0"
 VERT. 1" = 10'-0"



INVERT ELEVATION
 STATION

MAY-1 1963



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation (May 1962)

NO.	ELEVATION	STATION	OFFSET
1	406.0	15+28	3' LT.
2	407.0	14+06	41' LE
3	406.0	15+91	31' RT.
3A	406.0	15+93	2.4' RT.
4	404.0	13+77	39' RT.

NOTE

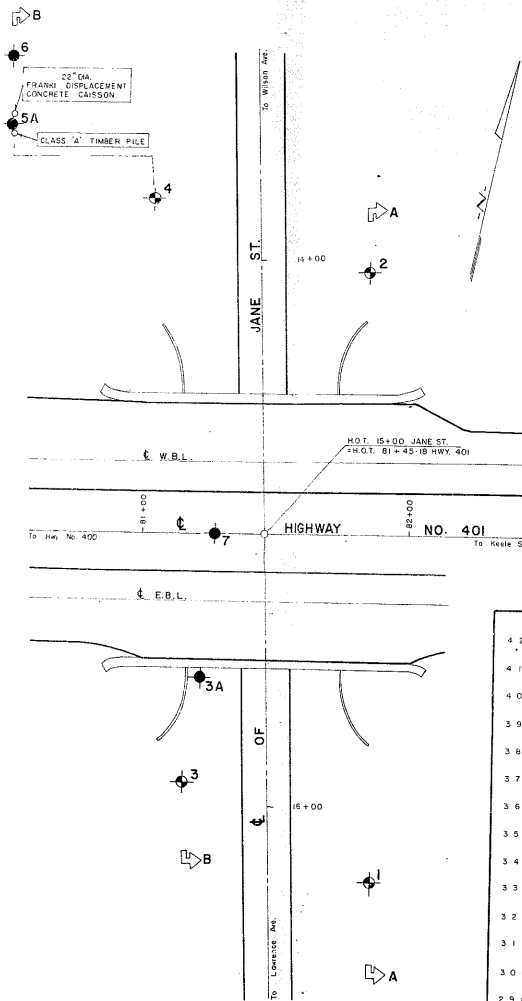
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & RESEARCH SECTION

JANE STREET AND HIGHWAY NO. 401

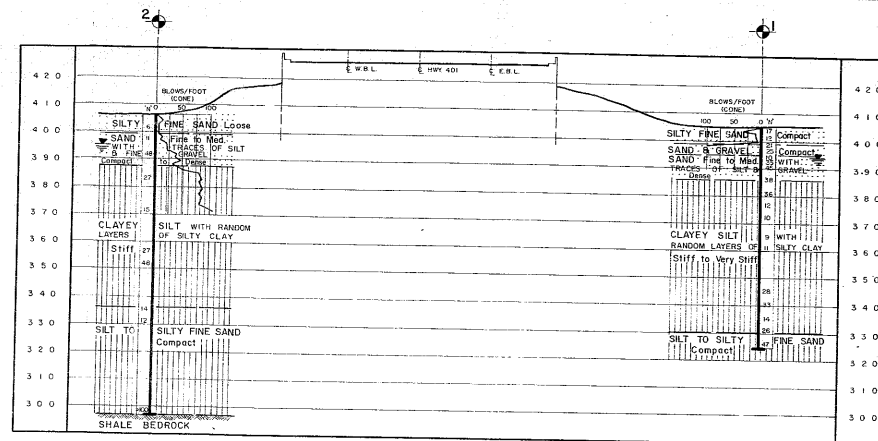
DESIGNED BY: H. W. FORD	DISTRICT NO.: 6	DATE: 21 JUNE 1962
CHECKED BY: H. W. FORD	HW-NO. 62-F-51	JOB NO. 62-F-51
APPROVED BY: H. W. FORD	CONTRACT NO.:	DRAWING NO.:

62-F-51A



PLAN

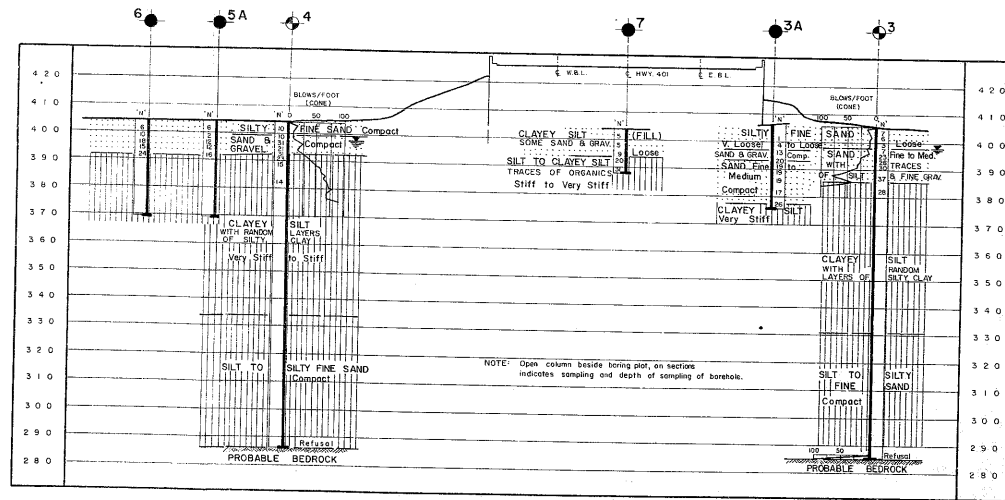
SCALE IN FEET
20 10 0 10 20



A - A

SCALE IN FEET
20 10 0 10 20

NOTE
The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downtown and at the TORONTO District Office. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.



B - B

SCALE IN FEET
20 10 0 10 20

NOTE Open column beside boring plot, on sections indicates sampling and depths of sampling of borehole.



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation, May 1962

NO.	ELEVATION	STATION	OFFSET
1	406.0	16+28	36' LT.
2	407.0	14+06	41' LT.
3	406.0	15+91	31' RT.
3A	406.0	15+53	24' RT.
4	404.0	13+77	39' RT.
5A	404.0	13+49	94' RT.
6	404.0	13+23	93' RT.
7	404.0	15+00	18' RT.
TIMBER PILE (TIP EL.)	366.5	13+52	93' RT.
FRANK PILE (BASE EL.)	376.0	13+45	93' RT.

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.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

JANE STREET

KING'S HIGHWAY NO. 401 DIST. NO. 6
CO. YORK
TWP. NORTH YORK LOT. 10 CON. IV & V

TEST PILE LOCATIONS, BORE HOLES & SOIL STRATA			
SUBM.D. K.S.	CHECKED <i>[Signature]</i>	W.P. NO. 232-60	K.S.B. DRAWING NO.
DRAWN <i>[Signature]</i>	CHECKED <i>[Signature]</i>	JOB NO. 62-F-51	62-F-51B
DATE MAY 12, 1963	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>	CONT. NO.		

REF. NO. D-5228-1

#

62-F-51B

#

N.P. 232.60

#

Hwy 401

JANE ST.

Contract # 23-63 1P

Mr. G. K. Hunter,
Sr. Project Design Engr.,
Toronto Regional Office,
Attention: Mr. J. H. Blevins.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
June 28, 1963

Your Memo - June 11/63

Proposed Hwy. #401 & Jane St. Bridge -
W.P. 23-60 - District #6 - W.J.62-F-51
(Protection of Existing Utilities)

We are in receipt of your memo dated June 11, 1963, regarding possible damage to the existing utilities due to future pile driving operations.

With regard to the possible effects of vibration on the utilities, we are not in a position to predict anything of a definite nature, but we believe that no undesirable effects will result if the following precautions are taken:

All piles within 10' of a particular sewer or water main should be prebored to a depth of about 4' below the invert level of the pipe.

If you have any further queries in connection with this matter, we shall be glad to be of assistance.

KGS/MdeF

cc: Foundations Office ✓
Gen. Files

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

Memo to Mr. A.G. Stermac, **Date** June 11, 1963
Principal Foundations Engineer,
Materials & Research, **Subject** Re: W.P. 232-60, Highway 401,
Lab Building. Proposed Jane Street Structure,
From J.H. Blevins, Toronto Reg. R.D.O. Your File: W.J. 62-F-51 and
62-F-51B.

Further to our telephone conversation of to-day, with reference to the possibility of damaging underground utilities during the pile driving operation on the project.

A meeting was held to-day with representatives of the Metro Water & Sewerage Department to discuss the type of protection which should be provided for their installations on Jane Street. The utilities have been located and will not be directly affected by the bridge construction, however, the Metro representatives were concerned about possible damage arising out of the pile driving operation.

We have enclosed herewith the following plans:

- (1) A print of a Metro plan showing the location of the 36"Ø watermain. As you will note, a free hand sketch of the encasement has been circled in red in the upper right hand corner.
- (2) A print of a drawing showing the location of the 48"Ø sanitary sewer. As you will note, we have also shown the proposed protection which is to be installed prior to the pile driving operation.
- (3) A plan showing the location of the sewer with reference to the footings of the proposed sewer.

Continued /2

June 11, 1963

Mr. A.G. Stermac - Re: W.P. 232-60, Highway 401.

The Metro Authorities were not so much concerned with the possible damage due to displacement, but rather due to vibration. Both Mr. Grebski of the Bridge Office and the writer felt quite sure that the utilities would not be damaged, however, we were not in a position to grant any assurances to the Metro representatives.

We would, therefore, request that you review the proposed work and advise this office of any reasonable precaution which may be taken to avoid damaging the utilities. It is acknowledged that the contractor's operator will have to be closely supervised during the pile driving operation and the Construction Staff is well aware of this, however, there may be some minor adjustments which you may wish to recommend and which would act as further safeguards during the pile driving operation.



J.H. Blevins

PROJECT DESIGN ENGINEER

For:

G.K. Hunter

SENIOR PROJECT DESIGN ENGINEER

JHB/GB.

Encl.

Copies to: Mr. H.F. Gilbert
Mr. C. Grebski

Mr. B. R. Davis,
Bridge Design Engr.,
Bridge Division.

Attention: Mr. C. Grebski.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

May 23, 1963.

Proposed Overpass at Hwy. #401
and Jane Street, District #6,
Toronto. (W.J. 62-F-51B)

In conjunction with the pile loading tests recently carried out at the above site, we have carried out some additional soil borings. The log sheets for these borings #5A, #6 and #7, together with Drawing #62-F-51B, are enclosed for your information and should be attached to your copy of Foundation Section Report #62-F-51.

For complete details and results of the aforementioned pile loading tests, you are referred to Foundation Section Report #62-F-51B.

KGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
G. K. Hunter (2)
C. Fraser
T. J. Kovich
A. Watt

Foundations Office
Gen. Files.

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

Mr. B. R. Davis,
Bridge Design Engr.,
Bridge Division.

Attn: Mr. C. S. Grebski

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
April 29, 1963

Jane St. Overpass, Hwy. #401,
District #6, Toronto

With reference to your memo of April 26, 1963, concerning the lateral forces acting on piles, we are herewith submitting our comments for your consideration:

1. Earth Pressure Computation:

If granular material is used as backfill to structures or retaining walls, as it should be, the intensity of the earth pressure acting on the structure will depend on the angle of internal friction of the material which, in turn, is a function of the material's density. The denser the material, the higher its angle of internal friction and, therefore, the lower the earth pressure. The relationship of the coefficient of active earth pressure to the angle of internal friction for the case of no wall friction reads:

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2 \left(45^\circ - \frac{\phi}{2} \right)$$

For ϕ	=	30°	35°	40°	45°
K_a	=	0.33	0.27	0.22	0.17

When wall friction exists, the above values give results on the conservative side.

If the backfill is properly compacted, it is quite justified to assume that the angle of internal friction of the granular material will be between 35° and 40°, and the corresponding value of the coefficient of active earth pressure, between 0.27 and 0.22. However, the unit weight of such a backfill should also be greater than 100 p.c.f. - say, 125 p.c.f.

Taking $K_a = 0.25$ and $\gamma = 125$ p.c.f., the value of 0.31 for the equivalent hydrostatic pressure is obtained.

Mr. B. B. Davis,
Attn: Mr. C.S. Grebski

- 2 -

April 29, 1963

2. Lateral Forces on Piles:

Based on a number of full-size tests, the following allowable horizontal loads acting on piles, have been proposed:

<u>Type of Pile</u>	<u>Pile Head</u>	<u>Type of Soil</u>	<u>Allowable Load per Pile</u> (lb. per pile)
Timber	Free end	Sand	1,500
(12-inch diam.)	Fixed end	Sand	4,500

Fixed end condition may be attained by embedding the pile head at least 24 inches in the concrete cap. The above loads allow for 1/4 inch lateral movement.

