

HAMILTON

AREA, STUDY OF

QUEENSTON

SHALE

30MS-95

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Chedoke Xpressway

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GEOCON

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Mumtala, Ontario,
August 19th, 1960.

Department of Highways, Ontario,
Materials and Research Section,
Downsview, Ontario.

Attention: Mr. L. C. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Engineering Study,
Properties of Onondaga Shale,
Proposed Chedoke Expressway,
Hamilton Area, Ontario.

Dear Sirs:

This letter summarizes the results of the above study which was authorized by your letter dated April 21st, 1960. A detailed report presenting the complete data upon which these results are based will be forwarded at a later date.

General Description

The Onondaga formation is a marine deposit of Ordovician age. It is of uniform composition over a large area and consists of a dark red brown shale interspersed with occasional irregular beds or pockets of olive green calcareous siltstone. It is highly fissile, and breaks frequently and easily parallel to the bedding planes. The latter properties render the shale particularly susceptible to weathering, and generally a zone of weathering up to 10 or 12 feet in thickness overlies the sound rock.

The shale is composed primarily of quartz and illite with relatively minor amounts of feldspar, calcite, and hematite. The average specific gravity of both the weathered and unweathered material is 2.78.

The effect of weathering on the engineering properties of the shale can be inferred from the results of the various index tests carried out on samples of both materials. As indicated by the grain size distribution curves

Department of Highways, Ontario,
August 1949, 1950,
Page 2.

General Properties (continued)

given on Figure 1 of Appendix II there is no significant difference between the two materials after pulverization with a mortar and pestle in accordance with standard soil testing procedures. Atterberg limits obtained from both materials also indicated no significant difference in the composition of the two materials; liquid limits of 30.6 and 27.5 were obtained for the weathered and unweathered shale respectively, with corresponding plasticity indices of 9.7 and 8.6. The corresponding liquid limits and plasticity index for the oven-dried, weathered shale were 25.8 and 8.3. These results confirm that the weathering process in the Queenston shale has been predominantly by mechanical as opposed to chemical processes, and imply that over the long term the engineering properties of both weathered and unweathered shale will be essentially similar.

The rate at which the material weathers is indicated by the fact that drilled cores of the sound rock usually disintegrated along the bedding planes almost immediately upon exposure to air. When immersed in water a sample of the sound rock immediately flaked along bedding planes and became totally disintegrated within about 10 hours. The mechanical effect required to break down the unweathered material in the laboratory was observed to be roughly comparable to that required to pulverize a silty clay glacial till of hard consistency.

The total unit weight of the sound shale determined on a sample of vanned core was 163 pounds per cubic foot at a water content of 4.1 percent. The computed dry density was 156 pounds per cubic foot with a computed saturation of over 100 percent.

In view of the indicated similarity between the sound and weathered shale, further detailed testing was carried out on a bulk sample of the weathered shale. This was prepared for testing in accordance with the procedure outlined in Appendix I.

Compaction Properties

Both Standard and Modified AASHTO compaction tests were carried out on the bulk sample of weathered shale. A series of three compaction curves were also obtained using the Harvard Miniature device with various compactive efforts. The results of these tests are shown on Figure 2 of Appendix II. From these results it may be seen that for a compactive effort of 10 layers and 25 blows per layer with a 40 pound spring, the compaction curve using the Harvard Miniature device approximates the Standard AASHTO curve. Furthermore, it was known from past experience that the Standard AASHTO curve adequately represents the degree of compaction gained in the field with this material. On this basis, the above compactive effort with the Harvard device was used for the study of compacted strength.

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Page 1.

Consolidated Research Proposition

The compacted, undrained shear strength properties with various confining pressures and at various moulding water contents are shown on Figure 3 of Appendix II. These results show the characteristic oyster curve of strength versus water content with the peak strength occurring at moulding water contents slightly less than the optimum water content for compaction.

A series of undrained strength tests with measured pore pressures were also carried out to determine the effective stress parameters of the compacted material. These were run at moulding water contents which were 1 to 3 percent less than optimum. The stress strain curves for these various tests are shown on Figures 4 to 6 inclusive in Appendix II. The first two series, shown on Figures 4 and 5 and from which the Mohr diagram given on Figure 9 was derived, were run according to conventional techniques. The tests shown on the following Figures 6 to 8 were run using various techniques and under various conditions in order to explain certain inconsistencies which were evident from the results of the first two series, and particularly in view of recent developments in shear strength theory and testing practice. These will be discussed in detail in the final report.

For the present it seems clear that the effective angle of shearing resistance, ϕ' , of 23 degrees indicated in Figure 9 is valid for practical purposes. However, the indicated apparent cohesion, C' , of 1360 pounds per square foot is questionable and should probably be lower by a factor of about 5. The implications of this for the long term stability of analysis of embankments is discussed below.

Practical Considerations

From the results of the laboratory work carried out on samples of the weathered and unweathered shale, particularly that relating to the nature and rate of weathering of the sound shale, it is considered that the long range stability of embankment fills must be based on the properties of the compacted weathered material.

It is understood that embankment fills are to be constructed to a maximum height of 80 feet with the weathered and unweathered shale and with side slopes of 2 horizontal to 1 vertical proposed. In order to maintain an adequate factor of safety for long term stability it is therefore necessary that some apparent cohesion, C' , be available. The cohesion indicated from the test results would provide a factor of safety in excess of 2 for an

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Page 4.

Practical Considerations (continued)

effective stress analysis of an 80 foot high embankment. However, in view of the uncertainty regarding the validity of this value for cohesion a chart has been prepared as Figure 10 of Appendix II which shows factor of safety with respect to strength as a function of cohesion.

From recent data reported by Dr. A. W. Bishop, it is considered that it would be conservative to assume that the apparent cohesion of the compacted shale would be about 3.0 pounds per square foot or 2 pounds per square inch which is approximately 1/5 of the value indicated in test series 1 and 2. This would provide a minimum factor of safety of about 1.1 for the conditions shown on Figure 10.

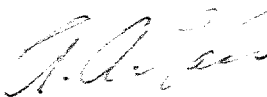
With regard to construction procedures with the weathered and un-weathered shale, little information is available. In the case of the unweathered shale, it has been found that it can be excavated by scrapers with occasional assistance from rippers. The most effective compaction of this material has been achieved using sheepfoot rollers with 200 p.s.i. foot pressure, compacting lifts of 6 inches in thickness in 6 passes. Due to the low plasticity of the material water content must be closely controlled, preferably in the range from optimum to 2 percent less than optimum.

It is considered that it may be possible to separate the unweathered shale by ripping and scraping, but no definite confirmation is known, and it may prove necessary to drill and blast. It is further considered that compaction by sheepfoot roller would be most effective. With regard to wet shrinkage or swelling of the excavated rock, it is recommended that a wet increase in volume of 10 percent after excavation and compaction be assumed for design.

We trust that this information will be sufficient for your present purposes. However, if you have any further questions, we would be pleased to discuss them with you.

Yours very truly,

GEOCON LTD



A. A. Gass, P. Eng.,
Senior Soils Engineer.

AAG/or
S-7050

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APPENDIX I

Preparation of Soil for Testing

Phase I: Bulk Sample Preparation

- (a) We have obtained a bulk sample consisting of about 4 cubic feet of the weathered shale which is to be used as fill material on the Chetoka Expressway. This sample is to be used for determining the compacted strength properties of the weathered shale. The following preparation of the sample is to be carried out before the commencement of testing in order to "homogenize" the material and thus minimize possible scatter of results due to material differences within the sample.
- (b) The total sample is to be spread out and air dried. As soon as it is reasonably dry, the total sample shall be screened through a #9 sieve (Tyler). The material retained on the #9 sieve shall be broken up and again screened, repeating the process as required. Pebbles or obviously unweathered pieces of shale shall be eliminated. The sample shall then be thoroughly mixed and stored in the humid room.
- (c) After sample preparation the following identification tests are to be carried out:
 - (1) Water contents (3)
 - (2) Sieve Analysis on "As-Prepared" Sample, i.e. no effort made to break up material further.
 - (3) Complete Mechanical Analysis - Standard Procedure
 - (4) Tests on Air Dried and Oven Dried Material
 - (5) Specific Gravity, if required.)

Phase II: Compaction Characteristics and Air-Compacted Strengths

- (a) The following compaction tests will be carried out initially:
 - (1) Modified AASHTO
 - (2) Harvard Miniature (3 layers, 25 blows, 40 lb. tamper)
 - (3) Harvard Miniature (10 layers, 25 blows, 40 lb. tamper)
- (b) For all compaction tests, the following procedure will be followed:
 - (1) Tests are to be run at water contents of about 6, 8, 10, 12, 14, 16, 18 percent.
 - (2) All samples should be mixed with the proper proportion of water and allowed to equilibrate for a period of about 12 hours (overnight).
 - (3) Unconfined compression tests are to be run on all Harvard Miniature samples, immediately upon molding. Water contents to be obtained on whole sample.

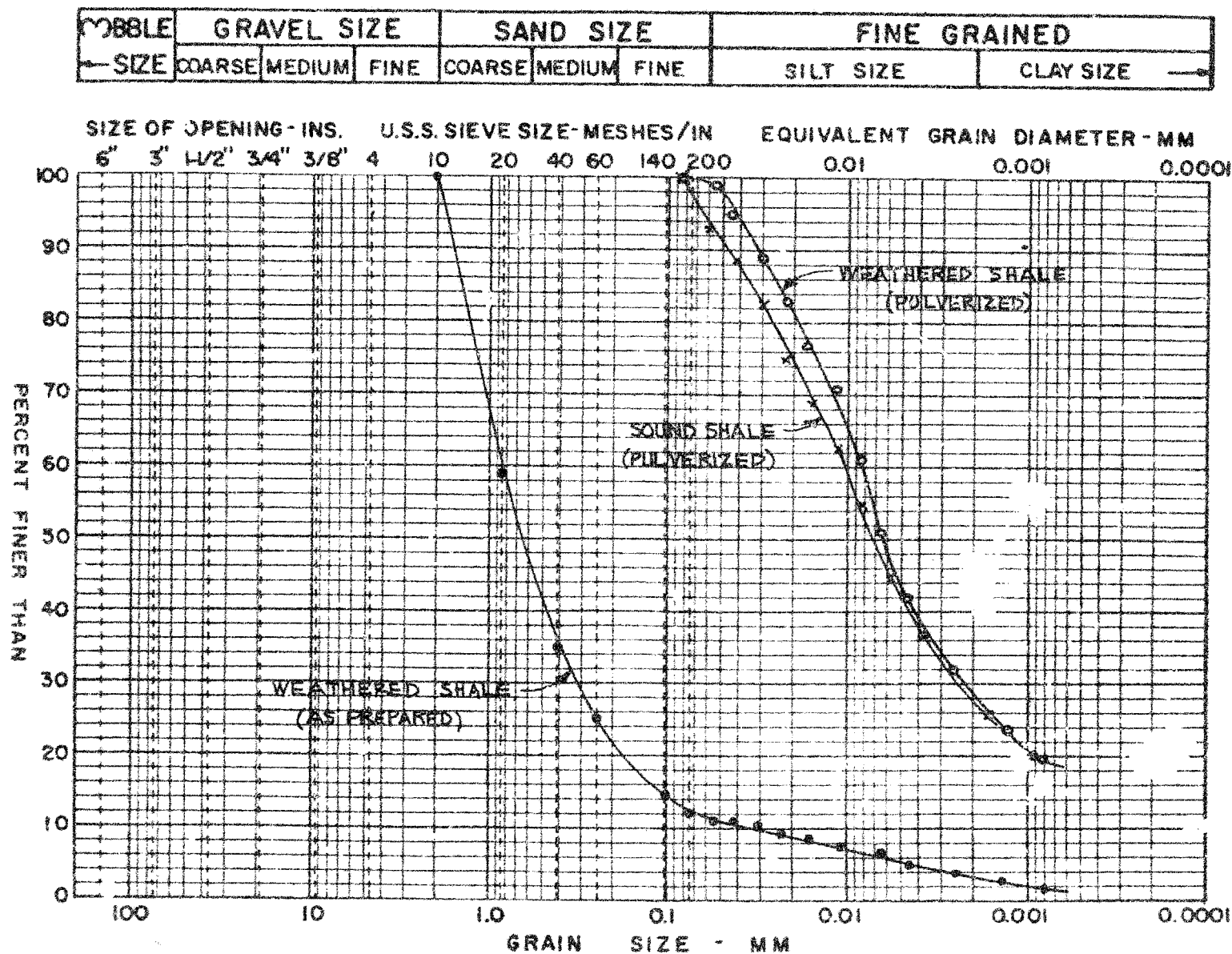
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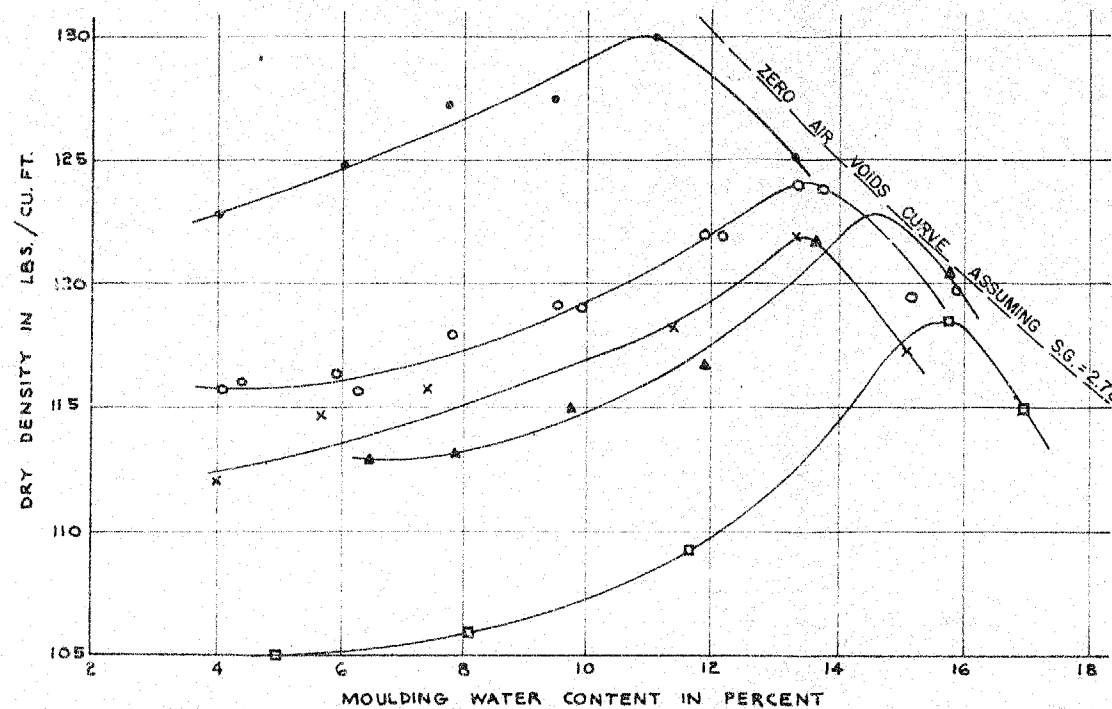
APPENDIX II

Figure

GRAIN SIZE DISTRIBUTION

APPENDIX I
FIGURE 1
PROJECT S7050



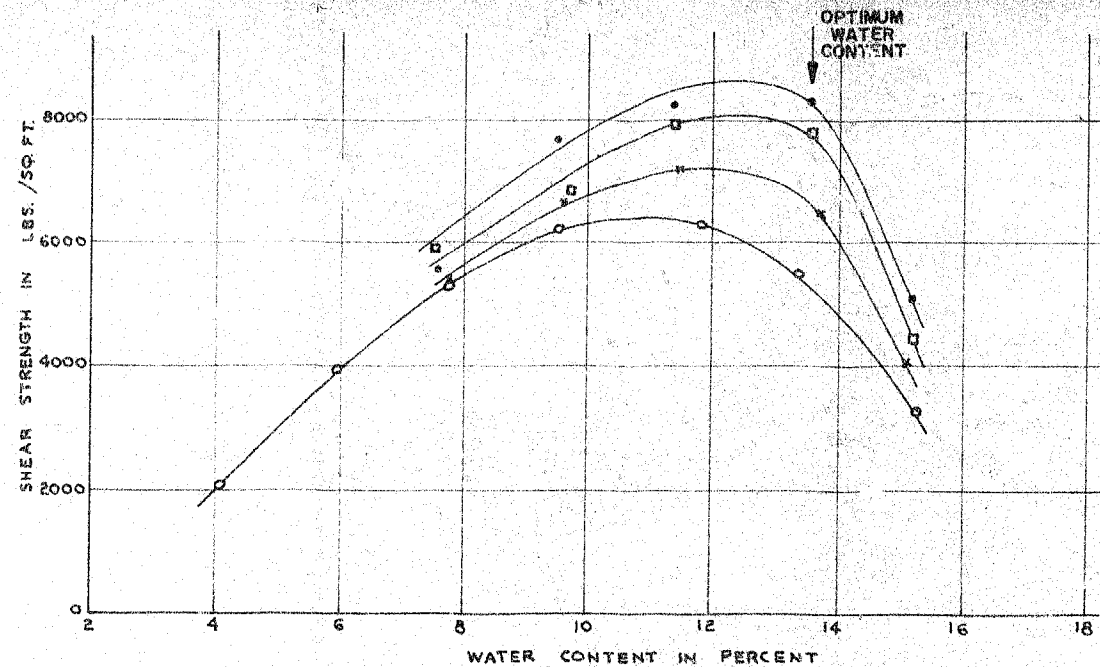


LEGEND		
SYMBOL	TYPE OF TEST	REMARKS
•	MODIFIED A.A.S.H.O.	—
○	HARVARD MINIATURE DEVICE	10 LAYERS - 25 TAMPS / LAYER - 40 LB. SPRING
x	STANDARD A.A.S.H.O.	—
▲	HARVARD MINIATURE DEVICE	5 LAYERS - 25 TAMPS / LAYER - 40 LB. SPRING
◻	HARVARD MINIATURE DEVICE	5 LAYERS - 25 TAMPS / LAYER - 20 LB. SPRING

LIQUID LIMIT — 30.0 %
PLASTIC LIMIT — 20.3 %
PLASTIC INDEX — 9.7 %

COMPACTION - STRENGTH PROPERTIES
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE 3
PROJECT S7050



LEGEND

SYMBOL CHAMBER PRESSURE, (PS.I)

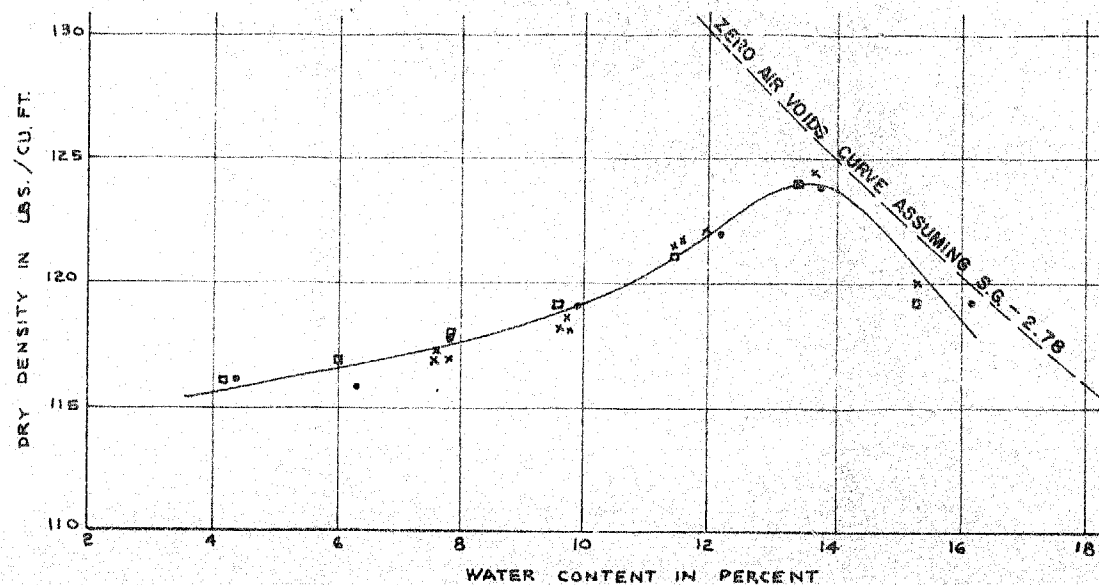
- UNCONFINED
- x 10 PS.I
- 20 PS.I
- 30 PS.I

COMPACTION TECHNIQUE

HARVARD MINIATURE DEVICE
10 LAYERS - 25 TAMPS/LAYER
40 LB. SPRING

LIQUID LIMIT - 30.0 %
PLASTIC LIMIT - 20.3 %
PLASTIC INDEX - 9.7 %

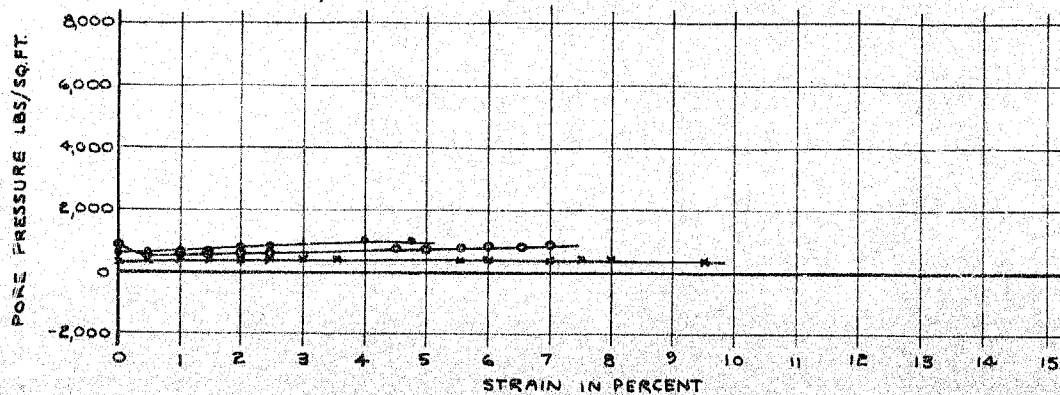
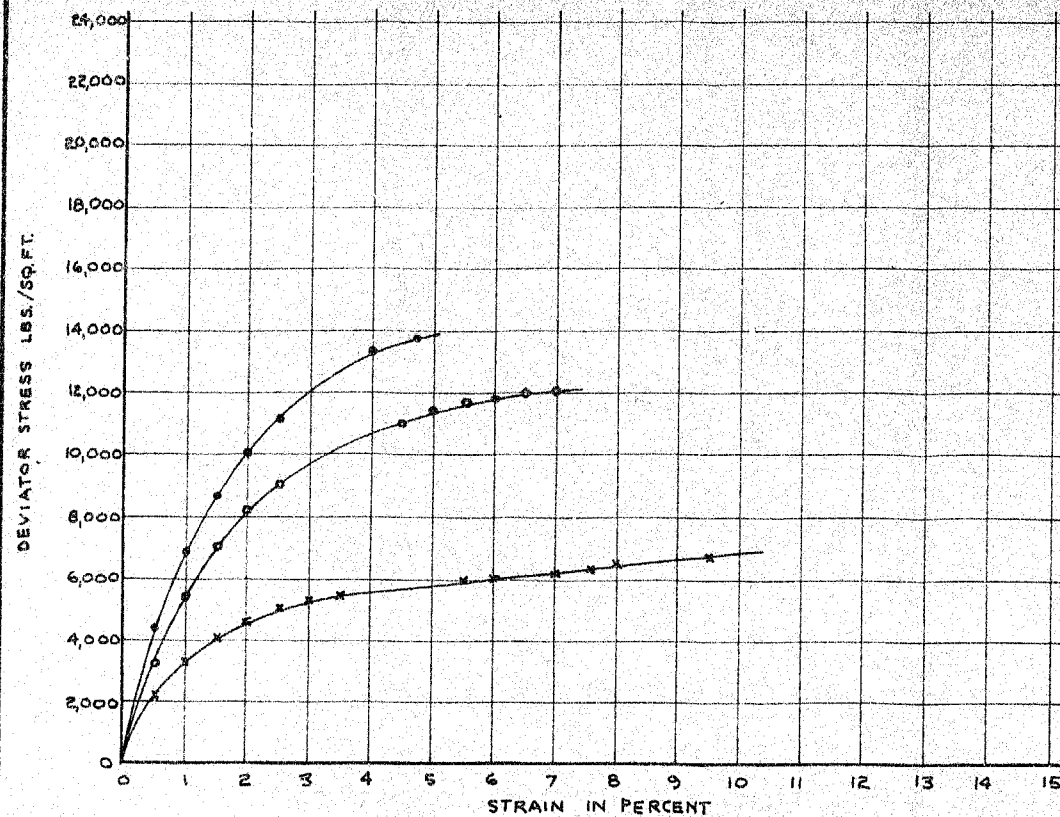
RATE OF AXIAL STRAIN, 2% / MINUTE



