

G.I.-30 SEPT. 1976

GEOCRES No. 41P-25DIST. 53 REGION W.P. No. 136-88-00CONT. No. W. O. No. STR. SITE No. 47-019HWY. No. 112LOCATION Bridge over Stony
CreekNo of PAGES - =====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

Submitted To:

TOTTEN SIMS HUBICKI ASSOCIATES
300 Water Street
Whitby, Ontario L1N 9J2

W.P. 136-88-00

FOUNDATION INVESTIGATION REPORT

**PROPOSED REPLACEMENT OF THE EXISTING
HIGHWAY 112 BRIDGE OVER STONEY CREEK
DISTRICT 53 - NEW LISKEARD
SITE NO. 47-019**

Submitted By:

**AGRA Earth & Environmental Limited
104 Crockford Blvd.
Scarborough, Ontario M1R 3C6**

**November 1997
TT97-8-3**

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**November 1997
TT97-8-3**

November 12, 1997.

Ref. No. TT97-8-3

Totten Sims Hubicki Associates
300 Water Street
Whitby, Ontario
L1N 9J2

Attn: Mr. Marko Prgin, P. Eng.

Dear Sir:

**RE: PROPOSED REPLACEMENT OF THE EXISTING
HIGHWAY 112 BRIDGE OVER STONEY CREEK
DISTRICT 53 - NEW LISKEARD
SITE NO. 47-019**

We take pleasure in enclosing two (2) copies of our Geotechnical Investigation Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil and rock samples will be retained for a period of three months, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Yours very truly,

AGRA Earth & Environmental Limited



Z.S. Ozden, P. Eng.,
Principal Engineer.

ZSO/dee

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1.0 INTRODUCTION

AGRA Earth & Environmental Limited (AEE), Consulting Geotechnical Engineers, was retained by Totten Sims Hubicki Associates, Consulting Engineers, to conduct a foundation investigation at the site of the proposed replacement of the existing bridge over the Stoney Creek, Highway 112 (District 53 New Liskeard - W.P. 136-88-00, Site 47-019).

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of exploratory boreholes and, based on the findings, to make recommendations pertaining to the design of the foundations of the proposed structure from a geotechnical engineering viewpoint.

The site is located in the District of Timiskaming, about 20 km north of Englehart, on Highway 112, a short distance north of the intersection with Highway 11. (See Key Plan - Enclosure No. 1).

The fieldwork was carried out on August 28 and 29, 1997, and consisted of putting down four sampled boreholes (Boreholes 1 through 4) and four probeholes, at the locations shown on Enclosure No. 2. Details of subsurface conditions encountered at each borehole location, including results of in-situ testing, are presented on the Log of Borehole Sheets, Enclosure Nos. 3 through 6. Composite Logs for Probeholes (Boreholes 5, 6, 7 and 8) are given on Enclosure No. 7.

2.0 SITE AND GEOLOGY

The topography in the area is characterized by rolling hills with deeply eroded channels and exposed bedrock outcrops. The existing bridge at Stoney Creek lies at the bottom of two hills, over a deeply eroded creek channel. Bedrock is exposed along the creek at the bridge abutments. The creek is poorly drained, emptying from a swamp to the west. The existing bridge is a single span rigid frame concrete structure. It has a span of 13 metres and crosses Stoney Creek at an angle of about 65 degrees with the creek banks. It was built in 1935 and is in a poor condition.

The physiography of the area has been shaped by its geological past. Glaciers receded from the area about 10,000 year ago, leaving behind glaciolacustrine clay and silt deposits. Northerly from the site, along Highway 112 the overburden cover diminishes leaving exposed bedrock at the surface or covered by a discontinuous overburden layer. Based on published data, the bedrock in the area are igneous intrusive rocks of the Archean Age (i.e. greater than 2.5 billion years old). The bedrock exposed at the site is a pinkish granodiorite, a member of

.../...

the granite family, which is slightly metamorphosed.

3.0 SUBSURFACE CONDITIONS

Subsurface conditions at the site were explored at four borehole (Boreholes 1, 2, 3 and 4) and four probehole locations (Boreholes 5, 6, 7 and 8). In addition a dynamic cone penetration test was performed from the bottom of Borehole 1. The locations of the boreholes and the probeholes are shown on the Borehole Location Plan, Enclosure No. 2.

Details of the stratigraphy encountered in the boreholes are given on individual Borehole Log Sheets, Enclosure Nos. 3 through 6 and the probehole results are presented on the composite borehole logs on Enclosure No. 7.

Boreholes 2 and 3 were located closest to the existing bridge and were drilled from the surface of the paved road (Elev. 260.9 and 260.8). These boreholes contacted 0.2 m of asphaltic concrete at the surface. Boreholes 1 and 4 were drilled further away from the bridge, from the shoulders of the Highway (Elev. 261.1 and 261.3 m). Beneath the asphaltic concrete in Boreholes 2 and 3, and the ground surface in Boreholes 1 and 4, granular road fill was encountered to a depth of about 1.5 m. The measured 'N'-values in the granular road fill range from 27 to 44 blows/0.3 m, indicating the material was compacted.

Underlying the granular road fill, the boreholes contacted fill generally ranging from sand with clay pockets to silty clay to depths ranging between 5.2 and 7.2 m below the existing ground surface. The measured moisture contents of samples from the clay (or primarily clay) fill range from 9 to 31%. Atterberg Limits test performed on a sample from Borehole 1 gave a liquid limit of 28%, a plastic limit of 18%, with a corresponding plasticity index of 10%. The measured moisture contents of the same sample were 25 and 31%. Samples recovered from the primarily sand fill indicated moisture contents generally ranging from 7 to 14%. Standard penetration tests performed in the fill yielded 'N'-values of 2 to 14 blows/0.3 m indicating that the fill has not received a systematic compaction.

In Borehole 2 refusal to augering was encountered at 7.2 m therefore the borehole was further extended by diamond drilling (coring). Continuous concrete and rock cores (recovery 100%) were obtained to a depth of 9.7 m.

