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AUGUST 12, 1999

HIGHWAY 403 AT MOHAWK ROAD / LINCOLN ALEXANDER PARKWAY INTERCHANGE PRELIMINARY REPORT – CROSSING STRUCTURES CULVERTS STANTEC CONSULTING LIMITED PROJECT NO. 650-00146

PREPARED FOR:

REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH
SPECIAL PROJECTS OFFICE

Soil-Mat

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PROJECT NO.: SM 97027-GB

August 12, 1999

Regional Municipality of Hamilton-Wentworth
Special Projects Office
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Attention: Mr. Gary Moore, P. Eng.

RE: HIGHWAY 403 AT MOHAWK ROAD / LINCOLN ALEXANDER PARKWAY INTERCHANGE
PRELIMINARY REPORT - CROSSING STRUCTURES AND CULVERTS
REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH
STANTEC CONSULTING LIMITED PROJECT NO. 650-00146

Dear Mr. Moore:

Submitted herein is our preliminary report for geotechnical engineering services on the above project. We have completed the fieldwork and laboratory testing in connection with the above noted project and trust that our preliminary report meets with your requirements. Our comments and recommendations, based on our findings at the [17] seventeen borehole locations, are presented in the following paragraphs.

1.0 INTRODUCTION

Soil-Mat Engineers & Consultants Ltd. were retained by the Regional Municipality of Hamilton-Wentworth [Special Projects Office] as geotechnical consultants for the Highway 403 at Mohawk Road / Lincoln Alexander Parkway Interchange project. This report presents the results of our geotechnical investigation for this project. The work was performed in general compliance with our discussions, and your authorization to proceed under Purchase No. P.O. 78707. The structural/highway consultants for this project are Stantec Consulting Ltd.

1.1 Location

The proposed interchange site is at the west end of the Lincoln Alexander Parkway [*Linc*], where it connects to Highway #403 at Mohawk Road in Ancaster, Ontario.

1.2 Project Description

The proposed project will include the construction of four [4] crossing structures [bridges] with associated entrance/exit ramps and earth retaining structures, and four [4] culvert structures. The various structures have been labeled as follows:

TABLE 1.1 STRUCTURE DESIGNATIONS AND LOCATIONS

Designation	Description	Approximate Location
Bridge No. 1	Ramp E-S over Hwy. N0. 403	Ramp E-S @ Sta. 10+425 to 10+500
Bridge No. 2	Ramp N-E over Hwy. N0. 403	Ramp N-E @ Sta. 10+750 to 10+825
Bridge No. 3	Mohawk Road over Ramp N-E	Ramp N-E @ Sta. 10+550 to 10+575
Bridge No. 4	Ramp E-S over Mohawk Road	Ramp E-S @ Sta. 10+675 to 10+825
Culvert No. 1	Crosses Ramp N-W	Ramp N-W @ Sta. 10+125
Culvert No. 2	Crosses Ramp E-W & W-N	Ramp E-W @ Sta. 10+275
Culvert No. 3	Crosses Ramp E-S & N-E	Ramp E-S @ Sta. 10+775
Culvert No. 4	Crosses Ramp N-E	Ramp N-E @ Sta. 10+850

This report summarizes our fieldwork, comments on the subsurface conditions, and provides geotechnical engineering information concerning the preliminary design of the proposed structures.

1.3 Geology

A review of the published geological information indicates that the bedrock in the area is overlain by deposits of Halton Till [clay and/or silt till] which were apparently deposited at the time of the last glaciation, about 12,000 to 13,000 years ago. The Ministry of Natural Resources publication entitled "Limestone Industries of Ontario" indicates that the bedrock consists of dolostone of the Guelph and Lockport formations.

The exposed face of the existing road cuts in the vicinity of the site consists of dolostone bedrock of the Guelph Formation. This rock is light brown-grey, very fine crystalline, and medium-bedded. The Guelph Formation dolostone is underlain by a medium brown, very fine crystalline, medium to thick bedded dolostone, which is the upper part of the Eramosa Member of the Lockport Formation. Thin bedded laminations and bituminous banding is common in the Eramosa Member dolostone, and vugs filled in part with gypsum, calcite or chert are common in the lower part of this member.

There is a potential for high horizontal stresses in the bedrock in this general area, which can result in rock trench squeezing, local natural pop-up, and small earthquakes. However, do to the proximity of the site to the escarpment and the relatively shallow excavations anticipated, high horizontal stresses in the bedrock are unlikely at the project site.

The dolomites forming the 'cap rock' of the Niagara Escarpment have been documented as being susceptible to the development of solution channels/cavities, Pluhar and Ford [1970].

1.4 Site Description

The proposed interchange is located at the site of the existing Highway #403/Mohawk Road Interchange and consists primarily of paved roads and ramps, grassed areas and rock cuts and outcrops. There is a significant grade change from north to south as Highway #403 climbs up the Niagara escarpment from the Dundas Valley, and the site is near the top of this transition. There are a number of significant slopes/embankments in the project area associated with the natural topography and the existing ramps.

2.0 PROCEDURES

The fieldwork consisted of exploring the subsurface conditions in a number of boreholes [with soil sampling and insitu testing], coreholes [with rock core sampling], and in a number of probeholes [no sampling]. All of these types of exploratory holes will generally be referred to as boreholes in the text. Table 2.1 summarizes the subsurface exploration program for the major structures. The approximate location of the boreholes is shown on the Borehole Location Plans, Drawing Nos. 1a, 1b, and 1c inclusive. The field investigation was carried out under the direction and supervision of Soil-Mat Engineers & Consultants Ltd. [SOIL-MAT]. The borehole locations were selected in the field by representatives of Soil-Mat Engineers.

2.1 Drilling

The exploration work was carried out between January 8,1998 and May 27, 1999 using hollow-stem, continuous-flight auger equipment. Coreholes were advanced into the bedrock using N-size rotary coring equipment. Probeholes were advanced to auger refusal to determine 'assumed' depths to bedrock. Auger cuttings from the probeholes were logged in the field.

2.2 Sampling

Soil samples were obtained from the boreholes at regular depth intervals using split spoon sampling equipment driven in accordance with the Standard Penetration Resistance [N-value] test procedure as specified under *CSA test specification A 119.1 [ASTM test specification D 1586]*. All samples were subjected to visual and tactile classification and to routine moisture content testing. Selected soil samples were tested for unit weight values, and hand penetrometer testing was carried out on the cohesive soil samples.

Rock Core samples were obtained using a diamond tipped core barrel sampler. Laboratory testing on selected core samples included Point Load Strength Testing and Uniaxial Compression Testing.

The results of the field investigation and laboratory testing are presented in the attached Appendix A - Borehole Logs and Appendix B - Laboratory Testing Results.

2.3 Surveys – Borehole, Corehole, and Probehole Locations and Elevations

Surveying to establish borehole and probehole locations and ground surface elevations in the field were performed by personnel from Stantec Consulting Ltd. It is understood that the elevations have been referenced to the geodetic datum.

TABLE 2.1 SUMMARY OF BOREHOLE INFORMATION

Structure	Borehole No.	Elevation in meters			
		Ground Surface	"refusal" or Top of Rock	Bottom of Borehole	Ground water
BRIDGES					
1 [Ramp E-S/403]					
	1051	218.25	217.08	211.24	-
	4990	213.62	213.62	208.33	-
	4989	212.80			-
2 [Ramp N-E/403]					
	1519	220.90	217.85	214.80	-
	5029	221.41	218.77	216.12	-
	1521	221.05	219.42	214.95	-
3 [Mohawk/Ramp N-E] 4 [Ramp E-S/Mohawk]					
	1003	220.899	218.13	215.11	-

		Ground Surface	"refusal" or Top of Rock	Bottom of Borehole	Ground water
	1004	223.112	217.70	214.58	-
	1022	225.273	218.26	218.26	-
	1023	226.130	218.41	216.88	-
	1031	223.516	217.88	217.88	-
	1032	220.184	217.46	217.46	-
CULVERTS					
#1					
	1011	218.984	216.79	216.79	217.79
	1012	219.425	216.51	216.51	-
#2					
	1022	225.273	218.26	218.26	-
#3					
	1031	223.516	217.88	217.88	-
	1032	220.184	217.46	217.46	-
#4					
	1041	220.200	219.64	219.64	-
	1042	220.812	219.29	219.29	-

3.0 SITE AND SUBSOIL CONDITIONS

3.1 Bridge #1: Ramp E-S over 403 [Borehole Nos. 1051, 4990, 4989]

Topsoil

A veneer of topsoil up to 230 mm thick was encountered at the borehole locations. Deeper deposits of topsoil and organic material may be encountered during the course of construction.

Sandy and Clayey SILT

A deposit of light brown sandy and clayey silt was encountered beneath the topsoil layer. In general, the sandy silt till was found to be medium dense and contained some fine gravel and rock fragments

Bedrock

The bedrock recovered from the coreholes for the bridge structure investigations was found to consist of grey bedded dolostone/limestone. The bedrock consists of the Guelph Formation dolomite and/or Niagara dolomite of the Lockport Formation [Eramosa Member].

Uniaxial compressive strength for the rock at this location varied from about 80 to135 MPa [based on Point Load and Uniaxial Compression Testing]. Recovery varied from 98 to 100%: and RQD varied from 35 to 88.

Groundwater

Free groundwater was not observed in any of these boreholes. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils or from the bedrock.

3.2 Bridge #2: Ramp N-E over 403 [Borehole Nos. 1519,1521, 5029]

Topsoil

A veneer of topsoil about 200 mm thick was encountered at the borehole locations. Deeper deposits of topsoil and organic material may be encountered during the course of construction.

Sandy SILT

A deposit of light brown sandy silt was encountered beneath the topsoil layer. In general, the sandy silt till was found to be medium dense and contained some fine gravel and rock fragments

Bedrock

The Boreholes were advanced to auger refusal on assumed bedrock at depths of between 1.6 to 3.1 m below grade [Elevation 219.42 to 217.85 meters].

The bedrock was found to consist of tan to grey bedded dolostone/limestone beneath the soils in all of the boreholes. The bedrock recovered from the core sampling was found to consist of grey, bedded dolostone/limestone. The upper portion of the bedrock consists of the Guelph Formation, which grades into Niagara dolomite of the Lockport Formation [Eramosa Member].

Uniaxial compressive strength for the rock at this location varied from about 46 to 95 MPa [based on Point Load Testing]. Recovery varied from 83 to 100%: and RQD varied from 11 to 87.

Groundwater

Free groundwater was not observed in any of these boreholes. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils.

3.3 Bridges 3-4: Ramp E-S & N-E/Mohawk Rd. [Borehole Nos. 1003, 1004, 1023]

Topsoil

A veneer of topsoil about 200 mm thick was encountered at the borehole locations. Deeper deposits of topsoil and organic material may be encountered during the course of construction.

Sandy Silt

A deposit of light brown sandy silt was encountered beneath the topsoil layer. In general, the sandy silt till was found to be medium dense and contained some fine gravel and rock fragments. A 1.5 m thick layer of clayey silt was observed within this layer of sandy silt.

Clayey Silt [Till]

A deposit of clayey silt till was encountered beneath the sandy SILT. This till was brown to greyish brown, stiff to very stiff, in consistency and contains fine gravel and rock fragments, occasional thin fine sand seams and was hard to stiff in consistency.

Bedrock

The Boreholes were advanced to auger refusal on assumed bedrock at depths of between 2.8 to 7.7 m below grade [Elevation 217.7 to 218.4 m].

The bedrock was found to consist of tan to grey bedded dolostone/limestone beneath the soils in all of the boreholes. The bedrock recovered from the core sampling was found to consist of grey bedded dolostone/limestone. . The upper portions of the bedrock consist of the Guelph Formation, which grades into Niagara dolomite of the Lockport Formation [Eramosa Member].

Uniaxial compressive strength for the rock at this location varied from about 40 to 156 MPa [based on Uniaxial compression and Point Load Testing]. Recovery varied from 77 to 100%: and RQD varied from 28 to 88.

Groundwater

Free groundwater was not observed in any of these boreholes. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils.

3.2 Culvert #1 [Borehole No. 1011 & 1012]

Fill

A deposit of FILL consisting of brown sandy silt and gravel was encountered at the borehole locations. The thickness of the fill varied from 0.76 m in Borehole No. 1011 to 1.83 m in Borehole No. 1012.

Sandy Silt

Light brown to grey sandy silt was encountered beneath the fill in both boreholes.

Bedrock

The boreholes terminated at practical refusal to the hollow-stem auger penetration in assumed dolomite/limestone bedrock in both boreholes.

Groundwater

Free groundwater was observed at a depth of 1.2 m in Borehole No. 1011. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils.

3.2 Culvert #2 [Borehole No. 1022]

Fill

A deposit of FILL consisting of brown sandy silt and gravel was encountered at the borehole locations. The thickness of the fill was approximately 450 mm thick.

Clay

A deposit of clay was encountered beneath the FILL. This clay was grey and very stiff to hard, in consistency and contained occasional thin fine sand seams.

Clayey Silt [Till]

A deposit of clayey silt till was encountered beneath the CLAY. This till was brown to greyish brown, stiff to very stiff, in consistency.

Silt

Grey, stiff to very stiff silt with some sand seams was encountered beneath the clayey silt TILL.

Sandy Silt

Light brown to grey sandy silt was encountered beneath the SILT. This material was compact.

Bedrock

The borehole terminated at practical refusal to the hollow-stem auger penetration in assumed dolomite/limestone bedrock.

Groundwater

Free groundwater was not observed in this borehole. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils.

3.2 Culvert #3 [Borehole No. 1031 & 1032]

Topsoil

A veneer of topsoil approximately 225 mm thick was encountered in the boreholes for Culvert #3. Deeper deposits of topsoil and organic material may be encountered during the course of construction.

Sandy Silt

Light brown to grey sandy silt with some clayey silt was encountered beneath the topsoil in both boreholes.

