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Naval Submarine Base Kings Bay and Bangor Soil Evaluations

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Abstract

This report provides soil evaluation and characterization testing for the submarine bases at Kings Bay, Georgia, and Bangor, Washington, using triaxial testing at high confining pressures with different moisture contents. In general, the samples from the Bangor and Kings Bay sites appeared to be stronger than a previously used reference soil. Assuming the samples of the material were representative of the material found at the sites, they should be adequate for use in the planned construction. Since soils can vary greatly over even a small site, a soil specification for the construction contractor would be needed to insure that soil variations found at the site would meet or exceed the requirements. A suggested specification for the Bangor and Kings Bay soils was presented based on information gathered from references plus data obtained from this study, which could be used as a basis for design by the construction contractor.

Nomenclature

AFV	air filled voids
DG	decomposed granite
RBC	Reentry Body Complex
MMM	Missile Motor Magazine
UFC	Uniform Facilities Criteria

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Executive Summary

This report provides soil evaluation and characterization testing for the new Reentry Body Complex (RBC) and Missile Motor Magazine (MMM) designs for the submarine bases at Kings Bay, Georgia, and Bangor, Washington, using triaxial testing at high confining pressures with different moisture contents. It also presents a set of soil specifications for soil-acceptance testing during the construction process. The properties for the Kings Bay and Bangor soils considered were compared to the properties of a reference material (Decomposed Granite), which was used extensively in vulnerability calculations. The comparison determines whether the properties of the proposed soils meet or exceed those of the reference material in order to ensure that the baseline computations are conservative.

The uniform size of the Kings Bay material resulted in a much higher maximum yield stress as a function of pressure than the reference material. The yield stress at 8% moisture content was more than 120 MPa. The maximum yield stress from 4% to 8% moisture content appeared to fall along the slope of the pressure-dependent yield stress curve. This could suggest that for these low moisture contents, the AFV had not been compressed to the point where fluid pore pressure supported any of the load.

The yield stress at a moisture content of 9% dropped to only 80 MPa for 105 pcf initial density. The yield stress for the reference material was 13 MPa at the same moisture content. For a moisture content of 11%, it appeared that the air-filled voids went to zero, resulting in low yield strength. There were some anomalies within the soil data; however, anomalies in soil data are the norm, not the exception. Based on the difference between the Kings Bay material and the reference material, it was concluded that the Kings Bay material should outperform the reference material.

The Bangor material was very similar in strength to the baseline decomposed granite material. At 9% moisture content, the two materials have the same maximum stress difference. At 7%, the material has about 30% more strength than the reference material. In general, the material properties for the Bangor material are very similar to the properties of the reference material. With the uncertainties associated with the variation in soil properties, and the unknown character of the equilibrium soil moisture content, one can conclude that material properties used in the reference calculation are representative of the Bangor material properties.

In general, the samples from the Bangor and Kings Bay sites appeared to be stronger than the reference soil. If the samples of the material were representative of the material at the site, then they should be adequate for use in the planned construction. Since soil could vary greatly over even a small site, a soil specification for the construction contractor would be needed to insure that soil variations found at the site would meet or exceed the requirements.

A suggested specification for the Bangor and Kings Bay soils was presented based on the information gathered from the indicated references plus the data obtained from this study, which could be used as a basis of design by the construction contractor.

1.0 Introduction

This report provides soil evaluation and characterization testing for the new Reentry Body Complex (RBC) and Missile Motor Magazine (MMM) designs for the submarine bases at Kings Bay, Georgia, and Bangor, Washington. Triaxial testing was used at high confining pressures with different moisture contents. This report also presents a set of soil specifications for soil acceptance testing during the construction process. The properties for the Kings Bay and Bangor soils considered are compared to the properties of a reference material (decomposed granite), which was used extensively in vulnerability calculations. The comparison determines whether the properties of the proposed soils meet or exceed the properties of the reference material in order to ensure that the baseline computations are conservative.

The design process is by its nature an iterative process. The calculations to evaluate new construction and upgrades to existing construction were not based on soil properties known at the time of the simulation. The lead time required to obtain samples and the time required to perform the necessary soil tests prevented the use of the actual soil properties measured from the building sites. The initial performance of the designs at Bangor and Kings Bay was based on soil properties for a Decomposed Granite (silty to clayey sand) that had previously been well characterized for a similar impact analysis.

Soil material properties can vary widely, and at the onset of this project, it was not clear that soil located at or near the site would be adequate for construction. One option considered was to import soil from off the site. The option of importing soil or gravel at each site would greatly increase the cost of construction.

As soil properties were made available through testing, they were compared to the baseline soil to verify comparable strengths under impact. If the soil properties were not comparable, then additional computations using the actual soil properties were performed to ensure that the structures would meet design goals.

2.0 Kings Bay and Bangor Soil Evaluations

2.1 Soil Behavior

Soil behavior is quite complex, and many factors can contribute to the wide response of soils to impact and blast loads. Porosity, water content, unit weight, grain size, and density are a few of the physical characteristics that determine soil behavior.

A simple view of soil under high compressive loads is shown in Figure 1. In this figure, the interaction of air, soil, and water are shown as acting independently. The water and air actually exist in the void space between the soil grains. During the initial crush phase, the air does not contribute significantly. The load is supported by the inter-granular contact between the grains of soil.

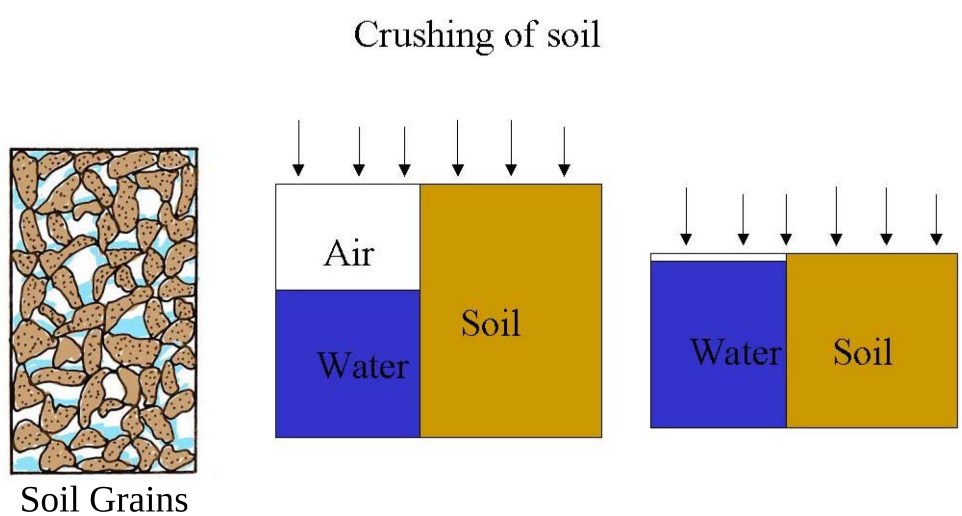


Figure 1. Soil, Water, and Air.

Deformation in soil is controlled by the interaction between individual particles, especially the sliding of particles. As sliding particles rearrange the void space between the particles, the soil volume is reduced. The soil then becomes denser. In addition to sliding, the contact interaction changes as the soil volume is reduced. Because of the movement of particles, more contacts are made. The normal force at each contact increases, resulting in a greater frictional force. This higher frictional force prevents the granules from sliding relative to each other and increases the soils' resistance to shear. At pressures above 2000 psi, the crushing of granular particles can occur, resulting in further volume reduction. The dependence of the frictional force between grains on the confining pressure leads to a shear strength (yield strength) that is a function of pressure.

The pore spaces between the soil particles can be filled with fluid and/or air. If the pore spaces are fully occupied by fluid, then the fluid will support load. In general, the fluid is much stiffer in compression than the soil matrix, resulting in a noticeable stiffness change when the air-filled voids (AFVs) are removed from the soil.

For static loads, the fluid and soil structure generates a time-dependent response (known as consolidation) as the fluid moves through the pores and flows away from the highly stressed region. For impact loads, it is assumed that the fluid will not have time to flow. Once the fluid has started supporting the load, the high pressure in the fluid limits the load on the soil structure and, thus, changes the way it behaves in shear. The contact forces at intra-granular soil boundaries no longer increase, and as a consequence, the shear strength no longer increases as the confining pressure increases.

In summary, moisture content can have a dramatic effect on the maximum shear strength. Moisture dependence is caused by the interaction of the response of the soil matrix and the response of fluid between the matrix of soil particles. At low pressures, the shear strength of the soil increases with pressure as the individual soil particles are pressed into contact. The fluid between the pores begins to support stress only when the air-filled void space between the soil grains is reduced to zero as a result of compressive volume strain. The shear strength will increase as a function of pressure until the fluid supports a portion of the load. Since the fluid is almost incompressible, only elastic changes in volume occur as the pressure increases beyond the limit of void space closure. Since the fluid pressure now supports the dilatational portion of the stress, the inter-grain contact pressure no longer increases. The shear strength then becomes, to a first approximation, independent of the pressure.

2.2 Laboratory Measurements

Based on the above observations of how the fluid, air, and soil granules interact, one can see that the void ratio to solid for a soil, combined with the water content, can dominate the soil behavior during impact. While the general trends in behavior are known, there is no theory for accurately predicting the response of soil based on the constituents. The soil properties must be measured.

To capture the soil behavior in a laboratory setting, the soil was tested using a triaxial testing machine that allows the application of a uniform pressure to the soil, followed by the loading along a single axis. Polyethylene shrink-fit tubing was used to jacket the sample to exclude the fluid used to apply confining pressure. All tests were conducted in a servo-controlled testing frame using a standard rock-mechanics testing setup, including a pressure vessel for applying the confining pressure and a loading ram for applying stress along the axis of the specimen. This is shown pictorially below in Figure 2. The soil was tested in an undrained state to reflect the lack of transport of fluid during impact. The testing procedure is described in detail in Appendix A, Section A2. From the triaxial data, the shear strength as a function of pressure can be measured.

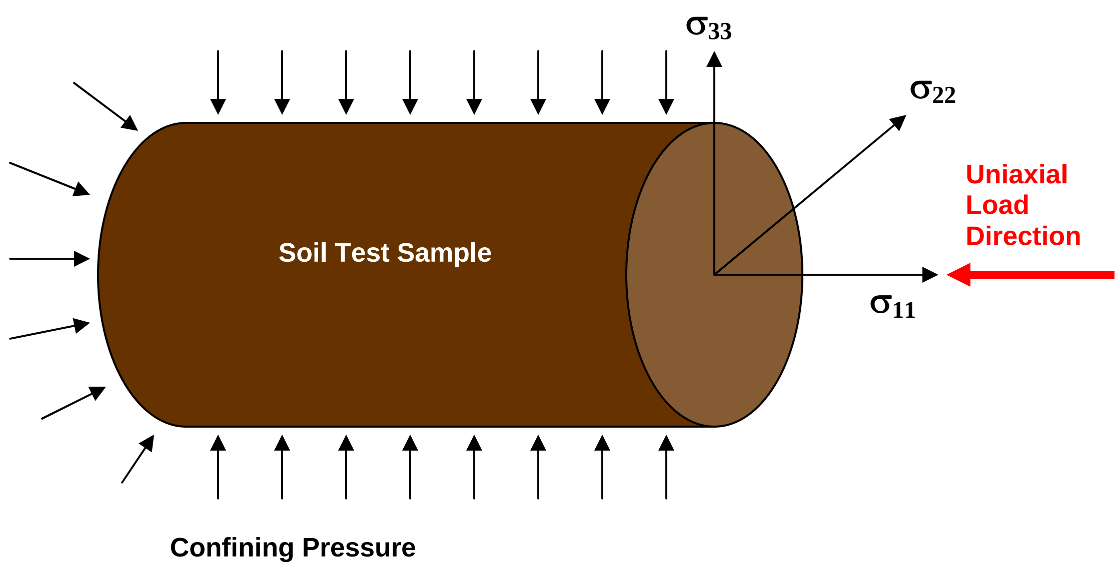


Figure 2. Triaxial Testing Pictorial

Figure 3 shows a plot of the triaxial data for the decomposed granite (DG) used in the initial impact calculations. The figure shows the stress difference $\sigma_D = \sigma_{11} - \sigma_{33}$ (proportional to shear strength) as a function of mean stress $\sigma_m = (\sigma_{11} + 2\sigma_{33})/3$ for different moisture contents. For very low soil moisture, the shear strength continues to increase with pressure, as can be seen for the 3% moisture content. For high soil-moisture content, (11%), the shear strength reaches a maximum of about 6 MPa. For a soil with 7% moisture content, the shear strength levels off at about 25 MPa.

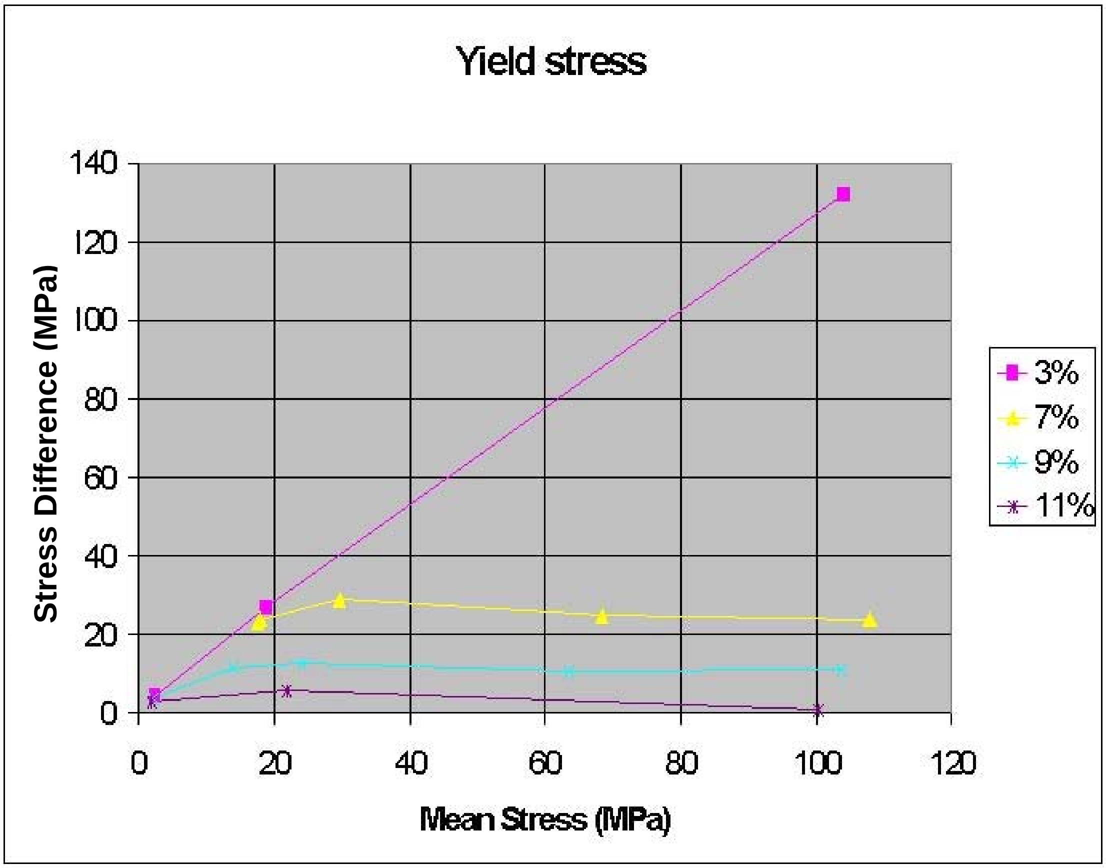


Figure 3. Yield Stress as a Function of Mean Stress for the Baseline DG Used in the Initial Impact Calculations.

Figure 4 shows the relation between the moisture content and the stress difference (proportional to maximum shear strength) for DG. This relation can be used to plot the soil strength as a function of depth, once the moisture content has been measured as a function of depth. A moisture gradient within the soil can be modeled by allowing the material properties to vary with the location. A simulation using varying moisture content is expensive, so we assumed a constant soil moisture content. This assumption is reasonable if care is taken to ensure that the soil is well drained and hydrologically designed to prevent flooding and water ponding.

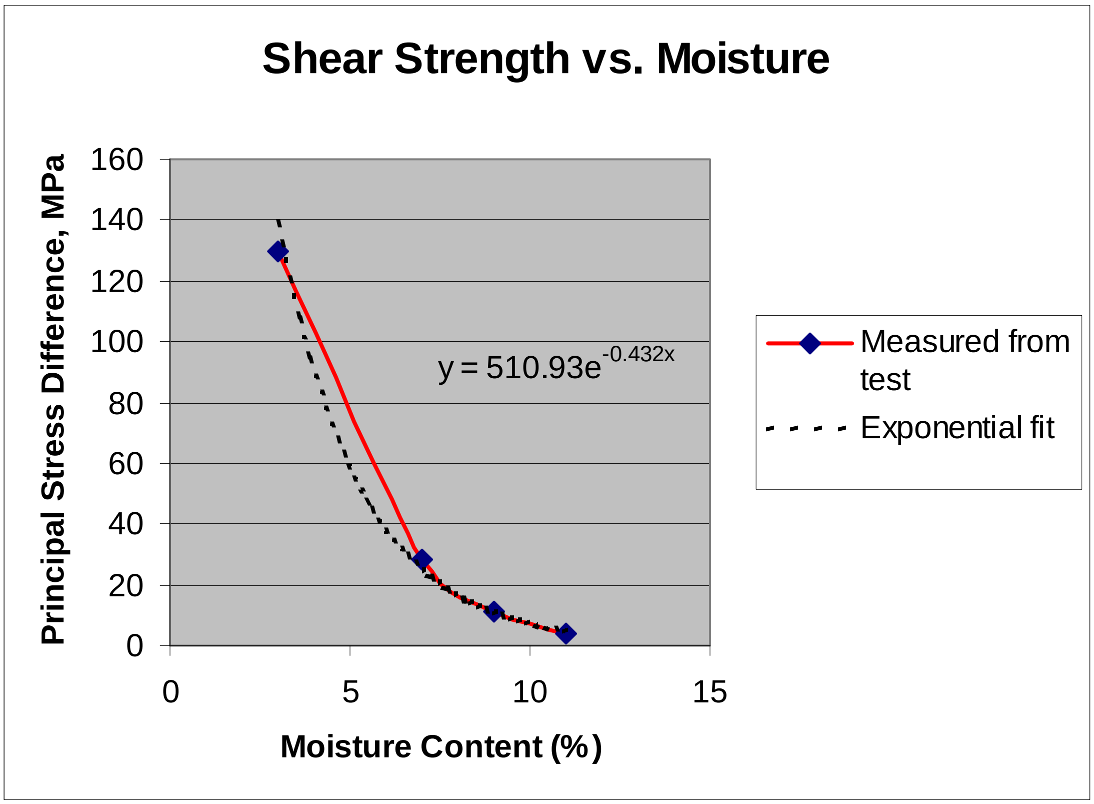


Figure 4. Maximum Shear Strength as a Function of Moisture for DG.