GEOCON

SERIES No.2
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

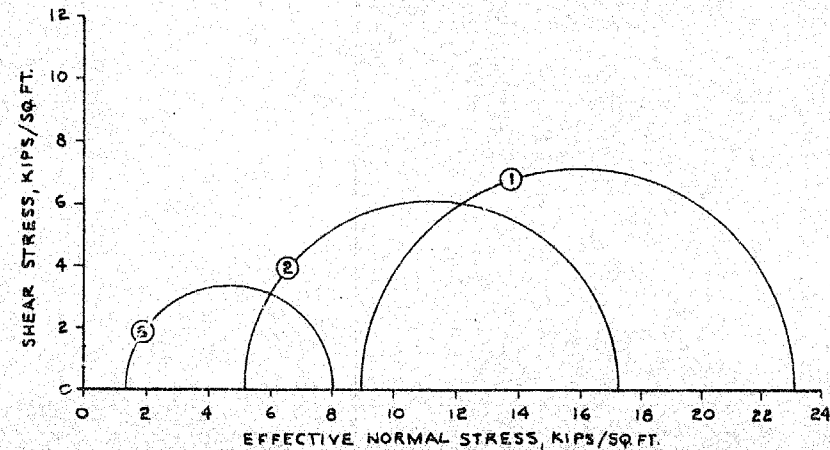
APPENDIX II
FIGURE 5
PROJECT S7050



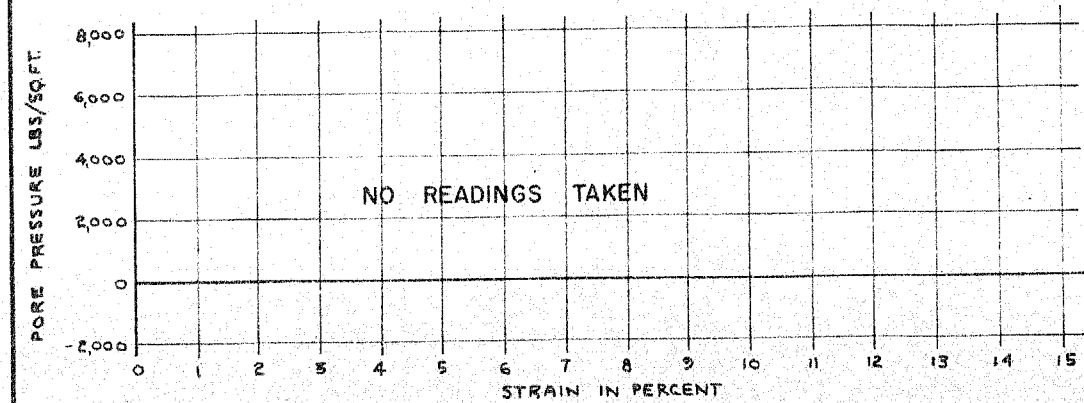
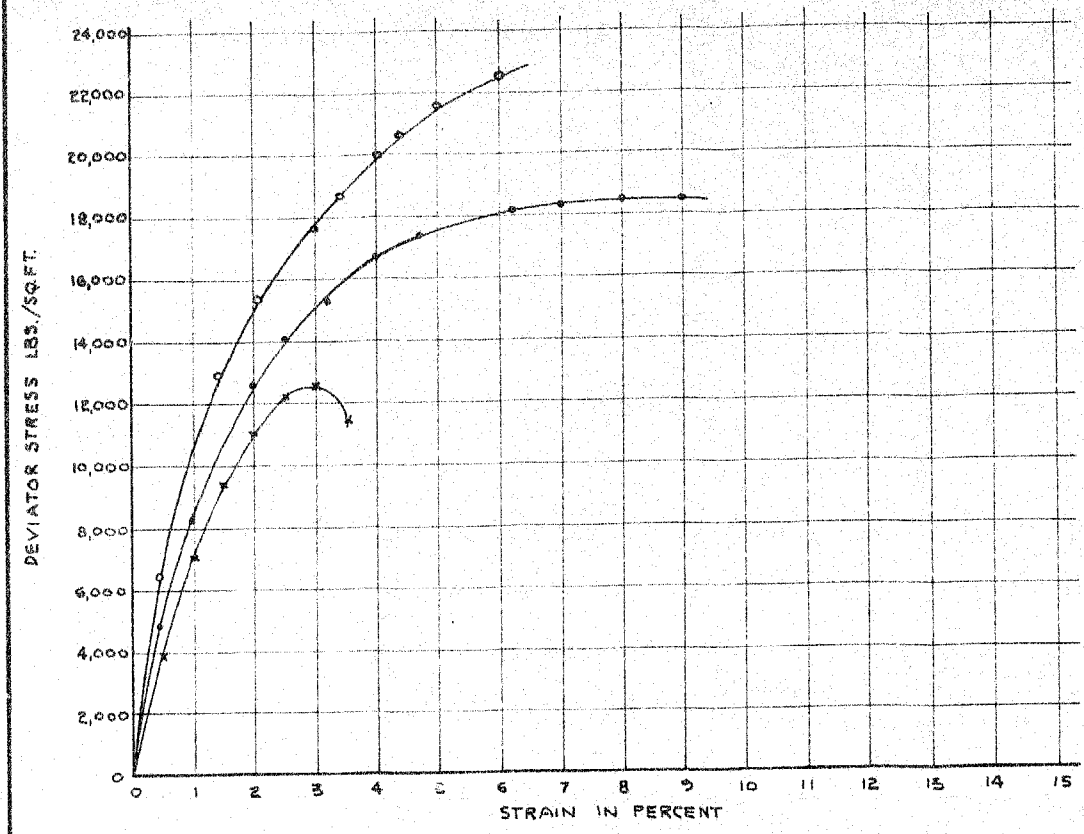
SYMBOL	TEST No.	σ_3 (psf.)	INITIAL CONDITION		FINAL CONDITION
			σ_d (psf.)	W (%)	W (%)
•	①	10200	121.2	11.1	13.2
○	②	6120	120.3	10.6	12.9
x	③	2040	120.5	10.9	14.3

REMARKS:

- 1) FILTER STRIPS & POROUS STONES TOP & BOTTOM
- 2) RATE OF AXIAL STRAIN, 1.4 % / HOUR
- 3) COMPACTION BY HARVARD MINIATURE DEVICE
10 LAYERS-25 TAMPS/LAYER-40*SPRING

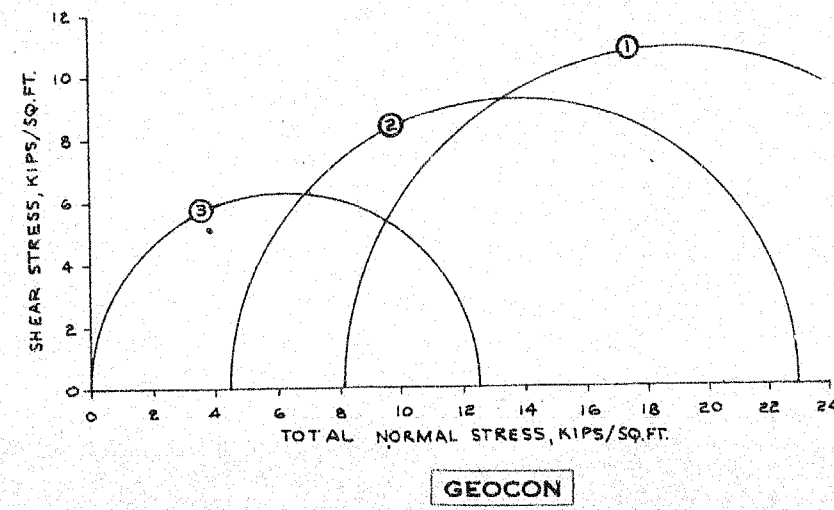


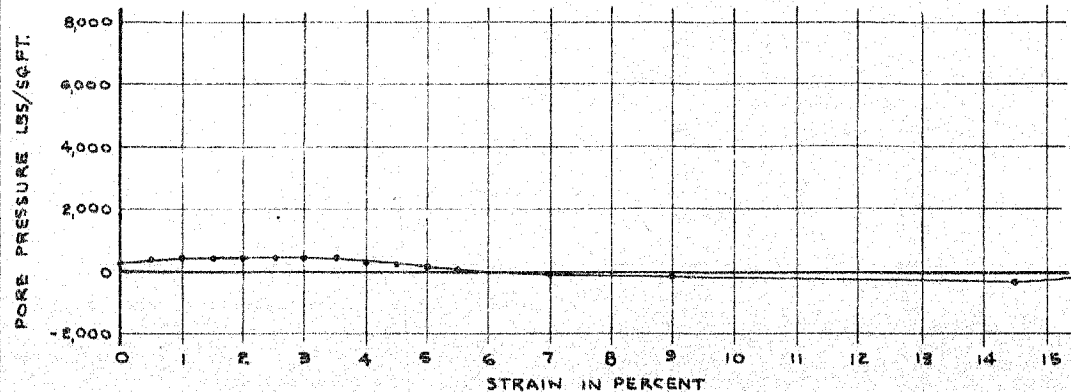
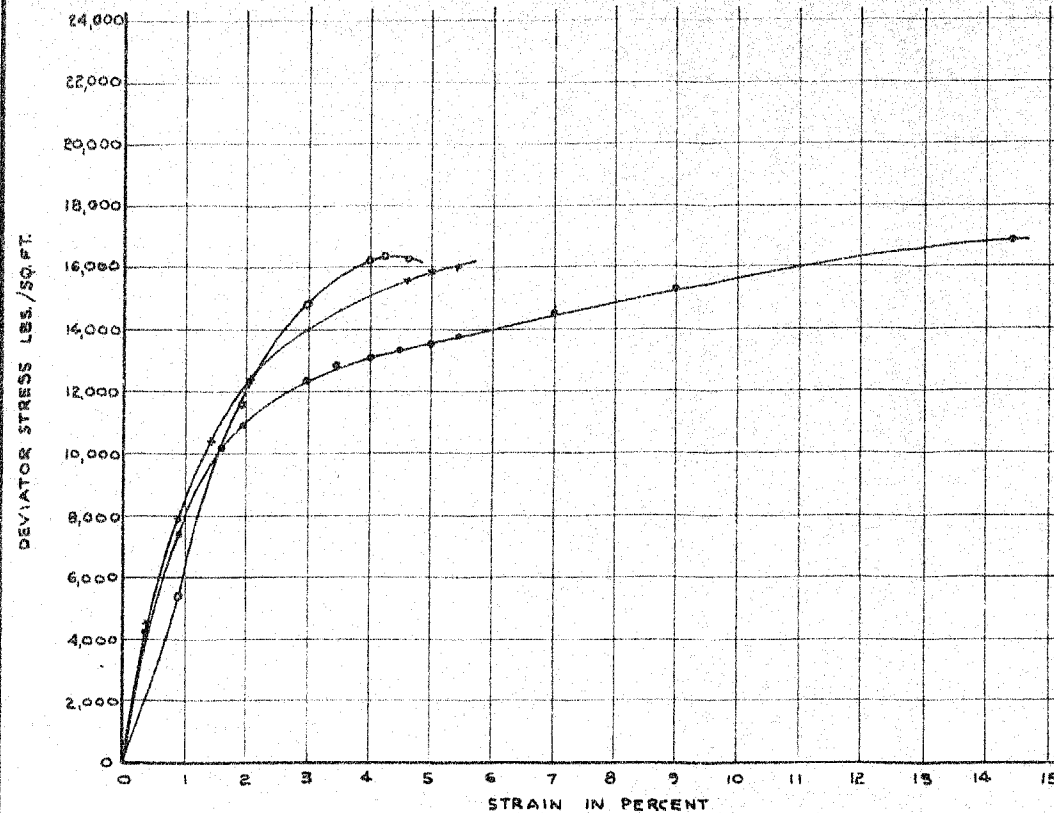
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SYMBOL	TEST No.	σ_3 (psf)	INITIAL CONDITION FINAL CONDITION		
			δd (pcf)	W (%)	W (%)
o	①	8160	124.0	12.1	10.2
•	②	4480	123.4	11.5	11.8
x	③	0	124.5	11.3	11.3

- REMARKS
- 1) NO FILTER STRIPS, LUCITE PLATES TOP & BOTTOM
 - 2) RATE OF AXIAL STRAIN, 1.4 % /HOUR
 - 3) COMPACTION BY HARVARD MINIATURE DEVICE
10 LAYERS-25 TAMPS/LAYER - 40*SPRING





SYMBOL	TEST No.	$\bar{\sigma}_3$ (p.s.f.)	INITIAL CONDITION				FINAL CONDITION		
			$\bar{\epsilon}_d$ (p.c.f.)	W (%)	S (%)	e	W (%)	S (%)	e
o	①	4320	123.2	12.3	79.6	.431	12.3	90.5	.379
x	②	4480	123.2	12.3	87.2	.396	12.3	91.0	.386
•	③	4480	123.4	12.1	86.4	.394	12.5	94.8	.371

REMARKS

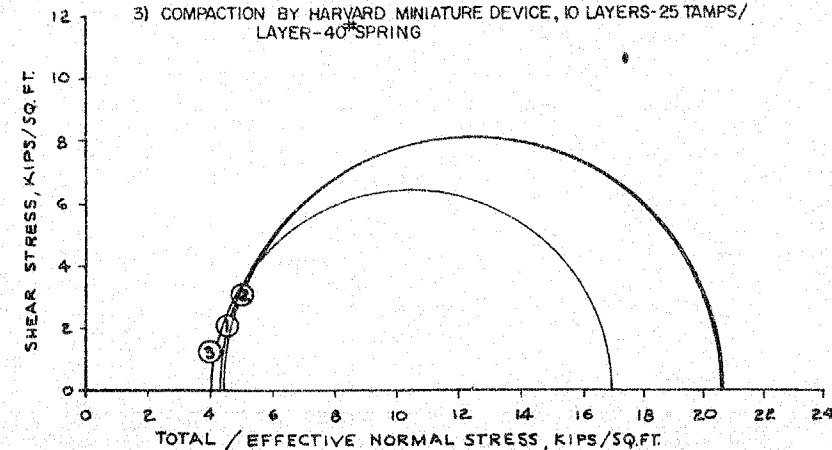
1) CONDITIONS OF TEST,

- ① NO FILTER STRIPS, LUCITE DISKS TOP & BOTTOM
- ② NO FILTER STRIPS, LUCITE DISKS TOP & BOTTOM
- ③ NO FILTER STRIPS, LUCITE DISKS AT TOP, POROUS STONES AT BOTTOM

2) RATE OF AXIAL STRAIN,

- ① 2.0 % / MINUTE
- ② 1.4 % / HOUR
- ③ 1.4 % / HOUR

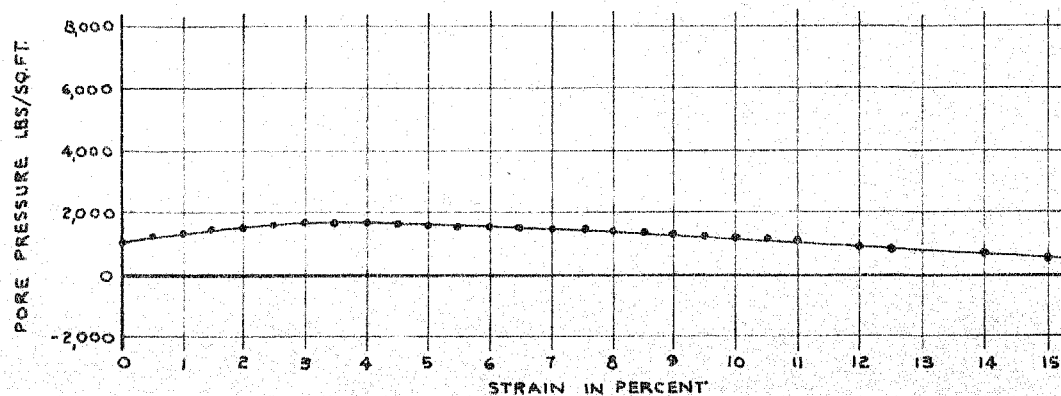
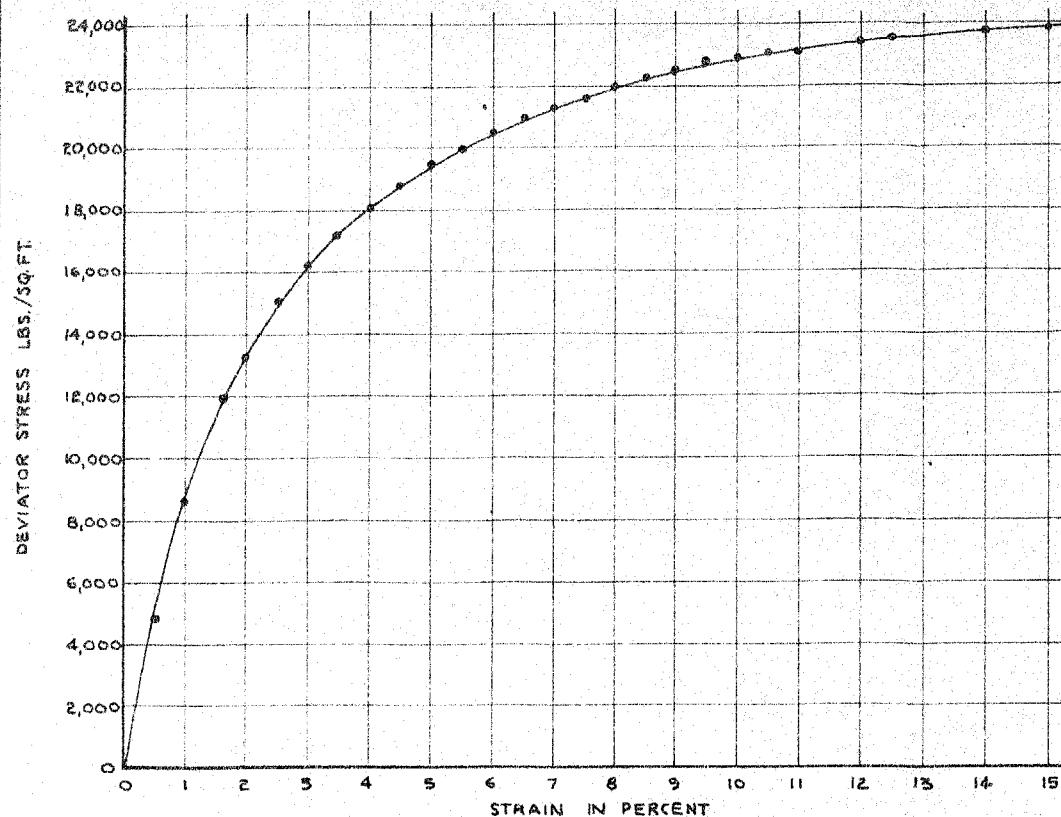
3) COMPACTION BY HARVARD MINIATURE DEVICE, 10 LAYERS-25 TAMPs/
LAYER-40° SPRING



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SERIES No.5
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

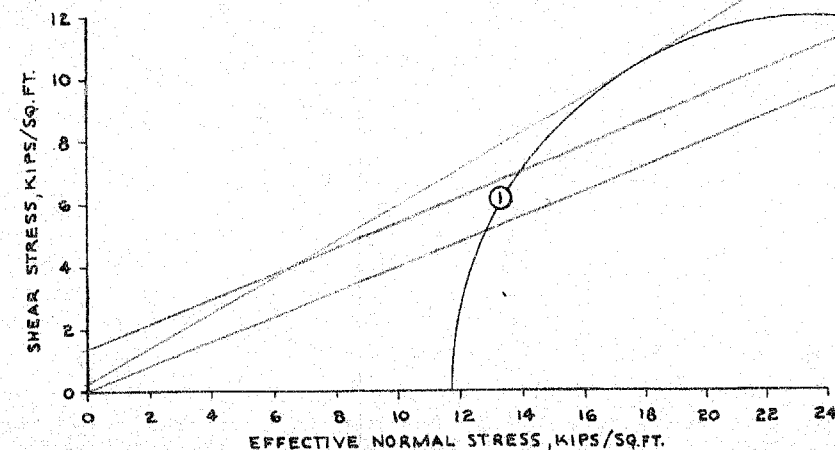
APPENDIX II
FIGURE 8
PROJECT S7050



SYMBOL	TEST No.	$\bar{\sigma}_3$ (psf.)	INITIAL CONDITION				FINAL CONDITION		
			γ_d (p.c.f.)	W (%)	S (%)	e	W (%)	S (%)	e
•	①	12340	123.2	12.3	87.2	.396	14.1	111.0	.356

REMARKS:

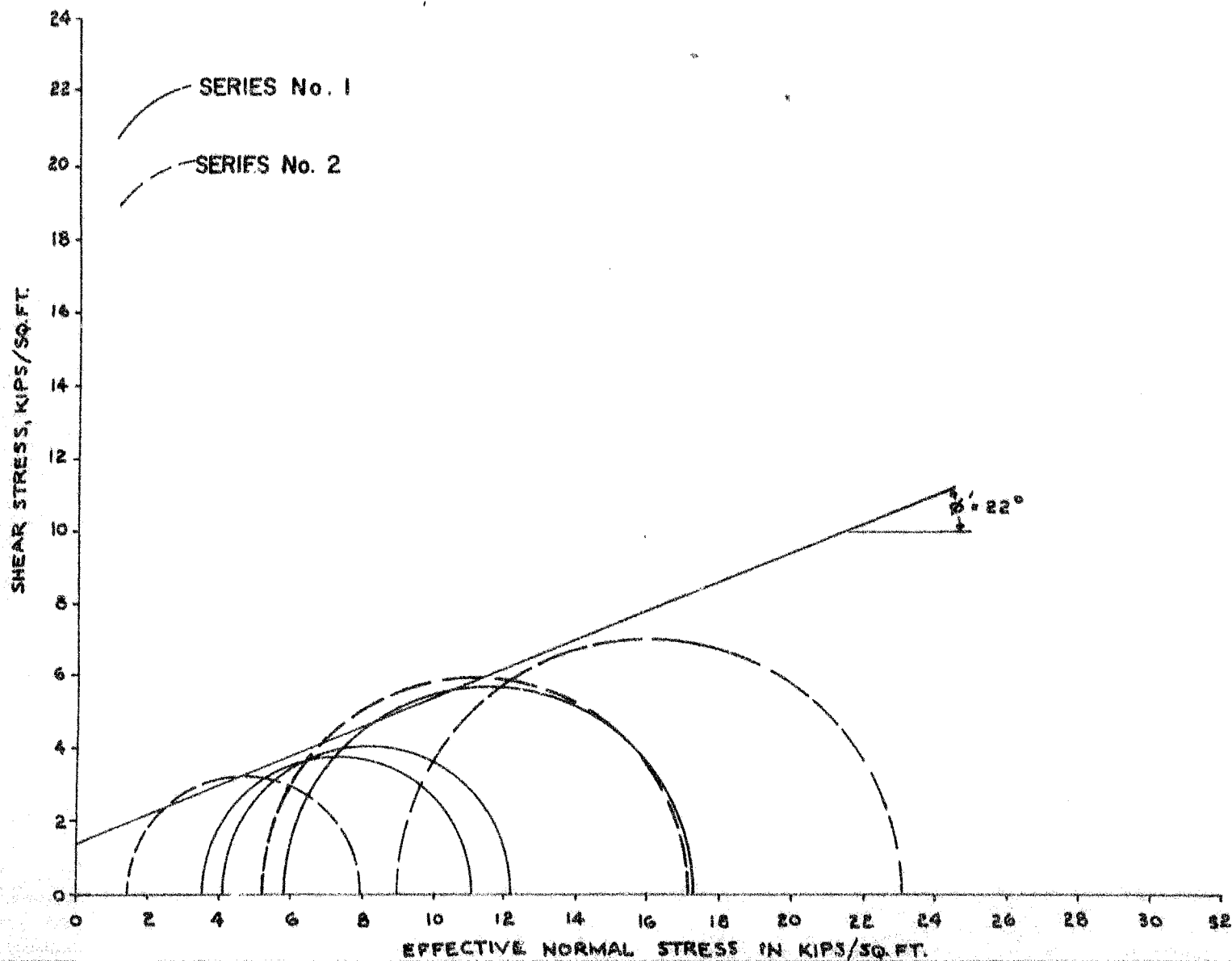
- 1) FILTER STRIPS & POROUS STONES TOP & BOTTOM
- 2) RATE OF AXIAL STRAIN, 1.4 % / HOUR
- 3) COMPACTION BY HARVARD MINIATURE DEVICE,
10 LAYERS - 25 TAMPS/LAYER - 40" SPRING



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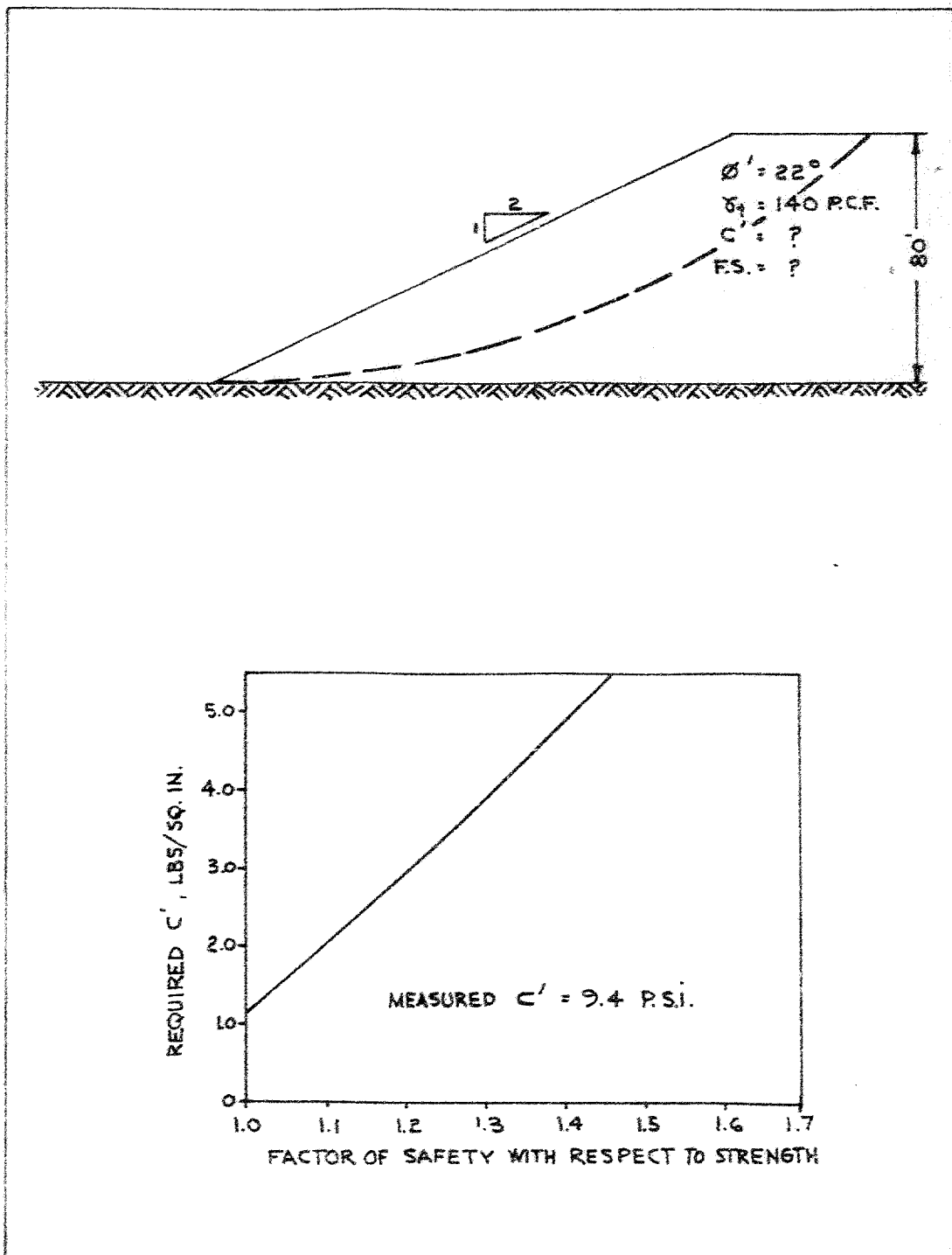
MOHR CIRCLE DIAGRAM
WEATHERED QUEENSTON SHALE
SERIES 1 & 2

APPENDIX II
FIGURE 9
PROJECT S7050



APPARENT COHESION VS FACTOR OF SAFETY
FOR
80' EMBANKMENT

APPENDIX II
FIGURE 10
PROJECT S7050



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180 VALLÉE ST., MONTREAL 18, QUEBEC
TELEPHONE UN. 6-7632

Rexdale, Ontario,
May 8th, 1961.

