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W.P. #444-64

CARLTON ST.

TUNNEL

WELLAND

CANAL

PRELIMINARY
REPORT TO
DEPARTMENT OF HIGHWAYS,
ONTARIO
RE: CARLTON STREET TUNNEL
UNDER WELLAND CANAL
W.P. 444-64

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September 4th, 1964

Your File W. P. 444-64
Our File 4011

Mr. H. W. Adcock
Assistant Deputy Minister - Engineering
Department of Highways Ontario

Dear Mr. Adcock: CARLTON STREET TUNNEL

Submitted herewith is our preliminary report in 8 copies to the Department of Highways Ontario and 4 copies to The St. Lawrence Seaway Authority.

We recommend the float-in-large-element scheme as the most practical and economical. The preliminary cost estimate totals \$8,000,000, from grade to grade, including contingency but excluding property and engineering. Construction is scheduled to begin in December 1964, and to end in November 1967. Preliminary borings disclose a soft clay bed for the tunnel with the possibility of artesian water in a deeper granular stratum. The recommended general scheme can cope with the most difficult conditions expected.

We recommend that during the '64-'65 ten-week winter dry-canal period the tunnel trench be partially excavated by removing about 100,000 cu. yds. from the canal bed. This will facilitate full scale study of slope stability and sedimentation and it will relieve the tightness of the next winter's construction programme. The St. Lawrence Seaway Authority have agreed that the '64-65 winter work will be undertaken in their name.

Foundation conditions are now being explored in detail and contract documents for the '64-'65 winter construction are now being prepared. The total project design is being developed with a view to completion of the design report before the start of construction in December 1964.

Yours very truly,

GENERAL ENGINEERING COMPANY LIMITED

per Norman D. Lea, Principal
MONTREAL TORONTO

VANCOUVER

CHAPTER 1

INTRODUCTION

By proposal of June 15th, 1964, and acceptance of July 14th, 1964, the Department of Highways Ontario has retained General Engineering Company Limited to carry out preliminary engineering on the Carlton Street Tunnel project under the Welland Canal at St. Catharines.

The Carlton Street Tunnel is one of three tunnels which the Department of Highways Ontario plans to build in conjunction with The St. Lawrence Seaway Authority to replace existing inadequate cross-canal roadway structures, these new crossings being scheduled with the canal twinning program. The Carlton Street Tunnel project is the subject of a Functional Report produced by the Functional Planning Section of the Planning Branch in July, 1964.

The results of the preliminary engineering work by GECO are described in this present report which marks the conclusion of the preliminary report phase. The objectives of this phase are:

1. A recommended general scheme for the project.
2. A preliminary cost estimate.
3. A construction schedule.

There are, of course, many design questions which have not been resolved in this phase. The design has, however, been pursued far enough to establish the technical feasibility and approximate cost of the several possible schemes.

Throughout the preliminary design work close liaison has been maintained with the engineering staff of the Department of Highways Ontario and the engineering staff of The St. Lawrence Seaway Authority. The preliminary field and laboratory foundation exploration program has been carried out by the Materials and Testing Division of the Department of Highways Ontario.

Discussions have been held with the several utility agencies concerning their present installations which will be affected by the project and concerning their desires for carrying utility services through the tunnel.

It is anticipated that this preliminary report will be followed in a few months by a design report which will present the results of the design development work to be carried out upon the adopted general scheme.

The GECO personnel responsible for the preliminary report are Norman D. Lea, William J. Swanson and T. J. Sluymmer.

CHAPTER 2

SITE

2.1 LOCATION

The site is situated on the outskirts of the City of St. Catharines about two miles south of the southern shore of Lake Ontario at Port Weller, which is the community where the Welland Canal joins Lake Ontario. The crossing is 1-1/2 miles north of the Garden City Skyway.

The Functional Planning Report considers two possible tunnel crossing locations in the northerly five miles of the Welland Canal and selects Carlton Street as the preferable of these two. This selection has not been reviewed as a part of this preliminary report, but from a general examination of the section of the Canal north of the escarpment, it is clear that Carlton Street is one of the best locations for the construction of a tunnel.

The Functional Planning Report suggests a tentative alignment for the tunnel as parallel to Carlton Street and 200 feet south.

The least expensive place to cross near Carlton Street would be about 1000 to 1200 feet south of Carlton Street. At this location the canal is substantially narrower than it is just 200 feet south of Carlton Street and the tie-up wall is absent. These two features would permit a very much less expensive tunnel crossing with a saving of at least half a million dollars. At this more southerly location, the tunnel might also be short enough that forced ventilation could be eliminated entirely, thereby effecting a further very substantial saving. This more southerly location would, however, greatly complicate the road connections and the saving in tunnel construction cost could well be consumed in the cost of additional road and street construction with the attendant property acquisition. The location closer to Carlton Street is therefore preferred.

Consideration has been given to the possibility of making the distance of the tunnel from Carlton Street somewhat more or less than the suggested 200 feet. For tunnel construction, the amount of the offset is not important on the east side of the canal because the construction work will take place upon Seaway Authority property. To the west of the canal the 200 feet offset will provide ample property between Carlton Street and the tunnel for construction provided the dyke is pierced by the steel-sheet-piling method. Should the dyke be pierced by the floated-in-element method however, using earth cofferdams, it would be necessary to take very substantially more property including several dwellings on the north side of Carlton Street. Should this method be selected for piercing the dyke, it would be necessary to move

the tunnel another 150 feet at least in order to avoid affecting these properties. The 350 foot offset thereby created would produce an unsatisfactory alignment and cannot be considered. Indeed any southward movement beyond the 200 feet produces alignment problems. Thus it is concluded that the offset should be not more than 200 feet and that the alignment of the tunnel for purposes of this preliminary report is parallel to Carlton Street and 200 feet south of it. During the design development phase, study will be given to reducing this 200 feet, possibly to 150 feet.

2.2 TOPOGRAPHY

The existing topography is shown on the attached drawing 4011-R-1. It will be observed that the countryside in this area is generally quite flat having been the bed or shore of a post glacial lake. The land slopes gently at about 20 feet per mile towards Lake Ontario.

The most significant topographical feature on the site is the man-made Welland Canal which, at this location, was constructed in the period 1920-1930 and open to traffic in 1931. The natural ground is about elevation 320 and the canal dyke rises about 20 feet above this to approximately elevation 340.

2.3 HYDROGRAPHY

The information which is currently available on the elevation of the bed of the canal is shown on drawing 4011-R-1. It will be observed that generally the bed of the 300 foot wide navigation channel is maintained at about elevation 305. The width of the channel will, of course, be increased when the canal is twinned but outside the channel the water depths will probably not be changed significantly. During the navigation season the water in the canal is maintained at elevation 335.5 with the extreme high elevation being 337.5.

The canal in this section which is the reach between Lock #2 and Lock #3 is drained each winter during the non-navigation season which extends from about mid-December to late March. The actual dates for the four years since the Seaway has been in operation are as follows:

<u>Year</u>	<u>Date Last Vessel Out</u>	<u>Date Drawdown of Reach above Lock 2 Completed</u>	<u>Date Refilling of Reach above Lock 2 Completed</u>
1960	Dec. 17	Dec. 22	March 25, 1961 (Probably)
1961	Dec. 16	Dec. 19	March 25, 1962
1962	Dec. 16	Dec. 26	March 23, 1963
1963	Dec. 18	Dec. 24	March 27, 1964

The normal minimum time is about two days for complete draw-down of the reach above Lock #2 and about the same time for refilling.

Since the water within the canal is entirely controlled, the summer currents are only those created by passing the water through the locks and by passing it over the spillway, which is shown on drawing 4011-R-1, to supply water to make up for evaporation and leakage downstream. The maximum rate of discharge is approximately 1200 cfs. giving a maximum summer current of approximately 0.07 fps at this location.

During the draining of the reach between Lock #2 and Lock #3, the gates may be open full giving a maximum discharge of 6000 cfs. and a maximum average drawdown velocity at the tunnel of about 0.35 fps. Following complete drawdown a drainage water course continues to flow in the bed of the canal carrying a flow of up to 500 cfs.

2.4 CLIMATE

The following temperature and precipitation data has been supplied by the Toronto Meteorological Office of the Federal Department of Transport:

Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year	Period of Record
<u>Temperature (°F.)</u>													
Mean Maximum Temperature													
32.3	32.5	40.2	54.3	66.7	77.2	82.2	80.6	72.7	61.2	47.1	35.3	56.9	1931-60
Mean Minimum Temperature													
21.0	19.9	26.1	35.9	45.5	56.2	61.8	60.9	54.1	43.8	34.5	24.4	40.3	1931-60
Mean Temperatures													
26.7	26.2	33.2	45.1	56.1	66.7	72.0	70.8	63.4	52.5	40.8	29.9	48.6	1931-60
Absolute Extremes - Maximum													
68.	87	80	85	90	97	104	101	97	88	77	63	104	1921-60
Absolute Extremes - Minimum													
-9	-11	-3	5	27	34	44	42	31	21	7	-12	-12	1921-60
<u>Average Precipitation (in)</u>													
Rainfall													
1.24	1.26	1.56	0.96	2.72	2.11	2.25	2.74	2.80	2.26	1.91	1.25	23.06	1931-60
Snowfall													
8.5	9.1	7.3	1.0	0	0	0	0	0	0	3.2	8.3	37.4	1931-60
Total Precipitation													
2.09	2.17	2.29	1.06	2.72	2.11	2.25	2.74	2.80	2.26	2.23	2.08	26.80	1931-60

The National Building Code, supplement No. 1, 1961 shows the following additional temperature and rainfall data:

2-1/2% Winter Design Temperature	-	5°F.
1% Winter Design Temperature	-	2°F.
Degree days below 65°F.	-	6700
Maximum 15 min. rain		1.0 in.
Maximum 1 day rain		3.5 in.

From the above it may be concluded that during the winter months of January, February and March

- both some rain and some snow may be expected
- the mean temperature during any week may be expected (with about 75% confidence) to be between 15°F. and 40°F.

The following wind records have also been supplied by the Toronto Meteorological Office of the Federal Department of Transport:

Wind Records - Jan. 1955 - Dec. 1962 (Broken)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year
Percentage Frequency (by directions)													
N	4	6	4	5	3	3	4	5	6	5	3	3	4
NE	11	13	17	17	9	11	13	15	13	12	6	9	12
E	7	6	10	9	6	4	3	4	7	6	5	6	6
SE	6	5	6	5	7	5	3	6	7	8	8	8	6
S	3	6	4	4	9	8	8	11	12	13	9	6	8
SW	45	34	25	30	36	38	39	34	30	29	43	47	36
W	7	8	12	11	11	8	8	7	9	8	12	9	9
NW	17	19	21	19	19	21	19	17	15	18	14	12	18
C	•	1	1	•	•	2	3	1	1	1	1	•	1

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year
Average Wind Speed in Miles per Hour (by directions)													
N	8.7	7.9	7.5	6.2	6.2	4.4	4.8	5.2	6.2	5.6	5.6	8.6	
NE	8.6	8.6	9.3	8.2	5.7	6.5	6.7	6.8	6.5	6.3	6.9	8.2	
E	4.8	5.4	6.8	6.6	5.4	3.7	2.8	2.9	2.7	3.2	5.0	5.0	
SE	5.8	5.7	6.4	7.1	6.7	4.5	4.6	5.3	5.3	5.7	7.1	7.9	
S	5.5	6.3	6.5	6.3	6.3	5.0	4.8	5.4	5.7	6.1	7.3	5.3	
SW	10.2	9.8	9.8	9.0	8.7	7.8	6.7	6.9	8.1	7.8	10.2	10.0	
W	7.9	7.9	8.9	6.1	5.8	4.6	4.4	4.0	5.5	4.3	8.6	8.2	
NW	12.8	12.9	11.5	9.5	8.4	7.4	6.9	7.1	7.4	9.6	11.1	11.8	
Average Wind Speed in Miles per Hour													
	9.4	9.1	9.1	8.1	7.5	6.5	6.0	6.1	6.5	6.8	9.0	9.1	7.8

This shows that the prevailing wind direction is south-west with an average speed of about 10 m.p.h. The National Building Code shows gusts to 80 m.p.h.

East and west winds are the least frequent and are generally of low speed. Wind should therefore cause a minimum of ventilation problems in the tunnel.

U. S. Corps of Engineers data indicates that the Design Freezing Index ranges between 750 and 1000 for the St. Catharines area.

2.5 FOUNDATION CONDITIONS

2.5a Available Data

The St. Lawrence Seaway Authority have made data available on ten borings made by them in the area. Three of these are old borings which were made at the time of the construction of the canal in the 1920s. The other seven are borings which were made within the last few years, some of them in the winter shutdown period taken from the bottom of the canal. These more recent borings show greater detail and also some laboratory test results. The location of the borings by The St. Lawrence Seaway Authority are shown on drawing 4011-R-8. The boring logs are shown diagrammatically on the stratigraphic sections on drawing 4011-R-10 and the significant strength results are plotted on drawing 4011-R-11.

It is known that a slip occurred during the construction of Lock #2 in the 1920s with the firm clay being the troublesome material. It has not been possible, however, to find sufficient information to make an analysis of .

this old slip. Considerable instability problems have developed in the canal dykes further upstream near the Garden City Skyway with a similar layer being the troublesome one. A substantial amount of technical data is available from this location but after examination of this data it has been concluded that it is not useful directly for the Carlton Street Tunnel project.

The St. Lawrence Seaway Authority has installed a number of piezometers around Lock #2. The location of these piezometers is shown on drawing 4011-R-12, and the results which have been observed in them are shown on drawing 4011-R-13. In July, 1964, piezometers W100-1, W97-1, W98-2, and W97 were examined in the field. The only one found to be sensitive was W98-2.

Professor R.E. Deane of the University of Toronto reports that there are numerous water wells in the general area, that they draw water from depths varying from 65 to 103 feet and that they consistently intercept water under hydraulic head. The water generally rises in the wells to about 22 feet below ground level and the wells are thus classified as non-flowing artesian.

Three preliminary borings, numbers 1, 2 and 3 shown in location on drawing 4011-R-8 were requested of the Department of Highways Ontario, Materials and Testing Branch with considerable attention being given to the strength of the firm clay layer. These borings were carried out during late July and early August. At each location three holes were made, two with vane tests through the firm material, and one with continuous piston samples. The boring logs from these three holes are shown on drawing 4011-R-9.

Professor Deane has reviewed the samples and commented upon the relevant glacial geology. His comments as well as the boring logs and laboratory data have been used in the preparation of the following discussion of stratigraphy.

2.5b Stratigraphy

The preliminary stratigraphy of the site is shown on the attached drawing 4011-R-10. The significant strata may be identified and described as follows:

Fill - Several of the borings have penetrated the dyke which is a stable, well graded material with substantial clay content.

Sand - From observation in adjacent excavations, and from one of the boreholes it is known that there is generally a few feet of sand and sandy topsoil immediately beneath the natural ground surface in this general area.

Hard Clay Till - This material is immediately below the sand or fill and grades from hard at the surface to stiff near its bottom. It contains all grain sizes from clay to

gravel with the clay content in the D.H.O. borings being adequate to give the dominant property to the deposit which is generally homogeneous. In some of the borings by the Seaway Authority, however, the lower part of this stratum is silty as its dominant characteristic with some samples being lost. Large stones are rare but pebbles up to one inch are not uncommon. Well defined stratified layers of any one grain size are absent but isolated pockets may be present. The colour grades from a brown at the surface to a grey at the base. The material is of low permeability and shows no apparent fissures or slickensides.

It is believed that this till was deposited beneath a re-advancing glacier that over-rode the underlying clay deposit. Hence much of the till is made up of the underlying clay material. It is possible that much, or all, of the strata which has been classified as firm to-stiff-brown-clay may be properly geologically classified as a till because it may have been disturbed and transported to some extent, by a glacier. In the attached logs and stratigraphic plots, however, the boundary between the hard till and the firm clay has been established primarily on the basis of Atterberg limits and of strength.

Firm Clay - The firm clay zone generally extends from about elevation 285 to about elevation 265. The lower part of the deposit is called "firm grey clay" and shows a well developed horizontal structure with varves or laminations of clay of varying plasticity with the least plastic being a slightly clayey silt. There are occasional thin layers or pockets of fine sand. The upper portion of the strata, called "firm to stiff brown clay", is similar in character to the firm grey clay with less evidence of silt and sand. The strength of this firm clay zone is of great significance to the tunnel project and therefore all of the presently available strength results have been plotted on drawing 4011-R-11. From this plot it is clear,

- a) that there is a well-defined zone of lower strength generally from elevation 265 to elevation 285, and,
- b) that further detailed studies of the strength properties are essential.

A shear strength of 750 lbs. per square foot has been selected for preliminary design purposes for both strata, although it is appreciated that the upper firm-to-stiff-brown-clay is generally somewhat stronger than the underlying firm-grey-clay. The contrast between the firm clay and the underlying material is a sharp one.