Borehole 1 was terminated in the fill at 6.9 m depth. A dynamic cone penetration test was extended from the bottom of the borehole from 6.9 to 11.0 m. The recorded values generally ranged from 22 to 36 blows/0.3 m.

.../...

In Borehole 4, underlying the fill, a 0.6 m thick layer of silty clay with some silt and fine sand was contacted. The material was identified as possible fill and from an 'N'-value of 15 blows its consistency is described as stiff. This material is underlain by fine sand, with some silt layers and occasional thin clay seams, to a depth of 7.6 m. Based on a recorded 'N'-value of 19 blows this material is described as compact. The borehole was terminated at 7.6 m upon encountering refusal to augering, probably on a boulder.

In Boreholes 2 and 3, which were extended by diamond drilling, a pink igneous granitic bedrock (described as granodiorite in published literature) was contacted at 9.7 and 6.1 m, respectively or at Elev. 251.2 and 254.7 m. The rock was cored for a vertical distance of 3.0 m. The percentage of total recovery (TCR)* ranged from 69 to 100%, and R.Q.D.** values from 25 to 77%. Based on these values the rock is described as a relatively sound rock of generally fair to good quality, with some fractured zones. The quality of the rock can be expected to improve with increasing depth below bedrock surface.

_____ *poor to good*

*TCR (Total Core Recovery) represents the total length of rock core recovered and is expressed as a percentage of the actual length of the core run (usually up to 1.5 m). Although the total core recovery is, to a certain extent, affected by the drilling method, nevertheless a high percentage of core recovery is indicative of a rock of good quality.

**R.Q.D. (Rock Quality Designation) value is obtained by measuring the total length of recovered rock core pieces which are longer than 100 mm (4 inches) and expressing their sum total as a percentage of the length of the core run. Breaks which are obviously caused by the drilling operation are disregarded.

The R.Q.D. values are an indicator of the quality and engineering properties of the rock. A relationship between rock quality and R.Q.D. indices, commonly used, is as follows:

R.Q.D. (%)	ROCK QUALITY
0 - 25	very poor
25 - 50	poor
50 - 75	fair
75 - 90	good
90 - 100	excellent

.../...

As shown on Enclosure No. 7, Boreholes 5 and 8 encountered refusal at ground surface on bedrock. Boreholes 6 and 7 encountered refusal at 0.2 to 0.3 m below the ground surface on shattered rock pieces.

All the boreholes were dry to depths ranging between 3.6 and 7.2 m below the ground surface upon their completion or prior to coring. In addition Borehole 1 remained dry to 6.9 m four hours after its completion. It is our opinion that the groundwater level would be largely controlled by the water level in the creek which was at Elev. $249.7 \pm$ m, or about 11.3 m from the bridge deck (the water in the creek was generally between 0.1 and 0.2 m deep).

4.0 DISCUSSION AND RECOMMENDATIONS

The proposed reinforced concrete culvert will replace the existing bridge which carries Highway 112 over Stoney Creek. As presently proposed, the culvert will be an approximately 50 m long and 5.2 m wide (clear width 4.0 m) rigid frame open concrete structure. The proposed structure is positioned at a 27 degree skew to the existing highway on the upstream side, and at 0 degree to the highway downstream from the bridge, beginning at about center line of the highway (See Enclosure. No. 8). We understand that it is proposed to raise the grade at the culvert location from the existing elevation by up to about 3.0 m.

This investigation has shown that, adjacent to the existing bridge (Elev. $261 \pm$ m) at Boreholes 2 and 3, the site is underlain by granular pavement fill to 1.5 m depth. The granular subbase course is underlain by mixed fill consisting of a mixture of fine sand with silt and clay, to silty clay, to depths of about 7 and 6 m, respectively (Elev. 253 and 255 m) in the two borings. The recorded 'N'-values indicate that while the granular road fill appears to be well compacted, the underlying mixed fill is uncompacted as evidenced by 'N'-values of 3 to 12 blows/0.3 m. Boreholes 1 and 4 drilled further away from the bridge along the highway indicated similar fill conditions.

Underlying the mixed fill, below about 7 m Borehole 2 contacted old concrete and rock, which extended to the surface of the pink granitic bedrock at about 10 m depth (Elev. 251 m). In Borehole 3 the surface of the bedrock was contacted at about 6 m or at about Elev. 255 m.

Beneath the centre of the bridge the creek bed elevation is at about Elev. 250 m and the bedrock is exposed on both sides of the creek.

.../...

Upstream from the existing bridge the bedrock is exposed on both sides (e.g. Borehole 8, Elev. 250 m). On the downstream side the bedrock is visible for a short distance beyond the existing bridge and is also exposed at Borehole 5 location at Elev. 248.6 m. Beyond this point however the subsurface conditions are not known.

4.1 Foundations

All footings may be founded on sound bedrock. For this purpose all loose or shattered rock under the footprint of the footing should be removed and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary.

For design purposes the following O.H.B.D.C. bearing resistance may be used:

Factored Bearing Resistance at U.L.S. = 2,000 kPa
Bearing Resistance at SLS will not govern.

At all footing locations the surface of the bedrock should be approximately level (e.g. no steeper than 10H:1V); where possible, sloping down towards the creek should not be allowed.

In choosing footing elevations, frost, scour, sliding resistance and safe distance from the face of rock cuts or rock ledges will have to be considered.

Bedrock would be prone to deterioration due to the opening of existing joints or fractures in shallow bedrock as a result of frost action. Provided that the surface water is diverted away from the footings, frost protection need not be provided for footings placed on massive, sound bedrock, although for added protection an earth cover of at least 0.3 m is recommended. The surface of the earth protection should be clayey to minimize the infiltration of surface water or the protection could be provided by concrete. If however the bedrock is not massive and water can accumulate in the joints or fractures of the rock (thus causing deterioration of the founding medium by expansion due to freezing) then full frost protection (i.e. 2.4 m) should be provided. For this purpose the surface of the proposed bearing surface can be inspected by qualified geotechnical personnel. If the rock is not massive then the excavation can be extended deeper until acceptable rock is found or to the full frost protection depth of 2.4 m, whichever comes first.