Whenever the loads are higher, batter piles are provided. It appears to be desirable to provide batter piles whenever piles are subjected to lateral loads in excess of about 1,000 lb. per pile.

We trust that the above answers your questions; however, should you require additional information, please feel free to call on our Office.

AGS/MdeF

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

cc: Foundations Office
Gen. Files.

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundations
Engineer,
Room 107, Lab. Bldg.

FROM: C. S. Grebski

DATE: April 26, 1963.

OUR FILE REF.

IN REPLY TO

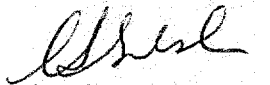
SUBJECT: Jane St. Overpass Hwy. #401
District #6.

We have designed this bridge with 12" \emptyset creosoted timber piles carrying 40 tons, however we are having difficulty taking care of the horizontal force due to earth pressure. Normally timber piles are loaded to 20 tons and there are then sufficient piles to take the horizontal force. The designer has used an equivalent fluid pressure of 36 lbs. and a weight of earth of 100 lbs.

During our discussion of earth pressures behind the retaining walls along Hwy. 401 it was pointed out that if the granular fill behind the walls was placed at a slope of $1\frac{1}{2}$ to 1 the horizontal earth pressure would be less than 36 lbs. and probably 28 to 30 lbs. Could we take advantage of this and design for the lesser value and if so what pressure and earth weight would you recommend?

With the Jane St. Structure being close by it would be possible to ensure backfill being placed as required.

CSG:go


C. S. Grebski,
Sr. Bridge Project Engineer.

cc: Foundations Office (RM. 110)
Mr. B. R. Davis,
Bridge Design Engr.,
Bridge Division.

Attn: Mr. Chester Grebski

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
April 11, 1963

Hwy. 401 and Jane Street -
W.P. 232-60 -- District 6.

With reference to your memo of April 10, 1963, concerning the above site, we would recommend that for a footing elevation of 40⁺.25, creosoted timber piles of 35-ft. length be used. This recommendation is based on the requirement of 33 ft. of embedded length of pile.

The footing elevation of 40⁺.25 was given to us verbally, by Mr. Chester Grebski, on April 11, 1963.

It is also recommended that the retaining walls be founded on piles because of the very heterogeneous soil stratification.

AGS/MdeF

cc: Foundations Office ✓
Gen. Files.

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



FRANKI

OF CANADA LIMITED

214 MERTON STREET,
TORONTO.

CABLEGRAMS
"FRANKIPILE"

TELEPHONE:
HUDSON 1-6426-7

April 11, 1963.

Mr. A. Stermac, P.Eng.,
Department of Highways, Ontario,
East Block, Parliament Buildings,
Queen's Park,
Toronto, Ontario.

Dear Tony:

Re: Bridge, Jane St. & 401 Hwy.

Enclosed is a copy of the sand pile layout for
this project.

Yours very truly,

FRANKI OF CANADA LIMITED

WEL:ag
Encl.

W. E. Lardner, P.Eng.,
Manager - Central Division.

MEMORANDUM

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

FROM: C. S. Grebski

DATE: April 10, 1963.

OUR FILE REF.

IN REPLY TO

SUBJECT:

Hwy. 401 and Jane Street,
W.P. 232-60 District #6.

As recommended creosoted timber piles with a design load of 40 tons is what we are presently designing for the Jane St. Bridge. The bottom of our footings are at elevation ^{404.25} 400.00. At what elevation should the pile tips stop? In other words what length of timber pile do you recommend?

CSG:go
c.c. B. Davis

CSG
C. S. Grebski,
for B. Davis,
Bridge Design Engineer.

Answered by phone on April 11, 1963.

*35 ~~FT~~ FT LONG PILES 33 FT OF IMBEDDED LENGTH.
~~TIP ELEV. AT 364.0~~*

A. Stermac

Mr. B. R. Davis,
Bridge Design Engr.,
Bridge Division.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

Attention: Mr. Chester Grebski.

April 9, 1963

62-F-51

Hwy. 401 & Jane Street Interchange
-- Results of Pile Loading Tests --

This is to advise you on the results of the pile loading tests carried out at Hwy. 401 and Jane Street interchange. Two types of piles were tested: one timber, and one Franki type pile. The details of the piles, test procedure and all numerical test data will be given in a special report which will be issued shortly.

The timber pile failed under a load estimated to be between 80 and 100 tons. A safe load per pile of 40 tons is therefore recommended.

The Franki type pile definitely failed under a load of 140 tons, but there are indications that failure occurred earlier. Since the test of this pile was entirely the responsibility of Franki of Canada Co., Ltd., the interpretation of the test was left to them. They recommended a safe load of 50 tons per pile.

The scheme of compacting the upper sand layer by driving sand piles, had to be abandoned because additional and subsequent field investigation revealed that the sand layer is non-existent in places under the present structure.

Both types of piles, the timber and the Franki, would be technically equally satisfactory.

Because of limited access, battered Franki piles could not be installed in two locations. However, it is considered that this would have only negligible effects and could, therefore, be ignored. It has also been pointed out to us by the Franki representatives, that for the driving of Franki piles, the site will have to be graded and level, and for battered piles, a clearance of 17 ft. from the centre of the pile to the toe of the slope and 22 ft. at 3 ft. above ground level, will be required.

cont'd. /2 ...

MR. D. M. DAVIS,
Attn: Mr. C. Grebski.

April 9, 1963

Franki of Canada's representative, Mr. W. Lardner, has given us a rough cost estimate for the installation of 280 twenty-two inch diameter Franki piles. The estimate is \$28,000. for the piles, and a lump sum of \$4,000. for mobilization. This estimate should not be considered as binding or final, but, rather as a guide.

In view of all above-mentioned facts and information, it appears to us that timber piles would represent a more favourable solution.

We have basically already advised you of all facts contained in this memo. However, should you require additional information, please feel free to call on our Office.

AGS/MdeF


A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files.

MEMORANDUM

To: Mr. A. Stermac
Principal Foundations
Engineer
Room 107 Lab. Bldg.

FROM: C. Grebski

DATE: February 22, 1963.

Att.: Mr. K. Selby

OUR FILE REF.

IN REPLY TO

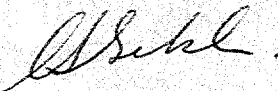
SUBJECT: Jane St. Overpass
W.P. 232-60
Hwy. #401 Dist. #6

Due to the short time allotted for construction of this bridge we recommend that a pile load test be done now. This will have the advantages of not holding up the contract and assures no changes will be required in the number of piles after the contract is let. In our opinion the piles should be tested to failure as this will yield the maximum information regarding allowable design load.

The award date for the construction of Jane St. Overpass is tentatively set for July 17, 1963. This means we should have all test results about two months minimum prior to this date i.e. May 17, 1963.

It might be advisable to look into using timber piles at Black Creek Barrel Arch and do a load test there at the same time.

CG:go
c.c. S. McCombie



C. Grebski,
for B. Davis,
Bridge Design Engineer.

Mr. B. Davis,
Bridge Engineer,
Bridge Division.

Attn: Mr. C. Grebski.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

February 21, 1963

Your Memo - Feb. 11/63

Jane St. Overpass -
W.P. 232-60,
Hwy. #401, Dist. #6.

We have reviewed the Preliminary Plan No. D-5228-P for the above-mentioned proposed structure and are in agreement with your proposals to utilize timber piles. We believe, however, that a pile load test should be carried out in order to check the proposed design load of 25 tons/pile.

With regard to your query concerning the use of Franki piles at this site, the upper granular deposit in which these latter could be founded, is too shallow in places to achieve a suitable design load. It would, therefore, in our opinion, not be practicable to use this type of pile at this location.

If we can be of any further assistance to you in this matter, please contact this Office.

KGS/MdeF

cc: Mr. S. McCombie

Foundations Office ✓
Gen. Files.

K. G. Selby
K. G. Selby,
SENIOR FOUNDATION ENGR.
For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM

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

FROM: C. S. Grebski

DATE: February 11, 1963

OUR FILE REF.

IN REPLY TO

SUBJECT: Jane St. Overpass
W.P. 232-60
Hwy. #401, Dist. #6.

Enclosed is one print of the preliminary plan for this bridge. As you know we were inquiring if Franki Piles could be used here and it was agreed additional borings would be required to determine this.

We have shown timber piles on the preliminary plan and possibly they are the best answer for this site, however, we will await your recommendation.

CSG/rt

c.c. S. McCombie



C. S. Grebski,
Sr. Bridge Project Engineer.

Mr. B. Davis,
Bridge Engineer,
Bridge Division.

Attention: Mr. C. Grebski.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

January 16, 1963

Re: Proposed New Structure at Jane St. &
Hwy. #401, Toronto, District #6,
W.P. 232-60 -- W.J. 62-F-51.
Proposed Piled Foundations

It is now proposed to construct a completely new structure at the above-mentioned site as an alternative to the widening which was formerly proposed. The new span will be approximately the same as that of the existing bridge. It is also proposed to support the new structure by means of piled foundations.

Following, are our recommendations pertaining to the latter:

The new structure should be founded on timber piles driven to approximate el. 375.0. It is believed that a design capacity of 25 tons/pile can be achieved, but in order to confirm this, a pile loading test should be carried out. Pile caps should be constructed at a sufficient depth for frost protection. It will be necessary to use treated timber piles because they will not be completely below the ground water level.

If we can be of any further assistance in this matter, please contact this Office.

K. G. Selby

KGS/MdeF

cc: Foundations Office
Gen. Files.

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

Phone call to C. Grebski 31st Jan '63
Bridge Office will send preliminary plan of new
bridge as soon as it is approved. Foundation
Section will then decide if further drawings are
necessary

K. G. Selby

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Division,
(Foundation Section)
Attention: Mr. S. McCombie.

June 29, 1962.

D.H.O. FOUNDATION INVESTIGATION
REPORT.
W.J. 62-F-51 -- W.P. 232-60.

Re: Proposed Widening of Hwy. #401 Overpass
at Jane Street, Toronto, District #6.

Attached, we are forwarding to you, our
detailed foundation investigation report on the subsoil
conditions existing at the above structure location.

We believe you will find the factual data and
recommendations contained therein, adequate for your future
design work. Should further information be required, please
feel free to contact our Office.

KYL/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
G. K. Hunter
C. Fraser
T. J. Kovich
J. Roy
J. E. Gruspier
E. R. Saint
F. Norman
A. Watt
Foundations Office
Gen. Files



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

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 - 5.3) Sand & Gravel.
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 - 7.2) Structure Foundation.
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-

FOUNDATION INVESTIGATION

For

Proposed Widening of Hwy. #401 Overpass
at Jane Street, Toronto, District #6,
W.J. 62-F-51 -- W.P. 232-60.

1. INTRODUCTION:

A request for a foundation investigation at the site of the proposed widening of the overpass at Hwy. #401 and Jane St. was received from the Bridge Location Section in a memo dated April 25, 1962.

A field investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the location of the proposed widening. Presented in this report are the results of this investigation, together with recommendations pertaining to the design of the proposed structure foundations.

2. DESCRIPTION OF SITE:

The overpass at Jane St. and Hwy. No. 401 is in the Township of North York in Metropolitan Toronto. It is proposed to widen the overpass approx. 85 ft. in each direction.

The site is located in the centre of the 700 ft. wide flood plain of Black Creek. At the overpass Hwy. 401 is built on a 20 ft. embankment, and Jane St. is about level with the plain. The present Black Creek is about 300 ft. east of Jane St.

2. DESCRIPTION OF SITE: (cont'd.) ...

Physiographically, the site lies in the South Slope Region which is the south slope of an interlobate moraine. This moraine consists of a clayey till deposited during the Pleistocene Ice Epoch. The site is located in a valley cut and refilled in the above moraine.

3. FIELD INVESTIGATION PROCEDURE:

A total of five sampled boreholes and four dynamic cone penetration tests was carried out in the field investigation. During the field work, disturbed and undisturbed samples were obtained. Disturbed samples were obtained by means of a standard split spoon sampler driven into the soil with an energy of 350 ft. lbs. per blow. Undisturbed samples were obtained by means of 2-inch I.D. Shelby tubes which were pushed into the soil hydraulically. Some disturbed samples were also obtained in the Shelby tubes. In-situ vane tests were carried out wherever possible at elevations 18" below the various sample depths. A measure of the relative density of the predominantly granular deposits was obtained by means of the Standard Penetration Tests. Ground water levels were recorded in the boreholes throughout the duration of the investigation.

cont'd. /3 ...

