

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M3-178

DIST. 4 REGION

W.P. No. 90-74-03

CONT. No. 86-102

W. O. No.

STR. SITE No. 18-175

HWY. No. 406

LOCATION Chestnut St. Overpass
S.B. Structure

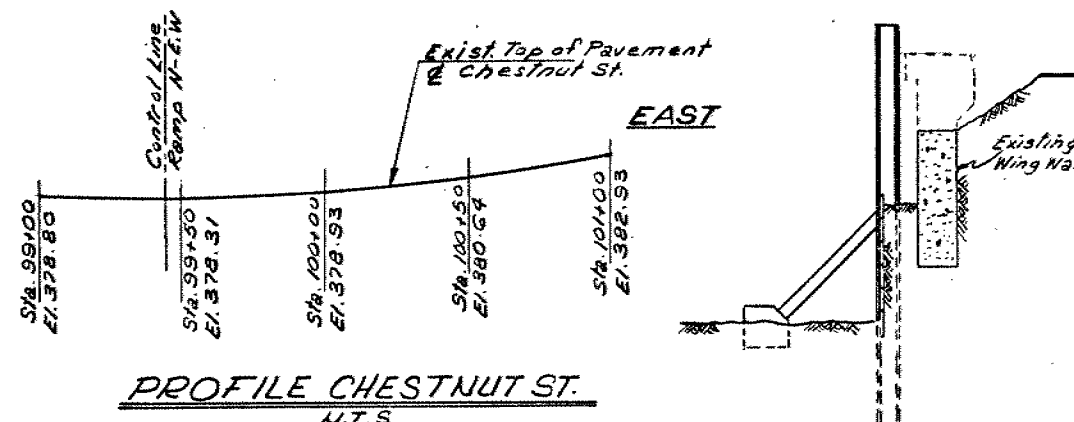
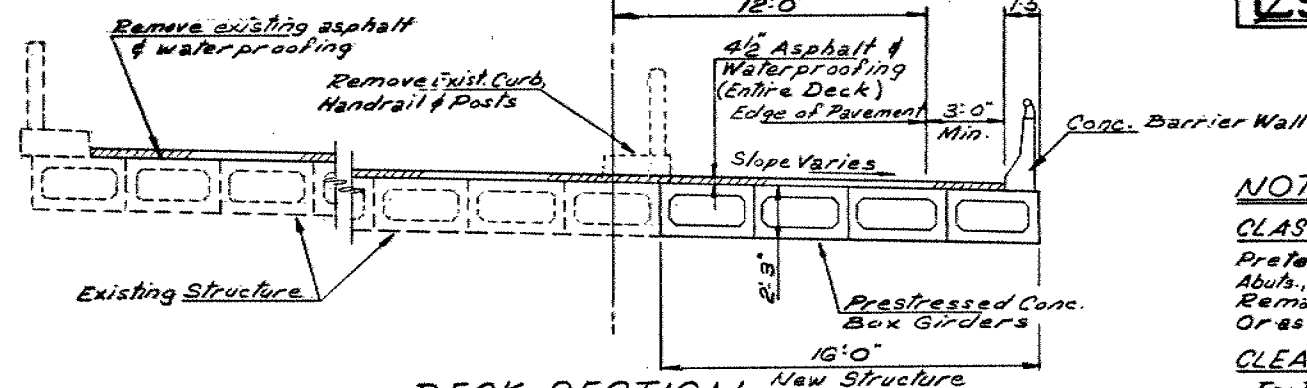
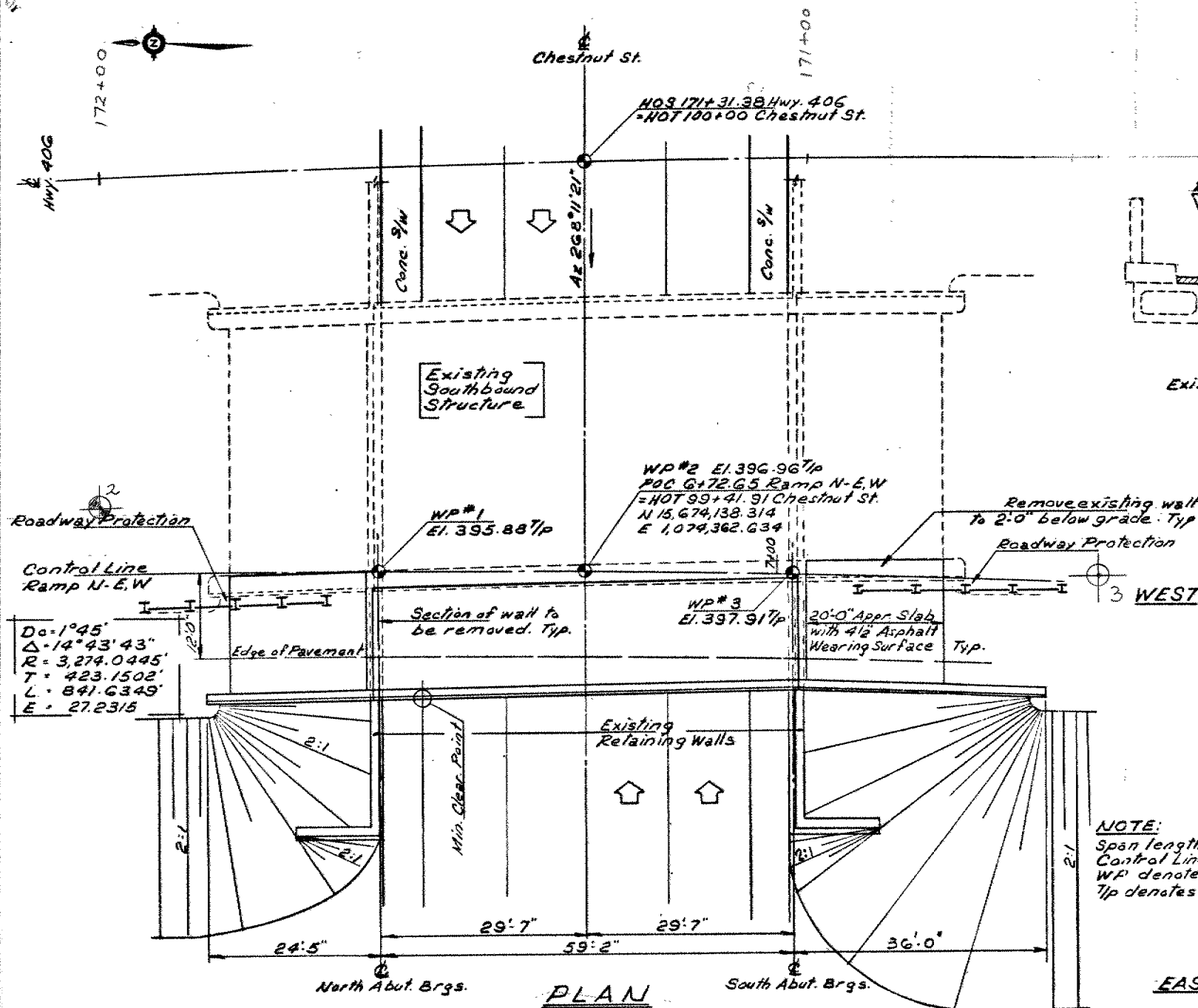
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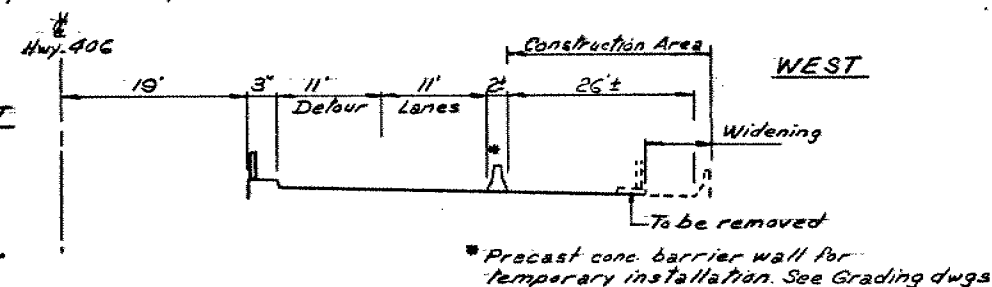
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976



