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# Laboratory and Field Strength of Mine Waste Rock

*D. J. WILLIAMS and*
*L. K. WALKER*



*esearch Report No. CE 48*
*November, 1983*

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Department of Civil Engineering,
University of Queensland,
St Lucia, Q 4067, Australia,
[Tel:(07) 377-3342, Telex:UNIVQLD AA40315]

# LABORATORY AND FIELD STRENGTH
# OF MINE WASTE ROCK

by

D. J. Williams, BE, PhD, MIEAust.
Lecturer in Civil Engineering,
University of Queensland

and

L. K. Walker, BE, MEngSc, PhD, MBA, MIEAust.
Principal,
Golder Associates Pty Ltd

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## Synopsis

*Laboratory testing of samples of waste rock fill at Bougainville, Papua New Guinea, has enabled the quality of the rock fill to be defined in terms of its particle size distribution. The strength of poor quality rock fill has been determined by triaxial testing and by back-analysis of observed waste dump slips, and slope stability charts have been prepared for waste dump design. The results obtained from the strength testing have been compared with the results of previous strength testing on Panguna andesite, and with the results of strength testing of rock fill materials for use in dam construction. The strength parameters obtained for poor quality Panguna andesite rock fill were substantially lower than the previously reported results.*

CONTENTS



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# 1. INTRODUCTION

Open pit mining operations at the Bougainville copper mine produce very large quantities of waste rock of variable quality. The waste rock comprises predominantly Panguna andesite with a degree of weathering ranging from moderately weathered to fresh. In addition, waste referred to as superficials, and comprising soil and extremely to highly weathered rock, is recovered from near surface stripping of the open cut area. The superficials are disposed of in a waste dump separate from the waste rock dumps.

The waste rock is transported down the valley from the pit in haul trucks and end-dumped down the steeply sloping sides of the valley, forming waste dumps up to 250 m vertically from crest to toe. The stability of these dumps is controlled largely by the strength of the rock fill, provided that the natural slope has been stripped of vegetation and soft materials. The strength of the rock fill in the waste dumps depends on the degree of weathering and fracturing of the rock, which is reflected by the particle size distribution of the rock fill. The waste rock fill at Bougainville ranges from the low strength, relatively high fines content, weathered, poor quality rock fill to the high strength, low fines content, virtually unweathered, good quality rock fill. Depending on the degree of weathering, appreciable particle breakdown can occur on haulage and end-dumping of the waste rock and further breakdown would accompany any subsequent slips in the waste dump. The occurrence of slips seems primarily related to the rapid dumping of waste rock, either on an existing waste dump or on a natural slope with a cover of soft materials. To date, there is little evidence of significant particle breakdown as a result of on-going chemical breakdown of the rock fabric.

This paper concerns one particular waste rock dump with a vertical height of about 200 m, where successive rapid dumping operations resulted in two substantial slips in the waste dump at the same location. The particular waste dump was of some significance since it was designed to carry a valley access road down its face, hence long term stability with an acceptable factor of safety was required. Following the slips, concern existed as to the properties of the waste rock being included in the fill, and a programme of laboratory testing was commenced to better define the quality of the rock fill and to compare its laboratory strength parameters with those assumed in the original design of the waste dump. In addition, it proved possible to undertake a back-analysis of the post-slip surface profile to determine likely field strength parameters. Correlating the results of the laboratory strength testing with the results of the back-analysis provided a basis for adopting strength parameters for the rock fill in the waste dump and gave confidence to the design of the slope.

## 2. LABORATORY TESTING PROGRAMME

### 2.1 Particle Size Distribution Analysis

To enable a partially quantitative classification of waste rock fill quality, a number of waste rock fill samples were subjected to particle size distribution analysis. Each sample was first passed through 200 mm and 60 mm aperture field sieves before laboratory sieving of the -60 mm fraction. Seven samples were taken from fresh rock waste dumps, two samples were taken of the moderately to slightly weathered rock fill in the area of the observed waste dump slips and one sample was taken from the superficials dump.

The range of particle size distributions obtained is shown on Figure 1. On the basis of the particle size distribution curves obtained,

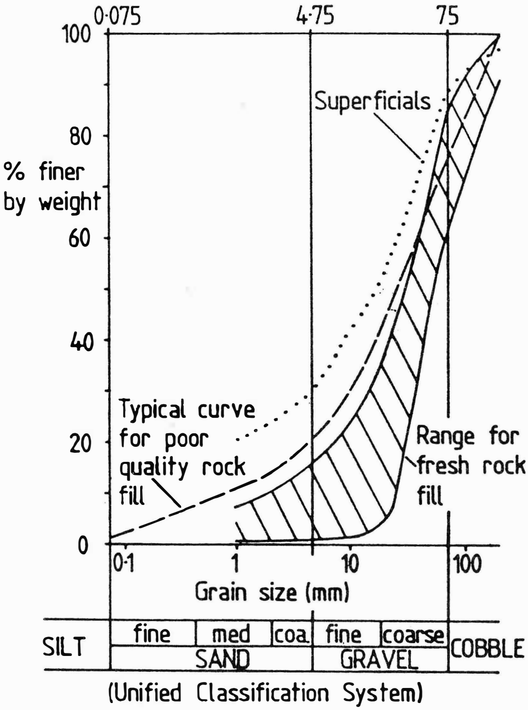



FIGURE 1: Particle size distribution curves

three categories of waste material were defined as follows.

(i) Good quality rock fill - less than 15% sand size or finer and about 25 to 35% cobble size or coarser, comprising fresh rock.

(ii) Poor quality rock fill - about 20% sand size or finer and about 25% cobble size or coarser, comprising moderately to slightly weathered rock.

(iii) Superficials - about 30% sand size or finer and about 10% cobble size or coarser, representing the upper limit on the particle size distribution plot for dumped waste material.