4. LABORATORY TESTS:

Samples were visually examined and classified at the site as well as in the laboratory.

Laboratory tests were carried out on silty clay and clayey silt samples to determine:-

- (a) Natural Moisture Content
- (b) Atterberg Limits
- (c) Unit Weight
- (d) Undrained Shear Strength.

Laboratory and field test results are summarized and included under Appendix I of this report.

5. SOIL TYPES AND SOIL CONDITIONS:

5.1) General:

The subsoil at the site consists of fluvial deposits ranging from sand and gravel to silty clay. The first 20 ft. depth from the ground level consists of sand and sand and gravel, which is underlain by clayey silt, silt to silty fine sand and finally by shale bedrock.

The boundaries of the various deposits are shown on the accompanying bore log sheets. The estimated stratigraphical profiles and cross sections shown on Drawing #62-F-51A are based on information from the boreholes. From ground level downwards, the various soil types are as follows:-

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.2) Silty Fine Sand:

This stratum was established in all boreholes and it extended from the ground surface to elevation 400', approximately. The depth of the material varied from 4.5 ft. to 9.0 ft. with an average depth of 7.0 ft. The silty fine sand had a brown colour, a result of oxidization, and a relative density varying from very loose to compact.

5.3) Sand & Gravel:

This material was found in borehole No's 1, 3A and 4 under the silty fine sand and it extended to elevations 396', 393' and 393', respectively. The thickness of the sand and gravel layer varied from 4' to 6.5'. The gradation of the material varied from a sand with gravel to a well graded gravel. The gravel particles were bulky and subrounded with a maximum size of 1½". The sand and gravel was found to be compact with an average 'N' value of 22 blows/ft.

5.4) Sand:

The sand stratum was found in all boreholes except in borehole No. 4. The stratum was found to be about 12 ft. in thickness and extended to elev. 387', approximately. The sand is mainly fine to medium and contains traces of silt and fine gravel. In borehole No. 3A the sand stratum extends to elev. 377'. The relative density of this material is compact to dense with an average standard penetration value ('N') of 25 blows/ft.

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.5) Clayey Silt:

The clayey silt underlies the sand stratum found in borehole No's 1 to 3A and the sand and gravel stratum in borehole No. 4. It is about 55 ft. in thickness and extends to elev. 326', approx. The clayey silt contains randomly distributed traces of coarse sand and layers of silty clay. The average properties of the clayey silt are:-

Liquid Limit	29.3%
Plastic Limit	17.8%
Water Content	21.9%
Unit Weight	131 lbs./cu.ft.
Shear Strength -		
Field Vane	1600 p.s.f.
Quick Triaxial &		
Unconfined Compression	1600 p.s.f.

5.6) Silt to Silty Fine Sand:

This stratum underlies the clayey silt and extends down to the shale bedrock. The stratum is about 45' in thickness and consists of silt to silty fine sand. The material is compact to dense with an average standard penetration value ('N') of 25 blows/ft.

cont'd. /6 ...

5. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.7) Shale Bedrock:

Probable shale bedrock elevations of 297', 286' & 286' were established in borehole No's 2, 3 and 4, respectively. In borehole No. 2 the bedrock was established by means of a 2" long split spoon sample and in borehole No's 3 and 4 the bedrock elevation was established by means of dynamic cone and auger refusal.

6. GROUND WATER LEVEL:

The ground water levels were observed in the boreholes during the time of the field investigation. The water level was observed at elevations 393.0', 396.5', 399.0' and 397.5' in borehole No's 1, 2, 3 and 4, respectively.

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General:

It is proposed to widen the Hwy. 401 and Jane St. overpass by 85' at each side. The extensions will be gravity wall abutments supporting pin jointed cross beams. The existing structure is designed with a foundation pressure of 2 T.S.F., approx.

7.2) Structure Foundation:

The structure extensions should be built as independent units butting against the existing bridge with vertical expansion joints. The extensions should be supported on approximately 6 ft. wide spread footings founded at elevation 397.0' using a design load of 2.0 T.S.F. This will involve an excavation some 9' in depth.

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

7.2) Structure Foundations: (cont'd.) ...

The design load is based on a maximum theoretical settlement of 2 inches. To improve conditions as regards settlement, the area should be surcharged with 25 ft. of fill placed as close to the proposed footings as possible and left for as long a period as possible.

It is anticipated that a dewatering scheme will have to be used during the construction of the footings. Therefore it is recommended to check the water level at a few locations at the time of construction and if the ground water is above elev. 397', a dewatering scheme utilizing sheeting or well points should be employed. If sheeting is used it should be driven to a depth below the footing base equal to the height of the prevailing water level above it.

No stability problems are anticipated for the embankments provided 2:1 slopes are used.

8. SUMMARY:

- (1) It is proposed to widen by 85' at each side, the overpass at Hwy. #401 and Jane St. in Metropolitan Toronto.
- (2) Subsoil at the site consists of 20 ft. of compact sand and sand and gravel mixtures underlain by 55 ft. of clayey silt, 45 ft. of silt to silty fine sand and finally, shale bedrock. The surface of the bedrock varies between elev. 297' and 286'.

cont'd. /8...

8. SUMMARY: (cont'd.) ...

- (3) It is recommended to build the extensions as independent units butting against the existing bridge with vertical expansion joints. The extensions should be supported on spread footings founded at elev. 397.0' using a design load of 2 T.S.F. This design load is based on a maximum settlement of 2".
- (4) To minimize any settlement likely to occur during or after construction, the area should be surcharged with a 25 ft. high fill for as long a period as is possible.
- (5) If the ground water level is observed to be above the base of the spread footings during construction, a dewatering scheme will be necessary. Recommendations for this are given under 7.2.
- (6) No stability problems are anticipated for the embankments provided 2:1 slopes are used.

9. MISCELLANEOUS:

The field work was undertaken during the period from May 17th to May 31st, 1962 by Mr. I. Holubec, who also prepared the report under the supervision of Mr. K. G. Selby.

July 1962.

APPENDIX I.

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

108 62-F-51

LOCATION Jane St., Sta. 16+28', 38' Lt. of 6

ORIGINATED BY I.H.

W. P. 232-60

BORING DATE May 17th, 1962.

COMPILED BY B.K.

DATUM 406

BOREHOLE TYPE 4 1/2" Auger

CHECKED BY I.H.

SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL		PLASTIC LIMIT — WP		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	WATER CONTENT — W	WATER CONTENT — WL	WATER CONTENT — WP	P.C.F.		
406.0	Ground Surface											
0	Silty Fine Sand Compact.		1	SS	17	400						
400.0			2	SS	12							
6.0	Sand and gravel		3	SS	21							
396.0	Compact		4	SS	25							
10.0	Sand, fine to med., with traces of silt and fine gravel, Dense		5	SS	10							
			6	SS	35							
			7	SS	45	390						
387.0			8	SS	38						137	
19.0			9	SS	36	380						
	Clayey silt with random layers of silty clay, stiff to very stiff.		10	SS	12							
			11	SS	10	370						
			12	SS	9						132	
			13	SS	11	360						
			14	TW	PH							
			15	TW	PH	350						
			16	TW	28						130	
			17	TW	33	340					131.5	
			18	TW	14							
331.0											141	
75.0	Silt to silty fine sand, compact to dense.		19	TW	26	330						
324.5			20	TW	47							
81.5	End of borehole.					320						
						310						

FOUNDATION SECTION

CHECKED BY I.H.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100					LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. O Quick Triaxial + Field Vane					WATER CONTENT % 10 20 30				
							500	1000	1500	2000	2500	WP	W			WL
407.0	Ground Surface															
0.0	Silty fine sand Loose.															
400.0			1	SS	6	400										
7.0	Sand, fine to med. with traces of silt and fine gravel.		2	SS	11											
	Compact to dense		3	SS	48	390										
388.0			4	SS	27											
19.0			5	TW	PH	380										
	Clayey silt with random layers of silty clay stiff.		6	TW	PH											
			7	SS	15	370										
			8	TW	PH											
			9	TW	PH	360										
			10	SS	27											
			11	TW	48	350										
			12	TW	PH											
			13	TW	PH	340										
337.0			14	TW	14											
70.0	Silt to silty fine sand, Compact.		15	TW	12	330										
			16	TW	PH											
						320										
						310										
						300										
297.0	Auger Refusal		17	AS												
110.0	Probable shale bedrock		18	SS	100											
	End of borehole				for 1"											

RECORD OF BOREHOLE NO. 3A

FOUNDATION SECTION

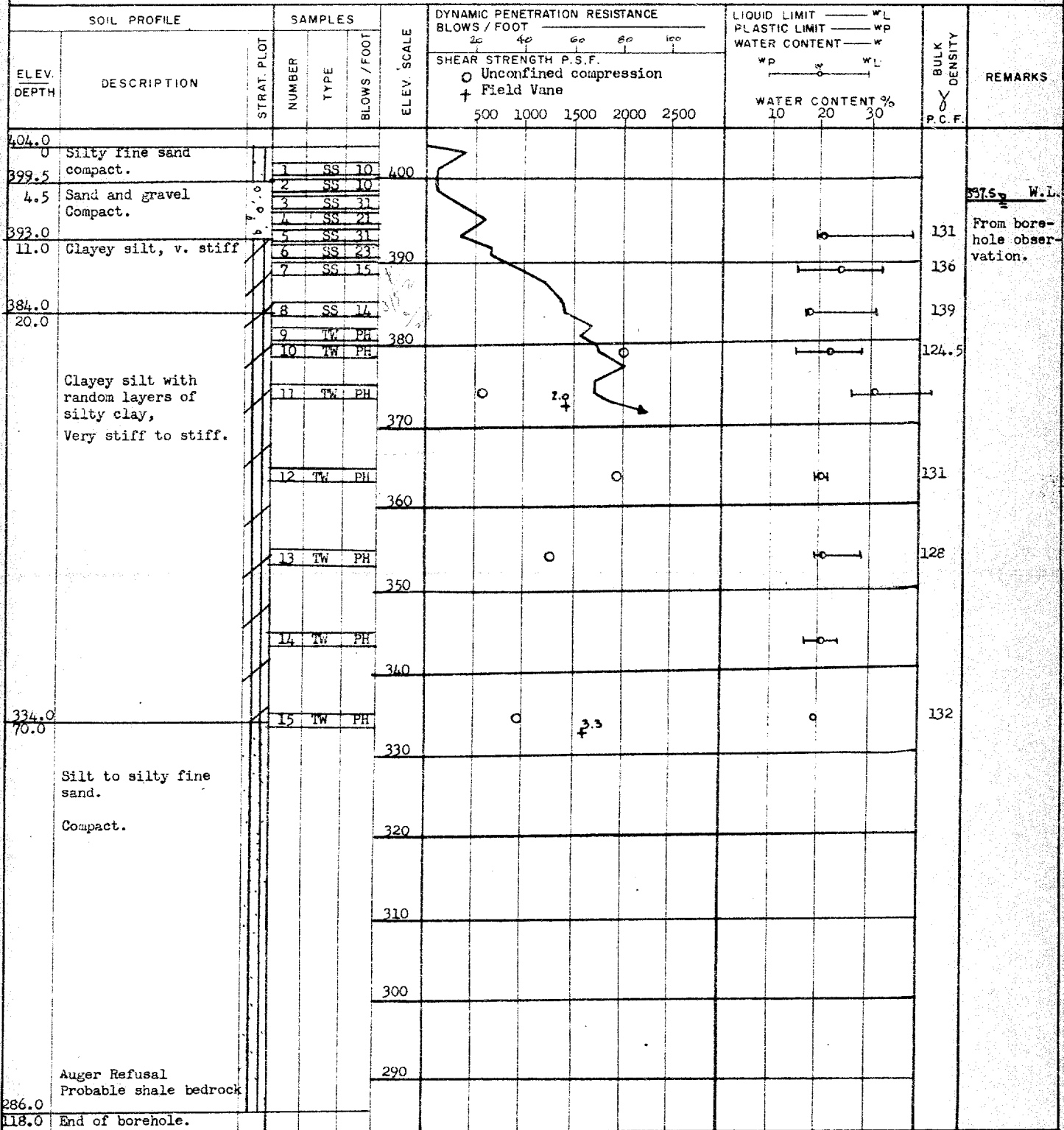
JOB 62-F-51 LOCATION Jane St., Sta. 15'53, 24' Rt. of C. ORIGINATED BY I.H.
W.P. 232-60 BORING DATE May 31st, 1962. COMPILED BY B.K.
DATUM 406 BOREHOLE TYPE 4 1/2" Auger CHECKED BY I.H.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W		BULK DENSITY P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.		WATER CONTENT %				
							500	1000	1500	2000	2500		
406.0	Ground Surface												
0	Silty fine sand Very loose to loose		1	SS	1	400							
399.0			2	SS	4								
7.0	Sand and gravel		3	SS	13								
393.0	Compact		4	SS	20								
13.0	Sand, fine to medium, compact.		5	SS	19	390							
			6	SS	19								
			7	SS	19								
			8	SS	17	380							
377.0													
29.0	Silty clay, very stiff		9	SS	26								134
375.5													
30.5	End of borehole.					370							

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 62-F-51 LOCATION Jane St., Sta. 13+77, 39' Rt. of E ORIGINATED BY I.H.
W.P. 232-60 BORING DATE May 28, 1962. COMPILED BY B.K.
DATUM 404.0 BOREHOLE TYPE 4 1/2" Auger CHECKED BY I.H.

