

Mr. John Walter,  
Design Engineer.

Attention: Mr. H. McMillan

F. C. Brownridge,  
Materials & Research Engineer.

March 27th, 1956

Re: Q.E.W. North End Hwy. 2  
Interchange to North End Burlington  
Bridge - Project No. 41-56.

The Soils Design Report for this project is attached herewith and included in the same cover you will receive Soils Profiles QEW A-12, QEW A-24 and Plan QEW A-14.

As you will see we are not, at this time, making any final recommendations for this job. We feel that, in view of the great difficulties involved, the importance of the final result as regards B.H.O. prestige, and the high costs involved, further study and discussion should take place before the final decision can be made. This report attempts to present the full problem and the various solutions which might be employed in its solution.

We would particularly comment upon the immediate confliction of interests involved - this is studied in detail in the report, but may be summarized as "haste versus prestige". The speed of construction which is required is the very thing which is liable to cause failures both during and after construction. The latter clearly cannot be tolerated and we would, in consequence, reiterate our strong recommendation that this contract include all such measures as may be necessary to (a) provide the design-solution of the problem and (b) ensure that adequate control is provided over the contractor's operations, as regards scheduling of the work and control of materials and methods.

DHS/MdeF  
Encl.

F. C. Brownridge,  
Materials & Research Engr.

c.c. Messrs. H.A. Tregaskes  
F.C. Brownridge  
D.H. Scholes  
File (4)

per

D. H. Scholes  
Soils Engineer

SOILS DESIGN REPORT

Queen Elizabeth Way

North End Hwy. 2 Interchange to North End Burlington Bridge: 0.75 miles

Project No. 41-56

Profile No.: F 2310-29Soils Nos.: QEW A-12  
QEW A-24Plan Nos.: F 2310-28 (50' : 1")  
F 2310-17 (400' : 1")Soils Nos.: QEW A-14  
QEW A-13GENERAL DATA:

The Design Criteria for this job as approved, on January 5th, 1956, by Mr. W. J. Fulton call for two 24 feet wide pavements separated by a 10 feet wide boulevard and with 10 feet wide shoulders and 2 feet rounding. Standard fill-slopes are proposed. Minimum vertical vision is to be 1000 feet, maximum grades are to be 3% and maximum curvature is to be 2°.

The contract is to be one of Grading, Culverts, Granular Base and Hot-mix Paving.

It is understood from Mr. D. Farran, the Project Design Engineer dealing with this job, that traffic is to be maintained during the construction of the Indian Point Overpass, W.P. 73-55 (at the intersection of the Q.E.W. and Highway 2, i.e. near the north end of the project under consideration herein) in 1956 by means of diversions to the east and north of the structure. These will use parts of the new interchange whenever possible.

OTHER INVESTIGATIONS AND REPORTS:

1. The Foundation of Canada Engineering Corporation Limited submitted on January 18th, 1955, a very comprehensive report of their investigations and design proposals for the Burlington High Level Bridge. A considerable number of very deep borings were performed in the course of their investigations, the most northerly being shown at Sta. 30+50 at 50 ft. Right of Q. (The structure ends at Sta. 27+35 or thereabouts, i.e. approximately three hundred feet away).
2. A report on the Foundation Investigation for the overpass for the Q.E.W. over Highway 2 was prepared by this Section and was submitted on March 4th, 1955. In the normal course of the Soils Section investigation holes were drilled on the location of the proposed approaches and a "fault" in the bedrock level was discovered at about 7+80 on Q. 'B'. Associated with this 25 foot fault was about thirty feet of peat which underlaid seven or eight feet of sandy material. (It has since been ascertained that this latter material is imported fill). Recommendations were made in this report for either diversion of the line or a limitation upon the height of fill. If this latter course was followed, considerable settlement was envisaged and it was then suggested that a period of five or six years

#### OTHER INVESTIGATIONS AND REPORTS: (cont'd.)

should be allowed to elapse before the laying of the permanent pavement. (This, it is realized, is not now practicable.) The possibility of using sand drains to expedite the settlement was also suggested in this report.

The peat referred to was not revealed by the consultant's investigation, even though it has been shown, by the investigations now being reported upon, to extend right up to the North end of the High-Level Structure itself - the depth of peat at Sta. 27+00 at 50' left of centre-line is about 8 feet. At this point it contains very much less organic matter and water than does the material nearer to Highway 2.

#### THE PRESENT INVESTIGATION:

This investigation was commenced in December 1955, using the "Swamp Buggy" which was equipped with flight-augers capable of drilling down to a depth of forty-five feet. A core-drill had later to be assigned to assist in the work and was used for sampling purposes now where the depth of peat was beyond the limits of the auger.

Some twenty-eight auger holes and eight core-drill holes were bored on line 'B' and on Ramp 'A' of the Highway 2 interchange. The locations of these are shown on Soils Plan No. QEW A-14 and the depths of peat, etc. found are plotted on the Soils Profiles Nos.: QEW A-12 and A-24.

#### SUMMARY OF CONDITIONS:

##### (a) Main Lanes on Line 'B' (and part on Line 'C')

The height of fill is, it is understood, as low as is possible and is now generally about seven or eight feet above ground-level. At the approach to the Highway 2 overpass, however, the height of fill increases to a maximum of twenty feet and at the end of the High-Level Structure it increases to the same height.

The sandy-loam fill which was found over most of the area varies considerably in depth. The average depth is, however, about ten feet.

Beneath this sandy-loam lies the original, rather swampy ground. This is, of course, usually well below the water table and has compressed considerably in the course of time until it might more accurately be described now as a soft and wet peat.

It's moisture content is high and samples taken have shown a range of from 70% to 215%. At the bottom of this peat lies either medium sand-gravel or hard clay which appears to fall towards the south-west (i.e. towards the Bay). This fall is small between the interchange and the High-Level Structure but is quite large within the limits of the interchange - e.g. the fall is about 1 in 10 between bore holes Nos. B.S.I 5 and B.H. 2 (see Soils Plan QEW A-14 for locations - both in the vicinity of Sta. 5+50 on Ramp 'A').

SUMMARY OF CONDITIONS: (cont'd.)

(a) Main Lanes on Line 'B' (and part on Line 'C') - (cont'd.)

The depth of peat over the former length, Sta. 17+00 to the High-Level Structure (at Sta. 27+35) averages about 10 feet, with about 8 feet at Sta. 27+00 (50' Left Q. Line 'C'). On the centre-line there appears to be a "fault" at about Station 7+50. The line of this fault would seem Cross Line 'B' at an acute angle - at only about 20°-30°. Furthermore, a certain amount of peat was found over the top of the fault which forms a thin layer only about seven or eight feet thick at an elevation at or above the top of the general area of peat. The maximum thickness of this peaty material on this line 'B' appears to be about 25 feet at Sta. 8+00. At Sta. 11+00 it is 20 feet and Sta. 15+50 it is about 11 feet.

It is not possible to conduct any useful strength tests upon the peat, and owing to its extremely variable moisture content and its consequent variation in stability, any results which might be obtained would be of little value.

(b) Ramp 'A' of Interchange - See Profile Q&A A-24.

It appears that the fault occurs around Station 3+00. At Sta. 2+50 (Ramp 'A') there is only about 6½ feet of peat with clay-loam at 16 feet, whereas at about Sta. 6+25 the peat commences within ten feet of the surface and extends to a depth of 55 feet from the surface. (This increased depth compared with line 'B' depths indicates the slope of the bottom.) By Station 11+30 the depth of peat has reduced to 18 feet (being seven feet deeper than the line 'B' equivalent) and Ramp 'A' then hooks-on to Line 'B'.

(c) East Leg of Interchange (Leg 'B')

It appears that no peat is encountered on this leg, north of the existing Q.E.W. (i.e. Sta. 23+50). The fault will, however, be encountered, (with its associated peat) at or about Sta. 24+50 which corresponds to approximately Sta. 9+50 on Line 'B'.

MATERIALS:

A large amount of borrow will be required. This material will be either (a) earth which may have to be hauled from the area north-west of Freeman, i.e. a haul-distance of some three or four miles, or (b) dredged sand or hydraulic fill. Arrangements could probably be made to dock quantities of this material quite close to the job - or alternatively pumping, similar to that done for the High Level Bridge footings, might be possible for some of the fill. Should the borrow selected be earth, it would in all probability be the light-medium clay of acceptable to borderline nature which is generally found in the area concerned.

The granular materials available are very limited and, apart from sub-base material out of the lake or the bay, crushed rock from the quarries to the north and west would have to be used. The haul distance would be about 8 or 11 miles for the Mount Ness and Dundas quarries respectively. There is a limited quantity of material suitable for G.B.C. 'B' available in the Waterdown area - 7 miles haul distance.

#### DISCUSSION:

The foremost considerations which must be borne in mind when the various alternative solutions and their respective costs are being evaluated are:-

1. A first-class highway, comparable in riding quality to the High-Level Structure itself, must be provided.
2. This highway must be completed and opened to traffic by the fall of 1957 by which time the High-Level Structure is scheduled for completion.
3. Finally, and most important, every possible effort must be made to ensure that, once open, this highway will not need immediate maintenance work of any kind which would destroy its smooth riding quality.

Two dangers have to be faced. They are (a) sliding of the fill, during construction, due to shearing of the peaty material and (b) settlement of the fill, due to compression of the peat. Either of these failures would, of course, be sufficient to cause severe distortion and/or rupture of the pavement.

The only positive solution against these two difficulties is complete excavation of all the peat. However, because of the depths involved this will not be possible. The best that could be done would be as follows:-

1. Excavate as far down into the peat as possible. A drag-line cannot usually operate in swamps at a greater depth than about 10 feet but in this case, owing to the 10 feet of sandy overburden which overlies the peat and to the slight cohesion of the peat itself, it may be possible to excavate to a depth of 5 feet or even more. Then:-
2. "Toe-shoot" the sack forwards by means of dynamite using drag-lines to cast the frontal-wave of peat to the sides.  
(Alternately liquefaction of the peat by dynamiting might be used to assist the load-fill in displacing it.)

The following factors involved in these operations should be considered:-

- (i) The width of excavation will be about 120 feet over the deepest part of the swamp, reducing to about 95 feet in a southerly direction.
- (ii) Assuming that the drag-lines excavate 15 feet down, they will only remove about 5 feet of peat leaving on line 'B', 23 feet of peat at the north end and 7 feet nearer to High-Level Structure.
- (iii) Toe shooting is not usually recommended (in conjunction with excavation) for swamps greater than about 20 feet deep and hence is not likely to be too successful where the peat is found to a depth of 38 feet.
- (iv) Underfill Blasting (or "liquefaction") is not considered to be a reliable method of swamp treatment and would merely assist in the settlement of the peat rather than actually removing it. In this case it is unlikely even to help much in this because of the firmness of the peat.
- (v) In short, these "positive" methods would be of greatest value where the peat depth is least (i.e. Station 15/50 to Sta. 27/35 - 11 feet decreasing to 8 feet beneath, in all cases, about 10 feet of sandy-fill). Elsewhere, the problem is more acute, the expense of any excavation would be vastly greater, and the solution will be less effective.



DISCUSSION: (cont'd.)

Alternately, the peat (and overburden) could be left untouched and all the effort to achieve a satisfactory solution applied from the top. The following suggestions are made as precautions against the danger of sliding:-

1. Apply the fill load as slowly as possible (Later recommendations will show that this is not meant to involve slowing down the whole operation but rather to organize it to better effect).
2. Stabilize the construction by some means such as sand-drains (See Q.E.W. and Hwy. 2 Structure report).
3. Provide berms to counterbalance the sliding forces.
4. Repair any slides if, and when, they occur. We would propose a combination of items (1) (3) and (4) but item (2) - sand drains - might, however, be considered further, in which case thought should be given to the very careful design and supervision of the work which would be required. The field control would present perhaps the most serious problem, and help of experienced personnel would be absolutely essential. Sheared-off sand piles are of no use whatever!

The second danger, that of settlement, cannot possibly be entirely avoided. The depth of fill has been reduced as much as possible and hence the total amount of settlement which will take place has been similarly reduced.

What can be done, however, is to ensure that as much as possible of this settlement occurs in the shortest possible time, i.e. before paving is commenced. To do this, sand piles as referred to above, might be used to increase the rate of settlement but, in view of the disadvantages already quoted, they are not advocated in this particular instance. Instead, two methods might be adopted:-

1. Provide a surcharge on top of the fill which would cause a larger proportion of the total eventual settlement to occur during the time this surcharge is in place. "Jetting" of the fill and/or subgrade would make this even more effective by increasing the load upon the peat.

The danger of the former procedure is that in addition to increasing the rate of settlement we will also increase a likelihood of shear failure occurring. The only safeguard then possible would be to further reduce the rate of application of the fill-load.

It would not be necessary for the applied surcharge to be wasted. The cross-section constructed could be designed so that some of the surcharge material could later be placed in the shoulders of the final cross-section. Appropriate scheduling of fill operations could also be used to advantage.

Furthermore, the abovementioned dangers of sliding and settlement apply not only to the main lanes of the Q.E.W. but also to Ramp 'A' (where the problem is the most acute (even though the "prestige-factor" is less vital) and to a part of Leg 'D' of the interchange.)

In view of the large risks involved it is suggested that Ramp 'A' be constructed very slowly with no surcharge and that it be given some years to settle before it is finally paved. (Traffic could always be diverted temporarily, if necessary, via the west leg crossing (on temporary construction) onto the west loop and thence onto the Q.E.W. lane.)

2. Prevent any settlement within the fill itself by ensuring completely uniform and adequate compaction of all material placed. The simpler expedient of using

continued....

DISCUSSION - (cont'd.)

2. - cont'd...

sand fill would do much to achieve this. (Hydraulic fill would, of course, provide the perfect answer.) This being achieved, even quite large settlements of the fill as a whole would be somewhat distributed or "cushioned" at pavement level and their destructive effects consequently diminished.

RECOMMENDATIONS:

1. Q.E.W. Main Lanes:

It is strongly recommended that all fill material used on the main lanes (and on interchange legs wherever peat is present in the subgrade), and all borrow used, should be sand material from the Lake or Bay - hydraulic fill may be used if desired. It would appear from experience on the first "Skyway" Contract that this hydraulic-fill material may well be the cheapest available borrow.

Consideration might be given to the possibility of using either dynamite or sand drains.

If dynamite is used the following additional precautions must be taken (details given later):-

- (i) Surcharging
- (ii) Jetting of the fill and the surcharge

If sand drains are used the following items should be included (details later):-

- (i) Uniform fill placing
- (ii) Surcharging
- (iii) Jetting of the fill and surcharge
- (iv) Provision of controlling personnel experienced in this type of work.

If neither of the above suggestions are adopted then the following practices would be the minimum treatment possible:-

- (i) Uniform fill placing
- (ii) Provision of berms
- (iii) Surcharging
- (iv) Jetting of the fill and surcharge

The following are the details of each of the above items:-

- (a) Uniform Fill Placing: It is suggested that "Special Information to Bidders" could require that, using normal placing and compacting procedure, the fill be placed uniformly throughout the whole length between Stations 6+50 and 27+35. Using the word layer to mean 6" layer as normally placed and compacted, it might be stated that this whole length of fill be constructed in 2 feet high stages. Alternation of efforts between the interchange and the aforementioned length might also be required and the contractor could be advised accordingly.

RECOMMENDATIONS - (cont'd.)

1. Q.E.W. Main Lanes - (cont'd.)

- (b) A berm, (which will be part of the interchange construction), on both sides of the fill is required and should be constructed concurrently with the main fill. This should be eighty feet wide and seven feet high between Sta. 6+50 and 11+00 and between Sta. 11+00 to 16+00 its width could taper off to zero. (On the east side the berm need not commence until Sta. 8+00.) Should slightly higher berms be desirable their width may be correspondingly lessened.
- (c) A surcharge should be placed between the outside pavement edges to a width of about 60 feet and to a height of 10 feet from Sta. 6+50 to Sta. 26+00. (To facilitate detour construction this could be dispensed with on the northbound lane (left lane) only between Sta. 6+50 and Sta. 7+25.) The full height of 10 feet surcharge is required at the stations quoted and it could be constructed with 1:1 slopes. It should be left in place as long as possible and should only be removed when paving operations are imminent - this should, of course, be delayed until near-completion of the High-level Structure. Surcharge material could then be used to complete the construction of Ramp 'A' and to flatten to 3:1 slope all fill slopes as already constructed to 1½:1 or 2:1 as desired.

A depth of 9" G.B.C.'A' over sand fill (dredged or pumped) is recommended for the Q.E.W. itself.

Although almost all the fill will be of sand, earth will be encountered in the interchange construction. On such material, providing that it is of acceptable nature, 9" of G.B.C.'A' and 15 inches of sand fill would be required.

2. Ramp 'A' Interchange:

Similar alternatives to those given above for the main lanes will apply. If dynamite is not used the following procedure should be adopted:

- (i) That part which is in effect the berms recommended above should be built immediately. Because of the great depth of soft material beneath this ramp between Sta. 3+00 and Sta. 6+50 the minimum of fill construction (without surcharge) - i.e. not including the general "filling-in" material which is eventually to be placed between Ramp 'A' and the main lanes - would be constructed up to grade and left to commence the consolidation of the subgrade material.
- (ii) When the surcharge is removed from the main lanes this material should be used to complete the basic construction of Ramp 'A' and all necessary grading connected with it.
- (iii) The fill should then be left - and used with a gravel or mulch surface if so desired - for period of at least two years, i.e. until 1959 or preferably later, before the final permanent surfacing is laid.