Dense Stratified Granular Material - Extending below the firm clay for a varying thickness is a stratified granular layer. This layer shows results of both water sorting and glacial action. It may have been deposited under the ice or by streams at the ice front. It contains layers of silt, sand and gravel, generally water sorted, and quite irregular.

This zone may well be an aquifer and therefore the piezometric pressure in it will be investigated carefully.

Hard Clayey Silt - The dense stratified granular material is underlain by hard clayey silt. This stratum is practically impermeable and has N values over 100. It would be expected to stop any piles that pass the overlying dense-stratified-granular material.

Very Dense Red Clay Till - The bedrock is overlain by a very dense red clay till deposit which contains enough fines to be essentially impermeable but which also contains a considerable amount of the reddish shaly material derived from the underlying bedrock.

Bedrock - The bedrock in this area is Queenston shale of Ordovician age. It is predominantly red in colour and is interbedded with thin beds of grey and green shale. The upper few feet is normally fractured and some of the older borings show a substantial thickness of "recompacted" or "bastard" shale.

2.5c Detailed Programme

Following the results from the first three borings made by the Department of Highways Ontario, a detailed soil investigation programme has been worked out and its implementation has been started without interruption because of the urgency of the schedule. This detailed programme includes four further exploratory borings, four borings with piezometer installations, and five slope indicator installations. It is described in greater detail in Appendix A.

In the boring programme and the laboratory testing program, the emphasis will be upon studying further, the short and long term strength properties of the firm clay, the long term strength properties of the hard-clay till and the variations of the stratigraphy throughout the site. In the piezometer installations, emphasis will be upon the determination of the piezometric pressures within the dense-stratified-granular-material and also in the hard-clay-till and the firm clay. The piezometers will be observed through the '64-'65 winter drawdown period. The slope indicators will be used to observe the performance of the dykes through the '64-'65 winter drawdown period and some of them throughout the construction programme. Particular attention will be paid to the movement of the slopes of the excavations made during the '64-'65 winter period as this

will give the best possible indication of the actual strength characteristics of the firm clay and the upper till.

2.5 EXISTING SERVICES

2.5a Storm Sewers

There is an existing storm sewer under the south shoulder of Carlton Street varying from 27" to 39" in diameter approximately 6 feet below the ground. This sewer drains into a ditch which runs north from Carlton Street along the west side of The St. Lawrence Seaway Authority's access road.

2.5b Sanitary Sewers

A 36" diameter sanitary sewer runs along Carlton Street and terminates at the corner of Dunkeld Road. The City intends to extend this sewer south in the future, and this will be taken into consideration for the design of the tunnel approaches. A new 8" sewer will be laid along the north side of Carlton Street from Dunkeld Road to Cushman Road by the end of this year.

2.5c Water Lines

A 4" water main is located approximately 13 feet south of the centre of Carlton Street and terminates at Cushman Road. Provision must be made for a future water main along Cushman Road.

2.5d Telephone

The Bell Telephone Company has an aerial cable along the south side of Carlton Street terminating at Cushman Road. On Cushman Road is an open wire pole line. On the east side of the canal there is an aerial cable and pole line parallel to Carlton Street.

The Company is preparing plans for a buried cable to be built this year along the West Seaway Road replacing the present Seaway Authority telephone line. It is intended to carry this cable on The St. Lawrence Seaway Authority poles for a distance of approximately 600 feet at the tunnel site to facilitate its relocation during construction.

2.5e Power

The St. Catharines Public Utilities Commission has an overhead 4 KV power line along the south side of Carlton Street, which turns north and crosses the bridge. There are no power lines along Cushman Road. The St. Lawrence Seaway Authority has a 13.8 KV line along the west side of the canal. In both cases, power and telephone services use the same poles.

On the east side of the canal, the power lines follow Carlton Street, and there are no services crossing the tunnel centre line.

2.6f Gas

The Provincial Gas Company has a gas line along the north side of Carlton Street, about 4 feet deep just inside the walk. This line varies in diameter from 6" at Dunkeld Road to 2" at Cushman Road where the line terminates.

2.6g Roads and Streets

The Functional Planning Report generally deals quite adequately with the existing road and street system and the changes necessary to it. Only one alteration is suggested to this recommended program and that is with regard to the new St. Lawrence Seaway East Side Haulage Road. It is suggested that this road be diverted from its present location to a permanent location several hundred feet westward where it can conveniently overpass the Carlton Street tunnel. This will remove an intersection with Carlton Street close to the east approach to the tunnel where it might cause future traffic difficulties.

Carlton Street has a concrete surface, 20 feet wide. Both Cushman Road and the West Seaway Road have asphalt surfaces about 18 feet wide. No loading restrictions are in force at any time on these roads.

2.6h Irrigation Water

Water is siphoned through the dyke, near Carlton Street, on the east side for irrigation purposes. The tunnel construction will not interfere with this procedure.

2.6i Drainage

The general drainage of the site outside the dykes is parallel to the canal and to the north. There are five main drainage channels to be considered.

The most westerly channel crosses the site in a north-easterly direction and underpasses Carlton Street in a culvert of approximately 12 sq. foot cross-section. It then joins with another channel running northward between Cushman Road and the West Seaway road, which latter channel underpasses Carlton Street in a culvert of approximately 12 sq. foot cross-section. The combined channel continues north to discharge into the canal, north of Lock No. 2.

On the east side of the canal, a main drainage channel runs parallel to the canal on the landward side of the dyke. Two ditches are located at the sides of the Seaway Authority's East Haulage Road, running parallel to the road.

Another drainage channel is located in the bed of the canal. The flow in this channel is discussed in section 2.3 of this chapter.

CHAPTER 3

REQUIREMENTS

3.1 HIGHWAY REQUIREMENTS

The highway requirements are defined in general by the Functional Planning Report issued in July, 1964, by the Planning Branch of the Department of Highways, Ontario. The general arrangement of the connections to the street system as presented in the Functional Planning Report is reproduced on the attached drawing 4011-R-2, and is not dealt with here-in except the discussion of the Seaway road in Section 2.6g.

The requirements for Cushman Road are presently under study by the Engineering Department of the City of St. Catharines.

The St. Lawrence Seaway Authority's East Side Haulage Road has a 30 ft. roadway with 10 ft. shoulders each side. Where crossing the tunnel, a 30 ft. roadway with 4 ft. shoulders each side is required, designed for 50 ton capacity trucks.

The St. Lawrence Seaway Authority's road on the west side is to cross the tunnel with a minimum of a 24 ft. roadway with 2 ft. shoulders on each side and H20S16 design load.

The Carlton Street Tunnel is to be designed as an urban arterial for 50 m.p.h. with a 350 ft. stopping sight distance and with a maximum grade of 6%. The significant cross-section dimensions for the tunnel are given on drawing 4011-R-2. The highway loading through the tunnel may be taken as H25S20.

At preliminary meeting with the Department of Highways, ventilation was discussed. Representatives from General Engineering Company Limited pointed out that for tunnels under about 800 feet in length it is quite common to rely on natural ventilation and to provide no forced ventilation, whereas for tunnels of about 1000 feet in length, the practice is mixed. It was decided that forced ventilation would be supplied, it being recognized that the ventilation would probably only operate in times of emergency.

At a preliminary meeting between GEICO and the Department of Highways Ontario, illumination was discussed. It was suggested that consideration be given to a comparatively high level of illumination which will provide adequate light even if the tunnel walls and roof are not cleaned but are allowed to become dirty.

3.2 SEAWAY REQUIREMENTS

The Seaway requirements are generally defined on drawing 4011-R-3.

Throughout the construction period, the Seaway Authority requires a 300 foot channel width extending eastward from the westerly tie-up wall and to a bottom elevation of 308.5 maximum. Following completion of the twinning project, the channel will be approximately 400 feet in width at this location, including the centre tie-up wall. The maximum bottom elevation will be 305.5.

The water elevation maintained in the canal during the navigation season is 335.5 average and 337.5 maximum. It is required that the concrete work of the tunnel be maintained at least 35 feet below the normal water level of 335.5 with bottom protection being permitted for up to a 5 foot thickness over the tunnel. The bottom protection may be eliminated for the width of the drainage ditch.

The Seaway Authority requires three entrance tie-up walls. The Westerly wall is already in existence and must be maintained operable throughout its length, throughout all navigation seasons during construction. The centre and easterly tie-up walls will be built by the Seaway Authority following the completion of the tunnel and the Seaway Authority has suggested that provision be made for them to be supported upon the tunnel structure. The maximum thrust anticipated against a tie-up entrance wall for purposes of design is that caused by a 30,000 long ton displacement vessel approaching the wall at a velocity of one third of a foot per second normal to the wall.

The main canal dykes are of course extremely important to the Seaway Authority. Their stability and water tightness must be very carefully protected and therefore winter construction of clay dykes is not normally permitted.

3.3 UTILITY SERVICES

The existing utilities which must be accommodated throughout and after tunnel construction are described in Chapter 2.

Discussions have been held with the several utility agencies concerning utility requirements within the tunnel. The utilities services for which space has been requested in the tunnel are as follows:

Telephone	-	12 - 4" O.D. ducts
Sanitary Sewer	-	1 - 24" to 30" dia.
Water	-	1 - 12" dia.
Power	-	8 - 5" O.D. ducts

Having obtained these requirements, the ability of the minimum tunnel cross-section to accommodate them was studied and it was found that the telephone and power ducts could be accommodated without change to the tunnel cross-section. In order to accommodate the sewer and water requests, however, it would be necessary to increase the tunnel in size with an attendant substantial increase in cost. Since both the sewer and water are requirements of the City of St. Catharines, the following letter was therefore written to the Mayor:

August 26, 1964

COPY

His Worship Mayor Ivan D. Buchanan
The Corporation of the City of St. Catharines
Municipal Building
Church Street
St. Catharines, Ontario

Dear Sir: RE: SERVICES THROUGH CARLTON STREET TUNNEL

We are acting as engineers for the Ontario Department of Highways in the design of this four-lane vehicular tunnel under the Welland Canal at Carlton Street. The project is presently scheduled for completion in 1968.

We have held preliminary discussions with the various utility authorities to determine what utilities may be required in the tunnel.

After studying the requirements of the various authorities, we find that the telephone and electrical requirements can be accommodated without affecting the cost of the tunnel. We find, however, that in order to accommodate a sanitary sewer, (perhaps 24 inches to 30 inches), and a water main, (possibly 12 inches), which might be required by the City of St. Catharines through the City Engineer's Department, or the St. Catharines' Water Commission, it would be necessary to increase the overall width of the tunnel by several feet and, consequently, the overall cost of the project as well. We estimate that the cost increase necessary to accommodate the water and sewer lines for St. Catharines would be in the order of \$150,000.

Since some construction work is to begin on this project during the winter of 1964-65, it is necessary to reach an early decision as to whether or not the tunnel is to be increased in size to accommodate the sewer and water lines.

We can find no reason to recommend that this additional cost to the tunnel structure of about \$150,000 should be borne by our client, the

Copy of letter to Mayor Ivan D. Buchanan (continued)

Ontario Department of Highways or by the St. Lawrence Seaway Authority, who is sharing in tunnel construction costs. We conclude, therefore, that if the tunnel is to be increased in size to accommodate these services, the additional cost of about \$150,000 would be borne by the City of St. Catharines, who may wish to distribute the cost in some way between the Water Commission and the City Engineer's Department.

We are not in a position to assess if this would be a wise capital expenditure at the present time for the City of St. Catharines. We conceive it as entirely possible that it would be a more sound financial programme to carry the sewer and water mains across the Canal independently of the tunnel as required. We shall, therefore, not increase the size of the tunnel to accommodate the possible sewer and water lines unless we receive appropriate advice from the City of St. Catharines.

Yours very truly,

GENERAL ENGINEERING COMPANY LIMITED

Norman D. Lea (Signed)

CC: Mr. J. Walter
 Department of Highways, Ontario

 Mr. L. H. Burpee
 The St. Lawrence Seaway Authority

CHAPTER 4

GENERAL SCHEMES

4.1 GENERAL

Five basic general schemes have been considered for constructing the main portion of the tunnel beneath the canal. These are as follows:

- 1) Float in elements.
- 2) Sheet pile walls.
- 3) Small pre-cast sections.
- 4) Cut and cover.
- 5) Shield driven.

Each of these schemes has been developed to take advantage of the winter season during which the canal is drained and each has provided for the maintenance of the westerly tie-up wall in operation during the navigation seasons during which construction is carried out.

4.2 SCHEME 1 - FLOAT IN ELEMENTS

This scheme is described in general on the attached drawings 4011-R-4 and 4011-R-5. The subaqueous tunnel is composed of concrete elements, each between 250 and 300 feet in length and each of sufficient cross-section to contain the four highway lanes of traffic. These elements are constructed in a dry dock on the east side of the canal. They will be so designed as to float with a small freeboard when temporary bulkheads are placed in either end. These elements will be floated into position at the end of a navigation season and will be lowered onto previously prepared beds in a trench in the bottom of the canal by the gradual lowering of the water in the canal. The joints between the tunnel elements will then be cast in the dry in the winter non-navigation season when the canal is drained.

The usual waterproof membrane will not be required for this tunnel since the tunnel elements will bear upon a clay bed and will be surrounded with a clay seal.

The method of penetrating the dykes is discussed in greater detail in Chapter 7. It will be observed, however, on drawing 4011-R-5 that two methods of dyke penetration are shown as illustrative. The westerly dyke is shown as being penetrated by building in place a concrete dam during a winter non-navigation season, while the easterly dyke is shown as being penetrated by a floated in section of tunnel with the attendant necessary surrounding earth cofferdam.

4.3 SCHEME II - SHEET PILE WALLS

This scheme is shown on the attached drawing 4011-R-6. Under this scheme the tunnel walls are composed of three parallel rows of piling. The two exterior walls are composed of interlocked sheet piling and the intermediate wall is composed of bearing piles. These piles are driven down into the dense-stratified-granular-material in the winter non-navigation season from the bottom of an excavation made to the elevation of the underside of the top roof slab. The reinforced concrete roof slab is then poured in place using the soil as the bottom form. The clay seal may then be placed over the roof of the tunnel and excavation proceeds inside the tunnel with the bottom slab and underdrains placed in short sections. At least two winter seasons are required for driving the piles and casting the roof slab.

4.4 SCHEME III - PRE-CAST SECTIONS

This scheme is illustrated on drawing 4011-R-7. The main tunnel with a cross-section similar to Scheme I is composed of a number of comparatively small pre-cast concrete units, each weighing not more than about 35 tons. These pre-cast concrete units are assembled in position in a trench during the winter non-navigation season.

This scheme may be compared readily with Scheme I since the cross-section of the tunnel is very similar and since both are pre-casting schemes. It has been demonstrated that Scheme I is substantially less expensive than Scheme III largely because of the less expensive method of handling and placing the pre-cast concrete elements. Because of its higher cost than Scheme I, the difficulties of insuring water tightness, this Scheme III has not been given further consideration.

4.5 SCHEME IV - CUT AND COVER

This scheme involves using a standard cut-and-cover method for constructing the tunnel in-situ. A trench would be excavated, the tunnel formed and concrete placed during the winter non-navigation season.

It has been demonstrated that the length of the non-navigation season is not adequate to be confident of completing construction of the tunnel by this method in three non-navigation seasons. This method was therefore found to be impractical and not considered further.

4.6 SCHEME V - SHIELD DRIVEN

Consideration has been given to the use of a shield with a tunnel lining being placed behind the shield as it is advanced by jacking.

This scheme, however, has been found to be impractical because of the small amount of cover above the shield and because of the varying soil conditions from hard till to firm clay which would be encountered within the height of the shield itself.

4.7 COMPARISON OF SCHEMES I AND II

It is apparent from the above that the most practical schemes are numbers I and II. Therefore these have been compared in some detail.

Scheme II, which involves the driving of three rows of piling is somewhat tighter in schedule than Scheme I and this is an advantage to Scheme I.

As discussed in Chapter 2, there is a possibility of artesian water in the dense-stratified-granular-material. If this water is encountered in the detailed soils investigation, and if it proves impractical to lower the head by deep well pumps, serious problems will be created for both Schemes I and II, but these problems would be almost insurmountable for Scheme II, whereas they could certainly be overcome with Scheme I.

The estimated construction cost of Scheme I is over half a million dollars less than that of Scheme II. This, therefore, is decisive in favour of Scheme I.

CHAPTER 5

TIE-UP WALLS

5.1 GENERAL

Provision for the tie-up walls which are required by The St. Lawrence Seaway Authority is an important consideration even in preliminary engineering. The westerly tie-up wall is different from the other two, not only in that it is existing, but also because it is the only one which may be made an integral part of the dyke. It is, therefore, considered separately. The Seaway Authority has given consideration to supporting the tie-up walls independently of the tunnel and GECO has considered supporting them upon the tunnel.

5.2 DESIGN CRITERION

Tie-up walls may normally be designed for a 30,000 ton ship with an approach velocity normal to the wall of one third of a foot per second. For fendering deflecting half a foot, this creates a force of about 100 tons. For the unusual case of a ship out of control the approach velocity could be 2 feet per second with the force increased about 50 fold.