Bedrock

Both boreholes terminated at practical refusal to the hollow-stem auger penetration in assumed dolomite/limestone bedrock in both boreholes.

Groundwater

Free groundwater was not observed in either of the boreholes along this section. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils.

3.2 Culvert #4 [Borehole No. 1041 & 1042]

Topsoil

A veneer of topsoil approximately 300 mm thick was encountered in the boreholes for Culvert #3. Deeper deposits of topsoil and organic material may be encountered during the course of construction.

Sandy Silt

Light brown sandy silt with some clayey silt was encountered beneath the topsoil in both boreholes.

Bedrock

Both boreholes terminated at practical refusal to the hollow-stem auger penetration in assumed dolomite/limestone bedrock in both boreholes.

Groundwater

Free groundwater was not observed in either of the boreholes along this section. Minor groundwater seepage should be expected during the course of construction from more permeable seams within the native soils.

4.0 DESIGN CONSIDERATIONS

4.1 General

Based on the geotechnical conditions indicated by the exploratory boreholes, the following preliminary engineering recommendations and comments are presented. It is assumed that the design will be in accordance with all applicable codes and standards. In addition, it is imperative that the recommendations and comments provided herein be reviewed by SOIL-MAT after the locations, elevations, and design of the structures is nearing completion.

4.2 Foundations

4.2.1 General

The subsurface conditions disclosed by this investigation indicated relatively good foundation conditions to support the proposed structures. The native clayey to sandy silt soils are capable of supporting conventional shallow foundations. The bedrock at the structure locations was at relatively shallow depths; therefore, foundations supported on the bedrock are a practical alternative for support of the structures

4.2.2 Bridge Structures

Shallow Foundations on Native Soils or Bedrock

Conventional shallow spread footing foundations may be used to support the bridge structures. All such footings should be founded on the undisturbed native clayey to sandy silts or bedrock. The footings should be founded at least 1.2 m [4 ft] below finish grade. Our engineering recommendations are based on the assumptions that the maximum span of these two bridges will be less than about 33 m and typical loading conditions for the types of structures proposed.

Shallow foundations on native soils should be designed using a factored Ultimate Limit state [ULS] bearing capacity of 500 KPa. The allowable bearing stress at Serviceability Limit State [SLS] should be limited to 325 kPa [based on total and differential settlements not exceeding 25 mm].

Shallow foundations on bedrock should be designed using a factored Ultimate Limit state [ULS] bearing capacity of 2500 kPa. These values are slightly lower than normal due to some relatively low strength and RQD values. The allowable bearing stress at Serviceability Limit State [SLS] will be controlled by the structure, as the rock would have to fail before the Service Limit deformations would be realized.

Resistance to lateral forces acting on the bridge structures may be determined using the following unfactored parameters:

FOUNDATION MATERIAL	Angle of Friction [degrees]	Adhesion [rough footing base] [kPa]
Cohesive soils	26	100
Sand soils	30	-
Dolomite/Limestone Bedrock	40	-

We do not recommend using passive soil resistance for resistance to lateral because of strain incompatibility and the possibility of future excavations, etc. If the frictional and/or adhesion resistance available is smaller than the horizontal force, then the footing may be 'keyed' into the soil to provide additional resistance.

All foundation excavations should be inspected prior to placement of the concrete. Excavations for shallow foundation in the native soils should be protected from disturbance due to construction activities, frost and/or freezing and from water, which if allowed to pond could cause softening of the bearing material.

Footing design must conform to the requirements of the current Highway Bridge Design Code - Ontario.

Deep Foundations

As an alternative to footing foundations, the bridge structures [Nos. 3 and 4] may be supported on deep foundations extended to the underlying bedrock. Engineering recommendations for two types of deep foundations, steel H-piles and drilled pier or caissons, are provided below.

Steel H-piles

Steel H-piles end-bearing on the bedrock may be used to support the bridge structures. The factored bearing capacity at [ULS] for H-piles on the dolostone bedrock of 75 MPa may be used for design of the H-pile foundation system, based on the cross-sectional area of the pile. Based on the anticipated loading requirements [verbal communications], we recommend that an HP 310 x 110 pile with additional plate welded to the web and flanges at the tip of the piles be used for supporting the bridge structures. This pile with an 8 mm plate having a total length of approximately 1m [additional area ~ 0.009 m^2] would have a capacity of:

Q[ULS] = 1690 kN;
Q[SLS] = 1150 kN.

Serviceability Limit State for the above pile is based on a pile length of 15 m and a maximum pile settlement of less than 6 mm. In actual fact for piles bearing on the bedrock, the SLS condition does not apply since more deflections would likely be tolerated in the structure than would be required to fail the rock, i.e., if larger than 6 mm settlements of the pile are acceptable, Ultimate Limit State would govern the design. Settlement of the piles in the loading range specified above is due to elastic compression of the piles, as settlement of the rock under these loading conditions is negligible.

Negative skin friction on the steel "H" piles is considered negligible.

The horizontal passive resistance for the above piles in the soils at the site is 200 kN for Ultimate Limit state and 110 kN for Serviceability Limit State. This recommendation is based on the assumption that the piles are rigidly connected to the pile caps and that loading tests are carried out to verify the lateral load capacity. For preliminary design 100 kN for Ultimate Limit state and 25 kN for Serviceability Limit State may be used.

Batter piles should have suitable rock-points on their tips to ensure that the piles penetrate the rock surface rather than slip along the top of the rock.

A pile driving hammer capable of developing sufficient energy to drive the piles through the hard soils and to refusal in the bedrock should be specified. A Berminghammer 400, Delmag D-22 diesel hammer, or equivalent would likely be suitable for this job. Final set of the piles should be established in the field based on the hammer used, the efficiency of the hammer and capblock system, and the weight / length of the piles.

It is important that the piles are not overstressed during pile driving as structural damage to the piles can occur as a result of over-driving. Piles driven to rock reach "practical refusal" quickly, and it is important to stop the driving when rock is reached to avoid 'overdriving' which can damage the piles.

Inspection and monitoring of the pile installation by a representative of SOIL-MAT is strongly recommended to confirm adequate set to achieve the required design capacities.

4.3 Lateral Earth Pressure

4.3.1 General

Lateral pressures on retaining structures [bridge abutments and retaining walls] are influenced by soil properties, groundwater conditions, surcharge loading, and the lateral movement of the structure. If the retaining structure design requirements and construction sequence are such that the structure can move slightly away from the backfill [unrestrained structure] shear strength in the soil is mobilized and the 'active' pressure case may be used for determining lateral earth pressure. Passive earth pressure in front of retaining structures should not include the top 1.0 m of the backfill soil due to the potential of adverse freeze-thaw action on this material.

For restrained structures an at-rest or 'backfill' condition should be assumed. An additional pressure due to the compaction operation should be incorporated into the lateral load calculations as indicated in the current Ontario Highway Bridge Design Code.

For an unrestrained retaining structure [less than 10 meters in effective height] the lateral loads may be determined based on the parameters [pressures per metre of wall height] detailed in Table 6 – 7.4.4 of the Ontario Highway Bridge Design Code equivalent fluid pressures as presented below:

Equivalent Fluid Pressure, Unrestrained Structures

Backfill Type	Granular 'A'	Granular 'B'
Angle of Internal Friction	More than 35^{O}	Between 30^{O} and 35^{O}
Active Pressure	5.9 kPa	6.9 kPa
Backfill Pressure	7.7 kPa	8.6 kPa

For all restrained retaining structures [less than 10 meters in effective height] the lateral loads may be determined based on the [minimum unfactored] parameters [minimum pressures per metre of wall height] detailed in Table 6 – 7.4.5 of the Ontario Highway Bridge Design Code equivalent fluid pressures as presented below.

Equivalent Fluid Pressures, Restrained Structure

Backfill Type	Granular 'A'	Granular 'B'
Backfill Pressure*	10.0 kPa	12.0 kPa

4.3.2 Bridge Abutments

Backfill behind bridge abutments should consist of free-draining granular material, and an adequate drainage system should be provided. The granular backfill should conform to OPSS specifications for Granular 'A' or 'B' material and be compacted to at least 95 per cent of the Standard Proctor maximum dry density. Assuming that the backfill behind abutments is compacted granular material as specified, the following parameters are recommended for determination of the lateral pressures acting on the abutment.

Lateral Earth Pressure Parameters	Ultimate Limit State [ULS]	Service Limit State [SLS]
Angle of Internal Friction *[unfactored]* =	32^{0}	
Angle of Internal Friction *[factored]* =	26.6^{0}	
Active Earth Pressure Coefficient *[factored]*, K_a =	0.39	0.31
At-Rest Earth Pressure Coefficient *[factored]*, K_0 =	0.47	0.47
Passive Earth Pressure Coefficient *[factored]*, K_p =	1.63	3.25
Fully Restrained Earth Pr. Coefficient [factored], K* =	0.69	0.55

For stability against sliding, in the case that the bridge abutment footings are placed on original ground, the spread footings should be proportioned such that the 'driving' active forces are balanced by the 'resisting' passive forces and frictional resistance or the adhesion between the base of the concrete footing and the bearing soil. The frictional resistance may be computed by assuming a static angle of friction of 28 degrees [unfactored], or adhesion between the 'rough' footing base and the underlying cohesive soils of 100 kN/m^2 [unfactored]. The above unbalanced horizontal force can also be resisted by battered piles in the case of a pile-supported structure.

Mechanically stabilized soil systems for abutments or retaining structures, such as, Reinforced Earth, use design parameters which are proprietary in nature. These systems should be reviewed by Soil-Mat Engineers prior to construction.

4.4 Culvert Nos. 1 to 4

The culverts are expected to be founded in the silty clay till material. The culverts may be designed using a factored Ultimate Limit state [ULS] bearing capacity of 500 kPa. The allowable bearing stress at Serviceability Limit State [SLS] should be limited to 325 kPa [based on total and differential settlements not exceeding 25 m]. We would recommend that an OPSS Granular 'A' be incorporated as a 'leveling' course material. The Granular 'A' material should be compacted to a minimum of 95 per cent standard Proctor density [using a static compactor]. The upstream end of the culvert structure should be 'keyed' into the silty clay to silty clay till to prevent 'piping' and undermining of the culvert. The culverts and head walls should be constructed in accordance with OPSS and the Regional Municipality of Hamilton-Wentworth specifications

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4.5 Excavations

All excavations must comply with the current *Occupational Health and Safety Act and Regulations for Construction Projects.*

Excavations in the silt clay to sandy silt overburden materials are expected to be relatively straightforward. Temporary excavations in native soils to the nominal depths expected on slopes of one horizontal: two vertical [1H : 2V] and in the fill material on slopes of one horizontal to one vertical [1H : 1V] should remain stable for the short construction periods provided surcharge loading and/or vibrations due to construction activities are not excessive. Groundwater seepage into excavations is expected to be limited and it should be controllable using conventional 'dewatering' techniques, i.e. pumping from sumps and ditches. All surface runoff should be directed away from the excavations.

Excavation into the dolostone/limestone bedrock will require the use of 'hoe-ramming' or blasting techniques. The strength of the rock determined from uniaxial compression testing and point load strength testing varies from approximately UCS = 40 to 150 Mpa. Given the relative shallow rock excavations anticipated for this project, the excavations in the rock should stand stable indefinitely on near vertical cut slopes. Such steep slopes would also reduce the width and height of ditch required to arrest rock falls from entering the traveled portion of the road. The strike and dip of the major joint sets and bedding planes in the bedrock in this case is of secondary importance, given the limited height of rock cut required. All loose rock should, of course, be scaled back from the face of the cut. If rock cuts are required in excess of about 5 m deep, then this office should be given an opportunity to reevaluate the stability of the rock cut. The strike and dip of major joint sets and bedding planes and the condition of the bedding planes and their infillings will then be determined with respect to the stability of the vertical cuts. The maximum height of any cut bench should be about 6 m, and a 2 m wide bench should be provided for each 6 m cut bench. This is to facilitate construction, maintenance, etc. Cuts significantly deeper than about 6 m become rather inefficient, since the impact energy from the pneumatic drill is absorbed in the drill stem, rather then used to make hole.

The majority of the excavated dolostone/limestone bedrock material is considered suitable for processing for use as granular roadway base and sub-base course material.

4.6 General Backfill Considerations

The excavated native materials on this project are generally considered suitable for structural backfilling purposes. However, some moisture content conditioning may be required for efficient compaction, depending on the weather conditions at the time of construction. The soils are considered to be currently 'dry' of their optimum moisture contents. These fine grained soils are sensitive to moisture absorption and may become practically impossible to compact using conventional compaction equipment if they become excessively wet. If the soils are too wet for efficient compaction, they should be exposed to the elements to dry before being reemployed as backfill, or should be discarded or used in non-settlement sensitive fill areas. After periods of heavy precipitation, any surface fill materials softened by excessive water should be removed and allowed to dry prior to re-use, or should be discarded.

The backfill material for roadway, berm/noise barrier acoustic wall construction should be compacted to 95 per cent standard Proctor density. The lift thickness of the fill will depend on the moisture content of the backfill material, the proximity of any underground pipes and other services and/or structures to the fill being compacted, and the size and type of the compaction equipment available for the project.

Any imported fill material should have its moisture content within 3 per cent of its optimum moisture content and meet the necessary environmental guidelines. We would recommend that a sampling/testing program of any imported material be undertaken prior to its shipment to the project site.

However, as previously noted, well graded free-draining granular material should be used as backfill against bridge abutments, etc. The fill behind the bridge abutments must also be suitably drained to prevent the build-up of water pressure behind the walls.

4.7 Corrosion Protection

We recommend that sulphate resistant cement be utilized for all buried concrete due to the concentrations of sulphate that has been detected in the native soils in this area [this includes partially buried structures such as, bridge abutments].