Note: This item will, in view of the depth of peat found here, be required irrespective of the treatment employed on Ramp 'A'.



## RECOMMENDATIONS - (cont'd.)

### 3. East Leg of Interchange (Leg 'B'):

Similar alternatives again apply for this leg but whichever is finally decided upon we would recommend that a 10 feet surcharge be applied between Sta. 24/00 (Sta. 9/00 approximately on Line 'B') and its southern end on the northbound lane of the Q.E.W. This would be constructed as detailed above and would similarly be removed in 1957 or at such time as the paving has to be done for the adjacent Q.E.W.

### 4. General:

Irrespective of the method adopted for treatment of the peat it is strongly urged that the weight of all fill material placed in the areas where peat is known to exist should be increased by the addition of water. The process of "Jetting" or "Hydraulic Fill Consolidation" should be used to ensure that the whole fill and surcharge is completely saturated. This process could well be extended to include saturation of the sandy material which is already covering the peat. (All fill used in the areas to be jetted must, of course, be of sand type.)

It is observed that no method of treatment herein suggested will remove all the peat in the deepest places and that some settlement is consequently inevitable. Hence the recommendations for (a) Surcharging and (b) Jetting are made irrespective of the procedure followed.

Some failures may occur during construction due to sliding. These should be attended to, as and when they occur. An increase in the contingent quantities of excavation and fill material would be advantageous.

All culverts south of Sta. 6/50 on the main lanes (or Sta. 3/00 on Ramp 'A', or Sta. 24/00 on Leg 'B') should be of flexible type, and should be laid with the upstream half horizontal and whole fall on the downstream half.

March 17th, 1956

David H. Scholes

MONTREAL OFFICE  
1900 SHERBROOKE ST. WEST  
MONTREAL 25

FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

206 BLOOR STREET EAST  
TORONTO 5

CABLE ADDRESS  
"FOUNDANENG"  
TORONTO

January 18, 1955.

Ontario Department of Highways,  
Parliament Buildings,  
Toronto, Ontario.

Attention Mr. W.A. Clarke,  
Chief Engineer

BURLINGTON HIGH LEVEL BRIDGE

Dear Sir:

This letter accompanies the preliminary report on the above project.

You will notice that general views of the structure which we recommend, are given on drawings numbered 1126-B-1 and 1126-E-6, which are bound at the rear of the report.

In general, the structure which we recommend has been designed with considerable emphasis upon economy. Only in the center span have strict economy requirements been waived for a structure of somewhat more pleasing form. Some ornamentation has been given to the feature piers which mark the change from one type of construction to another. Special ornamental treatment of the entrances to the bridge is dealt with in a separate letter.

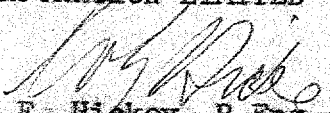
Throughout the preparation of our preliminary design we have kept in close contact with your representative, Mr. Arthur Sedgwick, and reviewed the alternatives on the various sections of the bridge with him. Your formal approval would be appreciated as quickly as possible however, so that we may continue with the final design.

**FENCO**

Ontario Department of Highways,  
January 18, 1955.  
Page 2

Our estimate for the cost of  
construction contracts is \$11,520,000.

Yours very truly,  
FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

  
W.E. Hickey, P.Eng.  
VICE-PRESIDENT

WEH/eh

PRELIMINARY REPORT  
TO THE  
ONTARIO DEPARTMENT OF HIGHWAYS  
REGARDING  
BURLINGTON HIGH LEVEL BRIDGE

DISTRIBUTION

4 copies Ontario Department of Highways  
1 copy Fenco, Montreal  
1 copy Dr. P.L. Pratley  
2 copies Fenco, Toronto

1126  
December, 1954

**FENCO**

## I N D E X

	<u>Page</u>
Introduction	1
Site	1
History	2
Fixed Requirements	3
Site Investigation	5
Fill Under Bridge	6
Foundations	6
General Arrangement	8
Central 3-Span Unit	9
Turn-Outs	10
Deck Truss Spans	10
Plate Girder Spans	10
Rolled Beam Spans	11
Rocker Bents	11
Piers	11
Entrance and Abutment	12
Deck Details	12
Lighting	14
Specifications	14
Estimate of Cost	15
Personnel	15
Appendix A - Geocon Report on Soil Exploration	
E - Loading Specification	
C - Settlement Studies	
D - Foundation Design Criteria	
E - Drawings	

## INTRODUCTION

By an agreement made on August 27, 1954, the Department of Highways of Ontario has retained Foundation of Canada Engineering Corporation Limited to act as its engineers for the design and construction supervision of the Burlington High Level Bridge.

One of the terms of the agreement is that Fenco is to present the Department with a preliminary report, presenting sketch plans and outline specifications on alternatives of merit. The document before you is the report called for.

Throughout preliminary studies Fenco has reviewed with the Department the various specific questions which have arisen, and thus we are able to present in this report a recommended design which we feel confident will meet with the approval of the Department.

Dr. P.L. Pratley, Consulting Engineer, is acting in association with Fenco as the consulting engineer on this project.

William R. Souter and Associates, Architects of Hamilton have been retained by Fenco as consulting architects.

## SITE

Hamilton Harbour is formed by the Hamilton Bar approximately four miles in length in a north-south direction. This bar, which is called Burlington Beach to the north of the channel and Hamilton Beach to the south of the channel, forms a natural breakwater several hundred feet wide. The bar is a typical bay-mouth bar composed of sands and gravels which have been deposited by a combination of long shore currents and the funneling effect of the shore lines upon wave action.

The proposed Burlington High Level Bridge is located on the beach to the harbour side of the bar, and parallel to the existing highway. The ground



SITE (continued)

traversed by the proposed structure is quite flat, varying in elevation between 240 feet and 250 feet above sea level. Normal water level is about el 246, thus most of the ground is normally submerged.

The soil conditions are described in detail in a report by Gecon Ltd dated November 18, 1954, and attached as appendix A.

Significant data on water levels is given in table 1. Since the fetch across the harbour is approximately five miles, considerable wave action can be expected, even on the harbour side of the bar.

TABLE 1

Regulated levels given are those proposed under Method No. 5 as published by the Special Projects Branch, Department of Transport, Canada.

	<u>Lake Level 1860-1952</u>		<u>Hamilton Harbour</u>	
	<u>Recorded</u>	<u>Regulated</u>	<u>Recorded</u>	<u>Regulated</u>
Maximum	249.3	249.1	249.5	249.3
Mean	246.0	246.4		246.4
Minimum	242.7	243.8	242.5	243.6
Minimum during navigation season	242.7	244.0		± 243.8

For the purposes of the Burlington High Level Bridge, regulation method No. 5 gives essentially the same results as the other methods of regulating the water level, which are being considered in connection with the St. Lawrence Seaway. At the date of writing, the final method of regulation has not been agreed upon.

HISTORY

The construction of a high level bridge across the Burlington channel has been discussed for a long time. The section of road along the Hamilton and Burlington beaches is the last remaining stretch of two

## HISTORY (continued)

lane highway in the Queen Elizabeth Way which extends from Toronto to Niagara Falls. This Burlington Beach link has become a serious traffic bottleneck for two reasons. First, because of the restricted speed through this four mile stretch of two lane highway in a built-up area without controlled access. Second, because of the interference with traffic by the operation of the bascule bridge when ships travel through the Burlington channel.

The first bascule highway bridge which spans the southern half of the present 300 ft channel was constructed in 1920 and is still in operation. The channel was widened in 1929-30 and a second and northern bascule span was constructed. This northern span was wrecked through collision by a ship in the spring of 1952. It was replaced by a temporary rolled beam structure supported by steel piles and blocking the northern half of the channel.

The presence of any obstructions, including piers, in the channel, is objectionable to shipping and thus the destruction of this bascule span has made it quite urgent, from the point of view of navigation, to provide a new bridge for carrying highway traffic.

The construction of the Burlington High Level Bridge will not completely solve the navigation problems because the pivot pier of the railway bridge will remain. Nevertheless, the new highway bridge is a step in the right direction, and it will make it possible to restore the north half of the channel to marine traffic.

## FIXED REQUIREMENTS

### Channel Clearances

A horizontal clearance of 300 ft parallel with and centered on the channel center line, and a vertical clearance of 120 ft above high water el 249 is required by the Department of Public Works

## FIXED REQUIREMENTS (continued)

### Channel Clearances (continued)

of Canada. Making due allowance for the floor system of the main span, and for the vertical curve, these clearance requirements have determined the roadway grades over the channel, shown on drawing 1126-E-1.

### Grade

The maximum grade has been fixed by the Ontario Department of Highways as 3.00 per cent.

### Terminating Elevation

A preliminary study has been made of the approach embankment. It showed that an embankment could be constructed within a 200 ft right-of-way to a maximum roadway grade of el 275. At least up to this elevation the construction of embankment would be considerably less expensive than that of bridge. In order to conform with possible land-scaping of the approaches, however, the Ontario Department of Highways has set the terminating elevation of the bridge deck at approximately el 265. This criterion has fixed the length of the bridge shown on drawing 1126-E-1.

### General Arrangement of Deck

The general arrangement of the deck has been fixed by the Ontario Department of Highways as that shown on their drawing D3320-2, namely; two 24 ft roadways, a 6 ft center strip partly consisting of an open grid, two refuge strips or walks, one on either side, each with a clear width of 3 ft.

### Appearance

The Burlington High Level Bridge will appear as a very prominent feature against the skyline as viewed from the City of Hamilton. Refer to drawing 1126-E-6. Its appearance is thus of great importance, and must be given major consideration, with due regard to economy.

FIXED REQUIREMENTS (continued)Live Loading

In view of the large investment in the structure and its long anticipated life, Fenco has recommended that the Burlington High Level Bridge be designed for the safe passage of trucks having a loaded weight of 40 tons. The need for such a heavy loading is shown by the fact that there are in use to-day in Canada, vehicles imposing twice the loading called for by the Canadian Standards Association 1952 Specification. These heavy vehicles are now restricted in their travel on highways. It seems probable, however, that in the future, public opinion may demand that the allowable vehicle weight on highways in such industrial areas be made more consistent with the weight of vehicles actually being manufactured. For this reason, the loading specification outlined on drawing 1126-R-1, and described in detail in appendix B is being adopted for the Burlington High Level Bridge.

SITE INVESTIGATION

A site investigation has been carried out by Geocon Ltd, an associate company of Fenco, which is so organized as to co-ordinate efficiently with Fenco designers and provide the information required for design.

Borings, penetration tests, pile driving and loading tests, plate bearing tests and wellpoint pumping tests were carried out in the field and appropriate tests on the properties of the soil were conducted in the laboratory. A detailed factual report by Geocon Ltd has been prepared in a sufficient number of copies that it can be distributed with the drawings and specifications when there is a call for tenders on the foundations.

Briefly the typical soil conditions are as follows:-

Ten feet of loose sand with some organic matter.

About thirty feet of compact to dense sand and gravel.

Eighty to one hundred feet of compact fine silty sand underlain by ten to twenty feet of the same material in a denser state.

SITE INVESTIGATION (continued)

Stiff clayey silt grading with depth into a stiff silty clay. This stratum is not found south of about station 90. North of station 90 the thickness of this stratum increases to at least one hundred and thirty feet.

FILL UNDER BRIDGE

Since the ground under the bridge is nowhere more than a few feet below water level, a study has been made of the possibility of filling the area under the bridge to an elevation slightly above the water level. This procedure is illustrated on drawing 1126-E-1. It has the following advantages:-

- (1) Construction is speeded up because this earthwork can be let early, as a separate contract.
- (2) Access to the site for construction and de-watering are facilitated. The water is generally too shallow for the use of marine plant. Thus, if the area were not filled before construction, it would be necessary for the contractor to build causeways or trestles for access. Sheet piling and bracing would be necessary for almost every pier. With the fill, however, access is very convenient and the ground water could be lowered before excavation, thus eliminating the necessity of sheet piling and bracing.
- (3) The over all cost of the bridge is decreased by about \$640,000.
- (4) Maintenance of the bridge is made considerably easier.

In view of the above considerations, Fenco has recommended to the Department on November 1st, 1954, that the area under the bridge be filled. It is anticipated that formal acceptance of this recommendation will be received shortly, and that tenders can be called early in 1955.

FOUNDATIONS

Piers must be supported below the upper loose sand stratum, either by means of piles or spread

## FOUNDATIONS (continued)

foundations. The compact to dense sand and gravel immediately beneath the loose sand is a good foundation material and adequate bearing capacity can be readily acquired in it. Whether this should be by means of excavation and spread footings or by short timber piles will be determined at each pier location on the basis of economy, considering the variations in thickness and properties of the upper two strata.

It is impractical to carry foundations through the deep compressible materials lying beneath the dense sand and gravel. Since these compressible materials are present, however, heavy load concentrations upon the dense sand and gravel must be avoided because of the possibility of causing excessive settlement.

Timber piles are ideally suited to this location because:-

- (1) They will be continuously submerged and therefore untreated piles can be accepted as permanent construction.
- (2) Only short piles are required to carry the load through the loose sand and spread it out by friction in the upper part of the compact to dense sand.

Any loads placed on the upper compact to dense sand and gravel will ultimately be transferred to the underlying looser and more compressible materials, and some settlement will result. A study has been made of the magnitude of these settlements under various possible loading arrangements. A summary of these studies is given in appendix C. It is concluded that even when the load is spread out as much as practicable in the compact to dense sand and gravel, settlement may still be of significant magnitude.

In order to satisfy the foundation requirements discussed above and to keep differential settlement within tolerable limits, a set of design criteria have been established. These are given in appendix D.

It is planned that each pier base will be so designed that it may be used either with or without piles.



## FOUNDATIONS (continued)

Typical foundation designs are shown on drawings 1126-T-102, 1126-T-103, 1126-T-104, and 1126-T-110.

The main piers flanking the channel require special treatment. These piers receive a much greater live and dead load than any other piers on the structure. It is planned that their foundation should consist of cellular type concrete caissons built within steel sheet piling and founded at a depth of about 37 ft below ground level upon either steel or timber piles. These piers are illustrated on drawing 1126-T-109. This type of construction has two advantages. One, because of the cellular type of construction, the net pressure upon the soil is reduced to such a value that settlement will be tolerable. Two, the caisson carries the foundation to such a depth that dredging for deepening or widening of the channel will have no effect upon the bridge foundations.

## GENERAL ARRANGEMENT

The recommended general arrangement of the bridge is shown on drawing 1126-E-1. Four different types of construction have been used to meet the changing conditions as the grade rises. The main 495 ft span with the two flanking spans is of steel cantilever truss construction with a six panel suspended span. Steel truss spans - eleven on each side of the canal - are used with spans of 250 ft, 200 ft, and 160 ft. Near the abutment, where the roadway is close to the ground, the short spans use 36 inch rolled steel beams. There are sixteen of these spans on each side of the canal. Nine steel plate girder spans on each side of the canal are used as a transition from truss to rolled beam spans.

Because of the anticipated settlements, continuity has been avoided in the longer truss spans. It is felt that any likely differential settlement between adjoining piers can be tolerated for continuous rolled beam and plate girder spans in the approaches. Provision is being made in the design for convenient jacking and shimming which may be required during the early life of the bridge.

## GENERAL ARRANGEMENT (continued)

Generally, piers carry alternately two fixed and two roller bearings. This arrangement uses only half the number of expansion joints which would be required for two different types of bearings on each pier. Typical roller and fixed bearings are illustrated on drawing 1126-T-108.

A few feature piers have been provided to break the monotony of the bridge, and to mark changes in type of construction. Simple lines have been given to the piers with little ornamentation and with only moderate emphasis upon the limestone pylons. This is in keeping with the modern trend of architecture, but the treatment is not so marked as to make the structure dated. Feature piers are illustrated on drawings 1126-E-2, and 1126-T-110.

The several parts of the bridge will now be considered in detail.

## CENTRAL 3-SPAN UNIT

A number of possible forms for the central 3-span unit have been studied. The various alternatives considered are shown on drawings 1126-T-100 and 1126-T-101. The arch-like form, drawing 1126-E-1, with the turn-outs on either end of the central 3-span unit, has been chosen because of its appearance from a distance, and because of the minimizing of obstruction of the view by truss members.

The length of the central span has been fixed by the requirement that the foundations for the main pier should be clear of the anchorage system of the channel walls. Because of the skew of the bridge to the channel, this has required a span of 495 ft.

Light weight concrete has been studied for the bridge in general and the central span in particular. Expanded slag concrete cannot be recommended because of the problems of placing and the inconsistencies of properties which are to be expected at the present time. A slab of haydite concrete is more expensive than an equivalent slab of standard concrete. This increase in the cost of the deck caused by haydite aggregate has been compared with the accompanying saving in steel.

### CENTRAL 3-SPAN UNIT (continued)

For all spans except the main one, the saving is not sufficient to justify the use of haydite aggregate. For the center span, preliminary studies suggest that there is an appreciable saving by the use of haydite aggregate.

### TURN-OUTS

Turn-outs or parking bays have been requested by the Department to serve as areas where a service truck, police car or a disabled vehicle could be temporarily parked. Locating the turn-outs at both ends of the central 3-span unit seems to give the best arrangement for the structure as a whole.

### DECK TRUSS SPANS

Studies have been made of the economic length for deck truss spans at two stations, 50 and 60. The arrangement of spans shown on drawing 1126-E-1 is based on the over all appearance of the structure with due consideration to the economic studies. It is found that from the point of view of appearance, and the possible usefulness of the area under the bridge, it is desirable to maintain the clearance somewhat greater than that dictated by economy alone.