In order to ensure stability, the footings should be located at a safe distance back from the face of rock or rock cuts, outside an imaginary plane defined by a 45 degree line from the toe of the rock. Depending on the orientation of joints and fractures, etc., rock bolting of the rock face may also be required.

.../...

Sliding resistance can be provided by penetrating into the bedrock (i.e. keying-in and utilizing passive rock resistance), utilizing the sliding resistance between concrete and bedrock, shear in grouted dowels and/or rock anchors. For the evaluation of the sliding resistance of the foundation (O.H.B.D.C. 6-8.4.3) the ultimate angle of friction between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) can be taken as 30 degrees. If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond capacity at ULS can be taken as 500 kPa and SLS will not govern. The minimum anchor embedment depth should be 2.5 m into sound bedrock. The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

All foundations should be backfilled and drained in accordance with OHBDC requirements.

4.2 Backfilling

Where applicable backfilling around the structure should conform to OPSD-803.02 and should consist of suitable materials, compacted in accordance with the MTO Standards. For fill below groundwater elevation or below the roadway, it is recommended that Granular 'A' or 'B' aggregates be used. The backfilling operation should be carried out simultaneously on both sides of the culvert, as per MTO specifications. Compactable select fill soils can be used as backfill up to a depth of 2.4 m below design grade (i.e. to frost line, as per OPSD-803.02). This fill should be compacted in lifts of not more than 150 mm loose thickness to at least 98% Standard Proctor maximum dry density (SPMDD). Above this level, backfill should consist of free draining OPSS Granular 'A' or 'B' sand and gravel compacted to 100% SPMDD. Where applicable the Granular 'A' or 'B' should extend to a depth of 2.4 m below finished road level and be tapered back from the culvert walls at 10 horizontal in 1 vertical to meet the granular road base, consistent with MTO Standards. Care should be exercised when compacting fill adjacent to the culvert structure and compaction equipment should be restricted in size as per MTO procedures.

Backfilling behind any retaining (wing) walls should consist of granular materials in accordance with the Ontario Ministry of Transportation Standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic pressure build-up.

.../...

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with the Ontario Highway Bridge Design Code, 3rd Edition (1991). For design purposes, the following properties can be assumed:

Compacted Granular 'A'

Angle of Internal Friction $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:IV	Backfill Sloping at 2H:IV
$K_a = 0.27$	$K_a = 0.34$	$K_a = 0.40$
$K_b = 0.35$	$K_b = 0.44$	$K_b = 0.50$
$K_o = 0.43$	$K_o = 0.56$	$K_o = 0.62$
$K^* = 0.45$	$K^* = 0.60$	$K^* = 0.66$

Compacted Granular 'B'

Angle of Internal Friction $\phi = 30^\circ$ (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:IV	Backfill Sloping at 2H:IV
$K_a = 0.33$	$K_a = 0.42$	$K_a = 0.54$
$K_b = 0.41$	$K_b = 0.52$	$K_b = 0.64$
$K_o = 0.50$	$K_o = 0.66$	$K_o = 0.76$
$K^* = 0.57$	$K^* = 0.74$	$K^* = 0.86$

Rock Fill

Angle of Internal Friction $\phi = 35^\circ$ (unfactored)

Unit Weight = 18 kN/m^3

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:IV	Backfill Sloping at 2H:IV
$K_a = 0.27$	$K_a = 0.34$	$K_a = 0.40$
$K_b = 0.35$	$K_b = 0.44$	$K_b = 0.50$
$K_o = 0.43$	$K_o = 0.56$	$K_o = 0.62$
$K^* = 0.45$	$K^* = 0.60$	$K^* = 0.66$

.../...

NOTE: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction effects.

K_o is the coefficient of earth pressure at rest

K^* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients.

Foundations on bedrock will be unyielding and the structure will be of the "rigid frame" type hence the at-rest condition will govern the earth pressure.

The use of vibratory compaction equipment behind the culvert and the retaining walls should be restricted in size as per current MTO practice. If rock fill is used, special care will be required in order to prevent damage to the retaining structures. In such a case, a cushion of Granular 'A' material or relatively finely graded rock fill (e.g. less than 250 mm nominal diameter) should be placed between the structure and the rock fill. This cushion should be at least 0.45 m wide and if Granular 'A' is used, proper filtering should be provided to prevent the loss of finer particles from the Granular 'A' cushion into the coarse rock fill.

4.3 Construction Comments

All excavations should be carried out in accordance with the Occupational Health & Safety Acts of Ontario, including regulations for Construction Projects and Regulations for Mines and Mining Plants (in bedrock).

Excavation of the bedrock by mechanical methods such as hoe-ramming or ripping will probably only be feasible within the uppermost weathered and fractured rock. In our opinion, in most cases blasting will be necessary. Blasting should however be carried out in a manner to minimize damage to the founding bedrock. Where blasting is required, controlled perimeter

.../...

(line) blasting is recommended in order to provide a neat excavation line and to minimize over break. In any event, by means of good construction techniques the integrity of the existing bridge should be maintained.

Temporary rock slopes should be stable at near vertical faces, but adequate precautions should be taken to protect the workers from spalling rock. The stability of rock faces will also depend on the inclination of planes of weakness in the rock mass.

If seepage into the excavation is encountered, dewatering can be achieved by pumping from temporary pumps, as necessary.

All rock excavations and footing bases should be inspected and evaluated by a qualified geotechnical engineer familiar with the findings of this report.

4.4 Approach Fills

Where supported on bedrock, the design of the approach fills will not be limited by the strength of the foundation material and therefore in such cases no instability problems are foreseen due to foundation conditions. Borehole 4, showed underlying the fill, the presence of a thin layer of stiff silty clay ($N' = 15$) followed by compact ($N' = 19$) fine sand. In this case too no instability problems are anticipated. If however weak clays or organic soils are present overlying the bedrock, problems could occur both with regards to stability and settlement. This aspect should be confirmed during the construction, and all organic or otherwise unsuitable (i.e. weak) soils should be removed within the footprint of the proposed embankment before placing the fill, as per MTO specifications.