The potential for corrosion of the reinforcing steel in concrete is also accelerated by the operating environment associated with de-icing salts on the roadways during the winter months.

5.0 GENERAL COMMENTS

The purpose of this preliminary geotechnical report was to determine the subsurface conditions at the borehole locations and to provide our comments and recommendations from a geotechnical point-of-view. The information contained in this geotechnical investigation report in no way reflects upon the environmental aspects of the site and therefore has not been addressed in this document, as this information is beyond the formatted scope and terms of reference.

The comments provided in this document are preliminary and intended only for the guidance of the design team. The borehole descriptions and logs are not to be considered descriptive of the conditions at locations other than at the borehole locations. Contractors bidding or undertaking this project should decide on their own exploration and own interpretation of the factual borehole results, so that they might decide on how the subsurface conditions will effect their operations.

We trust that this geotechnical report is sufficient for your purposes. Should you have any questions, please do not hesitate to contact this office.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Robert Horvarth, Ph.D., P. Eng.	Danny Schebesch, M.E.Sc., P. Eng.
Project Manager	Review Engineer

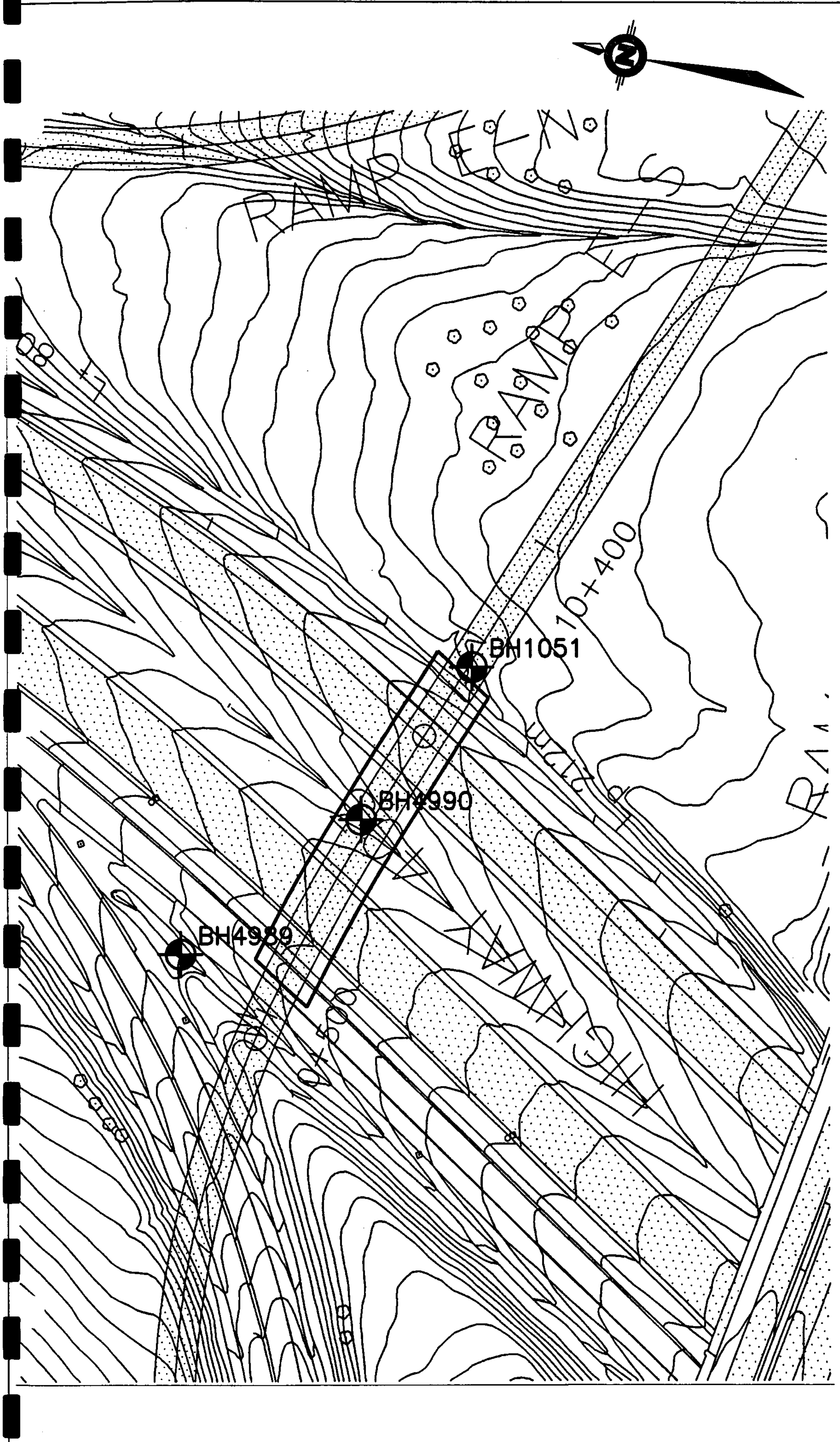
for

Enclosures: Plan Showing Borehole and Probehole Locations, Drawing Nos. 1a to 1c inclusive

Appendix A: Borehole Log Nos. 1003, 1004, 1011, 1012, 1022, 1023, 1031, 1032, 1041, 1042, 1051, 1112, 1519, 1521, 4989, 4990, 5029.

Appendix B: Laboratory Testing Results

Distribution: Regional Municipality of Hamilton-Wentworth [2]
Stantec Consulting Limited [2]



LEGEND

BH1 Borehole

NOTES:

1. This drawing should be read in conjunction with Soil–Mat Engineers & Consultants Ltd. report number SM 97027–G.
2. Soil samples will be discarded after 3 months unless directed otherwise by client.

Soil-Mat
Engineers & Consultants Ltd.

CLIENT	Stantec Consulting Limited
PROJECT TITLE	Geotechnical Investigation Hwy. 403 at Mohawk Rd. / "Linc" Interchange Hamilton, Ontario
DRAWING TITLE	Borehole Locations Bridge No. 1: Ramp E–S / Hwy. 403
PROJECT No.	SM 97027–G (Stantec # 650–00146)
SCALE	N.T.S.
DATE	August 1999
DESIGN	SS
DRWN	
FILENAME	027G1aSS.DWG

Drawing Ia

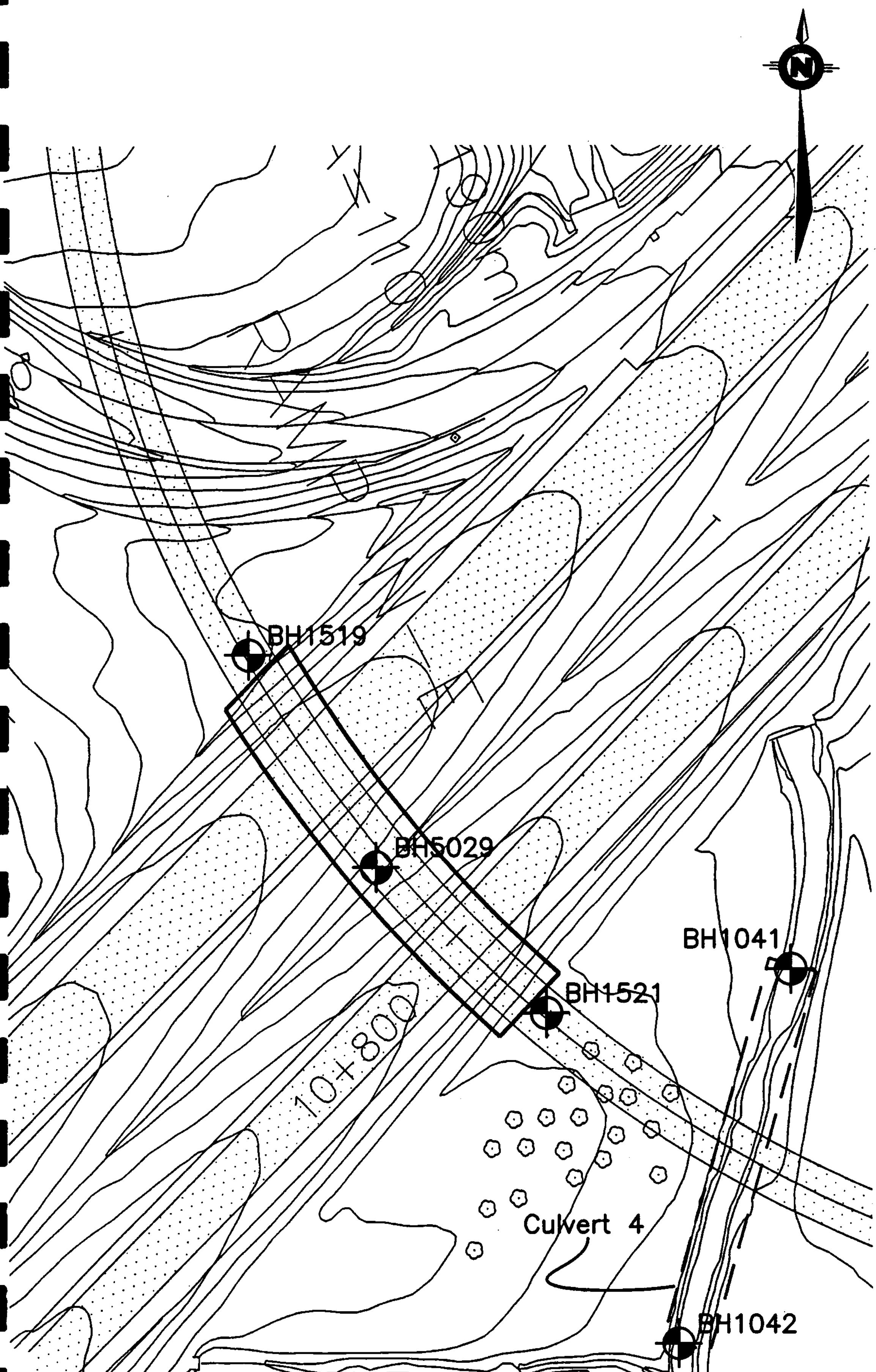
LEGEND

BH1 Borehole

NOTES:

1. This drawing should be read in conjunction with Soil–Mat Engineers & Consultants Ltd. report number SM 97027–G.

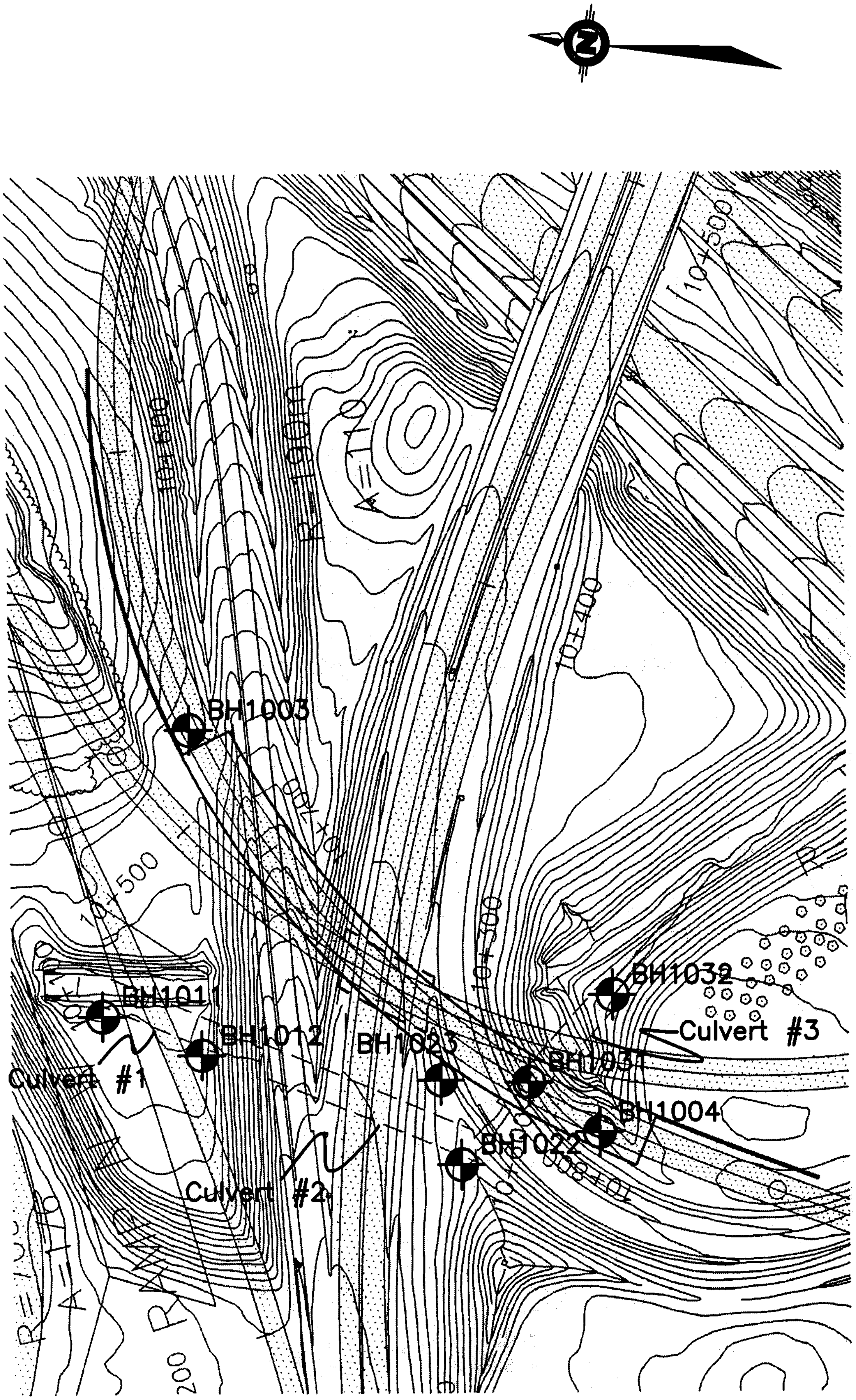
2. Soil samples will be discarded after 3 months unless directed otherwise by client.