SUGGESTED RDWY. PROTECTION



NOTES:

CLASS OF CONCRETE

Prestensioned Box Girders... 35 MPa.
Abut., Wing Walls & Barrier Wall... 30 MPa.
Remainder... 20 MPa.
Or as noted on the drawings.

CLEAR COVER TO REINFORCING

Footings... 3"
Abutments... 2"
Approach Slabs... 2"
Or as noted on the drawings.

REINFORCING STEEL

Grade 400. Bars with the designation
"C" shall be coated bars.

CONSTRUCTION NOTES

The Contractor is responsible for finishing
the bearing seats to the specified
elevations and slopes with a tolerance
of $\pm 1/8$ inch.

CONSTRUCTION SEQUENCE

1. Detour traffic and construct roadway protection.
2. Carry out demolition of Ret. & Wing Walls and Curb.
3. Constr. abut., install beams and backfill.
4. Strip asphalt & waterproofing - Stage 1 existing.
5. Install new seal, waterproof & asphalt (3") Stage 1.
6. Move traffic to west side of bridge Stage 2.
7. Strip asphalt & waterproofing - Stage 2 existing.
8. Install new seal, waterproof & asphalt (3") Stage 2.
9. Apply surface course of asphalt (1 1/2").

LIST OF DRAWINGS

1. General Layout
2. Borehole Locations & Soil Strata
3. Exist. Bridge Removal Details
4. Footing Layout & Details
5. Abutment Details
6. Prestensioned Box Beams
7. Barrier Wall
8. Railing For Barrier Wall
9. 20 Ft. Approach Slab
10. Standard Details I
11. Standard Details II
12. As Constructed Elevs. & Dim.

CONCRETE QUANTITIES

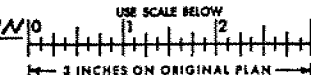
Concrete quantities are listed below
for the appropriate concrete lump
sum tender items.