In addition to the triaxial data, the pressure as a function of volume strain was also measured for an undrained specimen. Figure 5 shows a plot of the pressure as a function of volume strain for the baseline soil (DG) used in the baseline design calculations. The two-phase crush behavior is clearly visible in this plot, shown for a soil with an 8% moisture content and a total void volume of 30%. For volume strains of less than 15%, the pressures remain below 20 MPa. For volume strains beyond 15%, the volumetric stiffness response increases by an order of magnitude. At 16%, the pressures are more than 80 MPa.

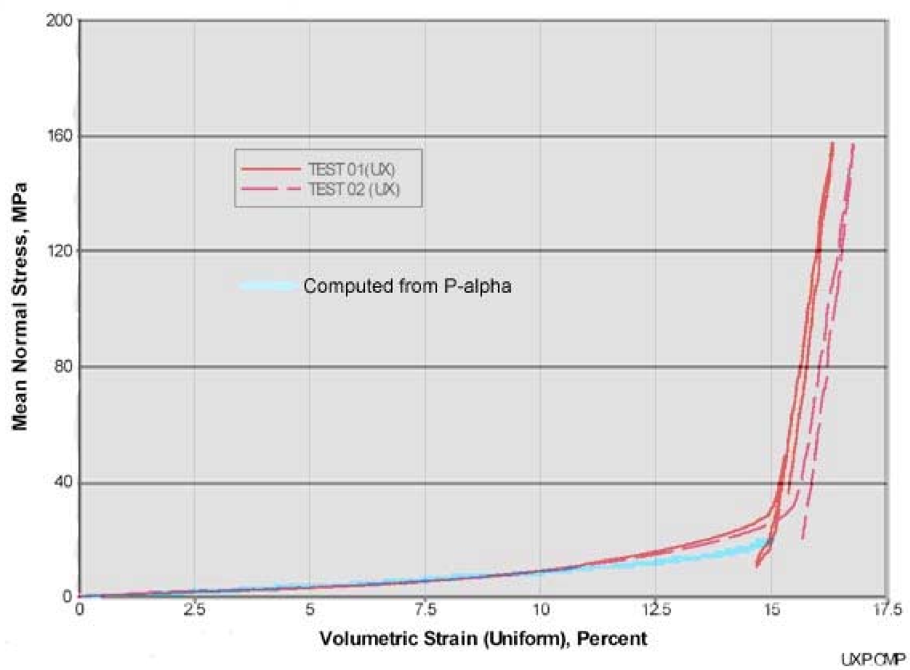


Figure 5. Pressure as a Function of Volumetric Strain for Baseline Soil (DG).

2.3 Desirable Soil Properties

Soil designs for mitigating aircraft impact require high shear strength, combined with the ability to absorb energy during volume crush. Impacting a soil with high water content limits the maximum shear strength and results in an elastic hydrostatic load, once the AFVs have been eliminated.

The energy absorbed during impact will be greater for soils with a higher percentage of AFVs. Obviously, a saturated soil will have no AFVs and, thus, will be a poor absorber of impact energy. The percentage of AFVs typically varies between 0% and 36% (see Table 8-3, page 8-14 of the Uniform Facilities Criteria (UFC) [UFC 3-340-01, 2002] for a list of typical AFV properties for several types of soil).

AFVs are greatest for nonuniformly graded soils. Well-graded soils have the least AFV. Clayey soils generally retain moisture through capillary action and have a very low percentage of AFVs. Hence, soils with a high clay content should be avoided for mitigating impacts. The void ratio corresponding to a cubic type arrangement of spheres is 0.91. For a pyramidal arrangement of spheres, the void ratio is 0.34. Granular soil particles are neither the same size nor perfect spheres. The smaller particles may occupy void space between the larger ones, which tends to reduce void ratio as compared with that for equal spheres. The irregular shape, however, of the particles generally increases the soil void ratio, as compared to ideal spheres. Typical void ratios for soils are shown in Table 1.

Table 1. Void Ratio for Different Soil Types.

Soil type	Maximum void	Minimum void	Minimum dry density (g/cc)	Maximum dry density (g/cc)
Gravel	0.6	0.3	1.6	2.0
Fine sand	0.85	0.4	1.4	1.9
Silty sand and gravel	0.85	0.15	1.4	2.3
Gravely sand	0.7	0.2	1.5	2.2

The soil water content is defined as the ratio of the water weight to the dry weight of the aggregate. This measure is usually expressed as a percent. As an example, consider a soil with a dry AFV ratio of 30%. Moisture content is measured by the ratio of the weight of water to the weight of the soil. The “solid” matrix typically has a density more than twice the density of water; thus, the soil becomes saturated at moisture contents between 12% and 18%. Figure 6 shows the relation between moisture content and saturation. In the example shown, the soil is 100% saturated, with the void volume completely filled by fluid at a moisture content of 16%. When the soil is 50% saturated, one-half of the void volume is filled with fluid. For this example, 50% saturation occurs at only 8% moisture content.

Gravimetric Water Content, ω

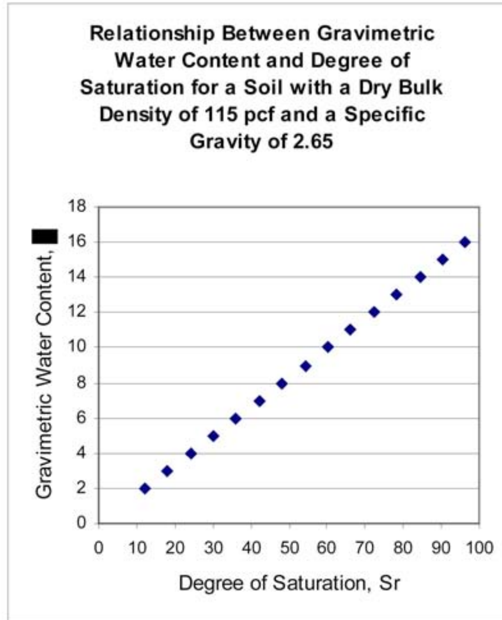
$$\omega = \frac{W_w}{W_s} \times 100$$

The ratio of the weight of the water in a sample to the dry weight of the soil in the sample expressed as a percentage

Degree of Saturation, S_r

$$S_r = \frac{V_l}{V_v} \times 100$$

The ratio of the volume of liquid in a sample to the volume of the voids in the sample expressed as a percentage



The functional relation between the degree of saturation and the gravimetric moisture content is given

$$\omega = S_r \times \left(\frac{62.43}{\gamma_d} - \frac{1}{G_s}\right)$$

Figure 6. Water Content as a Function of Saturation.

Part of the void may be occupied by air for coarse-grained soils located above the water table. Soil capillary action results in some water always remaining in the soil. A good measure for the amount of water in soil is the degree of saturation. The degree of saturation represents the ratio of the water volume to the void volume for a dry soil. The degree of saturation is usually described by words such as dry or moist (Dry 0, Humid 1–25%, Damp 26–50%, Moist 51–75%, Wet 76–99%, Saturated 100%).

Good energy absorption from soil requires adequate drainage so that the AFV percentage is maximized. A good way to maximize the drainage is to place a high-porosity material between the roof of the structure and the soil cap (e.g., 1 ft of ¾-inch gravel covered by a geotextile to allow moisture transport without soil transport). The roof below the soil should be sloped to insure adequate flow with provisions to guarantee that water does not stand. If a concrete cap is used, it should be designed to shed and remove water so that the soil does not become flooded.

The water-holding capacity is the smallest value to which the water content of the soil can be reduced by drainage. Figure 7 shows the ratio of air space to water as a function of soil grain size. Three curves are shown on this graph. Curves A and B are the results of soil drained using a centrifuge and a vacuum. Curve C is the result of drainage in the field. As can be seen, soil holds moisture even when drained. A grain size of 0.1 mm should drain to an air void ratio of 0.6. (60% of voids are filled by air). As a design goal to insure a high AFV, soil grain size should be as large as possible. Good impact performance should result from a soil with over 60% AFV.

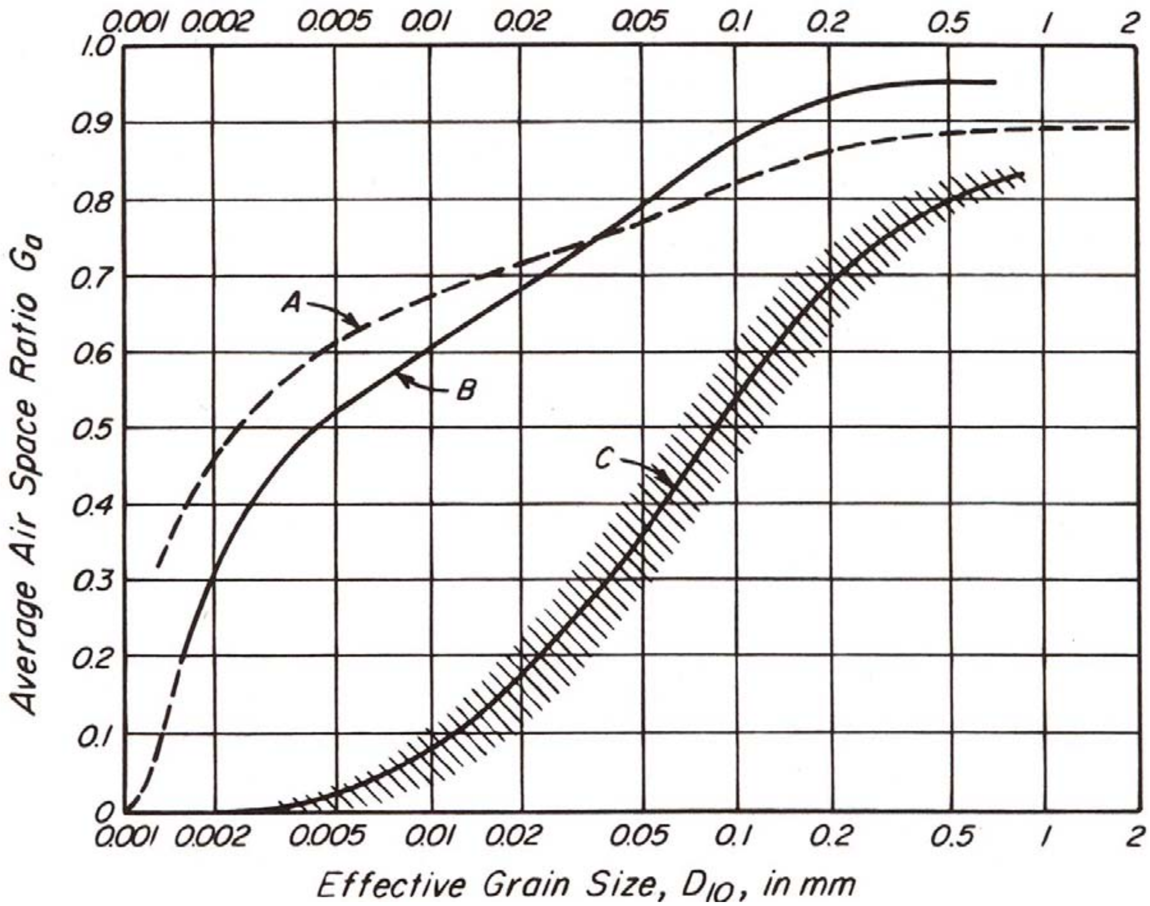


Figure 7. Air Space Ratio as a Function of Grain Size (Terzaghi et al., 1996).

2.4 Modeling Approach

When the load is shared between the soil and the pore fluid, one approach is to define an effective stress based on the difference between the soil stress and the pore pressure. In the material models used for computing the soil response to an aircraft impact, the effective stress will not be explicitly computed. Instead, a simple model for the volume response and the shear response for a soil at a constant moisture content will be used.

For modeling purposes, soil behavior is usually divided into its shear and dilatational response. The shear response is typically modeled using a pressure-dependent yield surface with an asymptotic maximum strength for high pressures. The volume response is then treated as independent of the shear response, with the pressure being a function of the volume strain.

Dilatational response of soil was modeled in CTH (large displacement Eulerian computer code) using the p-alpha model [Hertel and Kerley, 1998]. The model treats porous material by dividing the dilatational response into a crushable or porous zone and a solid matrix. Once the material has been “crushed,” a Mie-Gruneisen equation of state is used to treat the hydrodynamic response of the dense material.

The shear response was treated in CTH using the Geo-Yield material model. This model allows the user to set the maximum yield stress, the y-intercept, and the slope of the pressure-dependent part of the yield curve.

For the complete data on the soil properties, see Appendices A–E

3.0 Kings Bay Summary

The uniform size of the Kings Bay material resulted in a much higher maximum yield stress as a function of pressure than the reference material. Figure 8 shows the measured data points for the Kings Bay material. The yield stress at 8% moisture content exceeded 120 MPa. The maximum yield stress from 4% to 8% moisture content appeared to fall along the slope of the pressure-dependent yield stress curve. This could imply that, for these low moisture contents, the AFVs have not been compressed to the point that fluid pore pressure is supporting any of the load.

Two different initial densities were used in the Kings Bay testing. The yield stress for the Kings Bay soil with an initial density of 105 pcf and a moisture content of 9% drops to only 80 MPa. For the Kings Bay soil with an initial density of 120 pcf at a moisture content of 9%, the yield stress drop to 16 MPa. The yield stress for the reference material was 13 MPa at the same moisture content. At the higher initial density of 120 pcf, the strength was greater for moisture contents above 8%. For moisture contents below 9%, the denser material was weaker. Higher initial density combined with high moisture content results in the AFVs going to zero sooner than for the lower initial density material, resulting in lower yield strength for the same moisture content.

There were some anomalies within the soil data; however, anomalies in soil data are the norm, not the exception. Based on the difference between the Kings Bay material and the reference material, it was concluded that the Kings Bay material should outperform the reference material. Because of the nonlinear nature of the soil behavior, a separate set of material parameter data would be fitted to the test data, and the calculations for the performance of the proposed construction would be recomputed.

Further details of the Kings Bay soil evaluation are given in Appendices A, B, C, and E.

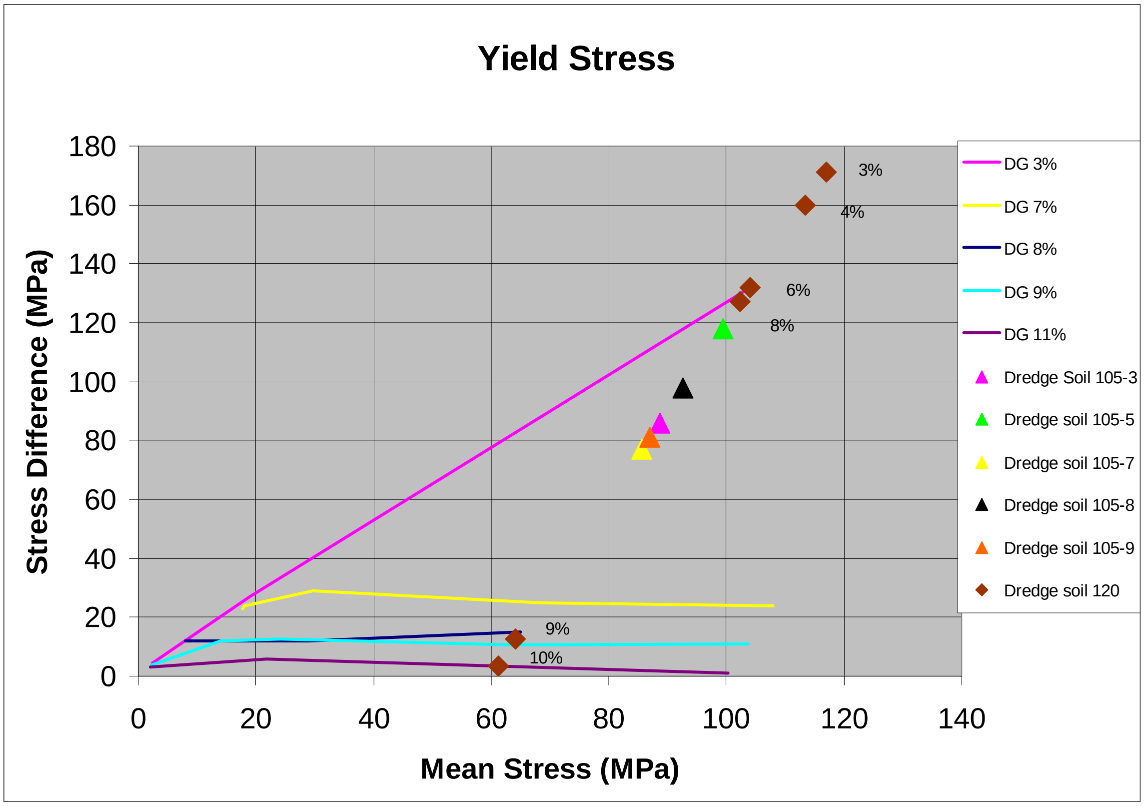


Figure 8. Kings Bay Yield Strength Compared with DG Baseline Material.