## 2.2 Triaxial Testing of Poor Quality Rock Fill

The triaxial testing was performed on the -19 mm fraction of poor quality rock fill samples 1 and 2 obtained from the area of the observed waste dump slips. The particle size distributions for the triaxial samples were obtained from the prototype distributions by scalping (that is, adopting a maximum particle size of 19 mm and scaling up the percentages passing all smaller sieve sizes to give 100% passing 19 mm). For both samples 1 and 2 the sieve sizes through which 50% of the samples pass ($D_{50}$) were about 5 mm and 25 mm for the triaxial samples and prototype rock fill, respectively, giving a scaling ratio of 1 to 5. Results presented by Frost (1974) indicated that a scaling ratio of 1 to 5, with a maximum triaxial sample particle size of 19 mm, would reduce the effective angle of internal friction obtained by 1 or 2 degrees.

The triaxial samples were nominally 150 mm in diameter by 300 mm in height. They were prepared either loose, to simulate waste dump conditions, or compacted, to assess the effect of initial sample density on strength. Testing was carried out at natural moisture content or under saturated conditions, and sample drainage during testing was permitted. Confining pressures of between 140 kPa and 640 kPa were applied to the sample with single stage axial loading to failure at a strain rate of 1.27 mm/min. The triaxial test conditions most closely representing the conditions prevailing in the waste dump are loosely placed, free draining rock fill at a moisture content of about 4% and under a confining pressure of up to 400 kPa (equivalent to a rock fill thickness of up to 25 m). The main aim of the triaxial testing was to establish the effective stress strength parameters for the poor quality rock fill in the waste dump under the prevailing conditions. However, the testing was also designed to assess the effects of a higher initial density and saturation on strength.

Details of the triaxial test procedure and a tabular summary of the tests performed on poor quality rock fill samples 1 and 2 are presented in Appendices A and B, respectively. The area corrected axial stress versus axial strain curves are presented in Appendix C.

The failure envelopes obtained from the triaxial testing of poor quality rock fill samples 1 and 2, under the various conditions, are presented on Figure 2. Comparisons of the five curves can be made to illustrate the effects of initial density and saturation on the strength parameters and a comparison can be made of the strength parameters of samples 1 and 2 under the conditions prevailing in the waste dump. The conclusions derived can be summarised as follows.

(i) Density has little effect on the shear strength of the material (curves A and B).

(ii) A significant reduction in shear strength results from saturation of initially loose samples (curves A and C). This effect diminishes with increasing effective confining pressure.

(iii) Saturation has a negligible effect on the shear strength of a compacted sample (curves B and D).

(iv) Sample 2 had a similar shear strength to that of sample 1 up to an effective confining pressure of about 300 kPa, but had a significantly lower shear strength at high confining pressures.

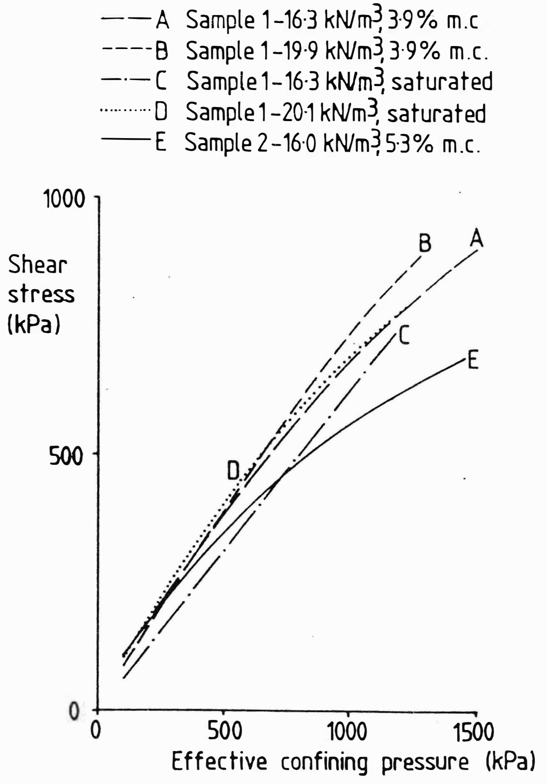



FIGURE 2: Comparison of failure envelopes

At an effective confining pressure of 400 kPa, which is appropriate to the field situation, samples 1 and 2 displayed similar shear strengths and the effect of initial density was negligible. However, saturation of initially loose material led to a reduction in shear strength of about 25% at an effective confining pressure of 400 kPa.

Particle size distribution analysis was performed on the triaxial samples after testing at a confining pressure of 640 kPa to assess the amount of particle breakage taking place during testing. This was aimed at giving some indication of the amount of particle breakage likely to occur on hauling, dumping and subsequent deformation of poor quality rock fill. The amount of particle breakage for triaxial samples from sample 1 was found to be small and largely independent of the initial test conditions, other than confining pressure. Typical particle size distribution curves for before and after triaxial testing at a confining pressure of 640 kPa are shown on Figure 3. The amount of particle breakage may be expressed quantitatively by summing the increases in the percentages retained on each sieve size after breakage compared with before to give the "B" factor. The B factor for the particle breakage indicated on Figure 3 is 12.5%.