1. Conc. in abut. & wing walls 10 cu. yds.
2. Conc. in barrier walls 11 cu. yds.
3. Conc. in approach slabs 21 cu. yds.

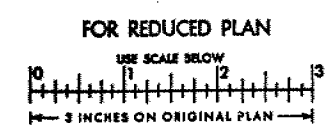
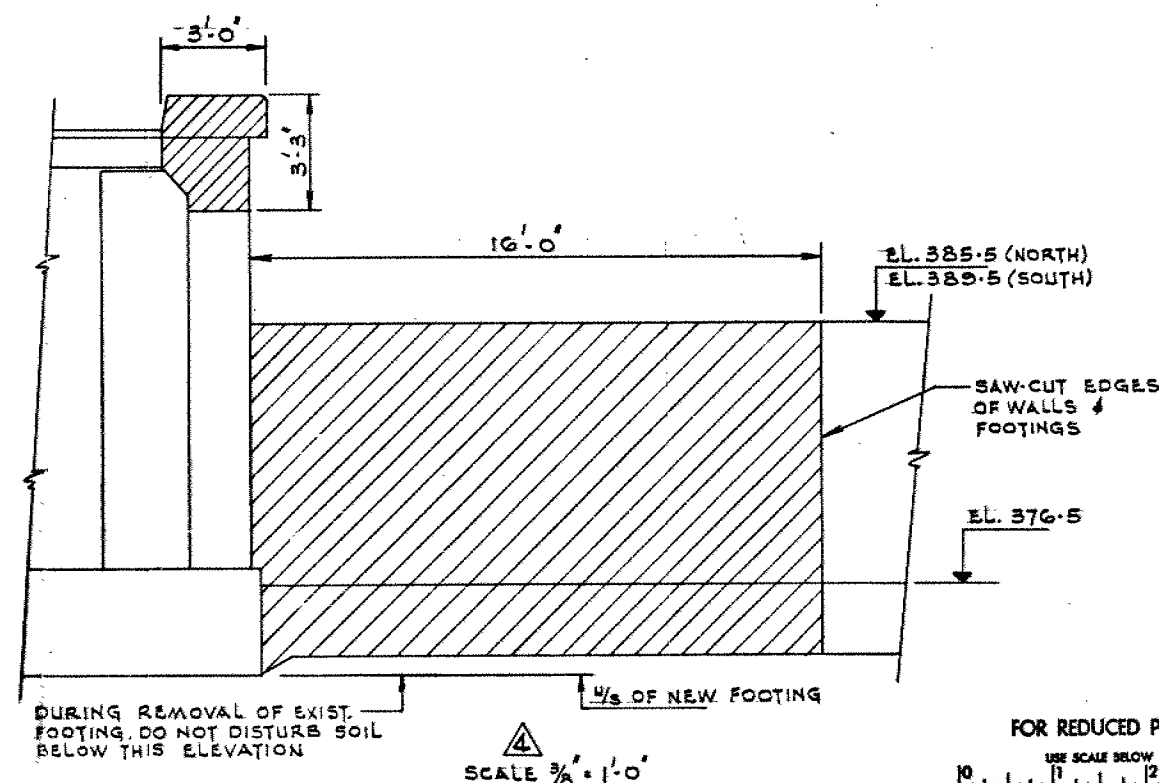
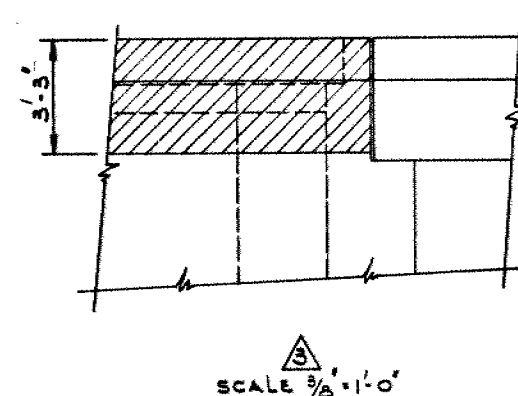
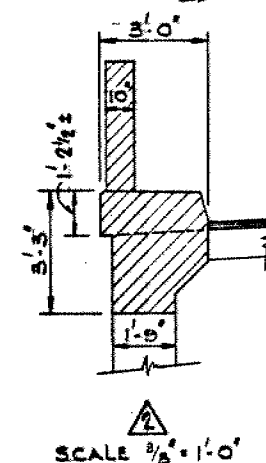
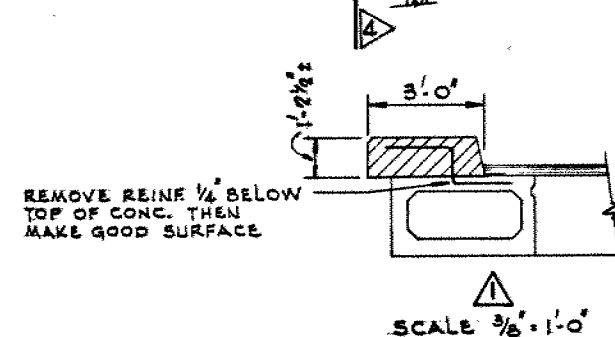
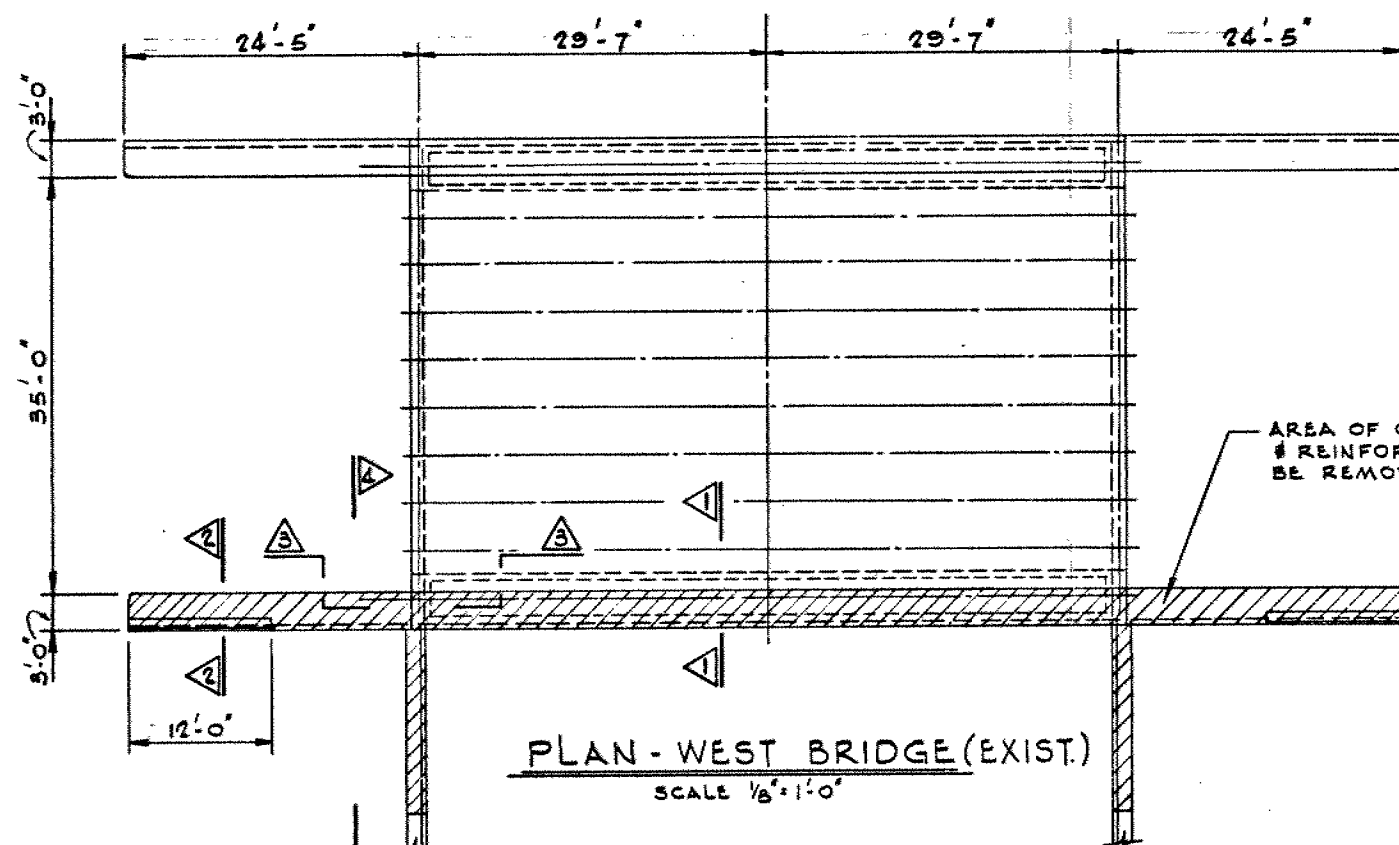