The use of two, three or four trusses for the deck truss spans has been studied and four trusses have been found to give the greatest over all economy.

A typical arrangement of deck recommended for the truss spans is shown on drawing 1126-T-105.

### PLATE GIRDER SPANS

Economic studies show that rolled beams are economical almost to the point where trusses become the least expensive type of construction. Although it would be possible to omit the plate girder spans without materially affecting the cost, it is felt that some plate girder spans should be used. They provide a transition from rolled beams to trusses.

## PLATE GIRDER SPANS (continued)

To take advantage of the economy of continuity, the plate girders have been made continuous over two supports.

A typical section through the floor system of the plate girder spans is shown on drawing 1126-T-106.

## ROLLED BEAM SPANS

For the spans closest to the abutment, rolled 36" W.F. beams are recommended. Concrete, including prestressed concrete, has been considered for these spans but it has been found to have no economic advantage and it has been ruled out because of the settlement considerations described in the section entitled "Foundations".

The economic span for rolled beams was found to be greater than the 48 ft which has been used. Because of considerations of vibration, however, the depth to span ratio has been restricted to not more than 16. For the floor system recommended and shown on drawing 1126-T-106, the lightest 36" W.F. section is somewhat stronger than required, but the deflection and vibration figures justify its use.

The rolled beam spans are made continuous over three supports to take advantage of continuity.

## ROCKER BENTS

Steel rocker bents are shown for the intermediate supports under the continuous plate girder and rolled beam spans, except for piers 35, 36, and 37 which are in concrete because of their low height. Typical rocker bents are shown on drawing 1126-T-103.

The appearance of these slim rocker bents is felt to be desirable for the supports of the comparatively low approach spans.

## PIERS

For the support of the truss spans, concrete two-shaft piers have been chosen for two reasons:-

PIERS (continued)

- (1) They have been found to be less expensive than three or four shaft piers because of the way they distribute the load onto the base.
- (2) Their appearance is preferred by our architectural advisers because they tend to increase the towering height effect of the structure.

The two shaft motif has been carried throughout the bridge except for a very few low piers near the abutments.

While no batter is actually required for structural considerations, a slight batter of 1:48 is used so that the piers will not appear to be slimmer at the bottom than at the top.

The lines of the piers have been kept simple, with horizontal rustications to accommodate construction joints.

Typical piers are illustrated on drawings 1126-T-102, 1126-T-104, 1126-T-110 and 1126-E-3.

ENTRANCE AND ABUTMENT

Special landscaping and architectural treatment of the entrances to the bridge is recommended. Since this is not strictly a part of the bridge design contract however, it is considered in a separate letter.

The recommended abutment is shown on drawing 1126-T-111.

A certain amount of differential settlement is possible between the approach embankment and the abutment, therefore a short transition span with one end resting on the fill and the other end on the abutment is included as a means of minimizing local grade changes.

DECK DETAILS

Several types of curbs have been studied and discussed with the Department. There seems to be no

DECK DETAILS (continued)

adequate reason however for departing from the Department's standard which is shown on drawing 1126-T-107. This standard curb is 10 inches high and slopes 2 inches. A curb bar is used to protect the concrete.

This curb should be effective in stopping a motor car out of control, although it may not prevent a heavy truck from mounting the sidewalk. It is not practical to design a curb and fence to stop the heaviest vehicle which might travel on the bridge, if it were out of control at high speed. Openings in the curb have generally been avoided so as not to allow salt water to drip down onto the steel.

The sidewalks are carried throughout the length of the bridge by a light truss on their outer edge. This, it is felt, will give a uniform appearance to the whole structure. The sidewalk is not designed for a truck wheel load but for a uniform loading of 100 lbs per s f. If a heavy truck were to jump the curb it would certainly cause damage to the sidewalk and railing, but such damage could readily be repaired and at the perpetrator's cost.

An open metal grill is used in the bridge deck between the roadways. This grill is so designed that any vehicle likely to get on it will not actually break through. The maximum size of openings will be about two inches. This is small enough to allow safe use by workmen.

Provision for cross-over from one roadway to the other is required by the Department. It is suggested that these cross-overs might be located close to feature piers. It is recommended that at the cross-over, the height of the curb be decreased from 10 inches to 4 inches. The 4 inch curb will be a deterrent against unauthorized vehicles using the cross-over, but it will permit the slow passage of maintenance and other official vehicles. This low curb is preferred to movable ramps or curb sections. Movable pieces seem to be out of place on such a structure. They may be lost or stolen. They could get out into the traffic. They certainly would have a shorter life than the rest of the structure.



## DECK DETAILS (continued)

Removable sections of the center strip grating are provided at each pier for inspection access. Ladders are provided at the same locations. It is planned to equip the underside of the steelwork with tracks to carry moving scaffolding for painting and maintenance.

Gutter drains are provided just uphill of each expansion joint. The drain pipes are of comparatively large size to overcome the possibility of being plugged by ice. Refer to drawing 1126-T-107. The cast grills covering the drain pipes are readily removable. The downpipes are of sufficient length to carry the salt water from the roadway down past the steelwork. Auxiliary curb scuppers are provided just downhill of the floor drains. These scuppers are a safety measure visualizing the possibility of the gutter drains being plugged for a short time. The scupper drains are provided with a lip so that under normal conditions they will carry no water.

## LIGHTING

It is recommended that the lighting fixtures be G.E. Form 206, or equal, using four 100 w fluorescent lamps. These fixtures would be spaced 120 ft to 150 ft and the fixture for each lane would be bracketed out from a common tapered aluminum standard placed in the center strip. For the mere purpose of illumination, the fastening of the fixture directly to the standard would be satisfactory, but short bracket arms as shown on drawing 1126-T-107 are recommended to improve the appearance. Provision is made for aircraft warning lights and telephones.

## SPECIFICATIONS

The specifications for the superstructure contract will be the Canadian Standards Association S6-1952 with very few supplementary and special clauses.

A complete set of substructure and concrete specifications will be provided. The 1935 Ontario Department of Highways Specifications will be used as

SPECIFICATIONS (continued)

a guide in writing these specifications. Canadian Standards Association Specifications will be referred to wherever possible.

An outline specification is proposed for each drawing.

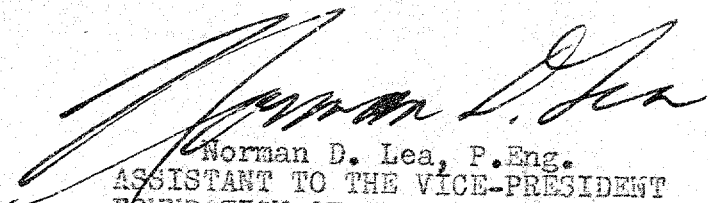
ESTIMATE OF COST

The cost of the structure recommended in this report is estimated as follows, exclusive only of property acquisition, interest and charges relative to Department services and personnel:-

Earthwork	\$ 380,000	
Foundations	3,430,000	
Steel Superstructure	5,960,000	
Deck	1,750,000	
Engineering	420,000	
Inspection & Testing	180,000	
	<u>12,120,000</u>	
Contingency - 10%	<u>1,212,000</u>	
TOTAL		<u>\$13,332,000</u>

PERSONNEL

This report has been written by N.D. Lea and reviewed by R.E. Chadwick and P.L. Pratley.

  
 Norman D. Lea, P.Eng.  
 ASSISTANT TO THE VICE-PRESIDENT  
 FOUNDATION OF CANADA ENGINEERING  
 CORPORATION LIMITED

APPENDIX A  
REPORT  
on  
SOIL CONDITIONS  
by  
GEOCON LTD



# GEOTECHNICAL SERVICES

DIVISION OF GEOCON LTD

402 OLD WESTON RD.  
TORONTO • ONTARIO  
Tel.: MURRAY 7581

November 18, 1954.

Foundation of Canada Engineering Corporation Limited,  
200 Bloor Street East,  
Toronto, Ontario.

Attention: Mr. N.D. Lea,  
Assistant to the Vice-President

Re: Site Investigation  
Proposed Burlington High Level Bridge,  
Burlington, Ontario.

Dear Sirs:

This letter accompanies our detailed factual report covering the above site investigation. The investigation which was carried out under your direction, included exploratory borings, dynamic and static penetration tests, pile driving and loading tests and plate bearing tests.

The borings and penetration tests show that the site is covered by a few feet of loose grey sand followed by up to 45 feet of dense sand. Below this is a thick bed of compact silty sand which becomes siltier with depth. North of station 89+50, the silty sand grades into a compact clayey silt which is underlain by a stiff grey clay stratum. The thickness of the clay stratum was not determined. South of station 89+50, the silty sand rests directly on the stiff clay which is of variable but limited thickness. Between stations 89+50 and 104+50, the clay is underlain by sound shale bedrock.

The results of the pile driving and loading tests and plate bearing tests are detailed in the report.

We believe our report gives all the information required for safe and economical design of the proposed bridge foundations. Should any additional information be required, we would be pleased if you would call us.

Yours very truly,  
GEOCON LTD

*James Morgan*  
JAMES MORGAN, P.Eng.

Manager - Geotechnical Services Division

O.S.6014

REPORT  
TO  
FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED  
ON  
SITE INVESTIGATION  
BURLINGTON HIGH LEVEL BRIDGE  
BURLINGTON, ONTARIO

Distribution:

- 30 copies - Foundation of Canada Engineering  
Corporation Limited  
Toronto, Ontario
- 2 copies - Geocon Ltd  
Toronto, Ontario

November 18th, 1954.

**GEOCON LTD**

## INDEX

	<u>Page</u>
Introduction	1
Procedure	1
Site and Geology	2
Soil Conditions	2
Water Conditions	8
Pile Driving and Loading Tests	8
Plate Bearing Tests	9
Personnel	10

Appendix I	Office Reports on Soil Exploration
Appendix II	Dutch Cone Penetrometer Results
Appendix III	Figures - Laboratory Testing
Appendix IV	Figures - Pile Driving and Loading Tests
Appendix V	Figures - Plate Bearing Tests
Appendix VI	Figures - Pumping Tests
Appendix VII	Photographs

Bound in rear of report -

OS-6014-1 Site Plan and Soil Stratigraphy



## INTRODUCTION

Geocon Ltd has been retained (proposals dated August 10th, 1954 and September 1st and 18th, 1954) to carry out a site investigation for the proposed Burlington High Level Bridge at Burlington, Ontario.

Foundation of Canada Engineering Corporation Limited has been retained by the Ontario Department of Highways as Designing Engineers for the bridge with Dr. P.L. Pratley as Consulting Engineer.

The purpose of the investigation was to obtain all the necessary information for safe and economical design of the proposed bridge foundations. This factual information is here reported in detail.

## PROCEDURE

A comprehensive site investigation program was carried out between August 17th, 1954 and October 21st, 1954. The investigation program included exploratory borings, dynamic and static penetration tests, pile driving and loading tests and plate bearing tests. To enable the plate bearing tests to be carried out at the required elevation, which was below the ground water table, a small wellpoint system was installed and pumping tests carried out during dewatering. Laboratory testing was carried out in the field laboratory set up on the site, and in the central laboratory of Geocon Ltd.

In all, twelve exploratory borings with dynamic penetration tests, eleven shallow wash borings and fourteen additional dynamic penetration tests were put down by standard machine drill-rigs, one of which was mounted on a barge as shown in Plate I of Appendix VII. The boreholes and other tests were carried out at specified locations which are shown together with the soil stratigraphy on Drawing OS-6014-1, bound in the rear of the report. Two static penetration tests using the Dutch Cone Penetrometer were also carried out and the results are presented graphically in Appendix II. A photograph showing the Penetrometer in use is given in Plate 2 of Appendix VII. The laboratory test results are plotted on the Office Reports in Appendix I and the Figures of Appendix III. Plate 3 of Appendix VII shows the field laboratory in operation.

**GEOCON LTD**

## PROCEDURE (continued)

The procedures for the Pile Driving and Loading tests and the Plate Bearing Tests are detailed under their respective headings later in this report.

## SITE AND GEOLOGY

The proposed new high level bridge is to be constructed roughly parallel to and about 350 feet to the west of the Queen Elizabeth Way and will extend for about 3500 feet each side of Burlington Canal.

The site is located on and to the west of the narrow strip of land comprising Burlington and Hamilton Beaches. The existing ground surface is up to 6 feet below the surface level of Burlington Bay except for a distance of about 1000 feet each side of the canal where the ground elevation rises up to 6 feet above the water level.

The site is part of a typical baymouth bar about 4 miles in length, formed probably in recent geological times by deposition of material eroded by wave action from the adjacent shores of Lake Ontario. From existing geological knowledge and from previous borings it is known that the site is covered by a considerable depth of sand.

## SOIL CONDITIONS

### Loose Grey Sand

A loose grey and dark grey sand stratum covers the site to a depth of up to 10 feet. It contains gravel of all sizes, organic matter in the form of sticks and twigs and numerous shell fragments. Where the stratum occurs above the water table it contains considerably less organic and calcareous material. The stratum has "N" values ranging between 2 and 14 with a general value of about 7. It is horizontally stratified both in gravel concentration and in colour, the latter probably due to variation in organic content. The stratification can be clearly seen in Plate 9 of Appendix VII. Some crossbedding

Loose Grey Sand (continued)

was observed during excavation.

Adjacent to the canal, the material has been replaced by about 5 feet of loose sandy fill in which wash borings encountered a variety of objects from small pieces of timber to gravel.

Compact and Dense Grey Sand

The loose grey sand is underlain by a stratum of compact and dense grey and dark grey sand for which typical grain size curves are given in Figures 1 to 3 of Appendix III. Its thickness varies between 20 feet and 45 feet but is generally about 30 feet. At the contact between the two strata there is a layer of very dark grey silty sand with a high organic content. The stratum tends to be horizontally stratified both in colour and composition, containing within the upper ten feet, layers or lenses of very sandy gravel and also shell fragments throughout. Gravel up to 3 inches in diameter was found dispersed throughout the stratum but the amount encountered and size generally decreased with depth. At the base of the stratum there are frequent thin layers of reddish brown fine and medium silty sand.

The stratum has "N" values ranging from 11 to 57 with a general value of about 32. A considerable variation in density with depth was indicated by the Standard Dynamic and Static Penetration Tests. This is however probably partly due to concentrations of gravel at various elevations. The Penetration Tests also indicated a general decrease in density with distance from the Burlington Canal. Average cone resistances of about 60 to 120 tons per square foot respectively, were obtained from two deep soundings with the Dutch Cone Penetrometer. The resistance diagram is plotted in Appendix II. The permeability of the stratum obtained from falling head tests carried out in boreholes varied between  $2.0 \times 10^{-3}$  and  $8.0 \times 10^{-3}$  cms. per second with a general value of  $4.5 \times 10^{-3}$  cms. per sec. A wellpoint system was installed to dewater an excavation required for plate bearing tests.

Compact and Dense Grey Sand (continued)

During dewatering of the excavation, readings of draw down taken in observation wells indicated permeabilities for this stratum of  $22.0 \times 10^{-3}$  to  $27.0 \times 10^{-3}$  cms. per sec. with a general value of  $25 \times 10^{-3}$  cms. per sec. A section through the wellpoint installation showing the cone of depression maintained by a discharge of 0.71 cusecs is given on Figure 1 of Appendix VI.

The following properties for the sand stratum have been determined from laboratory tests:

Wet Unit Weight	( $\gamma_w$ )	130 lbs./cu.ft.
Porosity	(n)	0.38
Relative Density	(Dr)	0.90
Angle of Internal Friction	( $\phi_s$ )	$43^\circ$
Sulphate Content	( $SO_3$ )	(a) Groundwater 32 to 167 ppm. (b) Soil 0.08% to 0.12%
Carbonate Content	( $CaCO_3$ )	(a) Groundwater 245 ppm. (b) Soil 13.1% to 29.6%
Free Carbon Dioxide	( $CO_2$ )	(a) Groundwater 10 to 17 ppm.

The results of the consolidated-drained triaxial tests from which the above angle of internal friction was obtained, are given in Figures 5 and 6 of Appendix III. Figure 4 of this Appendix gives the variation of relative density with depth.

Plate bearing tests which are described in detail later, were carried out in this sand stratum at elevation 239. Failure by breaking into the sand occurred, for the one foot square plate, at a loading of about 8 tons per square foot. The load-settlement, time-settlement and load-time curves for these tests are given in Figures 1 and 2 of Appendix V.

Compact Reddish-Brown Sand

The compact and dense grey sand changes through a depth of about five feet into a stratum of compact reddish-brown silty fine and medium sand becoming siltier with depth. The results of mechanical analyses are given in Figures 1 to 3 of Appendix III. The stratum which is between 75 and 110 feet thick is generally horizontally stratified throughout its depth, with the stratification appearing mainly as slight colour variations. Fine gravel sizes occur at random throughout the stratum. Below about elevation 180 there are numerous minute spherical cavities as shown in Plate 4 of Appendix VII with, in a few instances, cavities of 1/4 to 1/2 inches in diameter. Throughout the stratum, calcareous matter occurs in the form of fine shell fragments generally dispersed, but occasionally as thin concentrated layers. Organic matter in the form of vegetation and pieces of wood occurs at various elevations. At Borehole 97+00 - 50L, a 6 foot thick nest of logs was found at about elevation 162.

The stratum has indicated "N" values ranging between 7 and 50 with a general value of about 19. A variable penetration resistance was obtained from both dynamic and static tests. In general, the penetration resistance of the stratum at boreholes located in Burlington Bay, was about one half of the value at boreholes located on the land. Cone penetration resistances of about 40 and 60 tons per square foot respectively, were obtained from deep soundings with the Dutch Cone Penetrometer. Penetration Resistance diagrams for the latter are shown in Appendix II. A number of falling head field permeability tests carried out in the upper 10 feet of the stratum where the silt content is relatively low gave an average coefficient of permeability of about  $2.0 \times 10^{-3}$  cms. per second.