After stripping, the exposed subgrade should be inspected, evaluated and, where necessary, properly compacted from the surface in the presence of geotechnical personnel.

At the borehole locations, the design of safe slopes will be governed by the characteristics of the embankment fill: 2 (H) to 1 (V) slopes should be stable for properly compacted clean earth fills, or 1.25 to 1.5 (H) to 1 (V) slopes for rock fill. For embankment fills of 6 m or more in height, however, a 2 \pm m wide berm should be provided at approximately mid height of the embankment in order to control drainage and to improve surficial stability. Proper erosion control measures should be implemented.

.../...

Settlements of compacted fill embankment can be expected to be minor. Somewhat more settlements may occur within a rock fill approach. This settlement could be reduced, if desired, by surcharging the rock fill. In this instance, for embankment heights of up to 10 m an additional 1.5 to 2 m rock fill and a surcharge period of about several days would suffice.

In case of rock fill a geotextile separator should be provided between the granular road base and the rock fill to prevent the loss of granular soils into the core of rock fill.

Where the existing embankment will be left in place but will be heightened and widened, similar construction procedures should be followed, including the removal of unsuitable materials from the toe area. In this case, however, proper benching of the existing embankment should be applied as per MTO procedures. In case of rock fill, the bench width and height should be at least 1.2 m but may need to be increased depending on the rock sizes used for building up and widening the existing embankment. In case of rock fill, if the embankment widening is more than 2.5 m, 1.5H:IV side slopes should suffice (with an approximately 2 m wide mid-height berm for embankment heights of 6 m or more).

Another consideration is the fact that the existing embankment fills appear to be uncompacted below 1.5 m depth and this should be taken into account. For this reason and especially where the foundation soils are not competent, we recommend that the embankment platform be overbuilt (i.e. widened). Preloading can also be considered, if construction and timing constraints will permit this approach.


When using dissimilar materials (e.g. rock fill over existing frost-susceptible fill) for the construction of the embankment, differential frost effects should be considered (e.g. frost taper as per MTO specifications may be necessary).

5.0 CLOSURE

The Limitations of Report, as quoted in the Appendix, are an integral part of this report.


Respectfully submitted,

AGRA Earth & Environmental Limited


Z.S. Ozden, P. Eng.
ZSO/dee

.../...




L.S. Rolko, P. Eng.



AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

APPENDICES

APPENDIX A

AGRA Earth & Environmental Limited

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AGRA Earth & Environmental Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.



APPENDIX B

APPENDIX 'B'

PROCEDURES

The fieldwork for this project was performed on August 28 and 29, 1997 and consisted of drilling and sampling eight boreholes at the locations shown on the Borehole Location Plan, Enclosure No. 2.

Boreholes 1 through 4 were extended to depths ranging between 7.6 and 12.6 m below the ground surface. In Boreholes 5 through 8, refusal to augering was encountered at or near the ground surface on bedrock or shattered rock. The boreholes were put down using a truck mounted drilling rig owned and operated by Boart Longyear Limited. In Boreholes 1 through 4 sampling in overburden was effected at frequent intervals of depth by the Standard Penetration Test (S.P.T.) Method. As specified in ASTM Method D1586, this test consists of freely dropping a 63.5 kg (140-lb.) hammer a vertical distance of 0.76 m (30 inches) to drive a 51 mm (2-inch) diameter o.d. split-barrel (split-spoon) sampler into the ground at the sampling depth. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.305 (1-ft.) is recorded as the Standard Penetration Resistance or the 'N'-value of the soil and this gives an indication of the compactness condition or the consistency of the soil deposit.

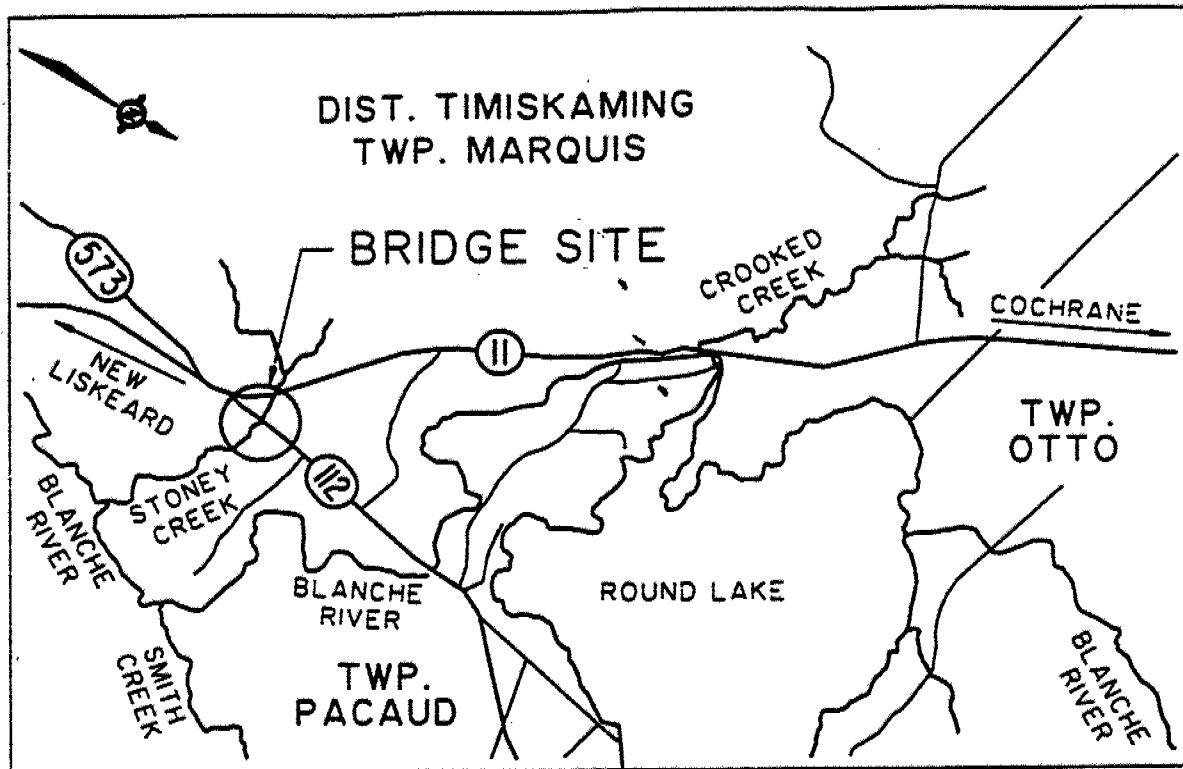
From the bottom of Borehole 1 a dynamic cone penetration test (DCPT) was performed. This test consists of driving a 51 mm (2-inch) diameter, 60 degree apex steel cone, attached to an 'A'-size (41.3 mm or 1 1/2-inch diameter) drill rod into the undisturbed ground without casing and by applying the same energy as in the SPT described earlier. The number of blows to advance the cone each 0.305 m (1-ft.) is counted and recorded and the result of the test is a continuous record of the driving resistance which indicates variations in the relative density or consistency of the subsurface overburden deposits.

