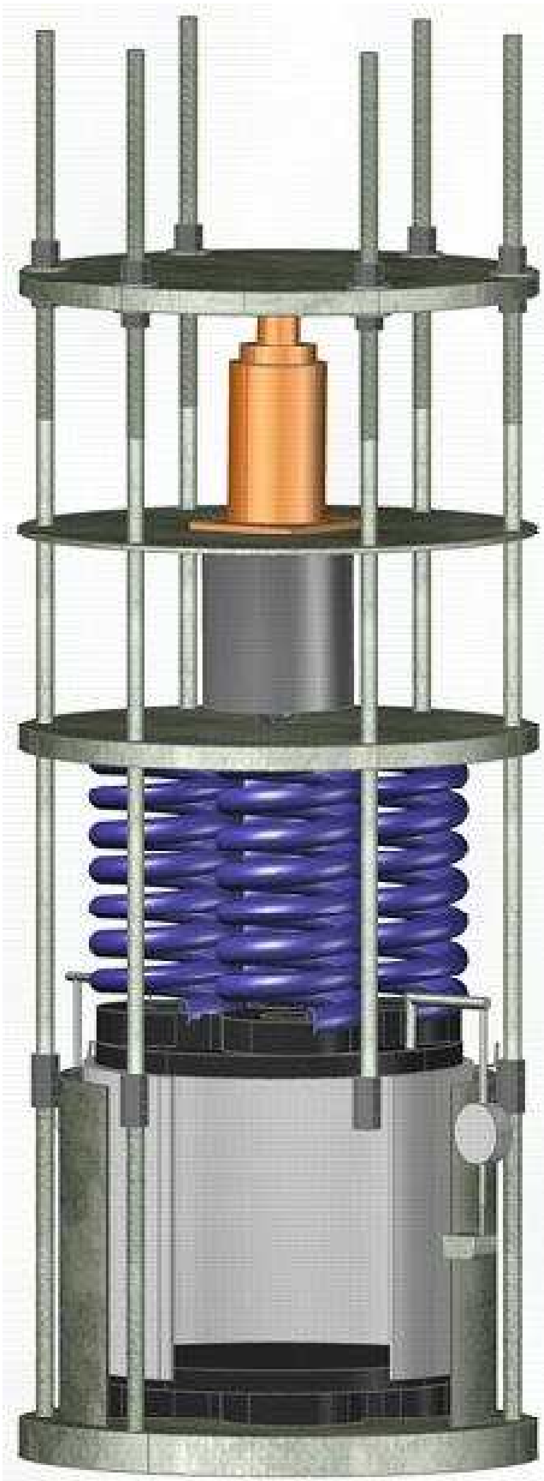

FINAL REPORT

LARGE SCALE CONSTRAINED MODULUS TEST

Presented to

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Section 1 Executive Summary

Constrained modulus (M_s) has recently found use in predicting deflection and buckling of buried pipe. Crushed rock (ASTM D 2321 Class I) is a preferred material for buried pipe support and only presumptive values of M_s are currently available. The purpose of this study was to measure M_s of crushed rock using a laboratory test. The study was designed to satisfy three objectives:

- Develop a test procedure for calculating the M_s of crushed rock.
- Experimentally determine M_s values for five crushed rock samples.
- Evaluate M_s values from Bureau of Reclamation archived data.

A one-dimensional compression test procedure was developed for measuring the constrained modulus[1] (M_s) of crushed rock and gravel. Various gradations of five different types of crushed rock from four sources were tested. The samples were hard, durable rock materials considered typical of crushed rock commonly used for pipe support. Each sample had uniform particle size and shape, and was tested at low, medium, and high densities and in wet and dry conditions. Test specimens were incrementally loaded to 150 lb/in^2, thereby simulating in excess of 150 feet of fill over a buried pipe. Twenty-eight tests were performed and M_s values were calculated to represent two stress ranges for each. The results and conclusions are summarized as follows:

- A test procedure for obtaining M_s of crushed rock was successfully developed.
- M_s values ranged between approximately 2,000 lb/in^2 and 10,000 lb/in^2.
- Load-deformation of crushed rock appears to be mainly due to fracturing of the particle edges. Accordingly, it is expected that distribution, angularity, shape, and hardness of the particles should affect the amount of fracturing, and therefore the magnitude of M_s. Indications of these effects were observed during the testing. A small moisture effect was also indicated.
- For each test, the load-deformation relationship was generally linear, indicating a constant M_s over the load range tested.
- For each crushed rock type, there was an approximately linear relationship between the constrained modulus and the placement density.

Additionally, M_s was calculated from data reported by the Bureau of Reclamation (BOR) for large-scale one-dimensional compression tests on soils containing gravel. These materials generally had more than 12 % fines and M_s values were between approximately 460 lb/in^2 and 6,700 lb/in^2.

Recommendations include:

- Further testing be performed to verify that higher M_s values may be possible using rounded to sub-rounded particles and/or a wider range of particles.
- Further testing be performed to evaluate the effect of rock hardness and shape on M_s.

[1] The constrained modulus is the ratio of stress to strain for the condition of no lateral strain (no net lateral particle displacement) during one-dimensional compression.

Section 2 Introduction

Clean gravel and crushed rock (12% fines or less) are preferred embedment for buried pipe (ASTM D 2321 Class I and Class II soils). Crushed rock, typically ¾ to 1½ inch size, is considered to be the stiffest embedment material. Estimated (presumptive) values of M_s for crushed rock are used in some instances to predict the deflection and buckling potential of buried flexible pipe. Presumptive M_s values for crushed rock used for pipe deflection calculations are derived from tests performed on finer soils and their validity has not previously been verified by laboratory experimentation. Hence, there is a need for testing to measure M_s for typical crushed rock samples so that a comparison can be made. This study developed a test procedure and measured M_s values for crushed rock. The results are presented herein.

A standard laboratory test procedure to measure the M_s of crushed rock does not exist. M_s can be measured in the laboratory by two traditional testing methods, one-dimensional compression tests and triaxial shear tests. These conventional tests are typically limited to fine-grained soils because cumbersome large-scale testing is needed for materials with gravel-size particles. A large-scale one-dimensional compression test method was developed for this study and implemented to measure M_s for crushed, hard rock that represents typical embedment material.

It is, in some cases, possible to extract M_s values from previously obtained data. The Bureau of Reclamation, BOR, has used large-scale tests to measure permeability and settlement of soils that contained gravel (USBR 5605). M_s can be calculated from this test data. However, the large-scale testing performed by BOR was generally for the purpose of evaluating soil permeability, rather than determining M_s.

This study fulfills three objectives. The first and primary objective is development of a test procedure for calculating the M_s of crushed rock and gravel. Second, M_s values for five crushed rock samples are determined by implementing this new procedure. Finally, M_s values for gravel-containing soils from BOR archived data are calculated and evaluated.

The remainder of this report is organized as follows. Section 3 presents background information describing the use of M_s to estimate the deflection and the buckling potential of buried flexible pipe, and thereby reinforces the need for this research. Section 4 summarizes the development of the test procedure. Section 5 discusses the M_s test conditions and results and includes a summary of supporting standard property tests performed on each sample. Section 6 presents the results of the search of BOR records for one-dimensional compression test data. Section 7 presents conclusions and recommendations. Section 8 contains a list of applicable test procedures and references.