Wet unit weight determinations gave a general value of 122 pounds per cubic foot. The individual results and natural water contents are plotted on the Office Reports in Appendix I. A general relative density of 0.80 was obtained. A plot of relative density with depth is given in Figure 4 of Appendix III. Consolidated-drained

Compact Reddish-Brown Sand (continued)

triaxial test results are plotted in Figures 5 and 6 of Appendix III. They indicate a minimum angle of internal friction of  $40^{\circ}$ . The compressibility of the stratum increases with depth as shown by the pressure void-ratio curves given in Figure 8 of Appendix III.

Compact Reddish-Brown Silt

The change from the preceding stratum to this stratum is a very gradual one. For the purpose of engineering analysis it has been taken to occur at the arbitrary boundary given. The stratum is a compact reddish-brown sandy silt grading to clayey silt with depth. Grain size distribution curves are given in Figures 1 to 3 of Appendix III. It was encountered only to the north of Borehole #89+50 - 50R, and varied in thickness between 6 and 71 feet. It is horizontally stratified throughout. The stratification occurs as slight colour variations in the upper portion of the stratum and in the lower portion, as alternating layers of sandy silt, clayey silt, and very silty clay. These layers vary between about 1/8 inch and several inches in thickness. At Boreholes #42+00 - 50R and #57+00 - 50R, pockets of dense gravel about 2 feet in thickness were encountered. The spherical cavities encountered in the overlying stratum were also present in this stratum. Calcareous matter in the form of dispersed fine shell fragments and traces of organic matter were also encountered.

The stratum has indicated "N" values ranging between 7 and 50 with a general value of about 25. Wet unit weight determinations gave a general value of 125 pounds per cubic foot. The individual test results together with natural water contents are plotted on the Office Reports in Appendix I. No consolidation tests were carried out on samples from this stratum. However, by extrapolation from available data, the compression index is estimated to vary from 0.08 to 0.12 with depth.

Stiff Grey Clay

Underlying the silt is a stratum of grey silty clay which was only penetrated completely at boreholes between Stations 89+00 and 105+00, where it is between 16 and 35 feet in thickness. Over the remainder of the site, all the borings excepting the two adjacent to the south bank of the canal were stopped in the clay between elevations 105 and minus 18. In the upper portion of the stratum, the clay is varved and at various elevations contains thin layers of clayey and sandy silt with some fine fissuring at Borehole #82+00 - 50L. The clay in general becomes more homogeneous with depth. A typical sample is shown on Plate 5 of Appendix VII.

Undisturbed samples of the clay were obtained by using thin-walled open and sleeve samplers. Some of the samples however, showed a visible expansion within minutes of removal from the ground. From a limited number of laboratory tests the properties given below were determined for the clay samples. Because of expansion of the latter, the stratum in situ is probably stiffer and less compressible than shown.

Wet Unit Weight	( $\gamma_w$ )	130 lbs./cu.ft.
Natural Water Content	(w)	19 to 28%
Liquid Limit	(Lw)	26 to 36%
Plastic Limit	(Pw)	20 to 16%
Unconfined Compressive Strength	( $q_u$ )	1.0 to 2.0 tons/sq.ft.
Quick - Triaxial Compressive Strength	(q)	1.0 to 2.5 tons/sq.ft.
Sensitivity	( $S_t$ )	2
Compression Index	(Cc)	0.142

The individual test results are plotted on the Office Reports in Appendix I and Figures 5 to 9 in Appendix III.



Bedrock

The bedrock was proved by core-drilling in AXT size for at least 10 feet in the three boreholes between Stations 89+00 and 105+00 but was not encountered elsewhere on the site. Its surface slopes downwards towards the north from elevations 127 to 109.5 respectively. Good core recovery was obtained except at one hole where grinding occurred. Bedrock is a sound medium-hard horizontally laminated reddish-brown shale. At various elevations it contains grey bands and soft layers several inches in thickness. The upper 2 feet of the shale appeared to be weathered.

WATER CONDITIONS

The greater part of the site lies below the water level of Hamilton Harbour which was at about elevation 246.5 during the investigation. Between Stations 57+00 and 78+00 where the ground surface is above the water level, observations during the investigation showed that the groundwater table is the same as the harbour level.

PILE DRIVING AND LOADING TESTS

In all, eight specially selected 50 foot timber piles and one 80 foot steel H-pile were test driven and one was test-loaded. The test piles were driven in two locations at Stations 62+00 and 42+00. To enable work to be carried out in the dry at the latter location, a causeway was built using mill-run blast furnace slag. Plate 6 of Appendix VII shows the equipment in operation on this causeway.

The results of the test driving at Stations 62+00 and 42+00 are given with all pertinent data in Figures 1 and 2 respectively of Appendix IV.

The pile loading test was carried out on the first pile driven at Station 42+00. The load was applied as specified, through a hydraulic jack bearing against a dead load supported by timber grillage as shown on the arrangement drawing in Figure 3 of Appendix IV. The upper 7 feet of the

pile was unsupported. Deflections were measured to one thousandth of an inch by dial gauges. General and detailed views of the actual loading test are shown on Plates 7 and 8 of Appendix VII. The results are presented by time - load - settlement curves in Figure 4 of Appendix IV.

The failure of the pile occurred during the increment of loading from 70 to 80 tons. Visible tilting accompanied by audible cracking of the wood fibres indicated that the failure was probably structural. The pile regained its vertical position upon release of the load.

PLATE BEARING TESTS

Plate bearing tests were carried out as specified at elevation 239.3 in an excavation at Station 63+00. The excavation was first made down to the groundwater table and then completed after the area was dewatered by a wellpoint installation which is shown in Plate 9 of Appendix VII.

The method of applying the test loads and measuring the deflection was essentially the same as described under "Pile Driving and Loading Tests". Bearing plates, one and two square feet in area and 7 feet apart, were used. General and detailed views of the tests in progress are given in Plates 10 and 11 of Appendix VII. During testing, the groundwater table was maintained within 3 inches of the bearing plates. The latter were founded 6 inches below general excavation level in small pits. The results for the test on the one square foot plate, for which failure by breaking into the ground occurred at a loading of about 8 tons per square foot, are given as time - load - settlement curves in Figure 1 of Appendix V. Similar results for the test on the two square foot area plate are given in Figure 2 of Appendix V. No failure by breaking into the ground occurred during the test which was discontinued after the total deflection had exceeded  $3/4$  inches.

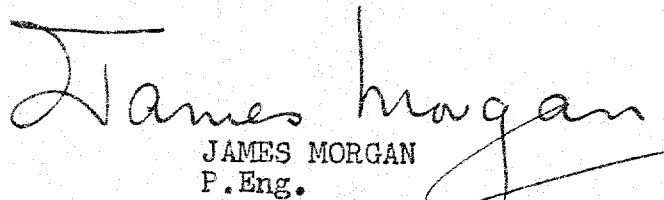
During dewatering of the test area to enable the excavation to be made, the permeability of the compact and dense grey sand stratum was checked by a pumping test. Figure 1 of Appendix VI gives the details and results of the latter. A coefficient

PLATE BEARING TESTS (continued)

of permeability (k) of  $25 \times 10^{-3}$  cms. per second was computed from this test.

PERSONNEL

This work was carried out under the direction of Mr. N.D. Lea, Project Engineer for Foundation of Canada Engineering Corporation Limited. The field and office work was supervised by Mr. M.A.J. Matich assisted by Messrs. B.I. Maduke and J.C. Osler, and at various stages by Messrs. G.W. Congdon and H. Sorenson. The report was written by Mr. M.A.J. Matich and reviewed by Mr. J. Morgan.

  
JAMES MORGAN  
P.Eng.

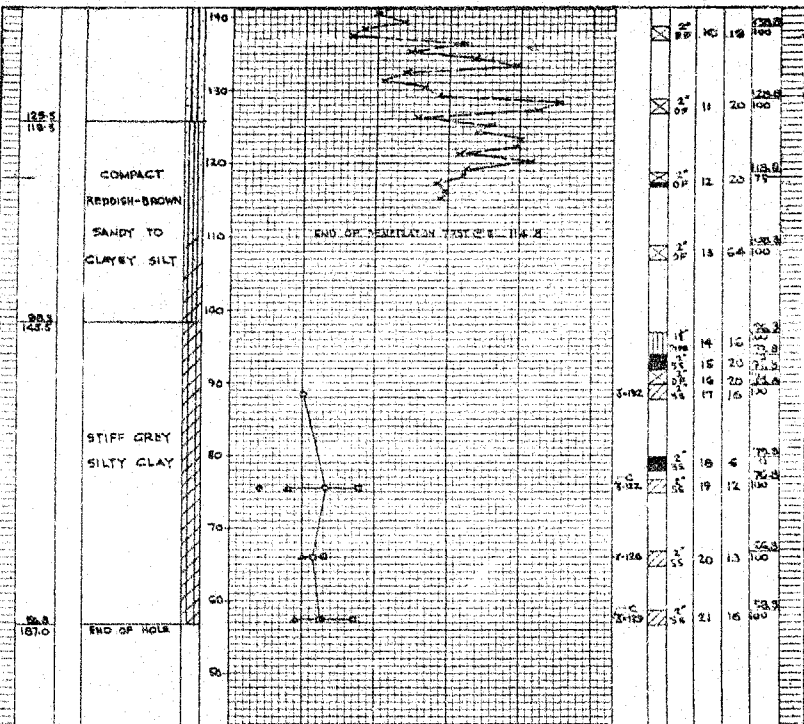
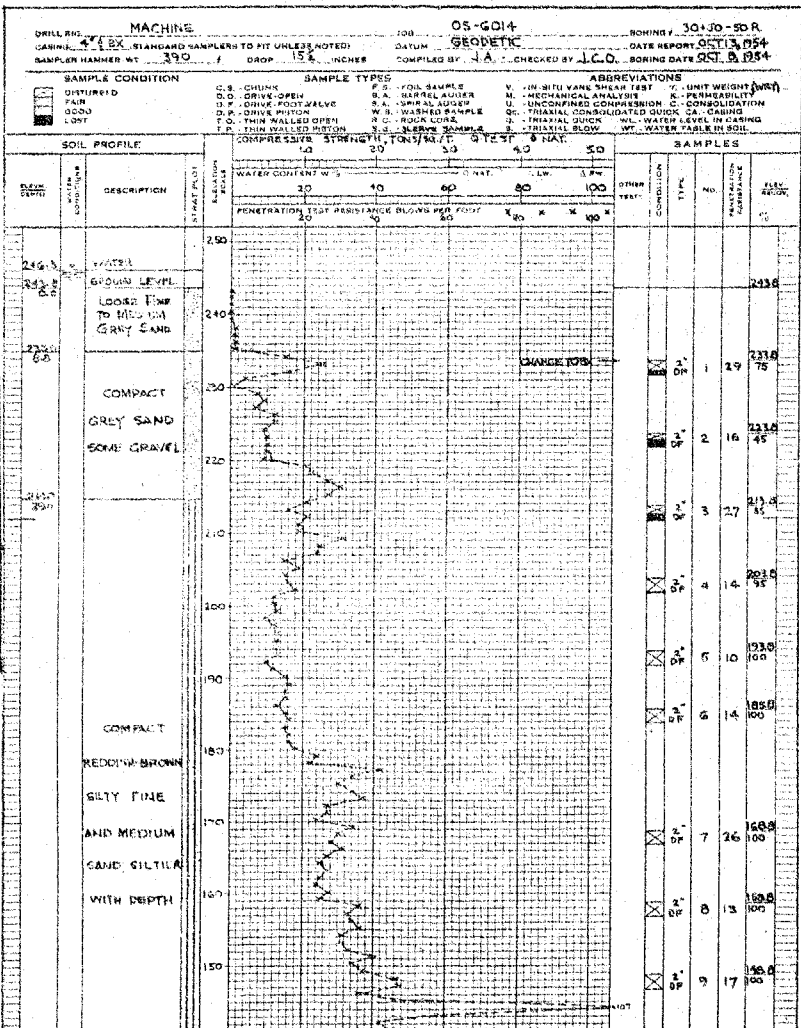
JM/eh

APPENDIX I  
OFFICE REPORTS ON SOIL EXPLORATION

**GEOCON LTD**

# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I  
FIGURE 1  
PROJECT OS6014