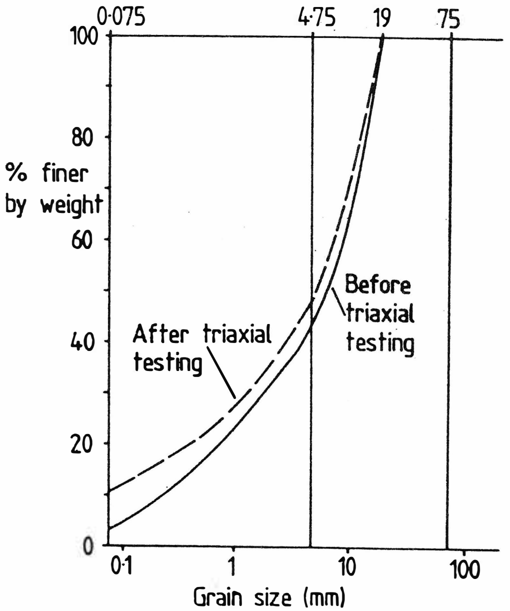



FIGURE 3: Particle size distribution curves for before and after triaxial testing of sample 1

Triaxial testing of sample 2 at a confining pressure of 640 kPa gave rise to significantly more particle breakage than did similar testing of sample 1. The before and after particle size distribution curves for sample 2 are shown on Figure 4, from which a B factor of 17.8% was calculated.

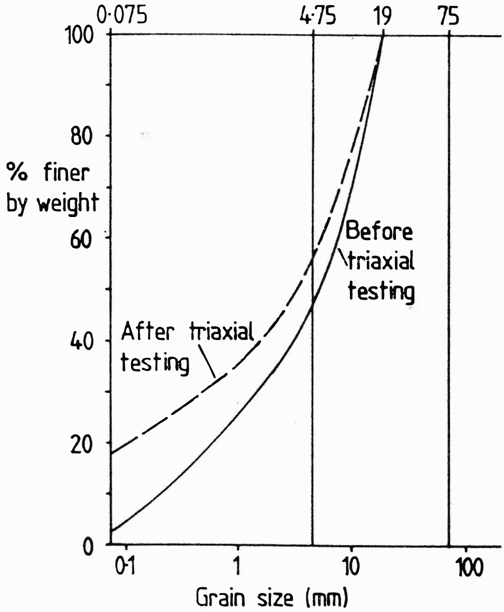



FIGURE 4: Particle size distribution curves for before and after triaxial testing of sample 2

Different degrees of breakdown are evident from Figures 3 and 4, but it is clear that a significant increase in the proportion of fines takes place on testing of the poor quality rock fill.

## 3. REVIEW OF PUBLISHED ROCK FILL STRENGTH DATA

Since 1969, a number of laboratory testing programmes has been directed at assessing the strength of Panguna andesite rock fill. The results of the testing were summarised by Scott (1977). To enable comparison of the results, the effective cohesion c' was assumed to be zero for all tests and the effective angle of internal friction $\phi'$ was calculated from the effective principle stresses at failure.

The values of $\phi'$ obtained are plotted against effective confining pressure to a logarithmic scale on Figure 5. Included on Figure 5 are the $\phi'$ values for samples 1 and 2 tested under conditions prevailing in the waste dump. The results indicate that $\phi'$ decreases with increasing effective confining pressure and reduces substantially as the degree of weathering of the rock fill increases. It is noted that the $\phi'$ values obtained from the triaxial testing of samples 1 and 2 are substantially lower than those reported by Scott.

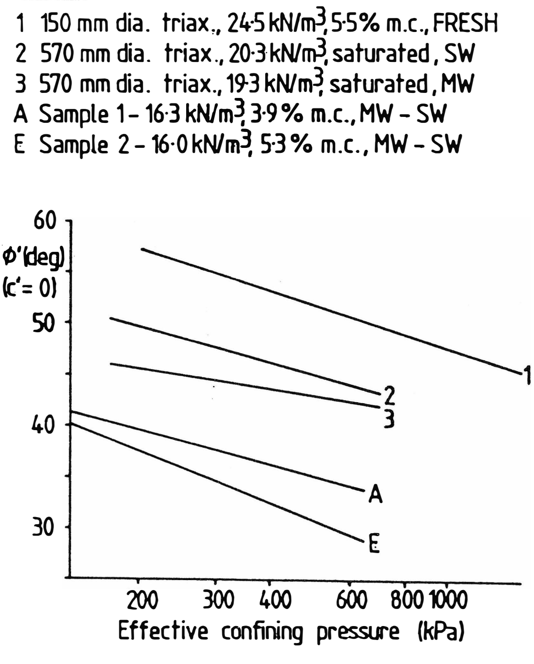



FIGURE 5: $\phi'$ versus effective confining pressure for Bougainville rock fill

Leps (1970) collected the results from a large number of triaxial tests on different rock fill types with maximum grain sizes in the range 19 to 203 mm. Based on these results, plots were made relating $\phi'$ and effective confining pressure for good quality (high density, well graded, strong particles), average and poor quality (low density, poorly graded, weak particles) rock fill. These relations are reproduced on Figure 6. More recent triaxial test results presented by Frost (1974) fall within the range defined by Leps. Also plotted on Figure 6 are selected results defining the range for Panguna andesite (lines 1, A and E from Figure 5). Line 1 plots above the relation for good quality rock fill defined by Leps, indicating negligible fines (passing 75 μm), a very high density and strong, well graded angular particles. Lines A and E plot below the relation for poor quality rock fill defined by Leps. This indicates that the poor quality rock fill from the area of the waste dump slips has a relatively high proportion of fines and comprises weak, more rounded particles. The relations defined by Leps are based on the testing of rock fill for dam construction, which would have negligible fines and therefore a higher strength than poor quality waste rock fill at Bougainville.