Appendix A details the development of the M_s test procedure and Appendix B presents the test procedure in ASTM format. Appendix C details the results of M_s tests and supporting laboratory standard property tests. Appendix D contains the data from past BOR tests.

Section 3 Background

The deflection and the buckling potential of buried flexible pipe are significantly affected by the stiffness of the soil used for the pipe embedment. For the purpose of calculating pipe deflection, soil stiffness has been expressed by E′ and by M_s.

E′ was initially used in the Iowa Formula, and subsequently other equations, to predict the deflection of buried pipe. Originally E′ was only two or three values loosely associated with embedment soil types. Tables relating soil classification and placement density to empirically determined E′ values were published by Howard in 1977 and updated in 2006 (Howard 1977, 2006). In 1987, E′ values were published that suggested that the value generally increases with pressure; and consequently burial depth (Hartley and Duncan 1987). In 1998, M_s was proposed to replace E′ because it: 1) is a measure of soil stiffness that is similar in magnitude to E′, 2) can be obtained from a laboratory test, and 3) generally increases with pressure (McGrath 1998). Having laboratory measurement capability provides the ability to relate measurements to site-specific embedment placement conditions. Having a stiffness value that increases with depth may be particularly useful for estimating deflection and buckling potential for pipe used in deeper burials. Presently, E′ tables are only recommended for cover depths less than 50 feet and M_s tables are limited to stress levels less than or equal to 63 lb/in^2.

Presumptive M_s values based on the work of McGrath are used in some design protocols (AASHTO 2006). Many of these presumptive values, however, are based on testing only one type of soil and extrapolating the data for other soil types (AASHTO 2006). For example, the AASHTO presumptive values for crushed rock are based on tests performed on sand.

This study provides M_s values determined from laboratory tests that can be used for comparison to the empirically derived E′ and presumptive M_s values just described.

Section 4 Procedure Development

4.1 Traditional Soil Tests

M_s can be determined from the results of two common soil tests. These tests involve one- or three-dimensional loading of cylindrical shaped soil specimens. However, these tests each have limitations that inhibit their application to crushed rock specimens. The scale effect is a significant limitation common to all methods, and becomes a more significant issue with increasing particle size. Conventional tests and the scale effect are discussed in this section.

M_s can be calculated from results of a one-dimensional compression test, such as the one-dimensional consolidation test for fine-grained soil described in ASTM D 2435. Laboratories typically perform this test on saturated fine-grained soils such as clays. In this test, the soil is placed in a rigid cylinder and axially loaded. M_s is calculated as the ratio of applied stress to measured strain for each applied load. The standard one-dimensional consolidation test described in ASTM D 2435 is applicable to fine-grained soils. There is no equivalent standard for coarse-grained soils, such as crushed rock.

M_s can also be calculated from the results of a laboratory triaxial shear test. In this test a cylindrical soil specimen is contained circumferentially by a flexible membrane and an external pressure, and then axially loaded. Triaxial shear test samples are allowed to deform laterally during axial loading, and therefore are not constrained. Hence, assumptions are inherent in the calculation of M_s from triaxial shear tests. These assumptions add to uncertainty in calculated results.

The scale effect is a constant concern when testing soil, and becomes even more so when evaluating larger sized particles. As an extreme example, consider the compression behavior of a crushed rock composed entirely of 1 inch gravel. If this material were tested in one-dimensional compression using a 1 inch diameter, 1 inch high container, then only one particle could be tested. Intuitively, the load-displacement behavior of the one particle would be different (stiffer) than that for a test specimen composed of many particles. The scale effect in this example is the expectation that crushed rock will behave more stiffly when the particle size approaches the size of the specimen container. Because of scale effects, specimen sizes used for conventional soil tests are commensurate with the maximum particle size present in the soil. A review of ASTM standard soil tests indicates a general acceptance that the smallest dimension of a test specimen should be at least 6 times larger than the maximum particle size. For example, a specimen diameter of 9 inches would be required when the largest particle in the soil is 1½ inches.

The US Bureau of Reclamation has used large-scale tests to measure permeability and settlement of soils containing gravel (USBR 5605) for use in the design of earth dams. The test uses a 19 inch diameter container so that soils containing particles as large as 3 inches can be tested. The percent compression of the soil specimen due to an applied load is measured during the test. M_s can be calculated from the test data. However, this test was not typically performed on clean gravels or crushed rock.

4.2 New One-Dimensional Compression Test for Crushed Rock

M_s is defined as the ratio of applied axial stress to measured axial strain for the condition of no lateral strain (no net lateral particle displacement) during one-dimensional compression. The test developed for this study combines: 1) the use of the floating ring method that is commonly applied in consolidation testing of fine-grained soil, and 2) a loading methodology used in BOR's procedure "Determining Permeability and Settlement of Soils Containing Gravel" (USBR 5605). Details of test development and the resulting test procedure, prepared in ASTM format, are contained in Appendices A and B respectively.

The test procedure is fairly straightforward. A suite of four tests on a single sample includes three different placement compaction efforts with one repeated in both dry and wet conditions. Approximately 800 lbs of crushed rock is required. Assuming the test is performed on a routine basis, the four tests require two persons four or five days to perform and report. It is expected that performing and reporting a single test would require 200 lbs of crushed rock, and take two people approximately two days. When testing is not performed on a routine basis, additional costs may include load-cell calibration and added time associated with initial equipment assembly.

Section 5 Results of M_S Tests

Twenty-seven tests were completed on five clean, crushed rock samples obtained from four sources in the Colorado area. The samples are identified as MS-1, MS-2, MS-3, MS-4, and MS-5. For each sample, a series of at least four one-dimensional compression tests was performed using three different compaction efforts in order to obtain a representative range of M_s values. Twenty-one of the tests were performed wet and six were performed dry. Table 1 summarizes the test conditions and results. Appendix C presents and discusses the test results along with a select set of standard properties tests performed on each sample. The remainder of this section discusses the effect of different variables on placement conditions and M_s results.

5.1 Effect of Compaction Effort on Placement Dry Density

Generally, three different compaction efforts were used to achieve different specimen placement dry densities with replicate tests being performed periodically. Compaction efforts are described as follows:

- Low - Hand placement with no compaction (not representative of construction practice but is used for geotechnical evaluation)

- Medium - Raining particles into the container in level lifts from a height of approximately two feet
- High - Hand compaction in shallow layers

The different compaction efforts produced the desired wide range of sample placement densities as evidenced by the data shown on Table 1. Sample placement densities for repeated tests using the same compaction efforts and the same sample are generally within 4 lbs/ft^3.