GEOTEC LTD

# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 2  
PROJECT-OS6014

2

DRILL NO. <u>140X</u> MACHINE <u>98-6014</u> JOB <u>OS-6014</u> BORING <u>34-50-50R</u>	
CASH NO. <u>140X</u> (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM <u>CEMETERY</u> DATE REPORT <u>AUG 26, 1954</u>	
SAMPLER HAMMER WT. <u>390</u> / DROP <u>17</u> INCHES COMPILED BY <u>B.J.M.</u> CHECKED BY <u>J.L.O.</u> BORING DATE <u>AUG 26, 1954</u>	
<b>SAMPLE CONDITION</b> DISTURBED <input type="checkbox"/> FAIR <input type="checkbox"/> GOOD <input type="checkbox"/> LOST <input type="checkbox"/>	
<b>SAMPLE TYPES</b> C.S. - CHUNK F.S. - POIL SAMPLE V. - IN-SITU VANE SHEAR TEST Z. - UNIT WEIGHT (WET) D.O. - DRIVE OPEN S.A. - SPIRAL AUGER M. - MECHANICAL ANALYSIS X. - PENETRABILITY D.F. - DRIVE FOOT VALVE W.A. - WASHED SAMPLE U. - UNCONFINED COMPRESSION C. - CONSOLIDATION O.P. - DRIVE POSITION W.S. - WASHED SAMPLE Q. - TRIAXIAL CONSOLIDATED QUICK C.A. - GUSHING Y.O. - YIELD WALLED OPEN R.C. - ROCK CUT S. - TRIAXIAL SLOW W.L. - WATER LEVEL IN CASING Y.P. - YIELD WALLED OPEN S.T. - TRIAXIAL SLOW W.T. - WATER TABLE IN SOIL	
<b>ABBREVIATIONS</b> C. - COMPRESSION S.T. - TRIAXIAL SLOW W.T. - WATER TABLE IN SOIL C. - COMPRESSION S.T. - TRIAXIAL SLOW W.T. - WATER TABLE IN SOIL	
<b>SOIL PROFILE</b> WATER CONTENT W. % 20 40 60 80 100 PENETRATION TEST RESISTANCE BLOWS PER FOOT X 50 100	
DEPTH FEET	DESCRIPTION 246.5 246.0 245.5 245.0 244.5 244.0 243.5 243.0 242.5 242.0 241.5 241.0 240.5 240.0 239.5 239.0 238.5 238.0 237.5 237.0 236.5 236.0 235.5 235.0 234.5 234.0 233.5 233.0 232.5 232.0 231.5 231.0 230.5 230.0 229.5 229.0 228.5 228.0 227.5 227.0 226.5 226.0 225.5 225.0 224.5 224.0 223.5 223.0 222.5 222.0 221.5 221.0 220.5 220.0 219.5 219.0 218.5 218.0 217.5 217.0 216.5 216.0 215.5 215.0 214.5 214.0 213.5 213.0 212.5 212.0 211.5 211.0 210.5 210.0 209.5 209.0 208.5 208.0 207.5 207.0 206.5 206.0 205.5 205.0 204.5 204.0 203.5 203.0 202.5 202.0 201.5 201.0 200.5 200.0 199.5 199.0 198.5 198.0 197.5 197.0 196.5 196.0 195.5 195.0 194.5 194.0 193.5 193.0 192.5 192.0 191.5 191.0 190.5 190.0 189.5 189.0 188.5 188.0 187.5 187.0 186.5 186.0 185.5 185.0 184.5 184.0 183.5 183.0 182.5 182.0 181.5 181.0 180.5 180.0 179.5 179.0 178.5 178.0 177.5 177.0 176.5 176.0 175.5 175.0 174.5 174.0 173.5 173.0 172.5 172.0 171.5 171.0 170.5 170.0 169.5 169.0 168.5 168.0 167.5 167.0 166.5 166.0 165.5 165.0 164.5 164.0 163.5 163.0 162.5 162.0 161.5 161.0 160.5 160.0 159.5 159.0 158.5 158.0 157.5 157.0 156.5 156.0 155.5 155.0 154.5 154.0 153.5 153.0 152.5 152.0 151.5 151.0 150.5 150.0 149.5 149.0 148.5 148.0 147.5 147.0 146.5 146.0 145.5 145.0 144.5 144.0 143.5 143.0 142.5 142.0 141.5 141.0 140.5 140.0 139.5 139.0 138.5 138.0 137.5 137.0 136.5 136.0 135.5 135.0 134.5 134.0 133.5 133.0 132.5 132.0 131.5 131.0 130.5 130.0 129.5 129.0 128.5 128.0 127.5 127.0 126.5 126.0 125.5 125.0 124.5 124.0 123.5 123.0 122.5 122.0 121.5 121.0 120.5 120.0 119.5 119.0 118.5 118.0 117.5 117.0 116.5 116.0 115.5 115.0 114.5 114.0 113.5 113.0 112.5 112.0 111.5 111.0 110.5 110.0 109.5 109.0 108.5 108.0 107.5 107.0 106.5 106.0 105.5 105.0 104.5 104.0 103.5 103.0 102.5 102.0 101.5 101.0 100.5 100.0 99.5 99.0 98.5 98.0 97.5 97.0 96.5 96.0 95.5 95.0 94.5 94.0 93.5 93.0 92.5 92.0 91.5 91.0 90.5 90.0 89.5 89.0 88.5 88.0 87.5 87.0 86.5 86.0 85.5 85.0 84.5 84.0 83.5 83.0 82.5 82.0 81.5 81.0 80.5 80.0 79.5 79.0 78.5 78.0 77.5 77.0 76.5 76.0 75.5 75.0 74.5 74.0 73.5 73.0 72.5 72.0 71.5 71.0 70.5 70.0 69.5 69.0 68.5 68.0 67.5 67.0 66.5 66.0 65.5 65.0 64.5 64.0 63.5 63.0 62.5 62.0 61.5 61.0 60.5 60.0 59.5 59.0 58.5 58.0 57.5 57.0 56.5 56.0 55.5 55.0 54.5 54.0 53.5 53.0 52.5 52.0 51.5 51.0 50.5 50.0 49.5 49.0 48.5 48.0 47.5 47.0 46.5 46.0 45.5 45.0 44.5 44.0 43.5 43.0 42.5 42.0 41.5 41.0 40.5 40.0 39.5 39.0 38.5 38.0 37.5 37.0 36.5 36.0 35.5 35.0 34.5 34.0 33.5 33.0 32.5 32.0 31.5 31.0 30.5 30.0 29.5 29.0 28.5 28.0 27.5 27.0 26.5 26.0 25.5 25.0 24.5 24.0 23.5 23.0 22.5 22.0 21.5 21.0 20.5 20.0 19.5 19.0 18.5 18.0 17.5 17.0 16.5 16.0 15.5 15.0 14.5 14.0 13.5 13.0 12.5 12.0 11.5 11.0 10.5 10.0 9.5 9.0 8.5 8.0 7.5 7.0 6.5 6.0 5.5 5.0 4.5 4.0 3.5 3.0 2.5 2.0 1.5 1.0 0.5 0.0 -0.5 -1.0 -1.5 -2.0 -2.5 -3.0 -3.5 -4.0 -4.5 -5.0 -5.5 -6.0 -6.5 -7.0 -7.5 -8.0 -8.5 -9.0 -9.5 -10.0 -10.5 -11.0 -11.5 -12.0 -12.5 -13.0 -13.5 -14.0 -14.5 -15.0 -15.5 -16.0 -16.5 -17.0 -17.5 -18.0 -18.5 -19.0 -19.5 -20.0 -20.5 -21.0 -21.5 -22.0 -22.5 -23.0 -23.5 -24.0 -24.5 -25.0 -25.5 -26.0 -26.5 -27.0 -27.5 -28.0 -28.5 -29.0 -29.5 -30.0 -30.5 -31.0 -31.5 -32.0 -32.5 -33.0 -33.5 -34.0 -34.5 -35.0 -35.5 -36.0 -36.5 -37.0 -37.5 -38.0 -38.5 -39.0 -39.5 -40.0 -40.5 -41.0 -41.5 -42.0 -42.5 -43.0 -43.5 -44.0 -44.5 -45.0 -45.5 -46.0 -46.5 -47.0 -47.5 -48.0 -48.5 -49.0 -49.5 -50.0 -50.5 -51.0 -51.5 -52.0 -52.5 -53.0 -53.5 -54.0 -54.5 -55.0 -55.5 -56.0 -56.5 -57.0 -57.5 -58.0 -58.5 -59.0 -59.5 -60.0 -60.5 -61.0 -61.5 -62.0 -62.5 -63.0 -63.5 -64.0 -64.5 -65.0 -65.5 -66.0 -66.5 -67.0 -67.5 -68.0 -68.5 -69.0 -69.5 -70.0 -70.5 -71.0 -71.5 -72.0 -72.5 -73.0 -73.5 -74.0 -74.5 -75.0 -75.5 -76.0 -76.5 -77.0 -77.5 -78.0 -78.5 -79.0 -79.5 -80.0 -80.5 -81.0 -81.5 -82.0 -82.5 -83.0 -83.5 -84.0 -84.5 -85.0 -85.5 -86.0 -86.5 -87.0 -87.5 -88.0 -88.5 -89.0 -89.5 -90.0 -90.5 -91.0 -91.5 -92.0 -92.5 -93.0 -93.5 -94.0 -94.5 -95.0 -95.5 -96.0 -96.5 -97.0 -97.5 -98.0 -98.5 -99.0 -99.5 -100.0 -100.5 -101.0 -101.5 -102.0 -102.5 -103.0 -103.5 -104.0 -104.5 -105.0 -105.5 -106.0 -106.5 -107.0 -107.5 -108.0 -108.5 -109.0 -109.5 -110.0 -110.5 -111.0 -111.5 -112.0 -112.5 -113.0 -113.5 -114.0 -114.5 -115.0 -115.5 -116.0 -116.5 -117.0 -117.5 -118.0 -118.5 -119.0 -119.5 -120.0 -120.5 -121.0 -121.5 -122.0 -122.5 -123.0 -123.5 -124.0 -124.5 -125.0 -125.5 -126.0 -126.5 -127.0 -127.5 -128.0 -128.5 -129.0 -129.5 -130.0 -130.5 -131.0 -131.5 -132.0 -132.5 -133.0 -133.5 -134.0 -134.5 -135.0 -135.5 -136.0 -136.5 -137.0 -137.5 -138.0 -138.5 -139.0 -139.5 -140.0 -140.5 -141.0 -141.5 -142.0 -142.5 -143.0 -143.5 -144.0 -144.5 -145.0 -145.5 -146.0 -146.5 -147.0 -147.5 -148.0 -148.5 -149.0 -149.5 -150.0 -150.5 -151.0 -151.5 -152.0 -152.5 -153.0 -153.5 -154.0 -154.5 -155.0 -155.5 -156.0 -156.5 -157.0 -157.5 -158.0 -158.5 -159.0 -159.5 -160.0 -160.5 -161.0 -161.5 -162.0 -162.5 -163.0 -163.5 -164.0 -164.5 -165.0 -165.5 -166.0 -166.5 -167.0 -167.5 -168.0 -168.5 -169.0 -169.5 -170.0 -170.5 -171.0 -171.5 -172.0 -172.5 -173.0 -173.5 -174.0 -174.5 -175.0 -175.5 -176.0 -176.5 -177.0 -177.5 -178.0 -178.5 -179.0 -179.5 -180.0 -180.5 -181.0 -181.5 -182.0 -182.5 -183.0 -183.5 -184.0 -184.5 -185.0 -185.5 -186.0 -186.5 -187.0 -187.5 -188.0 -188.5 -189.0 -189.5 -190.0 -190.5 -191.0 -191.5 -192.0 -192.5 -193.0 -193.5 -194.0 -194.5 -195.0 -195.5 -196.0 -196.5 -197.0 -197.5 -198.0 -198.5 -199.0 -199.5 -200.0 -200.5 -201.0 -201.5 -202.0 -202.5 -203.0 -203.5 -204.0 -204.5 -205.0 -205.5 -206.0 -206.5 -207.0 -207.5 -208.0 -208.5 -209.0 -209.5 -210.0 -210.5 -211.0 -211.5 -212.0 -212.5 -213.0 -213.5 -214.0 -214.5 -215.0 -215.5 -216.0 -216.5 -217.0 -217.5 -218.0 -218.5 -219.0 -219.5 -220.0 -220.5 -221.0 -221.5 -222.0 -222.5 -223.0 -223.5 -224.0 -224.5 -225.0 -225.5 -226.0 -226.5 -227.0 -227.5 -228.0 -228.5 -229.0 -229.5 -230.0 -230.5 -231.0 -231.5 -232.0 -232.5 -233.0 -233.5 -234.0 -234.5 -235.0 -235.5 -236.0 -236.5 -237.0 -237.5 -238.0 -238.5 -239.0 -239.5 -240.0 -240.5 -241.0 -241.5 -242.0 -242.5 -243.0 -243.5 -244.0 -244.5 -245.0 -245.5 -246.0 -246.5 -247.0 -247.5 -248.0 -248.5 -249.0 -249.5 -250.0 -250.5 -251.0 -251.5 -252.0 -252.5 -253.0 -253.5 -254.0 -254.5 -255.0 -255.5 -256.0 -256.5 -257.0 -257.5 -258.0 -258.5 -259.0 -259.5 -260.0 -260.5 -261.0 -261.5 -262.0 -262.5 -263.0 -263.5 -264.0 -264.5 -265.0 -265.5 -266.0 -266.5 -267.0 -267.5 -268.0 -268.5 -269.0 -269.5 -270.0 -270.5 -271.0 -271.5 -272.0 -272.5 -273.0 -273.5 -274.0 -274.5 -275.0 -275.5 -276.0 -276.5 -277.0 -277.5 -278.0 -278.5 -279.0 -279.5 -280.0 -280.5 -281.0 -281.5 -282.0 -282.5 -283.0 -283.5 -284.0 -284.5 -285.0 -285.5 -286.0 -286.5 -287.0 -287.5 -288.0 -288.5 -289.0 -289.5 -290.0 -290.5 -291.0 -291.5 -292.0 -292.5 -293.0 -293.5 -294.0 -294.5 -295.0 -295.5 -296.0 -296.5 -297.0 -297.5 -298.0 -298.5 -299.0 -299.5 -300.0 -300.5 -301.0 -301.5 -302.0 -302.5 -303.0 -303.5 -304.0 -304.5 -305.0 -305.5 -306.0 -306.5 -307.0 -307.5 -308.0 -308.5 -309.0 -309.5 -310.0 -310.5 -311.0 -311.5 -312.0 -312.5 -313.0 -313.5 -314.0 -314.5 -315.0 -315.5 -316.0 -316.5 -317.0 -317.5 -318.0 -318.5 -319.0 -319.5 -320.0 -320.5 -321.0 -321.5 -322.0 -322.5 -323.0 -323.5 -324.0 -324.5 -325.0 -325.5 -326.0 -326.5 -327.0 -327.5 -328.0 -328.5 -329.0 -329.5 -330.0 -330.5 -331.0 -331.5 -332.0 -332.5 -333.0 -333.5 -334.0 -334.5 -335.0 -335.5 -336.0 -336.5 -337.0 -337.5 -338.0 -338.5 -339.0 -339.5 -340.0 -340.5 -341.0 -341.5 -342.0 -342.5 -343.0 -343.5 -344.0 -344.5 -345.0 -345.5 -346.0 -346.5 -347.0 -347.5 -348.0 -348.5 -349.0 -349.5 -350.0 -350.5 -351.0 -351.5 -352.0 -352.5 -353.0 -353.5 -354.0 -354.5 -355.0 -355.5 -356.0 -356.5 -357.0 -357.5 -358.0 -358.5 -359.0 -359.5 -360.0 -360.5 -361.0 -361.5 -362.0 -362.5 -363.0 -363.5 -364.0 -364.5 -365.0 -365.5 -366.0 -366.5 -367.0 -367.5 -368.0 -368.5 -369.0 -369.5 -370.0 -370.5 -371.0 -371.5 -372.0 -372.5 -373.0 -373.5 -374.0 -374.5 -375.0 -375.5 -376.0 -376.5 -377.0 -377.5 -378.0 -378.5 -379.0 -379.5 -380.0 -380.5 -381.0 -381.5 -382.0 -382.5 -383.0 -383.5 -384.0 -384.5 -385.0 -385.5 -386.0 -386.5 -387.0 -387.5 -388.0 -388.5 -389.0 -389.5 -390.0 -390.5 -391.0 -391.5 -392.0 -392.5 -393.0 -393.5 -394.0 -394.5 -395.0 -395.5 -396.0 -396.5 -397.0 -397.5 -398.0 -398.5 -399.0 -399.5 -400.0 -400.5 -401.0 -401.5 -402.0 -402.5 -403.0 -403.5 -404.0 -404.5 -405.0 -405.5 -406.0 -406.5 -407.0 -407.5 -408.0 -408.5 -409.0 -409.5 -410.0 -410.5 -411.0 -411.5 -412.0 -412.5 -413.0 -413.5 -414.0 -414.5 -415.0 -415.5 -416.0 -416.5 -417.0 -417.5 -418.0 -418.5 -419.0 -419.5 -420.0 -420.5 -421.0 -421.5 -422.0 -422.5 -423.0 -423.5 -424.0 -424.5 -425.0 -425.5 -426.0 -426.5 -427.0 -427.5 -428.0 -428.5 -429.0 -429.5 -430.0 -430.5 -431.0 -431.5 -432.0 -432.5 -433.0 -433.5 -434.0 -434.5 -435.0 -435.5 -436.0 -436.5 -437.0 -437.5 -438.0 -438.5 -439.0 -439.5 -440.0 -440.5 -441.0 -441.5 -442.0 -442.5 -443.0 -443.5 -444.0 -444.5 -445.0 -445.5 -446.0 -446.5 -447.0 -447.5 -448.0 -448.5 -449.0 -449.5 -450.0 -450.5 -451.0 -451.5 -452.0 -452.5 -453.0 -453.5 -454.0 -454.5 -455.0 -455.5 -456.0 -456.5 -457.0 -457.5 -458.0 -458.5 -459.0 -459.5 -460.0 -460.5 -461.0 -461.5 -462.0 -462.5 -463.0 -463.5 -464.0 -464.5 -465.0 -465.5 -466.0 -466.5 -467.0 -467.5 -468.0 -468.5 -469.0 -469.5 -470.0 -470.5 -471.0 -471.5 -472.0 -472.5 -473.0 -473.5 -474.0 -474.5 -475.0 -475.5 -476.0 -476.5 -477.0 -477.5 -478.0 -478.5 -479.0 -479.5 -480.0 -480.5 -481.0 -481.5 -482.0 -482.5 -483.0 -483.5 -484.0 -484.5 -485.0 -485.5 -486.0 -486.5 -487.0 -487.5 -488.0 -488.5 -489.0 -489.5 -490.0 -490.5 -491.0 -491.5 -492.0 -492.5 -493.0 -493.5 -494.0 -494.5 -495.0 -495.5 -496.0 -496.5 -497.0 -497.5 -498.0 -498.5 -499.0 -499.5 -500.0 -500.5 -501.0 -501.5 -502.0 -502.5 -503.0 -503.5 -504.0 -504.5 -505.0 -505.5 -506.0 -506.5 -507.0 -507.5 -508.0 -508.5 -509.0 -509.5 -510.0 -510.5 -511.0 -511.5 -512.0 -512.5 -513.0 -513.5 -514.0 -514.5 -515.0 -515.5 -516.0 -516.5 -517.0 -517.5 -518.0 -518.5 -519.0 -519.5 -520.0 -520.5 -521.0 -521.5 -522.0 -522.5 -523.0 -523.5 -524.0 -524.5 -525.0 -525.5 -526.0 -526.5 -527.0 -527.5 -528.0 -528.5 -529.0 -529.5 -530.0 -5