5.3 WESTERLY WALL

This wall will be distinctively different depending upon whether the dyke is pierced by sheet piling or a floated-in tunnel section.

If sheet piling is used, the vertical load from the tie-up wall and its deck slab may be taken upon the tunnel roof and the thrust from the ship may be taken through the comparatively thin deck slab to the dyke itself.

Should the dyke be pierced by a floated-in tunnel section, then a temporary tie-up wall must be built for one navigation season during which the tunnel trench excavation is open. The most convenient structure would be a floated-in concrete crib supported independently of the tunnel.

5.4 CENTRE AND EAST TIE-UP WALLS

A preliminary design has been carried out for a tunnel supported structure containing one ventilation building, in the centre tie-up wall. Because the tie-up wall is limited in width to approximately 65 feet, the structural walls and the ventilation building become part of the same structure. With

a fendering system designed to deflect five feet the scheme is practical and could stop a ship out of control but the cost, including the accommodation of the extra forces in the tunnel structure, is high. Therefore, this arrangement would not be considered unless it were to effect a substantial saving in the ventilation system. As discussed in Chapter 8, it does not effect such a saving and, therefore, this scheme has been discarded.

Several schemes have been developed for a tunnel supported tie-up wall structure which will fail internally before stopping a ship which is out of control. The most practical of these involves a heavy concrete slab on a gravel mat on top of the clay seal over the tunnel. This slab becomes the foundation for a structural steel wall designed to fail at twice the normal wall design load. The slab would be designed so as not to slide even when the structural steel wall fails. This scheme is considered practical.

It is also considered practical to support the centre and easterly walls independently of the tunnel upon cells or caissons on either side of the tunnel. In this design, it would be important to keep the cells or caissons well clear of the tunnel - say 10 feet, so that if one were hit directly by a ship and moved several feet, it would still not damage the tunnel.

The practical schemes described in the preceding two paragraphs are both independent of the tunnel construction operation. It is understood that these tie-up walls may, in fact, not be built for several years after the tunnel. It is understood that the selection between the two schemes would be made by the Seaway Authority who would also design and build these walls. The centre and easterly walls are, therefore, not allowed for in the tunnel cost estimates. Provision will, however, be made in the tunnel design to support the tie-up walls, of the type designed to fail internally before damaging the tunnel, if this is requested by the Authority.

CHAPTER 6

PIERCING MAIN DYKES

The method of piercing the main dykes is one of the most significant design considerations. Because of the depth of the tunnel and the impracticality of driving the tunnel with a shield, it is necessary that a section of the main dyke on each side of the canal be removed and rebuilt where the tunnel must pierce through it.

There are two basic methods of achieving this. They are as follows:

- 1) A floated-in tunnel section with rebuilt earth dyke above it, and
- 2) A combined tunnel, dyke and ventilation building structure built in place in concrete in the winter.

Both of these schemes have been pursued far enough to establish their practicality. The selection between them will depend, to some extent, upon the detailed soils investigation and will be an outcome of the design development phase.

For purposes of illustration and for cost estimating at this preliminary report stage, the floated-in-section method has been employed for piercing the east dyke, and the built-in-place method for piercing the west dyke. Both methods are illustrated on drawing 4011-R-5.

The floated-in-section method requires that a large area to the landward side of the dyke be enclosed by an earth cofferdam. The main dyke is then removed, the tunnel section floated in and the dyke replaced. The dyke replacement is largely of gravel if it is built in the winter. The advantage of the floated-in-section method is that the entire closed tunnel section rests on a similar foundation. This method is more suited to the piercing of the easterly dyke where a temporary dyke can be built out into the canal to permit the rebuilding of the main dyke in the dry in the summertime.

For the built-in-place scheme, the dyke for some distance on either side of the tunnel is partially enclosed and protected in steel sheet piling. During the winter non-navigation season the gap is excavated and a short section of combined dyke and tunnel built in the dry of concrete supported on piles. The advantages of the built-in-place scheme are that it gives a shorter closed section of the tunnel, that it avoids irregular loading upon

the floated-in section which is a foundation on elastic support, and that only a minimum amount of the main dyke is removed.

The two methods are sufficiently close in cost that no selection can be made between them at this stage. Such a selection must await the more detailed studies of the design development phase, when more complete soil information will be available.

May 1941 - 1942 - 1943 - 1944 - 1945

CHAPTER 7

APPROACHES

The uncovered sections of the roadway at the east and west ends of the tunnel providing a transition from the normal highway type of construction to the subaqueous portion of the tunnel.

For purposes of the preliminary report, similar approach design has been adopted for all schemes. This consists of normal highway cross section as shown in the functional report and on the attached drawing 4011-R-2 down to a depth of about 10 feet below ground level.

From a depth of about 10 feet below ground level, down to the tunnel portal, this preliminary report is based upon a trough type concrete section with cantilever slabs on either side to engage additional soil. This cross-section is designed to resist all hydrostatic uplift and is a conservative and positive scheme. It is possible that in the design development phase a less expensive method of constructing this section of the approach employing under-drains beneath a normal roadway slab will be demonstrated to be safe and acceptable.

For a distance of about 150 feet before entering either portal, sunscreens have been included over both roadways. The supports for these sunscreens also act as struts between the concrete retaining walls. The length of the sunscreens is selected to permit the retina of the eye of a normal person to contract from the bright outside sunlight to the illumination provided within the closed section of the tunnel.

CHAPTER 8

MECHANICAL AND ELECTRICAL

This chapter contains a description of the mechanical and electrical systems which have been allowed for in the estimate accompanying this report. For purposes of the preliminary engineering, design work has not been carried out on the mechanical and electrical systems but selection has been made of comparable components from previous projects so that adequate monies are included in the preliminary estimate.

8.1 EXTERIOR DRAINAGE

The existing exterior drainage systems, of course, must be maintained and either diverted or carried across the tunnel excavation on an aqueduct. Around the rim of the tunnel approaches is constructed a low earth dyke with the elevation of its crest so selected that any flooding which might be created by overtopping of existing drainage ditches would not flood the tunnel approaches even though it might flood some of the surrounding land.

8.2 INTERIOR DRAINAGE

Within the drainage basin of the tunnel and the tunnel approaches three sumps are provided, one at each portal and one at the centre of the tunnel. Pumping from these sumps will be automatic only, activated by water level indicators, and will discharge into the canal.

8.3 LIGHTING

Continuous fluorescent lighting has been included to provide adequate illumination even if the tunnel walls and roof are not cleaned. The normal tunnel lighting will operate continuously, night and day, whereas some additional illumination just inside the portals is controlled by exterior light meters to operate only when the outside light exceeds a certain intensity. Overhead sun screens for about 150 feet outside each portal will provide a gradual transition from artificial light to daylight.

8.4 VENTILATION

The only ventilation system which need be considered for the Carlton Street Tunnel, because of its short length, is a longitudinal system. Trans-

verse or semi-transverse systems are much more expensive and are required only for longer tunnels.

Several schemes for longitudinal ventilation will be compared in greater detail at the design development stage. Some consideration has already been given to having one centrally-located ventilation building with all of the ventilation taking place from this one point. It has been found that the construction of the ventilation building with the associated civil work would be more expensive than the two buildings close to the ends and preliminary indications are that the mechanical and electrical systems would be just as expensive for the single installation, and perhaps more expensive than for two.

Even for ventilation buildings located near the portals, there are two possible schemes, one with the ventilation buildings located right at the portals and the other with the ventilation buildings located approximately 200 feet in from the portals. The latter has been adopted for present preliminary design considerations, but the two schemes will need to be investigated further. The system adopted for the present, therefore, consists of two vertically mounted axial-flow fans, one fan for each tube housed in separate ventilation buildings located at each end of the tunnel. Each tube is ventilated by means of fresh air drawn in through the fresh air intake and discharged obliquely into the tube in the normal direction of traffic for the particular tube. The quantity of air delivered in this direction by the fan will vary to some extent depending on traffic density, traffic speed and wind effects at the portals. The fans will be operated automatically by the fire safety system and the carbon monoxide safety system.

8.5 FIRE SAFETY

Three fire stations are provided at intervals along the centre partition of the tunnel, each station includes an emergency exit to the other tube, fire hose, hand dry chemical extinguishers, sand and emergency telephone.

The use of the fire hose or the extinguisher sends an alarm to the control room and to the fire department. A series of fire detection devices are also employed which operate on temperature or rate of temperature rise and which trigger the ventilation system as well as sending alarms to the control room and the fire department.

8.6 CARBON MONOXIDE SAFETY

The tunnel air will be continuously sampled from several points and continuously analyzed in carbon monoxide analysis. Should the carbon monoxide content exceed a certain predetermined value, alarms are sounded and the ventilation system activated.

8.7 CONTROL ROOM

A control room is provided in one of the ventilation buildings from which the mechanical and electrical equipment can be manually controlled and where all automatic control devices are maintained and signals from alarms are sounded. Alarm signals will also be sounded in the Department of Highways Administration Building at the Garden City Skyway, which point would be continuously staffed with emergency personnel for the operation of all major structures in the region.

8.8 HEIGHT CONTROL

Allowance has been made for height control devices at the approaches of the tunnel which would send out alarms and operate stop signs.

8.9 RAMP HEATING

Provision has also been made for electrical ramp heating although further study is necessary to determine whether or not this should be actually built.

8.10 EMERGENCY POWER

Diesel standby power for operation of pumps and emergency lighting has been included for in the preliminary report estimate. Further study is necessary of the possibility of tying in with the Seaway sources of emergency power rather than having a separate standby system.

CHAPTER 9

SCHEDULE

9.1 GENERAL

The proposed construction schedule covers three years, starting late in December 1964, and opening the structure to traffic in late 1967. The entire operation is, of course, geared to winter operations when the canal will be drained. This chapter contains both a written description of the schedule and an illustrative bar chart. The schedule is illustrated by the construction sequence drawing 4011-R-5.

9.2 WINTER 1964-65

The soil investigation program is to be completed by October 1, 1964, so that by November 1, 1964 the design drawings and tender documents may be completed for an earthwork contract to be called about November 1, 1964.

This proposed earthwork contract is to be for the excavation of a portion of the tunnel trench in the canal bed. Some of the material is to be stockpiled in preparation for building dykes the following summer. Some may be used for some drydock dyke construction during the winter.

The principal reason for undertaking some excavation the first winter is to permit a full-scale study of soil slope stability and of sedimentation conditions in the canal. The work done will be useful to the project and will significantly relieve the construction schedule for the second winter.

It is expected that tenders will be ready for call by November 1, and tenders should be received by November 20, 1964. Such a schedule would allow about ten days to award a contract, and a further fifteen days for the contractor to move onto the job before the canal is drained.

9.3 SUMMER 1965

An earth cofferdam will be constructed around the east end of the tunnel. This cofferdam will be built from the stockpiled material excavated from the canal bed during the previous winter. It may also utilize some

material from the eastern approach excavation.

On the west side of the canal another cofferdam will be constructed. Whether this cofferdam is composed of sheet piles or earth will depend upon the method finally selected for piercing the dyke.

Should the built-in-place method be adopted for piercing the dyke, piles would be driven through the deck of the tie-up wall during the summer of 1965.

9.4 WINTER 1965-66

During the period when the canal is drained, the main trench for the tunnel will be excavated for its full length. The tunnel bed will be prepared and drainage sumps built at each end.

Should water pressure in the dense-stratified-granular-material be a problem it may be necessary to do this excavation and bed preparation in the wet.

The concrete slab and dam will also be built in the breach of the west main dyke should this be the method of dyke penetration finally selected. Before the concrete can be placed, the excavation must be completed and both the sheet piles and the bearing piles must be driven.

9.5 SUMMER 1966

During the summer of 1966 the tunnel sections will be built in a prepared drydock while at the same time a dyke will be built around the drydock area utilizing material from the excavation. Work outside the dykes is generally not critical. Some will be done during the summer of 1966 and some during the summer of 1967.

The last two weeks or so, before navigation closes, would be used to inspect the tunnel bed by diver. It is expected that the tunnel bed will be in satisfactory condition to receive the tunnel elements; however, this must be confirmed by an inspection.

9.6 WINTER 1966-67

Immediately after the end of navigation a break will be made in the cofferdam dyke around the dry dock in order to enable the tunnel sections to be removed.

The tunnel sections will then be placed in position and sunk on the pre-

pared bed by lowering the water in the canal. When all sections are in place, and checked for location and elevation, the canal will be drained and the trench pumped dry. The tunnel sections will then be connected by concreting in the dry and the east ventilation building will be constructed.

Immediately after connecting the tunnel elements the tunnel will be back filled along the sides and over the top, using compacted impermeable clay for a complete seal plus granular scour protection where required. A cofferdam may then be built inside the permanent dyke location over the tunnel in order to permit reconstruction of the breach in the east main dyke in the summertime under dry conditions.

9.7 SUMMER 1966-67

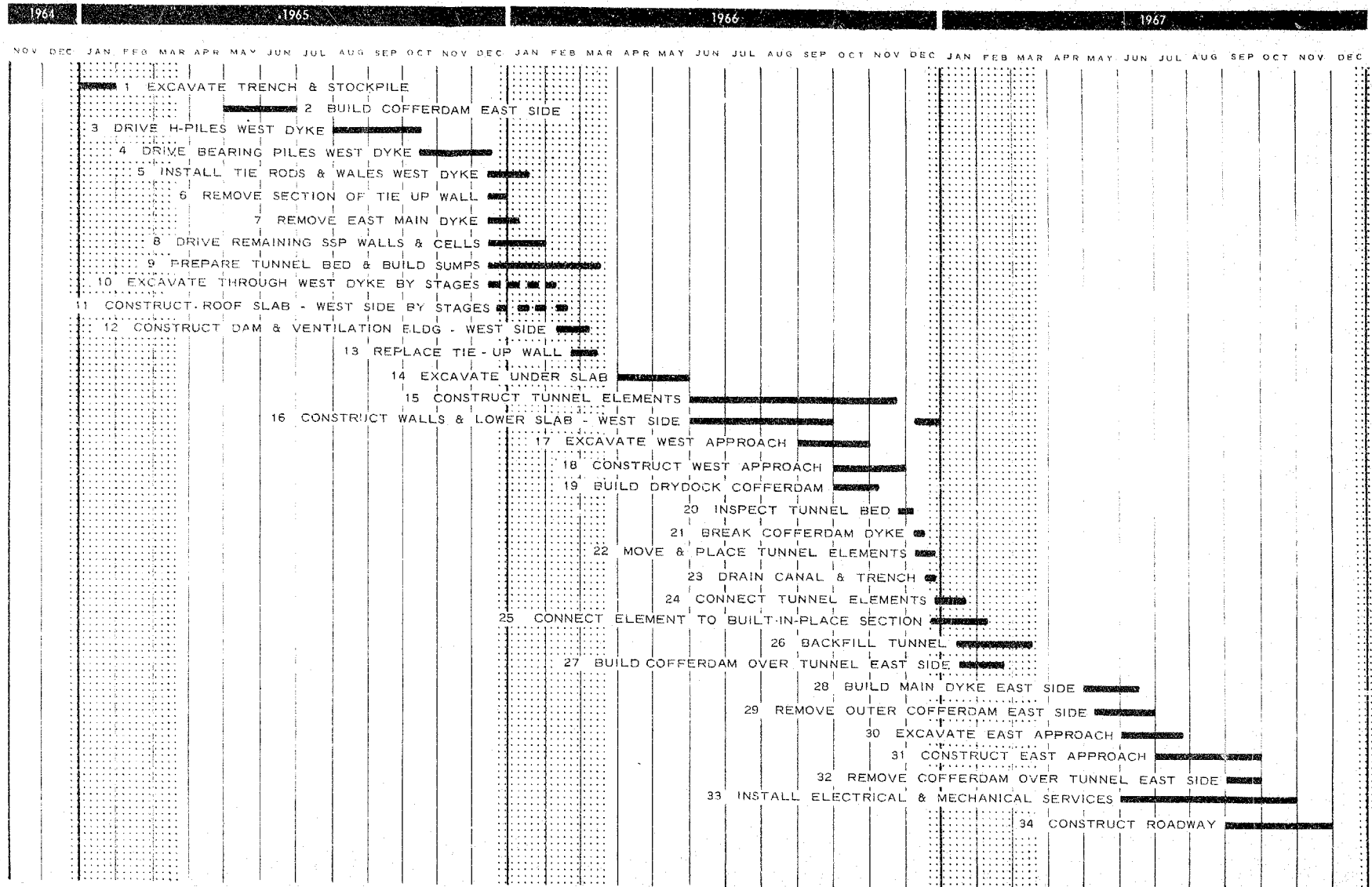
Construction of the east approach section will begin early in 1967. As soon as weather conditions permit the main east dyke will be constructed in order to permit the removal of the outer cofferdam. The excavation and construction of the east approaches may then be completed. The installation of the roadway in the tunnel, as well as the services and pipes required, will be completed during the summer of 1967.