Soil-Mat
Engineers & Consultants Ltd.

CLIENT	Stantec Consulting Limited
PROJECT TITLE	Geotechnical Investigation Hwy. 403 at Mohawk Rd. / "Linc" Interchange Hamilton, Ontario
DRAWING TITLE	Borehole Locations Bridge No. 2: Ramp N–E / Hwy. 403, Culvert 4
PROJECT No.	*SM 97027–G*
	(Stantec # 650–00146)
SCALE	*N.T.S.*
DATE	*August 1999*
DESIGN	*SS*
DRWN	
FILENAME	*027G1bSS.DWG*

Drawing Ib



LEGEND

BH1 Borehole

NOTES:

1. This drawing should be read in conjunction with Soil-Mat Engineers & Consultants Ltd. report number SM 97027-G.
2. Soil samples will be discarded after 3 months unless directed otherwise by client.

Soil-Mat
Engineers & Consultants Ltd.

CLIENT	Stantec Consulting Limited
PROJECT TITLE	Geotechnical Investigation Hwy. 403 at Mohawk Rd. / "Linc" Interchange Hamilton, Ontario
DRAWING TITLE	Borehole Locations Bridges No. 3 and 4: Ramp E-S / Mohawk Rd., Mohawk Rd. / Ramp N-E, Culverts 1, 2 and 3
PROJECT No.	SM 97027-G (Stantec # 650-00146)
SCALE	N.T.S.
DATE	August 1999
DESIGN	SS
DRWN	
FILENAME	027G1cSS.DWG

Drawing Ic

APPENDIX A

Borehole Logs

Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp E-S (Bridges # 3 and 4)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1003

(SM Corehole No.: A)

Corehole Location: Sta.: 10+679.1

Offset: 4.60

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE		SAMPLE						Moisture Content/Std. Penetration 'N'
Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	w% 10 20 30 40 / Blows/300mm 20 40 60 80
Ground Surface	0 220.9							
Topsoil Approximately 150mm.								
Fine Sandy Silt Light brown, compact.		SS	1	23				
	2.77 218.13							
Dolostone/Limestone Bedrock Thin-bedded to massive,Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core run #1: RQD=62, Recovery=91%		Core	1					
Core run #2: RQD=28, Recovery=100%		Core	2					
End of Corehole	5.79 215.11							

Depth scale: ft 0–32; m 0–8

NOTES:

1. Borehole advanced with hollow-stem continuous-flight auger to a depth of 2.77m and cored with a diamond tipped core barrel to a depth of 5.79m on January 20, 1998.

2. Terminated dry and backfilled on completion.

3. Samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: Hollow Stem

Drill Date: January 20, 1998

Hole Size: 200mm

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Sheet: 1 of 1

Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp E-S (Bridges # 3 and 4)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1004

(SM Corehole No.: B)

Corehole Location: Sta.: 10+800.7

Offset: 2.20

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE: Description	Depth/Elev.	SAMPLE: Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery
Ground Surface	0						
Topsoil Approximately 75mm.	223.11						
Sandy Silt Brown.	1.22 221.89						
Fine Sandy Silt Grey, traces of clay, dense.	2.8147 220.3	SS	1	32		22.1	
Clayey Silt / Fine Sand Light brown and reddish brown.	4.57 218.54	SS	2	12		20.6	
Fine Sandy Silt Grey, trace of rootlets, clay and gravel, staining, compact.	5.41 217.7	SS	3	18		21.3	
Dolostone/Limestone Bedrock Thin-bedded to massive, Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core run #1: RQD=62; Recovery=87%.		Core	1				
Core run #2: RQD=33; Recovery=78%.	8.53 214.58	Core	2				
End of Corehole							

Moisture Content/Std. Penetration 'N': w% 10 20 30 40; Blows/300mm 20 40 60 80

Depth: ft 0–32; m 0–8

NOTES: 1. Corehole advanced with hollow-stem contiuous-flight auger to a depth of 5.41m and cored with diamond tip core barrel to 8.53 m on January 21, 1998.
2. Terminated dry and backfilled on completion
3. Samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: Hollow Stem

Drill Date: January 21, 1998

Hole Size: 200mm

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Project No: SM 97027 G

Project: Expressway/403 Interchange

Location: Ramp N-W (Culvert #1)

Client: Reg.Municipality of Hamilton-Wentworth

Borehole # 1011

(SM Borehole No.: C1-1)

Borehole Location: Sta.: 10+119.4

Offset: 11.2

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE				SAMPLE						Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	▲ w% 10 20 30 40 ▲ / ● Blows/300mm 20 40 60 80 ●
0 / 0		Ground Surface	0							
		Fill 762 mm Topsoil, sandy silt, gravel and rock, moist.	218.98 / 0.762							
		Sandy Silt Grey, fine,compact, non-cohesive, and moist. Native material. Fractured rock encountered at 1.52 m.	218.22 / 2.19							
		End of Borehole	216.79							

Depth scale: ft 0–32; m 0–8.

NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 2.19m on January 8, 1998.
2. Terminated and backfilled on completion.
3. Water level observed on completion at a depth of 1.19m
4. No samples taken.

Drill Method: H.S.A.

Drill Date: January 8, 1998

Hole Size: 150 mm

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Project No: SM 97-027 G	**Borehole # 1012** (SM Borehole No.: C1-2)
Project: Expressway/403 Interchange	Borehole Location: Sta.: 10+131.9
Location: RAMP N-W (Culvert #1)	Offset: -11.1
Client: Reg.Municipality of Hamilton-Wentworth	Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE		SAMPLE						
Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	Moisture Content/Std. Penetration 'N' (w% 10 20 30 40; Blows/300mm 20 40 60 80)
Ground Surface	0 / 219.43							
Fill Silty clay and sandy silt mix, brown and moist. Topsoil in tip, stained, sandy, blackish, non-cohesive and compact.		SS	1	11				
	1.83 / 217.6	SS	2	9				
Sandy Silt Light brown, fine, compact, non-cohesive, and moist. Changing to grey with greater depth.		SS	3	30				
	2.92 / 216.51							
End of Borehole								

Depth scale: ft 0–32; m 0–8

NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 2.92m on January 8, 1998.
2. Terminated dry and backfilled on completion.
3. Soil samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: H.S.A.	SOIL-MAT ENGINEERS & CONSULTANTS LTD.	Datum: Geodetic
Drill Date: January 8, 1998	130 Lancing Drive, Hamilton, ON L8W 3A1	Checked by: RGH
Hole Size: 150 mm	Phone: (905) 318-7440 Fax: (905) 318-7455 e-mail: info@soil-mat.on.ca	Sheet: 1 of 1

Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp N-W Culvert #2)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1022

(SM Borehole No.: C2-2)

Corehole Location: Sta.: 10+778.2

Offset: 19.4

Project Manager: John Monkman

Soil-Mat

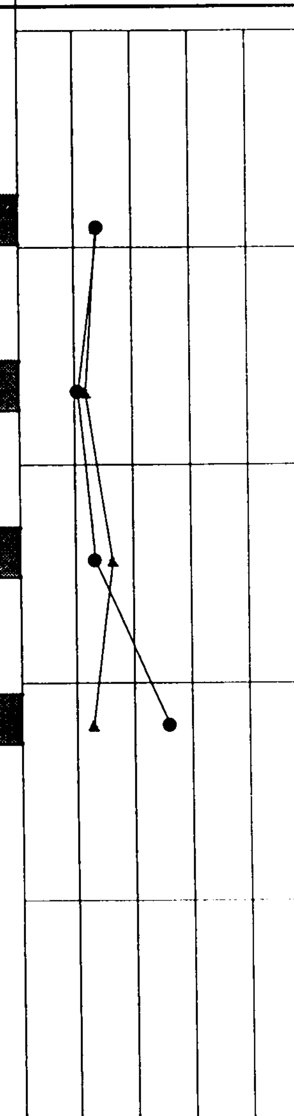
SUBSURFACE PROFILE			SAMPLE						
Description	Depth/Elev.		Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery
Ground Surface	0							
Fill Approximately 450mm.	225.27	0.9						
Clay grey, sand seams, very stiff to hard	224.37	3.05	SS	1	30		22.5	
Clayey Silt Grey, stiff to very stiff, with sand seams.	222.22	4.569	SS	2	15		22.3	
Silt Brown, compact	220.7	6.1	SS	3	20		21.4	
Sandy Silt Grey, compact	219.17	7.01	SS	4	14		20.3	
End of Corehole	218.26							

Depth: ft 0–32; m 0–8

Moisture Content/Std. Penetration 'N'

▲ w% 10 20 30 40

● Blows/300mm 20 40 60 80



NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 7.01 m on January 8, 1998.
2. Terminated dry and backfilled on completion.
3. Samples will be kept for 3 months unless otherwis directed by the client.

Drill Method: Hollow Stem

Drill Date: February 5, 1998

Hole Size: 200mm

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Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp E-S (Bridges # 3 and 4)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1023

(SM Corehole No.: C)

Corehole Location: Sta.: 10+761.6

Offset: 7.40

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE / SAMPLE

Depth/Elev.	Description
0 / 226.13	Ground Surface
0.45 / 225.68	***Fill*** Approximately 450mm.
4.569 / 221.56	***Sandy Silt*** Reddish grey and brown, occasional stones, compact.
6.1 / 220.03	***Silty Clay*** Grey and brown, very stiff.
7.72 / 218.41	***Sandy Silt*** Brown, very dense.
9.25 / 216.88	***Dolostone/Limestone Bedrock*** Thin-bedded to massive, Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core Run #1: RQD=64, Recovery=77% Core Run #2: RQD=88, Recovery=100%
	End of Corehole

NOTES:

1. Borehole advanced with hollow-stem continuous-flight auger to a depth of 7.72m and cored to a depth of 9.25m on February 5, 1998.
2. Terminated dry and backfilled on completion.
3. Samples will be kept for 3 months unless otherwis directed by the client.

Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)
SS	1	28		21.3
SS	2	21		21.7
SS	3	27		21.6
SS	4	52		21.8
Core	1, 2			

Moisture Content/Std. Penetration 'N'

w% 10 20 30 40

Blows/300mm 20 40 60 80

Drill Method: Hollow Stem

Drill Date: February 5, 1998

Hole Size: 200mm

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Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp S-E (Culvert #3)

Client: Reg. Municipal of Hamilton-Wentworth

Borehole # 1031

(SM Borehole No.: C3-1)

Borehole Location: Sta.: 10+786.2

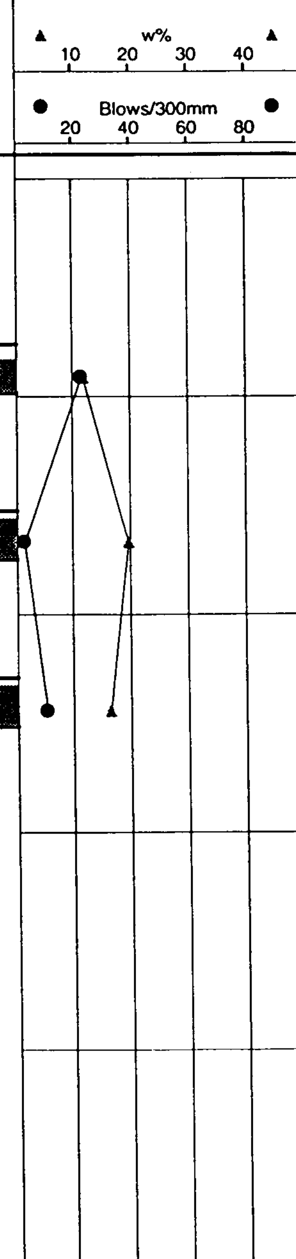
Offset: 0.6

Project Manager: John Monkman

Soil-Mat

Depth (m)	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery
0	Ground Surface	0 / 223.52						
0–0.2	***Topsoil*** Approximately 200mm.							
0.2–4.57	***Fine Silty Sand*** Some silt.	4.57 / 218.95	SS	1	23		21.9	
			SS	2	3			
4.57–5.64	***Sandy Silt*** Light brown to greyish black, trace of clay, organic.	5.64 / 217.88	SS	3	11		20.7	
5.64	End of Borehole							

Moisture Content/Std. Penetration 'N'



NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 5.64m on January 22, 1998.
2. Terminated dry and backfilled on completion.
3. Soil samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: Hollow Stem

Drill Date: January 22, 1998

Hole Size: 200mm

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Project No: SM 97027-G	**Borehole # 1032** (SM Borehole No.: C3-2)
Project: Expressway/403 Interchange	Borehole Location: Sta.: 10+612.6
Location: Ramp N-E (Culvert #3)	Offset: -13.4
Client: Reg. Municipal of Hamilton-Wentworth	Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE				SAMPLE						Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	▲ w% ▲ 10 20 30 40 / ● Blows/300mm ● 20 40 60 80
0 / 0		Ground Surface	0							
		Topsoil Approximately 225mm.	220.18 0.23 / 219.95							
2 – 8 ft / 2 m		***Clayey Silt / Sandy Silt*** Brownish grey to light brown, trace of organic material.	2.72	SS	1	7				●
		End of Borehole	217.46							

Depth scale: ft 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32; m 0, 2, 4, 6, 8

NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 2.72m on January 22, 1998.
2. Terminated dry and backfilled on completion.
3. Soil samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: Hollow Stem	SOIL-MAT ENGINEERS & CONSULTANTS LTD.	Datum: Geodetic
Drill Date: January 22, 1998	130 Lancing Drive, Hamilton, ON L8W 3A1	Checked by: RGH
Hole Size: 200mm	Phone: (905) 318-7440 Fax: (905) 318-7455 e-mail: info@soil-mat.on.ca	Sheet: 1 of 1

Project No: SM 97027-G	**Borehole # 1041** (SM Borehole No.: C4-1)
Project: Expressway/403 Interchange	**Borehole Location:** Sta.: 10+842.5
Location: Ramp N-E (Culvert #4)	Offset: -26.5
Client: Reg. Municipal of Hamilton-Wentworth	**Project Manager:** John Monkman

Soil-Mat

SUBSURFACE PROFILE				SAMPLE						Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	▲ w% 10 20 30 40 ▲ / ● Blows/300mm 20 40 60 80 ●
0 / 0		Ground Surface	0							
		Topsoil Approximately 300mm.	220.2 / 0.3							
		Fine Sandy Silt Light brown, slightly stained.	219.9 / 0.56							
		End of Borehole	219.64							

Depth scale: ft 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32; m 0, 2, 4, 6, 8

NOTES:

1.Borehole augered to practical refusal (assumed bedrock) at 0.56m on January 29, 1998.