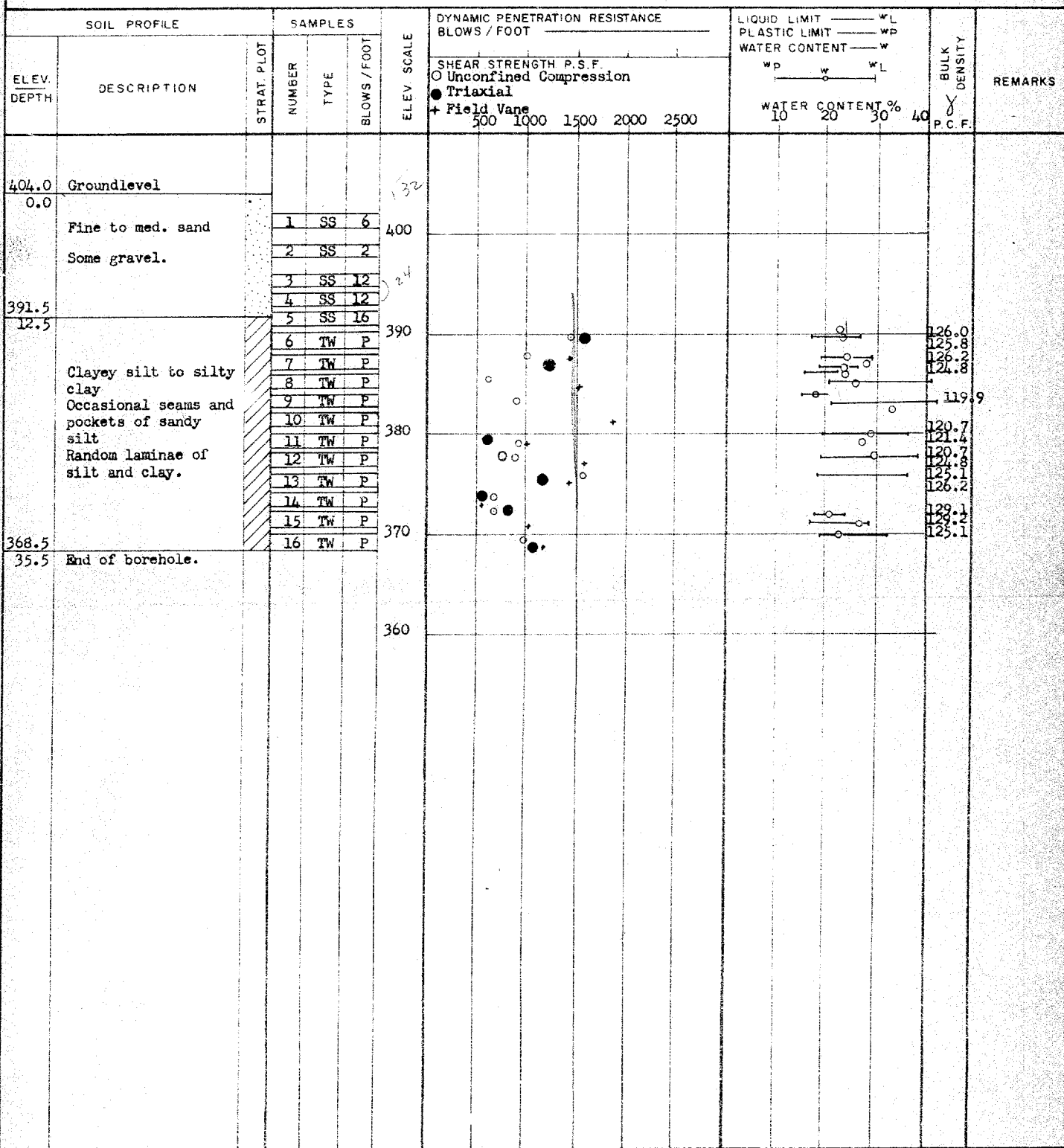


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 5A

FOUNDATION SECTION

JOB 62-F-51 LOCATION Sta. 13+49.94' Rt. 6 Jane St. ORIGINATED BY H.S.
 W.P. 232-60 BORING DATE April 3, 1963. COMPILED BY H.S.
 DATUM Geodetic BOREHOLE TYPE Washboring - NX Casing. CHECKED BY K.S.



RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 62-P-51

LOCATION Sta. 13/23 93' Rt. & Jane St.

ORIGINATED BY H.S.

W.P. 232-60

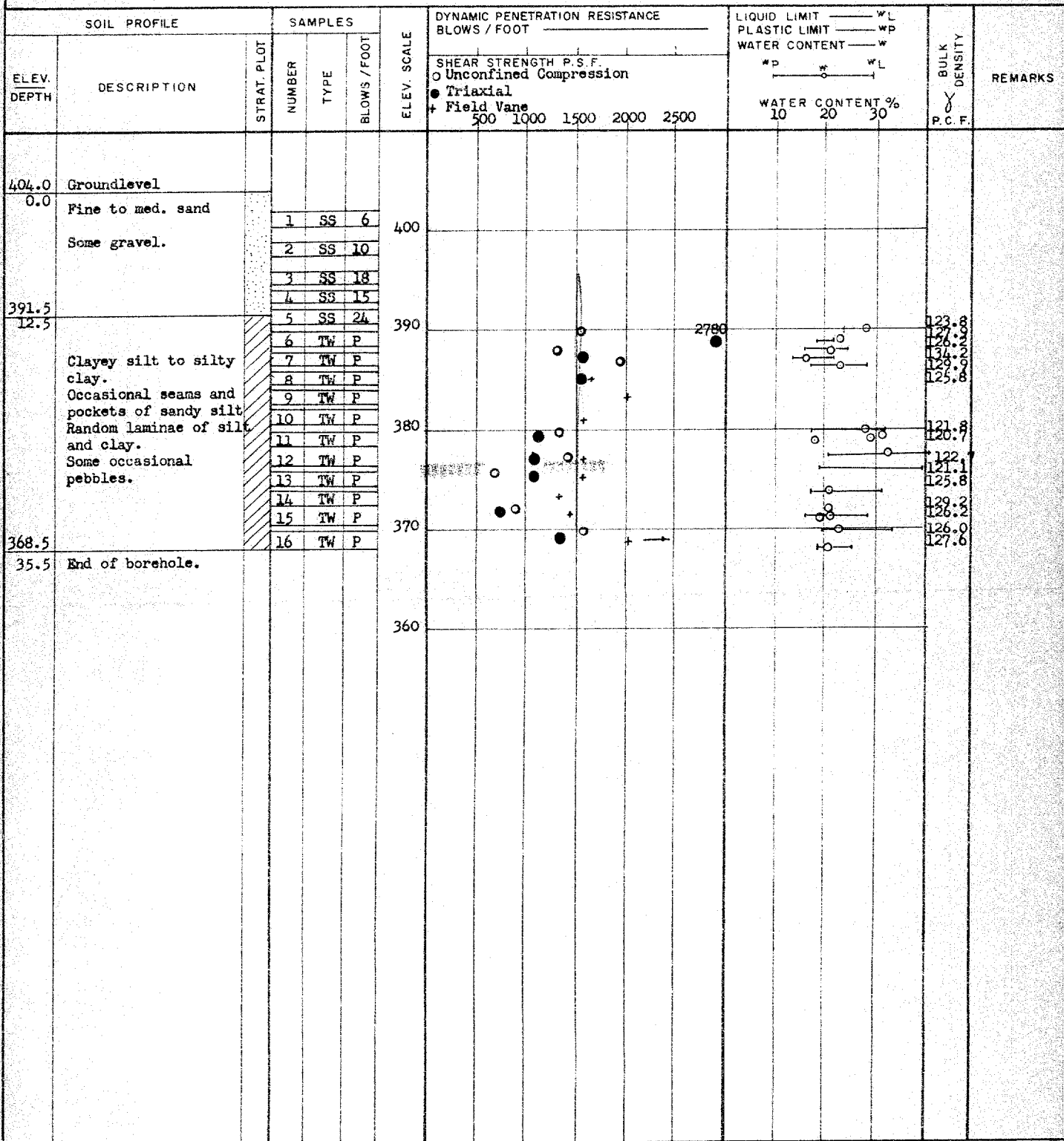
BORING DATE March 20, 1963.

COMPILED BY H.S.

DATUM Geodetic

BOREHOLE TYPE Washboring - NX Casing.

CHECKED BY K.S.



JOB 62-F-51 LOCATION Sta. 15+00 18' Rt. E Jane St. ORIGINATED BY H.S.
W.P. 232-60 BORING DATE April 6, 1963. COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Washboring. CHECKED BY K.S.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W		BULK DENSITY P C F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	WATER CONTENT %			
404.0	Groundlevel										
	Clayey silt with some sand and gravel- loose		1	SS	5	400					
	(Probably fill)		2	SS	5						
394.0			3	SS	9						
10.0	Silt to clayey silt with traces of organics stiff to very stiff.		4	SS	20	390					
387.5			5	SS	31						
16.5	End of borehole.					380					