It is expected that the tunnel will be open to traffic by December 1967.

9.8 CONTROL

It is important that careful construction scheduling control be exercised, particularly during the critical winter periods. Therefore, it is planned that the construction schedule will be set up on critical path and that weekly monitoring reports will be prepared for control during the critical period.

S C H E D U L E - S C H E M E 1



CHAPTER 10COST

The following is a summary of the preliminary estimate of the cost of the general scheme recommended in this report:

1,400,000 c.y.	Earthwork Total	\$ 2,100,000
sum	Concrete Tunnel Elements in place with approx. 25,000 c.y. concrete and making about 1200 l.f. closed tunnel	2,000,000
sum	Sumps and ventilation buildings	500,000
700 l.f.	Approach structures	1,000,000
sum	Mechanical & Electrical	1,000,000
500 l.f.	Remove and rebuild tie- up wall	400,000
	Contingency	<u>1,000,000</u>
	Total	<u>\$ 8,000,000</u>

CHAPTER 11

CONCLUSIONS

1. The Carlton Street area is a good practical location for a tunnel crossing of the Welland Canal.
2. An alignment parallel to the Carlton Street and 200 feet south has been adopted for this report. During design development, the economy of moving the tunnel about 50 feet north will be investigated. A relocation about 1,000 feet south would reduce the tunnel cost by over half a million dollars but would greatly complicate the connecting street system and, therefore, is not recommended.
3. All known tunnelling schemes have been investigated. The two most promising have been studied in some detail. The one involving the floating into place of large precast tunnel elements has been found to be most attractive, both in schedule and in price.
4. Allowance has been made for the existing westerly tie-up wall to be rebuilt where it must be removed for tunnel construction. The centre and easterly tie-up walls which the Seaway intends to build require special design where they cross the tunnel. These may be either completely independent structures with foundations well clear of the tunnel, or concrete slabs insulated from the tunnel by gravel pads and supporting steel superstructures designed to fail without damage to the tunnel if hit an unusual blow by a ship. It is expected that The St. Lawrence Seaway Authority will select, design and build one of these alternatives, sometime, after the tunnel is completed - possibly several years later.
5. Two methods have been developed for piercing the main dykes. Further study, soils information and design work is required to choose between them. The choice when made will substantially affect the property requirements particularly on the west side of the canal.
6. Gravity approach structures have been adopted for the present but other less costly types will be investigated during design development.

7. The following mechanical and electrical tunnel services have been allowed for:

- diverted exterior drainage
- three interior sumps with automatic pumping installations.
- continuous fluorescent lighting with automatic control of artificial light intensity at the portals
- sun screens for about 150 feet outside each portal
- longitudinal ventilation with vertically mounted automated axial flow fans discharging obliquely - one at each portal
- fire detection devices, fire hoses and extinguishers all sounding alarms and activating the ventilation
- carbon monoxide detection system also sounding an alarm and activating the ventilation
- control room in one ventilation building with alarms also sounding in Highways Administration Building
- vehicle height control with alarm and stop sign
- electrical ramp heating
- emergency diesel power

Each of these items require design development studies and each will be critically examined at that time with the possibility of some being eliminated.

8. Allowance has been made for dealing with all existing utilities which will be affected.
9. Utility agencies have expressed their desires for services through the tunnel. It has been found that power and telephone can be accommodated conveniently but that provision for the desired sewer and water would greatly increase the tunnel cost. The City of St. Catharines has, therefore, been given the opportunity of expressing their willingness to bear the extra cost of about \$150,000.

10. A schedule is recommended which calls for some excavation during the winter of 1964-65 and opening to traffic in the fall of 1967. The excavation is very important this winter to observe full scale slope stability and sedimentation. Delay of this by one year would delay the completion date by one year.
11. The preliminary cost estimate for the recommended scheme is \$8,000,000 for the structure from grade to grade for a total length of 2500 feet. This includes contingencies but excludes engineering and property.

APPENDIX "A"

NOTES RE DETAILED SOILS PROGRAMME

Exploratory Borings

- # 4 - West Portal
- # 5 - East Approach
- # 7 - In canal
- #11 - West Approach
- #13 - South Drydock

These boreholes to explore stratigraphy and obtain samples for laboratory identification tests. The dense-stratified-granular-material to be studied to detect possible artesian water pressure problem in the bottom of the excavation and, if this is present, to assess possibility of reducing pressure by pumping.

Vane tests and laboratory strength tests to be conducted, as required, to define the undrained strength of the firm clay and its variation throughout the site. A limited testing programme also required to explore long term strength of the firm clay and the upper hard clay till, also remoulded strength of the firm clay and the thixotropic, swelling and consolidation characteristics.

Exploratory Borings with Piezometer Installations

- # 6P - East Portal (Permanent)
- #8P -)
- #9P -) East Dyke (1964 and 1965 only)
- #10P -)
- #12P - West Portal (Permanent)

At these locations exploratory borings are required to determine the soil stratigraphy and gain further samples for laboratory testing if required.

At the same locations piezometers are to be installed to observe the piezometric water pressure in the various soil layers. At each location one piezometer is installed in the dense-stratified granular-material to intercept any aquifer which might exist in this material, one installed in the middle of each firm clay layer and one in the middle of the hard clay till. Should the dyke fill exist at the location, one is installed in the middle of it also. For P8, P9 and P10 another point is required at the upper quartile of the hard clay till.

Slope Indicators

S20)	West Dyke (Permanent)
S21)	
S22)	East Dyke (Permanent)
S23)	
S24		East Dyke (1964-1965 Only)

Five slope indicators are to be installed to observe movements in the dyke. These will each be deep enough to get their base into the very dense lower materials which will not move.

Stratigraphy will be observed in the boreholes made for slope indicator installations.

Slope indicators S20 to S23 are to be so located that they will not be damaged by construction and can thus be maintained through the construction period. Slope indicator S24 is located on tunnel centreline in the East Dyke where it can be used to observe carefully the effect of the winter 1964-65 excavation programme which will be so conducted as to cause some slight movement of the dyke, but not to endanger its stability.

Notes on Testing Programme

- field vane with mechanical strain control to be employed in an attempt to explain divergence of vane results.
- some laboratory vane tests to be carried out on fully remoulded specimens in an attempt to duplicate thixotropic effects observed with field vane.
- series of consolidation tests on samples from one borehole to be made so that an over consolidation depth plot may be constructed.
- for long term stability analysis effective strength

parameters c' & ϕ' to be determined from consolidated drained triaxial tests and consolidated undrained triaxial tests, with porewater pressure measurements on firm clay and hard clay till. Most of these to be extension tests.

- several swelling tests required on the firm clay for calculation of rebound and settlement movements during tunnel construction and thereafter.

Schedule

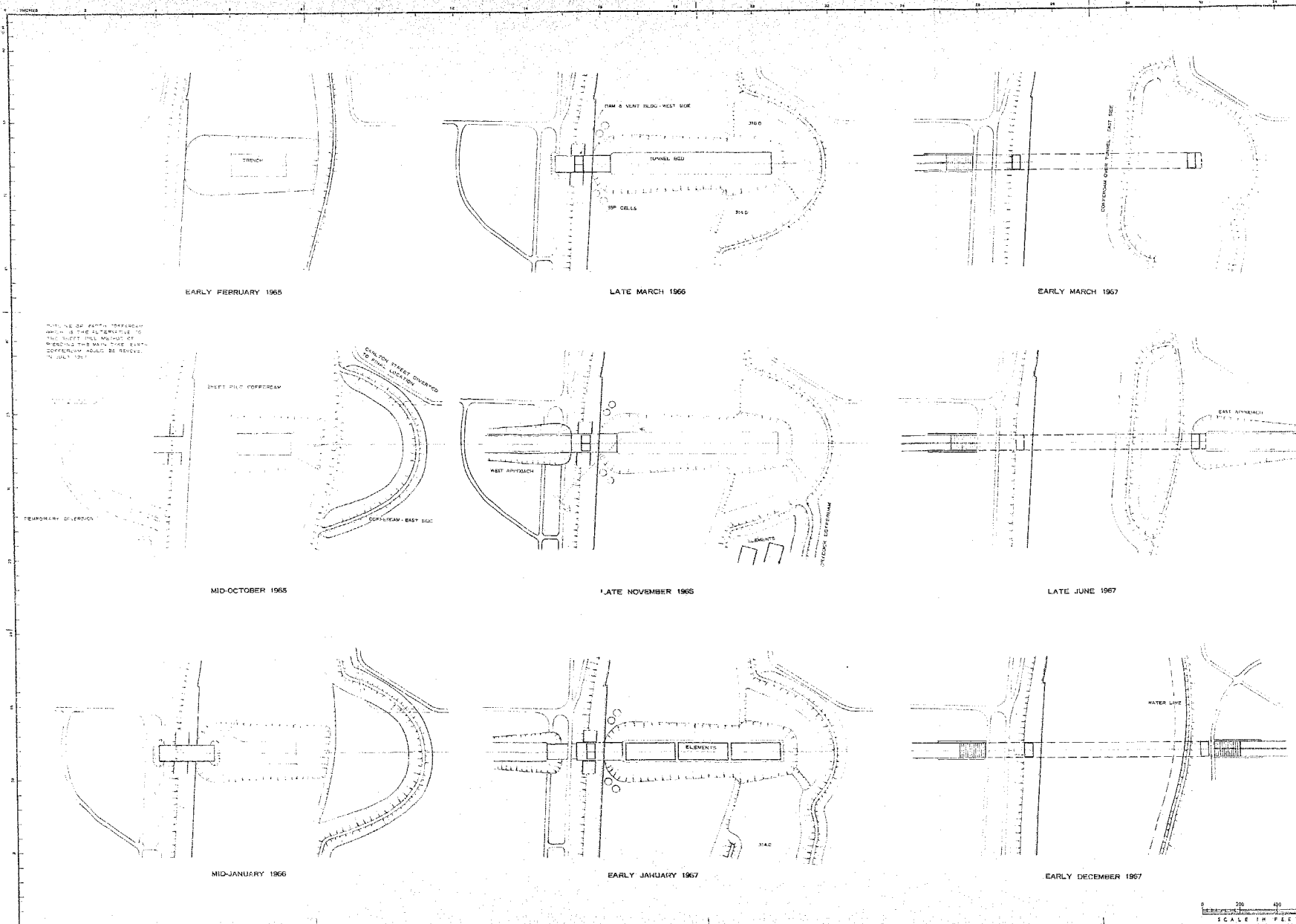
In order that plans may be complete for a November 1st, 1964 tender call for the 1964-65 winter programme, it is important that the exploratory borings, field vane tests and laboratory testing be completed and reported upon by the end of September.

Piezometer and slope indicators need to be installed by November 1st so that two reliable sets of observations may be made with a one month interval before the winter drawdown.

APPENDIX "D"

DRAWINGS

4011-R-1	Preconstruction Topography
4011-R-2	Department of Highways Requirements
4011-R-3	St. Lawrence Seaway Authority Requirements
4011-R-4	Scheme 1, Float-in-Elements General Arrangement
4011-R-5	Scheme 1, Float-in-Elements Construction Sequence
4011-R-6	Scheme 2, Sheet-piling General Arrangement
4011-R-7	Schemes 3 and 4 General Arrangement
4011-R-8	Soils Information Borehole Location Plan
4011-R-9	Soils Information Borehole Logs D.H.O. #1, #2 and #3
4011-R-10	Soils Information Preliminary Soil Stratigraphy
4011-R-11	Soils Information Composite Shear Strength Plot
4011-R-12	Soils Information Piezometer Location Plan
4011-R-13	Soils Information Piezometer Observations



REFERENCES	
DWG. NO.	DESCRIPTION
4011-R-5	SCHEME 1 GENERAL ARRANGEMENT

NOTE: THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH CHAPTER 3 OF THE REPORT

DWG. NO.	DATE	DESCRIPTION	APPROVED
4011-R-5	10/10/67	ISSUED FOR REPORT 24 SEPTEMBER 1967	
4011-R-5	10/10/67	CHECKED	
4011-R-5	10/10/67	APPROVED	

SCALE: AS SHOWN

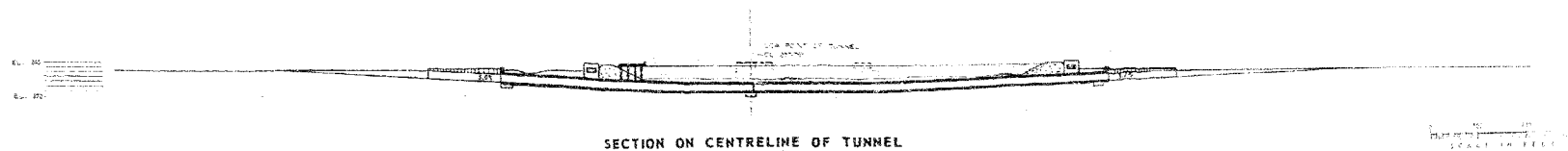
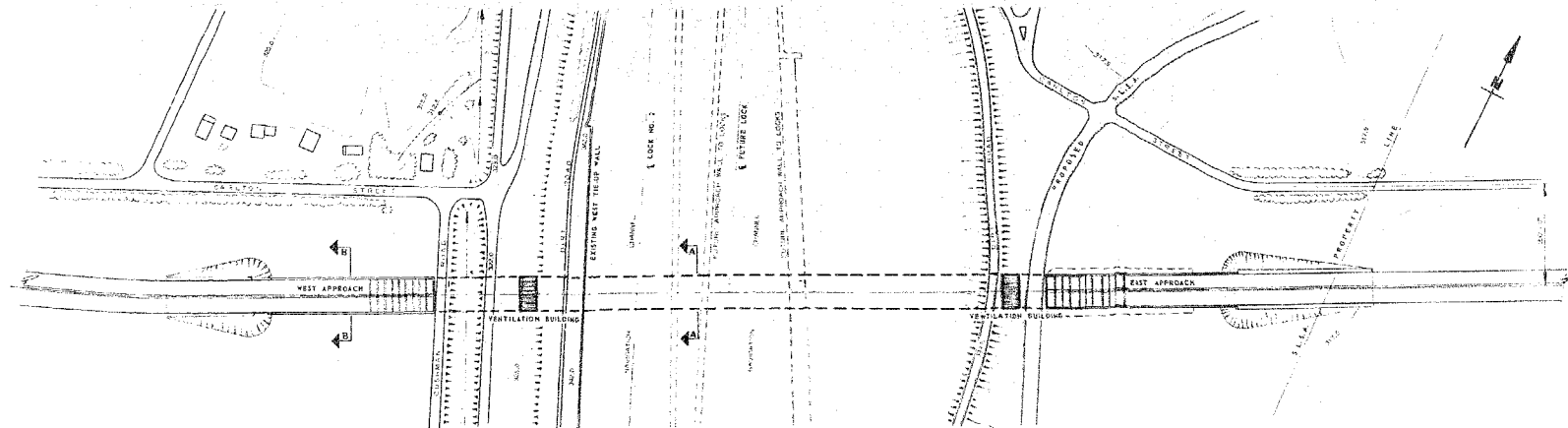
DEPARTMENT OF HIGHWAYS
ONTARIO

CARLTON STREET TUNNEL

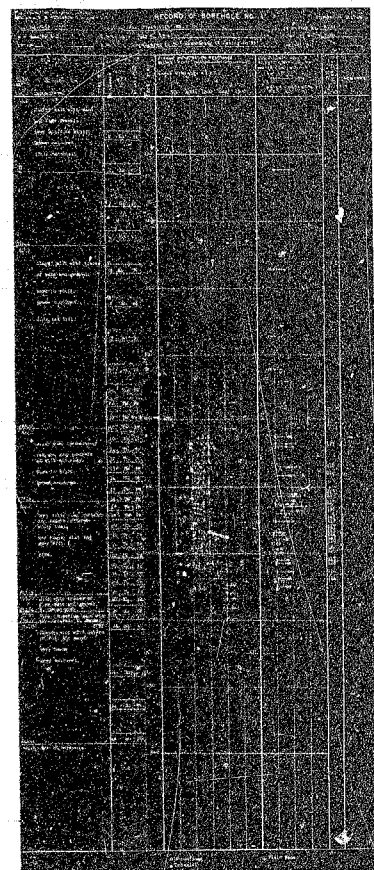
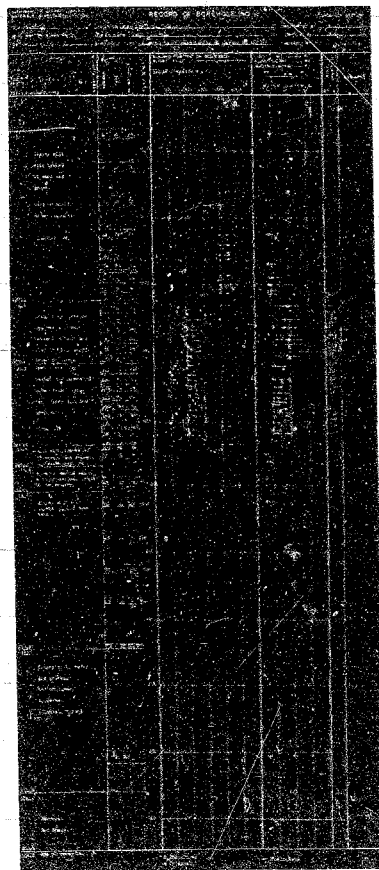
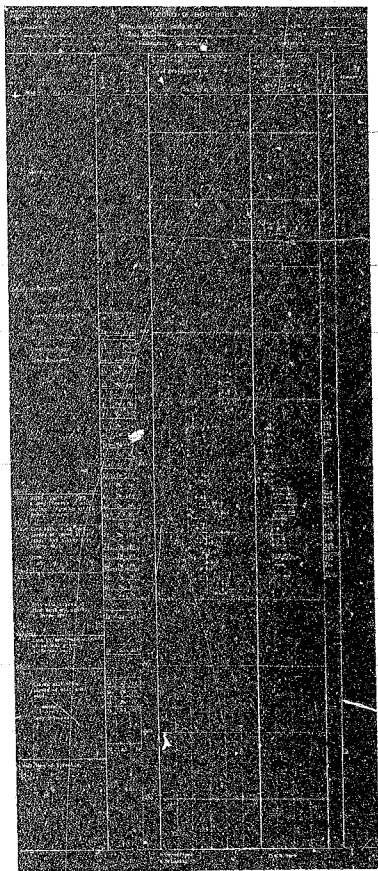
SCHEME 1
FLOAT-IN-ELEMENTS,
CONSTRUCTION SEQUENCE

DWG. NO. 4011-R-5

REV.