FOR REDUCED PLAN

2nd STAGE of CONSTRUCTION



REVISIONS	DATE	BY	DESCRIPTION
1	SEP 1990	C.S. LEPPER	DESIGN
2	OCT 1990	C.S. LEPPER	CHECK
3	NOV 1990	C.S. LEPPER	LOADING
4	DEC 1990	C.S. LEPPER	AS20-44
5	JAN 1991	C.S. LEPPER	DATE
6	FEB 1991	C.S. LEPPER	SEP 79
7	MAR 1991	C.S. LEPPER	DWG
8	APR 1991	C.S. LEPPER	1



REVISIONS	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			

DESIGN	CHECK	LOADING HS-20-44	DATE SEPT/79
DRAWING	CHECK	SITE No 18-175	DWG 3

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 86 - 102



Ministry of
Transportation and
Communications

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1	Index
2	Abbreviations & Symbols
3 - 11	Foundation Investigation Report for W.P. 90-74-03; Site 18-175 N-EW Ramp, Chestnut St. Widening

NOTE: For purposes of the contract this report supercedes all other foundation reports prepared by or for the Ministry in connection with the above-mentioned project.

90-74-03-17A

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION INVESTIGATION REPORT

For

N-EW Ramp, Chestnut St. Widening
W.P. 90-74-03; Site No. 18-175
Hwy. #406; District #4, Hamilton

INTRODUCTION

Presented in this report are the results of a foundation investigation carried out at the crossing of Hwy. #406 (formerly Hwy. #58, Line 'B') and Chestnut St. in the city of St. Catharines, during the period of Nov. 21-25, 1960 and was reported under W.P. No. 44-61. The fieldwork consisted of four sampled boreholes and three dynamic cone penetration tests.

Since the time of the original investigation the southbound and northbound structures were constructed under Contract No. 62-316. For the purpose of the proposed southbound structure widening, which will accommodate the future N-EW Ramp, we are utilizing the information obtained from Borehole #2 and #3 of the original field investigation. The data contained in this new report reflects the conditions which existed at the time of the fieldwork (Nov. 1960) and not necessarily the present conditions.

SITE DESCRIPTION

The site is located at the crossing of Hwy. #406 and Chestnut St. in the city of St. Catharines on an open strip of flat land which is bordered by commercial and residential areas.

Approximately 2500 feet of the site is a steep ridge which rises to the south from the land, 150 feet in height, to a flat limestone plain which forms the northern limit of the Niagara Escarpment.

Physiographically the site is located at the south end of the Iroquois Plain. •.

SUBSURFACE CONDITIONS

General

As mentioned above, the field investigation was conducted about 22 years ago. Since that time, the structures and the associated approach embankments were built. Locally obtained silty clay type materials were used for the fills and compacted according to the existing specifications. The original subsoil at this location was found to consist of a 50' + deep deposit of silty clay to clay followed by sandy silt of undetermined thickness. The boundaries between the different strata, together with the obtained field and laboratory test results, are shown on the Record of Borehole Sheets. A stratigraphical profile is shown on Drawing No. 2 of the contract documents. A brief description of the encountered different native soil types are given below.

Silty Clay to Clay

Immediately below the original ground surface there is an approx. 51.5' - 58.0' thick cohesive stratum. The material in the deposit consists of silty clay to clay with pockets and seams of silt and also, traces of sand. A plot of Plasticity Index versus Liquid Limit (Fig. 1) shows the majority of the points to fall within the CI zone. An overconsolidated zone, due to desiccation and/or weathering, with a thickness of about 20 feet was found to extend from the upper surface of the stratum. This zone is brown in colour due to oxidation and apart from the upper 3-4 feet (frost affected zone) has a very stiff to hard consistency; the 'N' values ranged from 19 to 74 blows per foot. Based on the standard penetration tests results only the undrained shear strength of this desiccated zone is estimated to be in the order of 2500 PSF to 8000 PSF. Below this crust the colour of the soil is grey and the consistency ranges from very stiff to firm. In general, the undrained shear strength decreases with depth.