4.0 Bangor Summary

The Bangor material is very similar in strength to the baseline DG material. Figure 9 shows the maximum stress difference for the Bangor material relative to the DG material. Note that at 9% moisture content, the two materials have the same maximum stress difference. At 7%, the material has about 30% more strength than the reference material. In general, the material properties for the Bangor material are very similar to those of the reference material. With the uncertainties associated with the variation in soil properties, and the unknown character of the equilibrium soil-moisture content, one can conclude that material properties used in the reference calculation are representative of the Bangor material properties.

Further detail of the Bangor soil evaluation can be found in Appendices A and D.

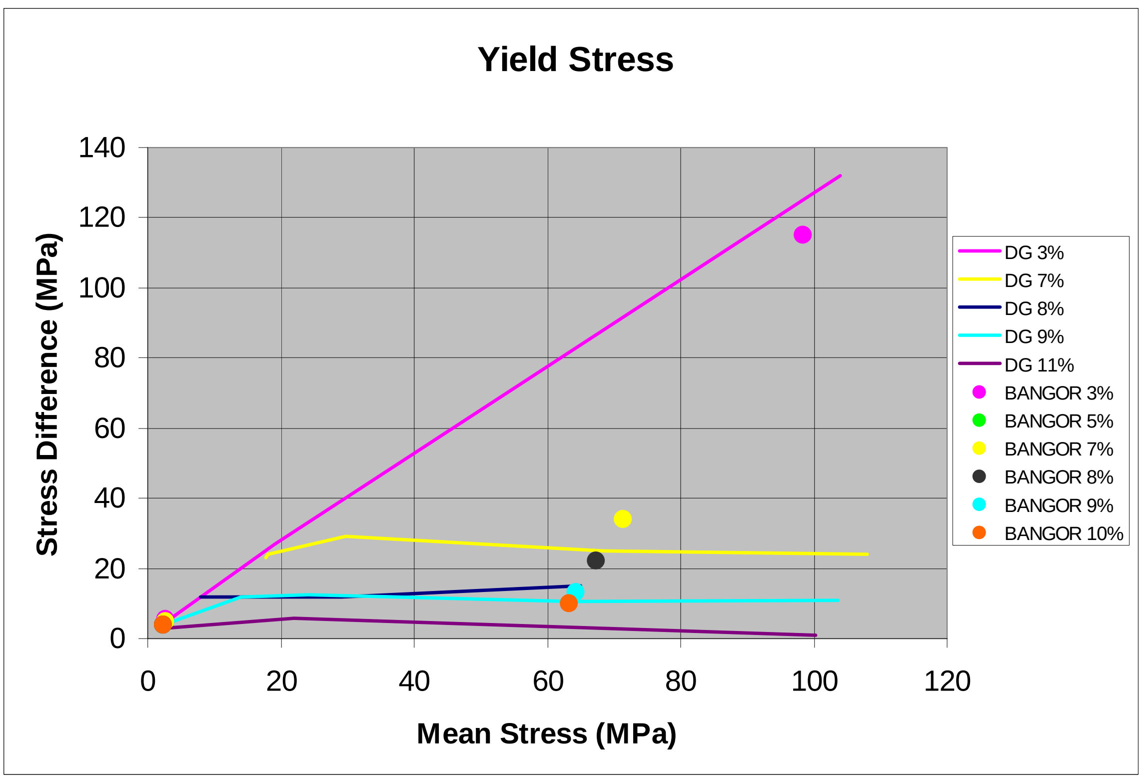


Figure 9. Yield Stress for Bangor Soil Compared with Baseline.

5.0 Soil Specification for Construction

In this section, the soil specification for a soil used in the construction process at Kings Bay and Bangor is defined.

Some good examples of soil behavior are shown on page 8-13 of the UFC [UFC 3-340-01, 2002]. Figure 8-10 [UCF page 8-12] shows typical mechanical property comparisons for several different backfill materials. Note that dry concrete sand is very strong, both in volume strain and in yield, as a function of pressure [see DSOIL3, 4, and 5 in Table 8-3, page 8-14]. The more AFVs present, the better. The DG material above had about 16% AFVs. Both the Bangor and Kings Bay material will have greater than 16 percent AFVs when well drained. The uniform sand from the Kings Bay site should have about 25% to 30% AFVs. Based on the above-referenced information and the data reported in Appendices A–E, a suggested specification for the Bangor and Kings Bay soils is shown in Table 2.

In addition to the sieve analysis testing performed at Sandia National Laboratories, the Bangor site was characterized in a report by HartCrowser, 1989. Both the Sandia and HartCrowser soil gradings for the Bangor material are summarized in Figure 10.

Also shown in Figure 10 are the soil gradings for the Kings Bay material. Note that the Kings Bay soil was washed in the dredging process. It thus appears to have no fines below the 0.25-mm size.

Soils used for impact mitigation are optimal when they have a high AFV content. To ensure a high AFV percentage, the soil should be well drained and uniform in size (poorly graded). The clay and silt content should be limited to insure low water content.

The samples from the Bangor and Kings Bay sites appeared to be stronger than the reference soil. If the samples of the material are representative of the material at the site, then those materials should be adequate for use in the planned construction. Since soil can vary greatly over even a small site, a soil specification for the construction contractor is needed to insure that soil variations found at the site will meet or exceed the requirements.

Compaction should be uniform with a minimum density of 85% as defined by a Proctor Test (ASTM D-698).

Table 2. Suggested Specification for Bangor and Kings Bay Soils.

Fill and structural backfill material shall consist of satisfactory, nonexpansive fill material, classified by ASTM D 2487 as GW, GP, GM, GC, SW, and SP and having a maximum size of 1-1/2 inches. Material shall be as close to uniform in size as possible, within the following limits as specified:

Sieve Size (in.)	Percent by Weight Passing
3	100
1 1/2	95–100
3/4	85–100
No. 4	70–100
No. 10	30–100
No. 40	15–95
No. 100	0–30
No. 200	0–10

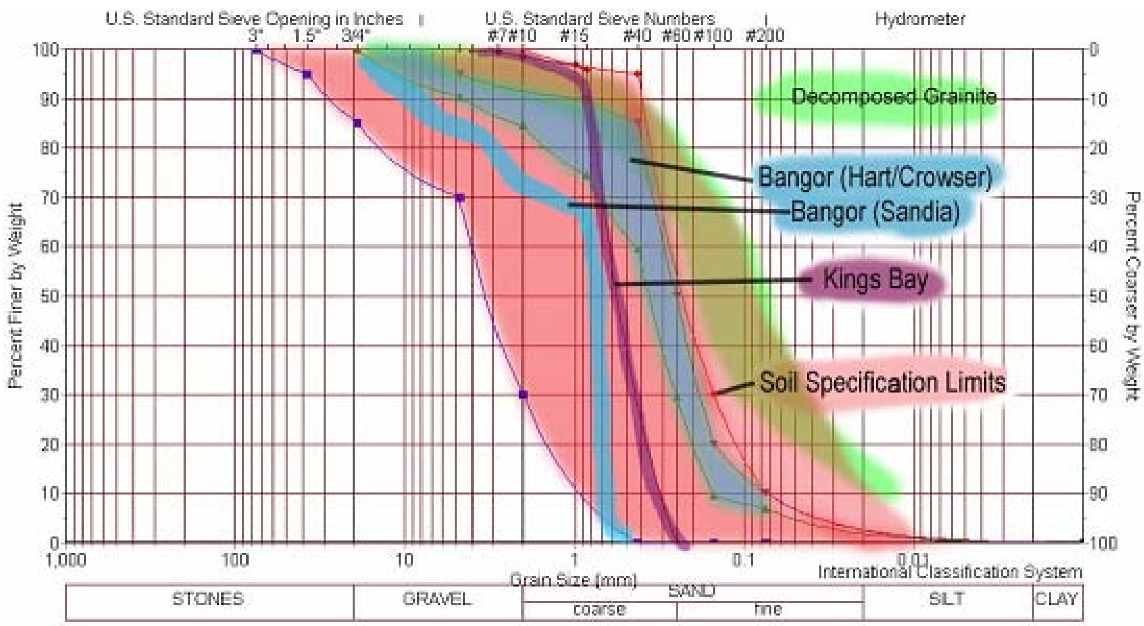


Figure 10. Graphical Representation of the Soil Specification.

6.0 Conclusions

Soil samples were received from the submarine bases at Kings Bay, Georgia, and Bangor, Washington, for characterization of mechanical properties. The Bangor as-received soil had a moisture content of 10.5%. Several sealed containers containing soil labeled "Lake" and "Dredge" from Kings Bay had been packaged to retain moisture. As received, the Kings Bay soil had a moisture content of 6.7% (weight).

The mechanical properties of two types of Kings Bay soil, Lake and Dredge, were determined at a confining pressure of 60 MPa. Two test series were completed on Dredge soil with starting dry densities of 105 (1.68) and 120 (1.92) lb/cu ft (g/cm^3) for a range of moisture contents ranging from nearly dry at 3% to nearly saturated at 10%. The as-received moisture content of both Lake and Dredge soils was 6.67%. Sieve analysis showed that both soils consisted almost entirely of grains with a size between 850 and 250 μm. The difference in appearance was because of the presence of organic particles and detritus in the Lake soil.

The Dredge material proved surprisingly strong, with no evidence of yield within the displacement limits of the testing equipment at a starting dry density of 105 (1.68) lb/cu ft (g/cm^3). Even at 9% water added by weight, the lower density specimens could support a stress difference in excess of 80 MPa at a confining pressure of 60 MPa. Significant sample shortening did occur before these stresses were reached. Failure to reach yield was an experimental limitation on the displacement of the hydraulic ram, not a fundamental limit.

When specimens were prepared with a higher initial density of 120 (1.92) lb/cu ft (g/cm^3), it was possible to observe yielding of the specimens, again at high, roughly constant shear stresses in excess of 120 MPa for moisture contents up to 8%. The maximum yield stress from 4% to 8% moisture content appeared to fall along the slope of the pressure-dependent yield stress curve. This could suggest that for these low-moisture contents, the AFV had not been compressed to the point where fluid pore pressure was supporting any of the load. At 9% added moisture, the shear strength fell sharply and was essentially zero at 10%.

There were some anomalies within the soil data; however, anomalies in soil data are the norm, not the exception. Based on the difference between the Kings Bay material and the reference material, it was concluded that the Kings Bay material should outperform the reference material.

The Bangor material was very similar in strength to the baseline DG material. At 9% moisture content, the two materials had the same maximum stress difference. At 7%, the material has about 30% more strength than the reference material. In general, the material properties for the Bangor material are very similar to the properties of the reference material. With the uncertainties associated with the variation in soil properties, and the unknown character of the equilibrium soil moisture content, one can conclude that material properties used in the reference calculation are representative of the Bangor material properties.

In general, the samples from the Bangor and Kings Bay sites appeared to be stronger than the reference soil. If the samples of the material are representative of the material found at the site, then they should be adequate for use in the planned construction. Since soil can vary greatly over even a small site, a soil specification for the construction contractor is needed to insure that soil variations found at the site will meet or exceed the requirements.

A suggested specification for the Bangor and Kings Bay soils was presented based on the information gathered from the indicated references plus the data obtained from this study, which could be used as a basis of design by the construction contractor.

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Appendix A. Results for Triaxial Testing of Kings Bay and Bangor Soils

A1 Overview

Soil samples were received from Kings Bay, Georgia, and Bangor, Washington, for characterization of mechanical properties. The Bangor as-received, soil had a moisture content of 10.5%. Several sealed containers of soil labeled “Lake” and “Dredge” from Kings Bay were received and had been packaged to retain moisture. As received, the soil had a moisture content of 6.7% (weight).

Material for testing was dried and then rehydrated to the desired moisture contents. Because of the unique nature of the intended use, appropriate choices for testing parameters were not obvious. In other work involving Decomposed Granite (DG), a dry density of 125 lb/cu ft had been adopted as the standard starting density. No problem was found in compacting the Bangor soil to that initial density. In contrast, we had difficulty compacting the Dredge material to that starting density. The Dredge material was too stiff to readily achieve 120 lb/cu ft, even using a hydraulic ram to compact the specimen. Therefore, we did a test series at a starting dry density of 105 lb/cu ft and moisture contents of 3, 5, 7, 8, and 9%. All tests were carried out under triaxial conditions: constant confining pressure P_C = 60 MPa and increasing axial stress σ_{11}. Tests were terminated when the sample yielded or the stroke limits of the machine were reached at a total sample shortening of about 20 mm.

Bangor soil responded to increasing moisture content in a manner very similar to DG; at 3% moisture content and 60 MPa confining pressure, high shear stresses were supported, but at higher moisture contents, the shear strength rapidly decreased.

At a starting dry density of 105 lb/cu ft, the Kings Bay Dredge soil proved surprisingly strong, with no evidence of yield within the limits of the testing equipment. Even at 9% water added by weight, the specimen could support a stress difference in excess of 80 MPa at a confining pressure of 60 MPa. Significant sample shortening did occur before these stresses were reached.

The high strength but large compaction indicated that it would be possible to preform specimens to a dry density of 120 lb/cu ft by using sufficient force and a stronger forming die. Whether this density would be achievable in the field is unknown. A second series was carried out with specimens containing 3, 4, 6, 8, 9, and 10% water by weight and the higher dry density. Results from this series confirmed that even at significant moisture contents, the Dredge soil could support high shear stresses. Only at 9% and 10% moisture contents was the yield strength significantly reduced, approaching zero at 10%.

Preliminary results indicate that the Dredge material is capable of supporting high shear stresses when confined and over a range of moisture contents. Only close to saturation was the shear strength reduced to near zero. The material does deform significantly with volume strains at a yield of about 19% and strains parallel to the maximum compressive stress of 20%, implying a low Poisson's ratio for much of the shear deformation. During the hydrostatic portion of the test, volume strains of about 10% were measured for the dryer, high-shear-strength specimens.

Testing of the Kings Bay Lake soil was less extensive because tests of Lake and Dredge soil at the as-received moisture content indicated that the mechanical response of the soils was identical, in spite of the very different appearance of the two soils.

A2 Experimental Procedure

Our test specimens were nominally 5.08 cm in diameter and 10.16 cm long. To avoid particle size effects, all samples were screened to remove either 4 or 2 mm and larger particles. Typically, there would be a small amount of shell fragments, pebbles, or organic debris in the removed fraction. The Bangor soil contained about 18% gravel, which was too large to include in our test specimens. We do not believe that results of testing would have been changed by the inclusion of these large particles in a suitably larger specimen.

A2.1 Sieve Analysis of Kings Bay Soils

A standard sieve analysis was performed to determine the range of particle sizes and to remove some of the large shell fragments and organic debris found in the samples.

A2.1.1 Dredge soil

Approximately 5 kg of Dredge soil was weighed and then dried overnight in an oven at 50 °C to determine the initial moisture content and to prepare the material for a sieve analysis. Initial moisture content was 6.7% by weight. Essentially, all the material passed through the 850-μm sieve and was retained on the 250-μm screen, indicating a narrow range of particle sizes. The fraction retained on larger screens consisted of shell fragments for the most part. Full results and photographs are in Appendix B.

A2.1.2 Lake soil

One kilogram of Lake soil was treated similarly, with full results reported in Appendix C. There was an absence of the larger shell fragments that comprised the large-size fractions in the Dredge soil. A smattering of organic particles and fibrous material was retained on sieves with passages larger than 850 μm, but essentially the entire sample (99.5%) passed through the 850-μm sieve and was retained on the 250-μm sieve. Full results and photographs are in Appendix C.

A2.2 Sieve Analysis of Bangor soil

Two kilograms of Bangor soil yielded about 18 % gravel, with many stones significantly larger than the 4-mm maximum mesh size used in preparing soil for use in test specimens. The balance of the material was heavily concentrated in the size fraction between 1 mm and 0.85 mm, which contributed 66 % of the soil by weight. Little material would pass through the 0.85-mm sieve. Full results and photographs are in Appendix D.

A2.3 Test Procedure

The objective was to determine the shear strength of the soil specimens at various moisture contents, under conditions of rapid loading and at high confining stresses. In particular, a confining pressure of 60 MPa was used for the high confining stress test series. Further testing was done at 1 MPa to determine the low confining stress strength of the specimens. Because the conditions of interest involve high loading rates, it is appropriate to use undrained specimens. Specimens were prepared from dried soil by adding the required mass of water and allowing equilibration time. A compaction die was used to precompact the soil to the desired dry starting densities of 105 (1.68) and 120 (1.92) lb/cu ft (g/cm^3). Beginning with a premeasured mass of dry soil to which the required amount of water had been added, the specimen was compressed to a fixed length of 4 in. (10.16 cm) at a fixed diameter of 2 in. (5.08 cm). This procedure resulted in uniform-sized specimens. Polyethylene shrink-fit tubing was used to jacket the sample to exclude the fluid used to apply confining pressure. All tests were conducted in a servo-controlled testing frame using a standard rock-mechanics testing setup, including a pressure vessel for applying the confining pressure and a loading ram for applying stress along the axis of the specimen.

A2.3.1 Specimen Preparation

1. Dry soil to remove existing moisture.
2. Weigh samples to provide requested density in a sample 5.08 cm diameter by 10.16 cm long (205.93 cc).
3. Add water to reach desired moisture content.
4. Mix soil and water thoroughly; this may require placing in a plastic bag for some time to equilibrate.
5. Prepare jacket and end caps shown in Figure A1. Jacket is 2-in. shrink tube formed over a 2-in. mandrel. Place jacket inside forming fixture. Place end cap without “O” ring and spacer in jacket before adding soil mixture.
6. Divide sample into six equal portions for placement in the jacket.