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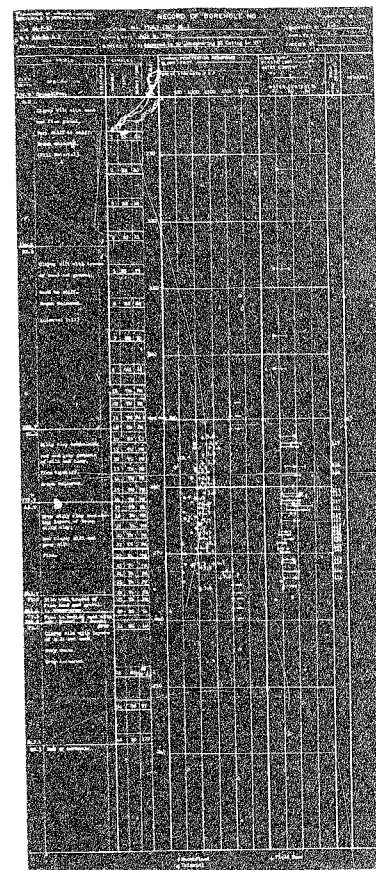
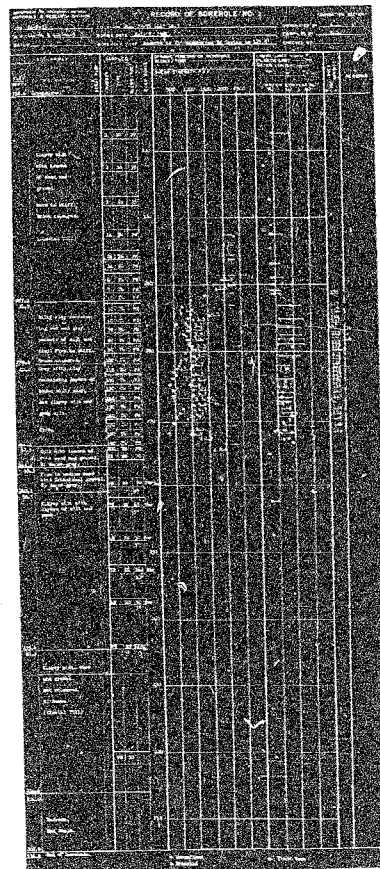
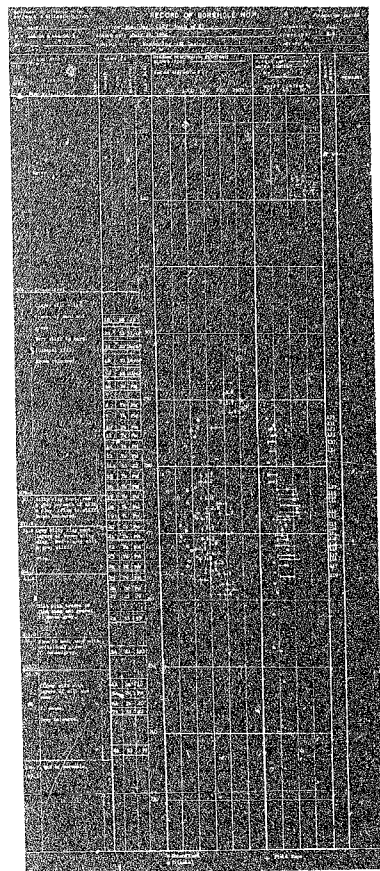


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SCALE	SCALE

DEPARTMENT OF HIGHWAYS	
ONTARIO	
GENERAL ENGINEERING	
CARLTON STREET TUNNEL	
SOILS INFORMATION	
BOREHOLE LOGS	
D.H.O. BOREHOLES 1, 2 & 3	
DWG. NO.	REV
4011-R-9	



GENERAL INFORMATION

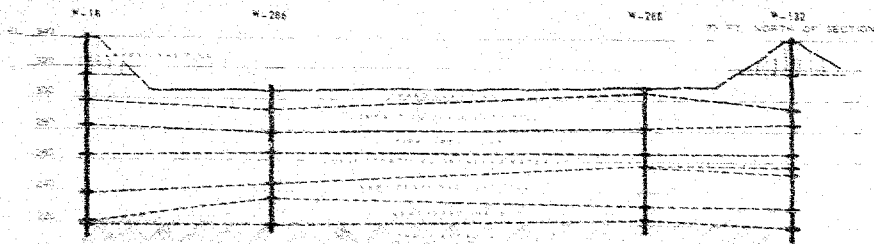
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SOIL INFORMATION

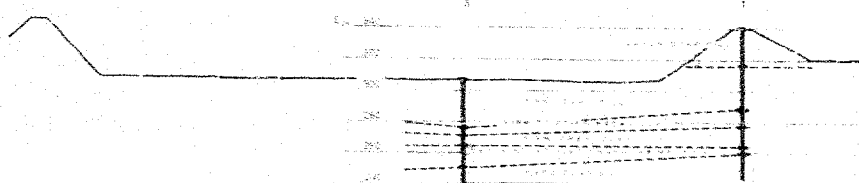
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DATE OF TEST

4011-R-9

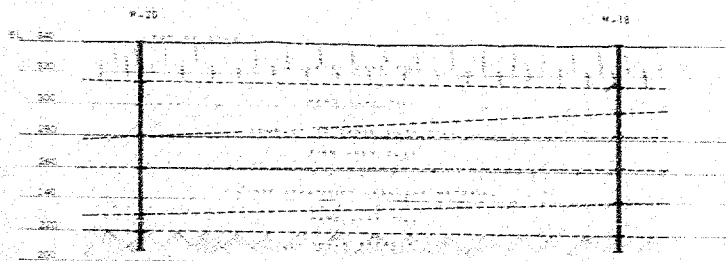


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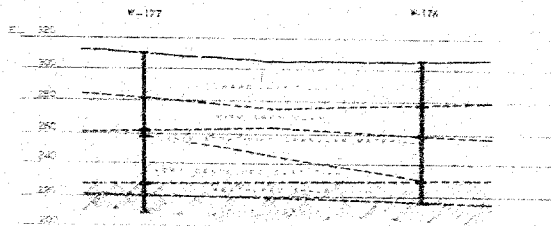


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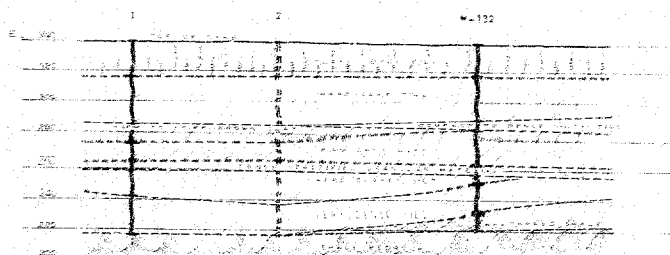
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| | HARD CLAY TILL | | WEATHERED SHALE |
| | COMPACT TO DENSE SILTY TILL | | SHALE BEDROCK |
| | FIRM GREY CLAY | | |
| | DENSE STRATIFIED GRANULAR MATERIAL | | |



SECTION C - C



SECTION D - D



SECTION E - E

DISTORTION RATIO 1:2



REFERENCES

DWG. NO.	DESCRIPTION
4011-R-8	BORERHOLE LOCATION PLAN
4011-R-9	BORERHOLE LOGS

NOTE: THIS DRAWING IS PREPARED FROM
FIELD BORERHOLE LOGS SHOWN ON
DRAWING NO. 4011-R-8 AND FROM
LOGS SUPPLIED BY ST. LAWRENCE
SEAWAY AUTHORITY

NO.	DATE	DESCRIPTION	PREPARED BY
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RECORD OF REVISIONS & SPECIAL ISSUES

ISSUED:
FOR REPORT DATED SEPTEMBER 1966

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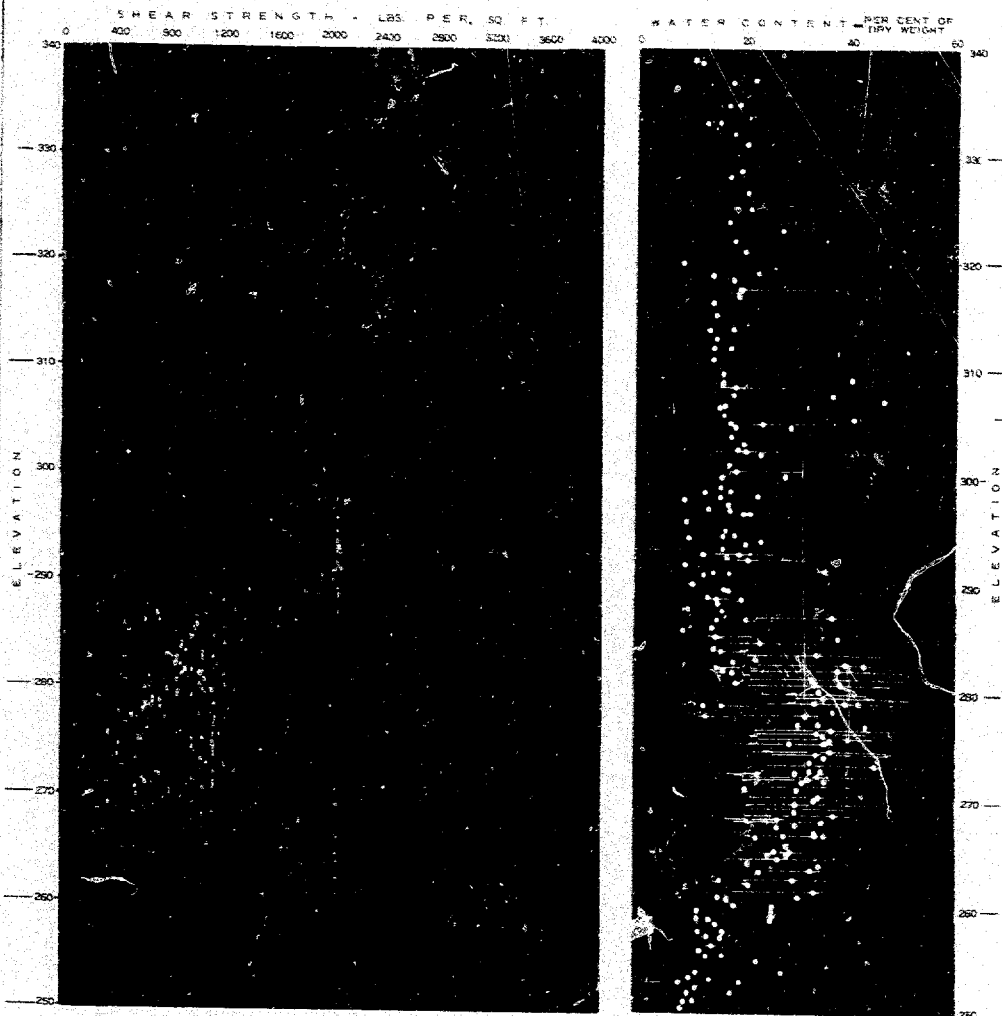
SCALE: AS SHOWN

DEPARTMENT OF HIGHWAYS
ONTARIO
GENERAL ENGINEERING
GENERAL ENGINEERING
COMPANY LIMITED
CARLTON STREET TUNNEL

SOILS INFORMATION
PRELIMINARY
SOIL STRATIGRAPHY

DWG. NO. 4011-R-10 REV.

WATER CONTENT



LEGEND

- FIELD VANE TEST BOREHOLE A
- UNCONFINED COMPRESSION TEST BOREHOLE A
- TRIAXIAL TEST BOREHOLE A

WATER CONTENT CODE

PLASTIC LIMIT	NATURAL WATER CONTENT	LIQUID LIMIT
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BOREHOLE CODE

BOREHOLE NO.	DESIGNATION	NOTE
1	A	BOREHOLES
2	B	
3	C	
		D. H. O.
W-177	D	1955-1964 BOREHOLES
W-176	E	
W-20	F	
W-286	G	
W-132	H	
		BY
W- 18	I	S. L. S. A.
W-288	J	

REFERENCES

DWG. NO.	DESCRIPTION
4011-R-R	BOREHOLE LOCATION PLAN

NOTE - THIS DRAWING IS PREPARED FROM
D. H. O. LOSS SHOWN ON DRAWING
NO. 4011-R-9 AND FROM LOGS
SUPPLIED BY ST. LAWRENCE SEARAY
AUTHORITY

NO.	DATE	DESCRIPTION	MADE BY	APP.
RECORD OF REVISIONS & SPECIAL ISSUES				

ISSUED:

FOR REPORT 3rd SEPTEMBER 1964

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DRAWN	J.E.	7 SEP 68
CHECKED	TRC	25 SEP 68
TECH. APP.	WJS	25 SEP 68
PROJ. APP.	WJS	25 SEP 68

APPROVED

[Signature] 2 Sept 68

SCALE:

DEPARTMENT OF HIGHWAYS
ONTARIO

GENERAL ENGINEERING

CARLTON STREET TUNNEL

SOILS INFORMATION

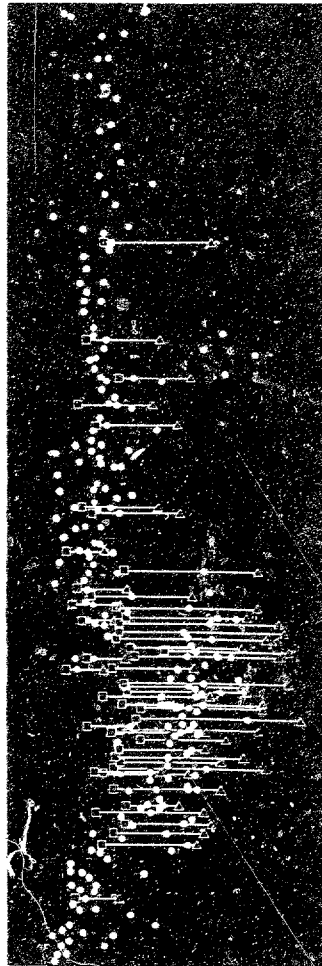
COMPOSITE

SHEAR STRENGTH PLOT

DWG. NO.	4011-R-11	REV.
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SHEAR STRENGTH