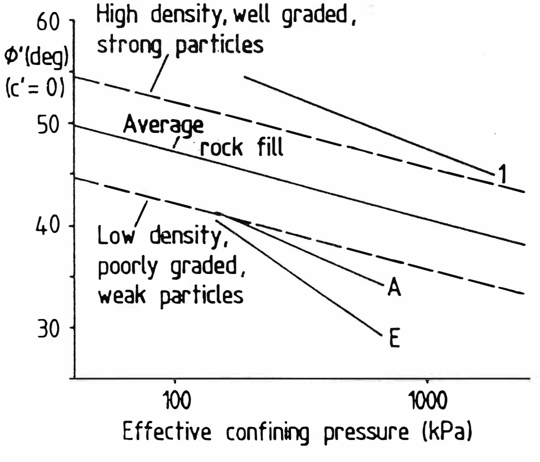



Figure 6: $\phi'$ versus effective confining pressure for other rock fill

## 4. SLOPE STABILITY ANALYSIS

A back-analysis has been performed on the area of the observed waste dump slips. The two slips were initiated at the crest of the dump by rapid dumping operations and involved waste rock fill quantities of about 100,000 $m^3$ and 50,000 $m^3$, respectively. Neither slip was actually observed in progress, however, both slips occurred rapidly and the post-slip surface profiles suggested a succession of four or five slips down the face of the waste dump. The post-slip surface profiles were characterised by a series of oversteep slopes above relatively flat benches. The rock fill within the area of the slips comprised moderately to slightly weathered rock and a relatively high proportion of fines, falling within the category of poor quality rock fill as defined in Section 2.1.

Observations of the post-slip surface profile of the waste dump indicated localised slope angles of up to $46.5^o$ standing to a vertical height of up to 25 m. The overall slope angle of the waste dump face after the second slip was about $28^o$ over a vertical height of about 200 m. A 25 m high slope at an angle of $4.65^o$ therefore represented the critical stability condition post-slip. A back-analysis of this critical condition was made using the fully drained slope stability chart of Hoek and Bray (1977) to establish a range of effective stress strength parameters for the waste rock fill in the area of the slips. As material was continually ravelling off this steep slope it was assumed to have a factor of safety of about 1.0 without earthquake loading. A unit weight of 15.8 kN/m , based on previous measurements at Bougainville (Scott, 1977), was assumed for the loosely dumped waste rock. The range of effective stress strength parameters for poor quality rock fill obtained from the back-analysis is shown on Figure 7.

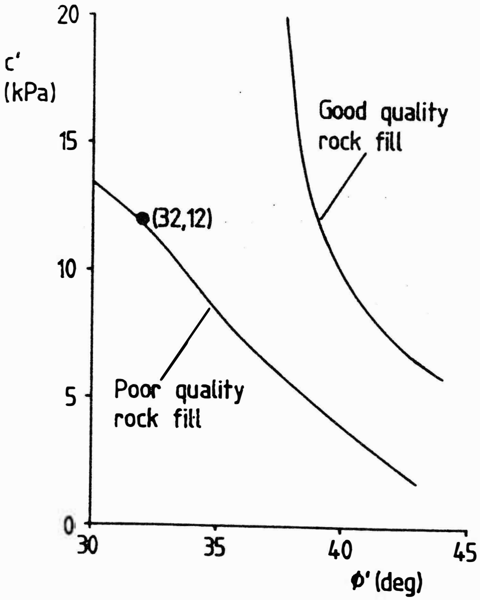



FIGURE 7: Back - calculated effective stress strength parameters

By way of comparison, from experience on Bougainville it is known that waste dumps comprising fresh rock fill rill at a slope angle of about $37.5^{\circ}$ and are stable at this angle to heights of up to 100 m under earthquake loading. Such slopes therefore possess a factor of safety without earthquake loading of at least 1.25. The range of effective stress strength parameters for good quality rock fill corresponding to this situation is also shown on Figure 7.

From Figure 7, an effective cohesion c' of 12 kPa and an effective angle of internal friction $\phi'$ of $32^{\circ}$ were selected as being representative of the poor quality rock fill in the area of the slips. Using these values, plots of slope height against factor of safety without earthquake loading may be made for various slope angles, as shown on Figure 8. These plots then form the basis for assessing the stability of the overall slope and of localised steeper slopes of limited height.

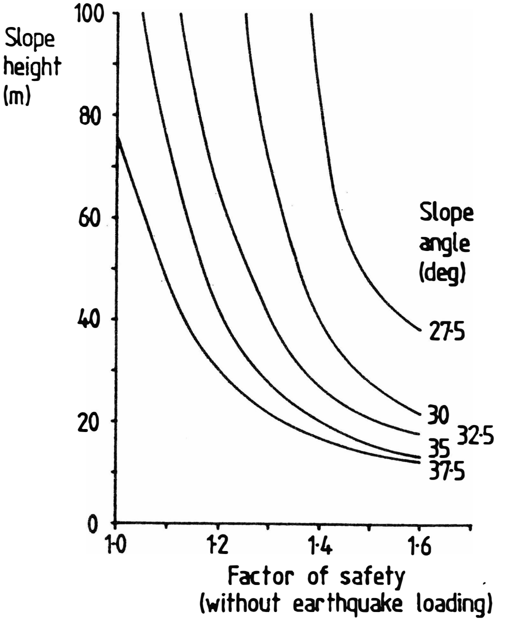



FIGURE 8: Slope height versus factor of safety for waste dump design

To enable comparison between the back-calculated effective stress strength parameters for poor quality rock fill and the results of triaxial testing of poor quality rock fill samples 1 and 2 under the conditions prevailing in the waste dump, an effective cohesion c' of 12 kPa was assumed. Values for the effective angle of interval friction $\phi'$ were obtained by drawing lines from a shear stress intercept of 12 kPa, tangential to the Mohr's circle plotted for each level of effective confining pressure applied. These $\phi'$ values are plotted against effective confining pressure and equivalent fill thickness on Figure 9 for poor quality rock fill samples 1 and 2.