In Boreholes 2 and 3, after encountering refusal on the augers, coring was effected by diamond drilling methods, using an NXL size core barrel.

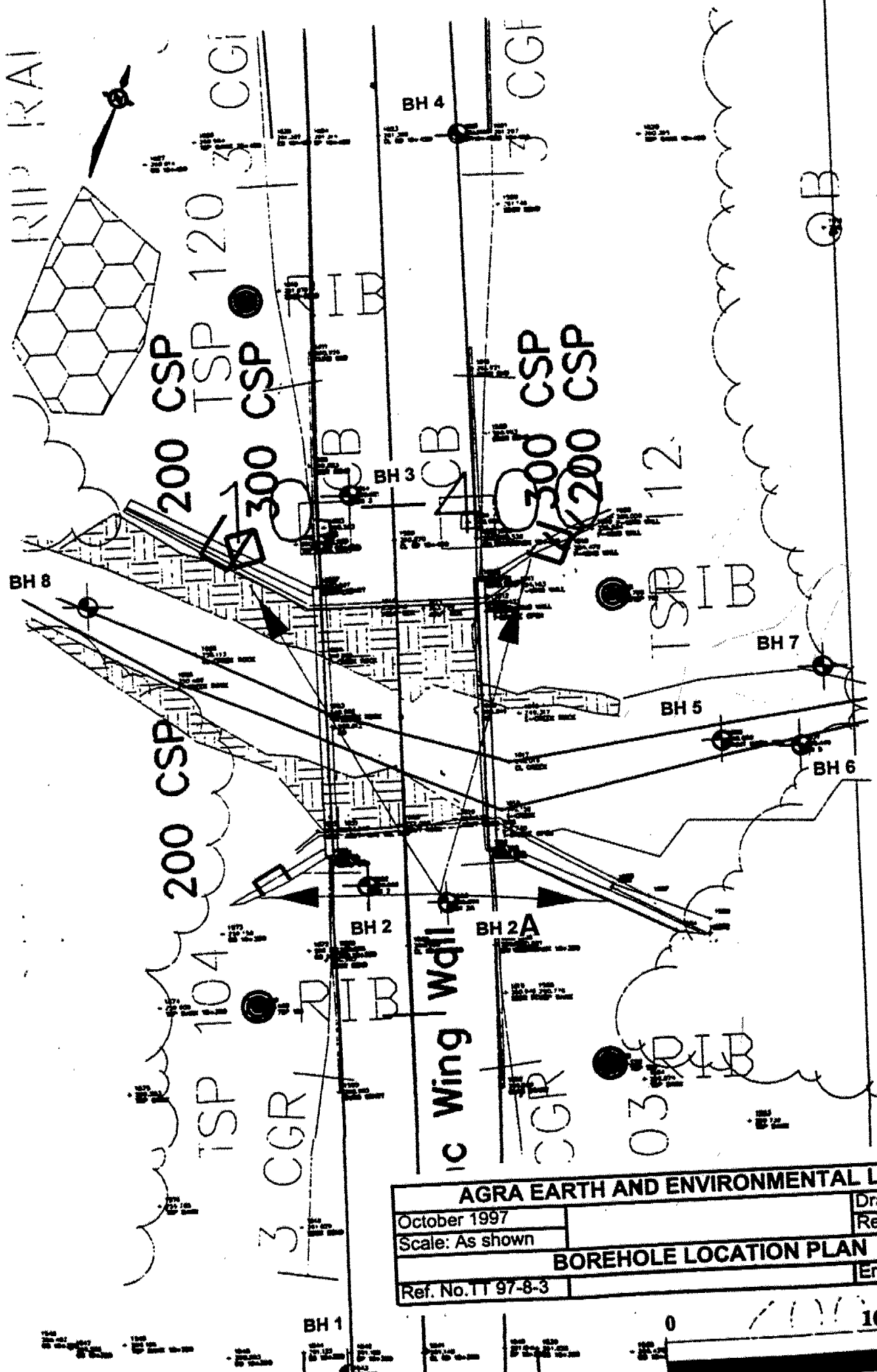
The fieldwork was performed under the supervision of a Professional Engineer from our office who was responsible for the layout of the holes, directing drilling and field tests, recording results and taking water levels. Upon completion of the fieldwork the soil and rock samples were shipped to our laboratory in Scarborough where they were further examined and classified. A laboratory testing programme consisting of natural moisture content and Atterberg Limits tests was performed on selected soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets.

The borehole locations and their elevations (except for Boreholes 7 and 8) were determined and provided to us by Totten Sims Hubicki Associates.