Table 1. Test Conditions and Results

Identification			Placement Conditions				Results (lb/in^2)	
Sample ID	Vendor Description*	Apparent Specific Gravity	Moisture Condition	Compaction Effort/ Method	Relative Density	Dry Density (lb/ft^3)	Ms Stress Range: 2.17 -151	Ms Stress Range: 2.17 -87
MS-1	1/2 in Granite	2.77	Dry	Low	14%	90	5059	5388
			Wet	Low	18%	91	3985	4230
			Wet	Low	19%	91	3969	4074
			Dry	Low	23%	92	5287	5367
			Wet	Low	31%	94	4939	5222
			Wet	Medium	44%	98	6890	7494
			Wet	High	69%	106	9772	9595
			Wet	High	74%	108	9811	9437
MS-2	3/4 in Granite	2.67	Wet	Low	12%	89	3339	4052
			Dry	Medium	39%	93	6950	6737
			Wet	Medium	62%	97	7171	7709
			Wet	High	97%	103	8484	8256
MS-3	1-1/2 in Granite	2.61	Wet	Low	15%	87	1727	2215
			Wet	Medium	82%	95	3515	4726
			Dry	High	94%	97	6190	5910
			Wet	High	101%	98	4930	6220
MS-4	3/4 in Limestone	2.65	Wet	Low	-29%	85	2087	2213
			Dry	Low	-3%	88	3564	4278
			Wet	Medium	33%	93	3675	4274
			Wet	High	64%	98	5428	7541
MS-5	3/4 in Quartzite	2.53	Wet	Low,Plate**	-47%	72	3835	4097
			Wet	Low,Plate**	-38%	73	3332	3843
			Wet	Low	-9%	76	3011	3601
			Wet	Low	9%	78	3816	4607
			Wet	Medium	38%	81	5414	6413
			Dry	Medium	61%	84	6351	6660
			Wet	High	93%	88	7306	7722

* These are the descriptions provided by the crushed rock vendors. More detailed geologic descriptions provided by representatives of the quarries are included in Appendix C.

** These unique tests were performed with a steel plate placed horizontally in the center of the specimen and are discussed in Appendix C. They are not used to assess effect of test variables on M_s in subsequent discussions.

5.2 Effect of Stress Level on M_s

The stress-strain responses of the crushed rock samples tested were approximately linear. For example, refer to the stress-strain data for Sample MS-3 tests that are shown on Figures 1 and 2. Figure 1 presents stress-strain curves created by connecting the individual measurements by line segments. Figure 2 presents linear trendlines (also known as "line of best-fit") to the loading portions of the curves. M_s, the ratio of stress to strain, is the slope of the best-fit line for the stress or strain interval of interest adjusted for scale and friction effects. The linearity of the best-fit line is visually apparent. The correlation coefficients, r, were above 0.90 for all the tests in this study.

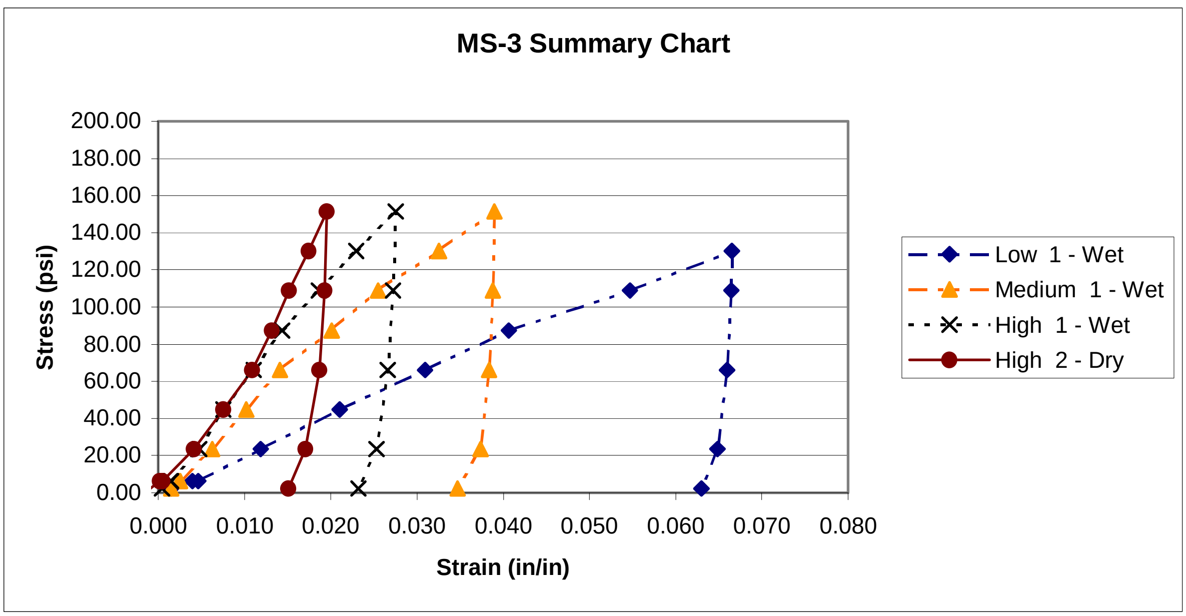


Figure 1. MS-3 Stress-Strain Plots for Various Compaction Efforts

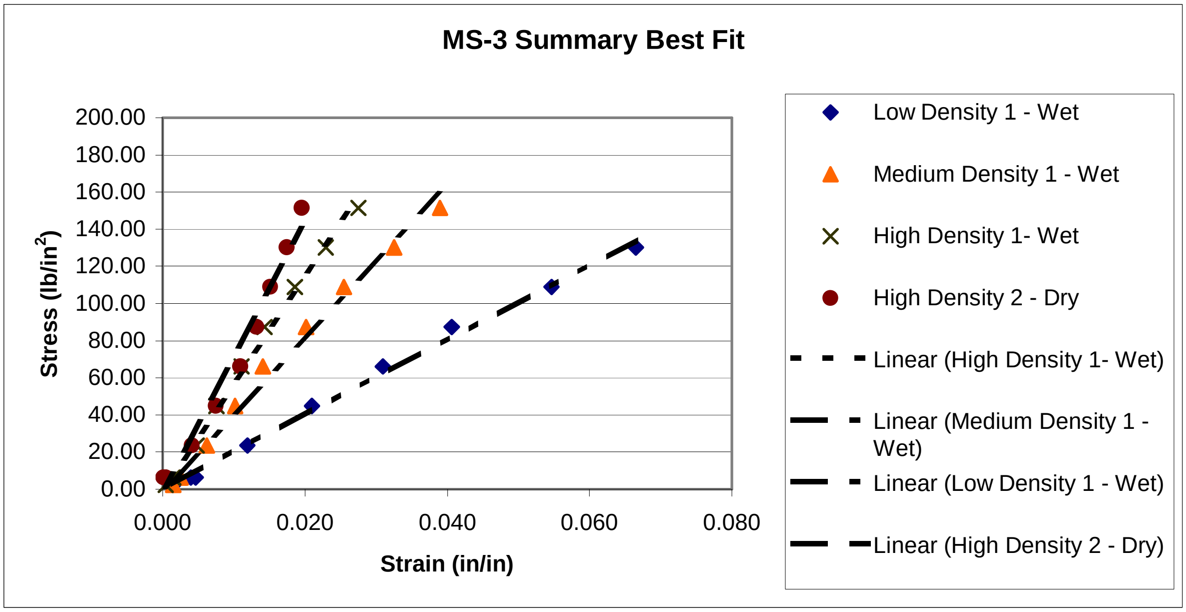


Figure 2. MS-3 Best-Fit Lines for Various Compaction Efforts

To further evaluate the effect of stress, M_s was calculated for the applied axial stress ranges 2.3 lb/in^2 to 87.5 lb/in^2 and 2.3 lb/in^2 to 151.5 lb/in^2 for all tests. These values for the 27 test specimens are tabularized in Table C.2, Appendix C. On the average, the M_s for the 2.3 lb/in^2 - 87.5 lb/in^2 range is approximately ten percent higher than the M_s for 2.3 lb/in^2 -151.5 lb/in^2 range.