WATER CONTENT



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NO. DATE DESCRIPTION ISSUED

RECORD OF REVISIONS & SPECIAL ISSUES

ISSUED FOR REPORT NO. SEPTEMBER 1961

BY DATE

APPROVED

SCALE

DEPARTMENT OF HIGHWAYS ONTARIO

CARLTON STREET TUNNEL

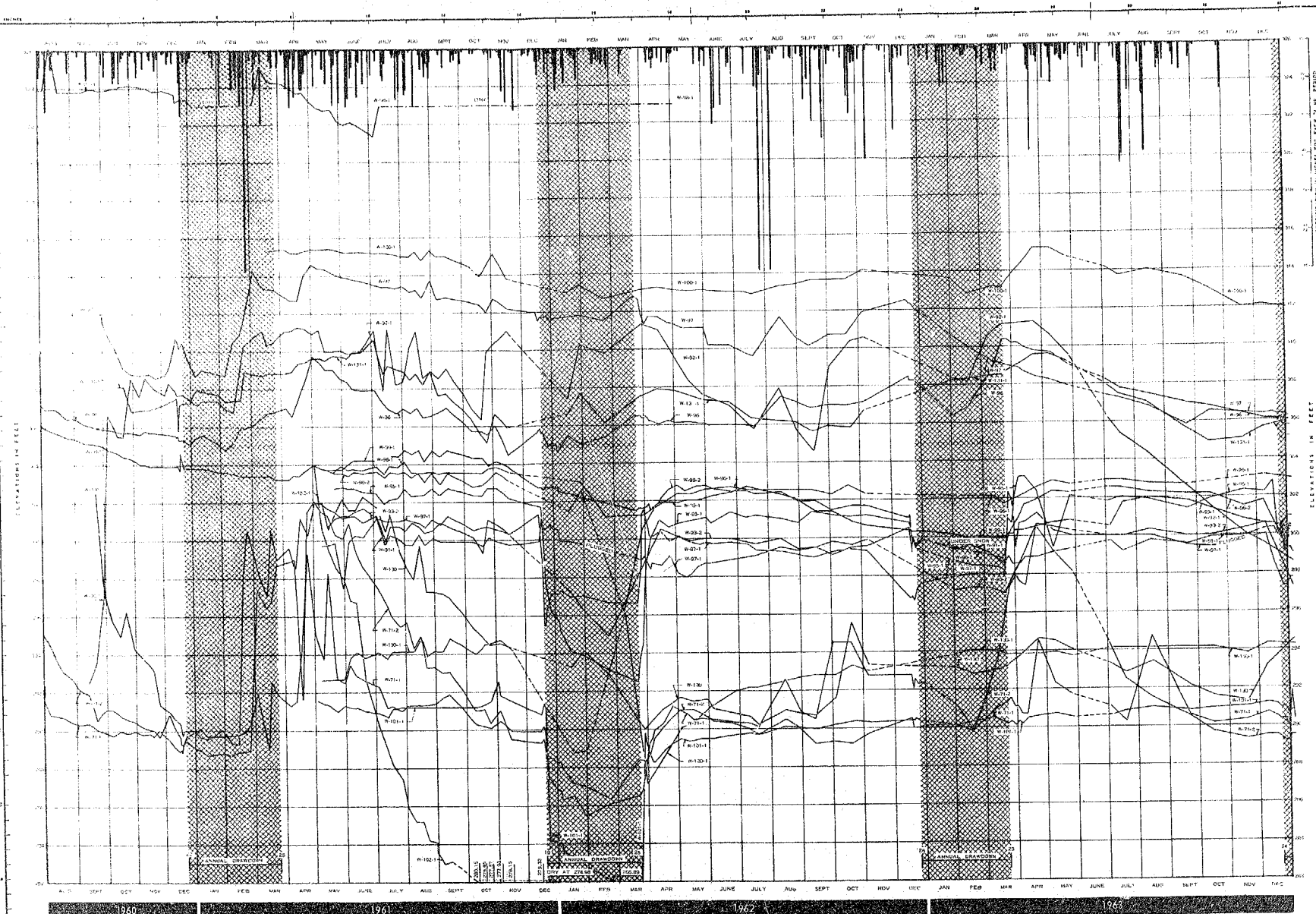
SOILS INFORMATION

COMPOSITE

SHEAR STRENGTH PLOT

DWG NO. 4011-R-11

REV.



REV. NO.

DESCRIPTION

REV. NO.

DESCRIPTION

NOTE: THIS DRAWING IS PREPARED FROM DATA SUPPLIED BY THE ST. LAWRENCE SEAWAY AUTHORITY.

PIEZOMETER LOCATION

STATION

DATE

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NO. DATE DESCRIPTION

RECORD OF REVISIONS & SPECIAL ISSUES

ISSUED FOR REPORT 34 SEPTEMBER 1962

DATE

BY

APPROVED

SCALE:

DEPARTMENT OF HIGHWAYS

ONTARIO

GENERAL ENGINEERING

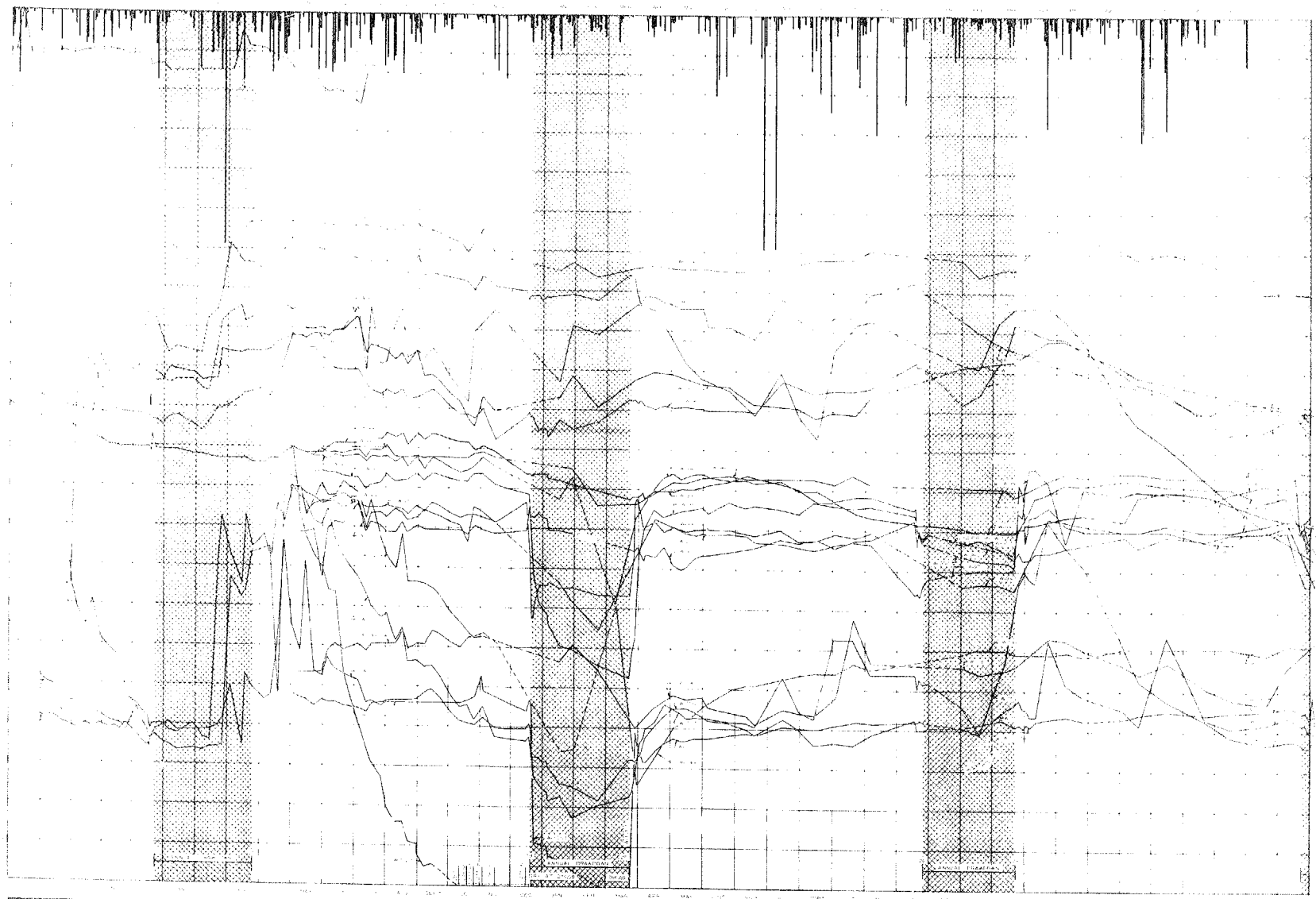
CARLTON STREET TUNNEL

SOILS INFORMATION

PIEZOMETER OBSERVATIONS

DWG. NO. 4011-R-13

REV.



REFERENCES

DATE: 1963-10-10
 BY: J. H. B. / J. H. B.
 FOR: J. H. B. / J. H. B.

ISSUED FOR PERIOD OF: 1960-1963
 APPROVED: [Signature]
 SCALE: 1 inch = 10 feet

DEPARTMENT OF HIGHWAYS
 ONTARIO

CARTON STREET TUNNEL

SOILS INFORMATION
 PIEZOMETER OBSERVATIONS

DWG. NO. 4011-R-13 REV.

ADDENDUM #1

October 2, 1964

SUPPLEMENTARY DESCRIPTIONS

1. INTRODUCTION

At a meeting held on September 18, 1964, between representatives of the Department of Highways Ontario, and General Engineering Company Limited, the preliminary report was reviewed chapter by chapter. The recommendations and conclusions of the report were accepted by the Department of Highways Ontario subject to certain more detailed information being submitted by GEICO. The purpose of this addendum is to supply the requested additional information as follows:

Description of Schemes

A more detailed description of Schemes I, II, III, IV and V and the reasons for their acceptance or rejection is given in Section 2 of this addendum which forms an addendum to Chapter 4 of the report.

Piercing Main Dykes

A more detailed description of the two methods of piercing the dykes is given in Section 3 of this addendum which section is an addendum to Chapter 6 of the report.

Cost Estimates

Itemized cost estimates for Schemes I, II, and III are given in Section 4 together with an estimate of the annual operating and maintenance costs for mechanical and electrical services.

2. DESCRIPTION OF SCHEMES

This section is an addendum supplying further information to supplement that given in Chapter 4 of the main report.

2.1 Scheme I - Float in Elements

Prior to placing of the elements, the excavation of the trench in the canal bed will be completed and the bottom prepared to receive the elements. The design of the side slopes will be dependent upon the results of the soil

investigation and the excavation during the winter of 1964-'65.

The concrete elements will be precast during the summer of 1966 in a drydock located on low-lying ground behind the main dyke on the east side of the canal south of the tunnel centre-line. At the end of the navigation season, temporary watertight bulkheads will be placed at both ends enabling the elements to float with a small freeboard. The drydock will then be flooded, thus floating the elements, and the dyke at the northern end of the drydock will be broken. The elements will then be winched out into the canal and anchored in position vertically above their final location. The canal will be drained for the winter and the trench pumped dry with the elements gradually coming to rest on the prepared bed.

The joints between the tunnel elements will then be cast in the dry and the complete tunnel backfilled. Armour against the ships' anchors will be placed above the tunnel.

The construction of the tunnel proper will be concluded by completing the replacement of the dykes, as described in Chapter 6.

The main advantages of Scheme I are:

- a) It is the most economical scheme.
- b) The construction work to be carried out within the canal during the winter is reduced to a minimum and the possibility of unforeseen difficulties disrupting the schedule is thereby minimized.

The main disadvantage of Scheme I is the necessity of leaving a large excavation open for considerable time in the firm clay. This problem is overcome by Scheme II but it may also be overcome by the investigation planned in conjunction with the full scale test excavation during the winter of 1964-'65.

2.2 Scheme II - Sheet Pile Walls

The preliminary scheme has been based upon prestressed concrete sheet piling of dimensions 4'-0" x 3'-6". In detailed design different sizes and materials would be considered. The piles are cast in a yard adjacent to the site and are driven down into the dense stratified granular material in the winter non-navigation season from the bottom of an excavation made to the elevation of the underside of the roof slab. The tops of the piles are then trimmed and a reinforced concrete roof slab cast in place using the soil as the bottom form.

Accesses are excavated to both ends of the pile structure and excavation proceeds inside the tunnel with the floor slab and underdrains being placed in short sections. Excavation beneath the canal would be done

only during the winter non-navigation season.

The main advantage of Scheme II compared with Scheme I is that excavation of the firm grey clay is carried out in a protected area.

The main disadvantages of Scheme II compared with Scheme I are:

- 1) About \$0.5 million more costly than Scheme I.
- 2) At least two winter seasons are required for driving the piles and casting the roof slab, resulting in a tighter schedule than for Scheme I.
- 3) The use of unusually large sheet piling with the possibility of unforeseen problems.
- 4) Artesian water, if encountered, would be almost prohibitively expensive to overcome whereas with Scheme I it could be overcome at moderate cost.

2.3 Scheme III - Pre-cast Sections

The concrete portals are built in a yard adjacent to the site and a concrete slab is cast at the bottom of a trench excavated in open cut. The excavation of the trench and the piercing of the dykes will be carried out in the same way as outlined for Scheme I. The pre-cast concrete units are assembled in position on the slab during the winter non-navigation season by cranes and are then tied together. The tunnel is then surrounded by a clay seal and armour above the tunnel is included with the back-fill. Construction is then completed in a similar method as for Scheme I.

This Scheme has no advantages over Scheme I.

The main disadvantages compared with Scheme I are:

- 1) More costly than Scheme I by about \$1.0 million because the cost of fabricating and placing small pre-cast elements by cranes is higher than the cost of fabricating and placing large pre-cast elements by floating, including the cost of a drydock.
- 2) Difficulty of ensuring watertightness.
- 3) A tighter schedule.
- 4) Artesian water, if encountered, would be almost prohibitively expensive to overcome whereas with Scheme I it could be overcome at moderate cost.

2.4 Scheme IV - Cut and Cover

This Scheme is illustrated on drawing 4011-R-7. The form of the tunnel

is the same as for Scheme I. The concrete is cast in place in the trench instead of in a drydock and floated into position.

This scheme has no advantages compared with Scheme I.

The main disadvantages compared with Scheme I are:

- 1) The extended length of the construction period by at least eighteen months and the uncertainties of completion within even such a lengthened schedule.
- 2) The cost is estimated as \$0.3 million greater than Scheme I due to the extension of the construction period and the higher cost of placing concrete during the winter.
- 3) Artesian water, if encountered, would be almost prohibitively expensive to overcome whereas with Scheme I it could be overcome at moderate cost.

2.5 Scheme V - Shield Driven

The tunnel proper would be made up of two tubes with concrete shells placed inside cast-iron linings made up of successive segmented rings bolted to one another. The shield in the form of a metal casing would be advanced by jacking with the tunnel lining being placed behind it. Due to the nature of soil, provision must be made in the estimate for construction of about 50% of the length under compressed air conditions.

The advantage of this scheme is that the dykes do not have to be removed.

The main disadvantages of this scheme are:

- 1) Considerably more costly than Scheme I. A comparable cost estimate totals \$14.5 millions.
- 2) The small amount of cover above the shield or the considerably longer tunnel to increase the cover.
- 3) Varying soil conditions from hard till to firm clay which would be encountered within the height of the shield.
- 4) Necessary divergence of tubes will result in a less desirable road alignment, and more costly approaches.

3. PIERCING MAIN DYKES

3.1 Floated-in-Section

The floated-in-section method requires that a large area on the landward

side of the dyke be enclosed with an earth cofferdam, built in summer, whose ends join the existing dyke. The location of the cofferdam is determined by the allowable side slopes of the trench excavation and the cofferdam. After the cofferdam is in position, a section of the main dyke is removed and the trench excavated to required grades. The tunnel elements are then floated into position and the trench is backfilled.

To enable the easterly dyke to be rebuilt during the summer, a cofferdam may be built during the winter of 1966-'67 within the limits of the canal but outside the navigation channel. This cofferdam, together with the mostly easterly cofferdam, fully enclose the section of the dyke to be reconstructed. Although this additional cofferdam is built in winter, full protection would be provided to the canal by the landward cofferdam which would be built during the summer.

For the westerly dyke however, the main dyke must be rebuilt in the winter because no temporary structure is allowed to encroach upon the navigation channel during the winter. In order to overcome problems of winter construction, it has been assumed for preliminary pricing purposes that the dyke would take the form of two small dykes of gravel containing a central core of clay. A temporary housing has been considered, to ensure that the clay is placed under good conditions and that ice lenses and snow are excluded from the area. Should this method be adopted, it is expected that a less expensive procedure would result from design development.

3.2 Built-in-Place

The built-in-place method requires that the dyke for some distance on both sides of the tunnel be partially contained and protected within steel sheet piling structures. During the winter non-navigation season, the area of the tunnel between the structures, and extending either side of the dyke, is excavated to the elevation of the underside of the roof in stages. Bearing piles are then driven down into the dense stratified granular material and the concrete roof is cast in place. The ventilation building, designed to act as a soil and water retaining structure, is built on the roof and tied into the existing dykes prior to the opening of the navigation season.

Access is then obtained from the landward side and excavation of the tunnel and the placing of the concrete walls and floor are carried out by stages.

In order that the excavation for the trench can be carried out without endangering the dykes, four steel sheet pile cells are constructed in strategic locations as stabilizing structures, with the clay being removed and replaced by gravel while the cells are full of water.

The advantages of the floated-in-section method are:

- 1) The entire closed tunnel rests on a similar foundation.
- 2) The replaced section of the dyke forms a homogeneous structure with the existing dyke.
- 3) More flexibility of construction methods are possible should artesian water become a problem.

The advantages of the built-in-place method are:

- 1) It avoids irregular loading upon the floated-in section which is a foundation on elastic support.
- 2) A minimum amount of the dyke is removed.

4. COST ESTIMATES

The following are summaries of the preliminary estimates of cost for the three schemes which have been studied in the greatest detail. These estimates have been prepared as accurately as practical in the preliminary design stage. Nevertheless, some of the quantities and units may be substantially in error. It is believed, however, that the breakdown given in Chapter 10 of the main report may be relied upon as giving an adequate budget figure at this stage. It is also considered that the cost differences shown to exist between schemes are quite accurate enough for selection of the general scheme.