ENCLOSURES



AGRA EARTH AND ENVIRONMENTAL LIMITED		
October 1997		Drawn By:
Scale: NTS		Reviewed By:
KEY PLAN		
Ref. No. TT 97-8-3		Enclosure No. 1



AGRA EARTH AND ENVIRONMENTAL LIMITED	
October 1997	Drawn By:
Scale: As shown	Reviewed By:
BOREHOLE LOCATION PLAN	
Ref. No. TT 97-8-3	Enclosure No. 2



LOG OF BOREHOLE 1

ENCL. No.: 3

REF. No.:	TT97-8-3	DRILLING DATA
CLIENT:	Totten Sims Hubicki Associates	
PROJECT NAME:	Bridge Replacement over Stoney Creek	Method: HolSt Augering
LOCATION	Highway 112, Ontario	Diameter: 250 mm
DATUM:	Geodetic	Date: August 28th, 1997

LABORATORY DATA				SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS					
PL	W	LL	DCPT	Su	No.	TYPE							N- Value				
%	%	%	blows/ 0.3m	Field Vane(kPa)													
GROUND SURFACE EL. 261.1 m																	
18	25	28							0.2m Sand & Gravel (FILL)	261		Borehole drilled from shoulder of the highway Borehole dry on completion and 4 hours thereafter					
									1	SS			30	Gravelly Sand (FILL)	260	1	
									2	SS			14	Sand with Clay pockets (FILL)	259	2	
									3	SS			3	Silty Clay (FILL)	258	3	
									43	4			TW		-	257	4
									> 110	5			SS		2	256	5
									> 110	6			SS		3	255	6
										7			SS		6	254	7
									8	SS			10	253	8		
									> 110						252	9	
															251	10	
				250	11												
								END OF BOREHOLE @ 6.9m			DYNAMIC CONE PENETRATION TEST PERFORMED FROM 6.6m TO 11.0m (FROM BOTTOM OF BOREHOLE)						
											DCPT: Dynamic Cone Penetration Test						

Vertical Scale: 1:80



AGRA Earth & Environmental Limited

Checked: ZSO

SHEET 1 OF 1 HOLE No. 1

LOG OF BOREHOLE 2

REF. No.: TT97-8-3		ENCL. No.:	
CLIENT:	Totten Sims Hubicki Associates	DRILLING DATA	
PROJECT NAME:	Bridge Replacement over Stoney Creek		
LOCATION	Highway 112, Ontario	Method:	HolSt Augering
DATUM:	Geodetic	Diameter:	250 mm
		Date:	August 28th, 1997

[illegible]

Vertical Scale: 1:80

**AGRA Earth & Environmental Limited**


Checked: ZSO

SHEET 1 OF 1 BH No. 2

LOG OF BOREHOLE 3

ENCL. No.: 5

REF. No.:	TT97-8-3	DRILLING DATA
CLIENT:	Totten Sims Hubicki Associates	
PROJECT NAME:	Bridge Replacement over Stoney Creek	Method: HolSt Augering
LOCATION	Highway 112, Ontario	Diameter: 250 mm
DATUM:	Geodetic	Date: August 29th, 1997

LABORATORY DATA					SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS	
PL %	W %	LL %	UNIT WT kN/m3	Su		No.	TYPE							N- Value
				Field Vane kPa	Lab. Compr kPa									
GROUND SURFACE EL. 260.8 m														
						1	SS	44		0.2m Asphalt 0.15m Granular Base Course Gravelly Sand (FILL)	260	1	Borehole dry prior to coring @ 6.1m	
10						2	SS	8		brown/grey Silty Clay, some sand layers (FILL)	259	2		
13						3	SS	3		brown, moist Sand with silty clay layers some clayey silt & clay lenses (FILL)	258	3		
14						4	SS	6			257	4		
18						5	SS	3			256	5		
18						6	SS	5		brown/grey Clay & Silty Clay (FILL)	255	6		
18						7	TW	-		grey Sandy Silt, clayey traces of organics & gravel (FILL)	254	7		
						8	RC	REC 100% RQD 60%	Pink IGNEOUS BEDROCK (Granitic)	253	8			
						9	RC	REC 100% RQD 77%		252	9			
										END OF BOREHOLE				

Vertical Scale: 1:80



AGRA Earth & Environmental Limited

Checked: ZSO

SHEET 1 OF 1 BH No. 3

LOG OF BOREHOLE 4

ENCL. No.: 6

REF. No.:	TT97-8-3	DRILLING DATA
CLIENT:	Totten Sims Hubicki Associates	
PROJECT NAME:	Bridge Replacement over Stoney Creek	Method: HolSt Augering
LOCATION	Highway 112, Ontario	Diameter: 250 mm
DATUM:	Geodetic	Date: August 29th, 1997

LABORATORY DATA						SAMPLES				SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS	
PL %	w %	LL %	UNIT WT kN/m ³	Su		No.	TYPE	N- Value								
				Field Vane kPa	Lab. Compr kPa											
GROUND SURFACE EL. 261.3 m																
										0.2m Granular Base Course	261				Borehole dry and caved-in @ 3.6m upon completion	
						1	SS	27		Gravelly Sand (FILL)		1				
14						2	SS	8		brown Silty Clay, trace of gravel (FILL)	260	2				
14						3	SS	5		brown Sand (FILL), moist	259	3				
17						4	SS	4		grey Silty Clay traces of organics (FILL)	258	4				
18						5	SS	13			257	5				
17						6	SS	10				6				
28						7	SS	15			brown Silty Clay, some silt & fine sand seams, wet (Possible FILL)	256	7			
22						8	SS	19		brown, compact, wet FINE SAND with some silt layers occ. clay seams	255					
											254					
						9	SS100/N.P.			END OF BOREHOLE AUGER REFUSAL Probably on a boulder					N.P. = no penetration	