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

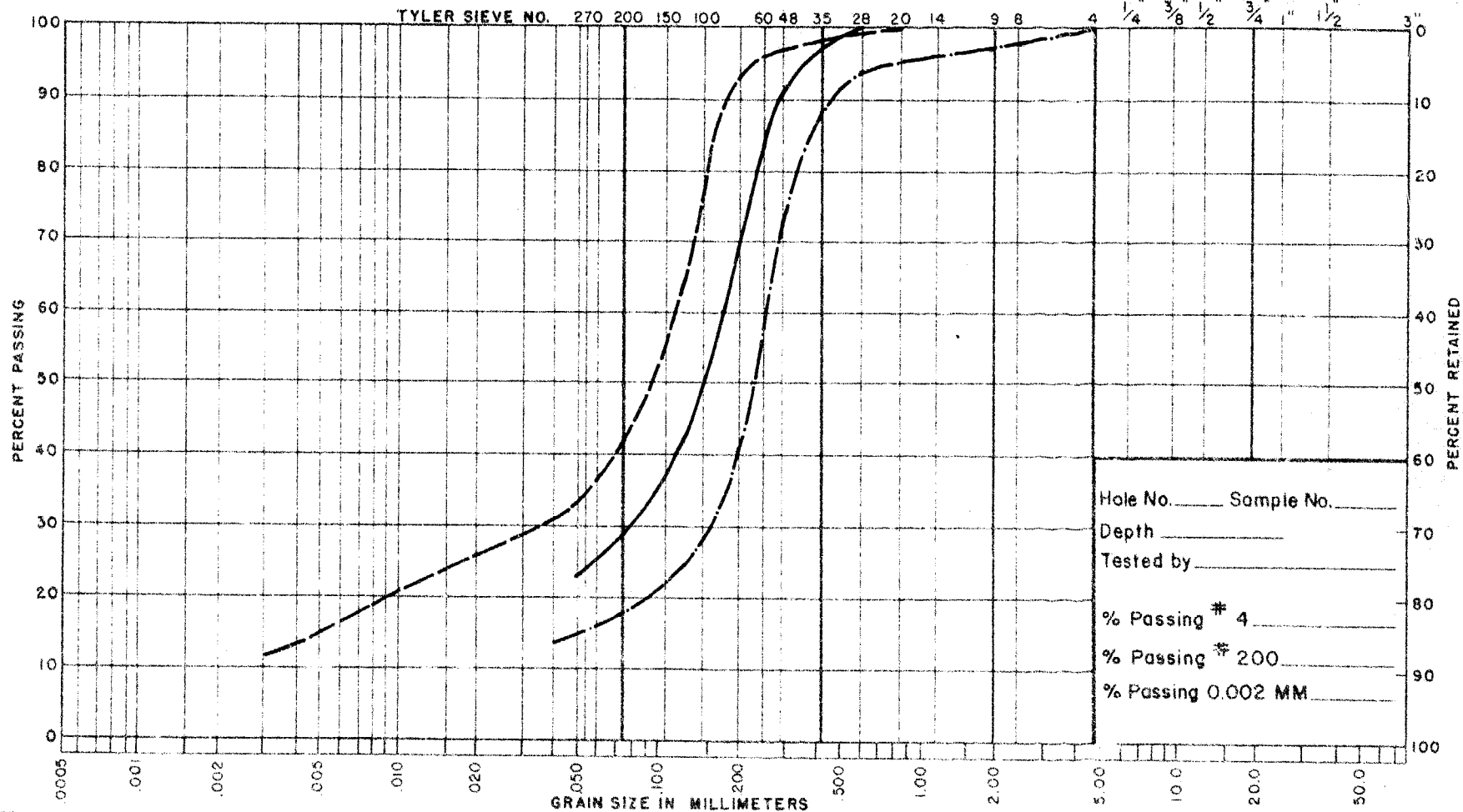
Fine

Medium

Coarse

Fine

Coarse



NOTES

BOREHOLE NO. 3, SAMPLE NO. 1 _____

BOREHOLE NO. 2, SAMPLE NO. 1 _____

BOREHOLE NO. 3, SAMPLE NO. 2 _____

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
GRAIN SIZE DISTRIBUTION

Job No. 62-F-51

W.P. No. _____

Location _____

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

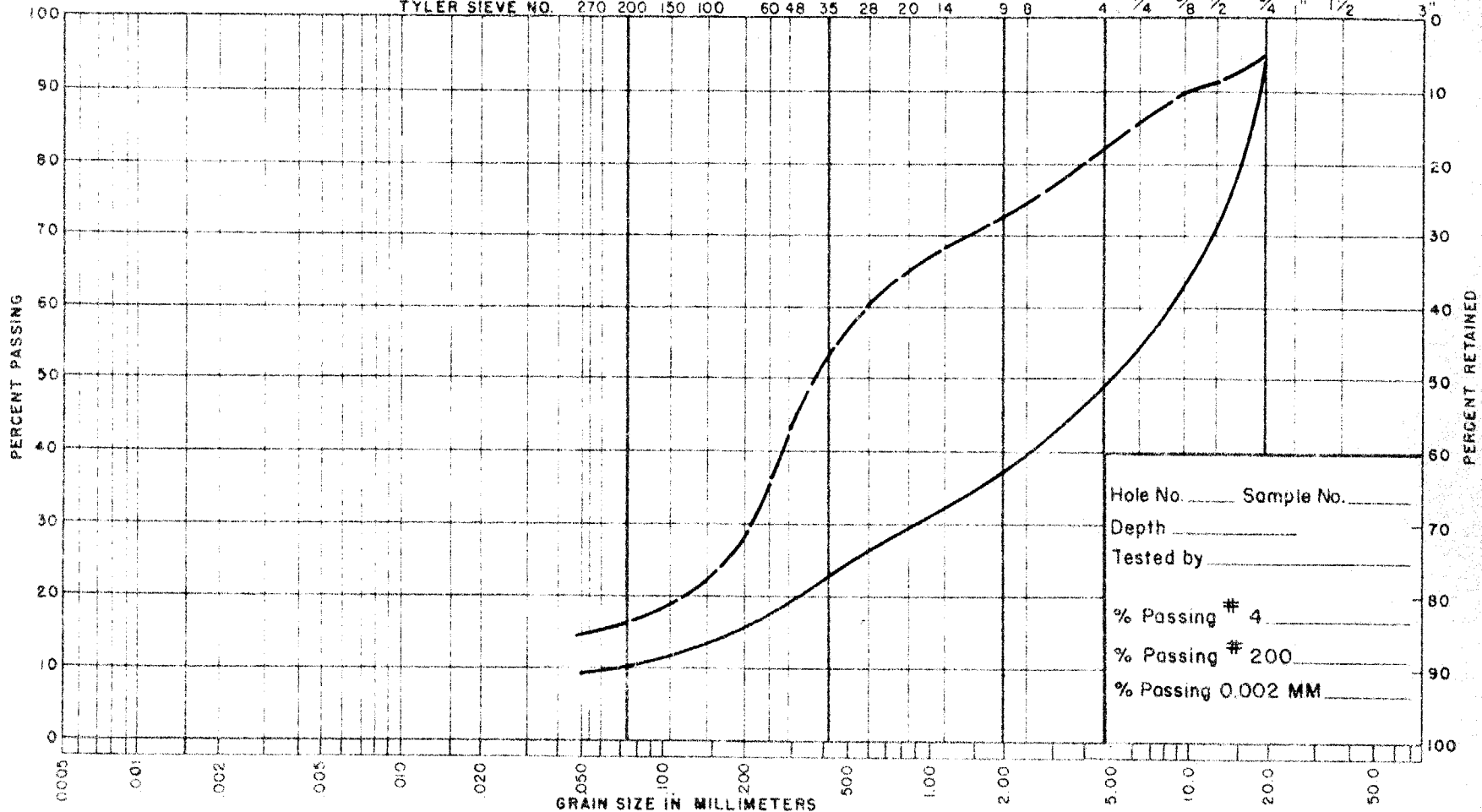
Medium

Coarse

Fine

Coarse

TYLER SIEVE NO. 270 200 150 100 60 48 35 28 20 14 9 8 4 1/4" 3/8" 1/2" 3/4" 1" 1 1/2" 3"



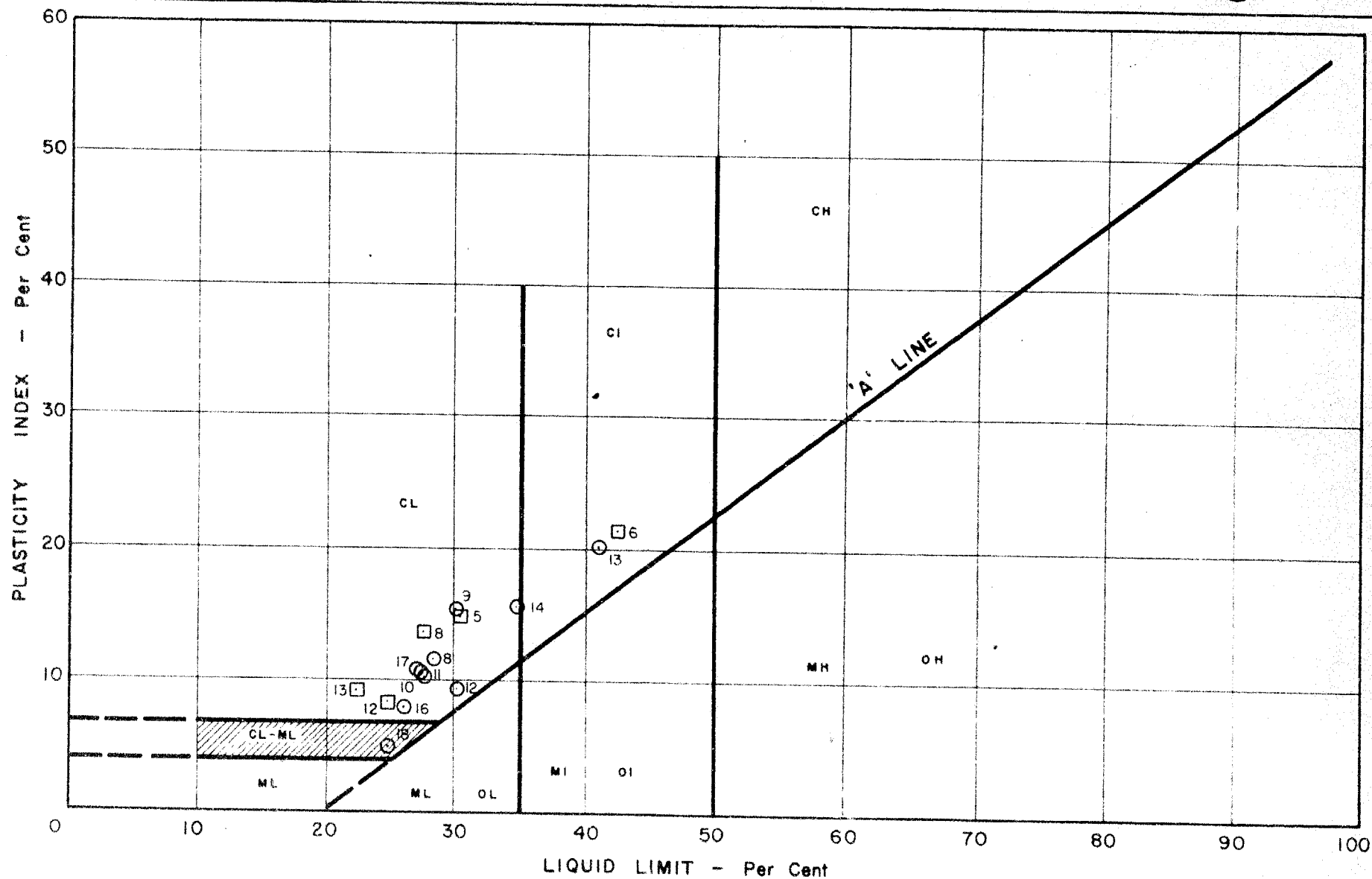
NOTES BOREHOLE NO. 4, SAMPLE NO. 3 _____

BOREHOLE NO. 1, SAMPLE NO. 3 _____

DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH SECTION
GRAIN SIZE DISTRIBUTION

Job No. 62 - F - 51 W.P. No. _____

Location _____



NOTES

BOREHOLE NO. 1 — ○

BOREHOLE NO. 2 — □

SAMPLE NUMBERS INDICATED BESIDES THE POINTS

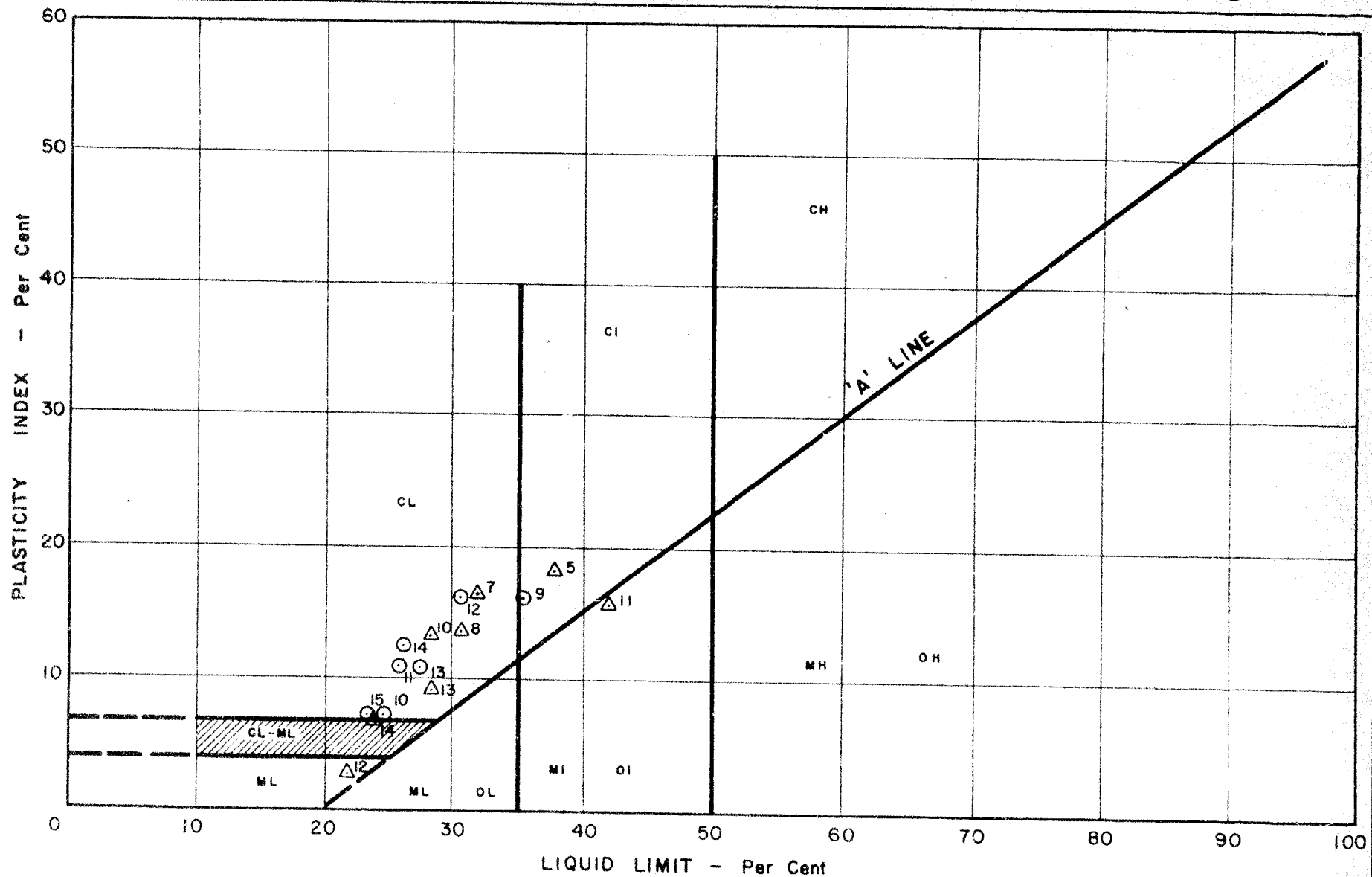
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH DIVISION