DISTRICT OFFICES

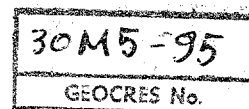
14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8641

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

30M5-95

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.



Re: Laboratory Studies,
Weathered Queenston Shale,
Chedoke Expressway - District 4,
Hamilton, Ontario.

Dear Sirs:

This letter presents the final results of the above study which was authorized by your letter dated April 21st, 1960. The object of the laboratory study was to provide data on the engineering properties of compacted weathered and sound Queenston shale, for the analysis of the stability of proposed highway embankments up to 80 feet in height for the Chedoke Expressway in Hamilton, Ontario.

Preliminary results of the testing were presented in an interim report dated August 19th, 1960. This final report presents the laboratory tests in more detail, together with a discussion of the results.

Department of Highways, Ontario,
May 8th, 1961,
Page 2.

SUMMARY OF RESULTS

The engineering properties of both weathered and pulverized un-weathered Queenston shale were found in the laboratory to be essentially the same. Compaction tests of varying compactive energies were carried out and gave optimum moisture contents ranging from 11 to 16 percent and dry densities ranging from 118 to 130 pounds per cubic foot. For the standard AASHTO test, which from experience is known to simulate most closely the compactive effort that the soil would be subjected to in the field, the optimum moisture content is 13.5 percent and the maximum dry density is 124 pounds per cubic foot.

Several samples were compacted to standard AASHTO density in the Harvard miniature device at moulding water contents of 9.6, 11.5, 13.6 and 15.2 percent. Undrained triaxial tests were performed on these samples and the results, when plotted on Mohr circle diagrams, showed the apparent cohesion to decrease from 5000 to 2500 pounds per square foot as the moulding water content increased from 9.6 to 15.2 percent. The apparent ϕ was 15 and 17 degrees obtained from the series at moulding water contents of 9.6 and 15.2 percent respectively but reached a maximum of 23 degrees for the series moulded at optimum moisture content of 13.6 percent. When the samples were saturated after compaction, the undrained shear strength was 2100 pounds per square foot for moulding water contents of 11.5, 13.0 and 15.1 percent.

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Page 3.

SUMMARY OF RESULTS (continued)

The so-called effective stress parameters obtained from the unsaturated Queenston shale, compacted at one or two percent below optimum moisture content, were c' of about 1500 pounds per square foot and ϕ' of about 22 degrees. Because of the difficulty of measuring the true pore air pressures in unsaturated soils these results are believed to be questionable. A series of consolidated undrained tests with pore pressure measurements run on material moulded at 13.5 percent and then fully saturated gave a c' of 500 pounds per square foot and a ϕ' of 31 degrees.

Tests to determine the variation of \bar{B} with degree of saturation bear out that \bar{B} increases rapidly as the degree of saturation increases beyond 90 percent.

GENERAL PROPERTIES

The Queenston formation is a marine deposit of Ordovician age. It is of uniform composition over a large area and consists of dark red brown shale interspersed with occasional irregular bands or pockets of olive green calcareous siltstone. It is highly fissile, and breaks frequently and easily parallel to the bedding planes. The latter properties render the shale particularly susceptible to weathering, and generally a zone of weathering up to 10 or 12 feet in thickness overlies the sound rock.

Department of Highways, Ontario,
May 8th, 1961,
Page 4.

GENERAL PROPERTIES (continued)

Geological literature indicates that the shale is composed primarily of quartz and illite with relatively minor amounts of feldspar, calcite, and hematite. The average of two specific gravity measurements on both the weathered and unweathered material is 2.78.

The effect of weathering on the engineering properties of the shale can be inferred from the results of the various index tests carried out on samples of both the weathered and sound shale. As indicated by the grain size distribution curves given on Figure 1 of Appendix II there is no significant difference between the two materials after pulverization with a mortar and pestle in accordance with standard soil testing procedures. Atterberg limits obtained from both materials also indicated no significant difference in the composition of the two materials; liquid limits of 30.0 and 27.5 were obtained for the weathered and unweathered shale respectively, with corresponding plasticity indices of 9.7 and 8.6. The corresponding liquid limit and plasticity index for the oven-dried, weathered shale were 28.8 and 8.5. These results confirm that the weathering process in the Queenston shale has been predominantly by mechanical as opposed to chemical processes, and imply that, for the problem under consideration, over the long term the engineering properties of both weathered and unweathered shale will be essentially similar.

Department of Highways, Ontario,
May 8th, 1961,
Page 5.

GENERAL PROPERTIES (continued)

Drilled cores of the sound rock split along the bedding planes almost immediately on exposure to air. When these drill cores were immersed in water, they immediately flaked along bedding planes and broke down within 10 hours. In contrast, bulk samples immersed in water exhibited little signs of flaking after 6 months immersion. It is possible that incipient fracturing of the core occurred during drilling, which resulted in crumbling on immersion. Because of the tendency of the rock to break-up under certain conditions both on exposure to air or immersion in water, it was considered appropriate to pulverize samples for the laboratory tests to determine the engineering properties for design.

The mechanical effort required to break down the weathered rock fragments in the laboratory was observed to be comparable to that required to pulverize a silty clay glacial till of hard consistency.

The total unit weight of the sound shale determined on a sample of waxed core was 163 pounds per cubic foot at a water content of 4.3 percent, which indicates complete saturation. The computed dry density was 156 pounds per cubic foot. The average water content of several samples of weathered shale was 11.5 percent.

Department of Highways, Ontario,
May 8th, 1961,
Page 6.

GENERAL PROPERTIES (continued)

In view of the similarity in the significant properties of the sound and weathered shale, as mentioned earlier, further detailed testing was carried out on a bulk sample of the weathered shale. This was prepared for testing in accordance with the procedure outlined in Appendix I.

COMPACTION PROPERTIES

Both standard and modified AASHO compaction tests were carried out on the prepared bulk sample of the weathered shale. A series of three compaction curves were also obtained using the Harvard miniature device with various compaction efforts. The results of compaction tests are shown on Figure 2 in Appendix II. It may be observed that the compaction curves obtained follow a rational pattern with maximum dry density increasing with compactive effort and optimum moulding water content decreasing with increased compactive effort. With a compactive effort of 10 layers and 25 blows per layer with a 40 pound spring, the compaction curve using the Harvard device approximates to the standard AASHO curve. The maximum dry density was about 123 pounds per cubic foot at a moulding water content of about 13 percent.

Department of Highways, Ontario,
May 8th, 1961,
Page 7.

COMPACTION PROPERTIES (continued)

From previous experience with field compaction studies on weathered Queenston shale in the Bronte-Burlington area, excavated by rippers and scrapers, placed in layers and compacted by sheepsfoot rollers, it is known that the standard AASHO compaction curve adequately represents the degree of compaction which would be obtained in the field with similar procedures. This field compaction was attained using 6 passes of sheepsfoot rollers with 200 psi foot pressure on lifts about 6 inches in thickness. The insitu, and thus the placement, water content ranged between 10 and 12.5 percent.

On the basis of the above results, the compacted strength properties were studied on samples prepared in the Harvard device with a compactive effort of 10 layers and 25 tamps with a 40 pound spring.

COMPACTED STRENGTH PROPERTIES

General

During construction of a rolled fill embankment, each layer of fill is compacted in as nearly an identical manner as possible, at about the same placement water content and to about the same unit weight. Immediately after compaction the soil in each layer is under no external load. However, capillary pressures or negative pore water pressures may

Department of Highways, Ontario,
May 8th, 1961,
Page 8.

COMPACTED STRENGTH PROPERTIES (continued)

stress the soil. These pressures will, for practical purposes, be equal in all directions in the soil layer. Hence, no shear stresses exist. As the height of fill is increased above a layer, normal and shear stresses will be applied causing volume change and changes in pore air and pore water pressures.

Thus, in the embankment, the shear stresses for each layer start at about the same void ratio, but as shear stresses are increased by the addition of fill some consolidation will occur. In general, non-hydrostatic conditions would exist in the field.

To simulate field conditions for unsaturated samples, a sample is placed in the triaxial apparatus and compressed, without drainage, to variable void ratios. In applying chamber pressures to bring all samples to different void ratios, pore air and pore water pressures will be developed but no change in water content takes place. The effective lateral stress will therefore be different for each sample. In this manner, when deviator stress is applied and pore pressures are measured, a strength envelope on the basis of effective stresses can be obtained for an unsaturated sample of soil which is comparable to field conditions to the extent that shear started at about placement water content.

COMPACTED STRENGTH PROPERTIES (continued)

General (continued)

The criterion of failure adopted in the laboratory tests is that of maximum deviator stress.

Test Results

The shear strength parameters, both total and effective, were investigated by undrained triaxial tests. The results of these tests are discussed below:

1. Total stress strength properties expressed as $\frac{\sigma_1 - \sigma_3}{2}$ are plotted against moulding water content on Figure 3 of Appendix II. This figure shows typical convex curves with the maximum deviator stress occurring at a water content slightly less than that for maximum dry density, in general about 12 percent as compared to 13 percent. The figure also shows that with an increase in the lateral pressure, the deviator stress becomes greater. This is due to compression of the air in the voids permitting the intergranular stresses to increase.

2. Stress-strain curves for the above unconfined and quick triaxial tests are given on Figures 4 and 5. These show that the strain at failure ranges from less than 1 percent for dry samples to greater than 15 percent for very wet samples. Thus the failure strain is a function of the moulding water content.

COMPACTED STRENGTH PROPERTIES (continued)

Test Results (continued)

3. The above unconfined and triaxial tests were grouped at different moulding water contents and the total stress results of four series are given on the Mohr circle diagrams, Figures 6 to 9. The tests were run at a strain of 2 percent per minute. The results are summarized below:

Moulding Water Content, percent	9.6	11.5	13.6	15.2
Apparent ϕ	15°	18°	23°	17°
Apparent c (kips/sq.ft.)	5	4.5	3.5	2.5

As can be seen from these results, it appears as if the apparent cohesion intercept c decreases, while the value of the apparent angle of shearing resistance ϕ varies, reaching a peak of 23 degrees at a moulding water content of 13.6 percent. From theory, it would be expected that the angle of shearing resistance obtained from unconfined and undrained triaxial tests would be more or less constant at low moulding water contents and then approach zero at higher moulding water contents where full saturation is approached. For any given moulding water content, the angle of shearing resistance should also decrease at higher cell pressures, because the degree of saturation will increase as the air in the voids is compressed. With the cell pressures used in this investigation, this effect was not too noticeable.

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COMPACTED STRENGTH PROPERTIES (continued)

Test Results (continued)

4. During construction of an embankment or a dam, the top surface at any one time may become saturated as a result of heavy precipitation. To simulate this condition in the laboratory, samples were saturated after compaction. No consolidation was allowed before shearing the samples and the shear strength ranged from 2100 to 2200 pounds per square foot as shown on Figure 10 for samples moulded at 11.5, 13.0 and 15.1 percent water content. This undrained shear strength is believed to represent the minimum value.

5. The results of undrained unconsolidated triaxial tests with pore pressure measurements carried out on unsaturated samples are shown on Figures 11, 12 and 13. The rate of strain in these tests was 1.4 percent per hour with a total time to failure of 4 to 5 hours. Figure 13 shows the average of two series, one moulded at about 12 percent water content and the other at slightly less than 11 percent. The best fit to these circles gives a ϕ' of 22 degrees and a c' of about 1.6 kips per square foot. It should be noted that the measured pore pressures were higher for samples moulded at the higher water content.

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COMPACTED STRENGTH PROPERTIES (continued)

Test Results (continued)

The pore pressures measured in these tests could be too high for several reasons. For example, since the pore pressures were measured at the base of the sample only and, with the rate of strain employed, some pore pressure gradient could have existed in the sample. In addition, during the testing programme, some softening of samples was observed, which would also affect the pore pressure.

In an attempt to explain the observed softening, a series of tests were carried out on unsaturated samples with the top and base of the samples completely sealed. These tests were run at a strain of 1.4 percent per hour with failure occurring between 4 and 5 hours from the start of the test. The results are shown on Figure 14. It may be noted that the apparent ϕ is about the same as ϕ' shown on Figure 13, but c is much greater than c' .

In a further series of tests on unsaturated samples shown on Figure 15, drainage conditions and strain rates were altered as shown and the strength results found to be almost identical. By a process of elimination it was deduced from this series of tests and the preceding set that the filter strips were the main medium through which the additional moisture was introduced to the sample.

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COMPACTED STRENGTH PROPERTIES (continued)

Test Results (continued)

A critical study of the standard testing technique showed that even in the short time that the routine de-airing procedure was in progress, the samples because of their affinity for moisture were in actual fact absorbing significant amounts of water.

6. To eliminate the effect of pore air pressures which are difficult to assess and also to determine the probable lower limit of effective cohesion, c' , it was decided to carry out a series of tests on fully saturated samples. It is understood that such a condition would in fact occur in practice because the lower part of the embankment would be fully saturated in places. The results of the tests in question are shown on Figure 16.

The saturation was accomplished by subjecting the sample to an upward hydraulic gradient of 30 psi for 24 hours followed by consolidation and then a back pressure of 24 psi. The moulding water content of the sample was about 13.5 percent.

The results of the triaxial testing are plotted on a stress path plot as well as on a Mohr's Circle diagram, and give an effective

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Page 14.

COMPACTED STRENGTH PROPERTIES (continued)

Test Results (continued)

angle of shearing resistance of 31 degrees and an effective cohesion intercept of 500 pounds per square foot.

On Figure 16A are given the results of two earlier tests carried out on saturated samples. Since these tests were carried out on samples compacted at a lower moulding water content, the results of the two series of tests are not directly comparable. Due to the narrow stress range represented by the two Mohr's circles on Figure 16A, a reliable envelope cannot be drawn.

DISCUSSION

From the results of the laboratory work carried out on samples of the weathered shale, particularly that relating to the nature and rate of weathering of the sound shale, it is considered that the long term stability of embankment fills will be controlled by the properties of the weathered material as tested.

It is understood that the embankment fill is to be constructed to a maximum height of 80 feet using the weathered and unweathered shale and having side slopes of 2 horizontal to 1 vertical.

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DISCUSSION (continued)

It is also understood that a portion of the embankment rests on a soil stratum with a shear strength less than that of the embankment fill. However, it is beyond the scope of this report to consider this underlying material and it has been assumed for stability check calculations discussed later, that the embankment rests on a competent bearing stratum.

Literature published during the time of this investigation (Factors Controlling the Strength of Partly Saturated Cohesive Soils by Bishop, Alpan, Blight and Donald) indicates that the cohesion intercepts obtained from tests on partially saturated compacted clay shale are too large where standard porous stones are used and/or where measurements of pore pressure are taken at the base of the sample for tests run at the standard time to failure with drainage from the base only. The recommended method for obtaining cohesion intercept with the use of standard laboratory equipment and techniques is to saturate the samples before testing.

Considering available data published for other tests on unsaturated compacted shales of similar plasticity, it is possible that the effective cohesion intercept could be as low as 300 to 500 pounds per square foot when pore air pressures are considered. As shown on

DISCUSSION (continued)

Figure 16, a value for c' of 500 pounds per square foot was obtained from the tests on saturated samples, which is in agreement with this data. Reference is also made to Figure 17, which indicates the decreasing trend in cohesion intercept with increasing moulding water content. Considering the specific case of a high embankment, where complete saturation of the base of the embankment could take place, it is conservative to assume that c' for this material when compacted as described would have a minimum value of 300 pounds per square foot.

From the data obtained, it is considered that an effective angle of shearing resistance, ϕ' , of 30 degrees and an effective cohesion, c' of 300 pounds per square foot should be taken as the effective stress parameters in stability computations. On this basis, the computed factor of safety of the 80 foot high embankment (with side slopes of 2 to 1 and resting on a competent foundation) would be 1.6. If the c' value is neglected, the factor of safety obtained is 1.2.

The end of construction case is generally the most critical for a compacted fill embankment. This is especially true in the present case where the permeability of the compacted shale is low and where large pore pressures can be built up during construction.

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DISCUSSION (continued)

In order to estimate the pore pressures that would be set up, laboratory testing was carried out to determine the variation of pore pressures with degree of saturation. The results are shown on Figure 18 where the coefficient B is plotted versus the degree of saturation. It can readily be seen that B increases rapidly after 90% saturation. The factor \bar{B} is slightly less than and varies proportionately with B. In this study the two factors were taken as equal. A plot of factor of safety versus pore pressure coefficient \bar{B} is shown on Figure 18 and this shows the great effect pore pressure will have on stability.

The total stress analysis can also be used to check the end of construction case. However the choice of undrained shear strength for use in design is subject to considerable judgement. For tests performed under ideal conditions in the laboratory, that is, no access to additional water, the results from Figures 11 to 13 give a c of 4000 pounds per square foot and a ϕ of 20 degrees. Using these parameters, the factor of safety is well in excess of 5.0. However, this shear strength is by no means conservative. The value of 2100 pounds per square foot for undrained shear strength as found in the saturated tests would result in a factor of safety of about 1.4 for an 80 foot embankment with slopes and foundations as before. However, this shear strength

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DISCUSSION (continued)

value is conservative for this case because no consolidation whatsoever has been considered.

We trust that this letter report, which was checked by Mr. M.A.J. Match, P. Eng., adequately covers the required soils engineering properties of the compacted shale for embankment design purposes. We would be pleased if you would give us a call, however, should you wish to discuss any aspects further, or if we can be of further assistance in the use of this information in design.

Yours very truly,

GEOCON LTD

F. J. Heffernan

F. J. Heffernan,
Senior Soils Engineer.

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APPENDIX I

PREPARATION OF SOIL FOR TESTING

A bulk sample consisting of about 4 cubic feet of weathered shale, which is to be used on the Chedoke Expressway, was obtained. This material was used for determining the compacted strength properties of the weathered shale. The following preparation of the sample was carried out before the commencement of testing in order to "homogenize" the material and thus minimize possible scatter of results due to material differences between, and within, the samples.

The total sample was spread out and air dried. After it was reasonably dry, the total sample was screened through a #9 sieve (Tyler). The material which was retained on the #9 sieve was broken up and re-sieved until all the lumps were eliminated. Pebbles or obviously un-weathered pieces of shale were removed from the sample. The sample was then thoroughly mixed and stored in the humid room.

In the preparation of samples for compaction tests the proper proportion of water was added and the samples were allowed to reach equilibrium for a period of about 12 hours (overnight). However, triaxial or unconfined compression tests were run on samples prepared in the Harvard miniature mould immediately upon moulding and the water contents were obtained on the whole sample in most cases.

APPENDIX II

FIGURES

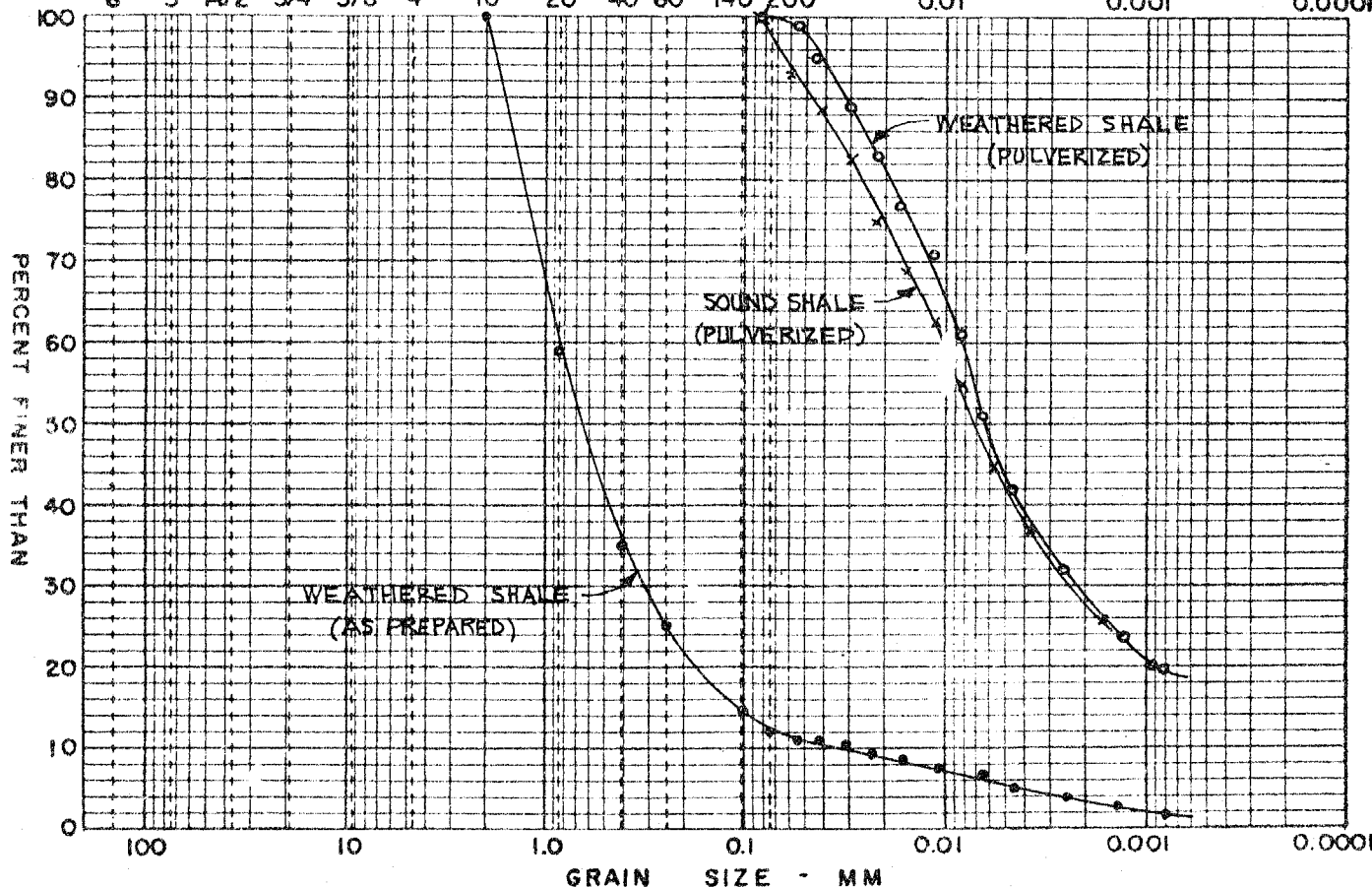
GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 1
PROJECT S7050

COBBLE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	
← SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE →

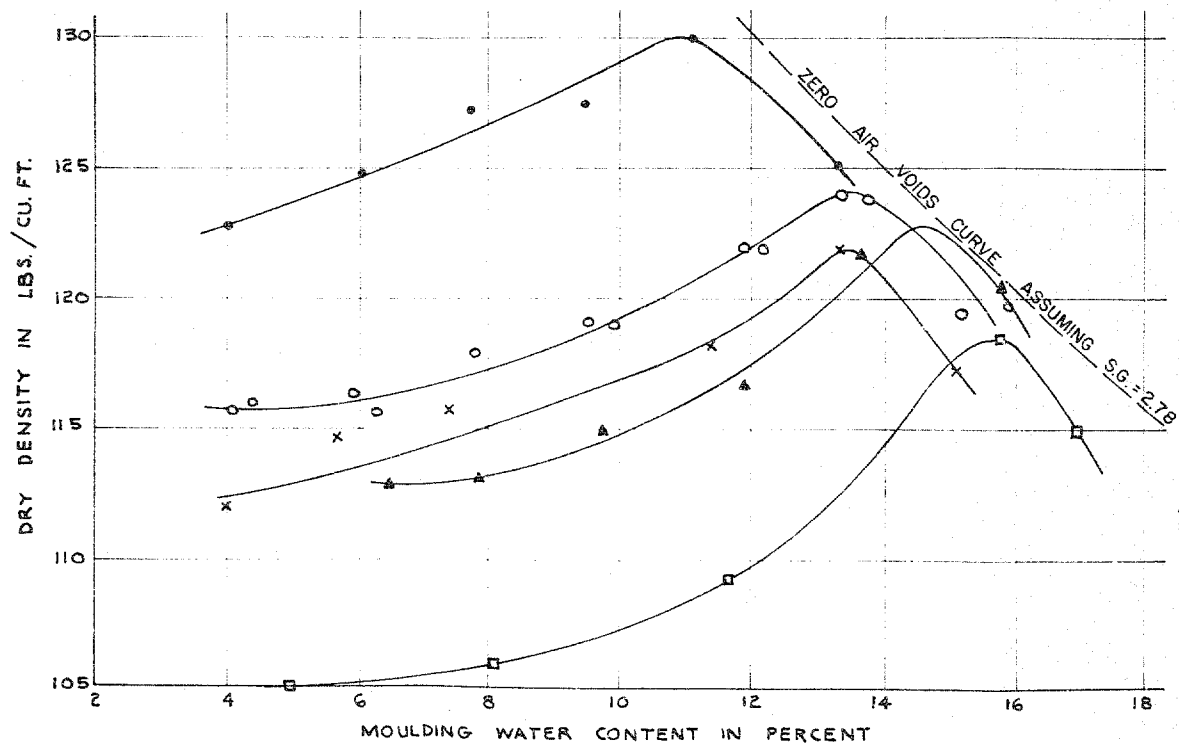
SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN EQUIVALENT GRAIN DIAMETER - MM

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 140 200 0.01 0.001 0.0001



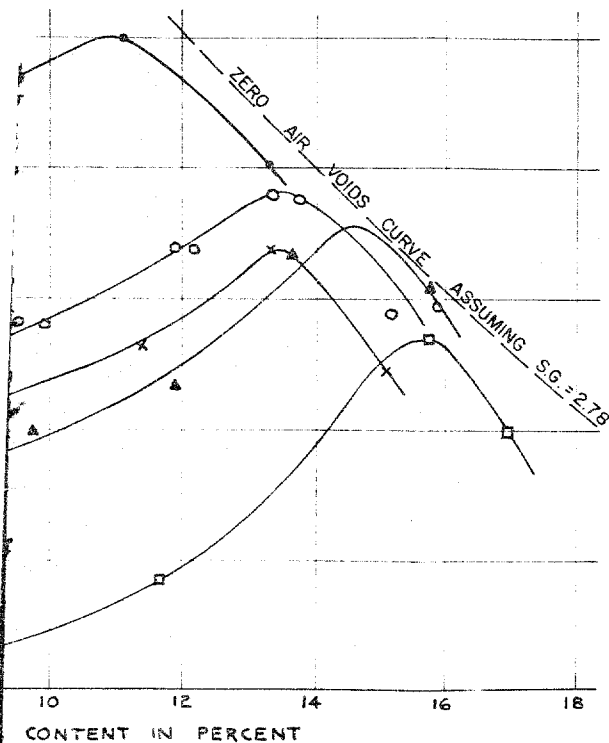
M.I.T. GRAIN SIZE SCALE

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COMPACTION PROPERTIES
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE 2
PROJECT S7050



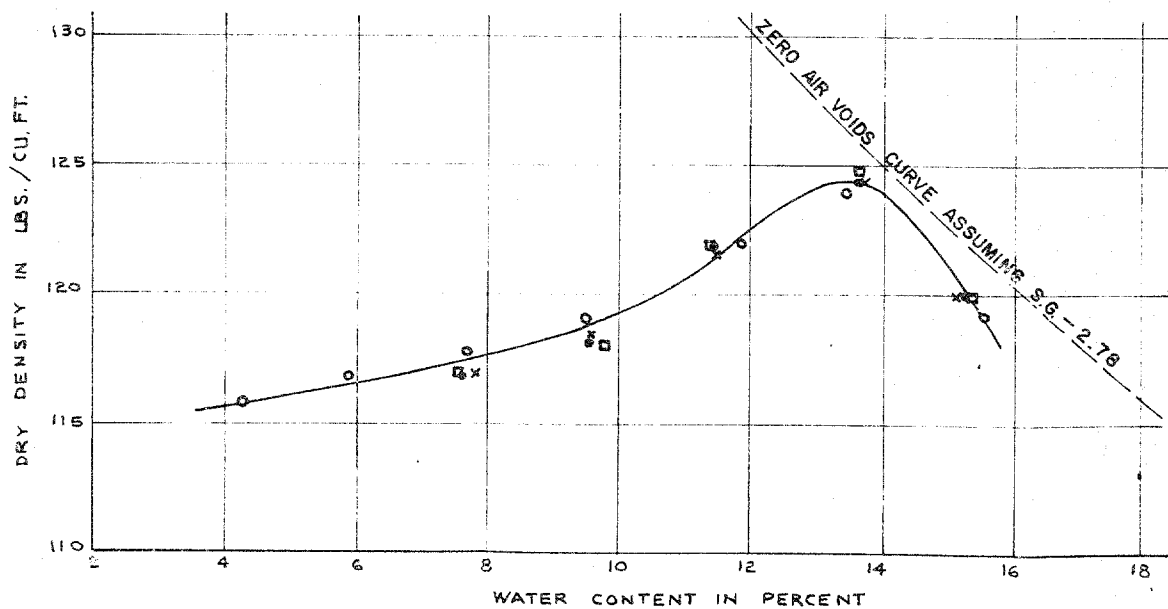
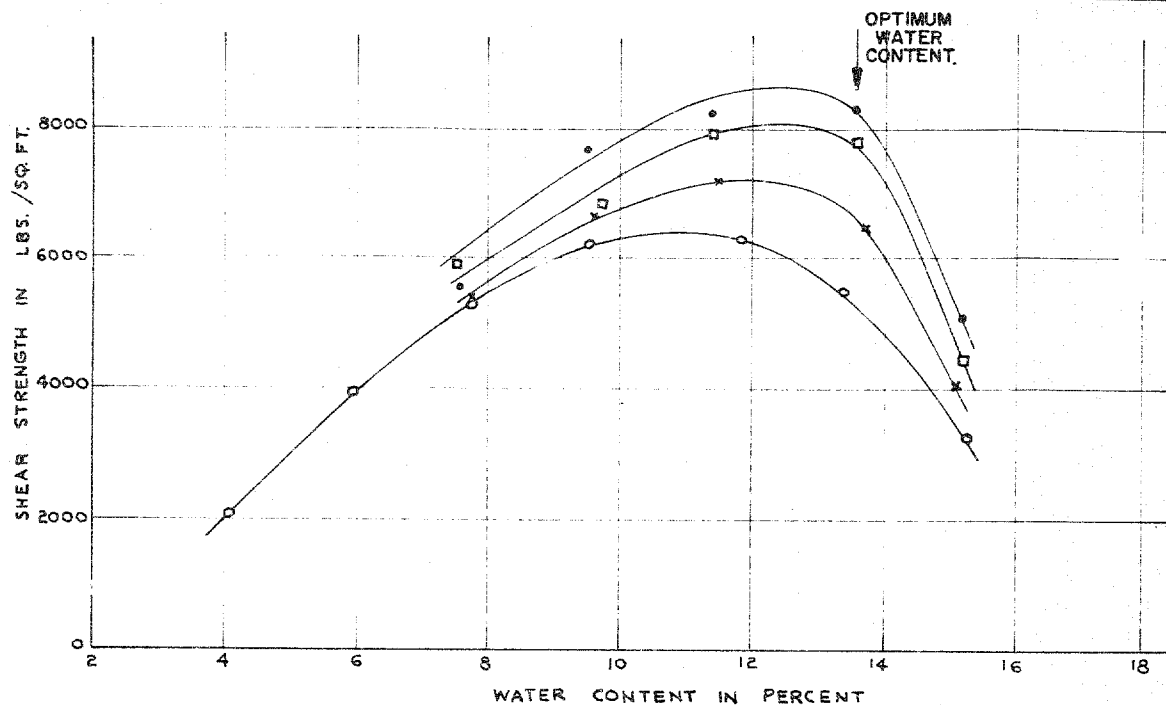
LEGEND		
SYMBOL	TYPE OF TEST	REMARKS
•	MODIFIED A.A.S.H.O.	—
○	HARVARD MINIATURE DEVICE	10 LAYERS - 25 TAMPs/ LAYER - 40 LB. SPRING
×	STANDARD A.A.S.H.O.	—
△	HARVARD MINIATURE DEVICE	5 LAYERS - 25 TAMPs/ LAYER - 40 LB. SPRING
□	HARVARD MINIATURE DEVICE	5 LAYERS - 25 TAMPs/ LAYER - 20 LB. SPRING

LIQUID LIMIT — 30.0 %

PLASTIC LIMIT - 20.3 %

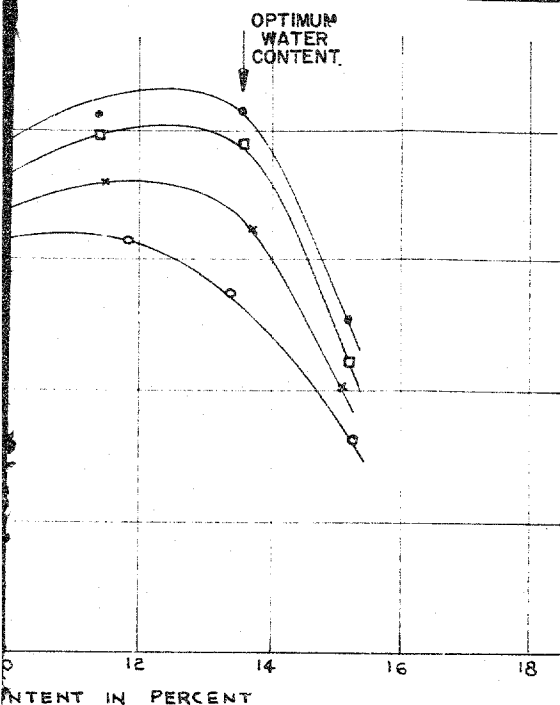
PLASTIC INDEX — 9.7 %

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COMPACTION - STRENGTH PROPERTIES
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE 3
PROJECT S7050



LEGEND

SYMBOL CHAMBER PRESSURE, (PSI)

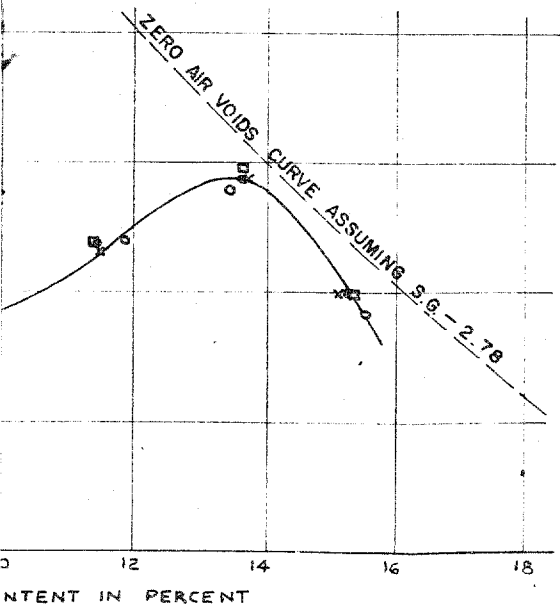
- UNCONFINED
- × 10 P.S.I.
- 20 P.S.I.
- 30 P.S.I.

COMPACTION TECHNIQUE

HARVARD MINIATURE DEVICE
10 LAYERS - 25 TAMPS/LAYER
40 LB. SPRING

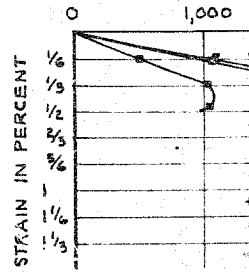
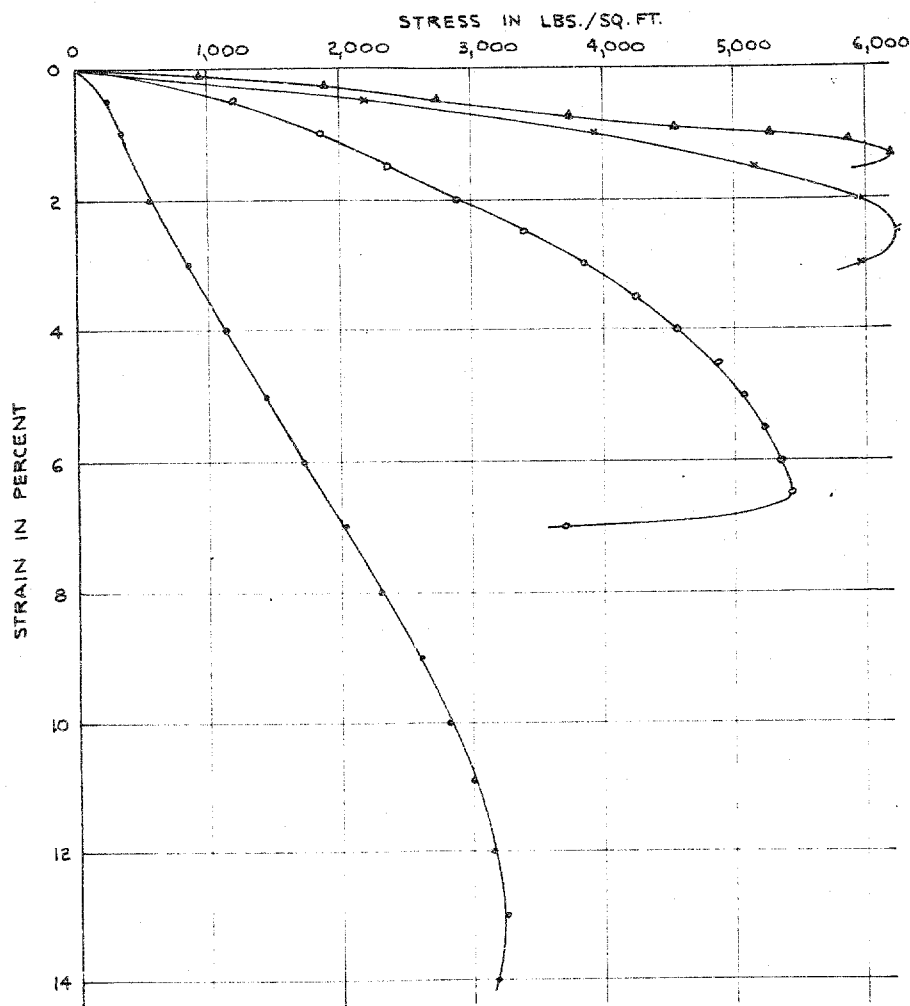
LIQUID LIMIT — 30.0 %
PLASTIC LIMIT — 20.3 %
PLASTIC INDEX — 9.7 %

RATE OF AXIAL STRAIN, 2% / MINUTE



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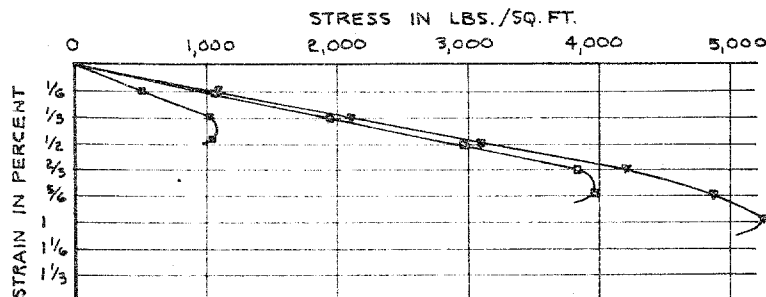
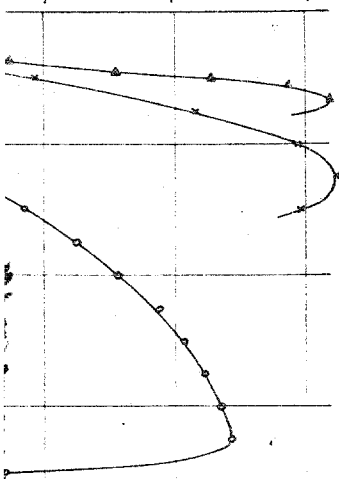
SYMBOL

■
◊
▼
▲
×
○
●

STRESS - STRAIN CURVES
COMPACTED WEATHERED QUEENSTON SHALE
UNCONFINED COMPRESSION TESTS

APPENDIX II
FIGURE 4
PROJECT S 7050

S./SQ. FT.
4,000 5,000 6,000

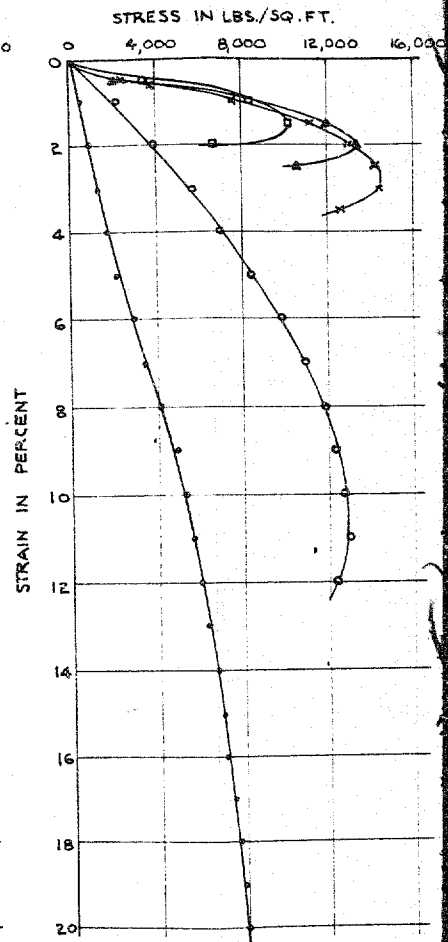
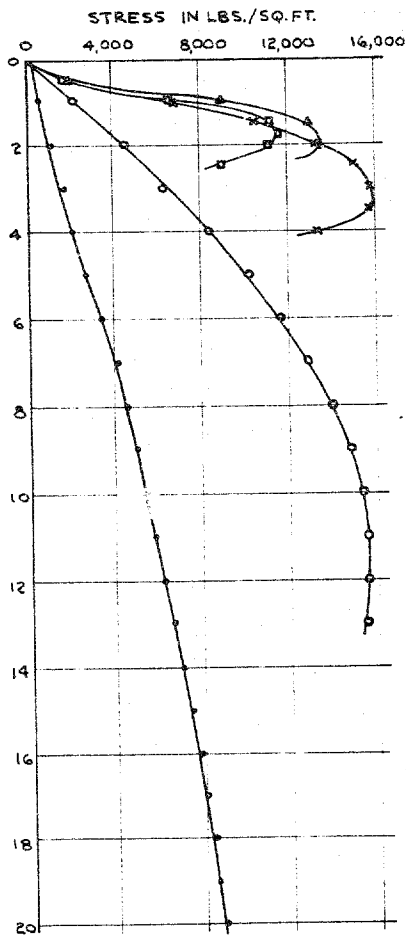
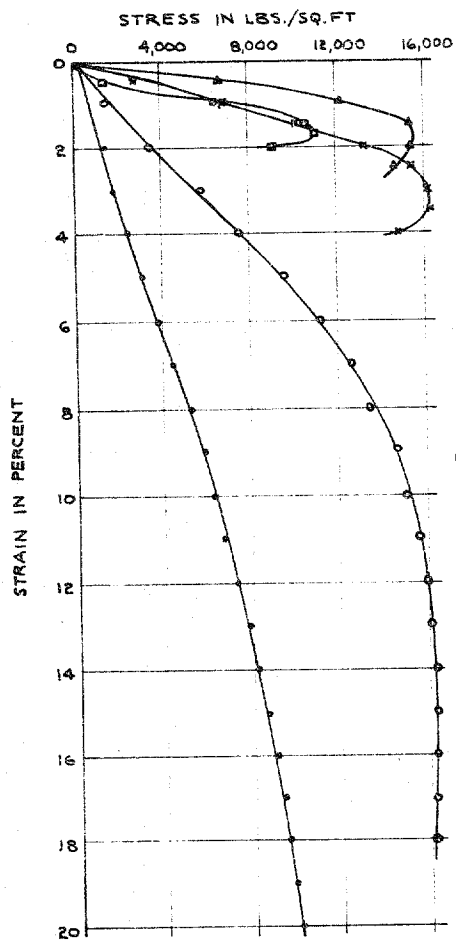


SYMBOL	MOULDING, Wc	DRY DENSITY, γ_d
■	4.03 %	115.8 P.C.F.
□	5.90 %	116.8 P.C.F.
▽	7.77 %	117.8 P.C.F.
▲	9.60 %	119.1 P.C.F.
x	11.50 %	122.0 P.C.F.
o	13.60 %	124.0 P.C.F.
9	15.20 %	119.3 P.C.F.

REMARKS

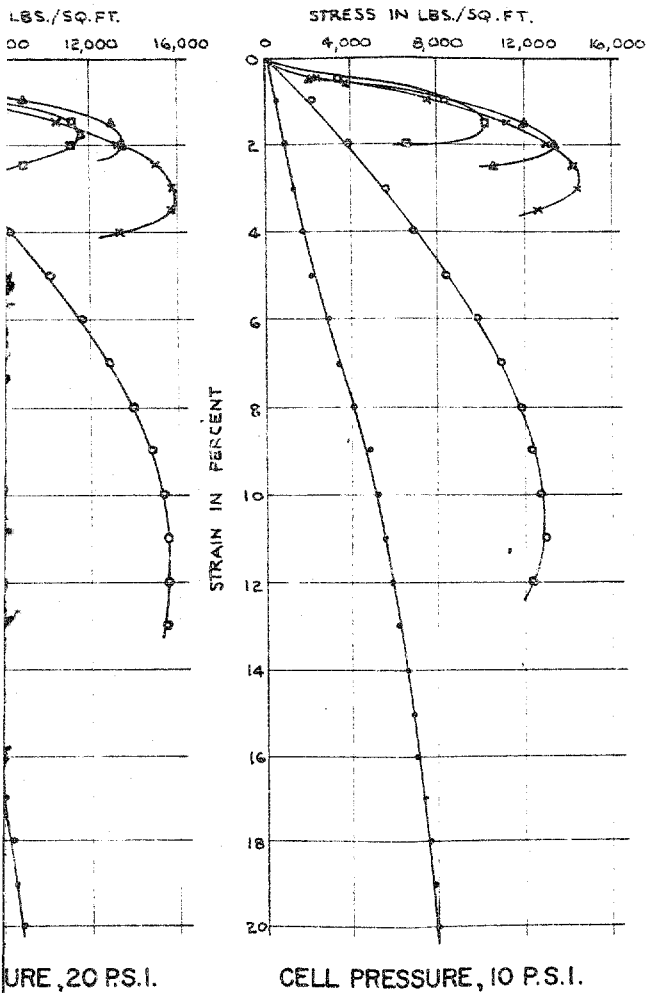
- 1) RATE OF AXIAL STRAIN, 2 % / MIN.
- 2) LUCITE PLATES TOP AND BOTTOM
- 3) COMPACTION TECHNIQUE, HARVARD
MINIATURE DEVICE - 10 LAYERS -
25 TAMP / LAYER - 40 LB. SPRING.