7. Add a portion and tamp ten strokes with a rod; repeat for all portions. Low-density samples may require light tamping to avoid overcompaction. This process should be completed quickly to reduce moisture loss.
8. Insert remaining end cap and spacer and compact in the small load frame to obtain consistent sample dimensions. Sample is proper length when platens contact both ends of the fixture.
9. Seal sample with "O" rings between end caps and jacket. Install clamp rings to complete seal.

Figure A1. Soil sample ready for testing.

A2.3.2 Strain measurements

Soil samples tend to deform inhomogeneously, making unreliable the usual techniques for measuring strain. Therefore, a dilatometric method was used. In essence, the technique consists of measuring the volume of fluid injected into the vessel as fluid pressure is applied and expelled and as the specimen is loaded along its axis by the hydraulically driven ram advancing into the vessel. For a rigid vessel and incompressible fluid, the measured volume of fluid injected, plus the volume of the ram that moves into the vessel, would equal the change in volume of the sample. Of course, in practice, a substantial correction must be made for the compressibility of the fluid. This is done by measuring the fluid volume needed to pressurize the vessel with a relatively rigid, dummy specimen in place of the compressible soil specimen. The difference between the calibration volume and the measured volume is equal to the specimen volume change during the hydrostatic portion of the test. We assumed that compression was isotropic during hydrostatic loading, allotting one-third of the volume strain to each of the three principal strains.

Individual strains can be determined during the application of shear stress using the displacement of the hydraulic ram combined with the change in fluid volume necessary to hold the confining pressure constant. To a good approximation, considering the low stiffness of the specimens, the displacement of the hydraulic ram, measured external to the pressure vessel, equals the axial shortening of the specimen, allowing ε_{11} to be determined. Volume strain $\varepsilon_{kk} = \varepsilon_{11} + \varepsilon_{22} + \varepsilon_{33}$ was then calculated using the dilatometric method, corrected for the slight change in volume from the loading piston moving into the vessel as the specimen shortened. Finally, assuming that the two principal strains perpendicular to the sample axis are equal, $\varepsilon_{33} = \varepsilon_{22}$, the third principal strain can be determined from the equation for volume strain.

A3 Results for Kings Bay Dredge and Lake Soils

Eleven tests were completed on the Dredge soil, six at an initial density of 1.68 and five at an initial density of 1.92 g/cm^3. All tests were done at a constant confining pressure of 60 MPa. The complete set of data plots may be found in Appendix E, arranged in the same order as the Table A1 entries.

Table A1. Test Conditions, Peak Stresses and Final Densities for Dredge Soil Tested at Starting Densities of 105 and 120 lb/cu ft and a Confining Pressure of 60 MPa.

Test ID	% H_2O	σ_{33}	σ_d (peak)	σ_{mean} (peak)	ρ_0 (dry)	ε_{kk}(final)	ρ_0 (dry,final)	ρ_0 (wet,final)
DRDG-4	3	60	86	88.7	105	0.23	136	140
DRDG-6	5	60	118	99.3	105	0.226	135	142
DRDG-1	7	60	77	85.7	105	0.23	136	145
DRDG-3	8	60	98	92.7	105	0.252	140	151
DRDG-2	9	60	81	87.0	105	0.225	135	147
							0	0
DRDG2B	3	60	53	77.7	120	0.19	148	152
DRDG6B	4	60	160	113.3	120	0.195	149	155
DRDG5B	6	60	132	104.0	120	0.19	148	157
DRDG4B	8	60	127	102.3	120	0.14	139	150
DRDG3B	9	60	12.5	64.2	120	0.05	126	137
DRDG-10	10	60	3.5	61.2	120	0.03	123	136

A3.1 Data Plot Discussion

Figures A2 and A3 are examples of the stress-strain data collected for each test. Similar plots for all the tests conducted may be found in Appendix E. Figure A2 shows the volume strain ε_{kk} measured dilatometrically while the confining pressure P_C increased to 60 MPa during the hydrostatic phase. Because of the various corrections and small offsets in the position of the intensifier ram, the volume strain at zero pressure typically does not equal exactly zero. The error was usually less than 0.01, and was small compared to the total volume strain. Each plot is labeled with the test ID and test starting conditions. Volume strains were observed to increase steadily up to the test pressure (60 MPa) with no obvious stiffening except for the highest water content samples tested at 120 lb/cu ft.

Figure A3 plots the stress difference $\sigma_D = \sigma_{11} - \sigma_{33}$ as a function of (ε_{11}, ε_{33}, ε_{kk}), which are the strains parallel to the maximum compressive stress, minimum compressive stress, and the volume strain, respectively. For ease of comparison, the strains during the shear loading phase have been zeroed at the beginning of shear loading. Appendix E contains additional plots for each test showing the total strains versus σ_{11}. All strains are an average over the sample length or volume, as post-test examination showed that deformation was typically not uniform. ε_{11} is a direct measurement of specimen shortening, made by using the displacement of the hydraulic ram. ε_{kk} is also a fairly direct measure of the total change in volume of the specimen. Because the pressure was not changing, there was no correction for fluid compression. The only correction required is for the volume of the loading piston that enters the pressure vessel as the axial load increased and that was easily determined from the displacement of the hydraulic ram. Lateral strain ε_{33} was measured indirectly using the equation $\varepsilon_{kk} = \varepsilon_{11} + \varepsilon_{22} + \varepsilon_{33}$ and assuming that the two lateral strains were equal. Yield strength was determined as the stress at which the stress-strain curves flattened. In Figure A3, clear evidence of yielding is seen in the flattening of the strain curves at a peak stress difference of 123 MPa. Continued loading would have produced additional strain at essentially constant load.

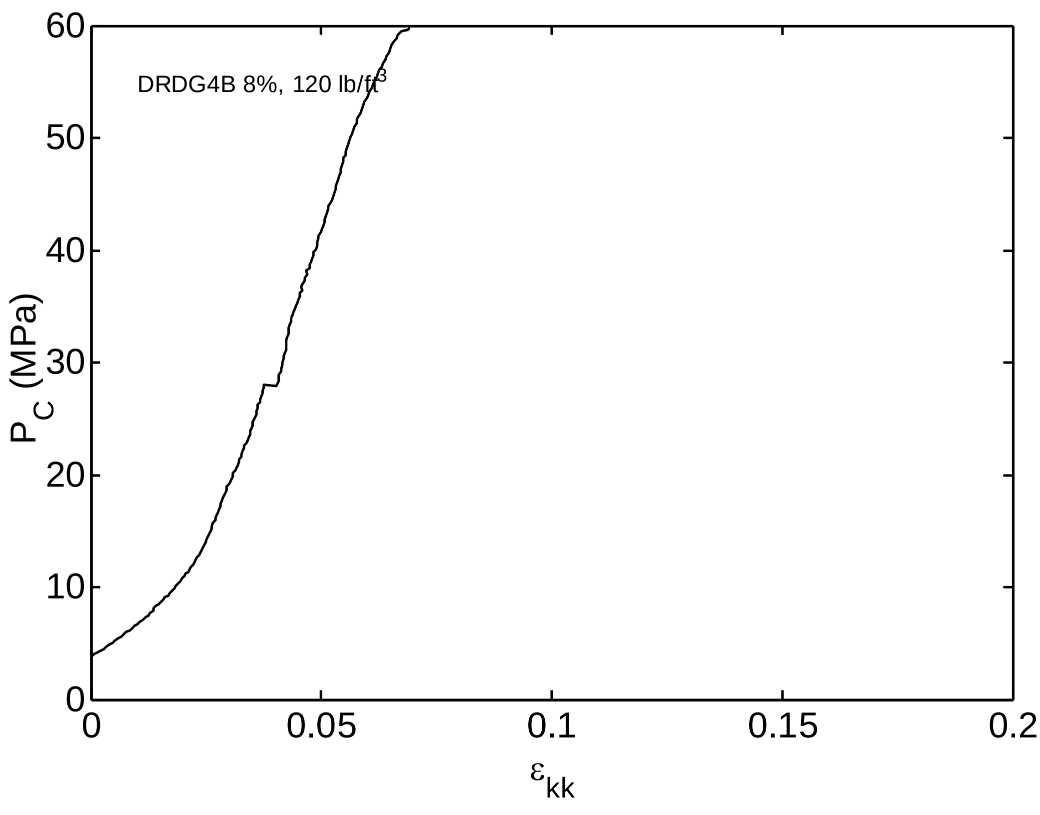

Figure A2. Hydrostatic loading of sample DRDG4B with a starting density of 120 lb/cu ft and added water corresponding to 8% by weight. Note the small offset from zero pressure at zero volume strain that is a result of various experimental errors and uncertainties.

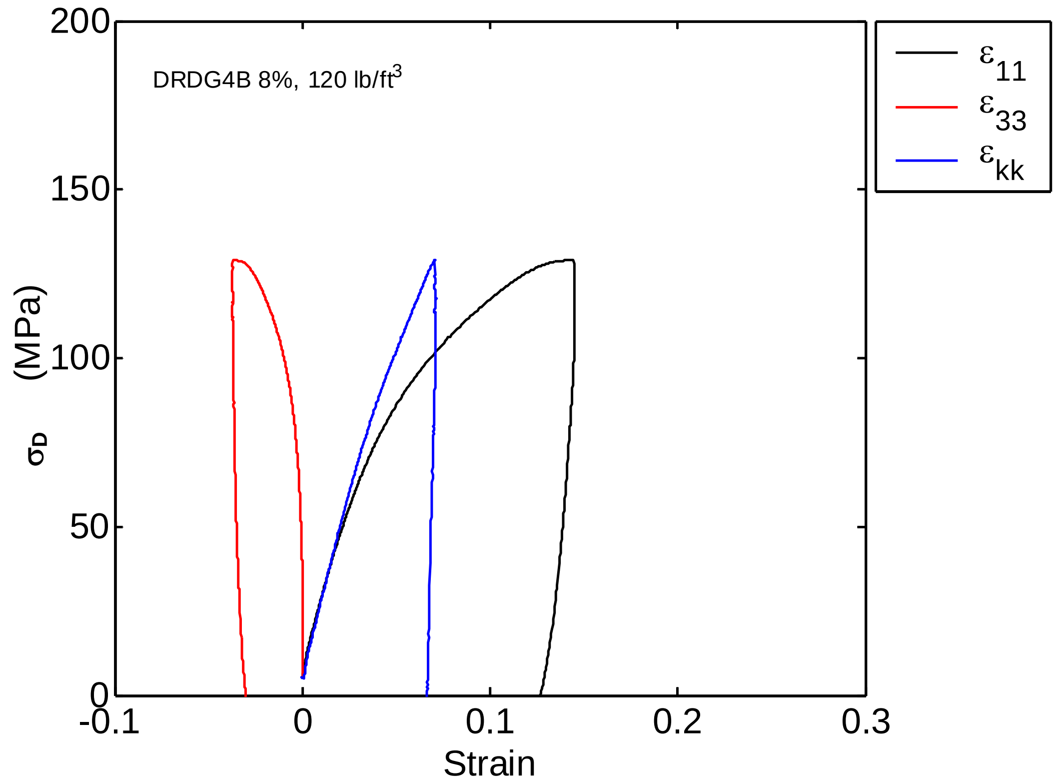

Figure A3. Shear loading of sample DRDG4B, with a starting density of 120 lb/cu ft and added water corresponding to 8% by weight. Strains have been offset to zero from the values at the end of the hydrostatic phase for ease of comparison.

A3.2 Soil Strength Discussion

Table A1 summarizes all the test results on Dredge soil. The first series of six tests was carried out using samples with an initial dry density of 105 lb/cu ft (1.68 g/cm^3) and the second series of five used specimens precompacted to 120 lb/cu ft (1.92 g/cm^3). As may be seen from the plots in Appendix D, we were unable to compress the lower density specimens to yield. This was a limit of the testing apparatus and was not fundamental. Thus, the values reported in Table A1 for peak stresses are lower limits. At the higher density, we were able to reach the yield stress as seen in Figure A3 and Appendix E.

Figure A4 plots the stress difference $\sigma_D = \sigma_{11} - \sigma_{33}$ as a function of mean stress $\sigma_m = (\sigma_{11} + 2\sigma_{33})/3$ for the two test series, using upward-pointing triangles for the results from lower density specimens to indicate that the values are a lower limit. The entire test series for the high-density specimens is shown by brown diamonds, with the moisture content shown next to the data point. An easier way of seeing the effect of moisture on yield strength is to plot σ_D as a function of moisture content, as in Figure A5.

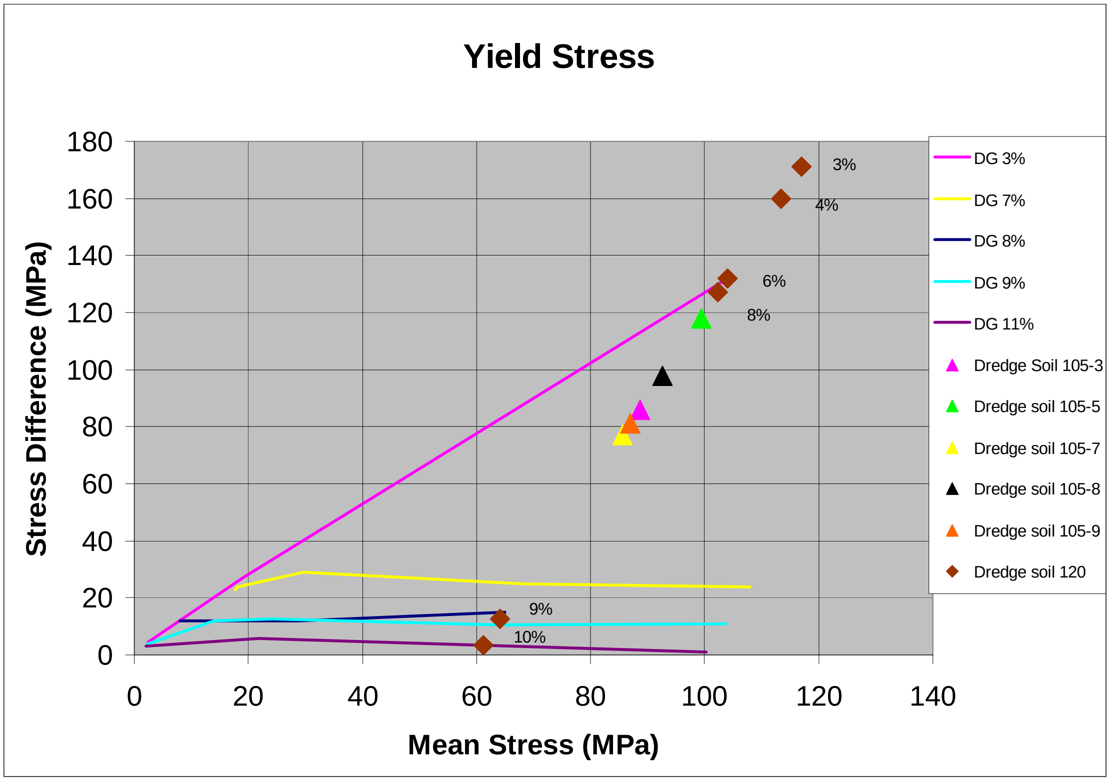

Figure A4. Stress difference as a function of mean stress for the Dredge soil, low-density specimens (colored, up-pointing triangles), high-density specimens (brown diamonds, with moisture content indicated) and, for comparison, the solid lines (marked DG 3%, etc.) show results for a cohesive soil from an arid environment (DG). Each colored line corresponds to a fixed moisture content.

Up to 8% moisture, the Dredge soil is almost unaffected by moisture content and can support high stress differences. Between 8% and 9% moisture content, the shear strength of the Dredge soil declines abruptly, becoming essentially zero at 10%.

For comparison, the results of a test series on a cohesive but sandy soil (Decomposed Granite, DG) are also plotted in Figure A4, using colored lines corresponding to various moisture contents, as indicated in the legend. Shear strength declines more gradually with moisture content than was observed for the noncohesive Dredge soil.

A test series conducted using Lake soil, which partially overlapped the Dredge soil test series, confirmed that the two soils were similar if not identical. Figure A5 contains the results from the two test series.

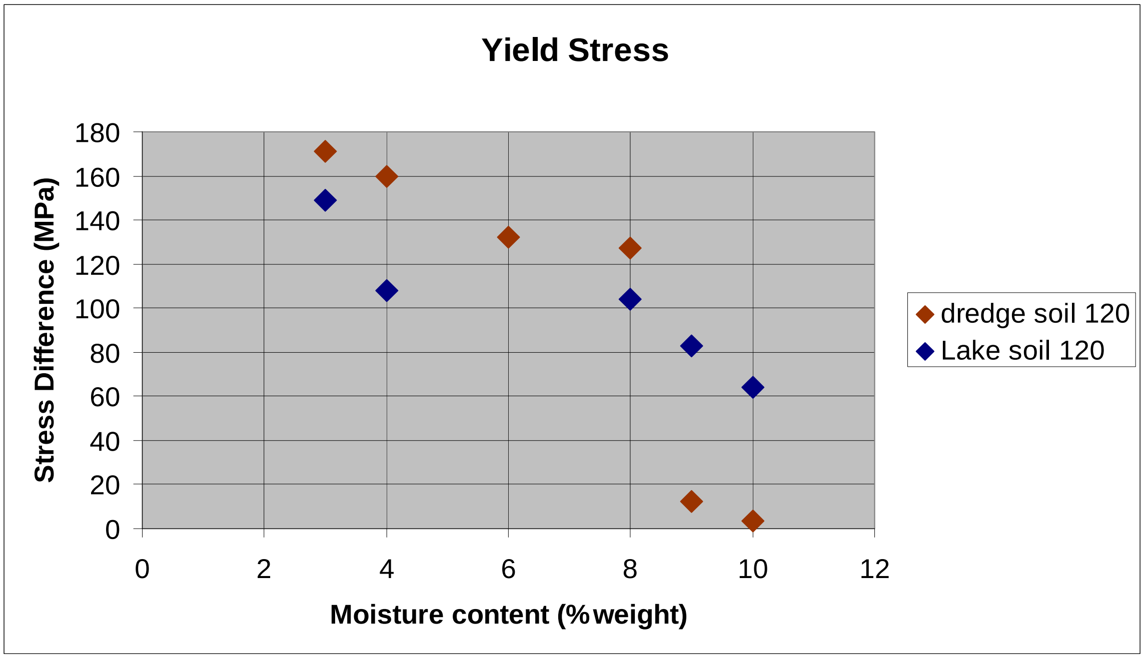

Figure A5. Stress difference at yield for the Lake and Dredge soils, tested at 60 MPa confining pressure and a starting dry density of 120 lb/cu ft (1.92 g/cm^3). Data are the same as in Figure A4.