The modulus is slightly greater at lower stress levels than at higher stress levels. Because the dependence on stress level is small, the stress-strain response for the crushed rock tested is considered approximately linear. The observed response is understandable considering the likely mechanism of compression that is detailed below.

When the load is increased on a crushed rock test specimen, the particle-to-particle contact stresses are increased. Some contact points are overloaded and break, resulting in the crushed rock specimen compressing. As the specimen compresses, the applied load is redistributed throughout a newly formed network of particle-to-particle contact points. The observed approximately linear stress-strain response suggests that particle-to-particle contact stresses are redistributed in a manner such that the distribution of contact stresses remains approximately unchanged as the crushed rock compresses. More generally, a linear response occurs if a load is applied resulting in approximately one percent of the contacts points breaking, and subsequent equivalent increments of load result in one percent of the contact points breaking. This is thought to be a consequence of the angular nature of crushed rock in combination with a narrow range of particle sizes and the small range of strain being considered. At larger strains, a significant amount of void space would disappear (commonly demonstrated in consolidation testing of fine-grained soil) and consequently a continually stiffer response would be expected. Consider the extreme condition - at some large strain all void space would disappear and subsequent application of load would result in a stress-strain response representing the rock mineral strength.

The validity of this mechanism of compression is supported by the presence of a significant amount of fractured rock observed in test specimens following tests (percent particle breakdown), and by the acoustic emissions that were easily recognizable during loading of all test specimens. The percent particle breakdown for each test is listed in Table C.2, Appendix C.

5.3 Effect of Placement Dry Density on M_s

As discussed in Section 5.1, three different compaction efforts were used in testing. This was done in an attempt to create test specimens having densities near the minimum index density (ASTM D-4254), the maximum index density (ASTM D-4253), and midway between these two densities. Each of the three compaction efforts, and their application to each sample, is described in Appendix C. An approximately linear relationship was observed between M_s and density.

Figure 3 demonstrates the approximately linear relationship between measured M_s and placement dry density for MS-1. Figure 4 shows each sample's relationship between M_s and placement dry density using the 19 specimens that were tested in a wet condition (two special tests performed on MS-5 using different test conditions were excluded). The strong linear relationship between M_s and placement dry density is apparent for all samples tested in wet conditions, as seen by the data shown on Figure 4. However, the same linear equation does not apply universally to all types of crushed rock, as most readily evidenced on Figure 4 by comparing the MS-5 data trendline located on the left side of the graph to the single trendline representing MS-1 through MS-4 data.

Evaluation of test data suggests that M_s can typically be estimated for a select dry density to within 500 lb/in^2 by completing the procedure in Appendix B for test conditions created by high, medium and low compaction efforts, and obtaining the slope of the line of best fit for the data obtained.