GEOCON



STRESS-STRAIN CURVES
COMPACTED WEATHERED QUEENSTON SHALE
UNCONSOLIDATED UNDRAINED TRIAXIAL

APPENDIX II
FIGURE 5
PROJECT S 7050



SYMBOL	AVERAGE W_c
□	7.5 %
△	9.6 %
x	11.5 %
o	13.6 %
e	15.2 %

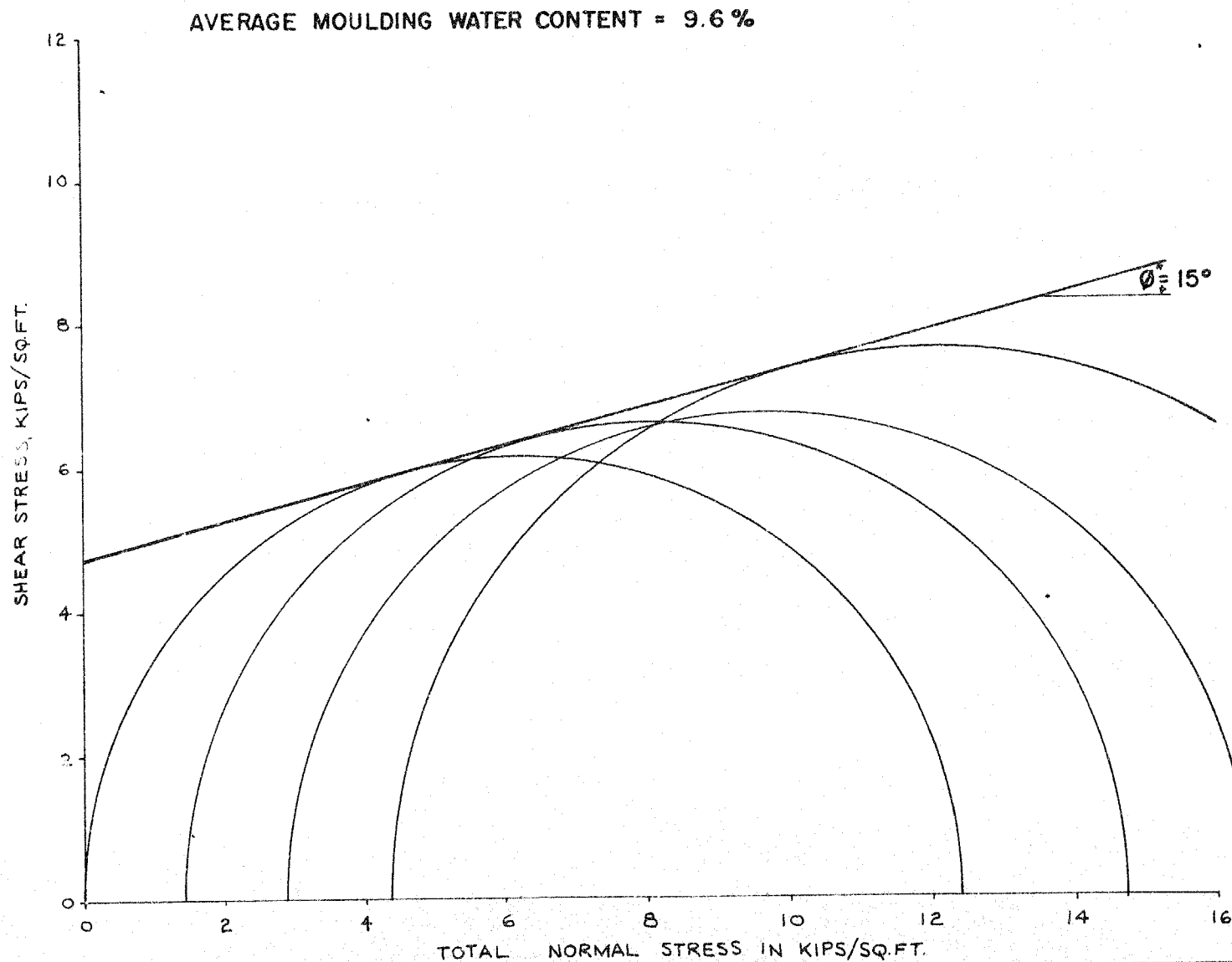
REMARKS

- 1) RATE OF AXIAL STRAIN, 2% / MIN.
- 2) LUCITE PLATES TOP AND BOTTOM
- 3) COMPACTION TECHNIQUE, HARVARD MINIATURE DEVICE - 10 LAYERS - 25 TAMPS / LAYER - 40 LB. SPRING.

GEOCON

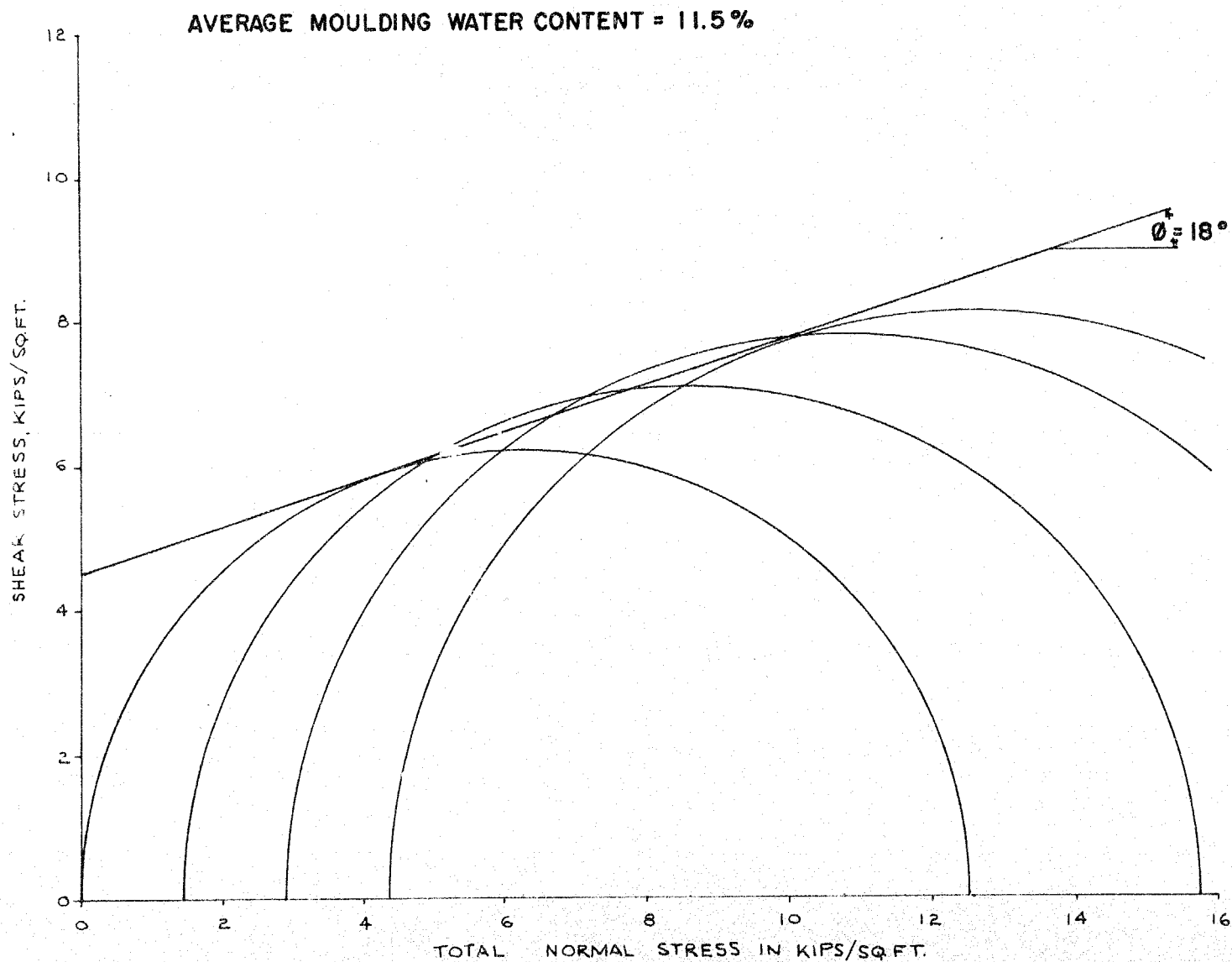
MOHR CIRCLE DIAGRAM
WEATHERED QUEENSTON SHALE
COMPACTED STRENGTH - TOTAL STRESS SERIES

APPENDIX II
FIGURE 6
PROJECT S7050



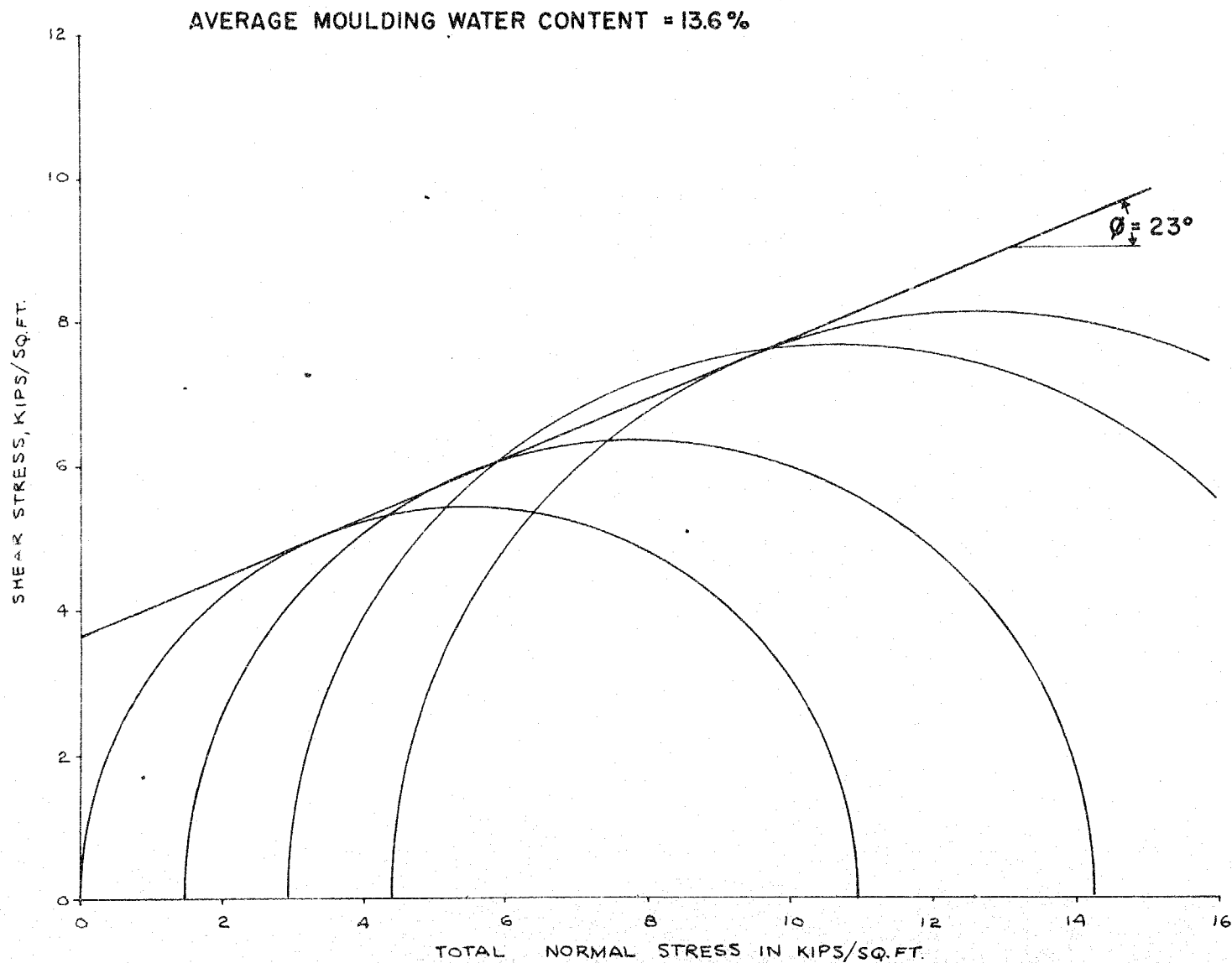
MOHR CIRCLE DIAGRAM
WEATHERED QUEENSTON SHALE
COMPACTED STRENGTH - TOTAL STRESS SERIES

APPENDIX II
FIGURE 7
PROJECT S7050



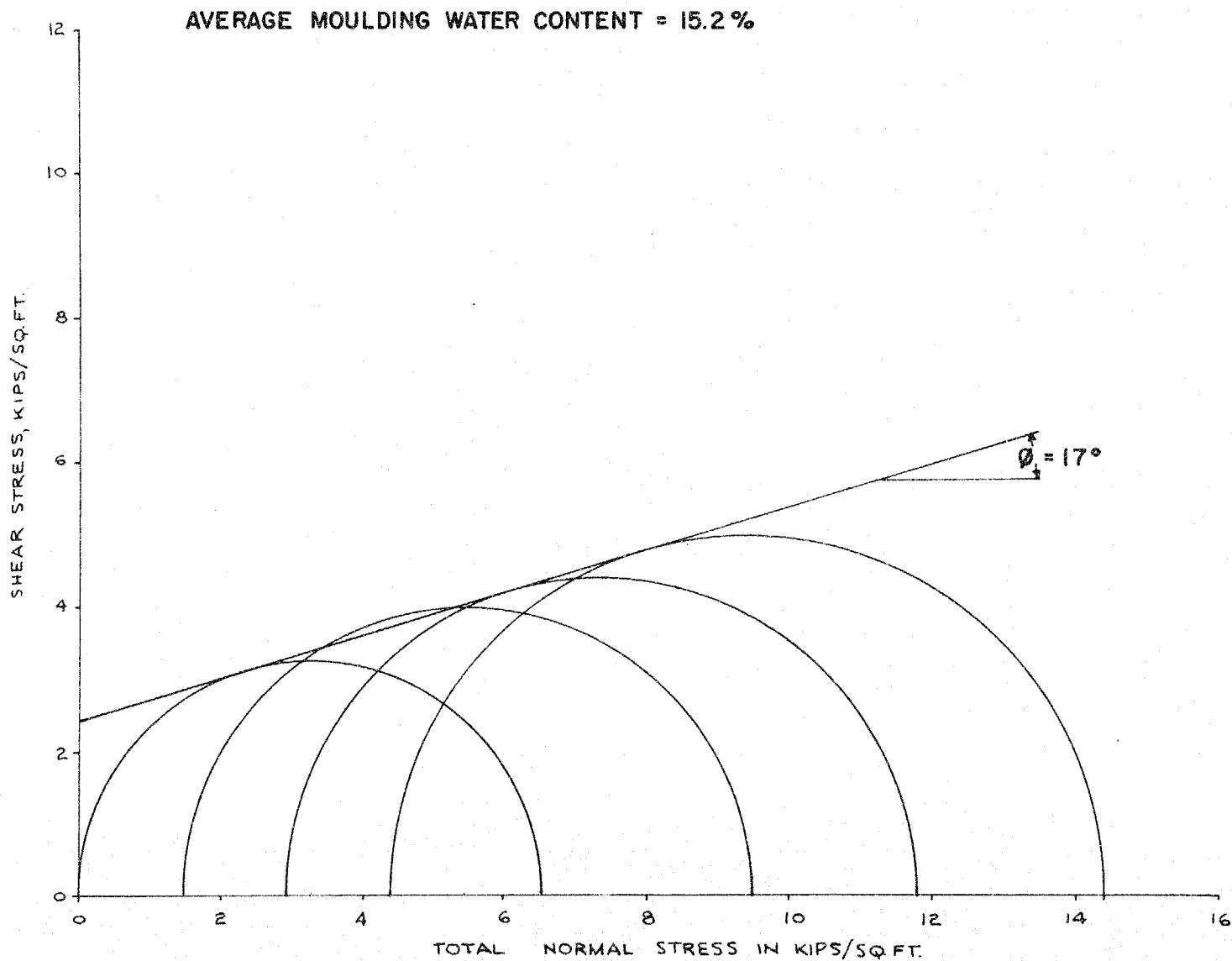
MOHR CIRCLE DIAGRAM
WEATHERED QUEENSTON SHALE
COMPACTED STRENGTH - TOTAL STRESS SERIES

APPENDIX II
FIGURE 8
PROJECT S7050



MOHR CIRCLE DIAGRAM
WEATHERED QUEENSTON SHALE
COMPACTED STRENGTH - TOTAL STRESS SERIES

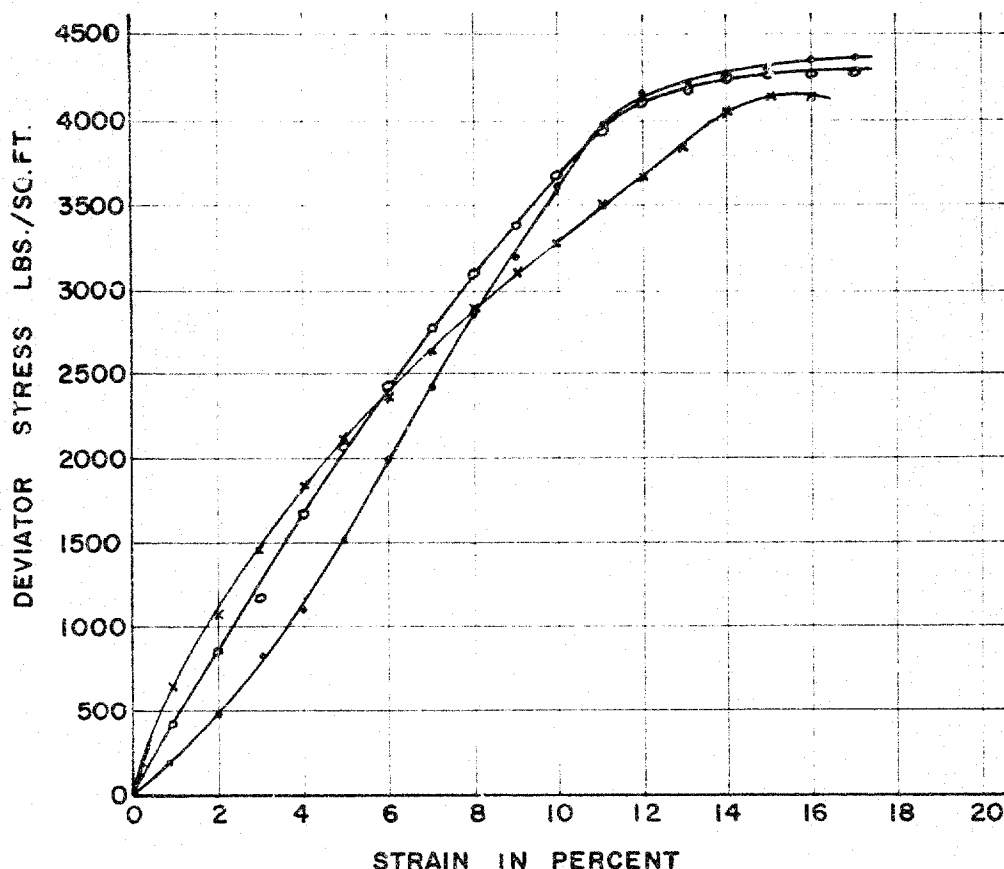
APPENDIX II
FIGURE 9
PROJECT S7050



UNDRAINED TRIAXIAL TESTS
SATURATED AND UNCONSOLIDATED
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE 10
PROJECT S7050

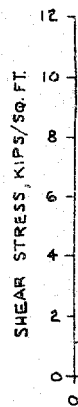
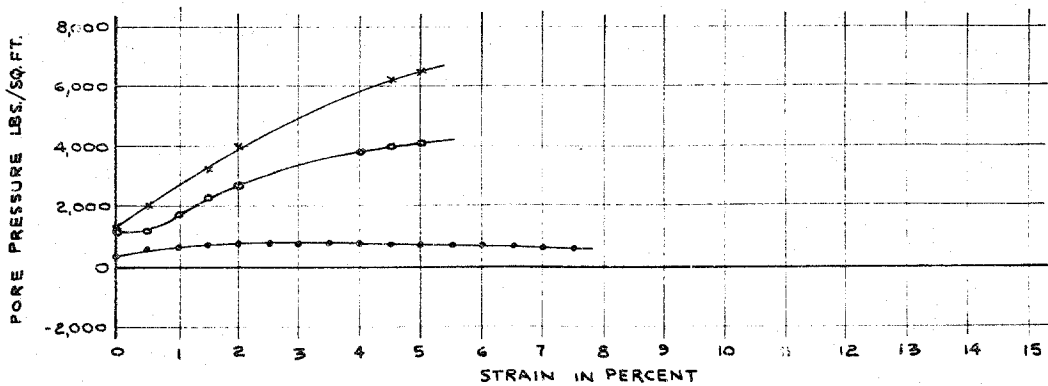
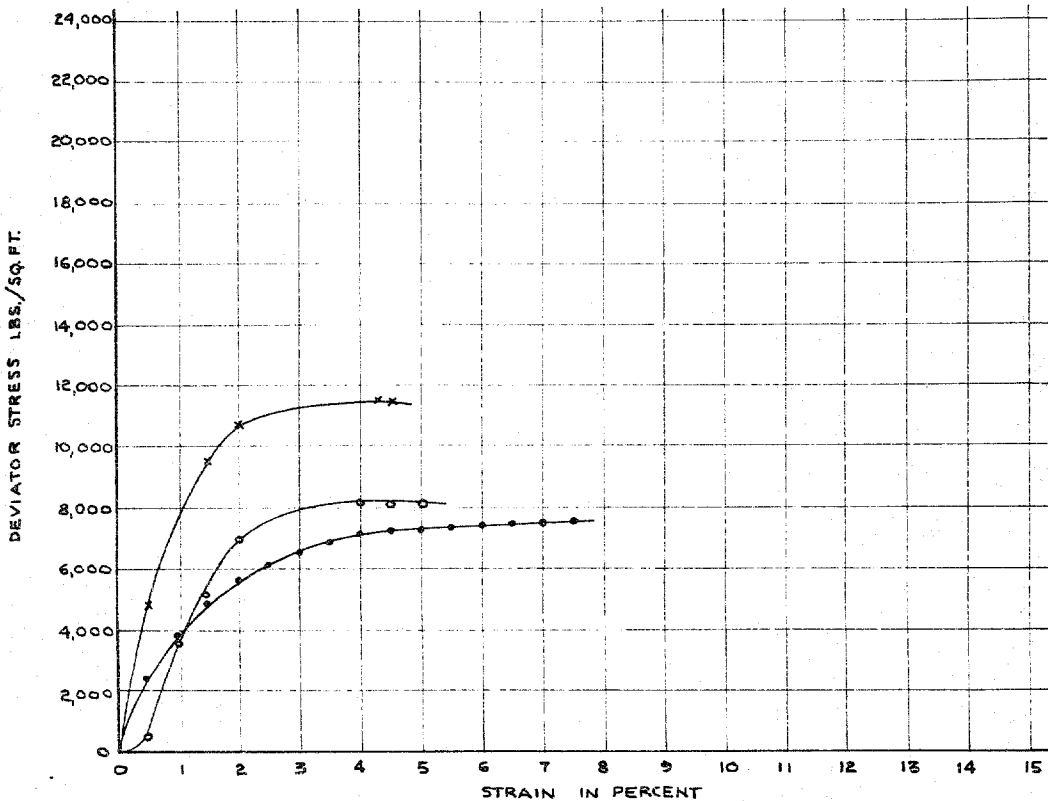
SYMBOL	TEST No.	γ_d (p.c.f.)	INITIAL CONDITION			FINAL CONDITION		
			W (%)	S (%)	e	W (%)	S (%)	e
x	①	127	12.8	97.1	.368	15.8	100	.475
o	②	126	13.0	95.6	.380	16.7	100	.510
•	③	121	15.1	97.0	.432	16.0	100	.450



- REMARKS 1) COMPACTION TECHNIQUE, HARVARD MINIATURE DEVICE, 10 LAYERS, 25 TAMPS/LAYER, 40 LB. SPRING.
- 2) SAMPLES WERE SATURATED IN TRIAXIAL CELL AFTER COMPACTION UNDER A CHAMBER PRESSURE OF 30 P.S.I., A HEAD OF 30 P.S.I. AND A BACK PRESSURE OF 20 P.S.I. THE PERIOD OF SATURATION WAS 24 HOURS.