APPENDIX 1  
FIGURE 3  
PROJECT-OS6014

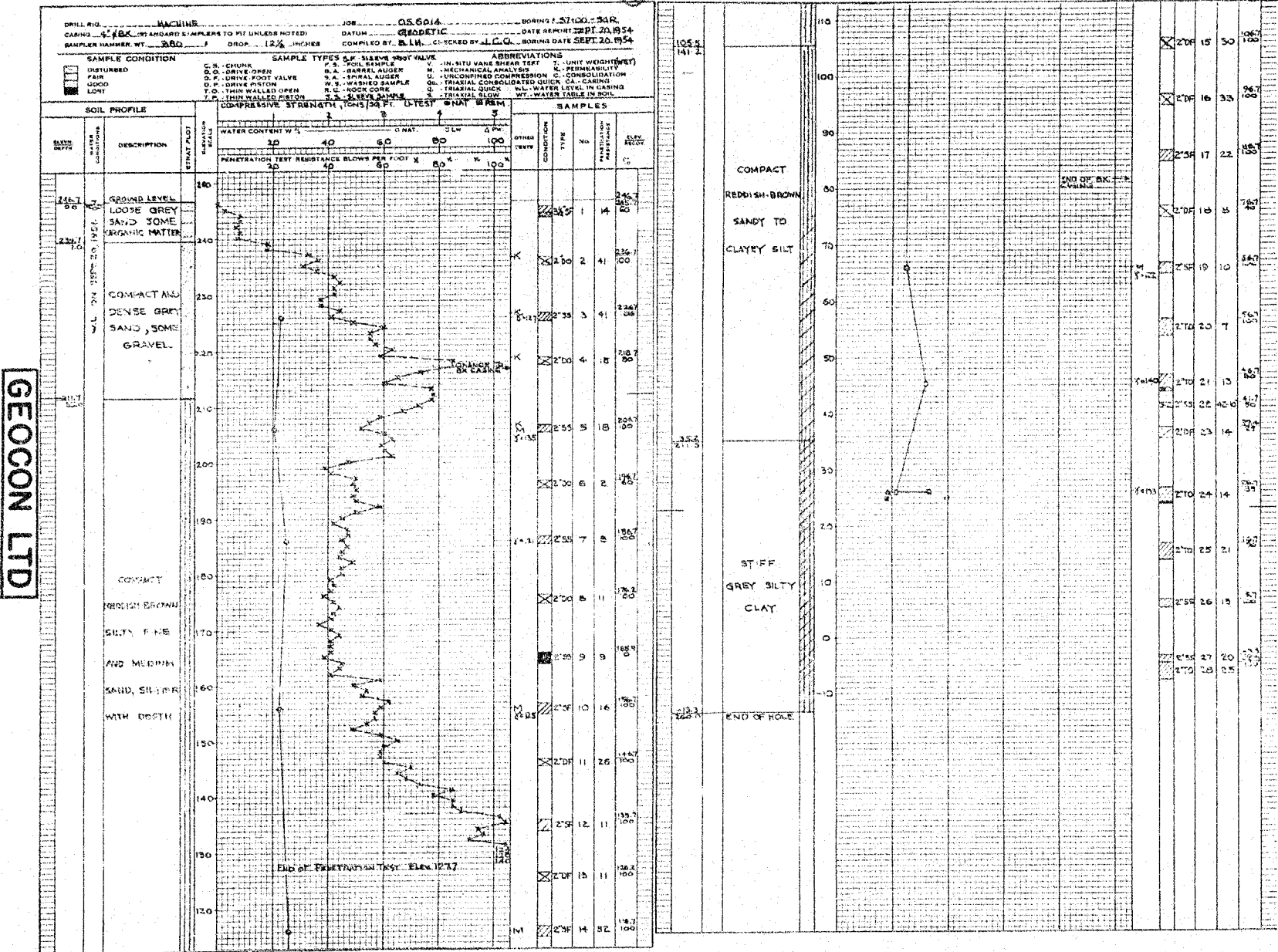
**GEOCON LTD**



APPENDIX I  
FIGURE 4  
PROJECT-OS6014

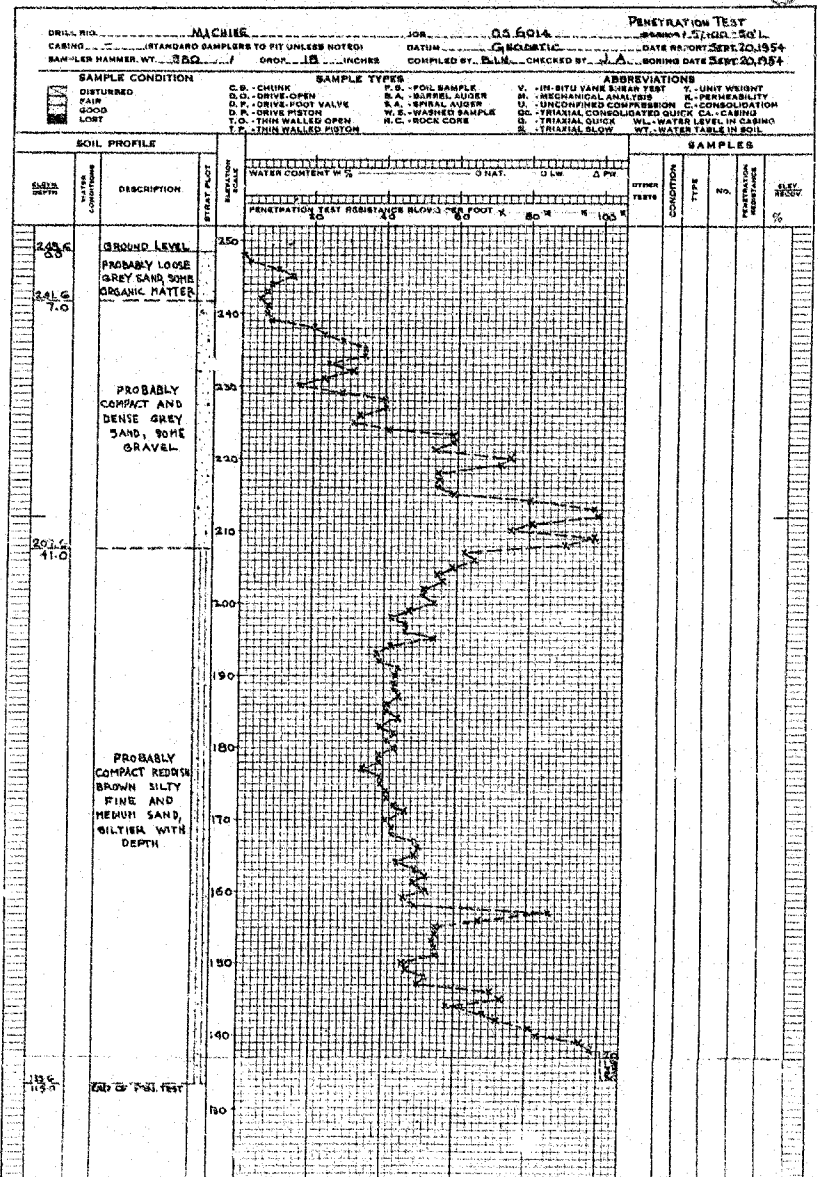


APPENDIX I  
FIGURE 5  
PROJECT-OS6014



# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 6  
PROJECT-056014



APPENDIX I  
FIGURE 7  
PROJECT-OS6014

[illegible]

APPENDIX I  
FIGURE 8  
PROJECT-OS6014

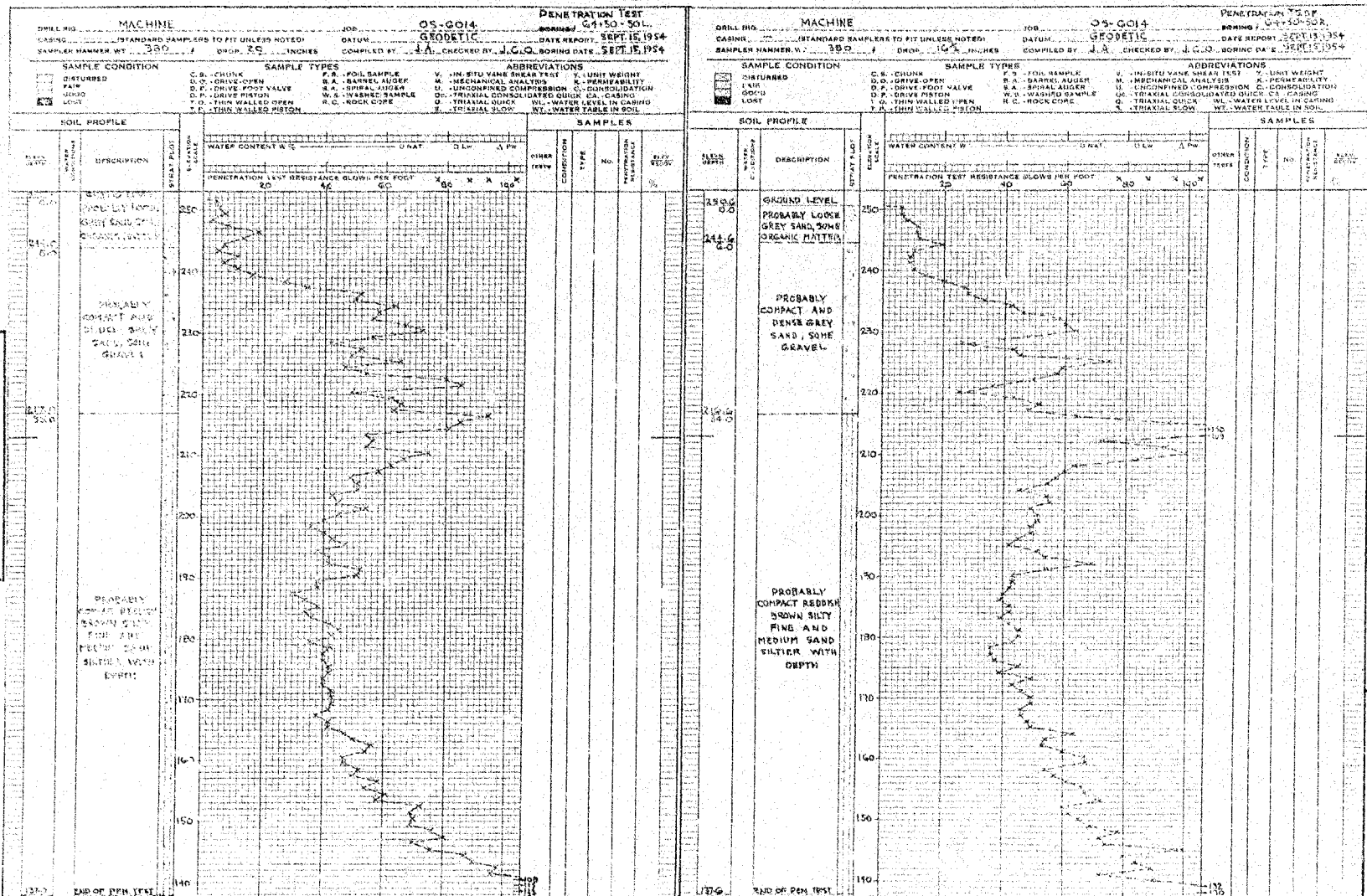




# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 9  
PROJECT-OS6014

(12)



GEOCON LTD.

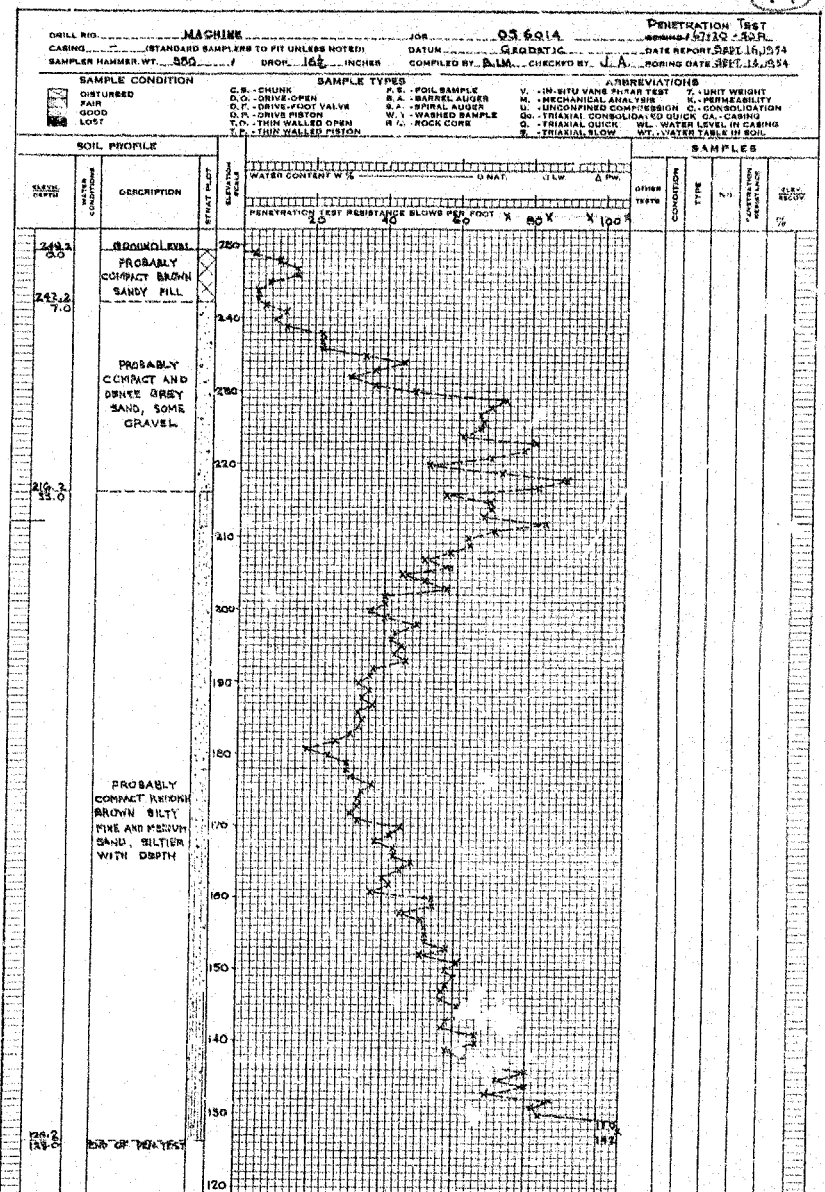
APPENDIX I  
FIGURE 10  
PROJECT-OS6014





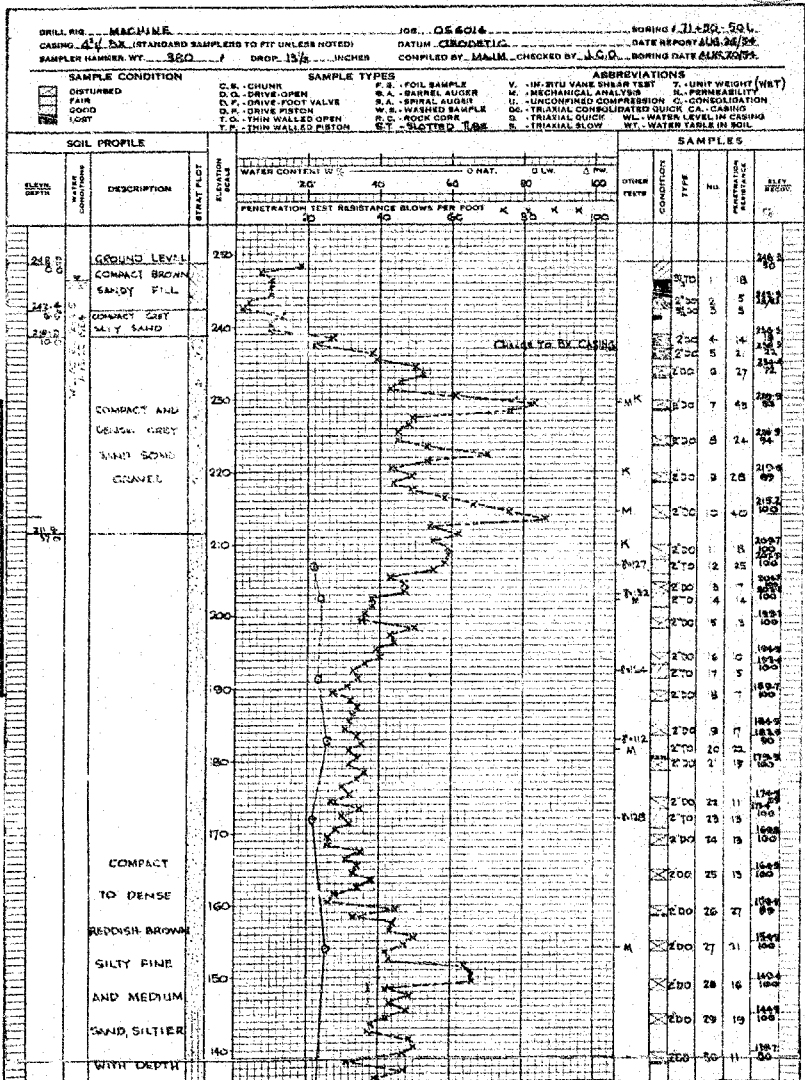
# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I  
FIGURE 11  
PROJECT-OS6014



GEOCON LTD

APPENDIX I  
FIGURE 12  
PROJECT-OS6014



**GEOCON LTD**

# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 13  
PROJECT-OS6014

GEOCON LTD

16  
WASHBORING  
DRILL NO. MACHINE JOB OS-6014 BORING 71+60-10R  
CASING 50 STANDARD SAMPLERS TO FIT UNLESS NOTED DATE REPORT 4 SEPT 54  
SAMPLER HAMMER WT. 1 DROP 1 INCHES COMPILED BY MAJM CHECKED BY JCO BORING DATE 20 AUG 54

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
<input type="checkbox"/> DISTURBED	C.S. - CHUNK	F.S. - FOIL SAMPLE	V. - IN-SITU VANE SHEAR TEST	T. - UNIT WEIGHT	X. - PERMEABILITY
<input type="checkbox"/> PAIR	D.O. - DRIVE-OPEN	B.A. - BARREL AUGER	M. - MECHANICAL ANALYSIS	C. - CONSOLIDATION	Q. - TRIAXIAL CONSOLIDATED QUICK
<input type="checkbox"/> GOOD	D.P. - DRIVE-FOOT VALVE	S.A. - SPIRAL AUGER	U. - UNCONFINED COMPRESSION	CA. - CASING	WL. - WATER LEVEL IN CASING
<input type="checkbox"/> LOST	D.R. - DRIVE PISTON	W.S. - WASHED SAMPLE	Q. - TRIAXIAL CONSOLIDATED QUICK	WT. - WATER TABLE IN SOIL	
	T.D. - THIN WALLED OPEN	R.C. - ROCK CORE	S. - TRIAXIAL SLOW		
	T.P. - THIN WALLED PISTON				

SOIL PROFILE		SAMPLES	
DEPTH	DESCRIPTION	TESTS	NO.
249.3 0.0	GROUND LEVEL		
240.0	GREY SAND AND GRAVEL		
239.3 10.0	END OF HOLE		

19  
WASHBORING  
DRILL NO. MACHINE JOB OS-6014 BORING 71+60-20R  
CASING 50 STANDARD SAMPLERS TO FIT UNLESS NOTED DATE REPORT 4 SEPT 54  
SAMPLER HAMMER WT. 1 DROP 1 INCHES COMPILED BY MAJM CHECKED BY JCO BORING DATE 20 AUG 54

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
<input type="checkbox"/> DISTURBED	C.S. - CHUNK	F.S. - FOIL SAMPLE	V. - IN-SITU VANE SHEAR TEST	T. - UNIT WEIGHT	X. - PERMEABILITY
<input type="checkbox"/> PAIR	D.O. - DRIVE-OPEN	B.A. - BARREL AUGER	M. - MECHANICAL ANALYSIS	C. - CONSOLIDATION	Q. - TRIAXIAL CONSOLIDATED QUICK
<input type="checkbox"/> GOOD	D.P. - DRIVE-FOOT VALVE	S.A. - SPIRAL AUGER	U. - UNCONFINED COMPRESSION	CA. - CASING	WL. - WATER LEVEL IN CASING
<input type="checkbox"/> LOST	D.R. - DRIVE PISTON	W.S. - WASHED SAMPLE	Q. - TRIAXIAL CONSOLIDATED QUICK	WT. - WATER TABLE IN SOIL	
	T.D. - THIN WALLED OPEN	R.C. - ROCK CORE	S. - TRIAXIAL SLOW		
	T.P. - THIN WALLED PISTON				