4.1 Scheme I - Float in Elements

1) Excavate Trench (Winter '64)	114,000CY @ 1.25	142,500
2) Construct East Cofferdam	86,700CY @ 1.25	108,300
3) Supply & Construct Wing Cofferdam incl. 1865 T of SS piles		570,000
4) Excavate Step by Step W. Cofferdam	20,000CY @ 3.50	70,000
5) Supply & Drive Bearing Piles (1500T)		450,000
6) Concrete in West End incl. Vent. Bldg. & Dam	8,200CY @ 85.00	696,000
7) Excavate Under Slab	18,000CY @ 2.50	45,000
8) Remove & Replace Tie-Up Wall	5,600SF @ 11.00	61,600
9) Supply & Drive SSP Cells	400 T	122,400
10) Excavate cells & Place Gravel	4,750CY @ 6.00	28,500
11) Excavate Trench & East Dyke	126,000CY @ 0.90	113,400
12) Construct Drydock incl. Cofferdam		90,500
13) Construct Tunnel Elements in Drydock	19,000CY @ 75.00	1,425,000
14) Construct Tunnel in place	1,500CY @ 85.00	127,500
15) Construct East Vent. Bldg		40,000
16) Prepare Bed & Sump	61,500SF @ 1.25	77,000
17) Inspect Bed, Break Dyke, Install Sections, Drain Canal & Trench		60,000
18) Backfill Tunnel	150,000CY @ 0.90	135,000
19) Construct Cofferdam over Tunnel East Side	147,000CY @ 1.25	183,750
20) Reconstruct East Dyke	65,000CY @ 1.25	81,250
21) Remove Cofferdams East Side	285,000CY @ 0.90	256,500
22) Excavate Approaches	95,200CY @ 0.90	85,700

23) Concrete Approaches	12,500CY @ 80.00	1,000,000
24) Backfill Approaches	34,000CY @ 0.90	30,600
25) Roadway & Sidewalk	16,000SY @ 4.00	64,000
26) Mechanical & Electrical		1,000,000
27) Contingency		1,000,000
TOTAL - SCHEME I		<u>\$ 8,065,500</u>

4.2 Scheme II - Sheet Pile Walls

1) Construct East Cofferdam	50,300CY @ 1.25	62,900
2) Supply & Construct Wing Cofferdams incl. 1865T of S.S. Piles		570,000
3) Excavate Step by Step W. Cofferdam	20,000CY @ 3.50	70,000
4) Supply & Drive Bearing Piles 1500T		450,000
5) Concrete in W. End incl. Vent Bldg & Dam	8,200CY @ 85.00	697,000
6) Excavate under Slab	18,000CY @ 2.50	45,000
7) Remove & Replace Tie Up Wall	56,000SF @ 11.00	61,600
8) Excavate Trench & East Dyke	83,000CY @ 0.90	74,700
9) Supply & Drive Concrete Sheet Piles	741 piles @ 24.00	1,778,400
10) Concrete in Roof Slab	10,000CY @ 95.00	950,000
11) Excavate under Slab	34,000CY @ 3.00	102,000
12) Filter under Slab	57,000 SF @ 0.20	11,400
13) Concrete Floor Slab	2,100CY @ 90.00	189,000
14) Construct East Vent. Bldg.		40,000
15) Drain Canal & Trench		20,000
16) Backfill Tunnel	32,700CY @ 0.90	29,430
17) Construct Cofferdam over Tunnel East Side	107,000CY @ 1.25	133,800
18) Reconstruct East Dyke	40,000CY @ 1.25	50,000
19) Remove Cofferdams East Side	157,300CY @ 0.90	141,600
20) Excavate Approaches	95,200CY @ 0.90	85,700
21) Concrete Approaches	12,500CY @ 80.00	1,000,000
22) Backfill Approaches	34,000CY @ 0.90	30,600
23) Roadways & Sidewalk	16,000 SY @ 4.00	64,000

24) Mechanical & Electrical	1,000,000
25) Contingency	<u>1,000,000</u>
TOTAL - SCHEME II	<u>\$ 8,657,130</u>

4.3 Scheme III - Pre-cast Sections

1) Excavate Trench (Winter '64)	114,000CY @ 1.25	142,500
2) Construct East Cofferdam	86,700CY @ 1.25	108,300
3) Supply & Construct Wing Cofferdams incl. 1865T of SS piles		570,000
4) Excavate Step by Step W. Cofferdam	20,000CY @ 3.50	70,000
5) Supply & Drive Bearing Piles (1500T)		450,000
6) Concrete in West End incl. Vent. Bldg. & Dam	8,200CY @ 85.00	697,000
7) Excavate under Slab	18,000CY @ 2.50	45,000
8) Remove & Replace Tie Up Wall	5,600CY @ 11.00	61,600
9) Supply & Drive S.S. Piles	400 T	122,400
10) Excavate Cells & Place Gravel	4,750CY @ 6.00	28,500
11) Excavate Trench & East Dyke	126,000CY @ 0.90	113,500
12) Fabricate Pre-cast Sections	14,400CY @ 120.00	1,728,000
13) Concrete in Place on Bed	8,400CY @ 80.00	672,000
14) Place Pre-cast Sections	850No. @ 600.00	510,000
15) Drain Canal & Trench		20,000
16) Construct East Vent. Bldg.		40,000
17) Backfill Tunnel	150,000CY @ 0.90	135,000
18) Construct Cofferdam over Tunnel East Side	147,000CY @ 1.25	183,750
19) Reconstruct East Dyke	65,000CY @ 1.25	81,250
20) Remove Cofferdams East Side	233,700CY @ 0.90	210,430
21) Excavate Approaches	95,200CY @ 0.90	85,600
22) Concrete Approaches	12,500CY @ 80.00	1,000,000
23) Backfill Approaches	34,000CY @ 0.90	30,600

24) Roadway & Sidewalk	16,000SY @ 4.00	64,000
25) Mechanical & Electrical		1,000,000
26) Contingency		<u>1,000,000</u>
TOTAL - SCHEME III		<u><u>\$ 9,169,410</u></u>

4.4 Operating and Maintenance Costs

The following preliminary estimate of operating and maintenance costs for the tunnel is based on the assumption that three tunnels will be maintained from a control point which will be manned continuously, requiring 5 operators, and which will also co-ordinate the efforts of a 6-man cleaning and maintenance crew.

Estimated annual costs for Carlton Street Tunnel:

Labour	\$ 19,000
Power	13,000
Materials	<u>8,000</u>
TOTAL	<u><u>\$ 40,000</u></u>

MEMORANDUM

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

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

Attention: Mr. F.I. Hewson
Consultant Liason Engineer.

DATE: October 9, 1964.

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

Proposed Carlton St. Tunnel under
the Welland Canal at St. Catherines,
Ontario.

W.P. 444-64

W.J. 64-F-53

Enclosed please find our detailed report on the subsoil conditions existing at the above mentioned proposed tunnel location.

It should be noted that slight revisions have been made to the preliminary borelog sheets #1-#3 which were given to Mr. N.D. Lea of General Engineering Co. Ltd. on August 26th.

We believe that the factual information and recommendations contained in the report will be adequate for immediate design purposes, however certain facets of the investigation dealing with ground water conditions and effective shear strength of the cohesive layers are still under study in the field and in the laboratory and will be fully reported at a later date.

Should any additional information be required or should any points in the report require clarification, please feel free to call our office.

AGS/PB
Attach.

cc: Messrs. A.M. Toye (2)
H.A. Tregaskes
H.D. McMillan

A. G. Stermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

G.K. Hunter (2)
H. Greenland Foundations Office
T.J. Kovich Gen. Files
W. Melinyshyn: General Eng. Co. Ltd. (2)

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FOUNDATION INVESTIGATION REPORT

FOR

Proposed Carlton St. Tunnel under
the Welland Canal at St. Catharines,
Ontario.

W.P. 444-64

W.J. 64-F-53

1. INTRODUCTION:

A preliminary foundation investigation, at the site of the proposed Carlton St. Tunnel, consisting of three sampled boreholes was requested by N. D. Lea, General Engineering Co. Ltd. in a letter to A. Putka, Materials and Research Engineer, dated June 24 1964. Following this request an investigation was commenced by this Section on July 6 and was completed by August 26 at which time the resulting information was made available to Mr. Lea in the form of detailed borelog sheets.

During the preliminary work it became apparent that a more detailed programme was necessary and due to the urgency of the tunnel project it was decided to complete the preliminary and detailed programmes in one operation. A memo from F. I. Hewson, Consultant Liason Engineer dated August 6, requested that the Foundation Section proceed accordingly.

The final form of the detailed soils programme was decided upon at a meeting between the Foundation Section and N. D. Lea held on August 25, 1964. Subsequent meetings were held during the course of the fieldwork to review the work progress.

This report contains the results of both field and laboratory tests carried out during the course of the investigation together with a detailed description of subsoil conditions existing at the site.

2. DESCRIPTION OF SITE:

The site is located in the west part of St. Catherines some 1100' south of the existing bascule type bridge which carries Carlton St. over the Welland Canal. The proposed Q_L of the tunnel intersects the Q_L of Lock #2 at Sta. 2N+16-Seaway Drainage. At this location the Welland Canal is some 700' wide and about 35' deep, the mean water level during the navigation season being el. 335.5. During boring operations for B.H.s #3 and #7 it was observed that the water level in the canal fluctuated approximately ± 1.5 feet about the mean level in periods of less than 30 minutes. The canal is contained by earth dykes which are approximately 20' higher than the surrounding terrain the latter being generally flat with an average ground surface elevation of approximately 320.0.

A geological report for this project has been prepared by Professor Deane of Toronto University and should be referred to if geological information is required.

3. FIELD INVESTIGATION PROCEDURE:

The fieldwork consisted of thirteen groups of boreholes, each group consisting of from one to six separate borings the grand total being forty seven. The pattern of the various groups

3. FIELD INVESTIGATION PROCEDURE: (cont'd)...

was arranged so that the minimum distance between adjacent holes was about four feet. At each of Boreholes #1 - #4, three separate holes were drilled, one being for the purpose of recovering continuous samples in the clay layers and the other two being for the purpose of carrying out field vane tests in the clay layers at intervals in elevation of nine inches. Boreholes #5, #7, #11 and #13 consisted of a single hole only in which samples were recovered at various intervals and field vane tests were carried out where applicable at elevations 12" below the various sample depths. Boreholes #6, #8, #9, #10 and #12 each consisted of a group of holes the deepest of which was used for sampling and vane testing the remainder of each group being drilled for piezometer installation only.

Boreholes #1 and #2 were drilled partly by means of a continuous flight auger 4-inch diameter and partly by means of conventional diamond drilling equipment adapted for soil sampling purposes. All other boreholes were drilled using the diamond drill only. In all instances where the diamond drill was used the borehole was advanced by means of wash-boring methods using NX casing down to the bottom of the clay layers and BX casing beyond this depth. Boreholes #3 and #7 were carried out in the canal in which case the drilling machine was mounted on a raft.

Undisturbed samples in the cohesive layers were obtained by means of 2-inch I.D. shelly tubes fitted with a piston sampler which were pushed into the soil either hydraulically

cont'd /4...

3. FIELD INVESTIGATION PROCEDURE: (cont'd) ...

by the drilling machine or by hand. 'Disturbed' samples in all layers were recovered by means of a standard 2-inch O.D. split spoon sampler driven into the soil with a 140 lb. hammer falling freely a distance of 30 inches. AX+ rock core samples were obtained in B.H.s #2, #5 and #1.

Field vane tests were carried out in the cohesive layers using a standard D.H.O. field vane having dimensions such that the shear strength equals twenty times the applied torque at failure. The vane tip was advanced eighteen inches into the undisturbed soil at which point the vane rods were clamped to a vertical bearing mounted on the top of the casing. This permits free rotational movement in the horizontal plane but prevents downward movement of the vane during the test. The torque was then applied and the value at failure recorded. The vane was then rotated six complete turns and a remoulded test was carried out in the same fashion. In order to determine the effect of rod friction in the vane tests, a number of tests were carried out using the same apparatus with the vane blades removed. Immediately after a friction test, a conventional vane test was carried out in order that a comparison might be made.

A number of piezometers were installed at five different locations numbered B.H.s 6P, 8P, 9P, 10P and 12P. At each location a Peaker Type piezometer connected to two 1/2" by 3/8" plastic tubes was installed at the bottom of the hole which was utilised for sampling purposes, and a bentonite

3. FIELD INVESTIGATION PROCEDURE: (cont'd) ...

seal was made some ten feet higher than the piezometer. The remainder of the hole was filled with sand. At each of B.H.s 6P and 10P four Geonor brass piezometers connected to single 3/8" by 1/4" plastic tubes were installed at various elevations between the sampling hole bottom and the ground surface. At each of B.H.s 8P, 9P and 12P five Geonor piezometers were installed in similar fashion. All Geonor piezometers were placed in separate borings which were drilled to within ten feet of the intended piezometer elevation, attached to 'A' rods or 'E' rods then driven the remaining ten feet by a hammer. The elevations of the various piezometers are shown on the pertinent log sheets contained in the Appendix of this report.

The locations and elevations of all boreholes were surveyed in the field by D.H.O. personnel from Engineering Surveys Section. Elevations are referred to Geodetic Bench Mark #563F located on Bridge #3. The Geodetic elevation on this landmark is 340.214 but the St. Lawrence Seaway Authority use a value 340.97, which has also been used during this project.

For the sake of simplicity all groups of boreholes are referred to as single boreholes numbered 1-13 incl. the centre of each group being given as the borehole location. The information from every hole in each group is therefore plotted on the single borelog sheet corresponding to a particular group. The borelog sheets are contained in the Appendix at this report.

cont'd /6...

4. SOIL TYPES AND SOIL CONDITIONS:

4.1) GENERAL:

Subsoil at the site, apart from the fill material in the canal dykes, consists of about five different deposits or types of deposit overlying shale bedrock. Generally speaking, conditions are fairly uniform over the whole site as regards soil type though somewhat variable as regards the depths of the various strata. The boundaries of the different deposits as determined in the boreholes are shown on the accompanying borelog sheets and the estimated stratigraphical profile contained in Drawing #64-F-53A is based on this information. From ground level downward the various soil types are as follows:

4.2) FILL MATERIAL:

This material was observed in B.H.s #1, #8P, #9P and 12P which were drilled through the artificial dykes constructed at the edges of the Welland Canal. The soil consists of brown coloured silty clay with traces of sand and fine gravel and is generally well compacted with a consistency ranging randomly from stiff to very stiff. On the east side of the canal the depth was observed to be 22.5 feet whilst on the west side the observed depth was 29 feet. Based on the results of field and laboratory tests the physical properties of the material are summarised as follows:-

Bulk Density	122 P.C.F.
Natural Moisture Content	...	14%-27%
Liquid Limit	31%-53%

cont'd /7...

4.2) FILL MATERIAL: (cont'd)...

Plastic Limit 16%-24%

Unconfined shear strength ... 1500-3900 P.S.F.

4.3) CLAYEY SILT WITH SAND AND OCCASIONAL GRAVEL:

(Upper Glacial Till)

This material extends from original ground level at M borehole locations for depths ranging from 25 to 39 feet and consists of a heterogeneous mixture of clayey silt, sand and gravel in the following average proportions:- clay 27%, silt 52%, sand 18%, gravel 3%. A wide range of 'N' values, 12 - 120 blows per foot, indicates some variation in consistency which is estimated to range from hard in the upper portion of the deposit to stiff in the lower portions. The texture of the material clearly shows the deposit to be of glacial origin with the predominant constituent being a clayey silt. A plot of plasticity index versus liquid limit shows the majority of the points confined to the CL portion of the chart. Natural moisture contents are generally close to the plastic limit. Generally speaking the lower ten feet of the deposit is somewhat softer than the rest and it was possible to obtain some shelly tube samples in this zone. Unconfined compression tests carried out on these samples gave undrained shear strength values ranging from about 800 to 1700 p.s.f. The worst location appears to be at B.H. #3 where the average was about 1000 p.s.f. Field vane tests on the other hand, were almost always greater than 2000 p.s.f. in this zone. No strength tests were carried out on material from the upper portion of the

cont'd /8...

4.3) CLAYEY SILT WITH SAND AND OCCASIONAL GRAVEL: (cont'd)...
(Upper Glacial Till)

deposit because of the hardness of the layer. Consolidation tests were carried out on samples obtained from B.H. #2 and #4 and indicated an overconsolidation pressure at about 1.5 tons in excess of the present overburden pressure. The effective overburden pressures quoted in this report have been computed from existing original ground level for convenience. It is believed that the overconsolidation pressure of the upper 20 feet of this layer is quite high. No attempt was made to estimate this during the present investigation.

Typical stress strain curves from unconfined compression tests, typical grain size distribution curves and a plot of plasticity index versus liquid limit are contained in the Appendix of this report on Figures 5, 3 and 1 respectively.