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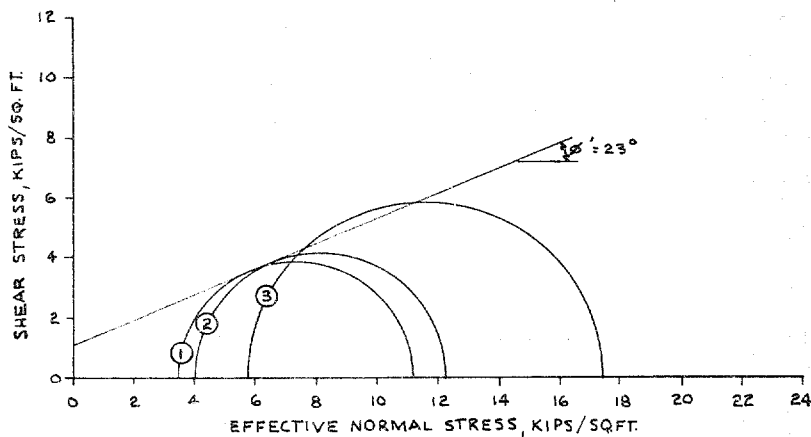
SERIES No. I
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE II
PROJECT S7050

SYMBOL	TEST No.	$\bar{\sigma}$ (psf.)	INITIAL CONDITION		FINAL CONDITION
			γ_d (p.c.f.)	w (%)	w (%)
•	①	4080	122.1	12.3	14.7
o	②	8160	121.9	12.1	15.1
x	③	12240	121.6	12.3	15.0

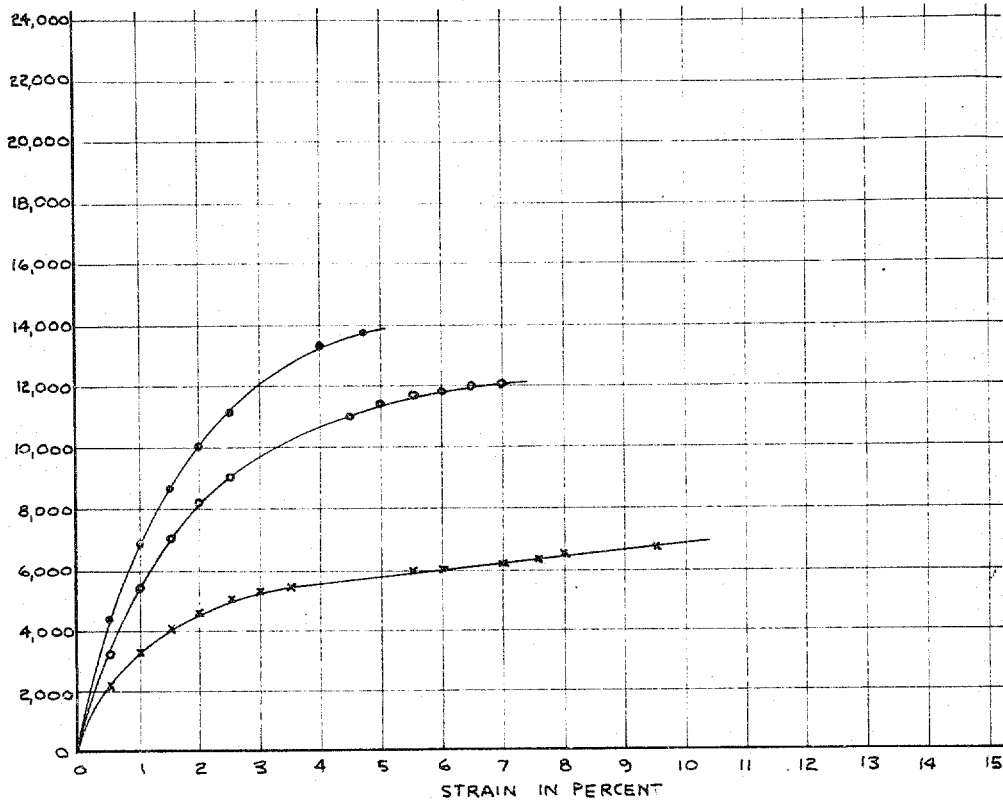
REMARKS:

- 1) FILTER STRIPS & POROUS STONES TOP & BOTTOM
- 2) RATE OF AXIAL STRAIN, 1.4 % / HOUR
- 3) COMPACTION BY HARVARD MINIATURE DEVICE
10 LAYERS - 25 TAMP/LAYER - 40° SPRING

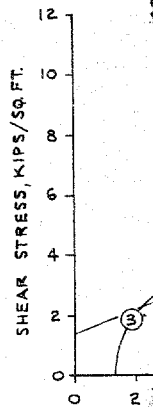
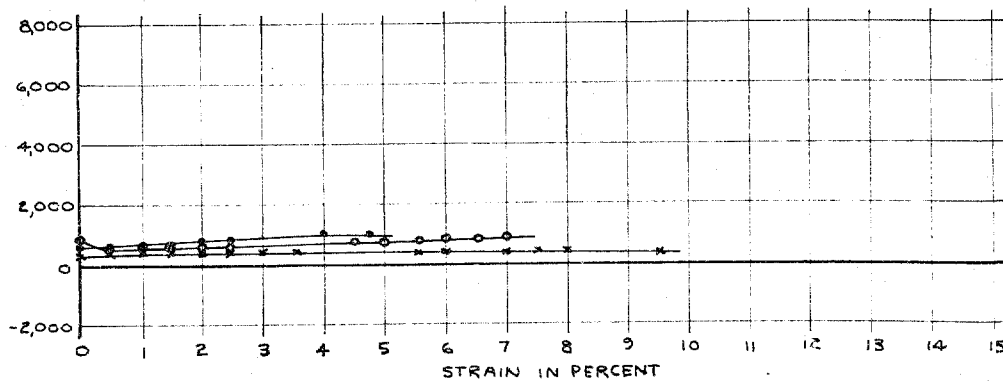


GEOCON

DEVIATOR STRESS LBS./SQ.FT.



PORE PRESSURE LBS./SQ.FT.



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SERIES No.2
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

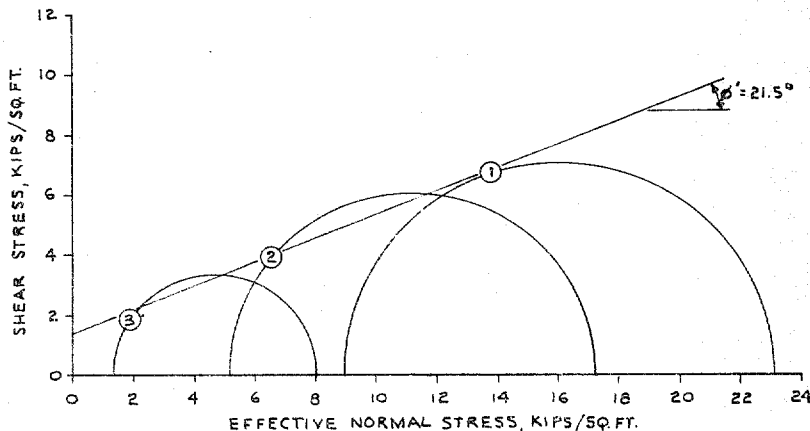
APPENDIX II
FIGURE 12
PROJECT S7050

SYMBOL	TEST No.	$\bar{\sigma}_3$ (psf.)	INITIAL CONDITION		FINAL CONDITION
			δ_d (p.c.f.)	W (%)	W (%)
•	①	10200	121.2	11.1	13.2
o	②	6120	120.3	10.6	12.9
x	③	2040	120.5	10.9	14.3

REMARKS: 1) FILTER STRIPS & POROUS STONES TOP & BOTTOM

2) RATE OF AXIAL STRAIN, 1.4 % / HOUR

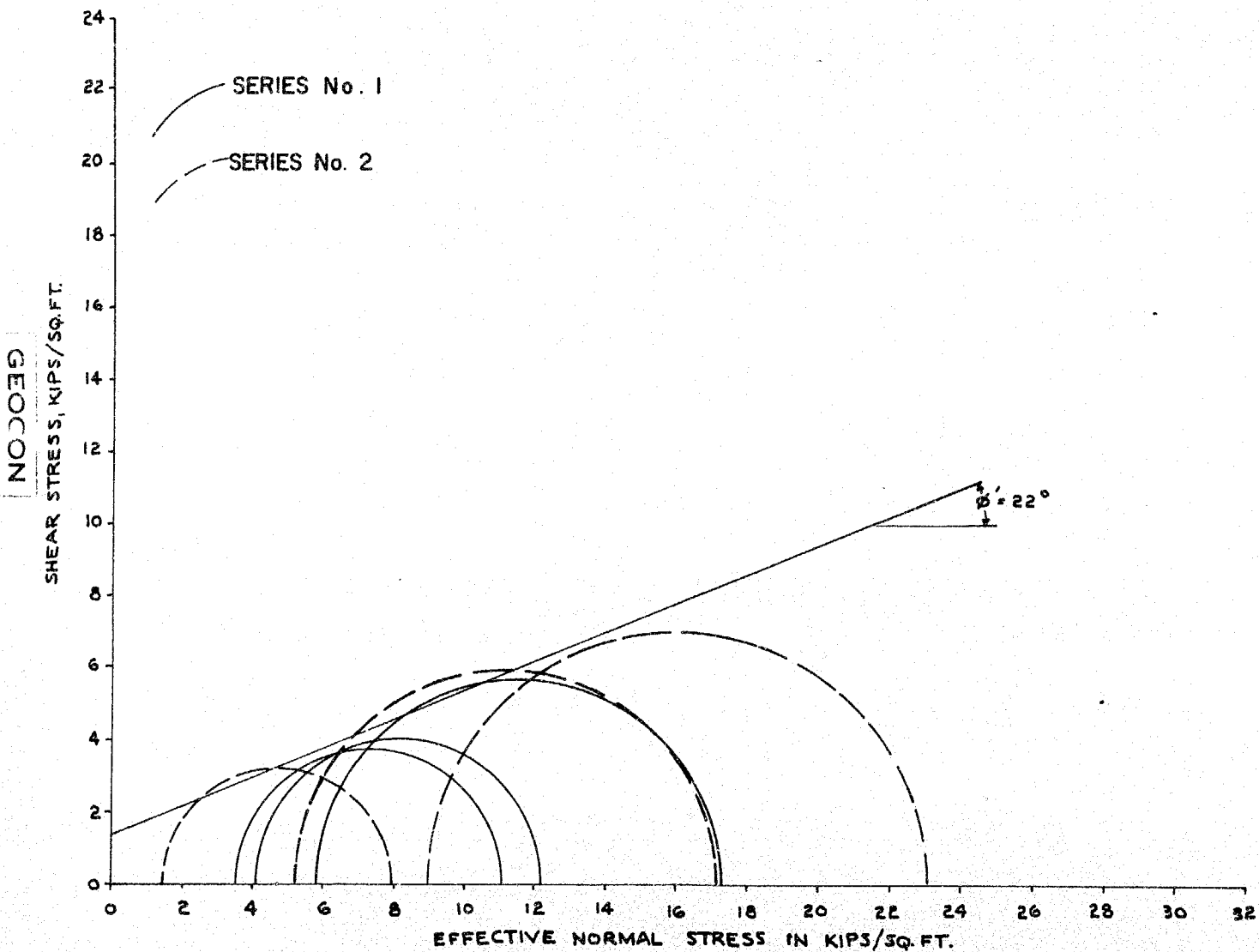
3) COMPACTION BY HARVARD MINIATURE DEVICE
10 LAYERS-25 TAMPS/LAYER-40th SPRING



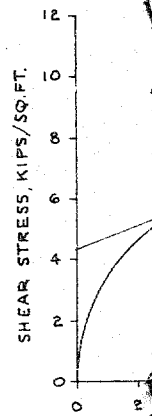
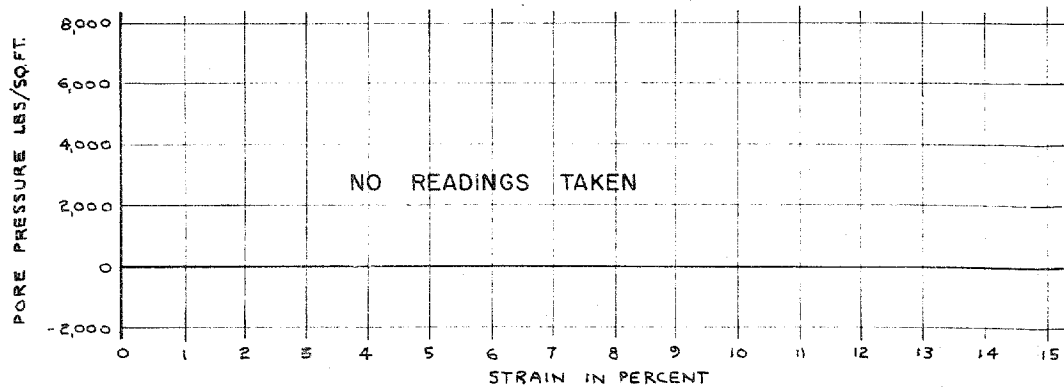
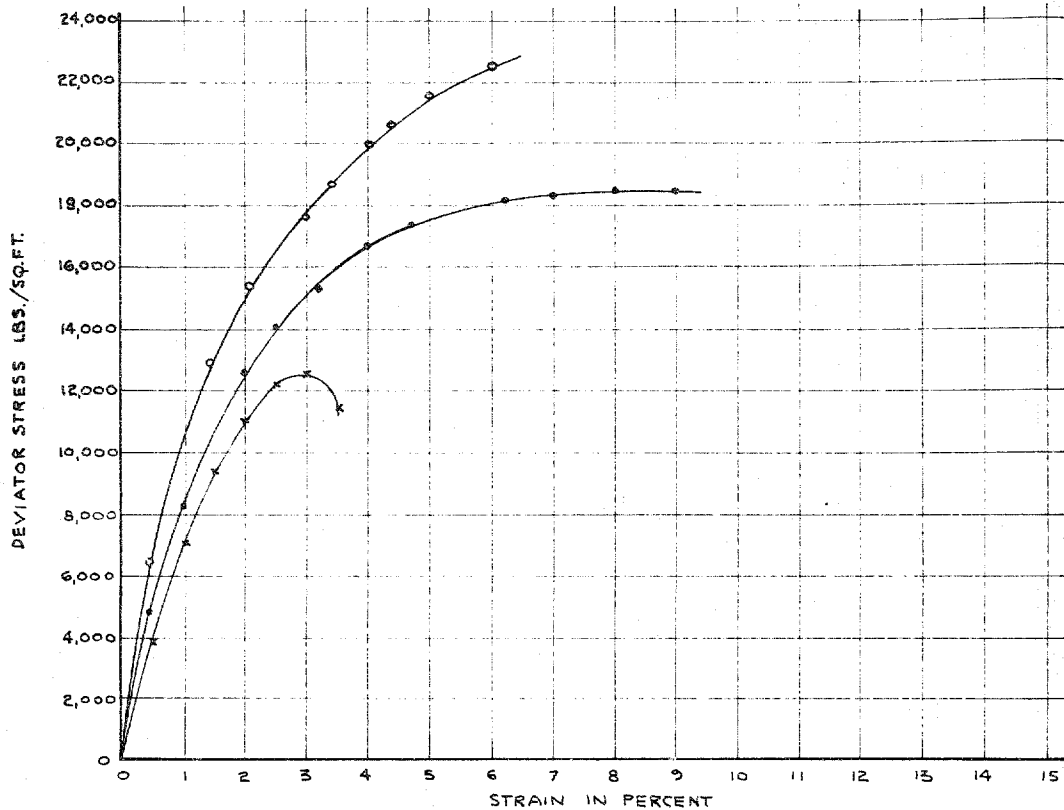
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MOHR CIRCLE DIAGRAM
WEATHERED QUEENSTON SHALE
SERIES 1 & 2

APPENDIX II
FIGURE 13
PROJECT S7050



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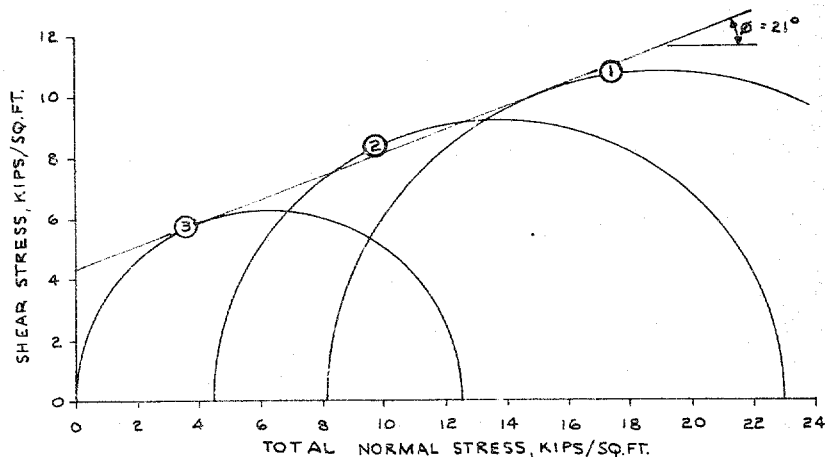
SERIES No.3
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE 14
PROJECT S7050

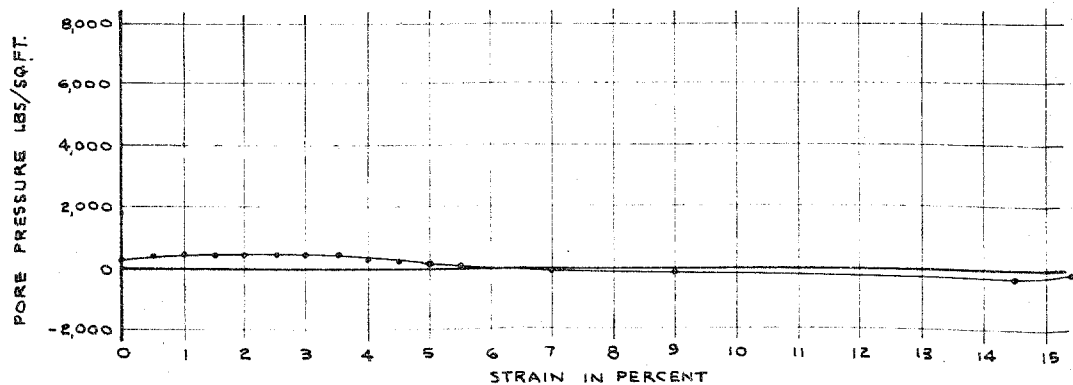
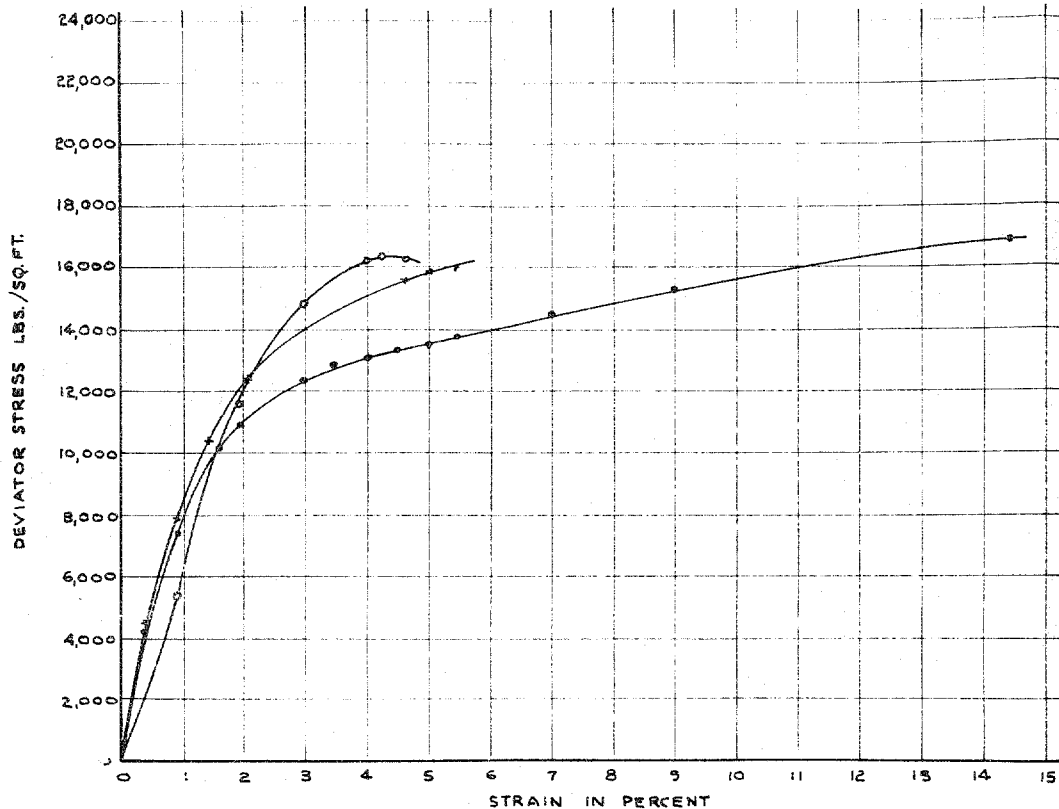
SYMBOL	TEST No.	$\bar{\sigma}_3$ (psf)	INITIAL CONDITION		FINAL CONDITION
			$\bar{\sigma}_d$ (pct)	W (%)	W (%)
o	①	8160	124.0	12.1	10.2
•	②	4480	123.4	11.5	11.8
x	③	0	124.5	11.3	11.3

REMARKS

- 1) NO FILTER STRIPS, LUCITE PLATES TOP & BOTTOM
- 2) RATE OF AXIAL STRAIN, 1.4 % / HOUR
- 3) COMPACTION BY HARVARD MINIATURE DEVICE
10 LAYERS-25 TAMPS/LAYER- 40* SPRING

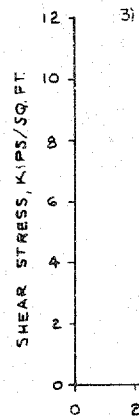


GEOCON



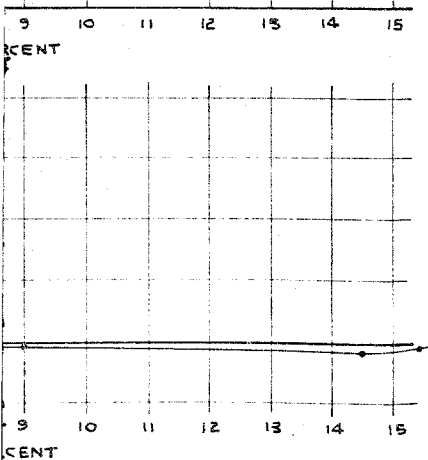
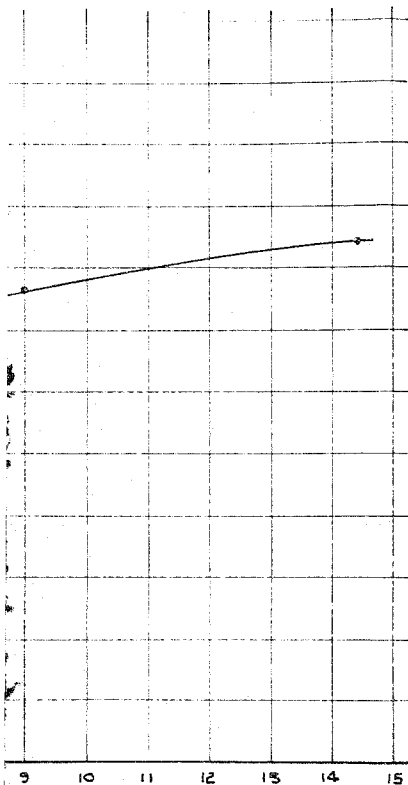
SYMBOL	TEST No
o	①
x	②
.	③

REMARKS



SERIES No.4
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

APPENDIX II
FIGURE 15
PROJECT S7050



SYMBOL	TEST No.	$\bar{\sigma}_3$ (ps.f.)	INITIAL CONDITION				FINAL CONDITION		
			$\bar{\epsilon}_d$ (p.c.f.)	W (%)	S (%)	e	W (%)	S (%)	e
o	①	4320	123.2	12.3	79.6	.431	12.3	90.5	.379
x	②	4480	123.2	12.3	87.2	.396	12.3	91.0	.386
•	③	4480	123.4	12.1	86.4	.394	12.5	94.8	.371

REMARKS

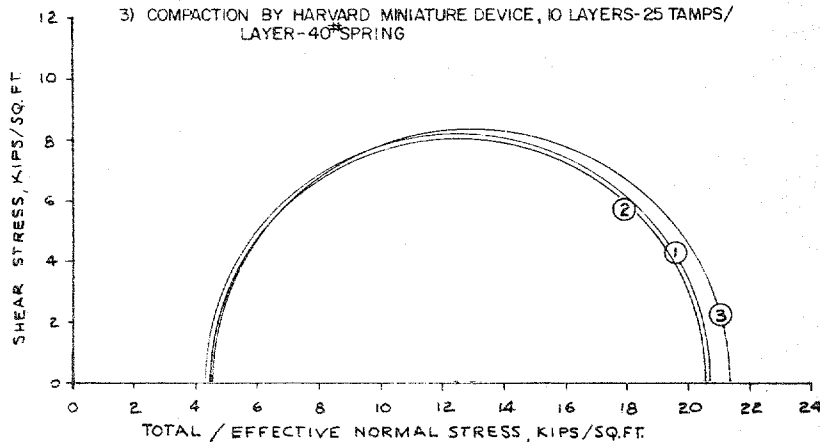
1) CONDITIONS OF TEST,

- ① NO FILTER STRIPS, LUCITE DISKS TOP & BOTTOM
- ② NO FILTER STRIPS, LUCITE DISKS TOP & BOTTOM
- ③ NO FILTER STRIPS, LUCITE DISKS AT TOP, POROUS STONES AT BOTTOM