SOIL PROFILE		SAMPLES	
DEPTH	DESCRIPTION	TESTS	NO.
249.3 0.0	GROUND LEVEL		
240.0	GREY SAND WITH GRAVEL		
239.3 10.0	END OF HOLE		

17  
WASHBORING 71+60-10R

SOIL PROFILE		SAMPLES	
DEPTH	DESCRIPTION	TESTS	NO.
249.3 0.0	GROUND LEVEL		
240.0	GREY SAND & GRAVEL		
239.3 10.0	END OF HOLE		

20  
WASHBORING 71+60-10R

SOIL PROFILE		SAMPLES	
DEPTH	DESCRIPTION	TESTS	NO.
249.3 0.0	GROUND LEVEL		
240.0	GREY SAND WITH GRAVEL		
239.3 10.0	END OF HOLE		

18  
WASHBORING 71+60-20R

SOIL PROFILE		SAMPLES	
DEPTH	DESCRIPTION	TESTS	NO.
249.3 0.0	GROUND LEVEL		
240.0	GREY SAND & GRAVEL		
239.3 10.0	END OF HOLE		

21  
WASHBORING 71+60-30R

SOIL PROFILE		SAMPLES	
DEPTH	DESCRIPTION	TESTS	NO.
249.3 0.0	GROUND LEVEL		
240.0	GRAVEL FILL		
239.3 10.0	END OF HOLE		

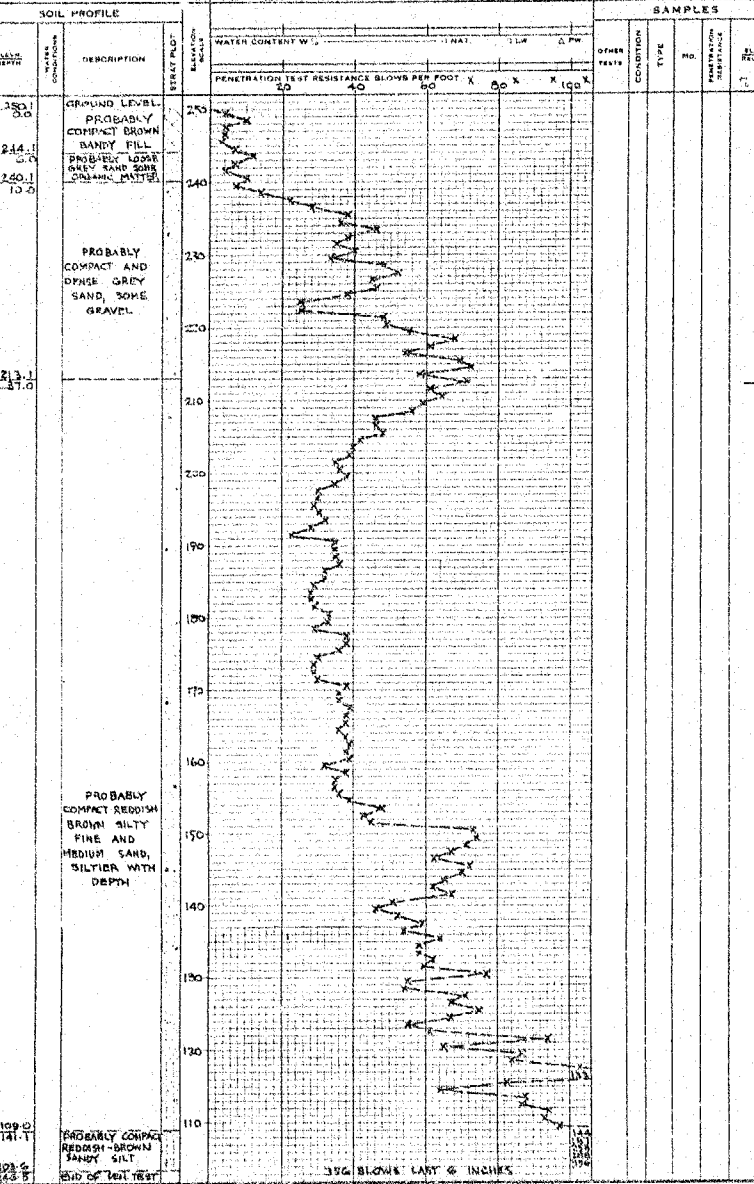
# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 14  
PROJECT-OS6014

23

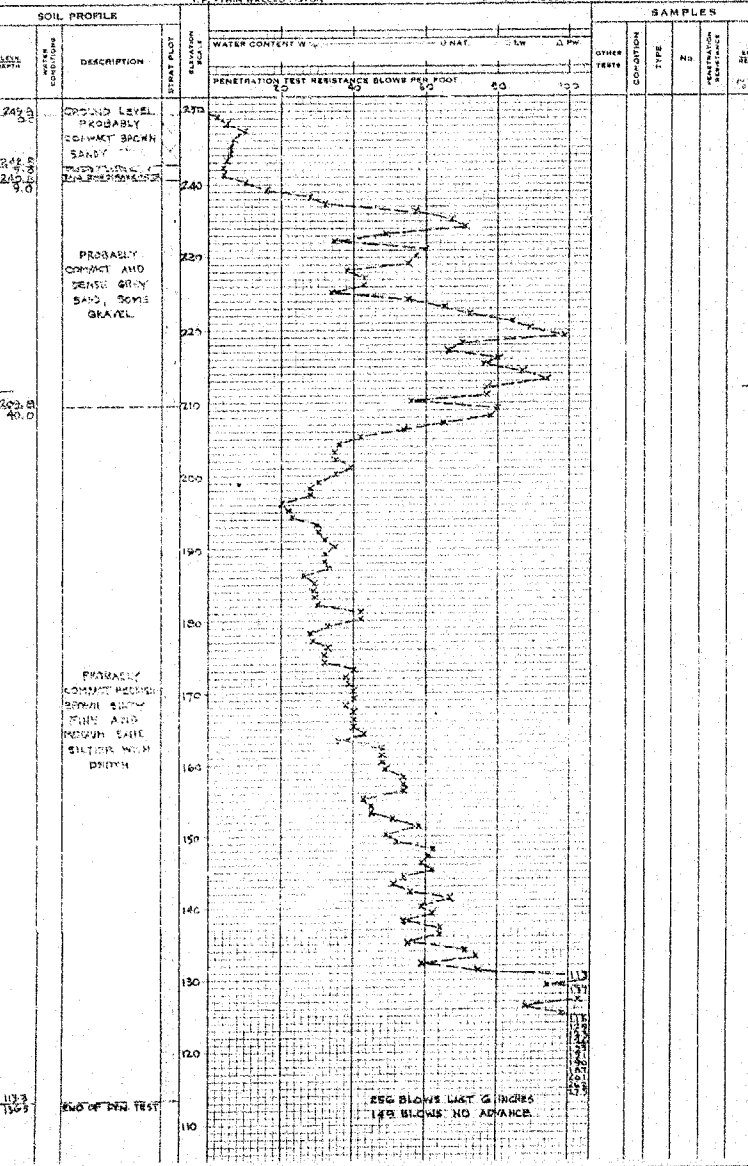
DRILL RIG: MACHINE  
CASING: STANDARD SAMPLERS TO FIT UNLESS NOTED  
SAMPLER HAMMER WT: 350  
DATE REPORT: 28.8.55  
BORING DATE: 28.8.55  
JOB: OS 6014  
DATHUM: GEODETIC  
COMPILED BY: B.J.M.  
CHECKED BY: J.A.  
BORING DATE: 28.8.55

SAMPLE CONDITION: DISRUPTED, FAIR, GOOD, BEST  
SAMPLE TYPES: C.S. - CHUCK, D.D. - DRIVE OPEN, D.F. - DRIVE FOOT VALVE, D.P. - DRIVE PISTON, T.O. - THIN WALLED OPEN, T.C. - THIN WALLED PISTON  
SOIL SAMPLES: S.1 - BARREL AUGER, S.2 - SPIRAL AUGER, W.3 - WASHED SAMPLE, R.C. - ROCK CORE  
ABREVIATIONS: V. - IN. SITU VANE SHEAR TEST, M. - MECHANICAL ANALYSIS, U. - UNCONSOLIDATED COMPRESSION, Q. - TRIAXIAL CONSOLIDATED QUICK, WL - WATER LEVEL IN CASING, WT - WATER TABLE IN SOIL



DRILL RIG: MACHINE  
CASING: STANDARD SAMPLERS TO FIT UNLESS NOTED  
SAMPLER HAMMER WT: 350  
DATE REPORT: 28.8.55  
BORING DATE: 28.8.55  
JOB: OS 6014  
DATHUM: GEODETIC  
COMPILED BY: B.J.M.  
CHECKED BY: J.A.  
BORING DATE: 28.8.55

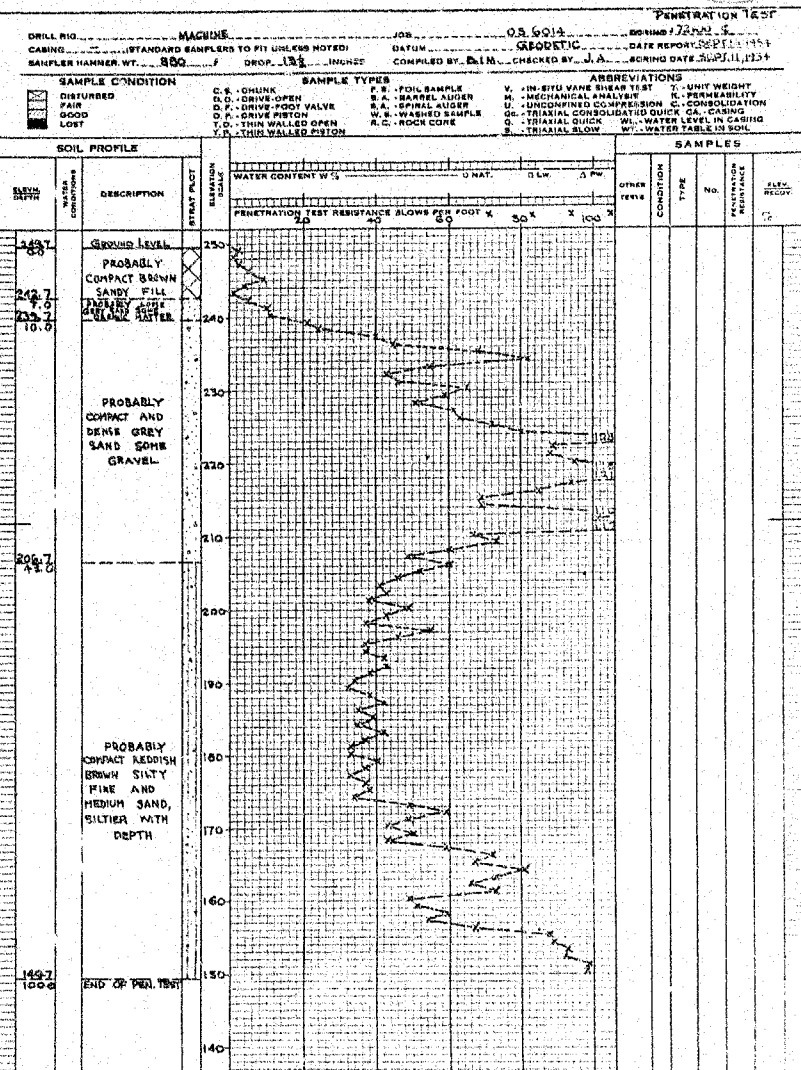
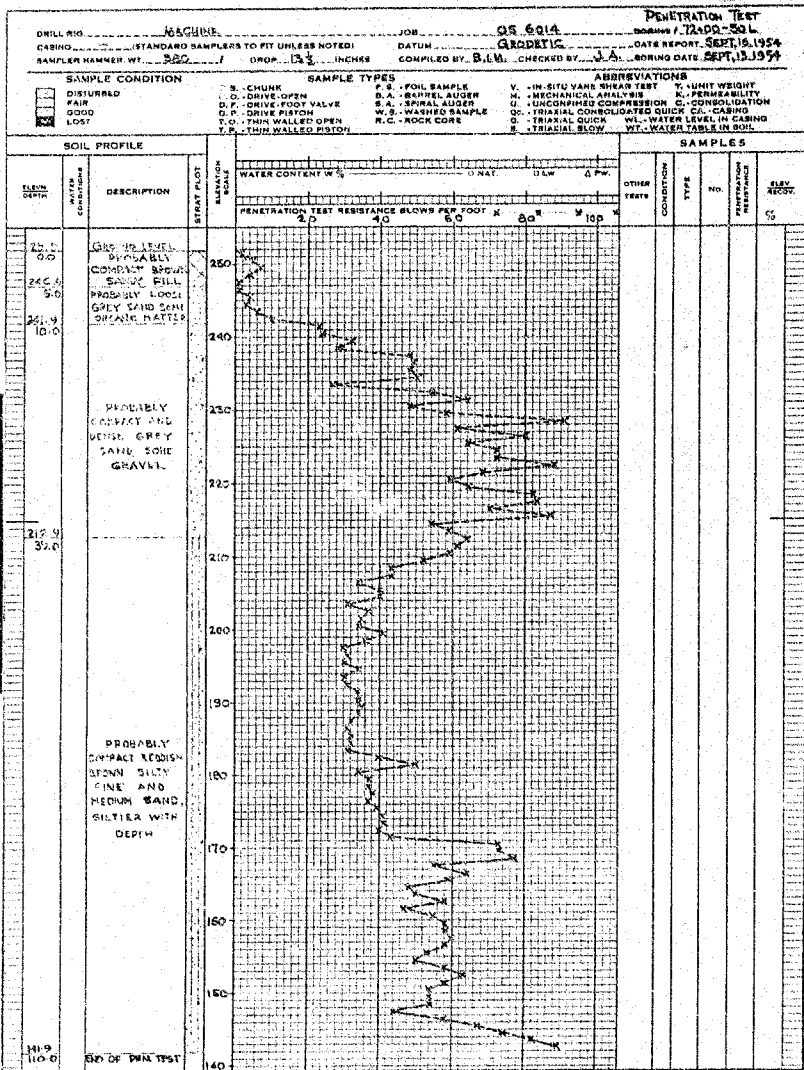
SAMPLE CONDITION: DISRUPTED, FAIR, GOOD, BEST  
SAMPLE TYPES: C.S. - CHUCK, D.D. - DRIVE OPEN, D.F. - DRIVE FOOT VALVE, D.P. - DRIVE PISTON, T.O. - THIN WALLED OPEN, T.C. - THIN WALLED PISTON  
SOIL SAMPLES: S.1 - BARREL AUGER, S.2 - SPIRAL AUGER, W.3 - WASHED SAMPLE, R.C. - ROCK CORE  
ABREVIATIONS: V. - IN. SITU VANE SHEAR TEST, M. - MECHANICAL ANALYSIS, U. - UNCONSOLIDATED COMPRESSION, Q. - TRIAXIAL CONSOLIDATED QUICK, WL - WATER LEVEL IN CASING, WT - WATER TABLE IN SOIL



GECON LTD

# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 15  
PROJECT-OS6014



GEOCON LTD

APPENDIX I  
FIGURE 16  
PROJECT-OS6014





# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX 1  
FIGURE 17  
PROJECT-OS6014

27

DRILL NO. MACHINE JOB OS-6014  
CASHING STANDARD SAMPLERS TO FIT UNLESS NOTED DATUM DEORETIC DATE REPORT 14 SEP 54  
SAMPLER HAMMER WT. 380 DROP 16 1/2 INCHES COMPILED BY MAW CHECKED BY J.A. BORING DATE 14 SEP 54

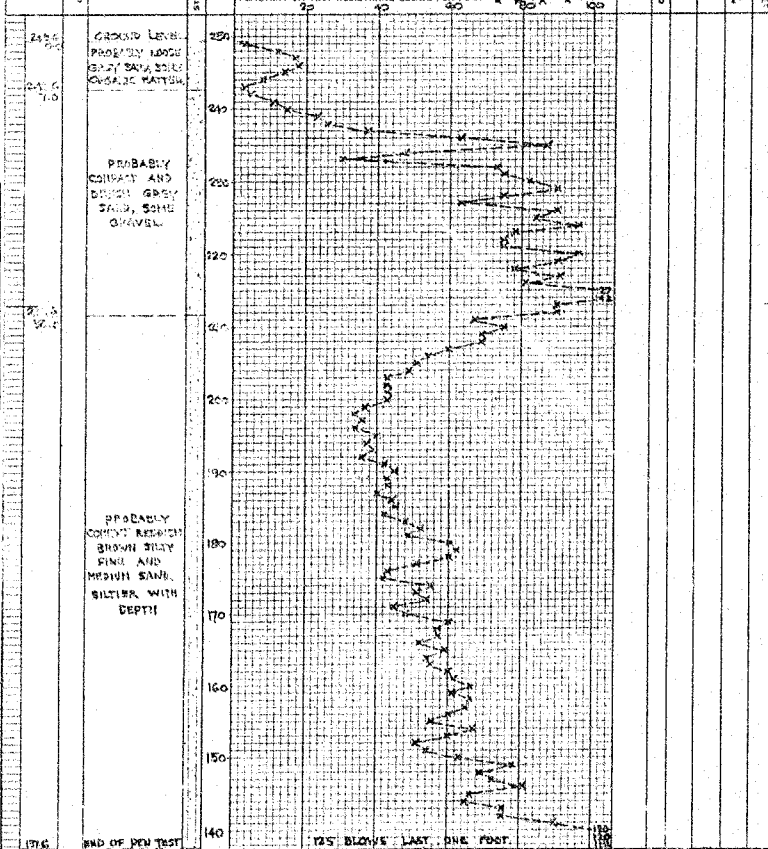
SAMPLE CONDITION  
DISTURBED  
FAIR  
GOOD  
LOST

SAMPLE TYPES  
C.S. - CHUNK  
D.O. - DRIVE-OPEN  
D.V. - DRIVE-FOOT VALVE  
D.P. - DRIVE PISTON  
T.O. - THIN WALLED OPEN  
T.P. - THIN WALLED PISTON