Physical properties of the material as determined from field and laboratory tests are summarized as follows:

	<u>Range</u>
Natural Moisture Content (%)	21-40
Liquid Limit (%)	30-57
Plastic Limit (%)	16-25
Bulk Density (PCF)	119-128
Undrained Shear Strength (PSF)	
Field Vane Test	1500-2000+
Lab. Vane Test	900-1500
Quick Triaxial Test	600-2050
Sensitivity	3-5

The results of the grain-size distribution tests are shown in an envelope form on Figure 2.

Sandy Silt

This stratum was found to underlie the above-described cohesive type deposit at El. 324+ (B.H. #2) and at El. 336+ (B.H. #3). The lower boundary was not determined since the borings were terminated within this zone, but was found to extend for a minimum depth of about 28 ft.

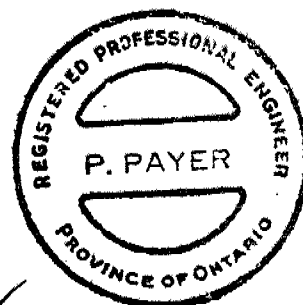
The material in the deposit is sandy silt. The results of the grain-size distribution tests are shown in an envelope form on Figure 3. The Natural Moisture Content is in the order of 8%. Standard Penetration Tests gave 'N' values in excess of 100 blows per foot. Based on the 'N' values it is estimated that this stratum is in a very dense condition.

Groundwater Conditions

The following groundwater levels were observed during the field investigation:

B.H. #2 El. 376.3 (6' Below Ground Level)

B.H. #3 El. 381.9 (5.6' " " ")



A handwritten signature in dark ink, appearing to read "P. Payer".

P. Payer, P. Eng.
Foundations Engineer

A handwritten signature in dark ink, appearing to read "K.G. Selby".

K.G. Selby, P. Eng.
Senior Foundations Engineer

RECORD OF BOREHOLE No 2

W P 90-74-03 LOCATION Sta. 172+00, 47' Lt. ORIGINATED BY G.G.C.
 DIST 4 HWY 406 BOREHOLE TYPE Washboring, Cone Penetration COMPILED BY B.K.
 DATUM Geodetic DATE Nov. 25, 1960 CHECKED BY LB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
382.3	Ground Level													
0.0	Silty Clay to Clay With Pockets and Seams of Silt Trace of Sand		1	SS	38									
			2	SS	24									
	Very Stiff to Hard		3	TW	19								126	
			4	TW	PM								122	
			5	TW	PM								123	
	Stiff to Very Stiff		6	TW	PM								124	
			7	TW	PM								121	
			8	TW	PM								119	
			9	TW	PM								122	
			10	TW	PM									
			11	TW	PM									
324.3			12	SS	108/ 8"									
58.0	Sandy Silt		13											
	Very Dense		14	SS	140									
306.3			15	SS	100/ 6"									
76.0	End of Borehole													

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

W P 90-74-03 LOCATION Sta. 170+59, 49' Lt. ORIGINATED BY G.G.C.
 DIST 4 HWY 406 BOREHOLE TYPE Washboring, Cone Penetration COMPILED BY B.K.
 DATUM Geodetic DATE Nov. 21, 1960 CHECKED BY *[Signature]*

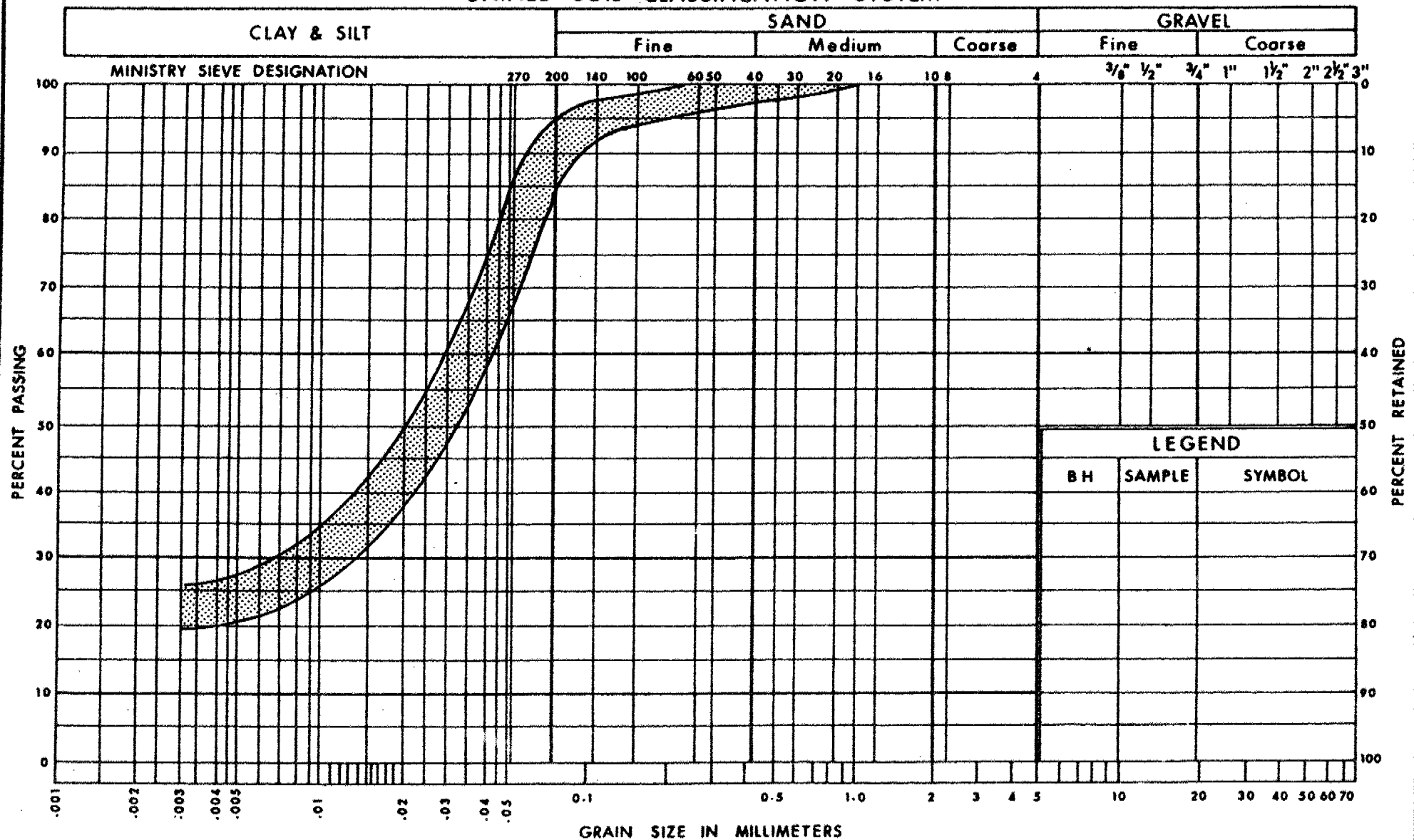
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH PSF					WATER CONTENT (%)				
								20 40 60 80 100					W _p	W	W _L		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										
							500 1000 1500 2000 2500					15 30 45					
387.5	Ground Level																
0.0																	
	Silty Clay to Clay Trace of Sand		1	SS	74	▼ W.L. 381.9	380							○	—	128	
			2	SS	25									○	—		
			3	TW	35									○	—	121	
	Very Stiff to Hard		4	TW	25									○	—		
			5	SS	16		370										
	Pockets and Seams of Silt		6	TW	15									○	—	123	
			7	TW	13		360							○		120	
			8	TW	12											123	
	Firm to Very Stiff		9	TW	14		350										
			10	TW	15									○	—	128	
			11	TW	16		340							○		124	
336.0			12	TW	12												
51.5	Sandy Silt Very Dense		13	SS	100/	5"								○			
			14	SS	100/	4"	330							○			
326.0			15	SS	100/	3"								○			
61.5	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (% STRAIN AT FAILURE)