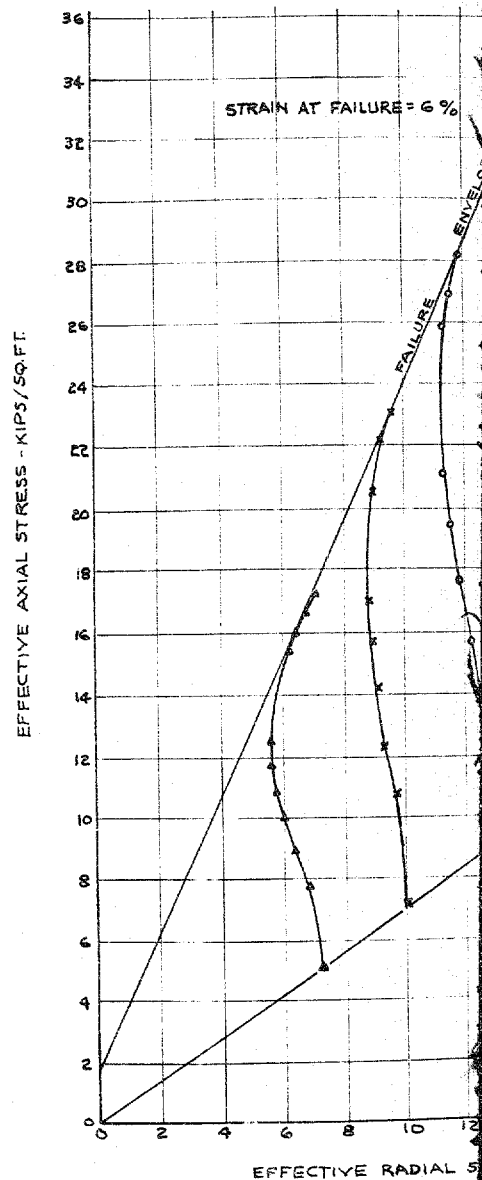
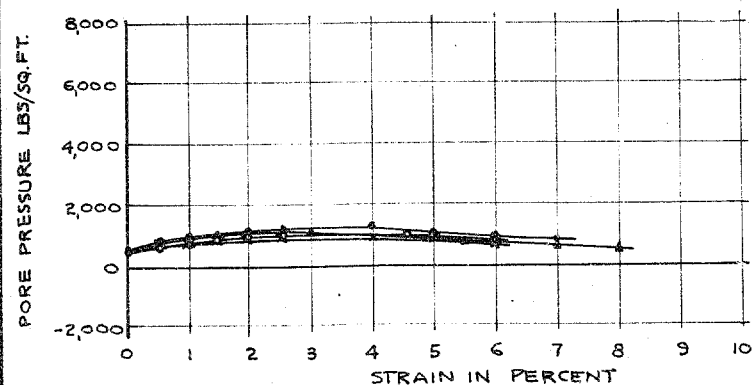
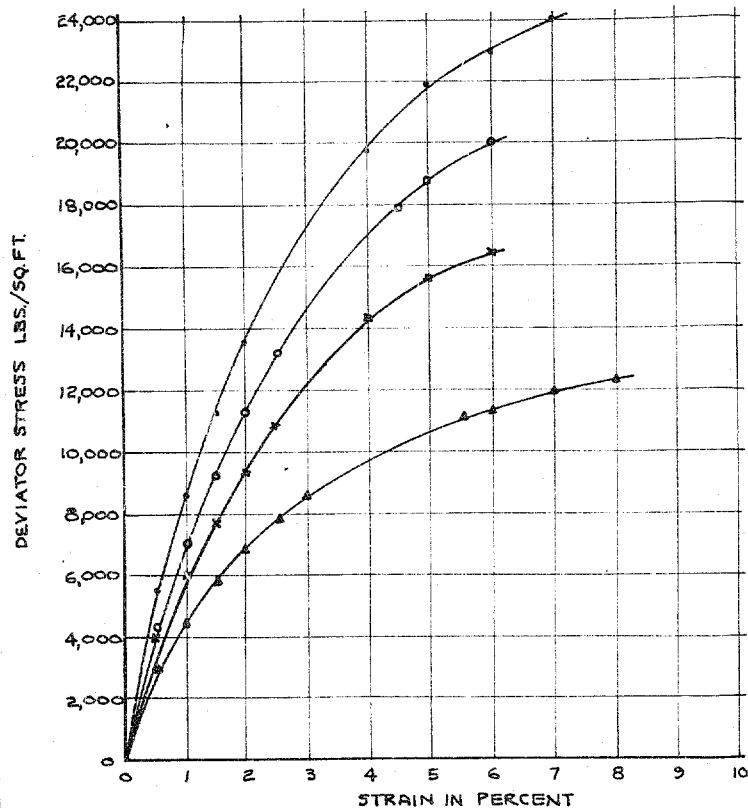
2) RATE OF AXIAL STRAIN,

- ① 20 % / MINUTE
- ② 1.4 % / HOUR
- ③ 1.4 % / HOUR

3) COMPACTION BY HARVARD MINIATURE DEVICE, 10 LAYERS-25 TAMP/ LAYER-40* SPRING

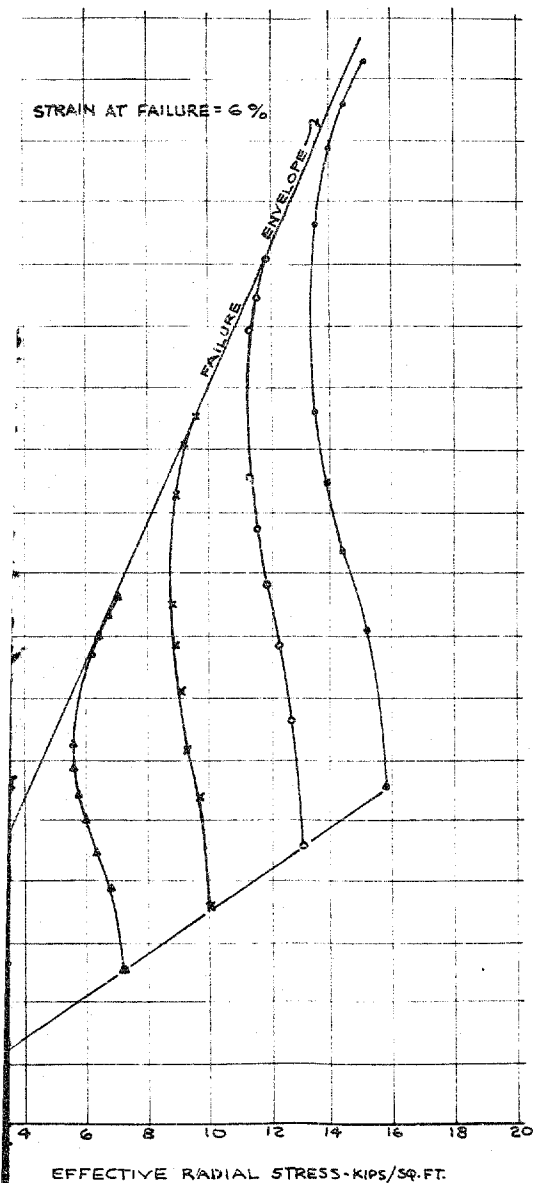


GEOCON



SERIES A
 CONSOLIDATED UNDRAINED
 WEATHERED QUEEN.

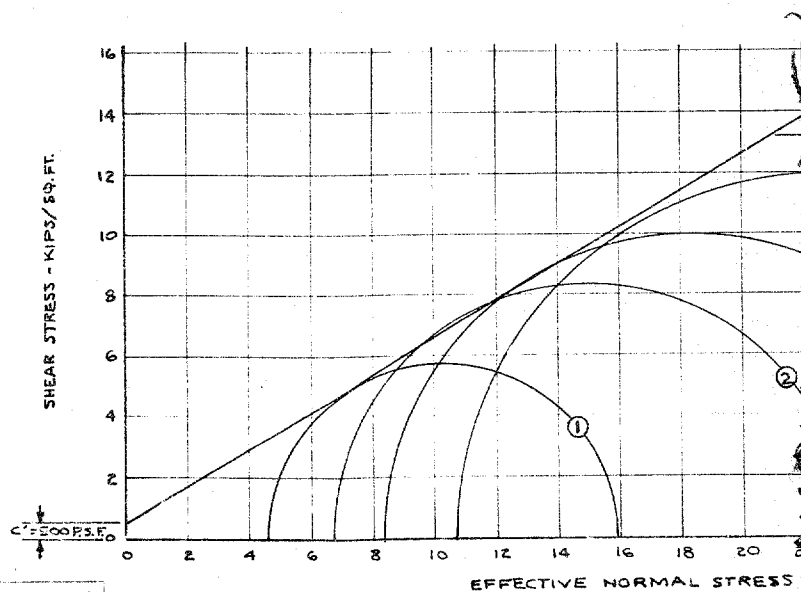
STRAIN AT FAILURE = 6 %



SYMBOL	TEST No.	σ_3 (p.s.f.)	σ_d (pcf)	INITIAL CONDITION		
				W (%)	S (%)	e
▲	①	2.55	123.3	13.5	92.4	.407
x	②	3.57	123.6	13.6	93.9	.404
o	③	4.59	123.5	13.4	92.0	.405
•	④	5.61	123.4	13.5	92.5	.406

REMARKS

- 1) FILTER STRIPS, POROUS STONES TOP AND BOT
- 2) RATE OF AXIAL STRAIN 1.4 % / HOUR.
- 3) COMPACTION TECHNIQUE, HARVARD MINIATURE D
25 TAMPS / LAYER - 40 LB. SPRING.
- 4) SAMPLES WERE SATURATED IN TRIAXIAL CELL AFT
A CHAMBER PRESSURE OF 30 P.S.I., A HEAD OF 30 P.
OF 24 P.S.I.. THE PERIOD OF SATURATION WAS 24
SATURATION THE SAMPLES WERE THEN CONSOLIDA



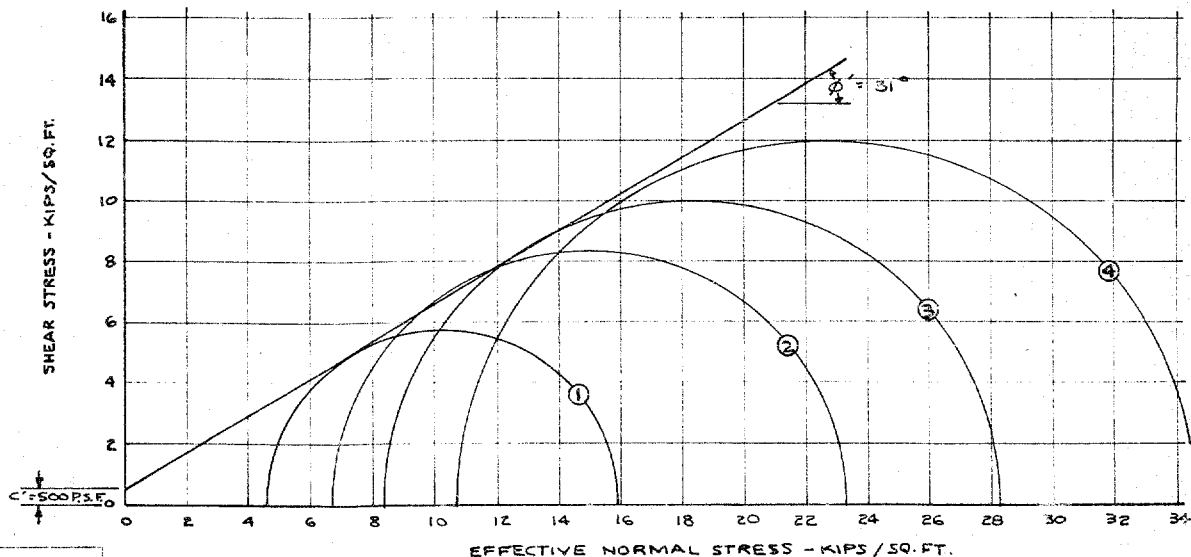
GEOCON

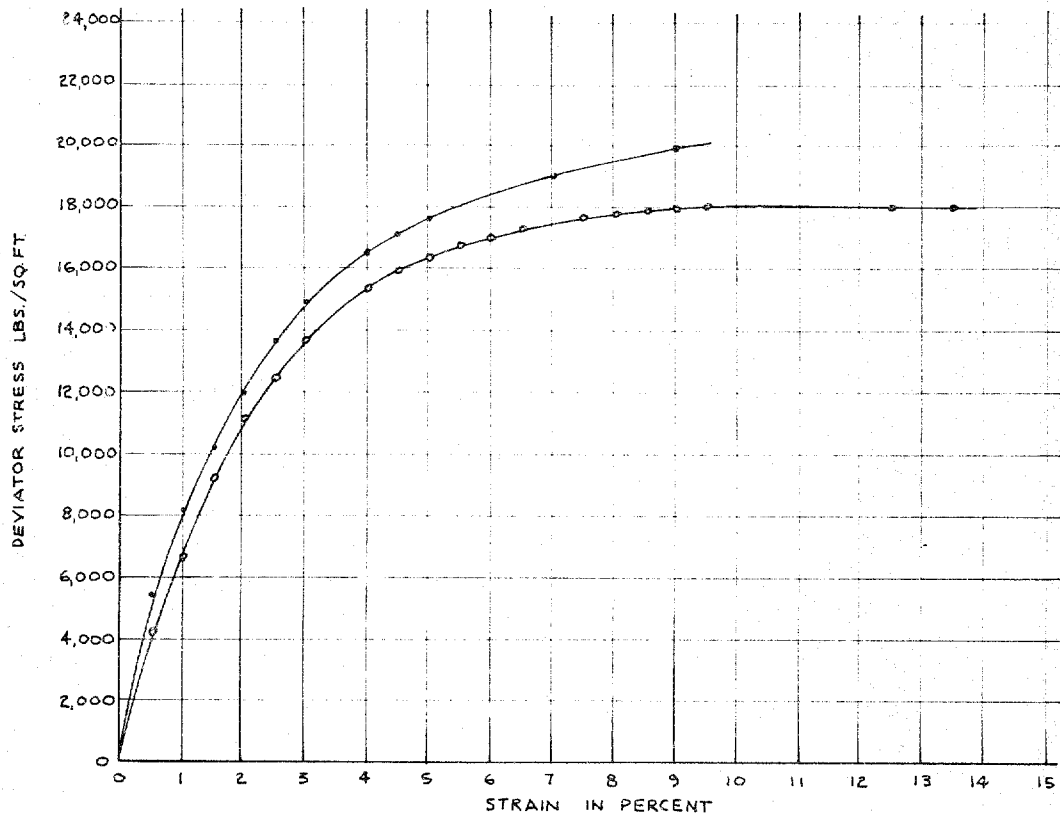
SERIES No.5
 CONSOLIDATED UNDRAINED TRIAXIAL TESTS
 WEATHERED QUEENSTON SHALE

APPENDIX II
 FIGURE 16
 PROJECT S 7050

SYMBOL	TEST No.	σ_3 (p.s.f.)	γ_d (pcf)	INITIAL CONDITION			FINAL CONDITION		
				W (%)	S (%)	e	W (%)	S (%)	e
▲	①	2.55	123.3	13.5	92.4	.407	14.2	100	.395
x	②	3.57	123.6	13.6	93.9	.404	12.9	100	.359
o	③	4.59	123.5	13.4	92.0	.405	13.0	100	.362
•	④	5.61	123.4	13.5	92.5	.406	13.5	100	.375

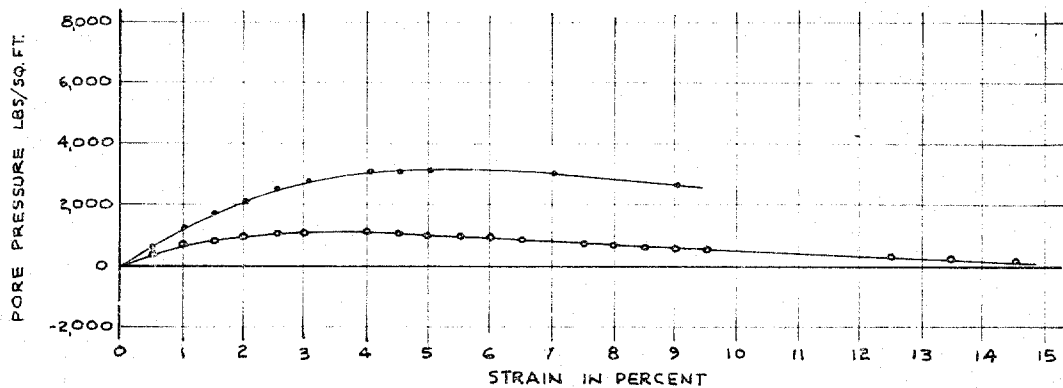
- REMARKS
- 1) FILTER STRIPS, POROUS STONES TOP AND BOTTOM.
 - 2) RATE OF AXIAL STRAIN 1.4 % / HOUR.
 - 3) COMPACTION TECHNIQUE, HARVARD MINIATURE DEVICE, 10 LAYERS - 25 TAMP / LAYER - 40 LB. SPRING.
 - 4) SAMPLES WERE SATURATED IN TRIAXIAL CELL AFTER COMPACTION UNDER A CHAMBER PRESSURE OF 30 P.S.I., A HEAD OF 30 P.S.I. AND A BACK PRESSURE OF 24 P.S.I.. THE PERIOD OF SATURATION WAS 24 HOURS. FOLLOWING SATURATION THE SAMPLES WERE THEN CONSOLIDATED BEFORE TESTING.



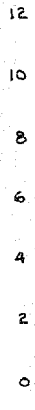


SYMBOL

REM



SHEAR STRESS, KIPS/SQ. FT.



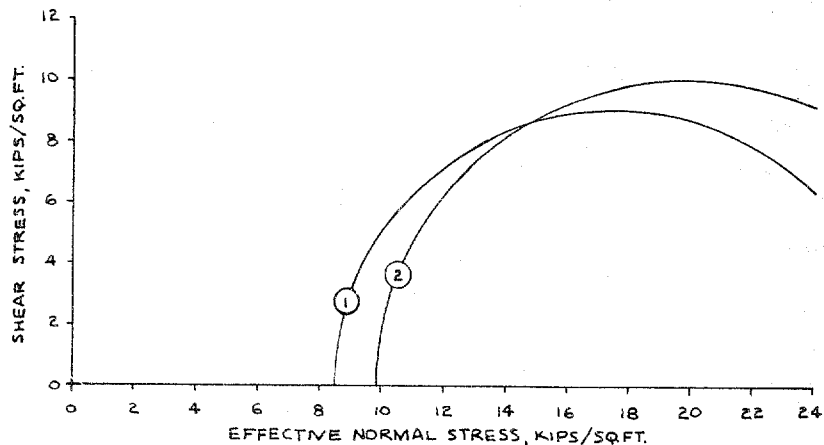
SERIES No. 5
CONSOLIDATED UNDRAINED TRIAXIAL TESTS
WEATHERED QUEENSTON SHALE

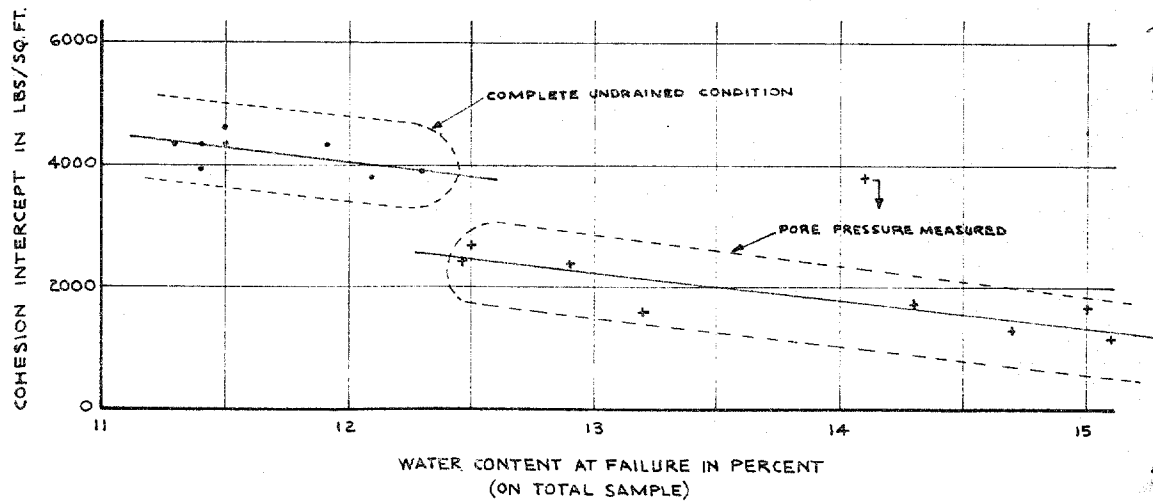
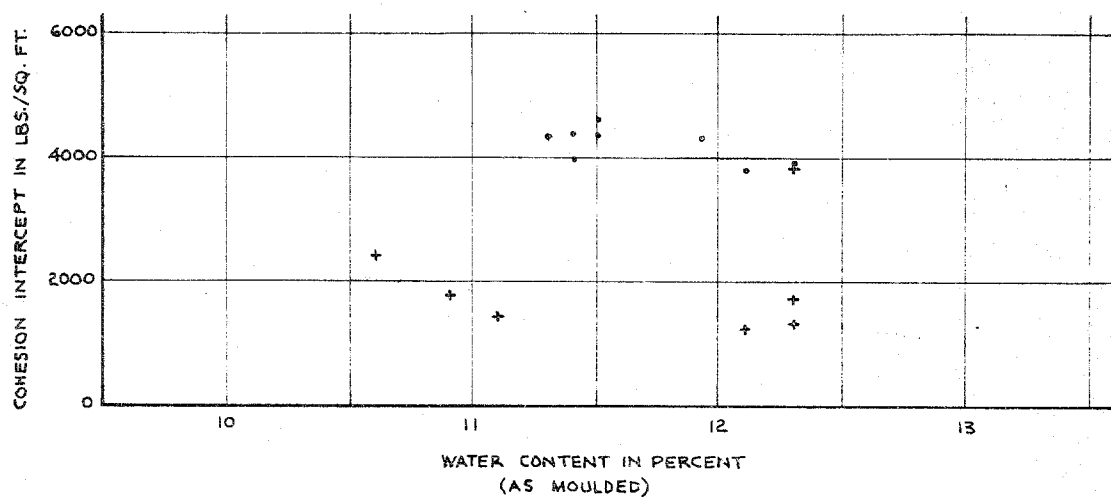
APPENDIX II
FIGURE 16A
PROJECT S7050

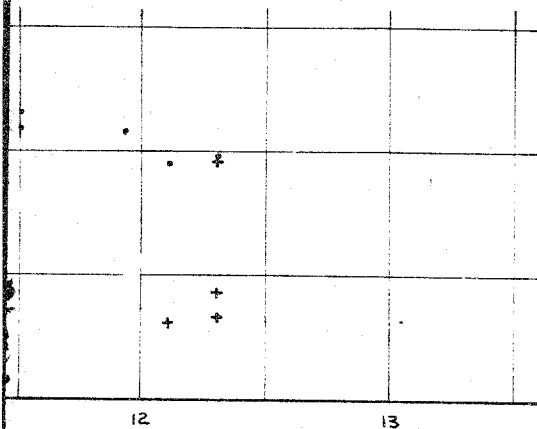
SYMBOL	TEST No.	$\bar{\sigma}_3$ (ps.f.)	INITIAL CONDITION			FINAL CONDITION		
			W (%)	S (%)	e	W (%)	S (%)	e
○	(1)	9180	12.3	86.3	.397	13.9	101.4	.381
•	(2)	11220	12.2	83.6	.406	13.7	98.2	.388

REMARKS

- 1) FILTER STRIPS, AND POROUS STONES TOP AND BOTTOM
- 2) RATE OF AXIAL STRAIN, 1.4 % / HOUR.
- 3) COMPACTION TECHNIQUE, HARVARD MINNATURE DEVICE, 10 LAYERS - 25 TAMPS/LAYER - 40 LB. SPRING
- 4) SAMPLES WERE SATURATED IN TRIAXIAL CELL AFTER COMPACTION UNDER A CHAMBER PRESSURE OF 60 P.S.I., A HEAD OF 60 P.S.I. AND A BACK PRESSURE OF 50 P.S.I. THE PERIOD OF SATURATION WAS 13 TO 19 HOURS. FOLLOWING SATURATION THE SAMPLES WERE THEN CONSOLIDATED BEFORE TESTING.