ABBREVIATIONS  
F.S. - FOUL SAMPLE  
B.A. - BARREL AUGER  
S.A. - SPIRAL AUGER  
W.S. - WASHED SAMPLE  
R.C. - ROCK CORE

OTHER TESTS  
V. - IN SITU VANE SHEAR TEST  
M. - MECHANICAL ANALYSIS  
U. - UNCONFINED COMPRESSION  
Q. - TRIAXIAL CONSOLIDATED QUICK  
C. - TRIAXIAL SLOW  
W. - WATER LEVEL IN CASING  
W.T. - WATER TABLE IN SOIL

SOIL PROFILE  
ELEV. DEPTH  
DESCRIPTION  
STAKE FOOT  
ELEV. DEPTH  
WATER CONTENT W %  
PENETRATION TEST RESISTANCE BLOWS PER FOOT



28

DRILL NO. MACHINE JOB OS-6014  
CASHING STANDARD SAMPLERS TO FIT UNLESS NOTED DATUM DEORETIC DATE REPORT 14 SEP 54  
SAMPLER HAMMER WT. 380 DROP 13 1/2 INCHES COMPILED BY MAW CHECKED BY J.A. BORING DATE 14 SEP 54

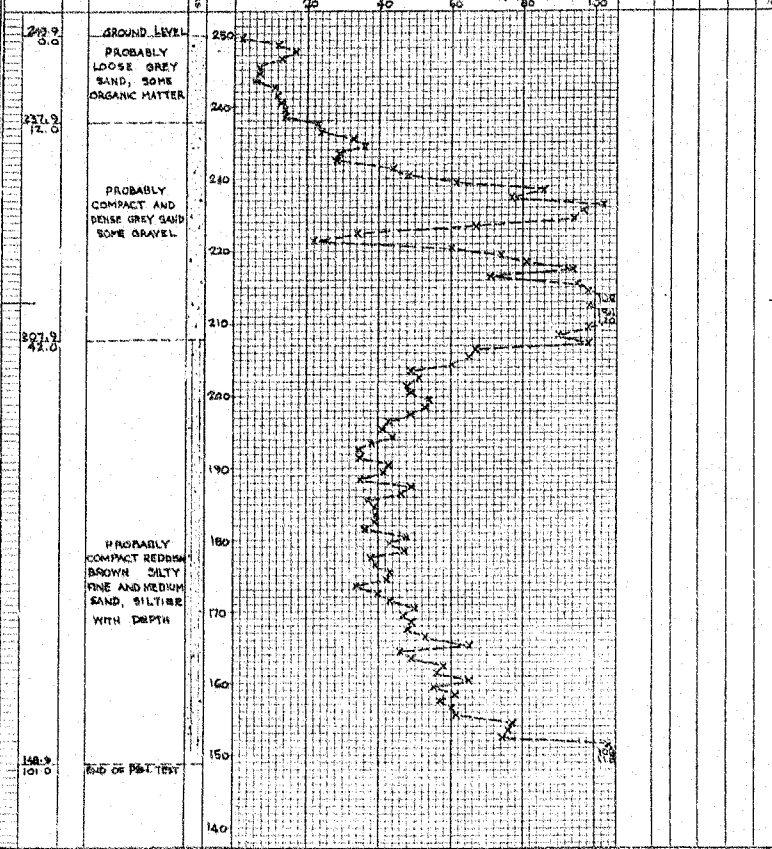
SAMPLE CONDITION  
DISTURBED  
FAIR  
GOOD  
LOST

SAMPLE TYPES  
C.S. - CHUNK  
D.O. - DRIVE-OPEN  
D.V. - DRIVE-FOOT VALVE  
D.P. - DRIVE PISTON  
T.O. - THIN WALLED OPEN  
T.P. - THIN WALLED PISTON

ABBREVIATIONS  
F.S. - FOUL SAMPLE  
B.A. - BARREL AUGER  
S.A. - SPIRAL AUGER  
W.S. - WASHED SAMPLE  
R.C. - ROCK CORE

OTHER TESTS  
V. - IN SITU VANE SHEAR TEST  
M. - MECHANICAL ANALYSIS  
U. - UNCONFINED COMPRESSION  
Q. - TRIAXIAL CONSOLIDATED QUICK  
C. - TRIAXIAL SLOW  
W. - WATER LEVEL IN CASING  
W.T. - WATER TABLE IN SOIL

SOIL PROFILE  
ELEV. DEPTH  
DESCRIPTION  
STAKE FOOT  
ELEV. DEPTH  
WATER CONTENT W %  
PENETRATION TEST RESISTANCE BLOWS PER FOOT



GEOCON LTD

APPENDIX I  
FIGURE 18  
PROJECT-056014





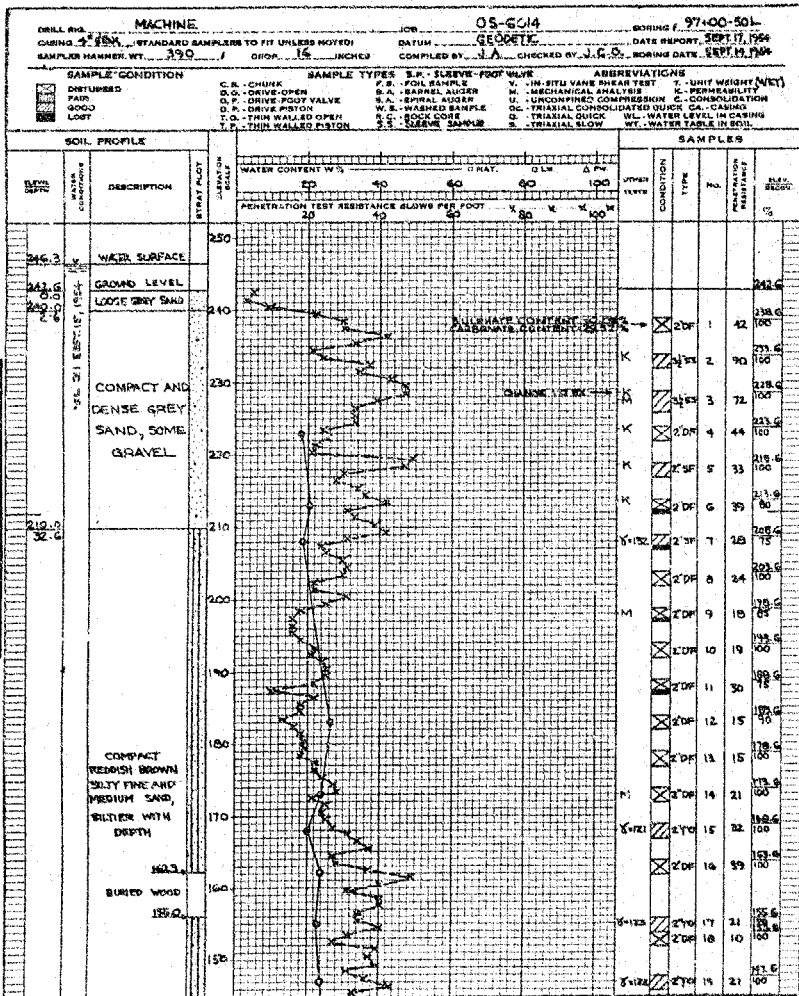
APPENDIX I  
FIGURE 19  
PROJECT-056014



# OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I  
FIGURE 20  
PROJECT-OS6014

GEOCON LTD



APPENDIX I  
FIGURE 21  
PROJECT-OS6014

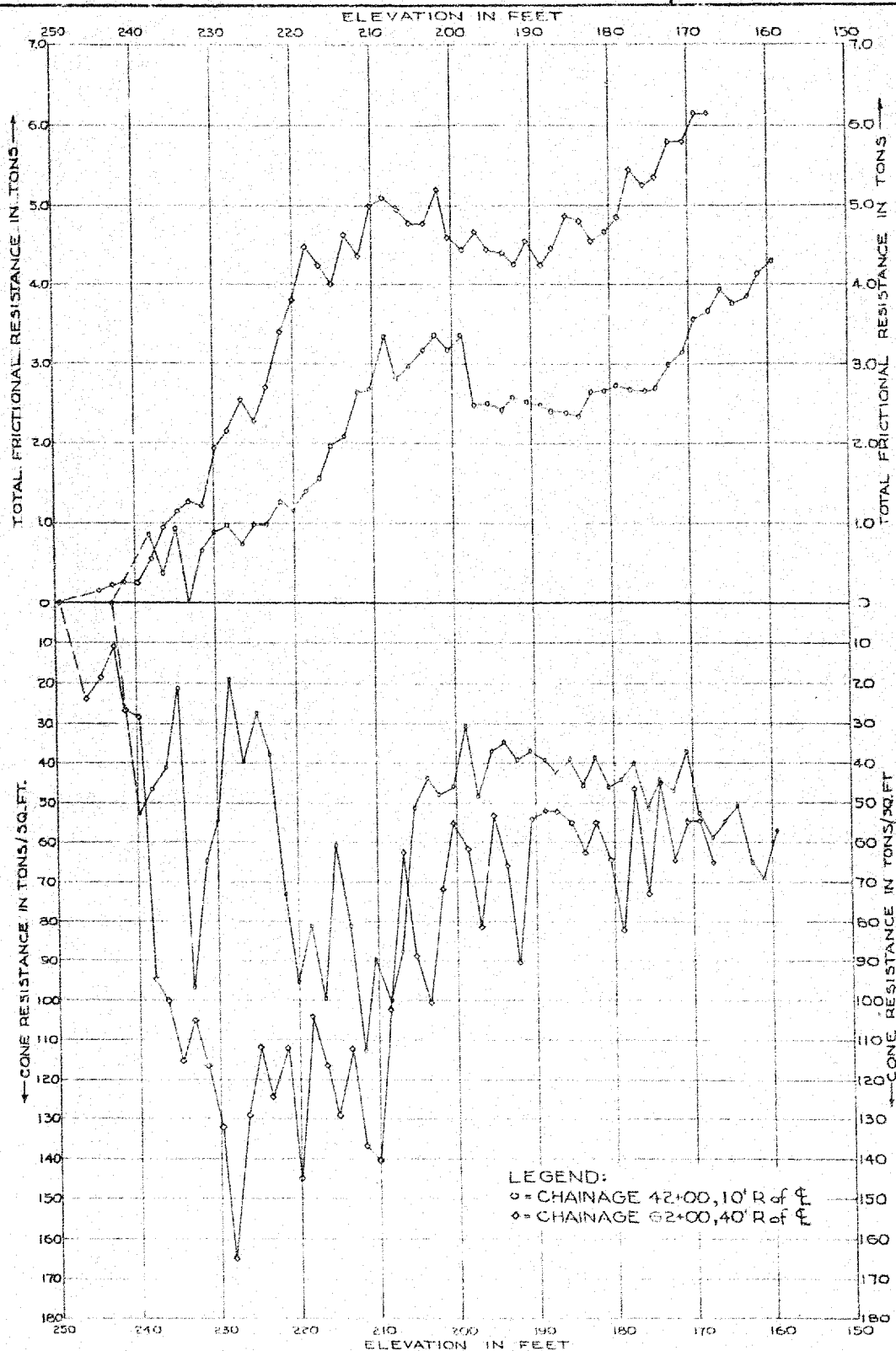


APPENDIX II

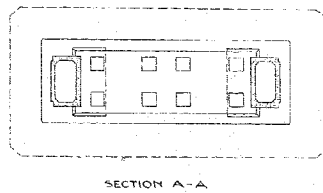
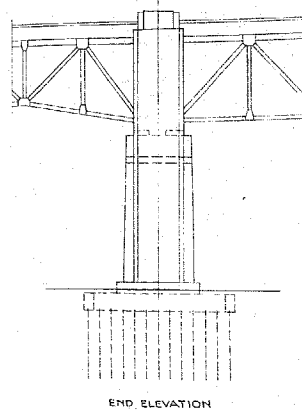
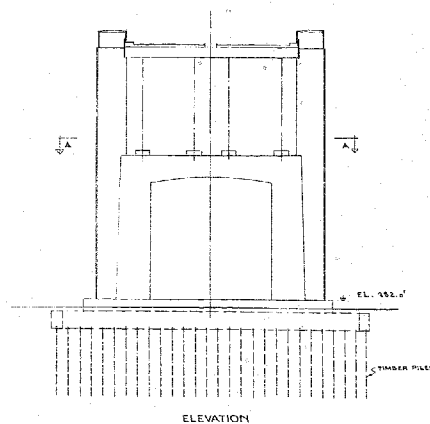
DUTCH CONE PENETROMETER RESULTS

# DUTCH CONE PENETROMETER RESULTS

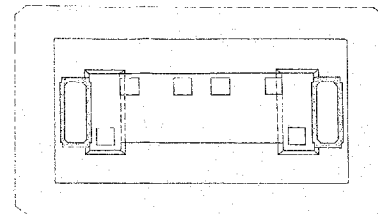
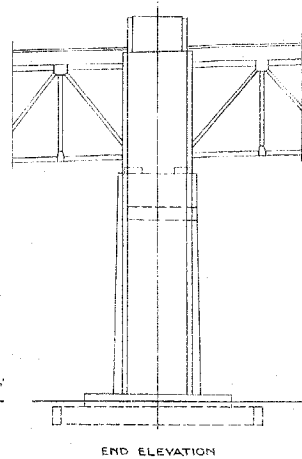
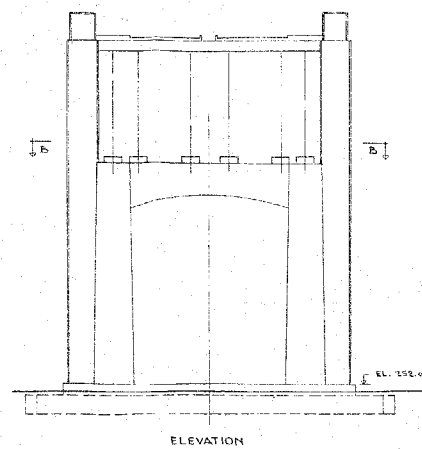
APPENDIX II  
FIGURE I  
PROJECT OS6014



GEOCON LTD



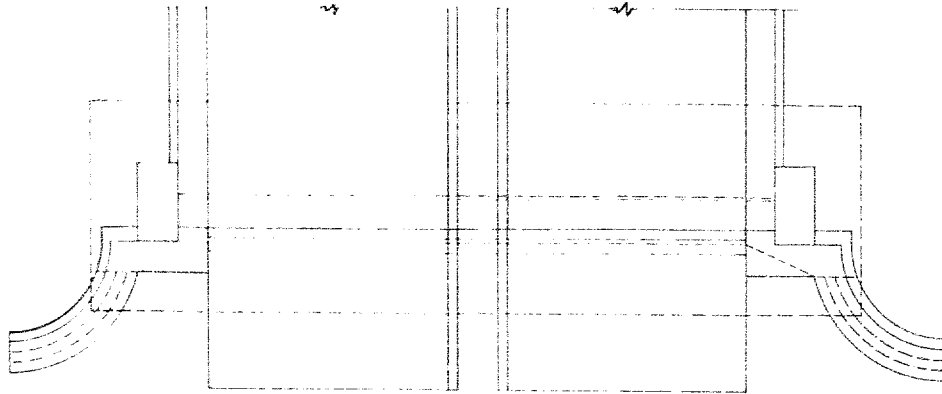
PIER No 7



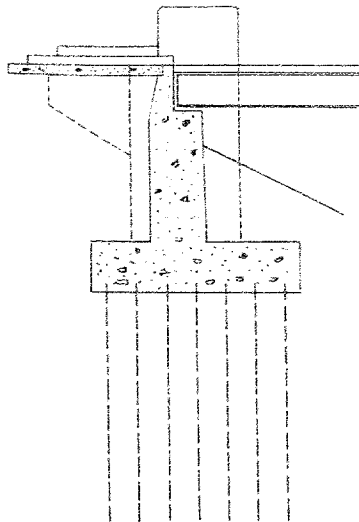
PIER No 2

0 10 20 30 40 50  
 SCALE IN FEET

ONTARIO DEPARTMENT OF HIGHWAYS BURLINGTON HIGH LEVEL BRIDGE	[FENCO] FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED	
	FEATURE PIERS No 2 & No 7	
	No. 1126 - T-110	



PLAN



SECTION

SCALE IN FEET

ONTARIO  
DEPARTMENT OF HIGHWAYS

BURLINGTON HIGH LEVEL  
BRIDGE

ABUTMENT

FENCO

FOUNDATION OF CANADA  
ENGINEERING CORPORATION  
LIMITED

No. 1126 - T - 111

REV. NO.