Vertical Scale: 1:80



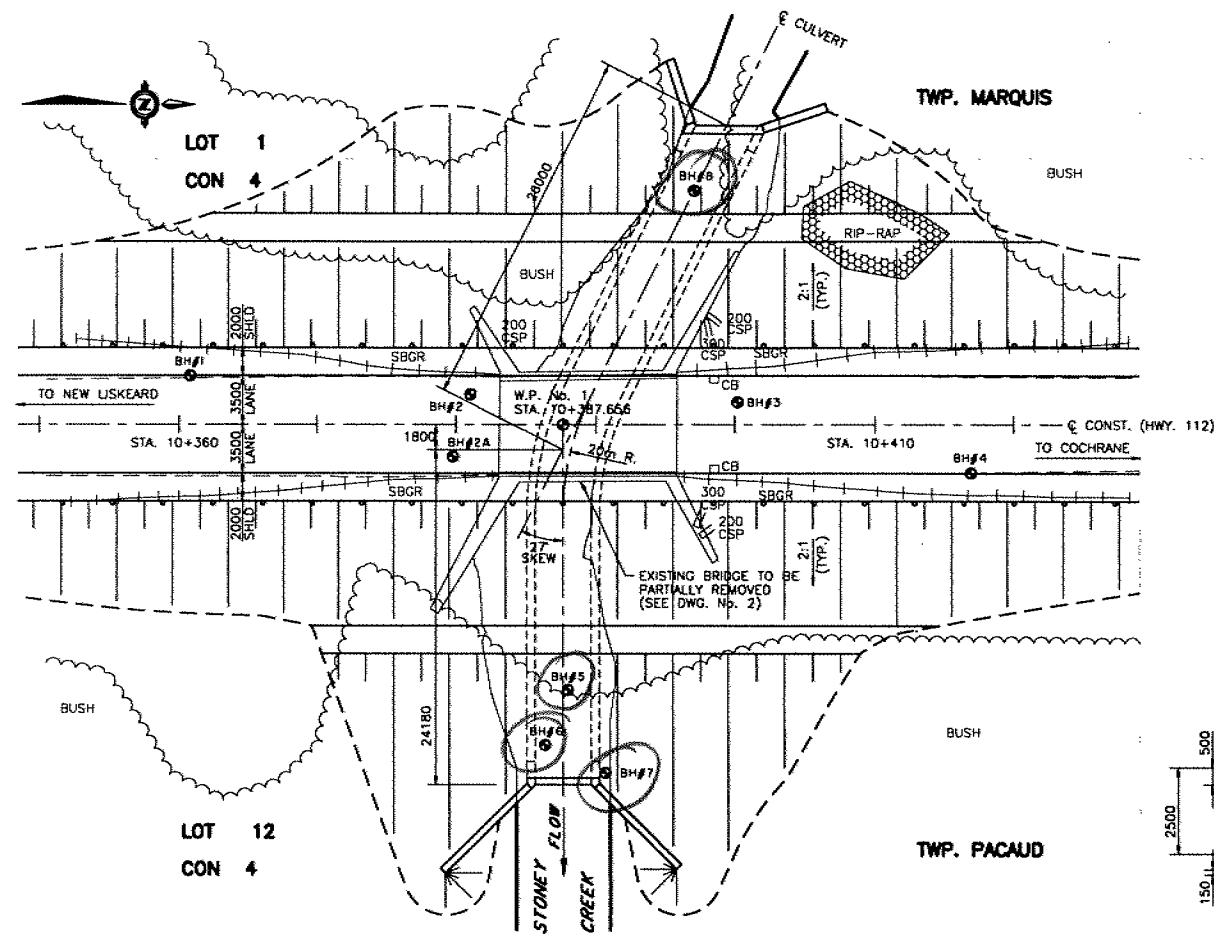
AGRA Earth & Environmental Limited

Checked: ZSO

SHEET 1 OF 1 BH No. 4

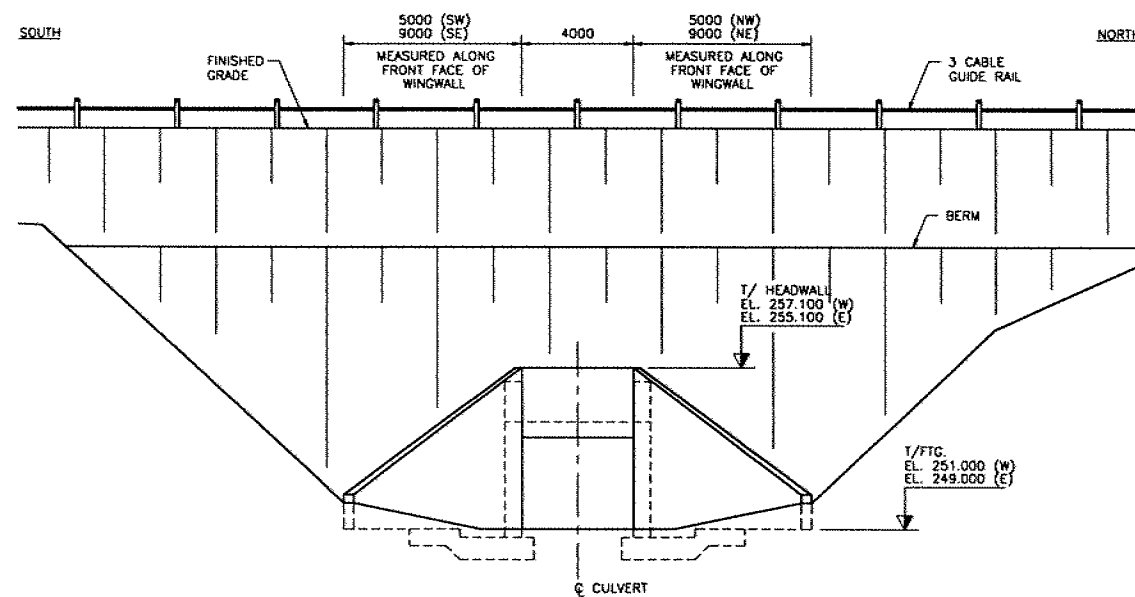
Composite Borehole Logs for Boreholes 5, 6, 7 & 8

Borehole No.5	Elev. 248.6m Exposed bedrock @ ground surface
Borehole No.6	Elev. 248.5m Probed 2x2m area 0.2 - 0.3m sand then refusal on Shattered rock
Borehole No.7	Elev. approximately 248.5m Probed 2x2m area 0.08m topsoil 0.08 - 0.3m sand Refusal @ 0.3m on boulders and shattered rock
Borehole No. 8	Elev. approximately 250.0m Exposed bedrock @ ground surface