PLASTICITY CHART

Job No. 62 - F - 51 W.P. No. 232 - 60

Location JANE & 401. TORONTO



NOTES B.H. NO. 3 — ○
 B.H. NO. 4 — △
 Sample numbers indicated besides the points

DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH DIVISION
 PLASTICITY CHART

Job No. 62-F-51 W.P. No. 232-60
 Location JANE & 401, TORONTO

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 :- 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

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE	ADVANCED HYDRAULICALLY
	P.M.	SAMPLE	ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	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_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
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_t	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

Contract # 23-63-182

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. B. R. Davis

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division,
May 28, 1963.

D.H.O. REPORT ON PILE LOADING TESTS AT -
Hwy. #401 & Jane Street, Toronto, Dist. #6.
W.J. 62-F-51 B -- W.P. 232-60

Attached, we are forwarding to you, our report on the results of pile loading tests carried out at the site of the proposed new bridge over Jane Street and Hwy. 401, Toronto By-Pass, Toronto.

We feel that the information contained in the report will be sufficient for your design purposes. If you have any further queries in connection with this matter, please do not hesitate to call our Office.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
G. K. Hunter (2)
C. Fraser
T. J. Kovich
A. Watt
Foundations Office
Gen. Files

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

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 4. PILE DRIVING.
 5. PILE TESTING.
 6. DISCUSSION OF TEST RESULTS.
 7. CONCLUSIONS.
 8. MISCELLANEOUS.
-

REPORT ON PILE LOADING TESTS AT

Hwy. #401 & Jane Street
W.P. 232-60 -- W.J. 62-F-51

1. INTRODUCTION:

The Foundation Section was requested by Mr. C. Grebski, Senior Bridge Project Engineer to initiate and subsequently supervise pile loading tests, at the site of the proposed new overpass at Hwy. #401 and Jane Street Toronto By-Pass. This request was made verbally on March 1st, 1963.

Following the above request arrangements were made to load two test piles at the site these being:-

- (1) A 35' long class 'A' timber pile,
- and (2) A 24' long, 22" dia., Franki Displacement Caisson.

This work was carried out by Franki Pile of Canada Ltd., under W.O. #36321.

The report which follows contains the detailed results of the pile driving records and the pile loading tests together with our recommendations pertaining to the particular design loads to be used for each type of pile. (The latter were previously given in a letter dated April 9th, 1963.)

2. DESCRIPTION OF SITE:

The site is located in the Township of North York in Metropolitan Toronto at the existing intersection of Hwy. #401 and Jane Street. The present bridge over Jane St. is a single

cont'd. /2 ...

2. DESCRIPTION OF SITE: (cont'd.)...

span structure supported on spread footings founded at or about elev. 397.0. The approach embankments to the structure are approximately 20' in height. The flood plain of the adjacent Black Creek extends for about 350' on each side of Jane Street, though the present water course of the creek is located some 300' east of Jane Street.

Physiographically, the site lies in the South Slope Region which is the south slope of an interbolate moraine. This moraine consists of a clayey till deposited during the Pleistocene Ice Epoch.

3. SUBSOIL CONDITIONS:

The subsoil at the site consists of fluvial deposits ranging from sand and gravel to silty clay. The upper layers consist of sand and gravel deposits of thickness varying from about 12' to 20'. The most unfavourable location was chosen for the pile tests and here the subsoil consisted of about 12' of compact sand followed by irregularly stratified layers of silt, clayey silt and silty clay. Vane tests carried out in the latter deposit gave results ranging from 1120 to 2000 p.s.f. down to a depth of 35.0'. Two log sheets for B.H. #5A and B.H.#6 which were carried out adjacent to the piles tested are included in the Appendix of this report. For a complete description of the subsoil conditions over the entire site

cont'd. /3 ...

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

the reader is referred to Foundation Report #62-F-51. The locations of the piles and the boreholes are shown on Dwg. #62-F-51B which is also included in the Appendix of this report.

4. PILE DRIVING:

(1) Franki Displacement Concrete Caisson:-

This pile was driven following the standard Franki procedure. A 22 inch \varnothing steel tube was advanced about 21' by means of a 7000 lb. drop hammer with a free fall of about 20 feet. A 40-inch diameter concrete base was then placed below the tube and a concrete shaft formed as the tube was withdrawn. Driving of this pile took place on March 14th, 1963, the total time of driving being about 12 minutes. All records pertaining to the driving of this pile are included in the Appendix of this report.

(2) Class 'A' Timber Pile:-

This pile was driven using the same equipment as for the Franki pile, somewhat modified. The pile was advanced by means of a 3000 lb. drop hammer with a fall of about 8' through a 10' steel follower clamped to the pile butt. The length of the pile was 35' and it was advanced to a depth of about 37' below ground level. Driving took place on March 14th and 15th, 1963, total time of driving being about 67 minutes. It is important to note that the driving time of 67 minutes is

cont'd. /4 ...

4. PILE DRIVING: (cont'd.)...

(2) Class 'A' Timber Pile:-

regarded as excessive and entirely due to the somewhat inefficient method of driving using equipment not properly designed for the purpose of driving this type of pile. In addition a breakdown occurred at 4:15 P.M. 14th of March and caused a delay in operations of several hours. It is estimated that using proper equipment this pile could have been driven in about 10 to 15 minutes. All records pertaining to the driving of this pile are included in the Appendix of this report.

5. PILE TESTING:

In general the loading tests were carried out in accordance with the National Building Code of Canada. The loads were applied by means of a jack acting between the pile and a reaction beam attached to a box structure containing about 250 tons of sand. Two tests were carried out on each pile, the first being a slow test carried out to a particular load over a period of about 45 hours and the second a quick test carried out to failure of the pile in a period of about 1 to 2 hours. The exact sequence of events for each pile is shown on the Load Test Records included in the Appendix of this report and is summarized as follows:

FRANKI PILE

The first test commenced at 8:30 A.M. on March 28th, 1963. The pile was loaded in increments of 20 tons up to

5. PILE TESTING: (cont'd.)...

FRANKI PILE

100 tons in about 5 hours and this load was maintained for a period of 25 hours. The load was then removed in increments over a period of 4 hours. Gross and net settlements for this pile were 0.872" and 0.672" respectively.

The second test commenced at 10:00 A.M. March 30th '63. The pile was loaded rapidly up to 100 tons then in increments of 10 tons until the point was reached at which no further load could be applied due to failure of the pile. This occurred after 55 minutes, the maximum load achieved being 150 tons. The load was then removed in increments in 14 minutes.

CLASS 'A' TIMBER PILE

The first test commenced at 12:17 P.M. March 30th '63. The load increments of 5 tons were applied to the pile reaching a maximum value of 50 tons in 2 hours 50 minutes. This load was then maintained for 24 hours then reduced by increments to zero in a further 20 minutes. Gross and net settlements for this pile were 0.196" and 0.036" respectively.

The second test commenced at 4:40 P.M. March 31st '63. The pile was loaded fairly rapidly in increments till no further increase in loading was possible due to failure of the pile. Total time of loading in this case was 73 minutes and the maximum load reached was 105 tons. The load was then reduced to zero in a further 8 minutes.

cont'd. /6 ...

5. PILE TESTING: (cont'd.)...

CLASS 'A' TIMBER PILE

For each test the following curves have been plotted:-

- (1) Load versus Time.
- (2) Settlement versus Time.
- (3) Settlement versus Load.

These curves are included in the Appendix of this report.

6. DISCUSSION OF TEST RESULTS:-

The loading tests described in the previous paragraphs were carried out for the purpose of determining an appropriate design load for each type of pile. It is usual to utilize a safety factor of 2.0 for design purposes when a pile loading test has been carried out. The slow tests carried out on the Franki Pile and the Timber Pile up to loads of 100 tons and 50 tons respectively gave results which indicated that design loads of at least 50 tons and 25 tons respectively may be used according to the provisions of the National Building Code of Canada. The tests to failure however, indicated that the above loadings might be somewhat conservative particularly in the case of the timber pile. The problem of analyzing load tests carried out to beyond the failure point becomes one of determining the actual failure load. This is sometimes defined as being the load at which further increase of load brings

cont'd. /7 ...

6. DISCUSSION OF TEST RESULTS:- (cont'd.)...

about an excessive rate of settlement. Load/Settlement curves do not usually show a clearly defined break at which the above phenomenon occurs, rather do they show a range in which it is possible to discern the trend towards failure. An examination of the Load/Settlement Curves in question for the two piles showed that in the case of the Franki Pile the initial stage of failure probably occurred at or about 125 tons whilst in the case of the Timber Pile failure probably occurred in the region of 80 tons.

The theoretical ultimate capacities of the piles have been calculated on the basis of full mobilization of shear strength of the cohesive stratum. This latter has been taken as 1600 p.s.f. and is an average of in-situ vane tests carried out within the stratum. The results of the calculations are as follows:-

(1) Franki Pile

$$Q = 9 \text{ } Ab \text{ } C + As \text{ } Ca \quad \text{where.}$$

$$\begin{aligned} Q &= \text{Ultimate capacity of pile.} \\ Ab &= \text{Area of base of pile} &= 8.78 \text{ sq. ft.} \\ C &= \text{Cohesion of soil at base of pile} &= 1600 \text{ p.s.f.} \\ As &= \text{Effective area of shaft of pile} &= 113.1 \text{ sq. ft.} \\ Ca &= \text{Adhesion of soil} &= 1600 \text{ p.s.f.} \end{aligned}$$

$$\begin{aligned} \text{Hence } Q &= \frac{9 \times 8.78 \times 1600}{2000} + \frac{113.1 \times 1600}{2000} \text{ tons.} \\ &= 63.3 + 90.5 = 153.8 \text{ tons.} \end{aligned}$$

cont'd. /8 ...

6. DISCUSSION OF TEST RESULTS:- (cont'd.)...

(2) Timber Pile

$$Q = 9 \text{ } A_b C + A_s C_a \quad \text{where.}$$

Q = Ultimate capacity of pile.

A_b = Area of base of pile. = 0.443 sq. ft.

C = Cohesion of soil at base of pile = 1600 p.s.f.

A_s = Effective area of shaft = 89.4 sq. ft.

C_a = Adhesion of soil = 1600 p.s.f.

$$\text{Hence } Q = \frac{9 \times 0.443 \times 1600}{2000} + 89.4 \times \frac{1600}{2000} \text{ tons.}$$

$$= 3.2 + 71.6 = 74.8 \text{ tons.}$$

As a result of the foregoing analyses it has been decided to recommend to following maximum design loads for the particular piles with the dimensions used in the tests:-

Franki Pile:- 60 tons. S.F. = 2.0

Timber Pile:- 40 tons. S.F. = 2.0

During a discussion with Franki Piling Co. the latter informed us that the maximum load that they would recommend for the Franki Pile would be 50 tons. This appears to us to be slightly conservative and in our opinion this value could safely be increased to 60 tons.