Physical properties of the material in the deposit are summarised as follows:-

Bulk Density	130-140 p.c.f.
Natural Moisture Content	12%-23%
Liquid Limit	17%-37%
Plastic Limit	11%-23%
Unconfined shear strength (bottom 10') ...		800-1700 p.s.f.

4.4) SILTY CLAY: (BROWN)

This deposit underlies the clayey silt stratum and extends for depths ranging from 3½ feet in B.H. #6P to 15 feet in B.H. #9P. It was observed in all boreholes and could be identified particularly by its brown to grey-brown colour and its

cont'd /9...

4.4) SILTY CLAY (BROWN): (cont'd)...

generally high plasticity. The material contains numerous tiny pockets of red and grey silt and clay. These pockets were observed to be in a much dryer state than the parent material and were uniformly dispersed throughout the whole layer. Some horizontal layering in the form of faint red bands and colour boundaries was observed indicating evidence of stratification. A plot of plasticity index versus liquid limit on Figure 1 shows the points to be concentrated about the CI-CH boundary on the chart. The latter is contained in the Appendix of the report. The liquidity index averages about 0.5 for the whole deposit. A number of unconfined and unconsolidated-undrained triaxial tests were carried out on samples from this layer and gave values of undrained shear strength ranging from about 400 to 1400, p.s.f. These values were in general lower than the field vane tests carried out at the corresponding sample elevations. Results of these tests are discussed in greater detail in the section headed 'Discussion'. A number of typical stress strain curves obtained from the compression tests are shown on Fig. 6. The overall consistency is estimated to range from firm to stiff.

Consolidation tests which were carried out on samples recovered from B.H. #1 indicate the stratum to be overconsolidated by about 3000 p.s.f. in excess of the existing overburden pressure. These tests are plotted on Fig. 4 in the Appendix of the report.

Physical properties of the material in the deposit as determined from laboratory and field tests are summarised as follows:-

cont'd /10...

4.4) SILTY CLAY (BROWN): (cont'd)...

Bulk Density	106-122 p.c.f.
Natural Moisture Content	28%-52%
Liquid Limit	40%-55%
Plastic Limit	21%-27%
Unconfined Shear Strength	400-1400 p.s.f.
Undrained Triaxial Shear Strength	600-1100 p.s.f.
Field Vane Shear Strength	500-1800 p.s.f.

Typical grain size distribution curves are shown in Fig. 4 of the Appendix.

4.5) SILTY CLAY WITH LAYERS OF CLAYEY SILT AND SILT:

(GREY)

This deposit underlies the brown silty clay stratum and extends for depths ranging from 7 feet in B.H. #3 to 14 feet in B.H. #1. It was observed in all boreholes and consisted of grey silty clay containing layers of varying thickness up to about 3 inches of red and grey clayey silt and grey silt. The layers were sensibly parallel and in the approximate horizontal plane. A plot of plasticity index versus liquid limit of the material from the cohesive layers shows a wide spread of the points along the 'A' line between the CI-CH and ML-CL boundaries. This plot is shown on Fig. 1 of the Appendix. The liquidity index ranges from about 0.5 to 1.0 the higher values generally representing the material of lower plasticity.

A number of unconfined and unconsolidated undrained triaxial tests were carried out on samples from this layer which showed

cont'd /11...

4.5) SILTY CLAY WITH LAYERS OF CLAYEY SILT AND SILT
(GREY): (cont'd) ...

an extremely wide scatter of results for the undrained shear strength. The range was approximately 300-1100 p.s.f. for both types of test the average value being 700 p.s.f. for each type of test. Typical stress strain curves for these tests are shown in Fig. 7 of the Appendix. Field vane tests were in general much higher than the laboratory tests. These results are discussed further in the section headed 'Discussion'. The overall consistency of the deposit is estimated to range from firm to stiff.

Consolidation tests carried out on samples from B.H. #2 indicated an overconsolidation pressure of about 1.5 t.s.f. throughout the stratum in excess of the existing overburden pressure. Results of these tests are plotted on Fig. 11 contained in the Appendix.

Physical properties of the material in the deposit as determined from laboratory and field tests are summarised as follows:-

Bulk Density	113-124 p.c.f.
Natural Moisture Content	20%-43%
Liquid Limit	22%-45%
Plastic Limit	14%-28
Unconfined Shear Strength	300-1100 p.s.f.
Undrained Triaxial Shear Strength	400-1000 p.s.f.
Field Vane Shear Strength	650-1800 p.s.f.

Typical grain size distribution curves are shown on Fig. 4 of the Appendix.

4.6) CLAYEY SILT, SILT AND SAND (LAYERED):

This material underlies the grey silty clay at all borehole locations and extends for depths ranging from 10 feet in B.H. #5 to 30 feet in B.H. #11. The overall deposit consists of a number of layers of varying thickness of material ranging in grain size from gravel and sand to clayey silt, with the predominant constituent being silt. Results of mechanical analysis are summarised in the accompanying borehole sheet. Definite boundaries between layers which were observed during sampling operations are shown on the borelog sheets contained in the Appendix. The 'N' values obtained from Standard Penetration tests ranged from 22 to more than 100 blows per foot. The relative density of the stratum is estimated to range from dense at the surface to very dense at the lower boundary. The permeability is estimated to range from high in the granular coarse grained layers to relatively low in the fine grained cohesive layers.

Physical properties of the material in the deposit as determined from field and laboratory tests are summarised as follows:

Bulk Density	111-130 p.c.f.
Natural Moisture Content	7%-46%
Liquid Limit (Cohesive layers)..		18%-55%
Plastic Limit (Cohesive layers).		12%-28%
Relative Density ('N' Values) ..		22-100 blows /ft.

During boring operations it was observed that at a number of locations natural gas started to emerge from the boreholes immediately this layer was intersected. The quantity appears to fluctuate from time to time with no definite observable

cont'd /13...

4.6) CLAYEY SILT, SILT AND SAND (LAYERED): (cont'd)...

pattern. It is believed that the gas may be present both in solution of the pore water and as free bodies in the pore spaces. Further information on this phenomenon will be reported when the results of our study on piezometer water levels are completed. The boreholes at which gas was definitely observed are as follows:- B.M. #2, 4, 6, 8, 9, 10, 11, and 12.

4.7) CLAYEY SILT SAND AND GRAVEL (LOWER GLACIAL TILL):

This deposit is of glacial origin and consists of a red to brown coloured mixture of clayey silt, sand and occasional gravel. The material is in a very dense state. 'N' values being in the order of 60 > 100 blows per foot. The observed depth of the deposit ranged from 12 feet in B.H. #5 to 21 feet in B.H. #2. Physical properties are summarised as follows:-

Natural Moisture Content	8%-21%
Plastic Limit	14%-17%
Liquid Limit	18%-23%

Results of mechanical analyses are summarized in the accompanying borelog sheets.

4.8) BEDROCK - RED SHALE:

Bedrock was observed at depths of 10¹/₂ feet 95 feet and 90 feet in B.H.s 2.5, and 11 respectively and consisted of a reddish coloured shale. The upper contact of the sound bedrock was somewhat difficult to determine due to the presence of weathered material which resembled the overlying till material very closely.

cont'd /14...

4.9) GROUNDWATER CONDITIONS:

A detailed investigation of piezometric water levels is at present underway in the field. At the present time the information available is incomplete and inconclusive. An additional report will be prepared covering this aspect of the subsoil conditions when our field investigation is completed.

5. DISCUSSION:

5.1) GENERAL:

It is proposed to construct a tunnel to replace the existing bascule type bridge at the intersection of Carlton St. and the Welland Canal at St. Catharines, Ontario. The tunnel will be constructed by the 'open cut' method and since the overall design is greatly affected by the geometry of the temporary slopes during construction and the permanent slopes for the tunnel approaches, the shear strength of the cohesive layers is of paramount importance. This aspect, together with the compressibility of the cohesive layers which must be taken into account in estimating the pressures exerted on the tunnel sections is discussed in some detail below:-

5.2) SHEAR STRENGTH OF THE CLAY LAYERS:

The stability of slopes immediately following construction is governed by the undrained shear strength and in the present case the most critical material is contained within the layers referred to as the brown silty clay and the grey silty clay. A wide range of scatter of undrained shear strength values has been obtained even for the same type of

5.2) SHEAR STRENGTH OF THE CLAY LAYERS: (cont'd)...

strength test and a fairly wide divergence occurs between field and laboratory tests. At this point it will be convenient to refer to the summary of the results of all undrained shear strength tests carried out in the two strata referred to as the brown clay and the grey clay.

Table I shows the average undrained shear strength computed numerically from a total of 145 field vane tests, 100 unconfined compression tests and 26 triaxial compression tests for each of B.H.s 1-12 together with the minimum values. The grand averages for the whole site are also given. In the table the brown clay and the grey clay layers are considered separately. In every case the field vanes averages are the higher. It can be observed however, that much closer agreement exists between the field vane averages and the compression test averages for the brown clay than for the grey clay. Since the grey clay consists of a number of stratified layers of silty clay, clayey silt and silt it is believed that the effects of disturbance caused by sampling in the field and by handling thereafter would be much greater than in the case of the brown clay which is a much more homogeneous material. With regard to the field vane tests it is believed that the presence of the silt layers in the grey clay deposit would tend to increase the measured value of the shear strength. In addition, a number of vane rod friction tests carried out during the course of the field work indicated that between the shear strength range 1000 - 1500 p.s.f. rod friction accounted for about five to 10 percent of the measured torque

cont'd /17...

B.H.	FIELD VANE AVERAGE MINIMUM		UNCONFINED AVERAGE MINIMUM		TRIAxIAL AVERAGE MINIMUM		FIELD VANE AVERAGE MINIMUM		UNCONFINED AVERAGE MINIMUM		TRIAxIAL AVERAGE MINIMUM	
1	1100	480	906	765	610	610	1147	640	722	535	834	546
2	1054	720	856	729	972	842	1108	760	491	287	600	525
3	1509	1120	775	575			1422	1040	834	635		
4	1091	960	1083	961	865	804	1065	640	765	655		
5	1280	1280	972	844			1413	1280	685	525	413	413
6			764	764			880	720	877	813		
7	1240	1120	1175	959			1108	960	874	741	732	690
8	1040	800	830	694			1013	880	678	455	742	564
9	970	720	849	663			1350	960	601	601		
10	1520	1520	387	387			980	800	534	370		
11	1040	960	1114	1010			1040	960	536	536	713	713
12	1360	1200	801	490			1350	1000	871	836	620	582
1-12	*1155/65 tests		*915/40 tests		*896/9 tests		*1160/80 tests		*707/60 tests		*701/17 tests	
1-4	1152/43 tests		887/22 tests		896/9 tests		1154/57 tests		687/38 tests		733/7 tests	

* Averages for all boreholes.

TABLE I - UNDRAINED SHEAR STRENGTH P.S.F.

5.2) SHEAR STRENGTH OF THE CLAY LAYERS: (cont'd)...

at failure. When all of the foregoing is taken into consideration it can be deduced that, whereas in the case of the brown clay the compression test results are probably fairly representative of the actual undrained shear strength, in the case of the grey clay they are somewhat lower than the true value.

For design purposes it is recommended that a value of undrained shear strength for the brown clay of 900 p.s.f. should be used.

For the grey clay, it is known that the laboratory strengths are too low due to greater sample distance. However, the anisotropic nature of the clay has not been determined. Previous experience has shown that for a layered material, the normal testing method employed, with the direction of the major principal stress at Failure perpendicular to the bedding planes of the soil, yields the maximum strength with respect to different orientation of the major principal stress. These two factors have a compensating effect and it is therefore suggested that the undrained strength used in design should be 700 p.s.f. which is the average of the laboratory tests. The actual strength can only be determined when the test section is carried out.

In order to examine the long term stability of approach cuts it is necessary to determine the effective shear strength parameters of the till and clay strata. It is also necessary to estimate the pore water conditions both during and after excavation of the cuts. In order to determine the parameters

cont'd /18...

5.2) SHEAR STRENGTH OF THE CLAY LAYERS: (cont'd)...

of the cohesive strata a series of undrained triaxial compression tests with pore pressure measurements and drained tests were carried out. The results of these tests are plotted on Figs. 8-10 of the Appendix of the report. Two multi-stage undrained tests with pore pressure measurements carried out on samples from the till stratum gave values of ϕ' of 26° and 31° with the values of effective cohesion C' being 110 p.s.f. and zero respectively. Drained tests carried out on samples from the brown clay stratum gave a value for ϕ' of 18° and a cohesion intercept of 300 p.s.f. Further tests are now underway and the results will be presented as an addendum to this report.

5.3) REMOULDED STRENGTH OF THE CLAY LAYERS:

A number of tests were carried out to try to ascertain the remoulded shear strength of the clay layers and the effect of time on this factor.

In the field, remoulded vane tests were carried out immediately after every natural vane test and in a few instances, additional tests at varying times after the natural test were also carried out. The results of the field tests indicated an immediate average remoulded value of about 25% of the undisturbed value and showed large regains in strength within a few hours.

In the laboratory unconfined compression tests were carried out on a number of samples to determine both the undisturbed undrained shear strength and the remoulded shear strength of varying times up to 8 days after remoulding.

5.3) REMOULDED STRENGTH OF THE CLAY LAYERS: (cont'd)..

The tests indicated that little or no increase in strength occurs beyond the value of the immediate remoulded strength and that this value is about 15% of the undisturbed value.

It is believed therefore, that the regain of the field vane test can be largely attributed to consolidation effects.

5.4) COMPRESSIBILITY OF THE COHESIVE LAYERS:

To study the compressibility and swelling characteristics of the cohesive deposits and thus to determine the possible movements of the tunnel units during construction and subsequent performance a number of consolidation tests were carried out. These are summarised in the Appendix of this report. The tests show that in general the clay layers and the lower 10 feet of the upper till layers are overconsolidated by about 1.5 t.s.f. in excess of overburden pressure except at the dyke locations where the overburden pressure has been artificially increased by about 2000 p.s.f. due to the weight of the fill.

From the general shape of the pressure-void ratio curves it is apparant that the samples indicate some degree of disturbance. It is therefore not possible to compute exact values for the compression index C_c . For computation purposes it is considered best to use the actual plot of void ratio versus the logarithm of pressure as shown on the figures.

To simulate the possible movements due to swelling and rebound the consolidation tests were subjected to various load cycles. In general, the cycles were commenced from overburden

5.4) COMPRESSIBILITY OF THE COHESIVE LAYERS: (cont'd)..

pressure as computed from the borehole results. The pressure was then reduced to 0.25 t.s.f. and the sample reloaded to a pressure in excess of overburden pressure. The resulting rebound curve was generally quite uniform in shape both for the lower section of the upper till layers and the clay layers for which the average values of the recompression indices C_{cr} are estimated to be 0.01 and 0.06 respectively. These values can be used to estimate movement due to swelling and rebound as the values of the swelling indices C_s and the values for C_{cr} will be essentially similar.

6. SUMMARY:

The results of a foundation investigation to determine the subsoil conditions existing at the site of the proposed Carlton St. Tunnel in St. Catharines, Ontario, are reported. It was found that the site is underlain by deposits of hard to stiff glacial till, firm to stiff clays, stratified layers of clayey silt, silt and sand, and a further layer of glacial till overlying shale bedrock. Depth to bedrock ranged from 90 to 104 feet.

The most critical material is contained within the clay layers and a detailed programme of field and laboratory testing has been carried out to determine values of undrained shear strength to be used for design purposes. This and other aspects are discussed in some detail in the main body of the report.

A study of piezometric water levels in the various strata is at present underway in the field. The results will be reported as soon as the study is complete.

cont'd /21...

7. MISCELLANEOUS:

The boring programme was commenced on July and is still underway at the time of writing this report. Equipment being used on the site is owned and operated by Dominion Soil Investigations Ltd. The fieldwork was supervised directly by Project Fdn. Engineers Mr. P. Payer and Mr. P. McGlone. The preparation of this report together with the general supervision of the fieldwork was carried out by Mr. K. Selby, Senior Fdn. Engineer. The report was reviewed by Mr. K. Y. Lo, Supervising Foundation Engineer.

October 9, 1964.

FILE

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

83-64-351
R. Draycott
Planning Section

To: A. Butka,
Materials & Testing Engineer.

From: Functional Planning Section,
Planning and Design Branch,
Central Region, Downsview.

Date: July 8th, 1964.

Our File Ref.

In Reply To

Subject: Proposed Vehicular Tunnel & Approaches - W.P. 444-64 & -5,
under Welland Ship Canal at Carlton Street,
City of St. Catharines & Twp. of Niagara,
District #4, Hamilton.
.....

Attached I am forwarding for your information and appropriate
action one.... (copy) (~~copies~~) of the Functional Planning Report for the
above project.

Prints of plans and profiles will be issued by Engineering Surveys
at a later date to those Sections that require them.

R. Draycott
R. Draycott

RD/ww
Attach.

for: R. G. Burnfield,
Reg. Functional Planning Engr.

[Signature]