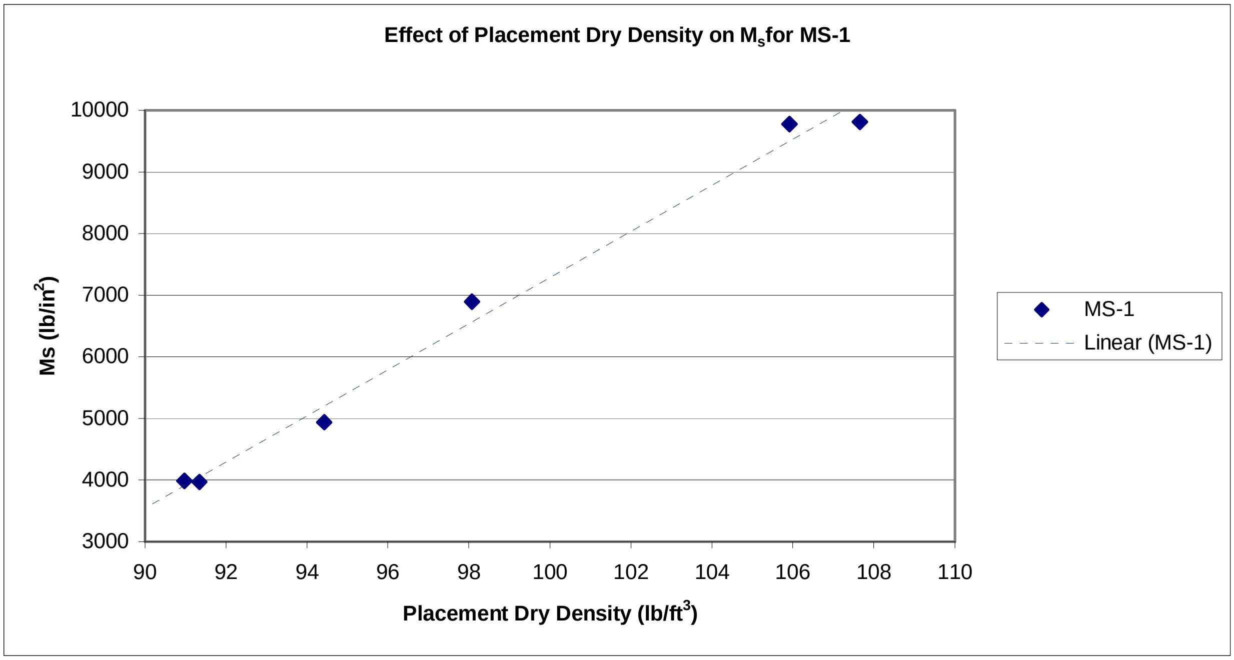


Figure 3. Effect of Placement Dry Density on M_s measured for Sample MS-1

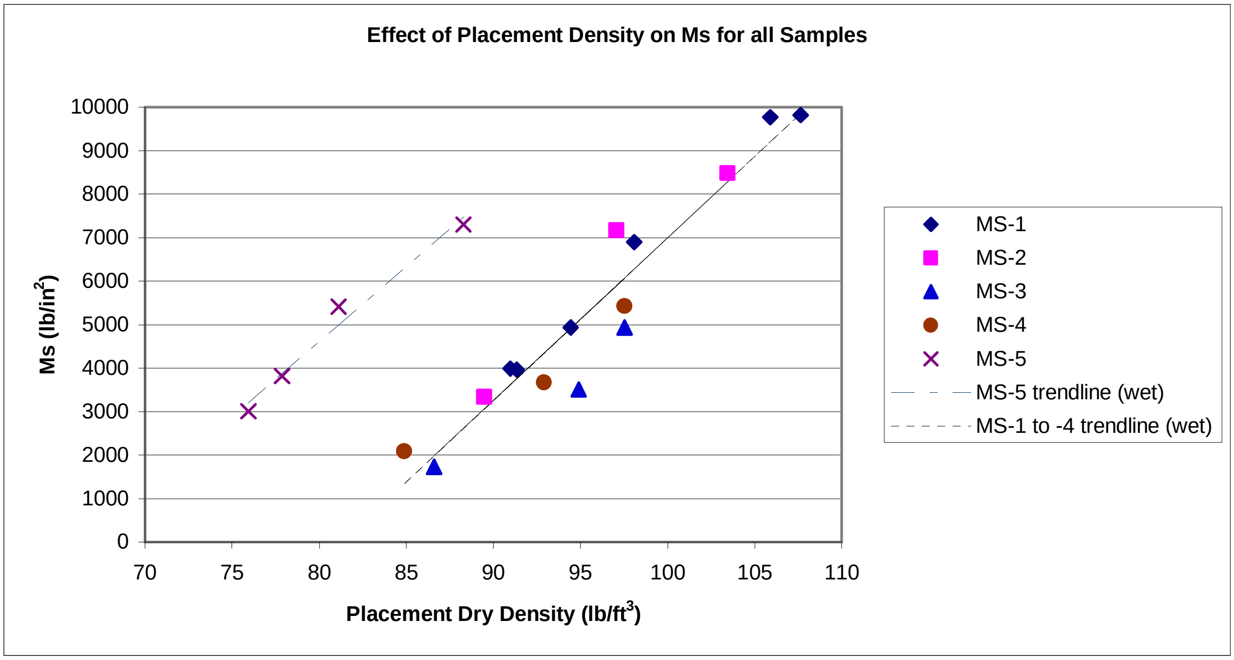


Figure 4. Effect of Placement Dry Density on M_s for all samples

Figure 5 presents the same test results shown on Figure 4, but compares the relative density (RD) to M_s. RD is a calculated value that expresses the dry density relative to minimum and maximum index densities. Again, each sample exhibits a strong, generally linear relationship, as indicated by trendlines on the figure. However, each trendline has a unique slope and location.

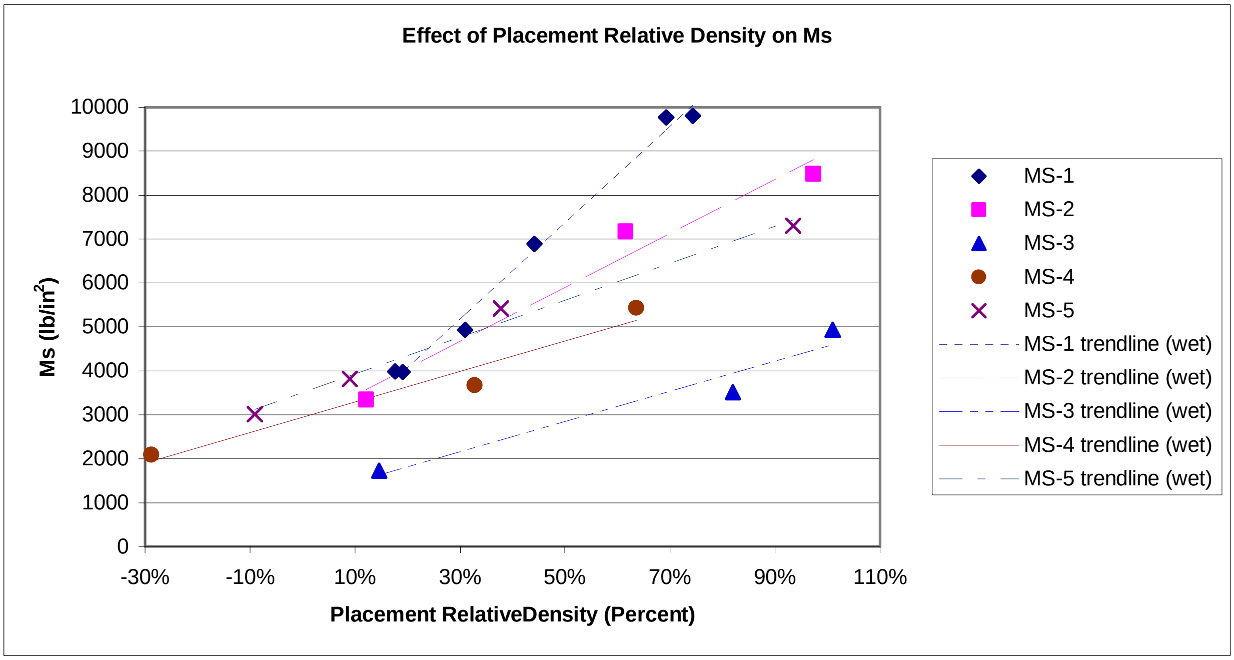


Figure 5. Effect of Placement Relative Density on M_s

5.4 Effect of Placement Moisture on M_s

Wet and dry tests were performed. Six specimens were tested dry and 21 were saturated with water during or immediately following placement in the test cylinder, and then tested. Figure 6 presents the results of dry tests superimposed on the results of wet tests that were previously presented on Figure 4.

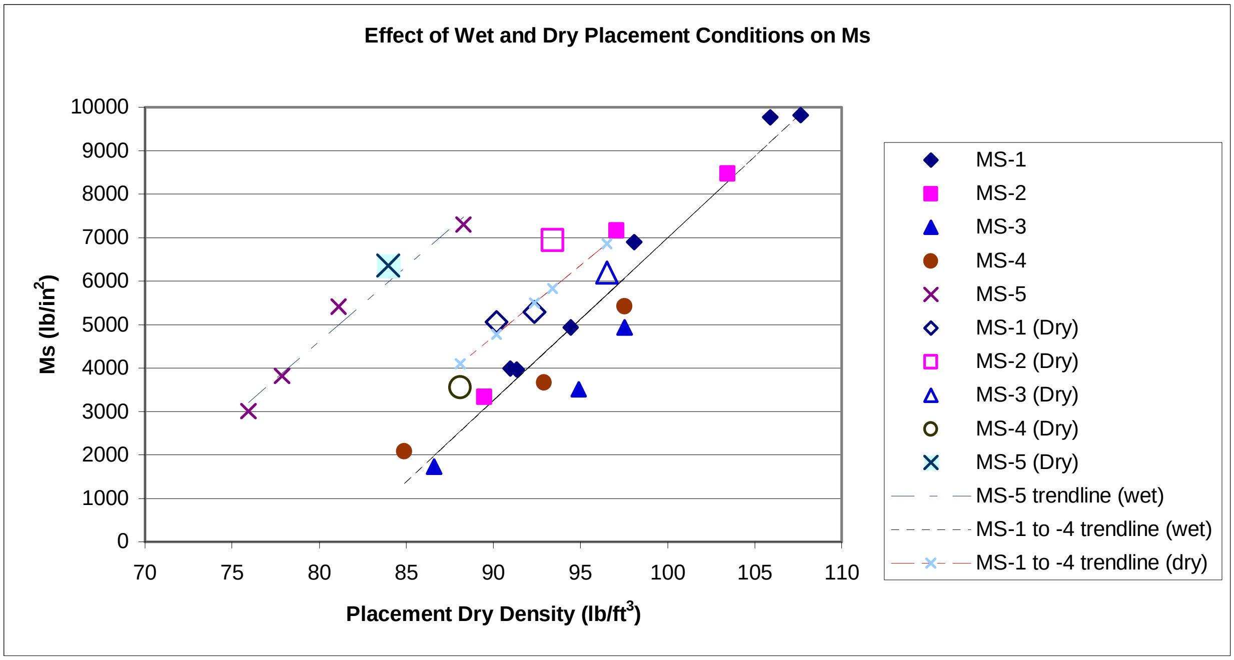


Figure 6. Effect of Placement Moisture Condition on Ms

Generally, the trendline for dry tests plotted above the trendline for respective wet samples. This indicates that wet conditions result in a lower M_s. However, the set of dry test results is too small to confidently establish the magnitude of this difference.