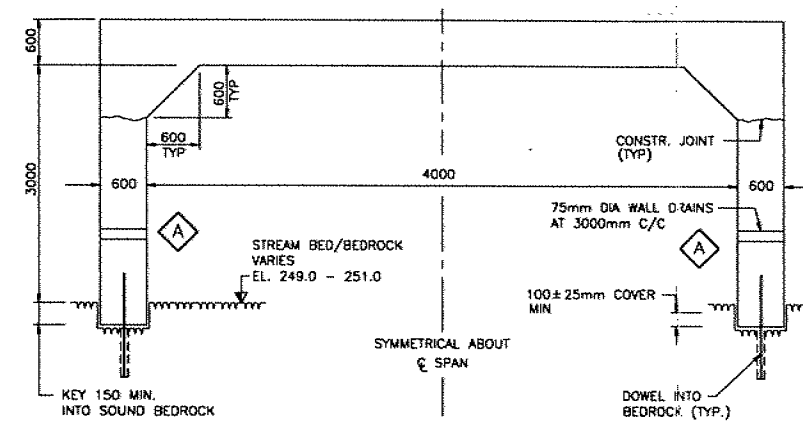
NOTES
W.P. - DENOTES WORKING POINT
● - DENOTES APPROXIMATE LOCATION OF BOREHOLE

PLAN
SCALE 1:250

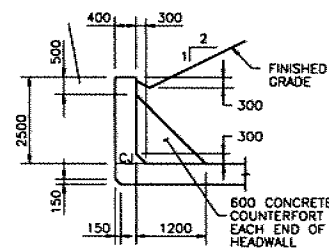


ELEVATION
SCALE 1:125

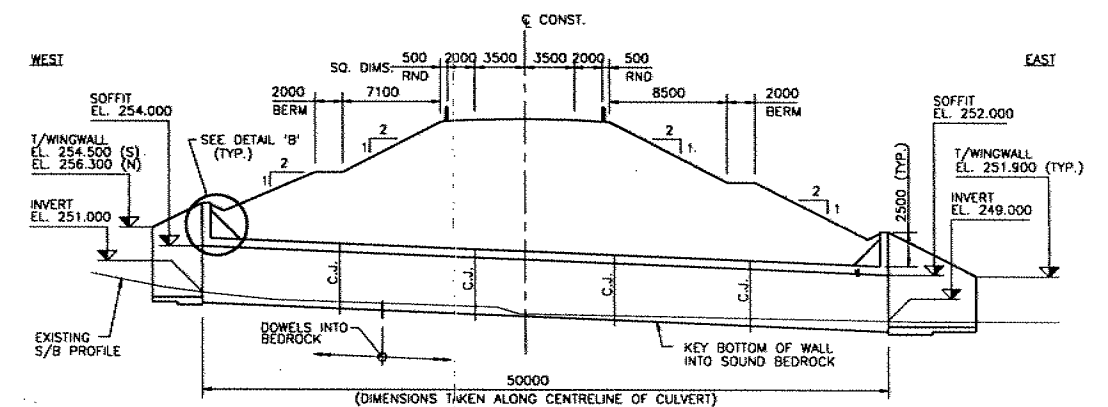
NOTE - FAR FACE OF FOOTINGS NOT SHOWN FOR CLARITY



TYPICAL CULVERT SECTION
N.T.S.

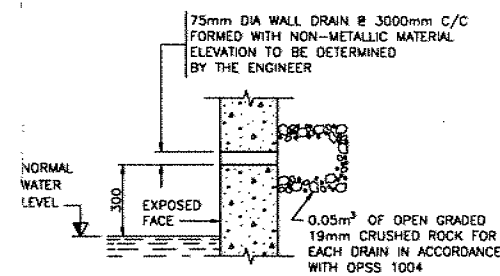


DETAIL 'B'
SCALE 1:100



LONGITUDINAL SECTION
SCALE 1:250

APPLICABLE STANDARD DRAWINGS
OPSD-4670.000 TYPICAL JOINT DETAILS
OPSD-803.02 GRANULAR BACKFILL



A

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DISTRICT No. 53
CONT No
WP No. 136-88-00



STONE CREEK
OPEN FOOTING CULVERT

SHEET

totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS

GENERAL NOTES

- 1 CLASS OF CONCRETE TO BE 30MPa
- 2 CLEAR COVER TO REINFORCING STEEL
BOTTOM OF TOP SLAB 40 ± 10 FOR SLABS ≤ 300 THK.
50 ± 10 FOR SLABS > 300 THK.
BOTTOM OF FOOTINGS 100 ± 25
REMAINDER 70 ± 20 UNLESS OTHERWISE NOTED
- 3 REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BARS MARKED WITH SUFFIX C DENOTE COATED BARS.
- 4 LEGEND
ALT DENOTES ALTERNATE
IF DENOTES INSIDE FACE
OF DENOTES OUTSIDE FACE
EF DENOTES EACH FACE

CONSTRUCTION NOTES

- 1 BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH SIDES OF CULVERT KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
- 2 SIDES OF FOOTING TO BE CAST AGAINST UNDISTURBED SOIL.
- 3 NO CONCRETE SHALL BE PLACED FOR ANY FOOTINGS UNTIL THE DEPTH OF THE EXCAVATION AND THE CHARACTER OF THE FOUNDATION HAVE BEEN APPROVED BY THE ENGINEER.
- 4 SITE No. AND DATE FIGURES SUPPLIED BY MTO.
- 5 MAXIMUM ALLOWABLE BEARING CAPACITY = 2000 KPa ULS AS PER REPORT ON SOIL CONDITIONS BY AGRA E & E.

STANDARD DRAWING
JUNE 1993
SS114-1
RIGID FRAME OPEN FOOTING CULVERT (MOD.)

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	S.D.	CHK	M.P.
DRAWN	DAC	CHK	S.E.L.
			SITE
			47-019
			STRUCT
			SCHEME
			DWG
			PI