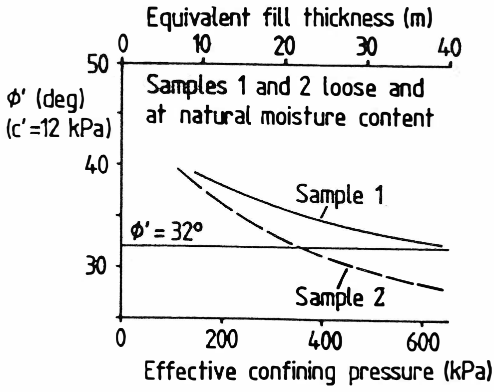



FIGURE 9: Strength versus effective confining pressure and equivalent fill thickness

The results presented on Figure 9 show that a $\phi'$ value of $32^{o}$ (for a c' value of 12 kPa) is appropriate for poor quality rock fill up to about 25 m deep. The close agreement between the laboratory and back-calculated effective stress strength parameters, as shown on Figure 9, gives considerable confidence to the use of Figure 8 for the design of waste dumps comprising poor quality rock fill.

## 5. CONCLUSIONS

The following conclusions are drawn from the results of laboratory testing and back-analysis presented in this paper.

(i) The strength characteristics of waste rock fill can be related specifically to the particle size distribution of the waste rock, with the proportion of sand size or finer being less than 15% for good quality rock fill and a least 20% for poor quality rock fill.

(ii) From the results of the triaxial testing of poor quality rock fill it is concluded that initial density has relatively little effect on the strength of the rock fill. However, saturation significantly reduces the strength of initially loose material, the effect diminishing with increasing effective confining pressure. The lower strength obtained for sample 2 compared with that for sample 1 appears to be related to its greater susceptibility to particle breakage on testing.

(iii) The results of laboratory testing and back-analysis showed a c' value of 12 kPa and a $\phi'$ value of $32^{o}$ to be appropriate for waste dumps comprising poor quality rock fill up to about 25 m deep.

(iv) Using these strength parameters, it is possible to prepare a family of curves relating slope height to factor of safety for a range of slope angle, enabling a satisfactory design for a waste dump of poor quality rock fill to be achieved.

## 6. ACKNOWLEDGEMENTS

The work on which this paper is based was carried out by Golder Associates Pty. Ltd. for Minenco Pty. Limited as client and Bougainville Copper Limited as owner. The authors acknowledge both client and owner for allowing the results of the work to be published.

APPENDIX A - TRIAXIAL TEST PROCEDURE

(1) Sample Preparation

- clamp 3.18 mm thick 95% latex membrane over bottom pedestal of triaxial cell (same membrane was used for all current tests)

- fill membrane with prepared sample at natural moisture content:
  (a) loose - place about 10 kg
  (b) compacted - compact about 10.25 kg in about 35 mm layers using a hammer on a steel plate

- place top platten inside membrane on top of sample and clamp membrane to it

- ensure that sample is vertical

(2) Setting up and Testing

- place cell top over sample and bolt to cell base (16 bolts to 65 foot - pound torque in correct order)

- mount cell on testing machine

- fill cell with water

- saturate sample if required through bottom pedestal

- connect cell pressure line

- apply cell pressure from two oil/water interface systems

(limited to about 640 kPa for tests at natural moisture content and 420 to 480 kPa for saturated tests which require tap water to supplement the volume of water stored in the oil/water systems)

. check seating of loading piston on top platten and zero proving ring

. to maintain negligible pore water pressure connect bottom drainage (of air) only for tests at natural moisture content and top and bottom drainage (of water) for saturated tests. (Pore water pressures generally do not exceed 10 kPa.)

. commence testing by applying axial load at a constant strain rate of 1.27 mm/min, recording proving ring (150 kN capacity) divisions every 30 seconds, and continue testing to limit of machine or piston travel.

(3) <u>Dismantling Cell</u>

. remove axial load and proving ring

. drain cell pressure

. drain cell and sample water, if any (takes 15 to 20 minutes)