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

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Communications

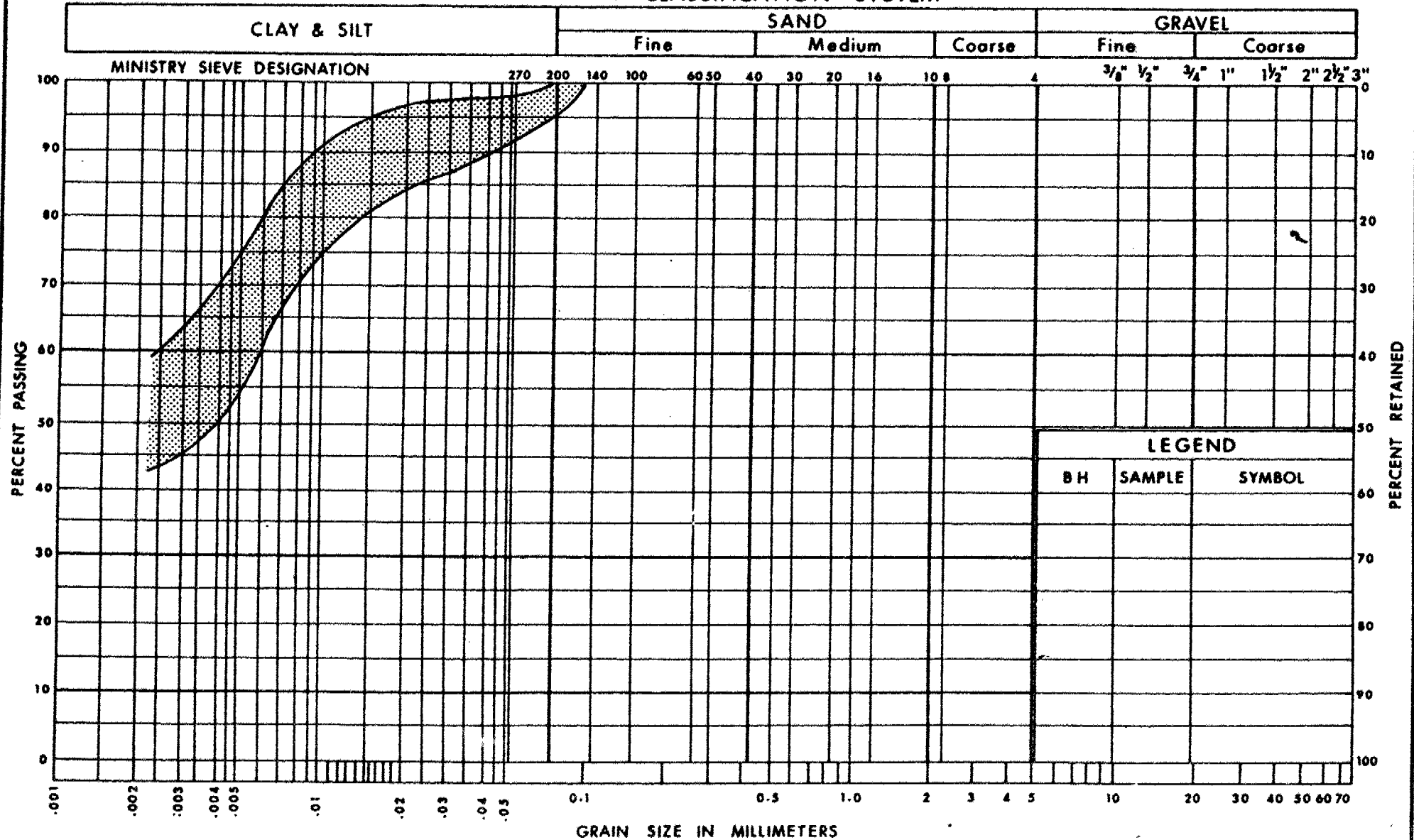
GRAIN SIZE DISTRIBUTION

SANDY SILT

FIG No 3

W P 90-74-03

UNIFIED SOIL CLASSIFICATION SYSTEM

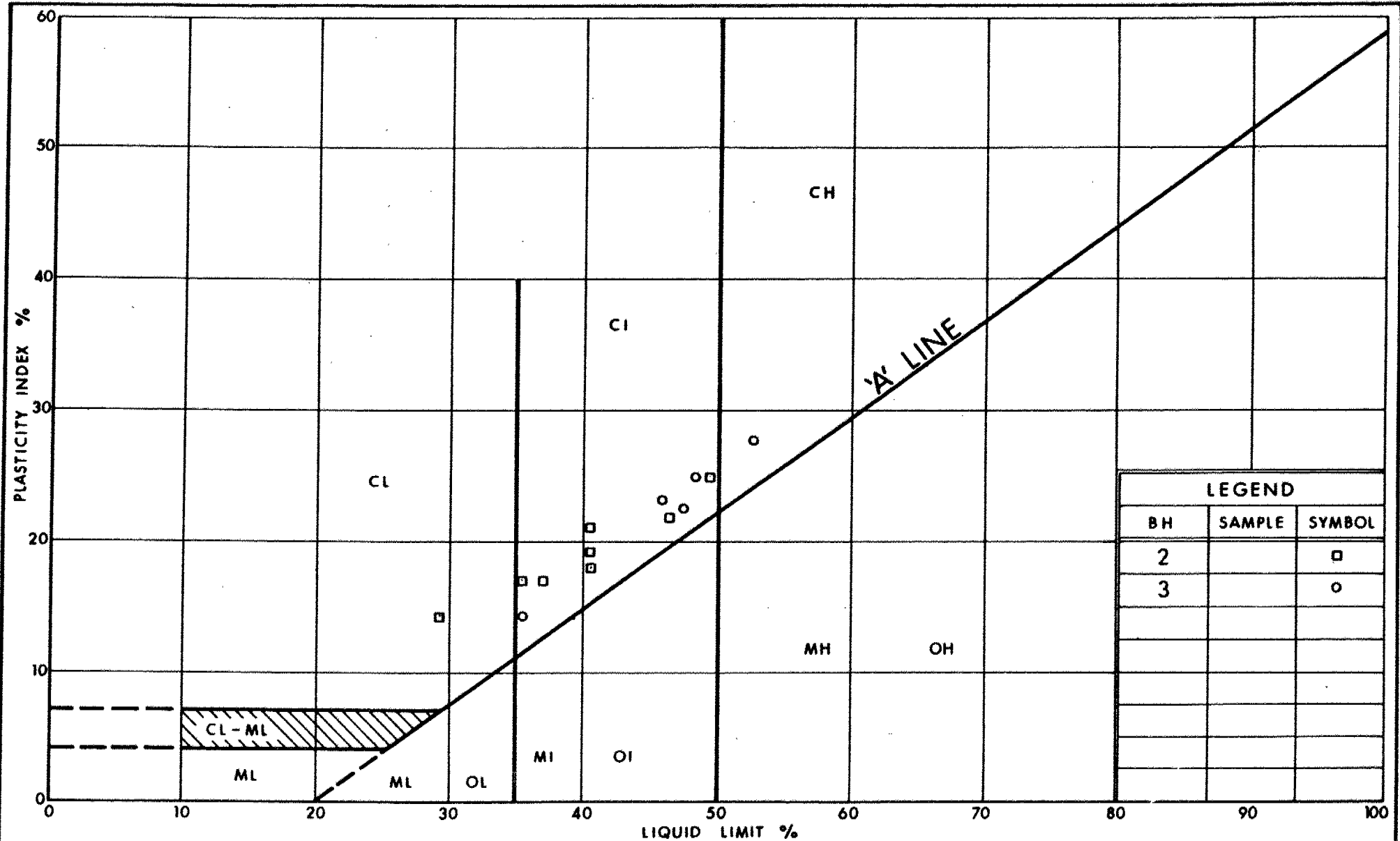


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GRAIN SIZE DISTRIBUTION SILTY CLAY TO CLAY

FIG No 2

W P 90-74-03



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Communications

PLASTICITY CHART SILTY CLAY TO CLAY

FIG No 1

W P 90-74-03

Mr. M. D. Bendayan,
Senior Structural Engineer,
Structural Section,
1st Floor, 5000 Yonge Street,
CENTRAL REGION

86 01 16

From: Engineering Materials Office,
Foundation Design Section,
Central Building, Room 315

Re: Widening of Hwy. #406
Chestnut St. Overpass S.B. Structure,
W. P. 90-74-03; Site: 18-175
District #4, Burlington

This is in reply to your memo of 86 01 08. We have reviewed recommendations provided to you in our memo dated 79 04 02 regarding the above-mentioned project. These are essentially still valid even though the amount of widening has been reduced from 4.9 to 1.8 m. For your convenience we are providing you with new recommendations in metric units and for purposes of the O.H.B.D.C.

1. The existing footings are at elevation 113.989 m. The footings for the widening should be placed at the same elevation and may be rigidly attached to the existing ones.
2. For design purposes assume a net safe bearing capacity of 200 kPa for the foundation soil at elevation 113.989 m. An adhesion value of 75 kPa may be assumed to apply between the new footing bases and the foundation soil.
3. For purposes of the O.H.B.D.C. the following applies to the foundation soil.

Factored Capacity at U.L.S.	340 kPa
Capacity at S.L.S. Type II	200 kPa

Earth Pressure	Active Condition applies
Backfill Gran. A	= 22.8 kN/m ³ $\phi = 35^\circ$ KA = 0.271
Gran. B	= 21.2 kN/m ³ $\phi = 30^\circ$ KA = 0.333

re: Widening of Hwy. #406
Chestnut St. Overpass S.B. Structure,
W. P. 90-74-03; Site: 18-175
District #4, Burlington

4. Additional fill will be required for the approach embankment widening. However, only the new shoulder will be supported by this new fill. Settlements due to the new fill are estimated to be 50 to 60 mm which can easily be tolerated by a shoulder. All pavement and approach slab widening will be supported by the existing shoulder which should not experience further significant settlement.