A3.3 Comparison of Lake and Dredge Soil

Visually, the Lake and Dredge soils appear very different; Lake soil is dark and organic in appearance while Dredge soil is light, resembling very fine beach sand. However, the similarity of the sieve analyses results and the identical as-received moisture contents hint that the difference may be superficial. A single test was done to compare the two soils when tested as received at 6.67% moisture content and 105 lb/cu ft (1.68 gm/cm^3). Results are shown in Figure A6, which plots the measured strains for both specimens versus σ_{11}, using solid lines for the Dredge soil and dotted lines for the Lake soil. It is clear that the mechanical response is essentially identical. It seems that the visual difference is the result of an insignificant admixture of organic detritus, as seen in the sieve analysis, which produces no effect on the mechanical properties.

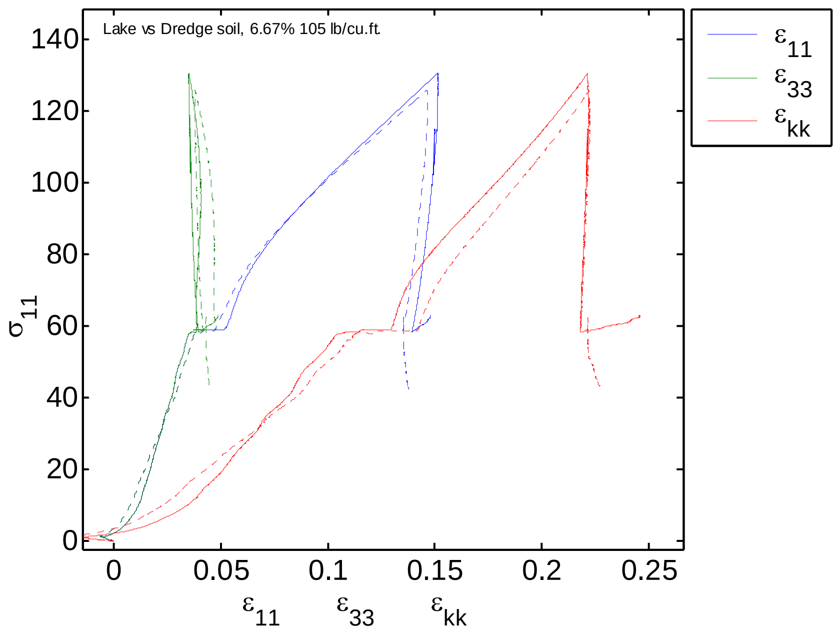

Figure A6. Comparison of stress as a function of strains for Dredge (solid lines) and Lake (dotted lines) when tested at the as-received moisture content of 6.67% and 105 lb/cu ft (1.68 g/cm^3). Their mechanical response is virtually identical.

A3.4 Summary

The mechanical properties of two types of soil, Dredge and Lake, were determined at a confining pressure of 60 MPa. Two test series were completed on Dredge soil with starting dry densities of 105 (1.68) and 120 (1.92) lb/cu ft (g/cm^3) for a range of moisture contents ranging from nearly dry at 3% to nearly saturated at 10%. The as-received moisture content of both soils was 6.67%. Sieve analysis showed that both soils consisted almost entirely of grains with a size between 850 and 250 μm. The difference in appearance is due to the presence of organic particles and detritus in the Lake soil.

The Dredge material proved surprisingly strong, with no evidence of yield within the displacement limits of the testing equipment at a starting dry density of 105 (1.68) lb/cu ft (g/cm^3). Even at 9% water added by weight, the lower density specimens could support a stress difference in excess of 80 MPa at a confining pressure of 60 MPa. Significant sample shortening did occur before these stresses were reached. Failure to reach yield was an experimental limitation on the displacement of the hydraulic ram, not a fundamental limit.

When prepared with a higher density 120 (1.92) lb/cu ft (g/cm^3), it was possible to observe yielding of the specimens, again at high, roughly constant shear stresses in excess of 120 MPa for moisture contents up to 8%. At 9 % added moisture, the shear strength fell sharply and was essentially zero at 10%.

Two tests were done to compare the Lake and Dredge soils at the as-received moisture content. Mechanically, the two soils appeared to be identical, contradicting their very different appearance.

The small range of particle sizes causes the porosity to be high, accounting for the difficulty in precompacting the Dredge soil. High initial porosity, combined with the high strength indicates that if moisture contents can be controlled, the Dredge soil could support large loads, albeit with significant compaction along the direction of maximum compressive stress.

A4 Results for Bangor Soil

Ten tests were completed on the Bangor soil, all at a density of 2.0 g/cm^3. Five tests were done at a confining pressure of 1 MPa and 5 at 60 MPa. All tests were conducted using the procedure and sample preparation technique described earlier, and are shown in Table A2.

Table A2. Test Conditions, Peak Stresses, and Final Densities for Bangor Soil Tested at Starting Density of 125 lb/cu ft and a Confining Pressures of 1 and 60 MPa.

Test ID	% H_2O	σ_{11}	σ_{33}	σ_d (peak)	σ_{mean} (peak)	ρ_0 (dry)
SS1-3	3	175	60	115	98.3	125
SS10-3	3	6.3	1	5.3	2.8	125
SS12-5	5	5	1	4	2.3	125
SS2-7	7	94	60	34	71.3	125
SS7-7	7	5.9	1.2	4.7	2.8	125
SS4-8	8	82	60	22	67.3	125
SS9-8	8	5	1	4	2.3	125
				0	0.0	
SS3-9	9	73	60	13	64.3	125
				0	0.0	
SS6-10	10	70	60	10	63.3	125
SS11-10	10	5	1	4	2.3	125

Table A2 summarizes all the test results on Bangor soil, all of which were carried out using samples with an initial dry density of 120 lb/cu ft (2.0 g/cm^3). Two test series were completed. One series of five tests was done at 1 MPa confining pressure and a second series at 60 MPa. Moisture contents ranged from 3% to 10 %. Figure A7 plots the stress difference $\sigma_D = \sigma_{11} - \sigma_{33}$ as a function of mean stress $\sigma_m = (\sigma_{11} + 2\sigma_{33})/3$ for the two test series.

For comparison, the results of a test series on a cohesive but sandy soil (DG) are also plotted in Figure A7, using colored lines corresponding to various moisture contents as indicated in the legend. Shear strength for both the Bangor and DG soils declined more rapidly with moisture content than was observed for the noncohesive Kings Bay soils discussed earlier. Another way of seeing the effect of moisture on yield strength is to plot σ_D as a function of moisture content (Figure A8). Strength decreased steadily as moisture content was increased, unlike the Kings Bay soil, which was almost unaffected by moisture content up to 8%.

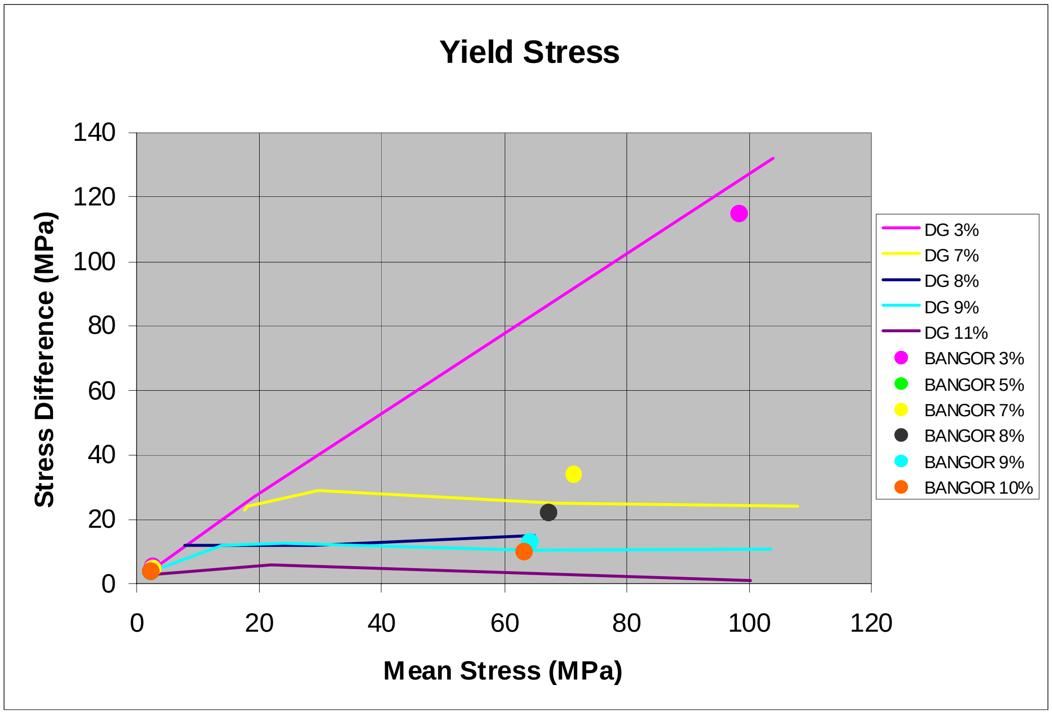

Figure A7. Stress difference as a function of mean stress for the Bangor soil, for an initial dry density of 2.0. For comparison the solid lines (marked DG 3%, etc.) show results for a cohesive soil from an arid environment (DG). Each colored line corresponds to a fixed moisture.

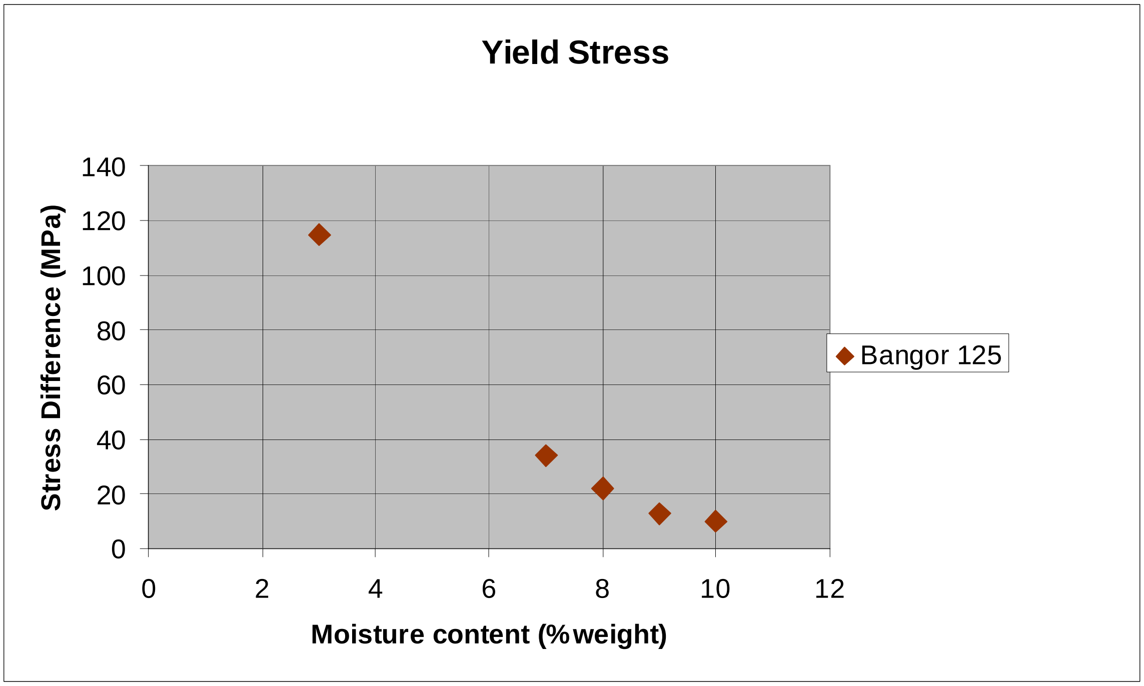

Figure A8. Stress difference at yield for the Bangor soils, tested at 60 MPa confining pressure and a starting dry density of 125 lb/cu ft (2.0 g/cm^3). Data are the same as in Figure A7 for the 60 MPa confining pressure series.

Appendix B. Sieve Analysis of Dredge Soil

STARTING MATERIAL

5.7568 kg of dried Dredge soil

SIEVE PROCEDURE

Sieve sizes – 4.0 mm, 2.8 mm, 2.0 mm, 1.0 mm, 850 μm
Material from each sieve was weighed and kept separated

SIEVE RESULTS

Sieve Size	Weight retained (g)	% retained
4.0 mm	11.70	0.2
2.8 mm	17.84	0.3
2.0 mm	50.30	0.9
1.0 mm	105.63	1.8
850.0 μm	50.62	0.9
250.0 μm	5520.00	95.9

Figure B1. Kings Bay Dredge Soil Sieve Test. 4.0 mm – 11.7 g.

Figure B2. Kings Bay Dredge Soil Sieve Test. 2.8 mm – 17.84 g.

Figure B3. Kings Bay Dredge Soil Sieve Test. 2.0 mm – 50.3 g.

Figure B4. Kings Bay Dredge Soil Sieve Test. 1.0 mm – 105.63 g.

Figure B5. Kings Bay Dredge Soil Sieve Test. 850 μm – 50.62 g.

Appendix C. Sieve Analysis of Lake Soil

STARTING MATERIAL

1.0 kg of dried Lake soil

SIEVE PROCEDURE

Sieve sizes – 4.0 mm, 2.8 mm, 2.0 mm, 1.0 mm, 850 μm, 250 μm
Material from each sieve was weighed

SIEVE RESULTS

Sieve Size	Weight (g)
4.0 mm	0.00
2.8 mm	0.00
2.0 mm	0.67
1.0 mm	2.84
850.0 μm	2.38
250.0 μm	994.00

Figure C1. Kings Bay Lake Soil Sieve Test. 2mm – 0.67 g.

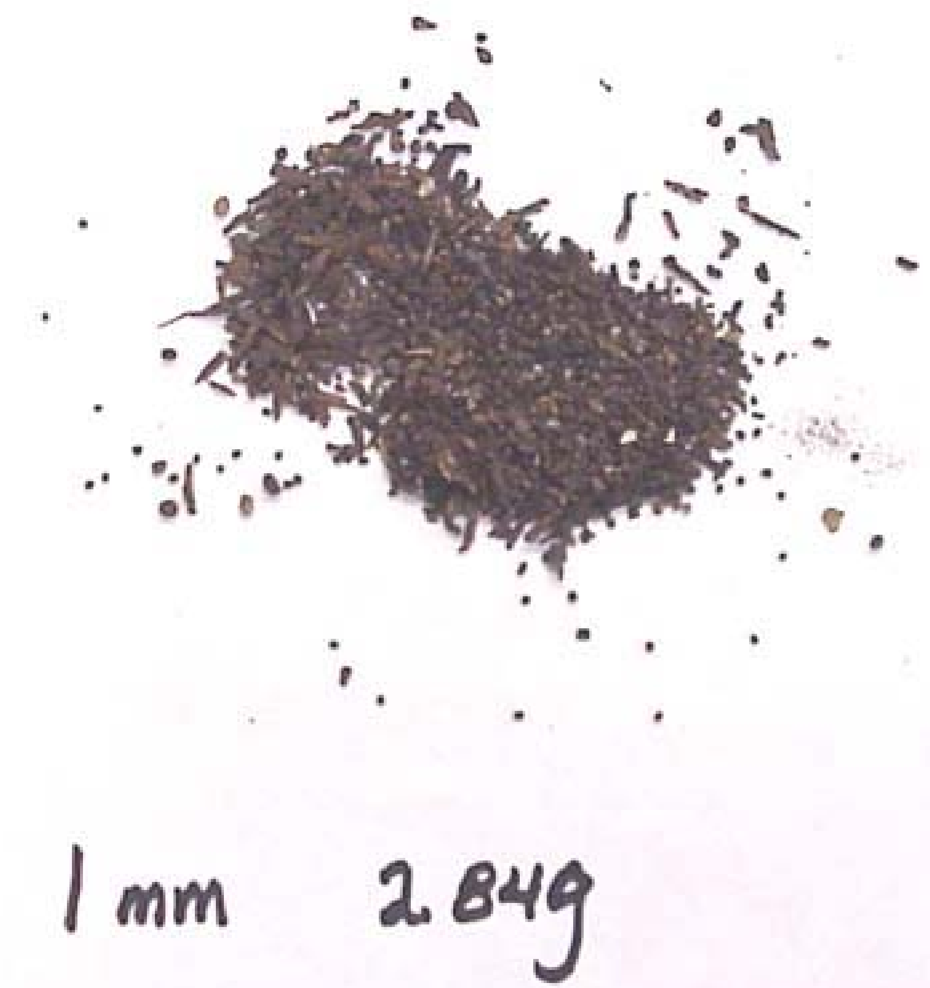

Figure C2. Kings Bay Lake Soil Sieve Test. 1 mm – 284 g.

Figure C3. Kings Bay Lake Soil Sieve Test. 850 μm – 2.38 g.