5.5 Effect of Particle Size Distribution on M_s

Particle gradation can significantly affect density and hence the volume of voids available for compression. It is reasonable to expect that a rock with less void space available for compression would be less compressible, all other things equal. This is evident in the density effect plot of Figure 4, which shows M_s decreasing with decreasing dry density, since decreasing density is known to be associated with increasing void space. Less uniformly graded material would be expected to result in higher placement densities (lower placement void space) and consequently exhibit higher M_s. Less uniform gradations are expected to distribute stresses among more contact points. Consequently, larger loads must be applied to cause stresses at contact points to exceed the rock compressive strength and result in particles breaking. For this reason, less uniformly graded material would be expected to result in higher M_s.

The particle size analysis results for the samples tested are provided in Table C.1, Appendix C, and are shown graphically on Figure 7. Because all samples tested were uniformly graded, the effect of particle size distribution cannot be discerned by this study. The particle distribution effect can be evaluated by additional tests using the following concept.

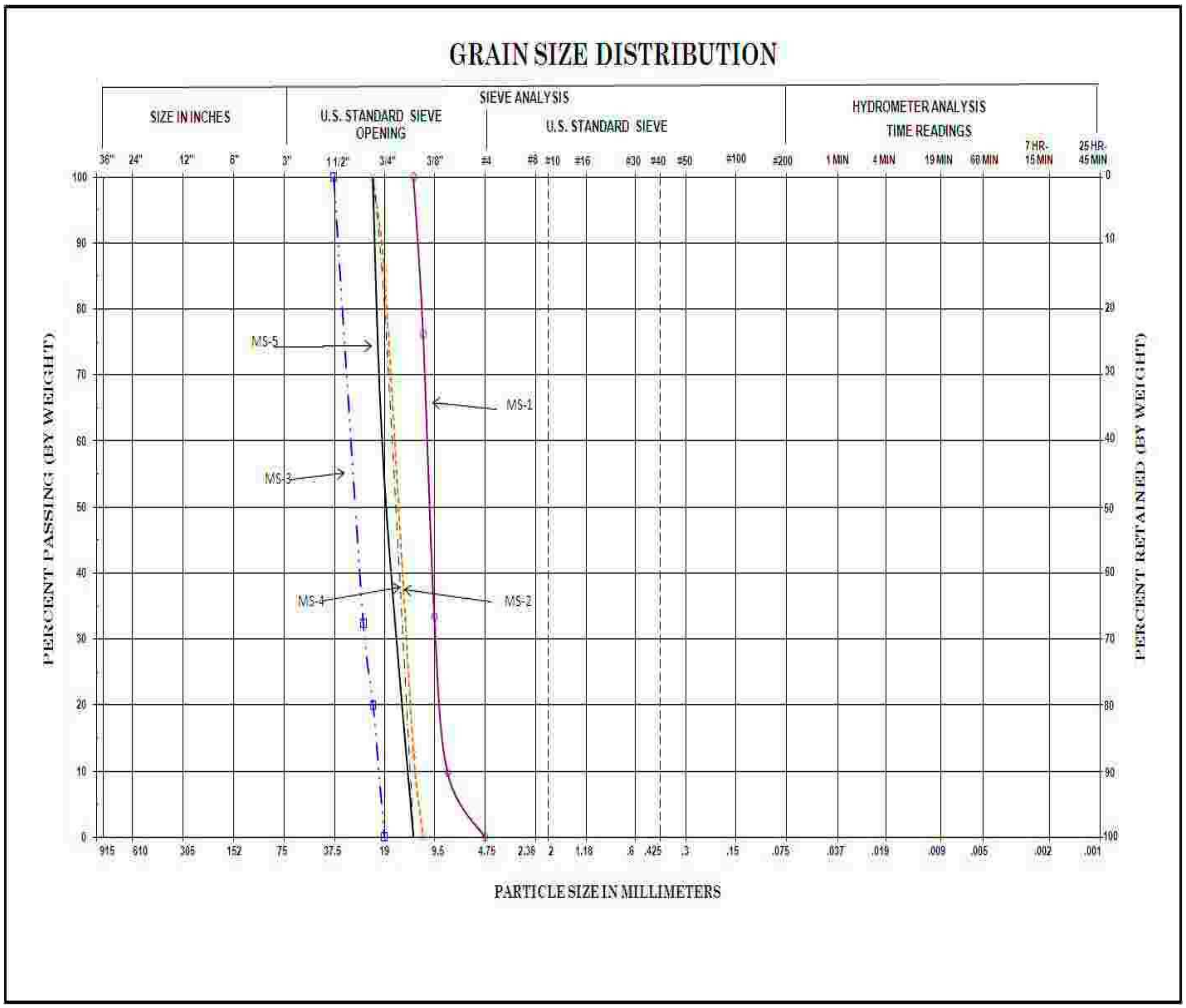


Figure 7. Grain Size Distribution for MS-1 through MS-5

Crushed rock representing a fine-grained mineral from a uniform geologic unit, such as MS-1, is expected to have the same hardness, angularity, shape and specific gravity for all particle size distributions. Therefore, the effect of particle size distribution on M_s can be assessed by selecting and testing different particle distributions from a single source of crushed rock. Replicate tests will be necessary to provide greater confidence in developed relationships. The effect of particle gradation on M_s may be different for crushed rock of different shape or hardness.

5.6 Effect of Hardness on M_s

The audible cracking of rock during all tests supports the hypothesis that particle fracturing significantly influences the stress-strain behavior of crushed rock. Sieve analysis of the post-test sample showed a 4 to 13 percent particle breakdown. This was determined after testing by measuring the percentage of particles that passed the largest sieve opening that previously retained all particles. Consequently, crushing at individual points of contact between particles appears to be the governing process resulting in volume change and is likely responsible for most compression of the test specimens observed in this test study. Since the hardness of the rock particles would have an effect on the amount of stress required to fracture the particle, particle hardness is considered a significant factor in crushed rock compression.

Rock hardness is directly related to the apparent bulk specific gravity and inversely related to the percent loss in the LA abrasion test. Apparent bulk specific gravity and percent loss by the LA abrasion method were determined for each of the five samples tested in an attempt to quantify the hardness. These results, reported in Table C.1, Appendix C, support a conclusion that the parent rocks from which the five samples were created (by crushing) are all of similar hardness. All five of the tested crushed rock samples, assessed qualitatively by handling and by visual appearance, are considered hard rock. Crushed rock from weaker rock source such as sandstone, or weathered granite, should be more compressible than the samples tested and would likely demonstrate lower M_s values, all other things equal.

5.7 Effect of Particle Angularity on M_s

The angularity of the particles affects how particles fracture under load and how easily they rearrange into a denser structure due to vibration, impact, kneading or pressure. It is possible that rounded to subrounded rock particles may arrange in a denser state during placement and not fracture as easily as more angular crushed rock. Particles with more rounded edges are expected to withstand higher point loads at contact points than sharp-edged angular particles. For these reasons, it is expected that M_s decreases with increasing particle angularity. Therefore, a higher constrained modulus may result from using rounded to subrounded particles. The effect of angularity should be further investigated.

Angular particles are expected to form steeper slopes than rounded particles, therefore, a measure of a materials slope-forming angle is a simple and reproducible index of particle angularity. The angle of repose test measures the slope formed by particles and was performed on each sample to provide a simple index of angularity (USBR 5380). The results are reported in Table C.1, Appendix C. The results were typical and ranged between 37 and 40 degrees for samples MS-2 to MS-5. MS-1 had an angle of repose five degrees higher (45 degrees) than the other samples suggesting it is a slightly more angular material. Rounded materials typically have a much lower angle of repose. Therefore, future evaluation of the effect of particle angularity should include measurement of the angle of repose.