2. Terminated dry and backfilled on completion.

3. No samples taken.

Drill Method: Hollow Stem	SOIL-MAT ENGINEERS & CONSULTANTS LTD.	Datum: Geodetic
Drill Date: January 29, 1998	130 Lancing Drive, Hamilton, ON L8W 3A1	Checked by: RGH
Hole Size: 200mm	Phone: (905) 318-7440 Fax: (905) 318-7455 e-mail: info@soil-mat.on.ca	Sheet: 1 of 1

Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp N-E (Culvert #4)

Client: Reg. Municipal of Hamilton-Wentworth

Borehole # 1042
(SM Borehole No.: C4-2)

Borehole Location: Sta.: 10+854.5

Offset: 29.5

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE				SAMPLE							Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	w% 10 20 30 40	Blows/300mm 20 40 60 80
0 / 0		Ground Surface	0								
		Topsoil Approximately 300mm.	220.81 / 0.3 / 220.51								
		Fine Sandy Silt Brown, slightly stained, oxidized.	1.52 / 219.29	SS	1	10					
		End of Borehole									

Depth scale (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32; (m): 0, 2, 4, 6, 8

NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 1.52m on January 29, 1998.
2. Terminated dry and backfilled on completion.
3. Soil sample will be kept for 3 months unless otherwise directed by the client.

Drill Method: Hollow Stem

Drill Date: January 29, 1998

Hole Size: 200mm

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Project No: SM 97027-G

Project: Expressway/403 Interchange

Location: Ramp S-E (Bridge # 1)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1051

(SM Corehole No.: D)

Corehole Location: Sta.: 10+423.4

Offset: -1.9

Project Manager: John Monkman

Soil-Mat

Depth (ft / m)	Description (Subsurface Profile)	Depth/Elev.	Sample Type	Sample Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery
0 / 0	Ground Surface	0 218.25						
	Clayey Silt	1.17 217.08						
	Dolostone/Limestone Bedrock Thin-bedded to massive,Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses.		Core	1				
			Core	2				
			Core	3				
	End of Corehole	5.77 212.48						

Moisture Content/Std. Penetration 'N': w% 10 20 30 40; Blows/300mm 20 40 60 80

NOTES:

1. Borehole advanced with hollow-stem continuous flight auger to a depth of 1.17m and cored to a depth of 5.77 m on February 6, 1998.

2. Terminated dry and backfilled on completion.

3. Samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: Hollow Stem

Drill Date: February 6, 1998

Hole Size: 200mm

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Project No: SM-97027-G

Project: Expressway/403 Interchange

Location: Ramp N-E

Client: Reg. Municipality of Hamilton-Wentworth

Borehole # 5019

(Sta. 10+375)

Borehole Location: Sta: 10+390.4

Offset: -17.4

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE			SAMPLE							Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Description	Depth/Elev.	Type	Number	Blows/150mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	w% 10 20 30 40	Blows/150mm 20 40 60 80
0	Ground Surface	0 / 226.03								
0–0.15 m	***Topsoil*** 150 mm									
	Fill Mix Light and reddish brown, some staining some clay, some gravel, some sandy silt, compact		SS	1	20		21.7			
		3.01 / 223.02								
	Fine Sandy Silt Grey, some clay, trace of roots, compact		SS	2	27		21.2			
		3.81 / 222.22								
	End of Borehole									

NOTES:

1. Borehole augered to practical refusal (assumed bedrock) at 3.81m on May 26, 1999.
2. Terminated dry and backfilled on completion.
3. Soil samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: HS/SS

Drill Date: May 26, 1999

Hole Size: 200 mm

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Datum:

Checked by:

Sheet: 1 of 1

Project No: SM 97027-G

Project: Expressway/ 403 Interchange

Location: Ramp N-E (Bridge # 2)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1519

(SM Corehole No.: 519)

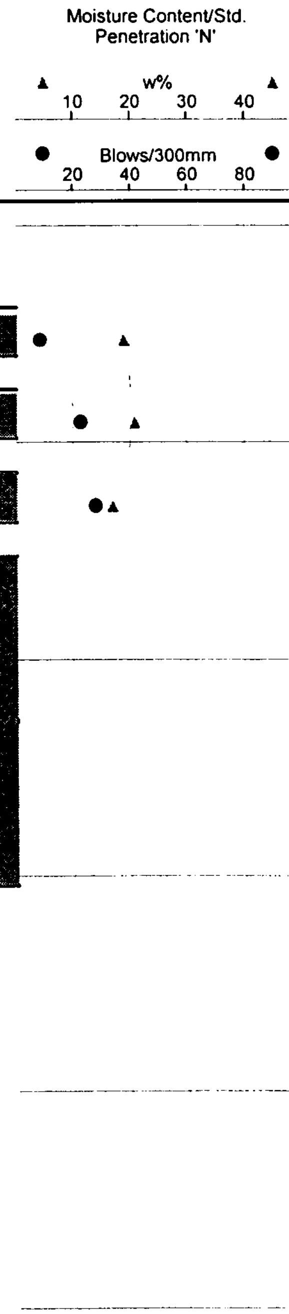
Corehole Location: Sta.: 10+748.6

Offset: -0.3

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE		SAMPLE						
Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	
Ground Surface	0 220.9							
Topsoil 300mm	0.3 220.6							
Silty Sand/ Sandy Silt Brown and light brown, fine to medium, trace of rootlets, oxidized, loose..	1.52 219.38	SS	1	9		20.1		
Fine Sandy Silt Light brown, thin grey clay seams, oxidized, compact.	2.29 218.61	SS	2	23		20.7		
Till Grey, with clay and limestone.	3.05 217.85	SS	3	28		20.4		
Dolostone/Limestone Bedrock Thin-bedded to massive,Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core run #1: RQD=68; Recovery=83%		Core	1					
Core run #2: RQD=87; Recovery=96%	6.1 214.8	Core	2					
End of Corehole								



NOTES:

1. Borehole advanced with hollow-stem continuous flight auger to bedrock at 3.05 m and cored with diamond tipped core barrel to 6.1 m on January 28, 1998.

2. Borehole backfilled on completion.

3. Soil samples will be kept for 3 months unless otherwise directed by client.

Drill Method: Hollow Stem

Drill Date: January 28, 1998

Hole Size: 200mm

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Datum: Geodetic

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Project No: SM 97027-G

Project: Expressway/ 403 Interchange

Location: Ramp N-E (Bridge # 2)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 1521

(SM Corehole No.: 521)

Corehole Location: Sta.: 10+816.5

Offset: -0.3

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE				SAMPLE						Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	▲ w% 10 20 30 40 ▲ / ● Blows/300mm 20 40 60 80 ●
0 / 0		Ground Surface	0 221.05							
		Topsoil Approximately 200mm.								
2–4		***Fine Sandy Silt*** Light brown, limestone pieces, trace of rootlets, oxidized, loose to compact	1.63	SS	1	10			■	●
6 / 2		***Dolostone/Limestone Bedrock*** Thin-bedded to massive,Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core run #1: RQD=42; Recovery=100%	219.42	Core	1				■	
10–14 / 4		Core run #2: RQD=11; Recovery=97%		Core	2				■	
16–20 / 6		Core run #3: RQD=14; Recovery=97%	6.1	Core	3				■	
20		End of Corehole	214.95							

NOTES:

1. Borehole advanced with hollow-stem continuous flight auger to bedrock at 1.63 m and cored with diamond tipped core barrel to 6.1 m on January 30, 1998.

2. Borehole backfilled on completion.

3. Soil samples will be kept for 3 months unless otherwise directed by client.

Drill Method: Hollow Stem

Drill Date: January 30, 1998

Hole Size: 200mm

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Sheet: 1 of 1

Project No: SM 97027-G

Project: Mohawk/ 403 Interchange

Location: Ramp E-S (Bridge # 1)

Client: Reg.Municipality of Hamilton/Wentworth

Corehole # 4989

(SM Corehole No.: E)

Corehole Location: Sta.: 10+498.1

Offset: 18.5

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE			SAMPLE						Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	w% 10 20 30 40; Blows/300mm 20 40 60 80
0 / 0		Ground Surface	0 212.8							
		FILL sand ,loose rock	0.94 211.86							
		Bedrock Thin-bedded to massive,Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core 1: RQD=79; Recovery=100%		Core	1					
		Core 2: RQD=88; Recovery=100%	3.89 208.91	Core	2					
		End of Corehole								

NOTES:

1. Borehole advanced with hollow-stem continuous-flight auger to a depth of 0.94 m and cored to a depth of 3.89 m on June 18, 1998.

2. Terminated dry and backfilled on completion.

3. Samples will be kept for 3 months unless otherwise directed by the client.

Depth scale: ft 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32; m 0, 2, 4, 6, 8

Drill Method: Hollow Stem

Drill Date: June 19, 1998

Hole Size: 150mm

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Project No: SM-97027-G

Project: Expressway/403 Interchange

Location: Ramp E-S (Bridge # 1)

Client: Reg. Municipality of Hamilton-Wentworth

Borehole # 4990

Borehole Location: Sta.: 10+460.3

Offset: 1.3

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE				SAMPLE						Moisture Content/Std. Penetration 'N'
Depth (ft / m)	Symbol	Description	Depth/Elev.	Type	Number	Blows/150mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	w% 10 20 30 40; Blows/150mm 20 40 60 80
0 ft / 0 m		Ground Surface	0 213.62							
		Dolostone/Limestone Bedrock Highly fractured, loose to depth of 1.53 m. Thin-bedded to massive,Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses.								
		Core Run #1: RQD=35, Recovery=100%		CB	1					
		Core Run #2: RQD=64, Recovery=98%		CB	2					
		Core Run #3: RQD=84, Recovery=100%		CB	3					
		End of Borehole	5.29 208.33							

NOTES:

1. Borehole advanced with hollow-stem continuous-flight auger to a depth of 1.53m and cored with a diamond tipped core barrel to a depth of 5.29m on May 27, 1999.
2. Terminated dry and backfilled on completion.
3. Samples will be kept for 3 months unless otherwise directed by the client.

Drill Method: HSA/Core Barrel

Drill Date: May 27, 1999

Hole Size: 200 mm/N

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Sheet: 1 of 1

Project No: SM-97027-G

Project: Expressway/403 Interchange

Location: Ramp N-E (Bridge # 2)

Client: Reg. Municipality of Hamilton-Wentworth

Corehole # 5029

Corehole Location: Sta.: 10+786.2

Offset: 0.2

Project Manager: John Monkman

Soil-Mat

SUBSURFACE PROFILE		SAMPLE						Moisture Content/Std. Penetration 'N'
Description	Depth/Elev.	Type	Number	Blows/300mm	PP (kgf/cm2)	U.Wt.(kN/m3)	Recovery	w% 10 20 30 40 / Blows/300mm 20 40 60 80
Ground Surface	0 221.41							
Topsoil	0.76 220.65							
Clayey Silt brown, firm w/some sand	2.64 218.77							
Dolostone/Limestone Bedrock Thin-bedded to massive, Dark to light brown-grey, fine crystalline, with vug-filled mineralization and some chert nodules and lenses. Core Run #1: RQD=35, Recovery=92%		CB	1					
Core Run #2: RQD=81, Recovery=100%	5.67 215.74	CB	2					
End of Corehole								

NOTES:

1. Borehole advanced with hollow-stem continuous-flight auger to a depth of 1.53m and cored with a diamond tipped core barrel to a depth of 5.29m on May 27, 1999.
2. Terminated dry and backfilled on completion.
3. Samples will be kept for 3 months unless otherwise directed by the client.

Depth scale: ft 0–32; m 0–8

Drill Method: HSA/Core Barrel

Drill Date: May 27, 1999

Hole Size: 200 mm/N

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Sheet: 1 of 1

APPENDIX B

Laboratory Testing Results

UNCONFINED [UNIAXIAL] COMPRESSION TEST

Unconfined [Uniaxial] Compression testing of rock may be used to determine the strength and deformation behaviour of intact rock specimens. The compressive strength values obtained may also be used to provide an approximate index of drilling and excavating properties of the rock mass.

The rock samples, obtained from the core drilling, were tested at the McMaster University Geotechnical Laboratory. The specimens were approximately 46.5 mm in diameter and 105 mm in length, with a length/diameter ratio, L/D of approximately 2.2. the test results are summarized in Table B1, and the load-displacement curves are presented in Figures B1 to B9.

Table B1. Uniaxial Compression Test - Summary

Borehole		Sample				
No.	Elevation [m]	No.	Elevation [m]	Unit Weight [kN/m³]	UCS [MPa]	E_{S50} [MPa]
1003	220.90	A-1	217.53	26.4	156	54,160
1003	220.90	A-2	216.25	24.92	107	35,151
1004	223.11	B-1	217.31	26.7	130	32,480
1004	223.11	B-2	215.16	26.0	94	22,661
1023	226.13	C-1	218.43	26.3	101	26,198
1023	226.13	C-2	216.63	25.9	115	32,849
1051	218.25	G-2	214.45	26.3	136	44,455
F		F-1		25.9	111	35,892
F		F-2		25.4	93	41,607

POINT LOAD TESTS

The Point Load Test is an index test which can be used to test irregular size and shape rock specimens. The load required to split the rock, compressed between two conical steel points and the specimen size determine the Point Load Index, I_p . The Unconfined [Uniaxial] Compressive Strength of the rock may be estimated from the Point Load Index using empirical relationships.