Appendix D. Bangor Soil Sieve Test

STARTING MATERIAL

2 kg of dried Washington sandy soil as received

SIEVE PROCEDURE:

Sieve sizes of 4 mm, 2.8 mm, 2 mm, and 1 mm; a sieve size of 850 μm was also used, but the soil would not pass through after about one hour of shaking.

SIEVE RESULTS:

Sieve Size	Weight retained (g)	% retained
4.0 mm	376.4	19
2.8 mm	140	7
2.0 mm	79.1	4
1.0 mm	86.52	4
850.0 μm	1317	66
250.0 μm	trace	0

Figure D1. Bangor Soil Sieve Test. 4.0-mm Sieve – 376.4 g.

Figure D2. Bangor Soil Sieve Test. 2.8-mm Sieve – 140.32 g.

Figure D3. Bangor Soil Sieve Test. 2.0-mm Sieve – 79.07 g.

Figure D4. Bangor Soil Sieve Test. 1.0-mm Sieve – 86.52 g.

Appendix E. Plots of Test Results

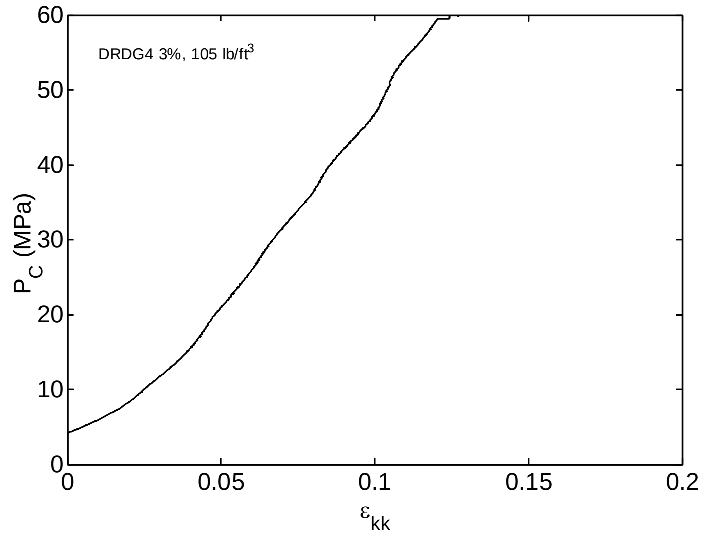

Figure E1. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG4 tested at a starting density of 105 lb/cu ft.

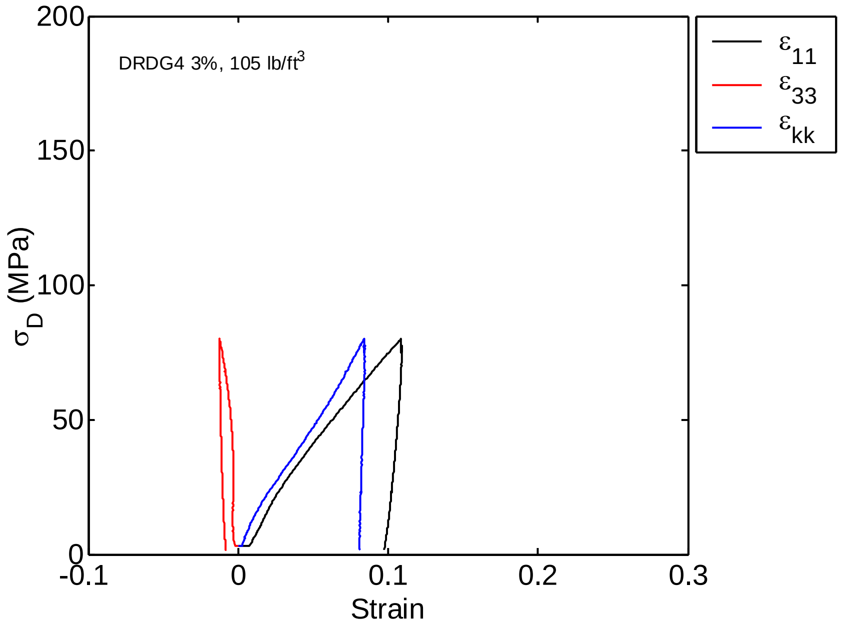

Figure E2. Shear loading of sample DRDG4, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 3% by weight.

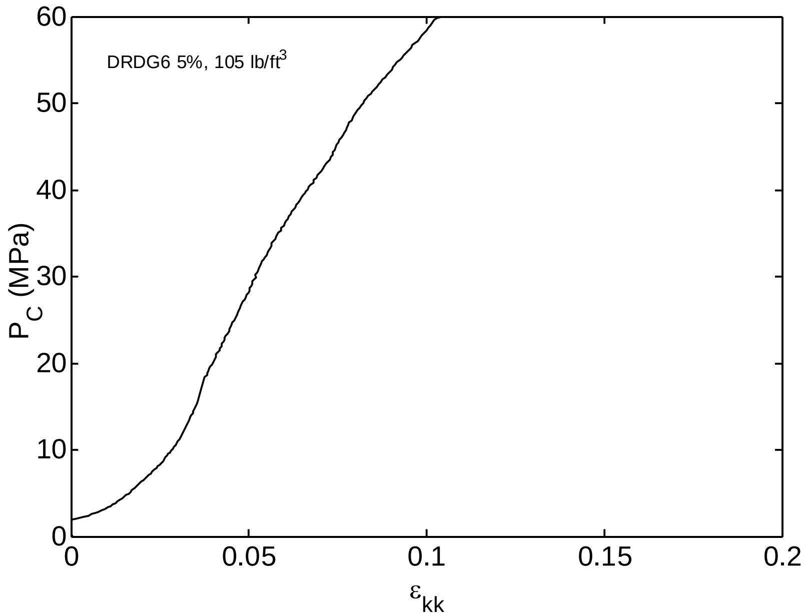

Figure E3. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG6 tested at a starting density of 105 lb/cu ft and added water corresponding to 5% by weight.

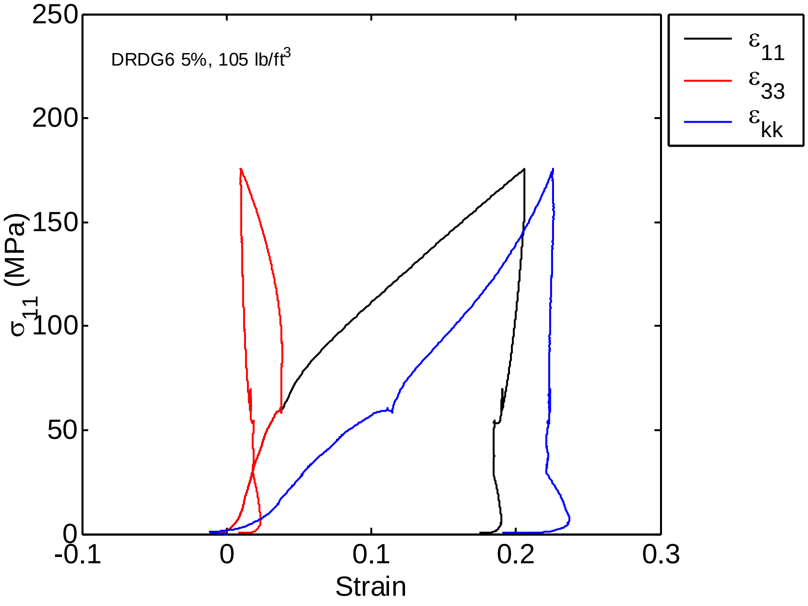

Figure E4. Stress as a function of strain for sample DRDG6, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 5% by weight.

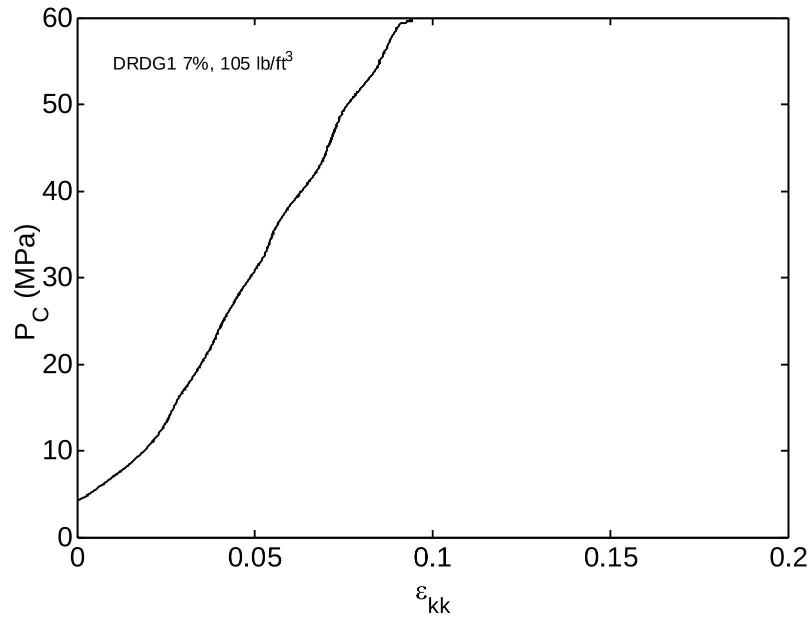

Figure E5. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG1 tested at a starting density of 105 lb/cu ft and added water corresponding to 7% by weight.

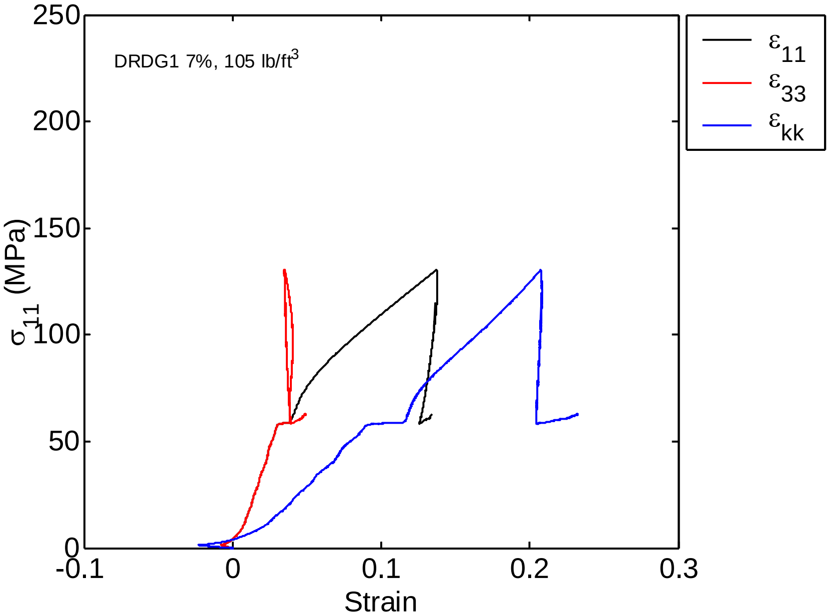

Figure E6. Stress as a function of strain for sample DRDG1, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 7% by weight.

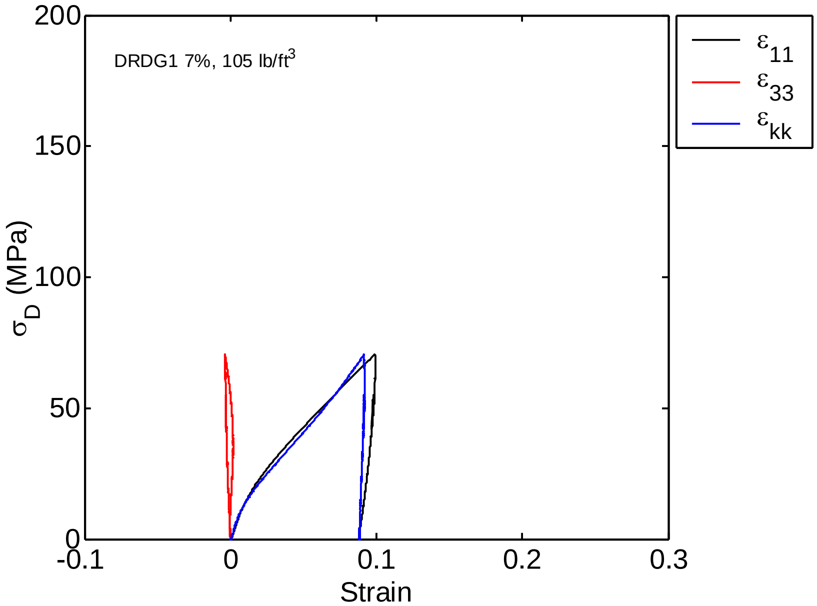

Figure E7. Shear loading of sample DRDG1, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 7% by weight.

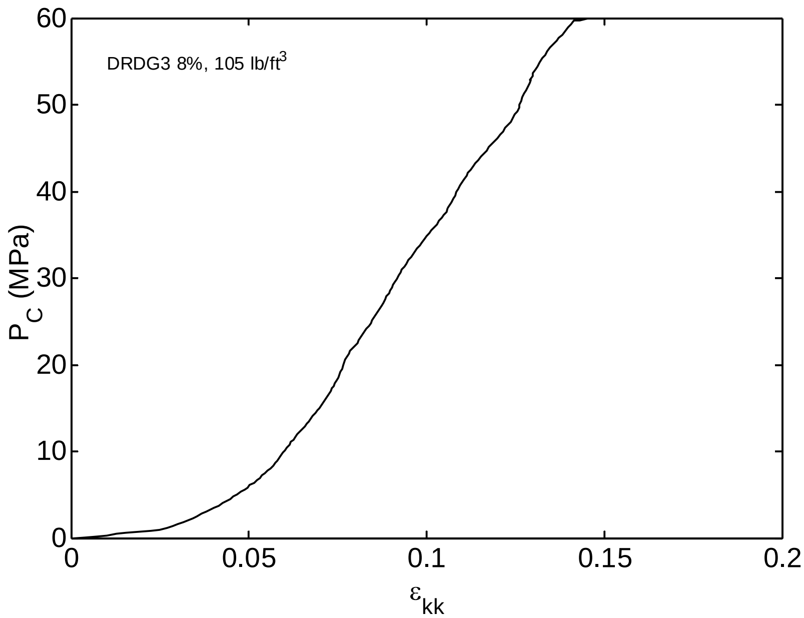

Figure E8. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG3 tested at a starting density of 105 lb/cu ft and added water corresponding to 8% by weight.

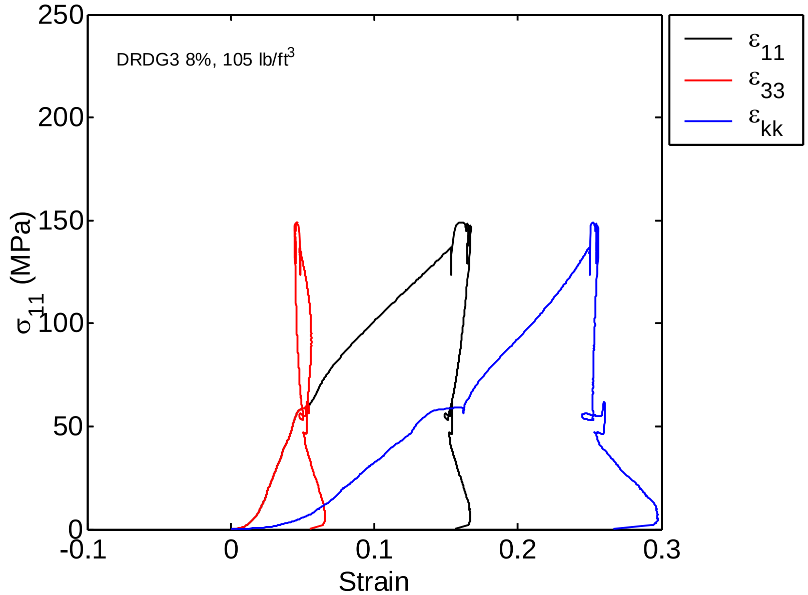

Figure E9. Stress as a function of strain for sample DRDG3, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 8% by weight.

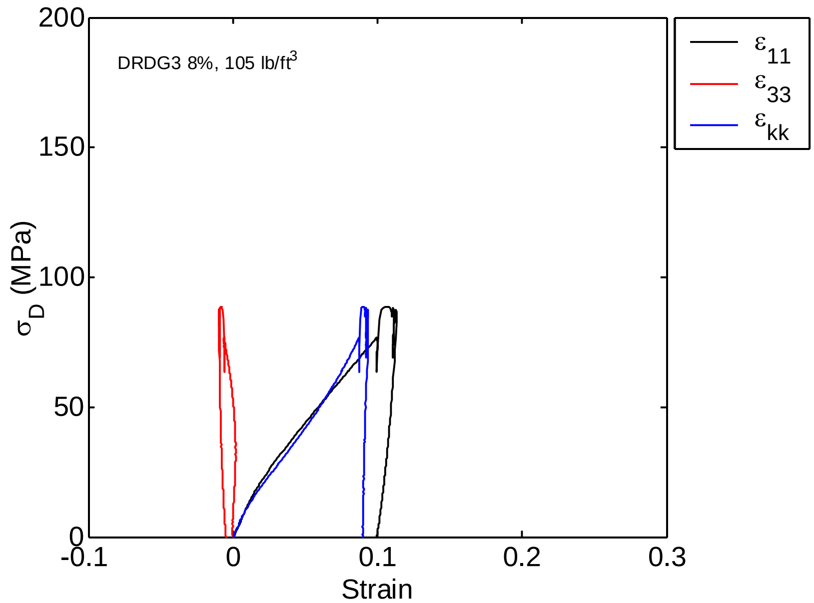

Figure E10. Shear loading of sample DRDG3, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 8% by weight.