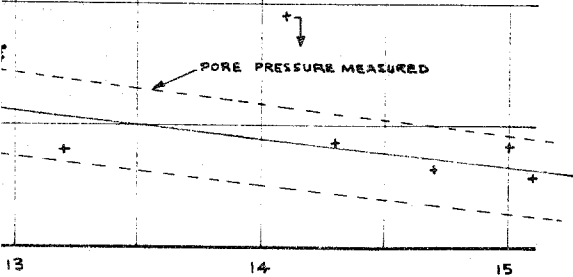






WATER CONTENT IN PERCENT (UNDETERMINED)

UNDRAINED CONDITION



WATER CONTENT IN PERCENT (UNDETERMINED)

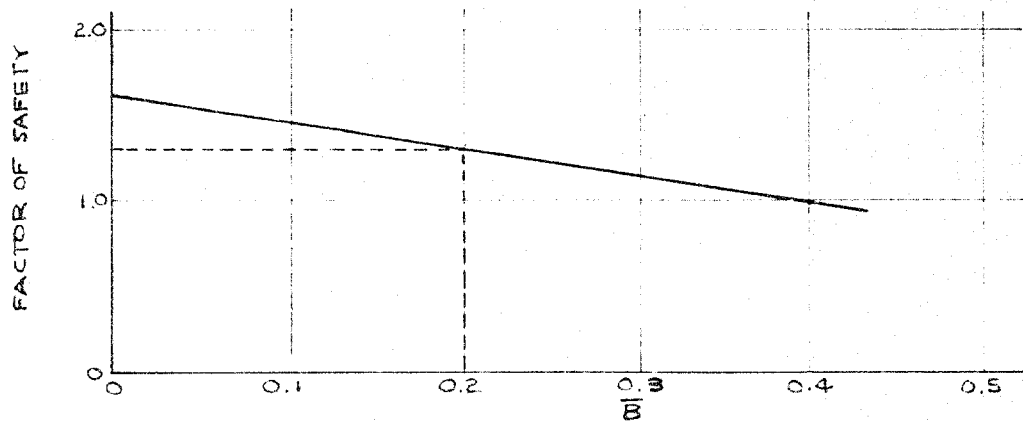
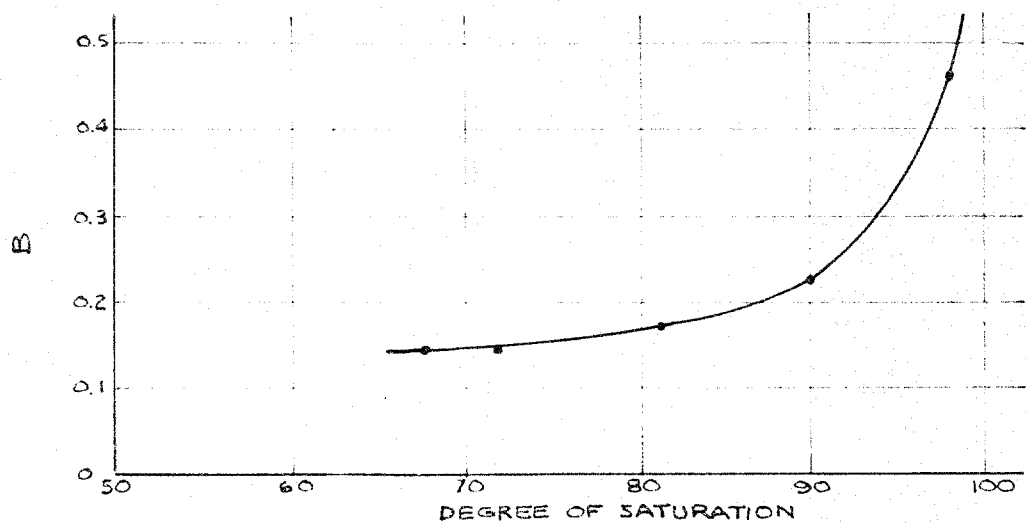
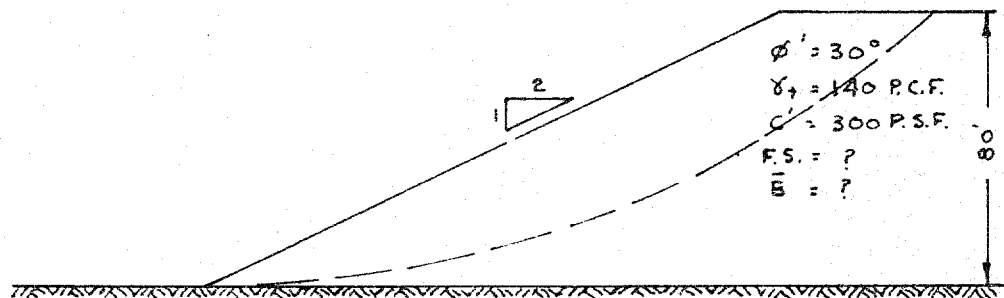
SAMPLE END CONDITIONS DURING TEST

+ POROUS STONE

• LUCITE PLATES

END OF CONSTRUCTION FACTOR OF SAFETY
FOR
80' EMBANKMENT

APPENDIX II
FIGURE 18
PROJECT S7050



GEOCON

Materials and Research Section.

May 23, 1961.

C.C. Parker & Parsons, Brinckerhoff, Ltd.,
Consulting Engineers,
795 Main Street West,
Hamilton, Ontario.

Attention: Mr. J. W. Richer.

Re: Laboratory Studies,
Weathered Queenston Shale,
Chedoke Expressway - District 4,
Hamilton, Ontario.

Dear Sir:-

Attached, we are forwarding to you the above mentioned report submitted by the consultant, Geocen, Ltd., of Toronto.

We have reviewed the report and have found that it contains all the information required from such an investigation. The results of the laboratory testing are well presented and the discussion conclusive.

Should you desire any additional information, or should there be any problems that you would like to discuss, please feel free to contact this Office.

Yours very truly,

L. G. Goderman,
PRINCIPAL FOUNDATION ENGR.
Per: *After May*

(A. G. Sternac,
SUPERVISING FOUNDATION ENGR.)

AGG/WLEW
Attach.

cc: C. C. Parker & Assoc. (4)

A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
I. C. Campbell
J. C. Thatcher
T. J. Kovich
A. Watt
Foundations Office
Gen. Files.

30M5-95
GEOCRES No.

30M5-95
GEOCRES No.

S7219

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

LABORATORY TESTING

PROPOSED WELLAND TUNNEL

WELLAND

ONTARIO

Distribution:

10 copies - Department of Highways, Ontario,
Downsview, Ontario.

2 copies - Geocon Ltd,
Rexdale, Ontario.

GEOCON

GEOCON LTD

30M5-95

GEOCRES No.

HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 6-7632

Rexdale, Ontario,
May 15th, 1961.

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8641

1425 WEST PENDER ST.
VANCOUVER 8, B.C.
TEL. MU. 1-8926

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Laboratory Testing,
Proposed Welland Tunnel,
Welland, Ontario.

Dear Sirs:

This letter accompanies our Drawings Nos. S7219-1 and -2 showing the results of undrained triaxial compression tests performed on two samples from the above project.

Drawing No. S7219-1 shows the results obtained on a sample from a depth of 50 feet in borehole No. 9B. When plotted on the Mohr circle diagram the results give an effective angle of shearing resistance of 30 degrees and an effective cohesion of 100 pounds per square foot.

Drawing No. S7219-2 shows the results obtained on a sample from a depth of 20 feet in borehole No. 9B. Test specimens obtained from this sample showed a variation in plasticity, moisture content and unit weight as noted on the drawing. From the stress path plots it is apparent also that one of the test specimens behaves differently from the other two. From the Mohr circles of stress an effective angle of shearing resistance of 26 degrees and no cohesion has been assigned to one specimen. The corresponding parameters for the other two specimens are an effective angle of shearing resistance of 23 degrees and again no cohesion.

We trust that this letter contains all the information you require from the testing. If we can be of any further assistance, please do not hesitate to call us.

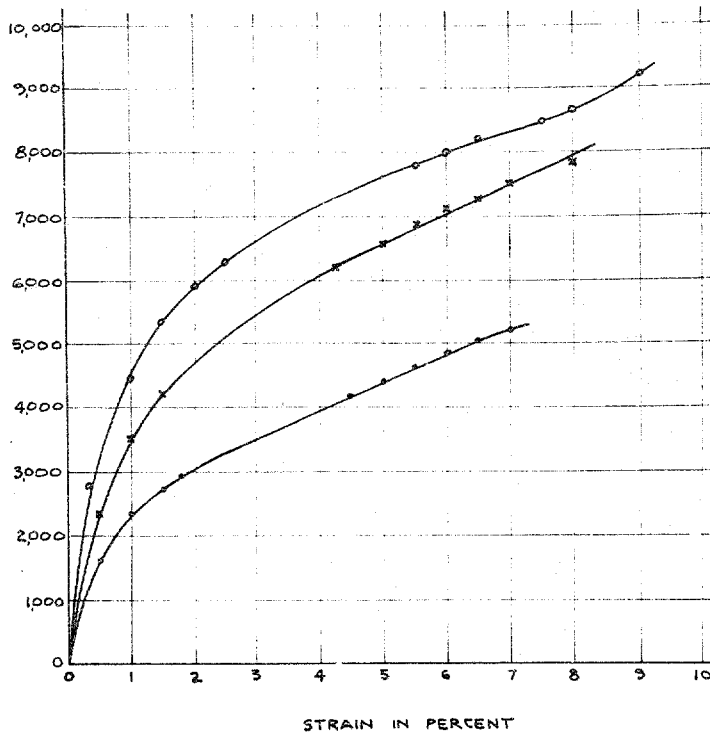
Yours very truly,

GEOCON LTD

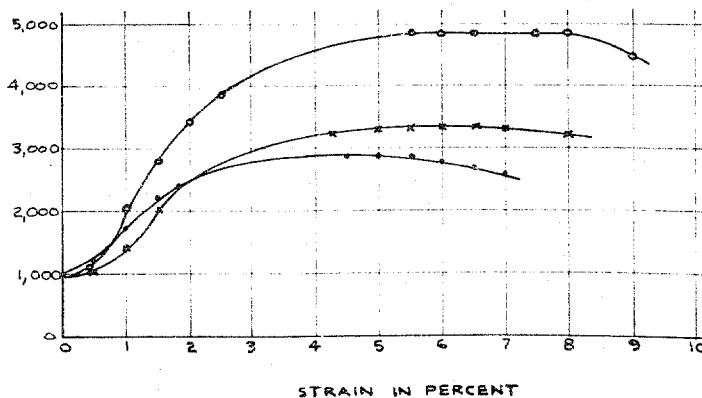
K. H. King
K. H. King, *per J. J. H.*
District Soils Engineer.

KHK/dw
S7219

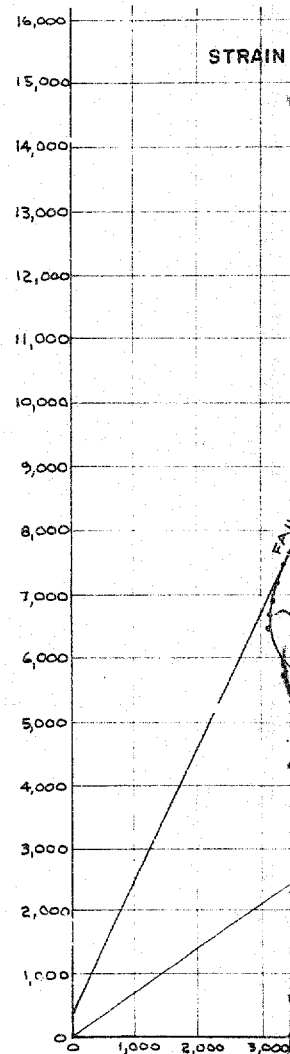
DEVIATOR STRESS LBS./SQ. FT.

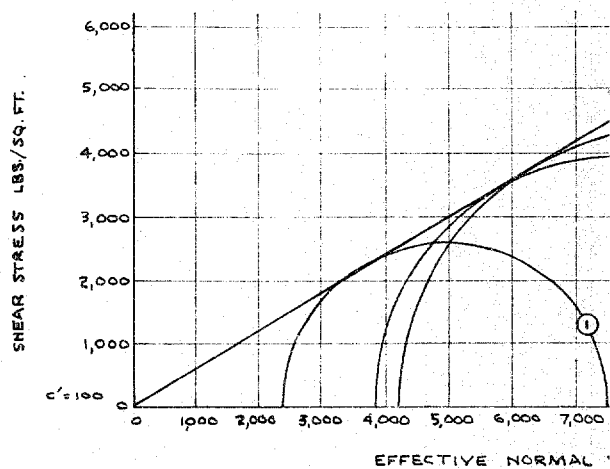
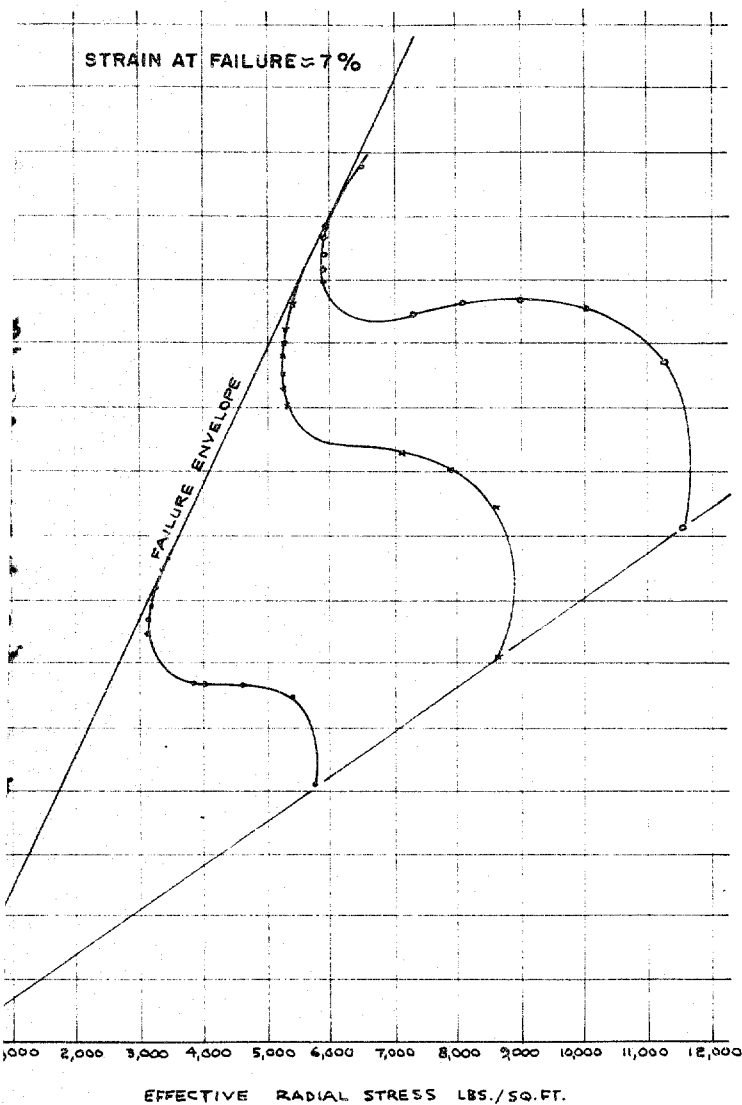


PORE PRESSURE LBS./SQ. FT.



EFFECTIVE AXIAL STRESS LBS./SQ. FT.





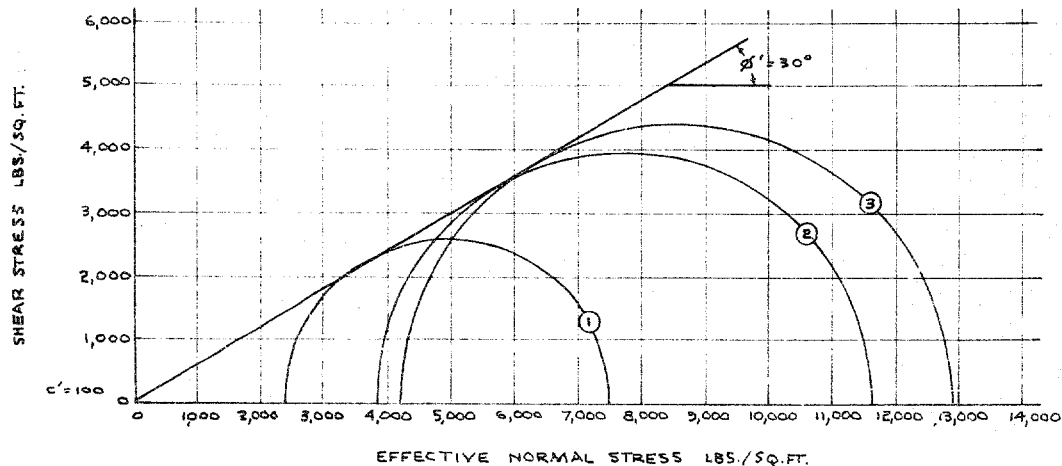
SYMBOL	TEST	γ	M.C.	L.L.
•	1	133	20.5	-
x	2	132	21.3	28.7
o	3	133	20.6	-

NOTES: 1) FILTER STRIPS, POROUS

2) RATE OF AXIAL STRAIN 0.

BOREHOLE No. 9B, S

DEPARTMENT OF HIGHWAYS, ONT
TORONTO
PROPOSED WELLAND TUNNEL
WELLAND
UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASURE



SYMBOL	TEST	γ	M.C.	L.L.	P.L.	P.I.
*	1	133	20.5	-	-	-
x	2	132	21.3	28.7	17.6	11.1
o	3	133	20.6	-	-	-

NOTES: 1) FILTER STRIPS, POROUS STONES TOP AND BOTTOM.

2) RATE OF AXIAL STRAIN 0.27 % / HOUR.

BOREHOLE No. 9B, SAMPLE 4, DEPTH 500' - 515'.

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED WELLAND TUNNEL
WELLAND ONTARIO
UNDRAINED TRIAXIAL TESTS
WITH PURE PRESSURE MEASUREMENTS

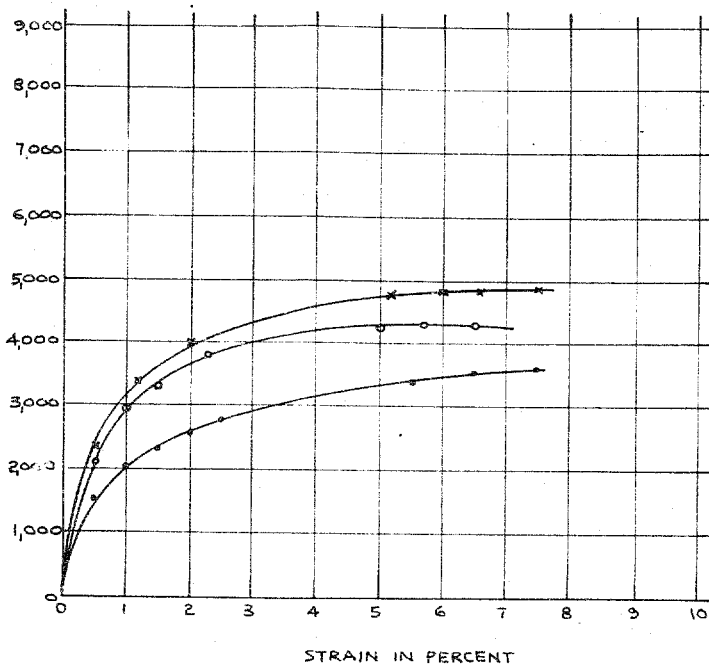
GEOCON LTD

DATE. MAY 5, 1961 SCALE. AS SHOWN

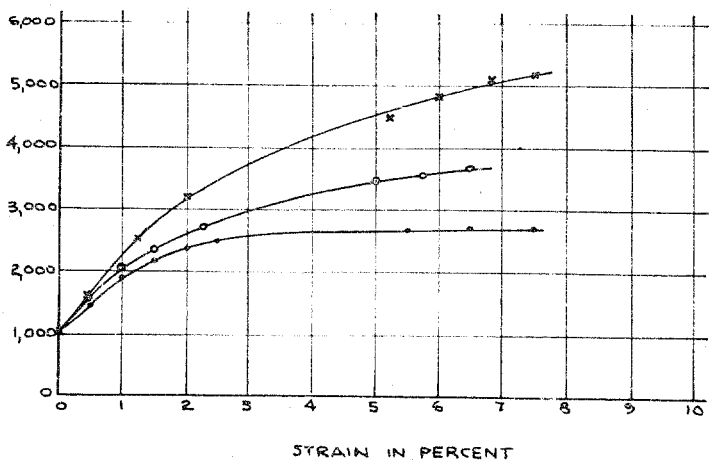
MADE CHKD. APPD.
M.W. F.H. K.H.

No. S 7219-1

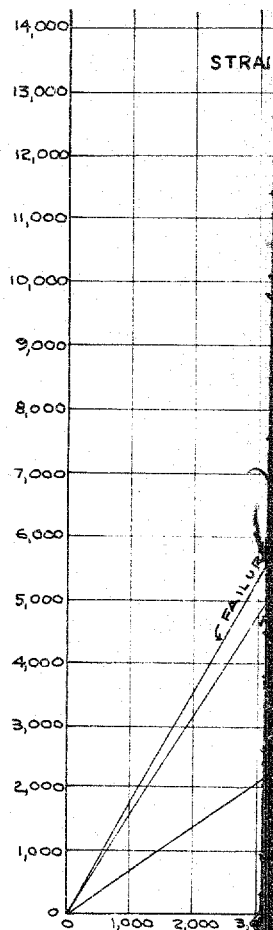
DEVIATOR STRESS LBS./SQ. FT.



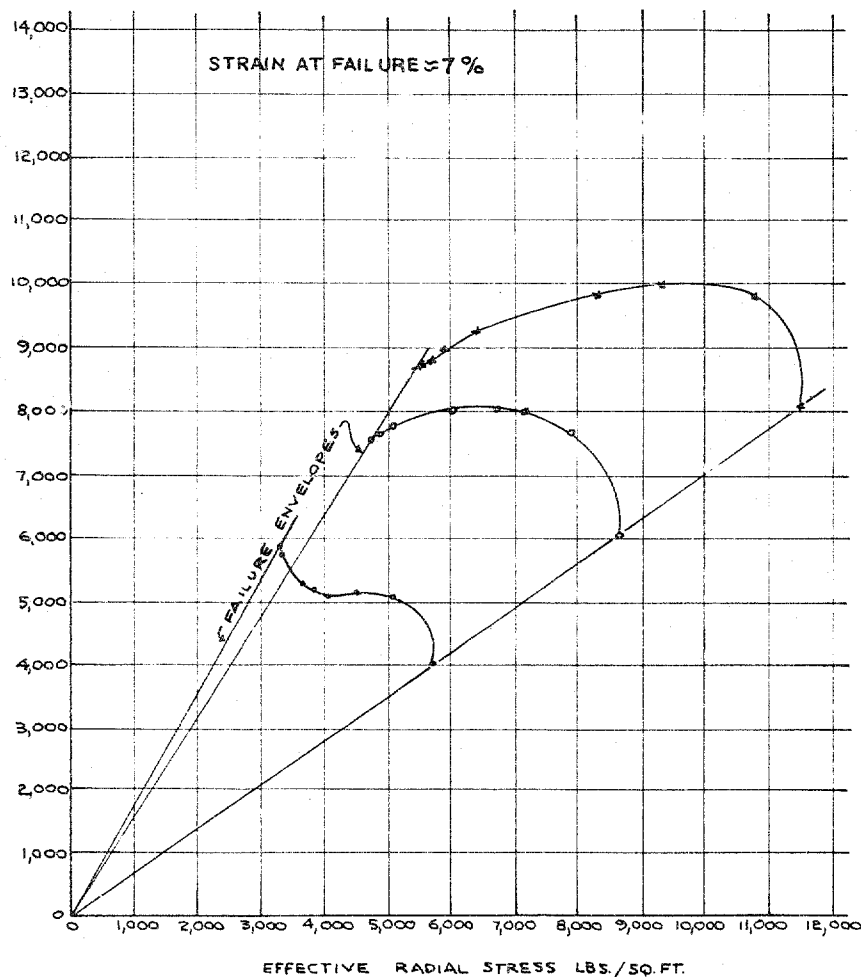
PORE PRESSURE LBS./SQ. FT.



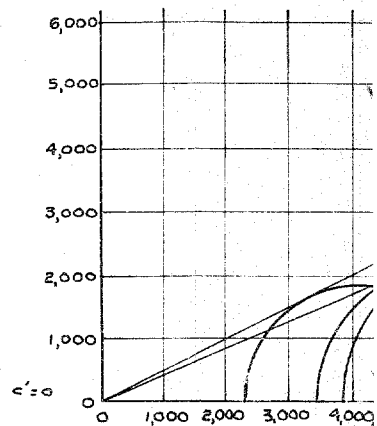
EFFECTIVE AXIAL STRESS LBS./SQ. FT.



EFFECTIVE AXIAL STRESS LBS./SQ. FT.



SHEAR STRESS LBS./SQ. FT.



SYMBOL TEST

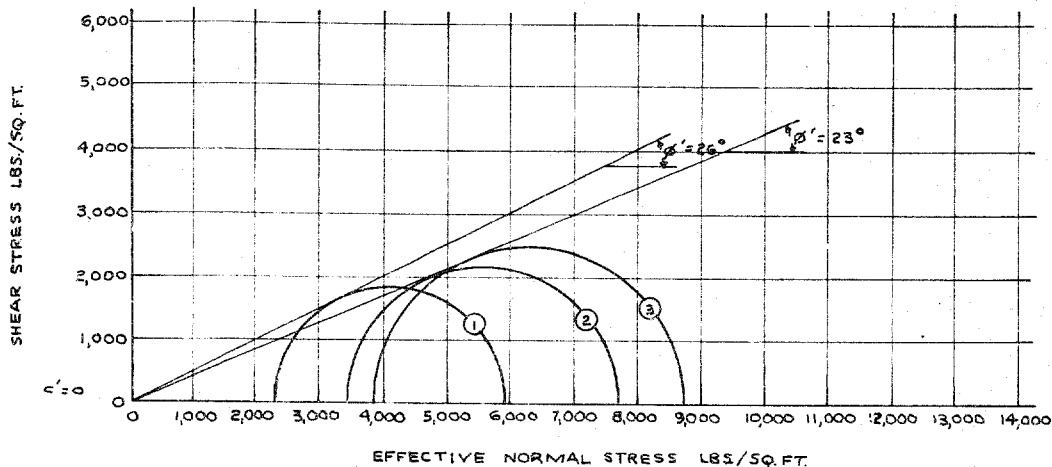
• 1
o 2
x 3

NOTES: 1) FILTER

2) RATE OF

BOR

DEPARTMENT OF
TORONTO
PROPOSED W
WELLAND
UNDRAINED
WITH PORE PRESS



<u>SYMBOL</u>	<u>TEST</u>	<u>X</u>	<u>M.C.</u>	<u>L.L.</u>	<u>P.L.</u>	<u>P.I.</u>
•	1	123	30.8	39.2	20.0	19.2
o	2	120	33.0	—	—	—
x	3	117	37.6	57.8	24.6	33.2

NOTES: 1) FILTER STRIPS, POROUS STONES TOP AND BOTTOM.

2) RATE OF AXIAL STRAIN 0.27 % / HOUR.

BOREHOLE No. 9B, SAMPLE 1, DEPTH 20.0' - 21.5'.

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED WELLAND TUNNEL
WELLAND ONTARIO
UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS

GEOCON LTD

DATE MAY 10, 1961 SCALE AS SHOWN

MADE CHKD APPD
M.W. F.H. [initials]

No. S 7219 - 2