Please advise if we can be of further help.



K. G. Selby,
Chief Foundations Engineer
(West)

KGS:ma

cc: K. Bassi
R. Fitzgibbon

memorandum



To: Mr. K.G. Selby
Chief, Foundation Engineer (West)
Foundation Design Section
Room 315, Central Building

Date: 86-01-08

RE: Widening of Highway 406 - Chestnut Street Overpass
(S.B. bridge, only)
W.P. 90-74-03, Site 78-175
District 4, Burlington

Back in 1979 your office was requested to give foundation recommendations for the subject work. This was done in a memo from Mr. P. Stuart dated 1979-04-02 (copy attached).

The design was then completed and stockpiled. The overall Highway 406/Glendale Avenue (Regional Road 89) planning improvements have recently be reviewed with the following main results:

- a) The new N-E, W ramp bridge over Glendale Avenue will not be constructed now.
- b) The widening required now for the Chestnut Street south bridge deck is much less ($1.8 \pm m$) than originally planned ($4.9 \pm m$).

Item a) does not require any action at the present time.

Regarding item b), would you please review and update your previous recommendations bearing in mind that this project is metric.

We attached for your information one print of drawing S-6885-1 "Initial Geometrics for the Chestnut Street Overpass", dated 85-12-31, as prepared by Cole, Sherman & Associates Ltd.

...../2



Would you please send us your updated recommendations by the end of January, 1986 to enable the design to proceed accordingly.

You will have the opportunity to comment on the final design package when completed, early in June, 1986.

Thanking you in advance.



M.D. Bendayan
Senior Structural Engineer
for:
G.C.E. Burkhardt,
Head, Structural Section.

MDB/df

c.c.: R. Fitzgibbon
G. Smolskis



118-175
Ministry of
Transportation and
Communications

Memorandum

To: Mr. G.C.E. Burkhardt
Head, Structural Section
Central Region
3501 Dufferin Street, Downsview
Attention: Mr. M.D. Bendayan

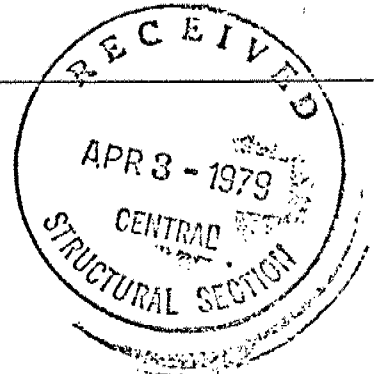
From: Soil Mechanics Section
Engineering Materials Office
Room 315, Central Building

Date: 79 04 02

Our File Ref.

In Reply to

Subject: Re: NE, W Ramp Glendale Avenue Overpass
W.P. 90-74-02, Site 18-170
Widening of Chestnut Street Overpass
W.P. 90-74-03, Site 18-175
Hwy. 406, District 4, Hamilton



To comply with the accelerated design schedule for these structures it was requested that preliminary foundation recommendations be made by letter. Recommendations for the design of the new structures are provided in this memorandum based on both the performance of the existing structures and on the data gathered in the field investigation for their design. Since no further fieldwork is required no new report will be issued until the final design stage when a contract report and a new subsoil stratigraphy drawing will be prepared.

Chestnut Street Overpass

Subsoil at this site consists of from 50 to 60 feet of hard to stiff silty clay overlying a deposit of red very dense sandy silt of undetermined thickness. The existing structure is founded on spread footings at elevation 374. It is recommended that the structure widening also be supported on spread footings at approximately this same elevation with a design load of two tons per square foot. Resistance to sliding may be calculated employing a design adhesion value of 2000 lb per square foot. It is estimated the maximum settlement will be less than one inch.

As an alternative the widening may be supported on steel H piles with driving controlled by SS-3-11. It is estimated that design loads equal to the structural capacity of the section chosen will be achieved at tip elevations 325 for the south abutment and 310 for the north abutment. The pile tips should be reinforced with standard flange plates to prevent damage by boulders in the till zone.

The base of all footings or pile caps should be protected from frost action by a minimum four feet of cover.

cont'd.....


N-E, W Ramp Glendale Avenue

Subsoil at this site consists of approximately 30 feet of hard to very stiff silty clay overlying from 50 to 60 feet of very dense red silt to clayey silt. Red shale bedrock was encountered at elevation 321, some 85 feet below the ground surface.

The existing structures at this site are constructed with the abutments supported on spread footings on compacted granular cores within the approach fills and with the piers on spread footings in the upper hard portion of the silty clay. It is recommended that a similar scheme be adopted for the new structure. The pier footings will support a design load of three tons per square foot at approximate elevation 402. The abutment footings may also be designed for a load of three tons per square foot with the granular A core constructed in accordance with the attached figure. Resistance to sliding may be calculated using a design adhesion value of 2000 pounds per square foot for the pier footings and a coefficient of friction of 0.5 for the abutment footings on compacted granular A fills. It is estimated that the settlement for both pier and abutment footings will be less than one inch.

As an alternative any or all of the footings may be supported on steel H piles with driving controlled by SS 3-11. It is estimated that design loads equal to the structural capacity of the section chosen will be achieved at elevation 345. The pile tips should be reinforced with standard flange plates to prevent damage from boulders in the till zone.

The base of all footings or pile caps should be protected from frost action by a minimum of four feet of cover.



P.J. Stuart
Project Engineer

PJS/gs

Attach.

cc: W. Lin
R. Fitzgibbon

Files

D. Mullett

H. Cameron

B. Hard (66), (Lancaster)