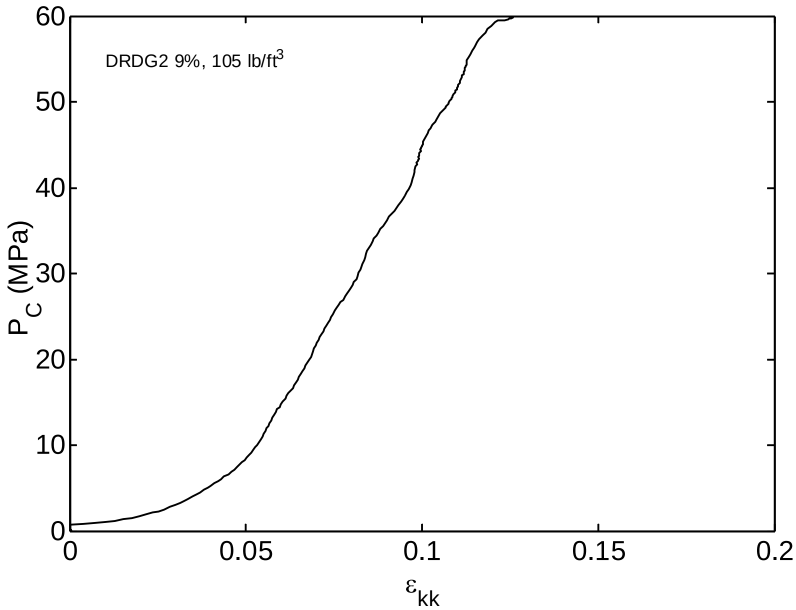

Figure E11. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG2 tested at a starting density of 105 lb/cu ft and added water corresponding to 9% by weight.

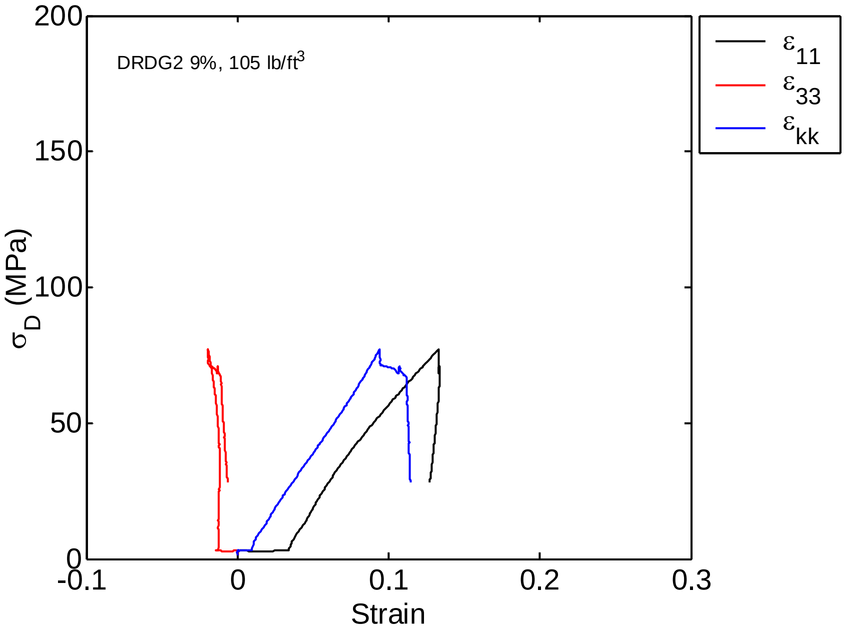

Figure E12. Shear loading of sample DRDG2, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 9% by weight.

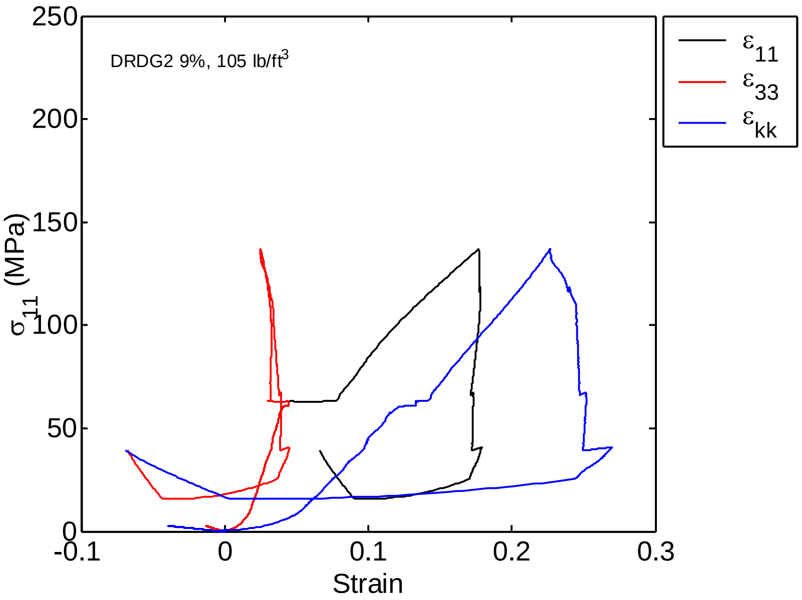

Figure E13. Stress as a function of strain for sample DRDG2, with a starting density of 105 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 9% by weight.

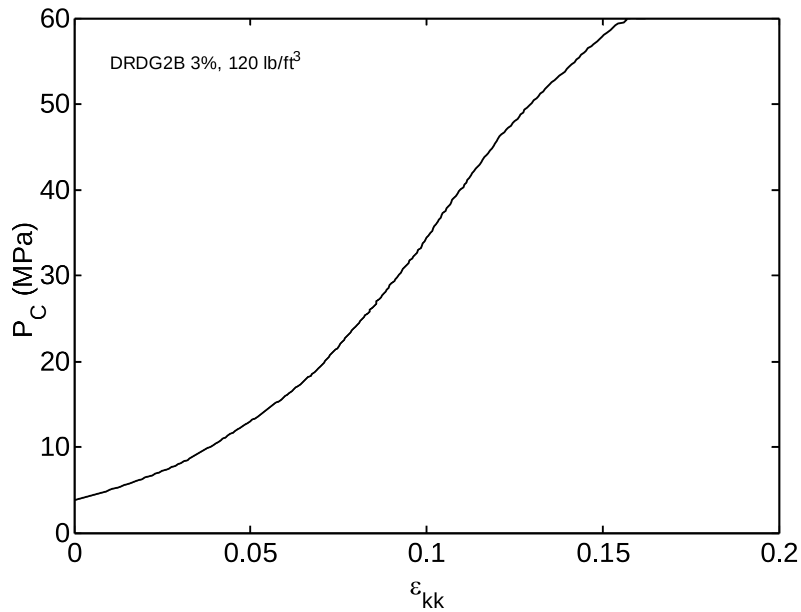

Figure E14. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG2B tested at a starting density of 120 lb/cu ft and added water corresponding to 3% by weight.

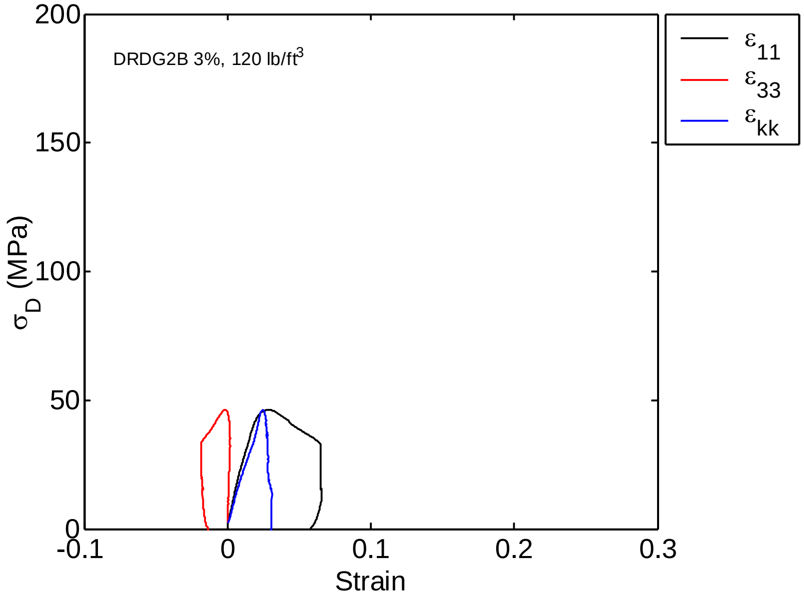

Figure E15. Shear loading of sample DRDG2B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 3% by weight.

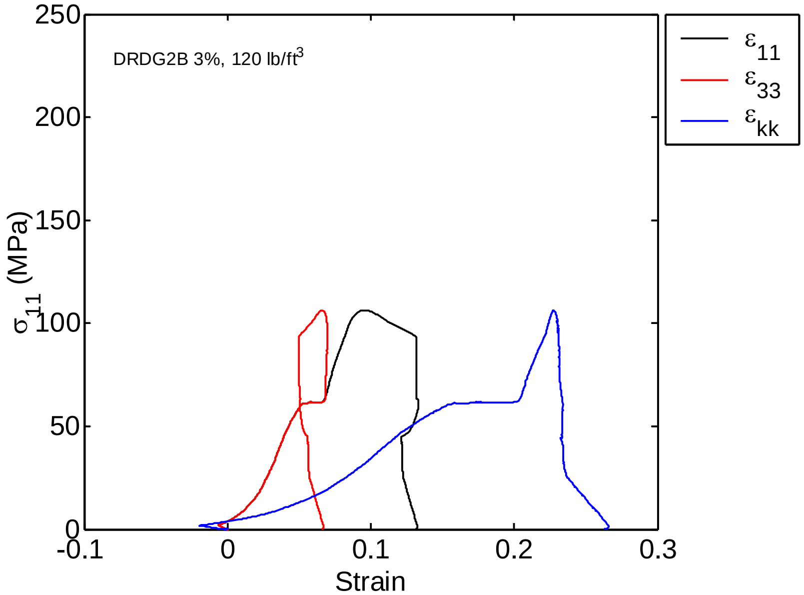

Figure E16. Stress as a function of strain for sample DRDG2B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 3% by weight.

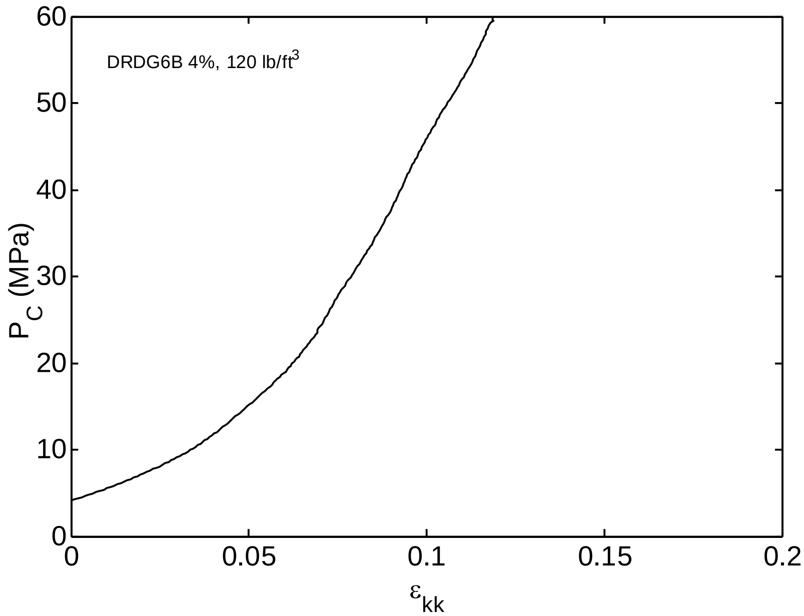

Figure E17. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG6B tested at a starting density of 120 lb/cu ft and added water corresponding to 4% by weight.

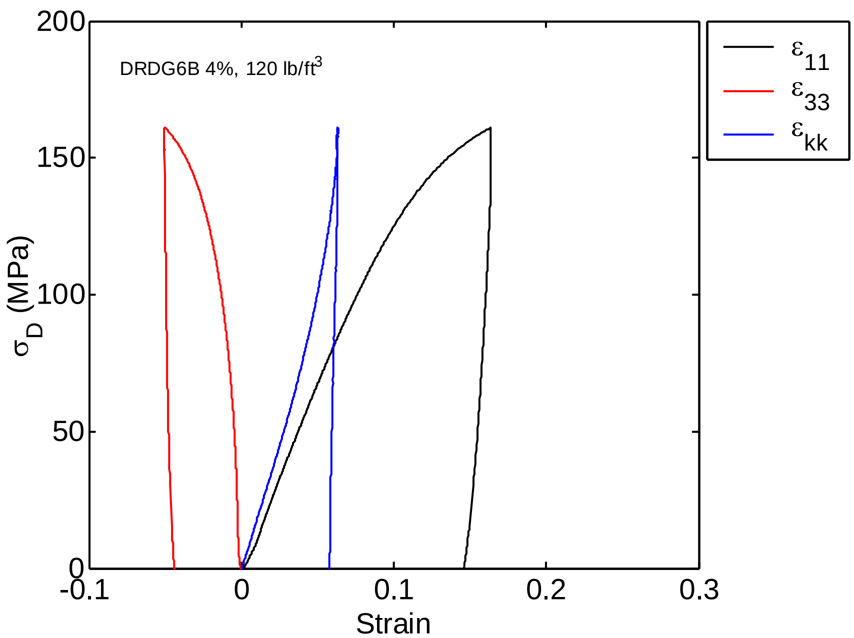

Figure E18. Shear loading of sample DRDG6B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 4% by weight.

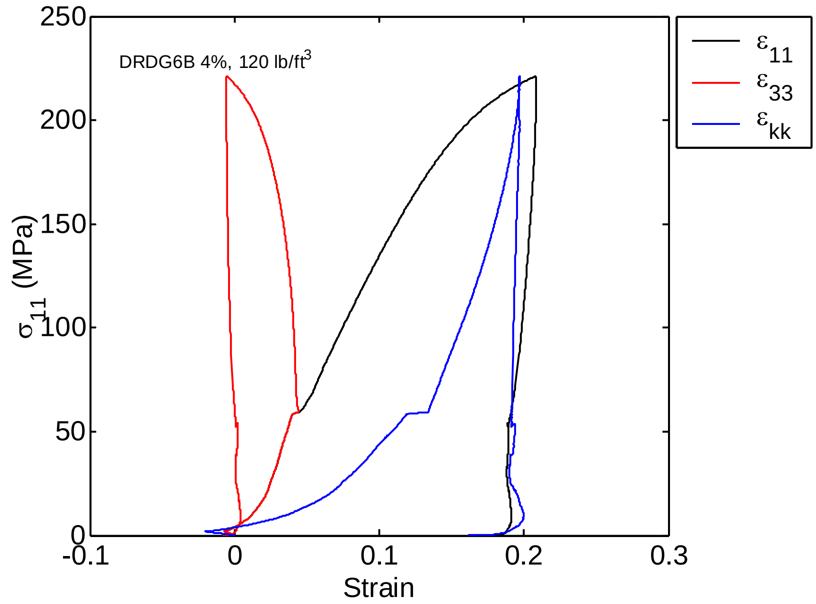

Figure E19. Stress as a function of strain for sample DRDG6B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 4% by weight.

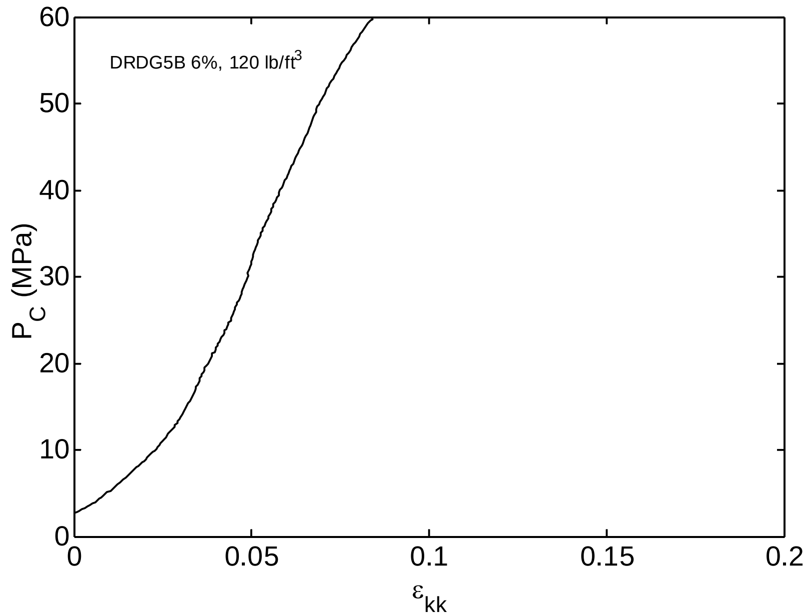

Figure E20. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG5B tested at a starting density of 120 lb/cu ft and added water corresponding to 6% by weight.

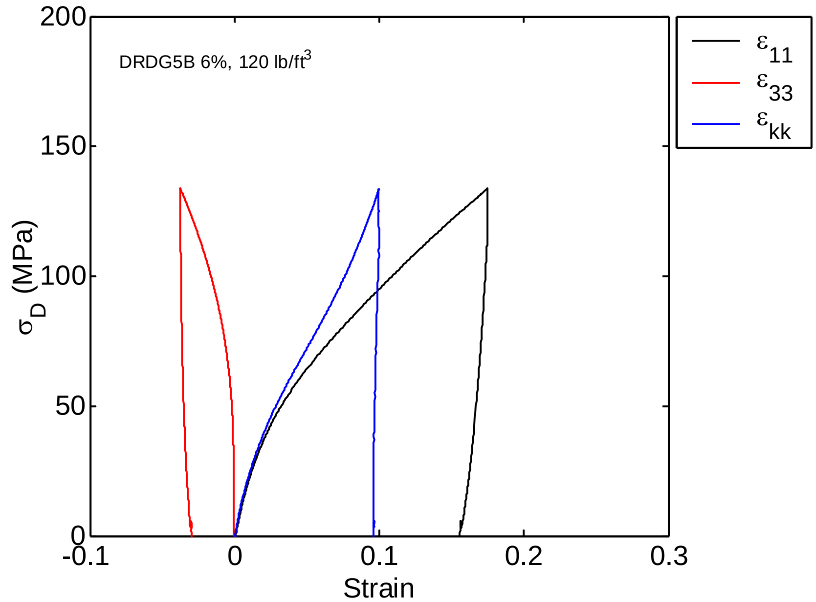

Figure E21. Shear loading of sample DRDG5B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 6% by weight.