5.8 Effect of Particle Shape On M_s

The shapes of grab samples of each rock tested were measured in accordance with ASTM D 4791, and the results are summarized in Table C.1, Appendix C. ASTM D 4791 provides for measuring shapes of gravel particles to determine percentages of “flats” and “elongates”. The test is typically used to evaluate concrete aggregate because a significant portion of flats and/or elongate particles in a soil may affect mix design and placement. It is also known that the presence of a large percentage of flat and elongated particles also affects the ability to compact soil. No flats were found in 30-particle representations of each sample. Two elongates were found in MS-1 and no elongates were found in the other samples. Based on the low occurrence of flats and elongates, it is concluded the samples tested were all approximately uniform in shape.

Elongated and flat shaped particles are expected to be more susceptible to bending and breaking at particle interior sections than more equal-dimensional particles, all other things equal. Therefore, it is expected that M_s will generally decrease with increasing percentages of flat and elongated particles.

Section 6 BOR Data

The BOR procedure "Determining Permeability and Settlement of Soils Containing Gravel" outlines a procedure for determining permeability and settlement characteristics of compacted soils containing gravel particles 3 inches or less in diameter (USBR 5605). The procedure was used most extensively in the 1960's to evaluate the one-dimensional compressibility of gravels used in embankment dams. The test results were reported in Earth Materials (EM) reports that are archived in paper copy and on microfiche at the BOR offices in Denver, Colorado. The EM reports were reviewed and data related to settlement of soils containing gravel was extracted. This data was used to calculate the M_s values that are presented in Appendix D.

USBR 5605 was typically performed to evaluate permeability of gravel and gravelly soils. Settlement characteristic measurements were often not the property being sought. Consequently, many tests were never loaded beyond 1 lb/in^2 to 3 lb/in^2. Only one test was found that acquired data adequate to plot stress vs. strain.

Typically, USBR 5605 involved applying a pressure on a dry or moist specimen and measuring the settlement as percent strain. Then, without changing the applied pressure, water was added to the sample, and the settlement was again calculated. The data in Appendix D shows that a small strain often occurred upon addition of water.

BOR data was used to calculate M_s for 27 tests for which pressures applied ranged from 20 lb/in^2 to 100 lb/in^2. Fines contents of samples tested ranged between 1 and 51 percent and gravel contents ranged between 0 and 79 percent. Calculated M_s ranged between 460 lb/in^2 and 6,700 lb/in^2.

USBR 5605 was better suited for measuring gravel permeability than M_s, and as a result a few potential problems exist. The issues associated with using this test to measure M_s are discussed in the following paragraph, along with the way these problems were addressed and corrected in the new test method developed in Appendix A and presented in Appendix B.

USBR 5605 used the deflection of calibrated springs to determine the magnitude of the applied load. This made it difficult to make rapid load measurements. Also, USBR 5605 included a ¼ inch thick rubber liner glued to the inside of the test chamber. It is likely that the compression of this rubber layer is responsible for some of the measured settlement. Additionally, friction that developed between the side of the rubber-lined test chamber and the sample may have significantly reduced the pressure felt by the soil at increasing depths within the sample, making the effective applied load at the bottom of the test specimen somewhat less than the load applied at the top. These problems were addressed in the new procedure presented in Appendix B by: 1) using a load cell to measure applied load, 2) not using a rubber membrane but rather Teflon coating the test cylinder to reduce friction, 3) adopting a floating ring to reduce the depth of specimen affected by friction and 4) using a friction correction factor when calculating M_s.

USBR 5605 presumed the settlement behavior observed in the test was insignificantly affected by interference of the cylinder walls with the movement of immediately adjacent particles. This is not a correct assumption. To help understand this problem, consider a particle on the interior of the test specimen. It can move in any direction into an adjacent void during loading. However, a particle resting against the edge of the container cannot move in the direction of the container wall. The effect of this constraint is a stiffer response of the test specimen. This stiffer response is more significant as the particle sizes become larger, but is probably of acceptably low significance when the maximum particles size is less than about 1-½ inch. This scale effect may have been significant to settlement measurements when a BOR sample contained a significant percentage of 3 inch particles. This problem was addressed in the new procedure by limiting the maximum particle size in the test specimen to 1-½ inch and by using a scale correction factor when calculating M_s. The development of this scale correction factor is briefly outlined in Appendix A.

Section 7 Conclusions and Recommendations

A test procedure was developed to determine M_s for crushed rock or clean gravel having maximum particle size 1-½ inch or less. Using this procedure, it was demonstrated that M_s of hard, uniformly graded and shaped crushed rock:

- is typically constant regardless of stress level;
- is strongly dependent on sample compaction effort;
- is weakly dependent on placement moisture condition;
- varies with changes in crushed rock particle characteristics.

The independence of M_s on stress level is understandable given the observed importance of particle crushing on the load-deformation behavior.

A strong dependence of M_s on compaction effort was expected and observed. This dependence is demonstrated by test results presented and discussed in Section 5.1 and summarized below in Table 2. In Table 2, the average of measured M_s values representing low, medium and high compaction efforts and wet samples are rounded to the nearest 500 lb/in^2 to enhance visual clarity, and to suggest an appropriate level of precision and accuracy.

Table 2. Summary of Constrained Modulus Results (lb/in2)

			Compaction Effort	
Sample	Vendor Description	Low (hand placement)	Medium (raining particles)	High (hand tamping)
MS-1	1/2 in Granite	4000	7000	10000
MS-2	3/4 in Granite	3500	7000	8500
MS-3	1-1/2 in Granite	2000	3500	5000
MS-4	3/4 in Limestone	2000	3500	5500
MS-5	3/4 in Quartzite	3500	5500	7500
	Range	2000 - 4000	3500 - 7000	5000 - 10000

A weak dependence of M_s on moisture was observed in tests performed using the new procedure and also in BOR results.

The large variation in M_s values between samples suggests a strong influence of rock particle characteristics not controlled by this study. Four important crushed-rock-particle characteristics not controlled during testing are:

- hardness
- size distribution
- angularity
- shape

All samples tested in this study were composed of particles that were hard, had a narrow size-distribution range, were angular, and generally uniform in shape. Because these properties did not vary measurably between samples, their effect on M_s could not be demonstrated. However, it is evident by the differing load-displacement behavior of each rock type that small variations in one or more of these properties are significant.

The procedure provided in Appendix B may be used to determine M_s for a specific crushed rock or gravel in wet or dry conditions using a range of placement densities. Such measurement will reduce uncertainty in M_s used to estimate pipe deflection, and may be used to establish appropriate construction compaction effort.

It is concluded that although the crushed rock samples selected for testing were typical of crushed rock used as pipe embedment, they do not represent the full range of possibilities. It is recommended that the influence of the above listed variables be evaluated by additional experimentation. As discussed in Sections 5.5 and 5.7, testing should be performed to verify the expectation that higher M_s values are likely using rounded to sub-rounded particles and a wider range of particles.

Section 8 References

8.1 ASTM Standards

American Society for Testing and Materials, Annual Book of ASTM Standards, Vol. 04.08.