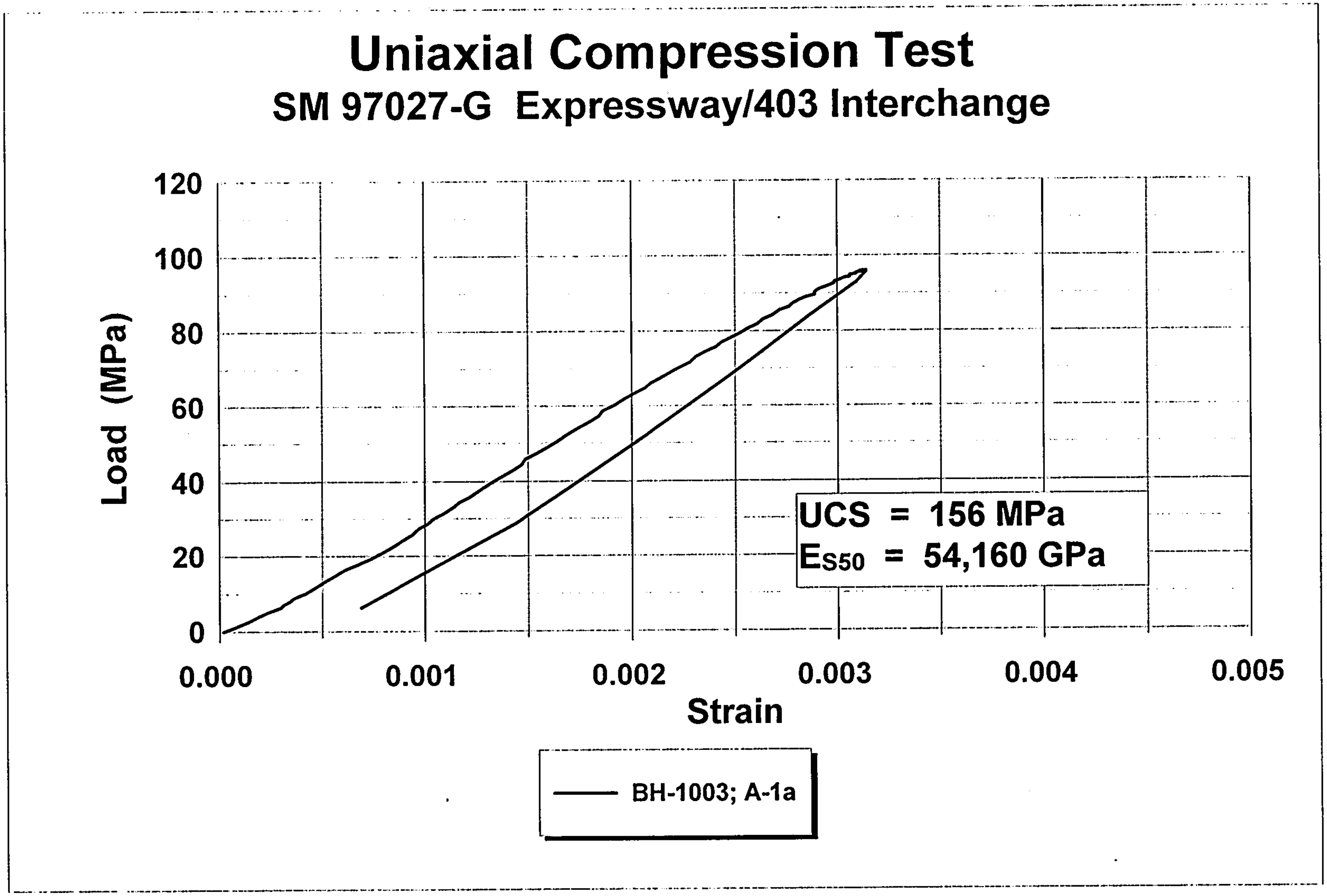
Point Load Tests were carried out on numerous rock specimens obtained from the core drilling. The results of these tests are summarized in Table B2.

Table B2. Point Load Test Results

Borehole No.	Borehole Elev. [m]	Point Load Index *Is = P/D2* [MPa]	Estimated Strength UCS [MPa]
1003	***220.90***		
	215.90	4.25	90
	216.15	2.96	65
	216.45	3.29	70
	216.45	3.30	70
	217.00	3.22	70
	217.00	3.29	70
	217.30	2.08	45
	217.30	4.15	90
	217.67	2.93	65
	217.67	4.29	95
1004	***223.11***		
	214.91	2.29	50
	214.91	1.80	40
	215.91	1.92	40
	215.91	2.06	45
	216.61	2.88	60
	216.61	1.48	30
	216.91	2.64	60
	216.91	1.83	40
	217.61	5.58	115
	217.61	3.69	80

1023	***226.13***		
	216.83	4.39	95
	216.83	2.32	50
	217.23	4.10	90
	217.23	3.00	65
	218.13	3.57	75
	218.13	2.88	65
	218.58	2.65	55
	218.58	2.72	60
4989	***212.80***		
	213.99	1.59	35
	214.25	3.47	75
	214.53	6.90	155
	214.60	5.42	120
	216.60	4.28	110
	216.60	6.02	145
F	***??***		
	-6.90	3.61	80
	-6.90	3.34	70
	-6.80	4.85	110
	-6.80	3.72	85
	-6.80	2.02	45
	-6.80	3.01	70
	-6.40	3.66	75
	-6.40	2.72	60
	-6.10	2.72	60
	-6.10	3.09	65
	-5.79	4.25	95
	-5.79	4.88	110
	-5.00	4.01	90
1519	***220.90***		
	215.10	2.89	65
	215.10	2.36	50
	215.60	3.28	70
	215.60	2.08	45
	216.15	2.17	45
	216.15	1.16	25
	217.10	3.37	75
	217.10	3.98	85
	217.60	2.20	50
1521	***221.05***		
	215.25	4.81	105
	215.25	2.72	60
	216.95	3.15	70

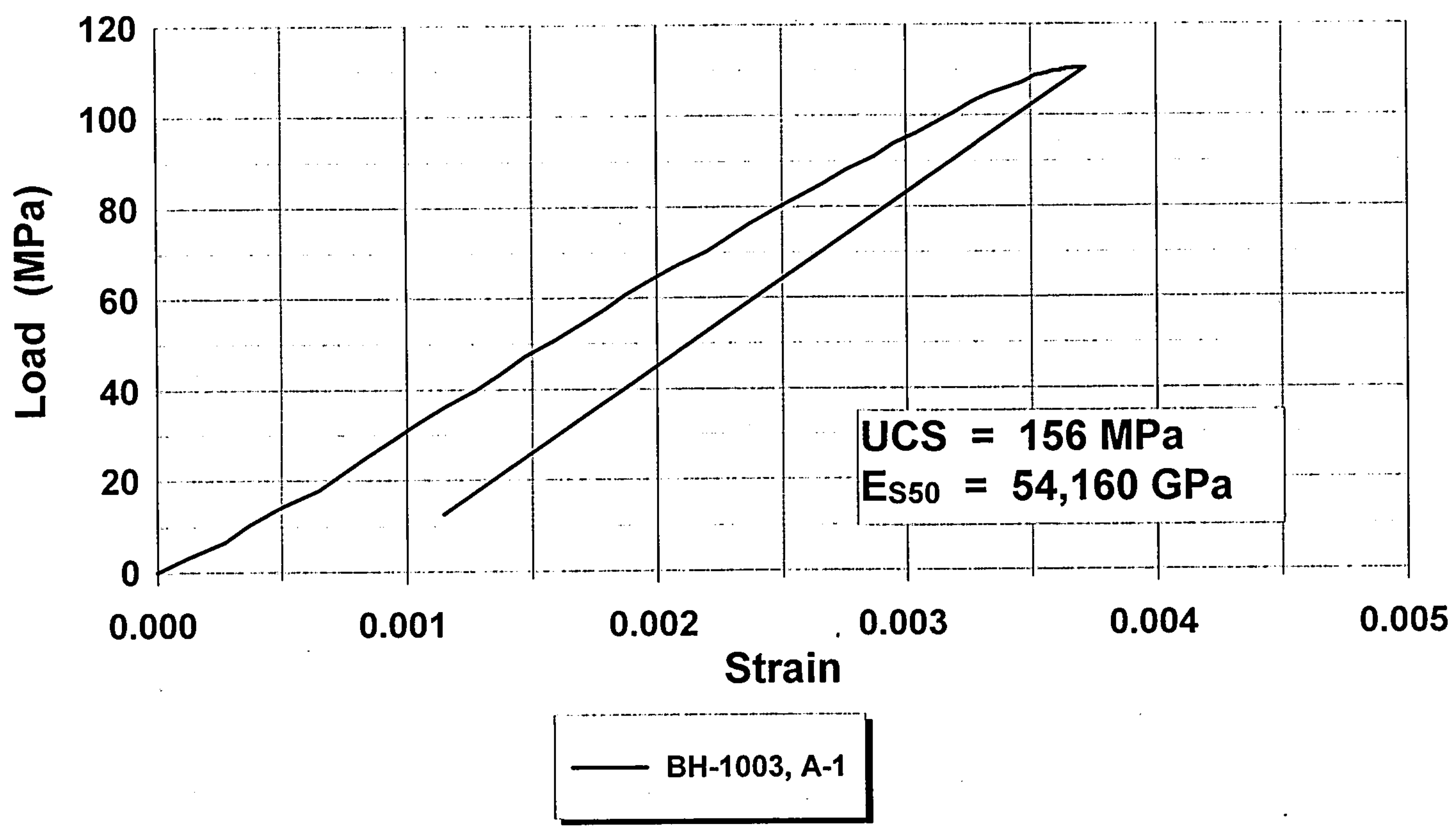
	216.95	2.44	55
	218.65	4.64	100
	218.65	2.64	60
	219.42	3.02	65
	219.42	2.88	65

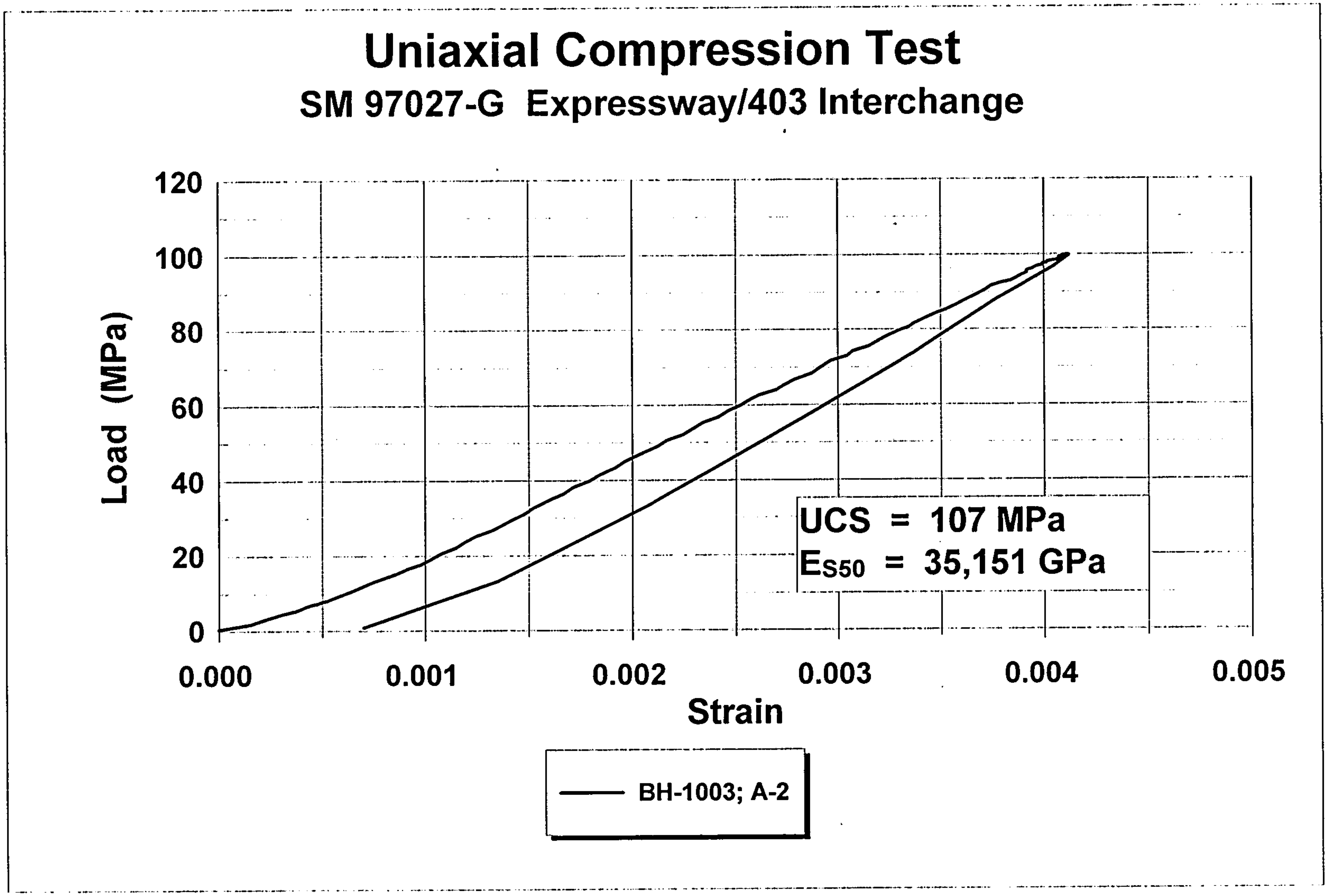


Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
Load (MPa)
120
100
80
60
40
20
0
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 156 MPa
ES50 = 54,160 GPa
BH-1003; A-1a