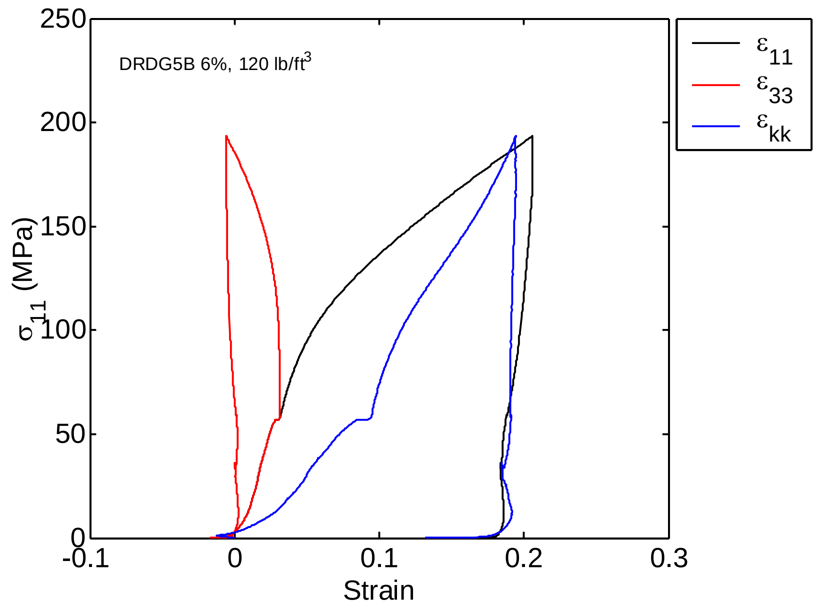

Figure E22. Stress as a function of strain for sample DRDG5B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 6% by weight.

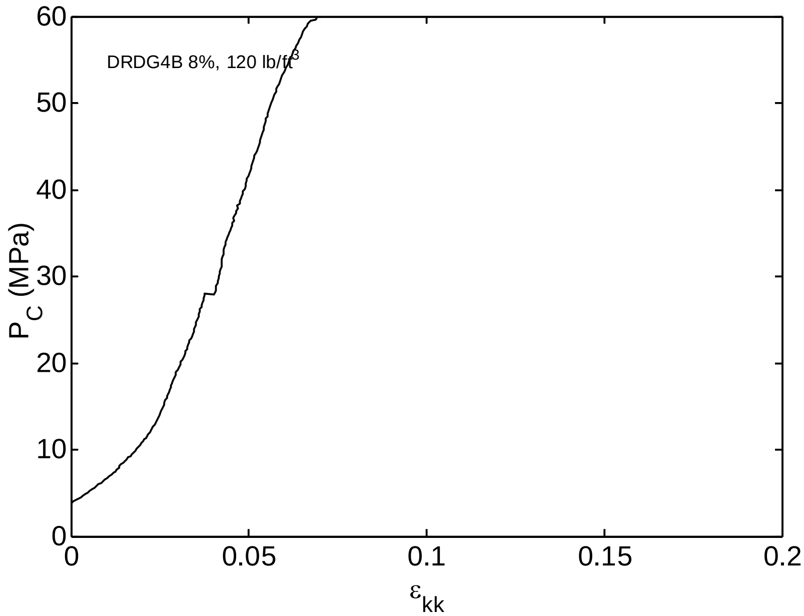

Figure E23. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG4B tested at a starting density of 120 lb/cu ft and added water corresponding to 8% by weight.

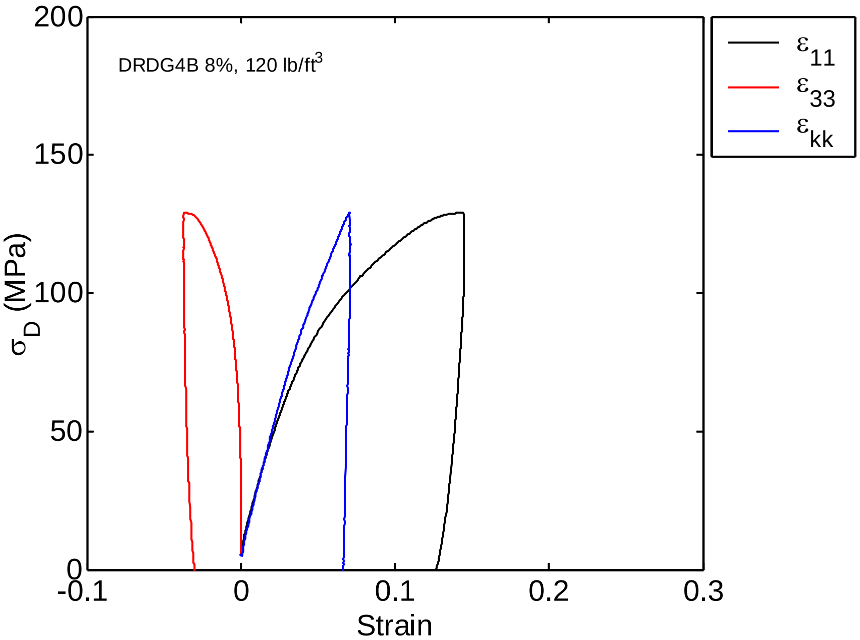

Figure E24. Shear loading of sample DRDG4B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 8% by weight.

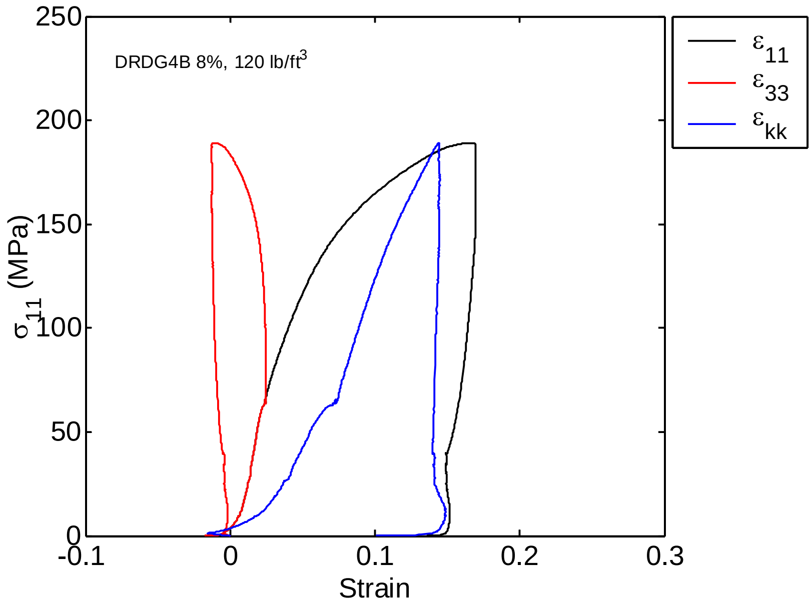

Figure E25. Stress as a function of strain for sample DRDG4B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 8% by weight.

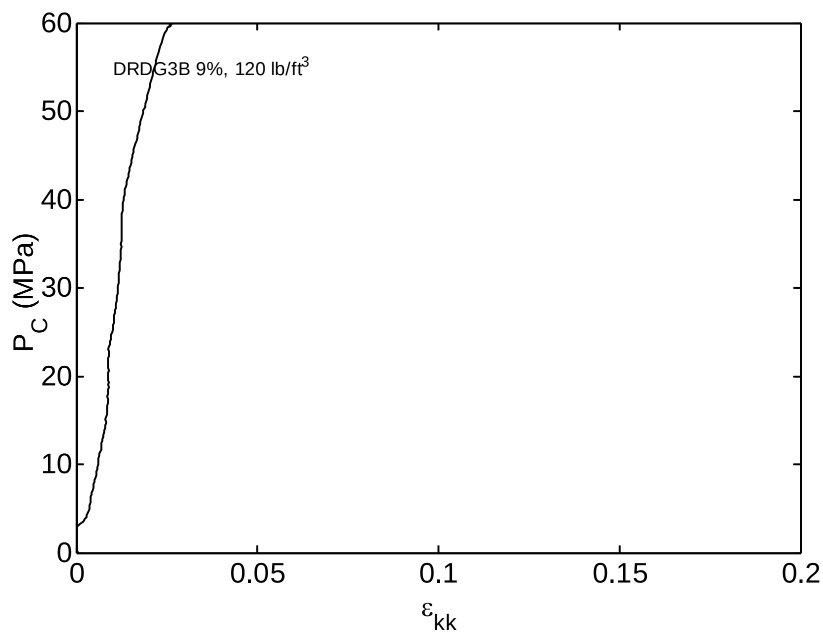

Figure E26. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG3B tested at a starting density of 120 lb/cu ft and added water corresponding to 9% by weight.

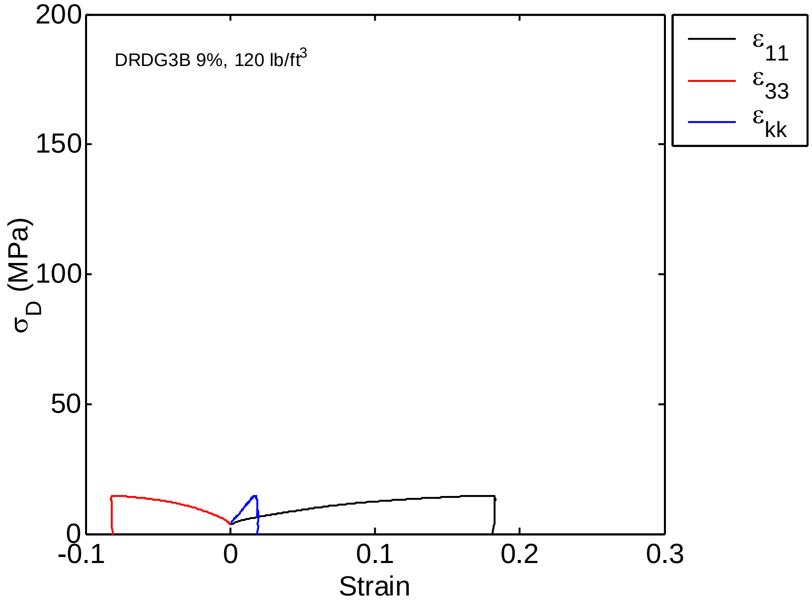

Figure E27. Shear loading of sample DRDG3B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 9% by weight.

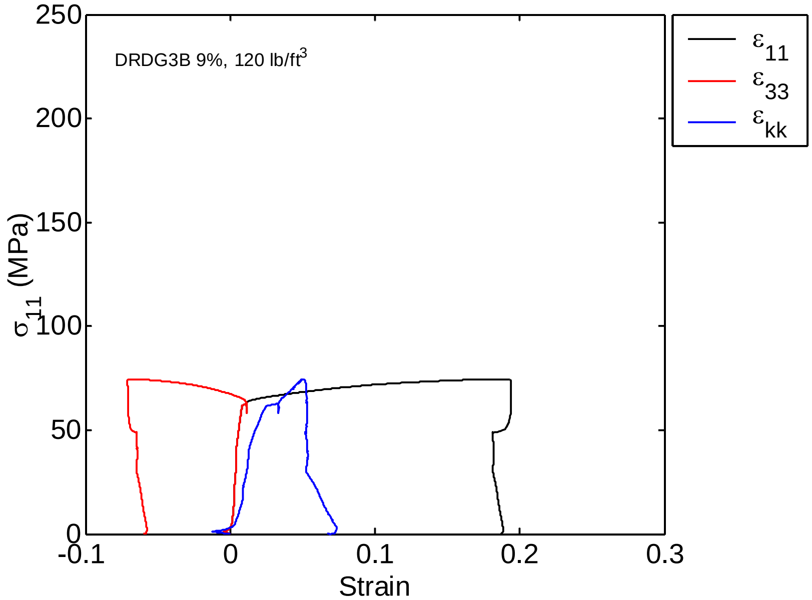

Figure E28. Stress as a function of strain for sample DRDG3B, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 9% by weight.

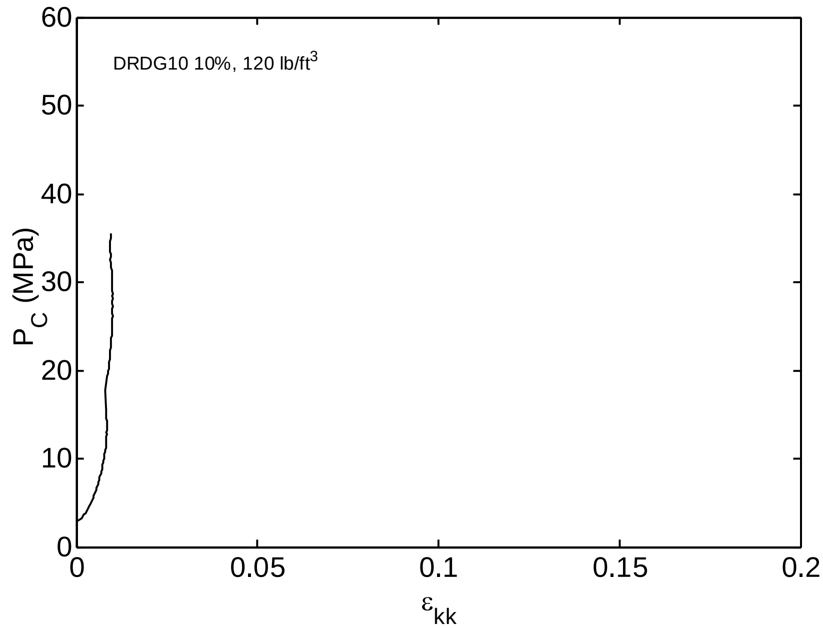

Figure E29. P_c(Confining Pressure) - ε_{kk} (Volume Strain) plot for sample DRDG10 tested at a starting density of 120 lb/cu ft and added water corresponding to 10% by weight.

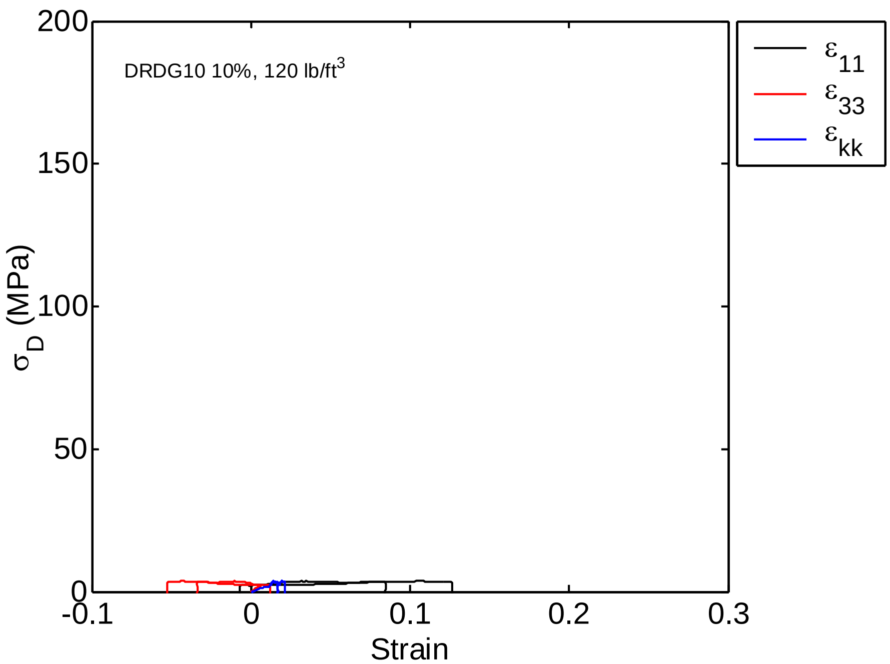

Figure E30. Shear loading of sample DRDG10, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 10% by weight.

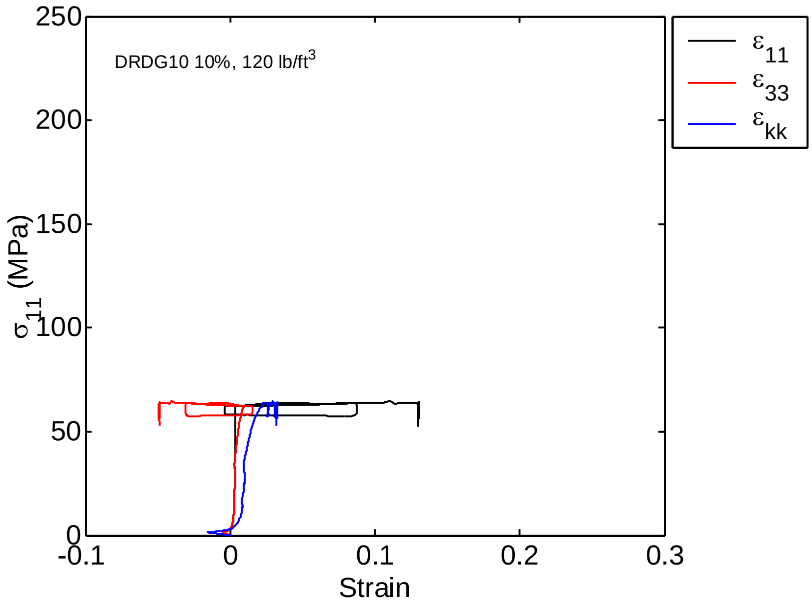

Figure E31. Stress as a function of strain for sample DRDG10, with a starting density of 120 lb/cu ft, a confining pressure of 60 MPa, and added water corresponding to 10% by weight.

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