7. CONCLUSIONS:

The results of loading tests on a 22" Ø 24' long Franki Displacement Caisson, and a 35' long class 'A' Timber Pile are reported. The tests were carried out at the site of the proposed new Hwy. #401 and Jane St. Overpass. Subsoil consists of about 12' of compact sand followed by an extensive layer of stiff to very stiff silty clay to clayey silt.

The test results are compared with the theoretical ultimate capacities computed assuming full mobilization of the shear strength of the cohesive soil. The comparisons are reasonably favourable.

As a result of the tests design loads of 40 tons for the Timber Pile and 60 tons for the Franki are recommended. A safety factor of 2.0 has been assumed.

8. MISCELLANEOUS:

The pile driving and subsequent load tests were carried out during the period March 14th to March 31st, 1963, by Franki Pile of Canada Ltd., under the supervision of the Department of Highways Foundation Section. This report was prepared by Mr. K. G. Selby.

May 1963.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION
FOUNDATION SECTION

BRIDGE CONSTRUCTION - PILE DRIVING RECORD

OVERPASS

DISTRICT NO. 6 ^{N.P.} CONTRACT NO. 232-60 STRUCTURE HWY. NO 401 & JANE ST.

CONTRACTOR FRANKI PILE CO. DESIGN LOAD OF PILE TO BE DETERMINED

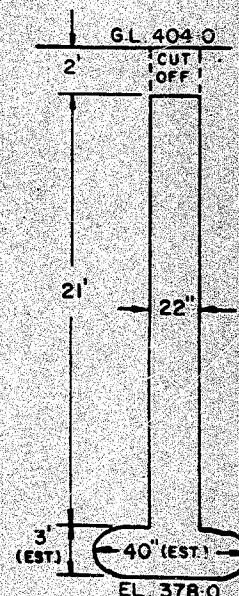
HAMMER DETAILS: TYPE DROP HAMMER WEIGHT 7,000 HEIGHT OF FALL OR ENERGY 15'

TYPE OF ANVIL OR CAP STANDARD FRANKI WEIGHT OF ANVIL OR CAP NOT APPLICABLE

PILE DETAILS FRANKI PILE

PILE NO. 1 LOCATION STA. 13+45 93' RT. & JANE ST. DATE DRIVEN 14 MAR. 63

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.
// 55 AM	1	EXCAVATED		26			51	
	2			27			52	
	3	2		28			53	
	4	4		29			54	
	5	3		30			55	
	6	5		31			56	
	7	4		32			57	
	8	7		33			58	
	9	9		34			59	
	10	8		35			60	
	11	6		36			61	
	12	6		37			62	
	13	5		38			63	
	14	5		39			64	
	15	5		40			65	
	16	5		41			66	
	17	4		42			67	
	18	5		43			68	
	19	5		44			69	
	20	5		45			70	
	21	5		46			71	
	22	5		47			72	
1207 PM	23	6		48			73	
	24			49			74	
	25			50			75	



DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES	NOT RECORDED					
FINAL LENGTH OF PILE <u>23'</u>	FINAL CUT OFF ELEVATION <u>402.0'</u>					

REPORT TO BE SENT TO: - PRINCIPAL FOUNDATION ENGINEER
MATERIALS & RESEARCH DIVISION
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS
TORONTO, ONTARIO

SIGNED H. Szymanski
NAME (PRINT) H. SZYMANSKI

DATE 14 MARCH 1963

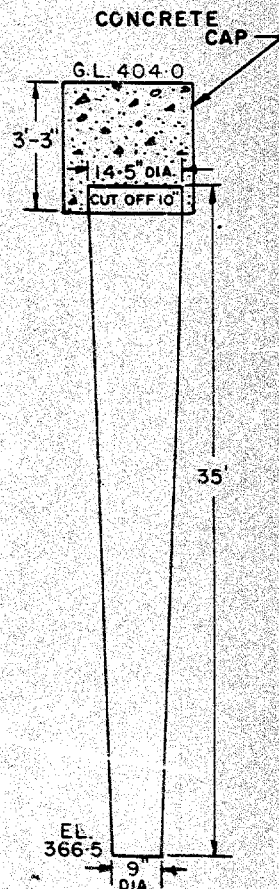
ATTACH SKETCH OF PILE NUMBERING SYSTEM

BRIDGE CONSTRUCTION - PILE DRIVING RECORD OVERPASS

DISTRICT NO. 6 ^{W.P.} CONTRACT NO. 232-60 STRUCTURE HWY. NO. 401 & JANE ST.
CONTRACTOR FRANKI PILE CO. DESIGN LOAD OF PILE TO BE DETERMINED
HAMMER DETAILS: TYPE DROP HAMMER WEIGHT 3000 HEIGHT OF FALL OR ENERGY 8' & 6"
TYPE OF ANVIL OR CAP STEEL FOLLOWER WEIGHT OF ANVIL OR CAP 30 LB/FT. X 10' = 300 LB/FT.
PILE DETAILS CLASS 'A' TIMBER PILE

PILE NO. 2 LOCATION STA. 13+52 93' RT. & JANE ST. DATE DRIVEN 14 MAR. 63

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.
	1			26	41		51	
	2			27	29		52	
	3			28	31		53	
	4			29	18		54	
	5			30	19		55	
	6			31	18		56	
3:50 PM	7 1/2			32	16		57	
	8			33	21		58	
	9			34	25		59	
	10			35	28		60	
	11			36	30		61	
	12			37	31		62	
	13			38			63	
	14			39			64	
	15			40			65	
	16			41			66	
	17			42			67	
	18			43			68	
	19			44			69	
	20			45			70	
	21			46			71	
	22			47			72	
	23			48			73	
	24			49			74	
4:15 PM	25			50			75	



DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	3	3	3	3	3	3
MEASURED REBOUND IN INCHES	1	1	1	1	1	1
FINAL LENGTH OF PILE <u>34' 2"</u>	FINAL CUT OFF ELEVATION <u>400.75</u>					

REPORT TO BE SENT TO: - PRINCIPAL FOUNDATION ENGINEER
MATERIALS & RESEARCH DIVISION
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS
TORONTO, ONTARIO

SIGNED H. Szymanski
NAME (PRINT) H. SZYMANSKI
DATE 15 MARCH 1963

ATTACH SKETCH OF PILE NUMBERING SYSTEM

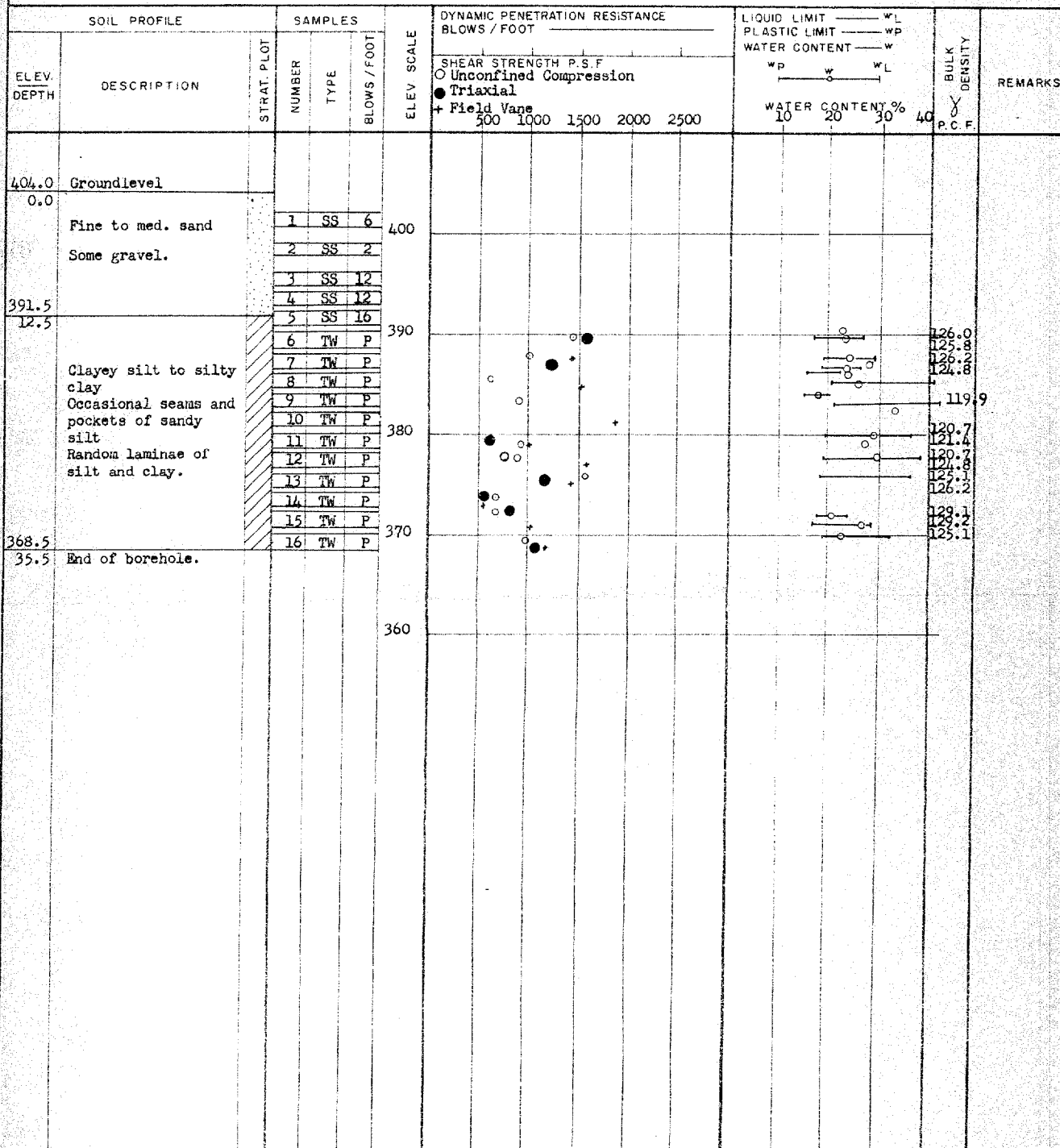
APPENDIX I.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 5A

FOUNDATION SECTION

JOB 62-F-51 LOCATION Sta. 13+49 94' Rt. 6 Jane St. ORIGINATED BY H.S.
W.P. 232-60 BORING DATE April 3, 1963. COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Washboring - NX Casing. CHECKED BY K.S.