. unload cell from machine and dismantle

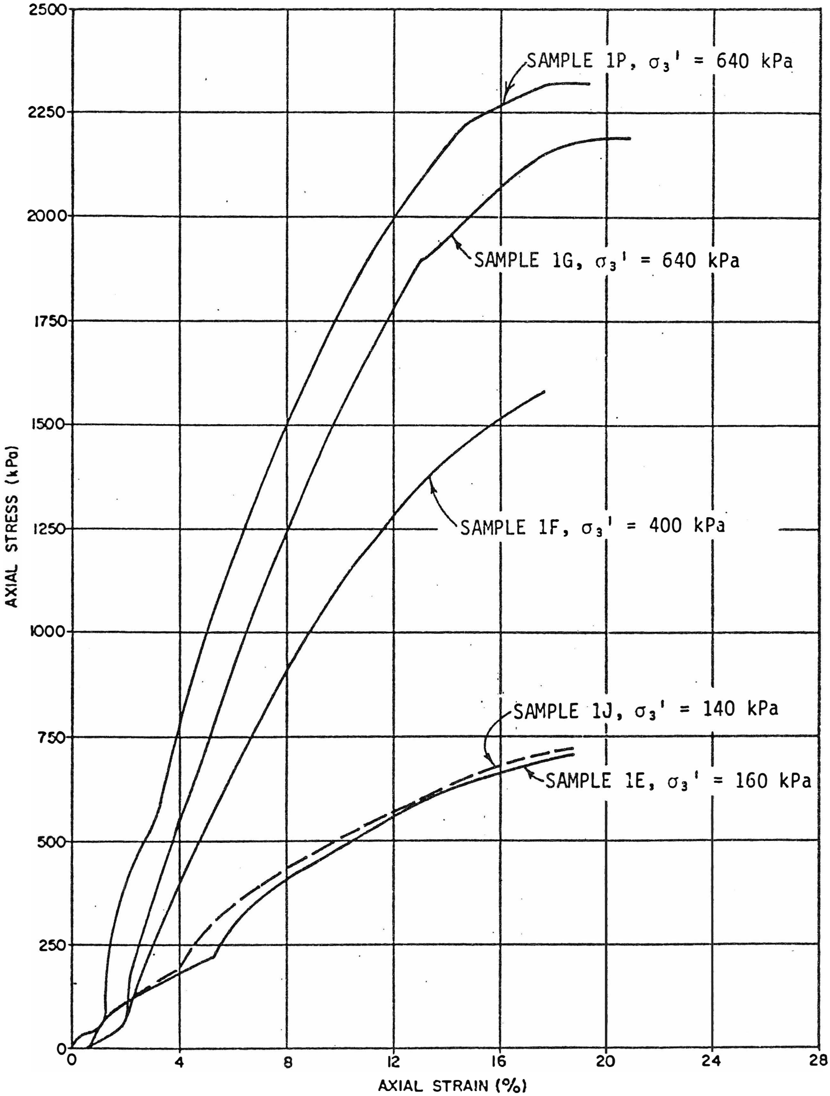
. resieve sample if required

# APPENDIX B - SUMMARY OF TRIAXIAL TESTS

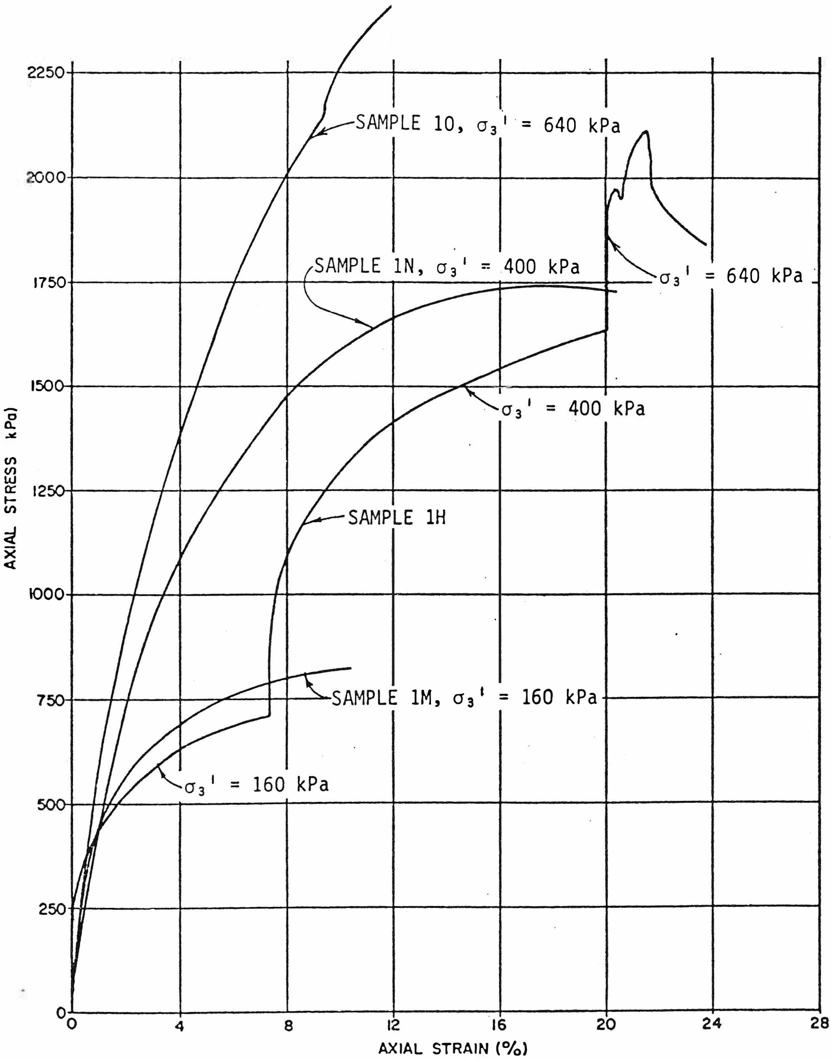
| Sample No. | Test Conditions | Initial Dry Density $\gamma_d$ (kN/m³) | Confining Pressure $\sigma_3'$ (kPa) | Peak Conditions* Axial Stress $\sigma_1'$ (kPa) | Peak Conditions* Axial Strain $\varepsilon_1$ (%) | Strength Parameters c' = 0 $\phi'$ (deg.) | Strength Parameters Straight line of best fit c' (kPa) | Strength Parameters Straight line of best fit $\phi'$ (deg.) |
|---|---|---|---|---|---|---|---|---|
| 1E | A - loose natural moisture content (3.9%) | 16.6 | 160 | 704 | 19 | 39.0 | 74 | 31.1 |
| 1F | | 15.8 | 400 | 1572 | 18 | 36.5 | | |
| 1G | | 16.2 | 640 | 2190 | 20 | 33.2 | | |
| 1J | | 16.4 | 140 | 728 | 19 | 42.6 | | |
| 1P | | 16.5 | 640 | 2314 | 19 | 34.5 | | |
| | | Av. = 16.3 | | | | | | |
| 1H | B - compacted, natural moisture content (3.9%) | | 160 | 713 | 7 | 39.3 | 87 | 31.5 |
| multi-stage test | | 20.2 | 400 | 1630 | 19 | 37.3 | | |
| | | | 640 | 2115 | 21 | 32.4 | | |
| 1M | | 20.3 | 160 | 827 | 10 | 42.5 | | |
| 1N | | 19.6 | 400 | 1736 | 17 | 38.7 | | |
| 1O | | 19.3 | 640 | 2487 | 17 | 36.2 | | |
| | | Av. = 19.9 | | | | | | |
| 1U | C - loose, saturated | 16.3 | 160 | 394 | 10 | 25.0 | -76 | 38.9 |
| 1X | | 16.6 | 300 | 964 | 13 | 31.7 | | |
| 1W | | 16.1 | 420 | 1533 | 12 | 34.7 | | |
| | | Av. = 16.3 | | | | | | |
| 1S | D - compacted, saturated | 19.8 | 160 | 737 | 15 | 40.0 | 25 | 36.0 |
| 1V | | 19.7 | 320 | 1294 | 23 | 37.1 | | |
| 1T | | 20.9 | 480 | 1971 | 17 | 37.5 | | |
| | | Av. = 20.1 | | | | | | |
| 2A | E - loose, natural moisture content (5.3%) | 17.0 | 140 | 672 | 21 | 40.9 | 117 | 24.0 |
| 2B | | 15.3 | 400 | 1351 | 17 | 32.9 | | |
| 2E | | 15.7 | 640 | 1857 | 16 | 29.2 | | |
| | | Av. = 16.0 | | | | | | |