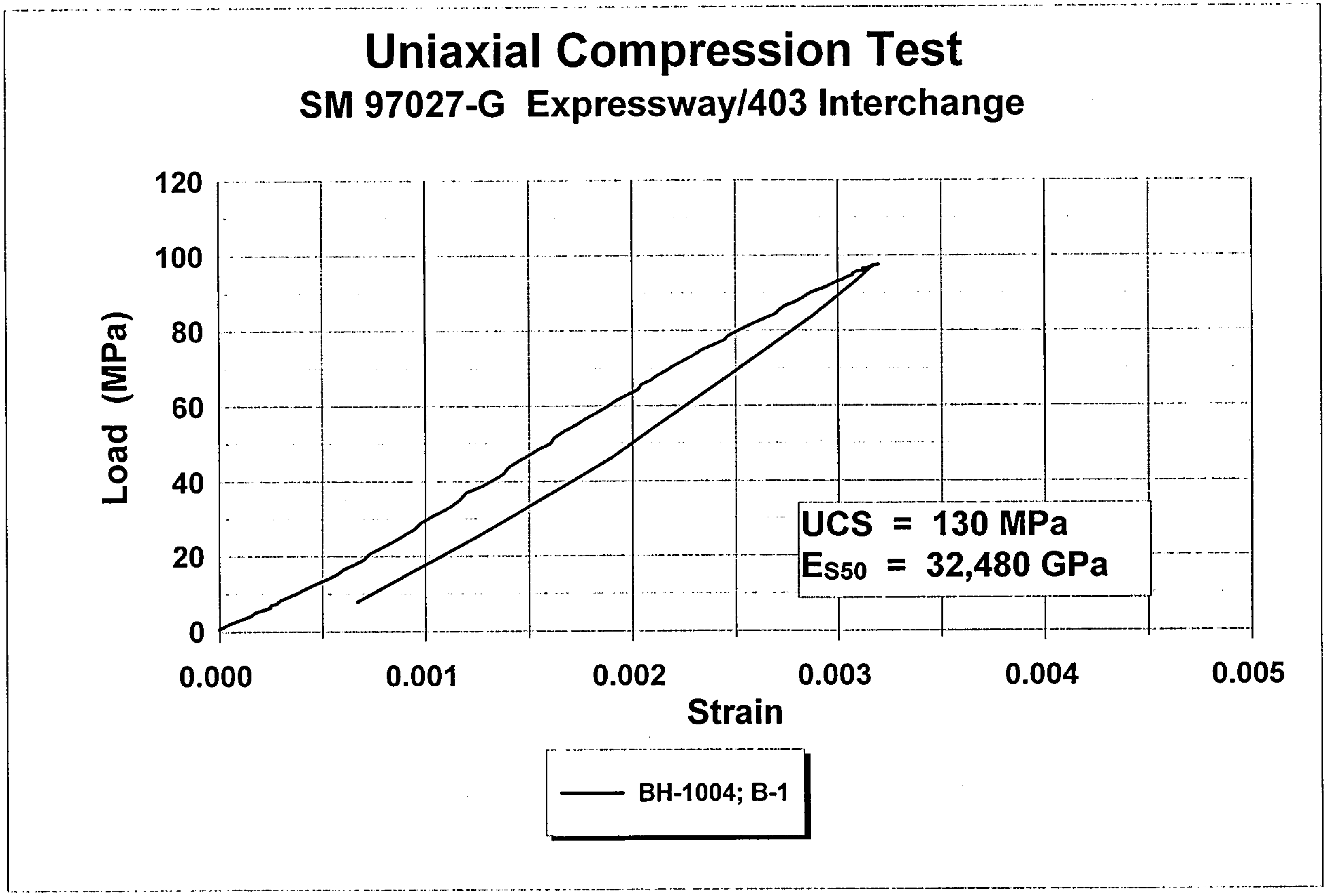
Uniaxial Compression Test

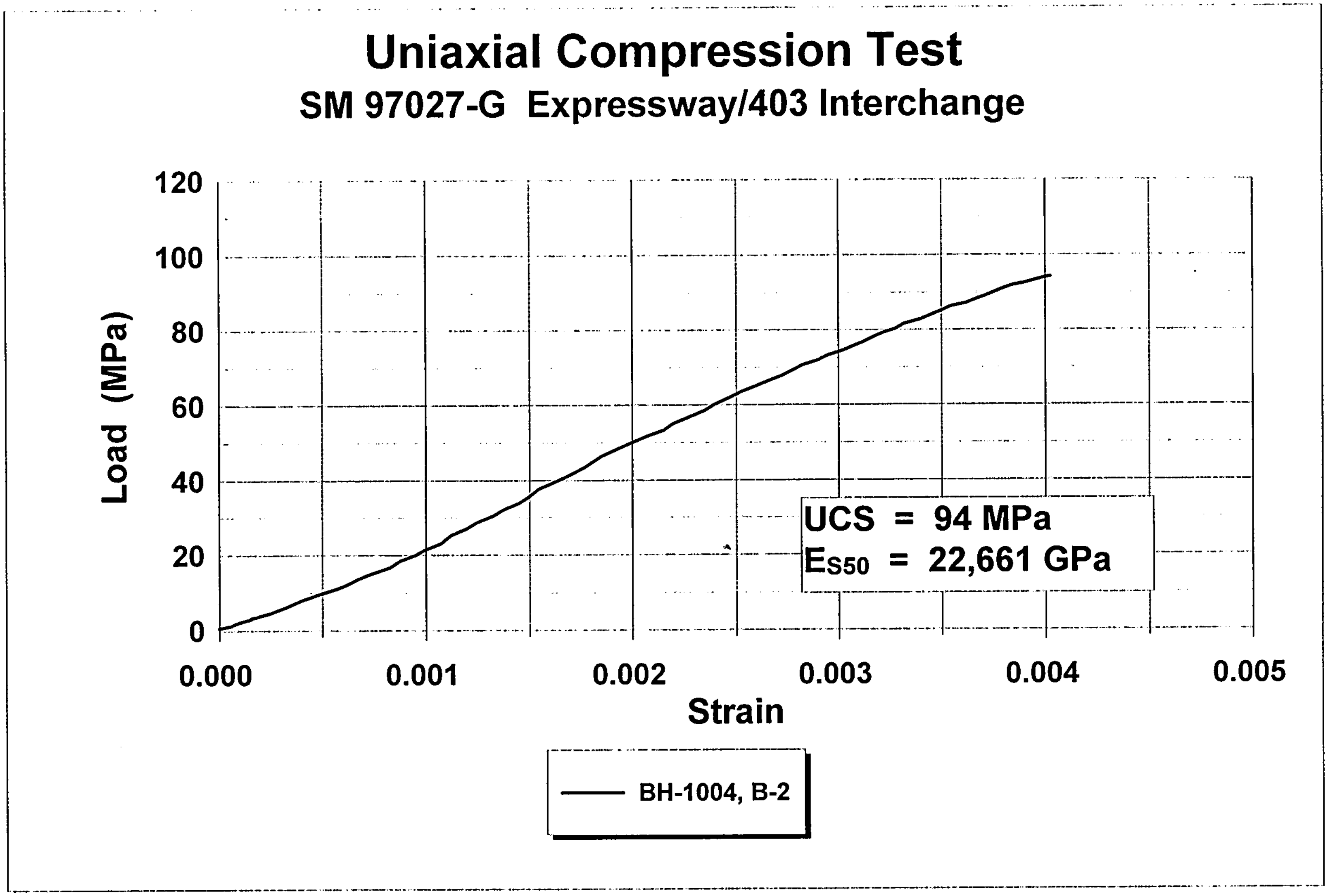
SM 97027-G Expressway/403 Interchange



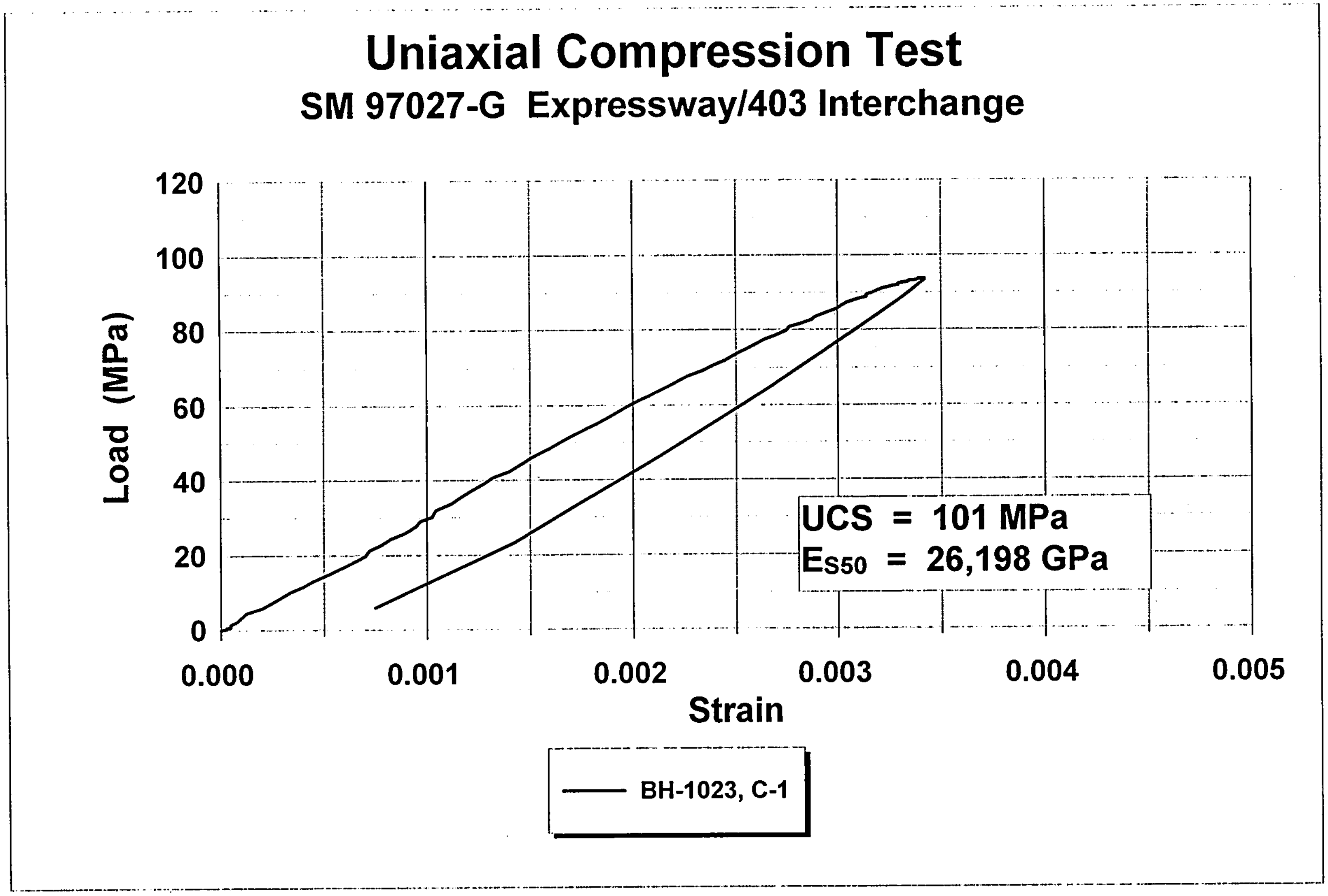


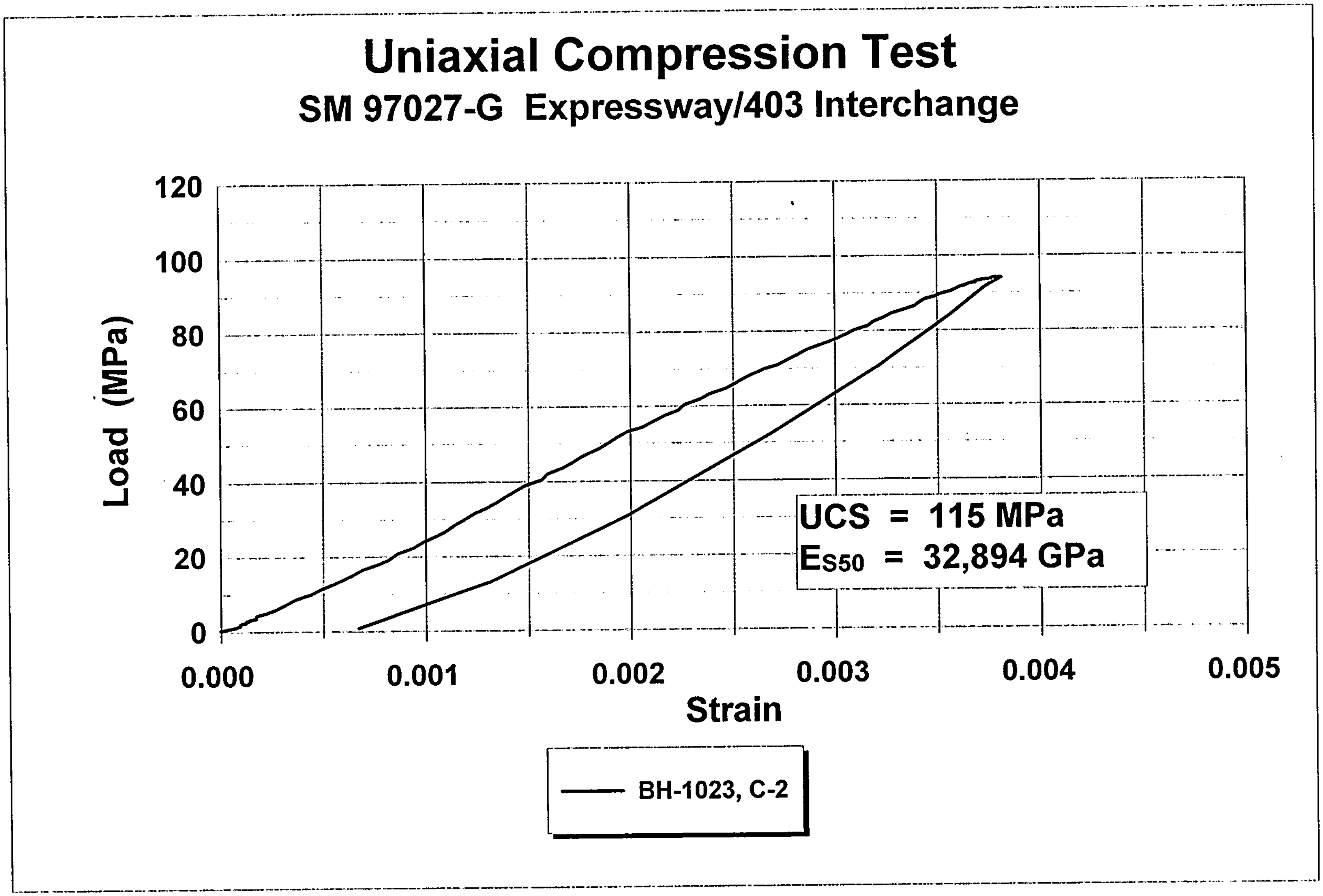
Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
Load (MPa)
120
100
80
60
40
20
0
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 107 MPa
ES50 = 35,151 GPa
BH-1003; A-2


Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
Load (MPa)
120
100
80
60
40
20
0
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 130 MPa
Es50 = 32,480 GPa
BH-1004; B-1

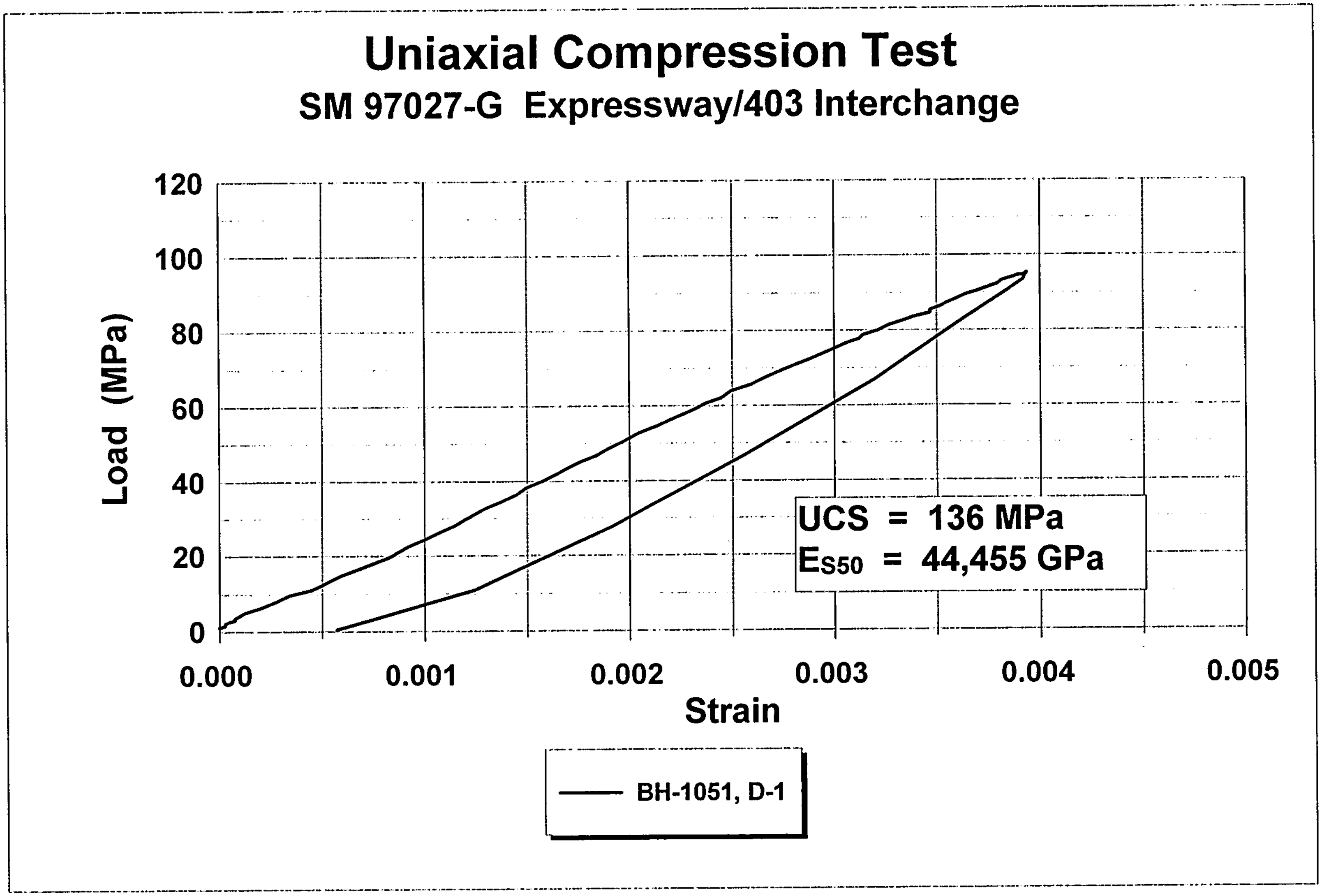


Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
Load (MPa)
120
100
80
60
40
20
0
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 94 MPa
E_{S50} = 22,661 GPa
BH-1004, B-2


Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
120
100
80
60
40
20
0
Load (MPa)
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 101 MPa
ES50 = 26,198 GPa
BH-1023, C-1

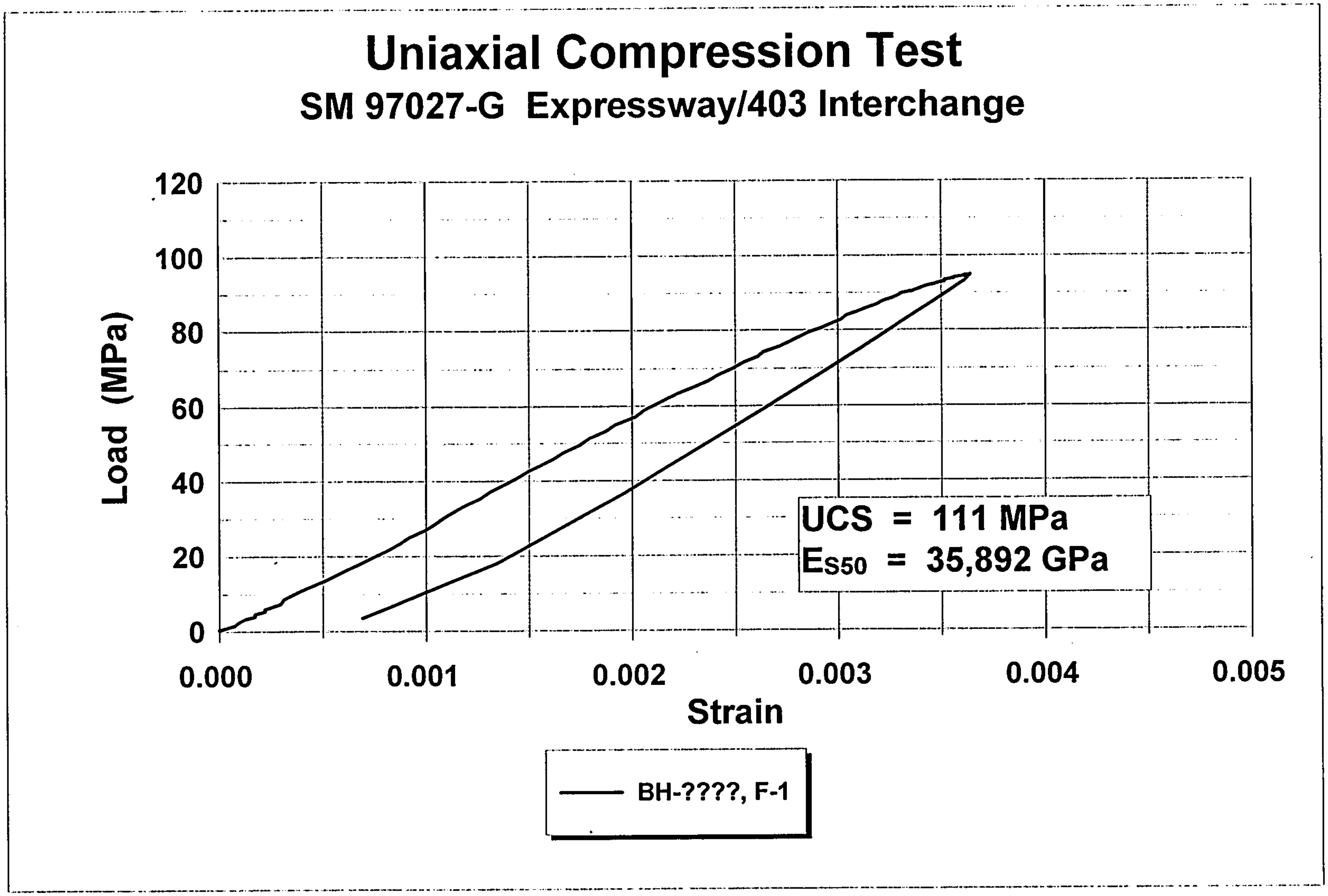

Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
Load (MPa)
120
100
80
60
40
20
0
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 115 MPa
Es50 = 32,894 GPa
BH-1023, C-2

SM 97027-G

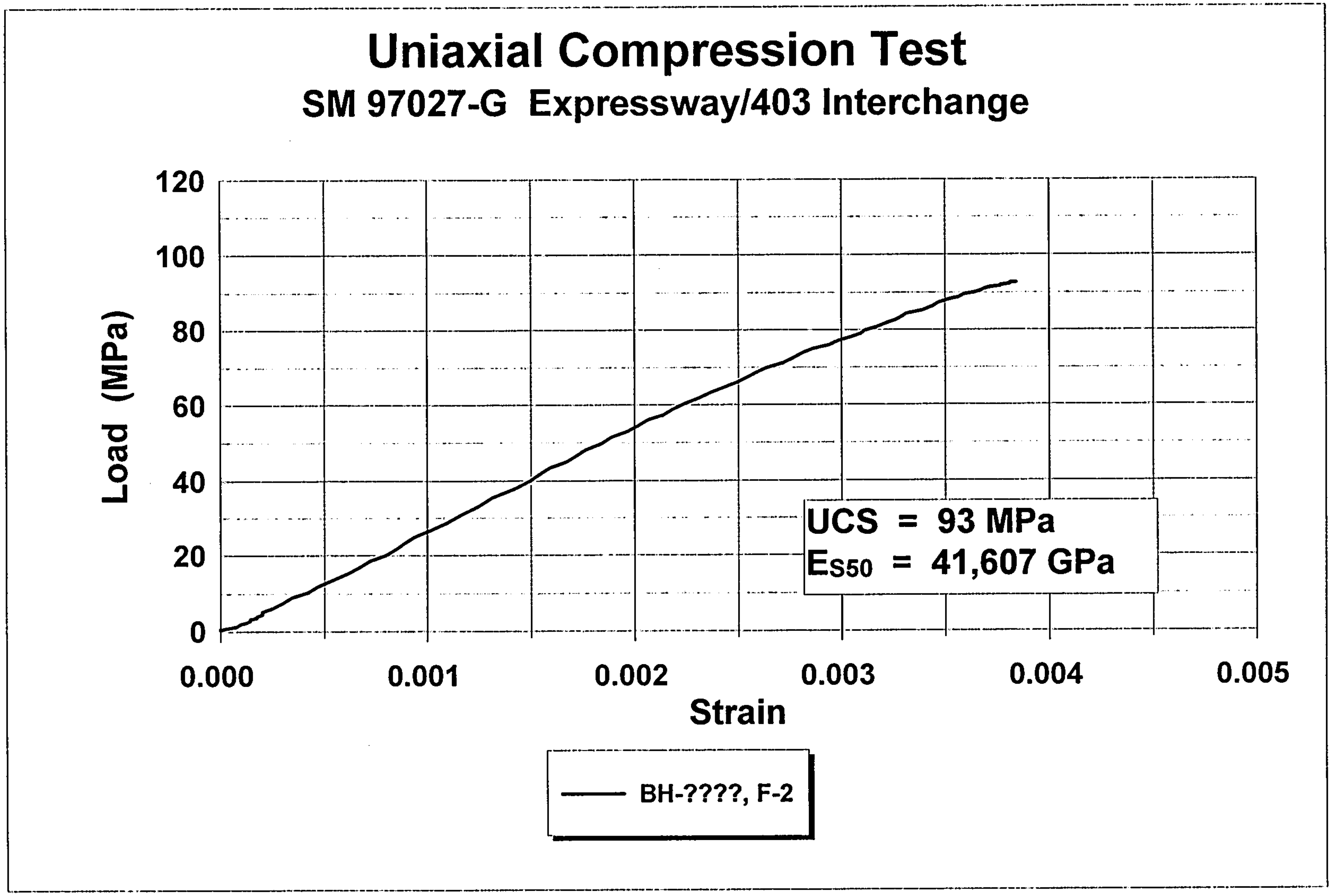


ucs_tests.wb3

SM 97027-G



ucs_tests.wb3


Uniaxial Compression Test
SM 97027-G Expressway/403 Interchange
120
100
80
60
40
20
0
Load (MPa)
0.000
0.001
0.002
0.003
0.004
0.005
Strain
UCS = 93 MPa
Es50 = 41,607 GPa
BH-????, F-2