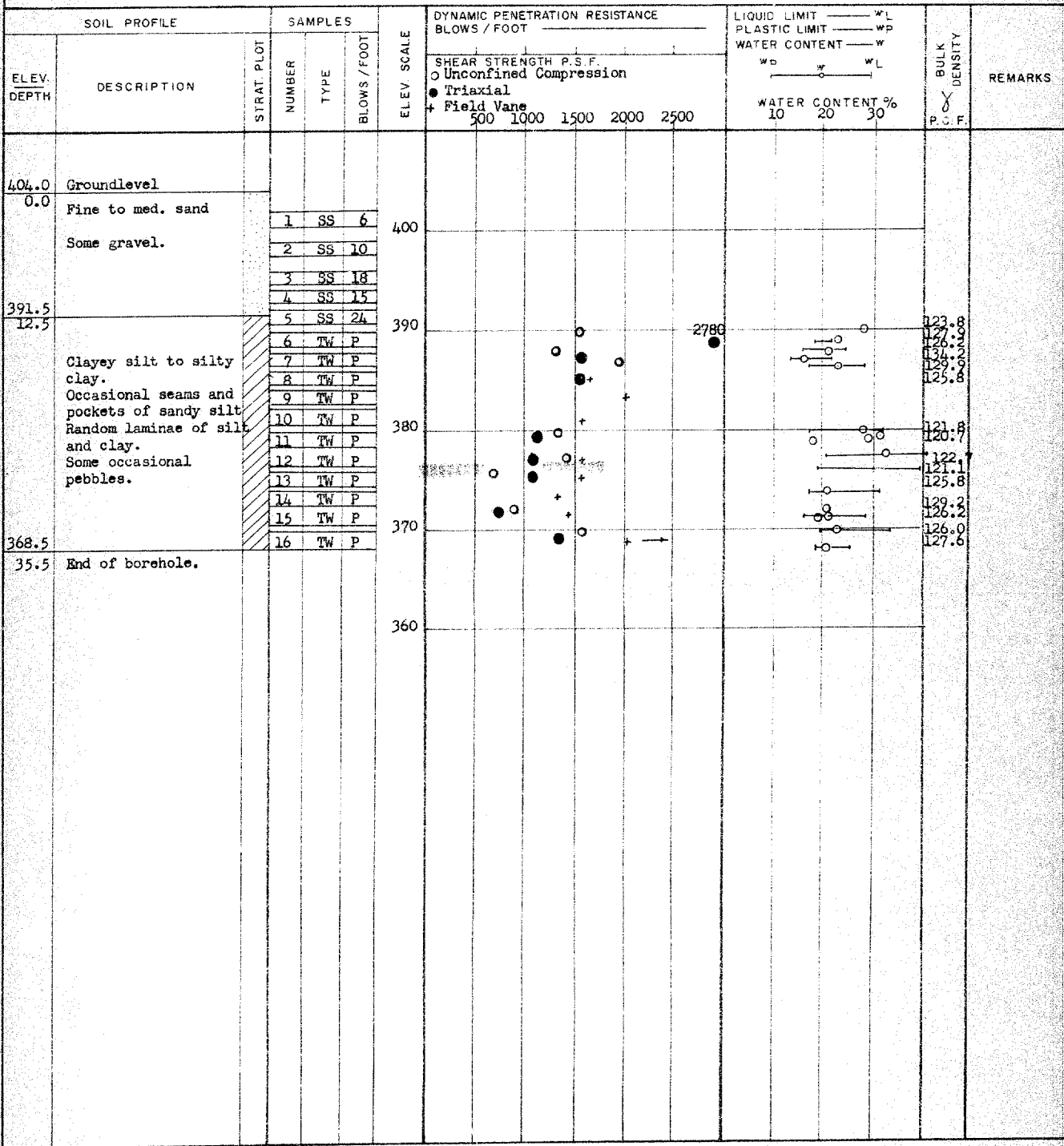


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 6

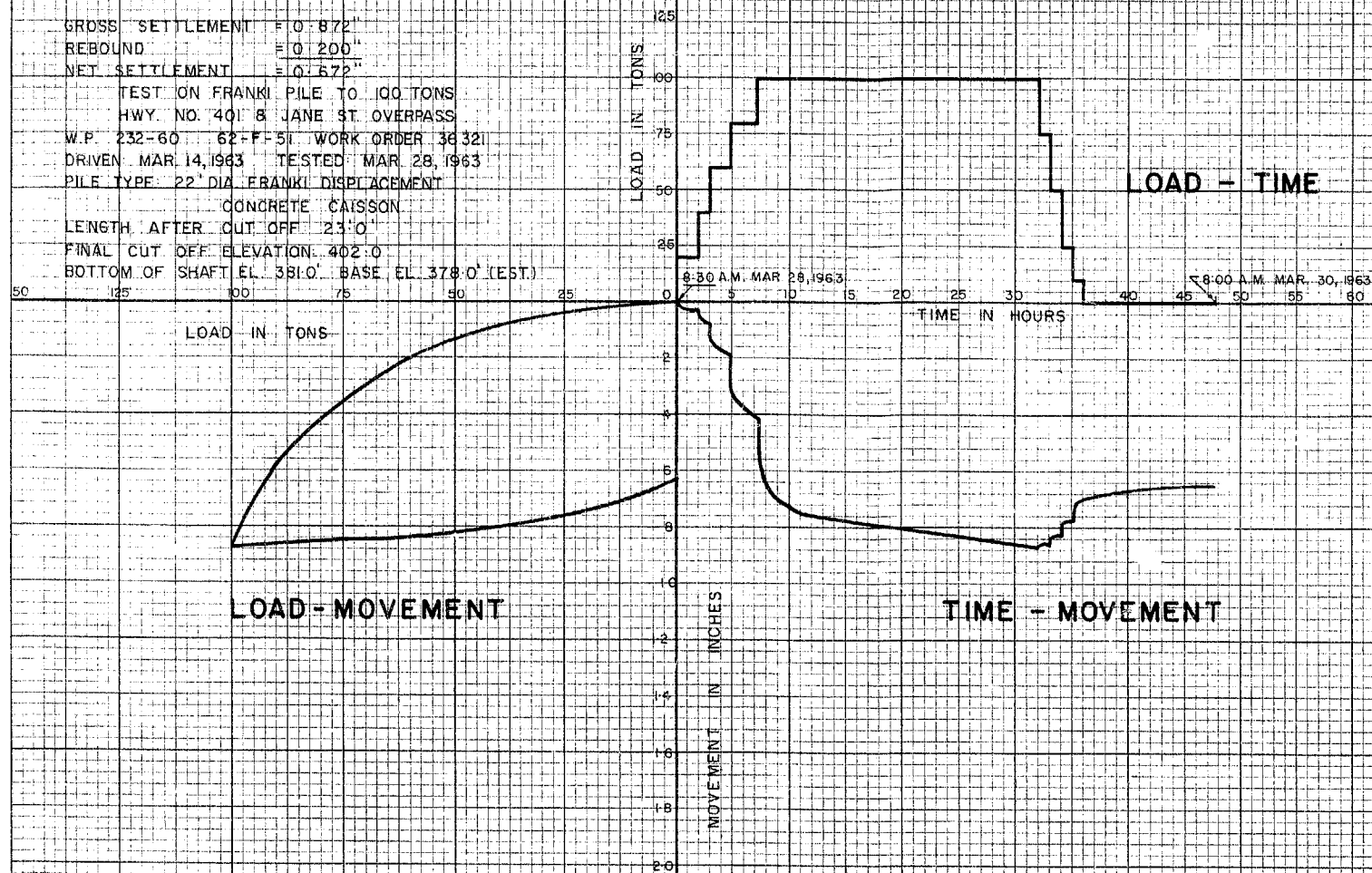
FOUNDATION SECTION

JOB 62-F-51 LOCATION Sta. 13+23 93' Rt. & Jane St. ORIGINATED BY H.S.
W.P. 232-60 BORING DATE March 20, 1963. COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Washboring - NX Casing. CHECKED BY K.S.



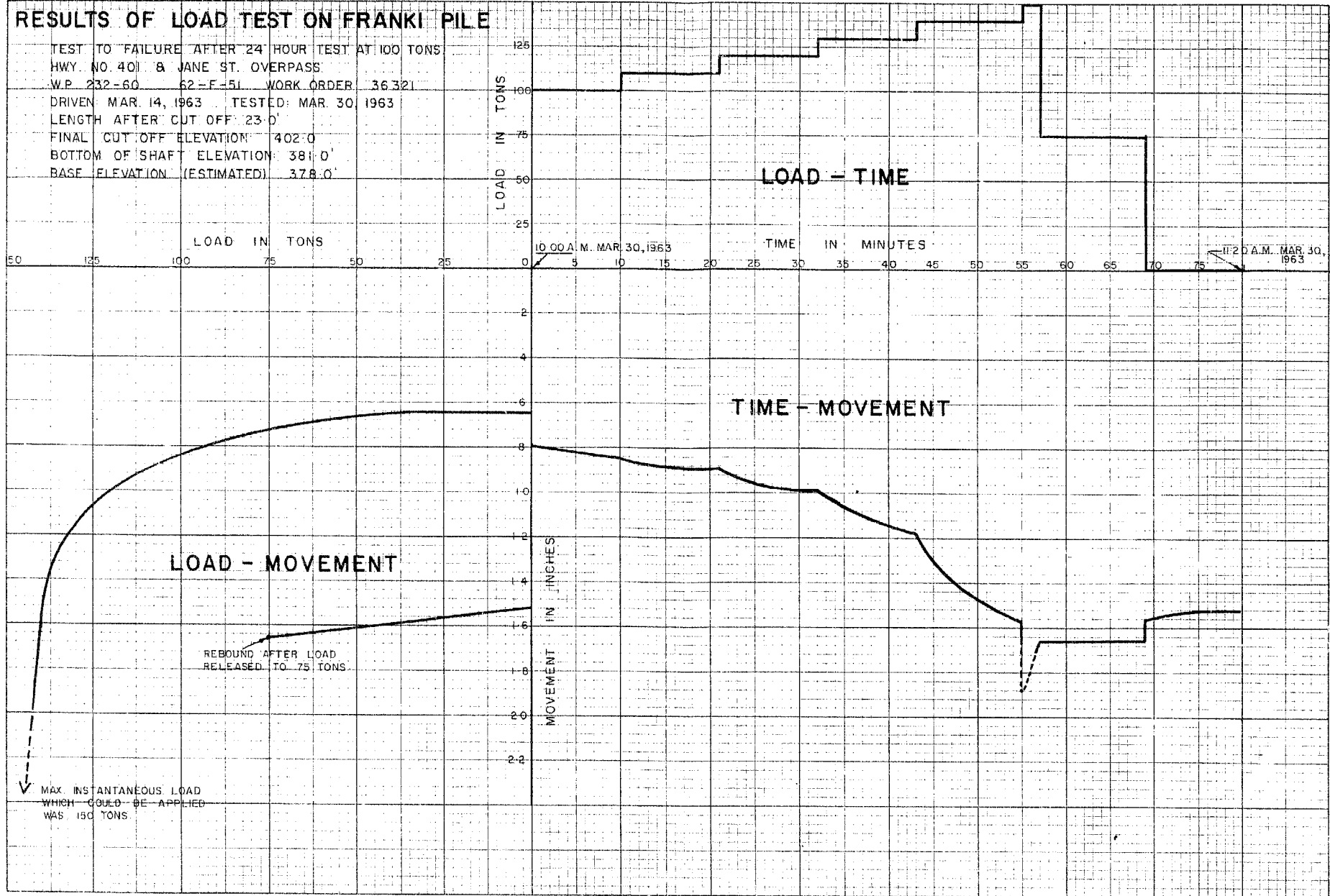
K&L **REPLIST DIEDEN CO** **WILSON & A**
10 X 10 TO THE INCH **320-2DLC**

TEST ON FRANKI PILE TO 100 TONS
HWY. NO. 401 & JANE ST OVERPASS
W.P. 232-60 62-F-51 WORK ORDER 38321
DRIVEN MAR. 14, 1963 TESTED MAR. 28, 1963
PILE TYPE 22" DIA FRANKI DISPLACEMENT
CONCRETE CAISSON
LENGTH AFTER CUT OFF 23'0"
FINAL CUT OFF ELEVATION 402.0
BOTTOM OF SHAFT EL. 381.0 BASE EL. 378.0



RESULTS OF LOAD TEST ON FRANKI PILE

TEST TO FAILURE AFTER 24 HOUR TEST AT 100 TONS
 HWY. NO. 401 & JANE ST. OVERPASS
 W.P. 232-60 62-F-51 WORK ORDER 36321
 DRIVEN MAR. 14, 1963 TESTED: MAR. 30, 1963
 LENGTH AFTER CUT OFF 23'-0"
 FINAL CUT OFF ELEVATION 402'-0"
 BOTTOM OF SHAFT ELEVATION 381'-0"
 BASE ELEVATION (ESTIMATED) 378'-0"



NOT TO SCALE TO THE INCH 320-2070

RESULTS OF LOAD TEST ON TIMBER PILE

GROSS SETTLEMENT = .196
REBOUND = .160
NET SETTLEMENT = .036

TEST ON TIMBER PILE TO 50 TONS
HWY. NO. 401 & JANE ST. OVERPASS
W.P. 232-60 62-F-51 WORK ORDER NO. 36321
DRIVEN MAR. 14, 1963 TESTED MAR. 30, 1963
PILE TYPE TIMBER PILE TOP DIA 14.5" BOTT DIA 9"
LENGTH AFTER CUT OFF 34.2'
FINAL CUT OFF ELEVATION 400.75'
TIP ELEVATION 366.50'

LOAD IN TONS

LOAD - TIME

TIME IN HOURS

MOVEMENT IN INCHES

LOAD - MOVEMENT

TIME - MOVEMENT

NOTED BY: J. H. CO. 320-2070

RESULTS OF LOAD TEST ON TIMBER PILE

TEST TO FAILURE AFTER 24 HOUR TEST AT 50 TONS
HWY. NO. 401 @ JANE ST. OVERPASS
W.P. 232 - 60 62-F-51 WORK ORDER NO. 3632
DRIVEN - MAR. 16, 1963 TESTED - MAR. 31, 1963
PILE TYPE: TIMBER TOP DIA. 14.5" BOTT. DIA. 9"
LENGTH AFTER CUT OFF: 34' 2"
FINAL CUT OFF ELEVATION: 400.75'
TIP ELEVATION: 386.50