*Area Correction Applied.

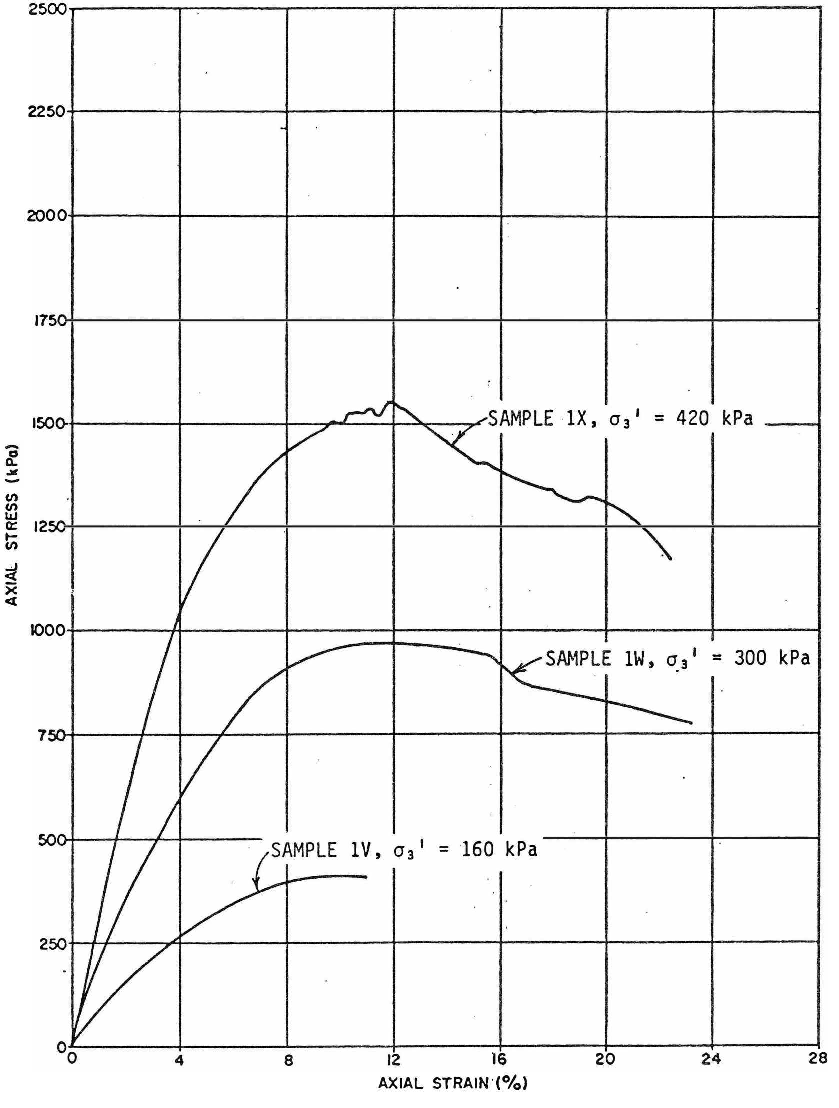
APPENDIX C - AXIAL STRESS VERSUS AXIAL STRAIN CURVES



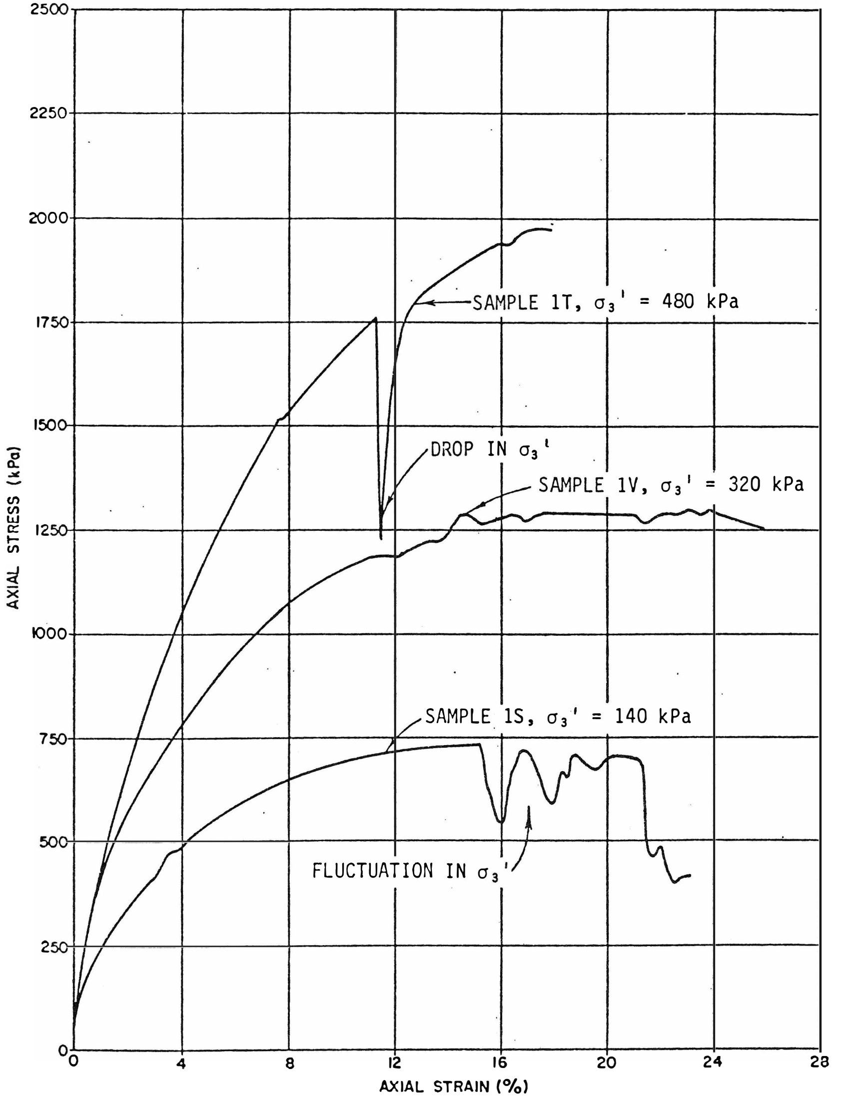

SAMPLES 1 E, F, G, J, P
MOISTURE CONTENT = 3·9%
AVERAGE DRY DENSITY = 16·3 kN/m$^3$
DRAINED TEST
AREA CORRECTION APPLIED



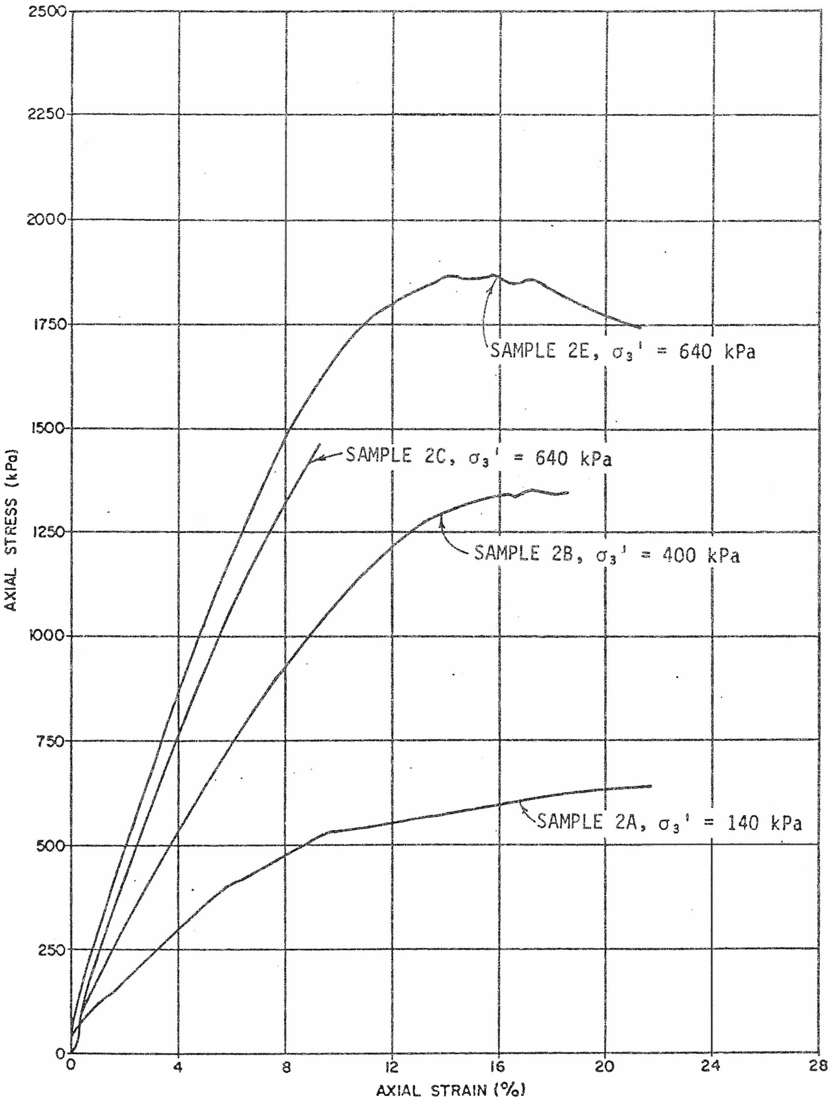

SAMPLES 1H, M, N, O
MOISTURE CONTENT = 3·9%
AVERAGE DRY DENSITY = 19·9 kN/m$^3$
DRAINED TEST
AREA CORRECTION APPLIED




SAMPLES IV, W, X
MOISTURE CONTENT — SATURATED
AVERAGE DRY DENSITY = 16·3 kN/m$^3$
DRAINED TEST
AREA CORRECTION APPLIED




SAMPLES 1 S, T, V
MOISTURE CONTENT — SATURATED
AVERAGE DRY DENSITY = 20·1 kN/m$^3$
DRAINED TEST
AREA CORRECTION APPLIED




SAMPLES 2A, B, C, E
MOISTURE CONTENT = 5·3%
AVERAGE DRY DENSITY = 16·0 kN/m$^3$
DRAINED TEST
AREA CORRECTION APPLIED

## APPENDIX D - NOMENCLATURE

| Symbol | Meaning |
|---|---|
| $D_{50}$ | Sieve size through which 50% of the sample passes |
| $c'$ | Effective cohesion |
| $\phi'$ | Effective angle of internal friction |

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