D 653 "Terminology Relating to Soil, Rock, and Contained Fluids"

D 2435 "One-Dimensional Consolidation Properties of Soils Using Incremental Loading"

D 2488 "Description and Identification of Soils (Visual-Manual Procedure)"

D 4791 "Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate"

8.2 USBR Standards

US Bureau of Reclamation, Earth Manual, Third Edition, Part 2, 1990.

USBR 5380, "Determining the Angle of Repose of Soils"
(There is no equivalent ASTM standard)

USBR 5605, "Determining Permeability and Settlement of Soils Containing Gravel"
(There is no equivalent ASTM standard)

8.3 Cited References

(Hartley and Duncan 1987) Hartley, J. P. and J. M. Duncan, "E' and its Variations with Depth," *Journal of Transportation Engineering*, American Society of Civil Engineers, Vol 113 No 5, 1987.

(Howard 1977) Howard, A. K., "Modulus of Soil Reaction Values for Buried Flexible Pipe," Journal of the Geotechnical Engineering Division, ASCE, Vol 103, No GT1, January 1977.

(Howard 2006) Howard, Amster, "The Reclamation E' Table 25 years Later," Proceedings, *Plastic Pipes XIII*, Washington DC, Oct 2-5 2006.

(McGrath 1998) McGrath, T. J., "Replacing E' with the Constrained Modulus in Flexible Pipe Design,"ASCE Conference Proceedings – *Pipelines in the Constructed Environment*, American Society of Civil Engineers, 1998.

Appendix A: *Procedure Development- Procedure for Determining the Constrained Modulus of Clean Gravel and Crushed Rock*

APPENDIX A. PROCEDURE DEVELOPMENT - CONSTRAINED MODULUS OF CLEAN GRAVEL AND CRUSHED ROCK

INTRODUCTION

The Constrained Modulus of Clean Gravel and Crushed Rock procedure is a one-dimensional compression test that was developed, as its name suggests, to determine the constrained modulus (M_s) of crushed rock. The constrained modulus is defined as the ratio of applied axial stress to measured axial strain, for the condition of no net lateral strain, i.e. lateral strain is constrained. The developed test combines 1) the use of the floating ring method that is commonly applied in consolidation testing of fine-grained soil, and 2) a loading methodology commonly used by the Bureau of Reclamation for applying axial loads to constrained gravel specimens for large scale permeability tests (BOR 1990). The following discusses the development of testing equipment; the development of appropriate adjustment factors to correct for expected friction and scale effects; the development of quality control measurements; and presents reasoning for the supporting crushed rock physical properties tests. The resulting procedure, written in ASTM format, is included in Appendix B.

DEVELOPMENT OF TESTING EQUIPMENT & EFFECT FACTORS

Equipment

The Constrained Modulus of Crushed Rock test is designed to fulfill practical application purposes. The equipment can be readily fabricated and easily and safely operated. The equipment is generally a modified version of the equipment used by the Bureau of Reclamation in its procedure “Determining Permeability and Settlement of Soils Containing Gravel” (USBR 5605). The equipment modifications were necessary to add a load cell for more accurate force measurement and a Teflon coated floating ring (test cylinder) to reduce frictional effects. Photographs of the USBR apparatus (BOR 1974) and the modified loading system are shown on Figure A-1.

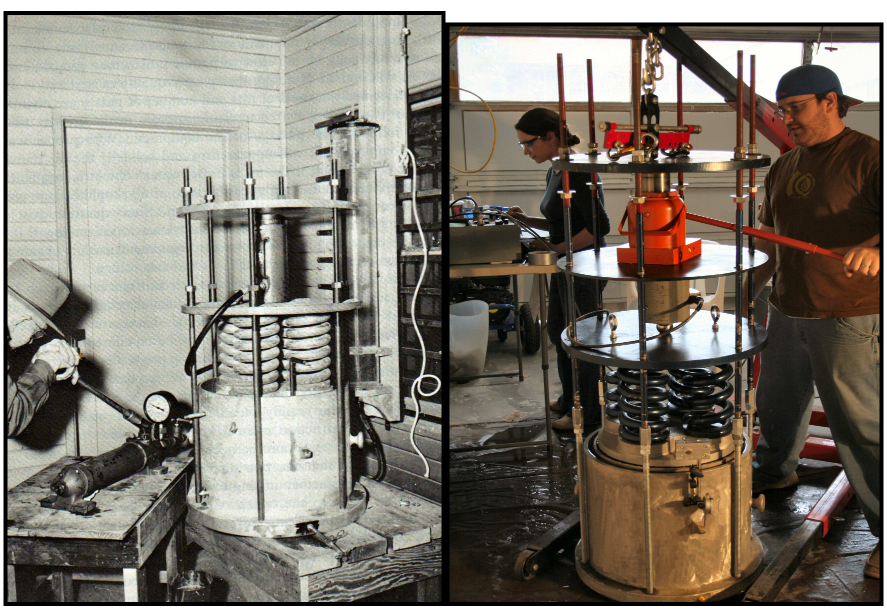


Figure A-1 Typical Equipment for USBR 5605 vs. Constrained Modulus Test Modified Loading System

The primary components of the testing equipment are shown on Figure A-1 and consist of: a 19 inch inside diameter aluminum test chamber, two steel loading platens, an approximately 17 inch inside diameter steel testing cylinder, two dial indicators (with reference brackets), three reaction plates, six threaded steel tension rods, four loading springs, and a 2 inch steel ball bearing. A hydraulic jack is used to apply loading, and a load cell is used to measure the loads.

The Test may be run on wet or dry specimens. The 17.3 inch i.d. test cylinder is placed inside the 19.0 inch i.d. test chamber, suspended temporarily by metal pins to be centered over the lower steel loading platen and floating above the base of the test chamber.

Figure A-2 Test Cylinder, Ready for Loading

After or during specimen placement, the test chamber may be filled with water to saturate the specimen.

Figure A-3 Filled Test Cylinder; Ready for Saturation

The second steel loading platen is centered on top of the sample and acts as a base for the four loading springs; and subsequently the remainder of the loading apparatus. It also serves as the connection point between the dial gages and the dial gage reference brackets, which extend beyond the edges of the test chamber at two locations to allow measurement of the top loading platen displacement. The dial gages measure the deflection upon loading.

Figure A-4 Zeroing Dial Gages

The six equidistant steel tension rods are slipped through the center three reaction plates, and threaded into the test chamber base. The top reaction plate is positioned parallel to the base of the test chamber and held in place by threaded nuts. The vertical placement of the three plates provides spacing for the load cell (between the bottom and middle reaction plates) and the hydraulic jack (between the middle and upper reaction plates). The load cell is placed in the center of the bottom reaction plate on the 2 inch steel ball bearing (a groove is molded into the center of the bottom reaction plate to center the load cell, and allows for simplified placement). This entire section (steel rods, reaction plates, hydraulic jack, and load cell) is placed on top of the loading springs using a hoist, as demonstrated in Figure A-5.

Once the weight of the upper portion of the testing apparatus is released from the hoist, and the steel rods are threaded into the test chamber base; testing can commence. The weight of the springs, center two loading plates, jack, and load cell is referred to as the seating load. An additional 1000 lb load is applied to create enough force to hold the test specimen inside the test cylinder. The steel pins that suspend the test cylinder above the base of the test chamber are removed after application of this load. The load cell is used to measure the loads applied by the hydraulic jack; and at each loading increment, readings are taken from the dial gages affixed to opposing sides of the upper reaction plate. Stress and strain are calculated for each increment of applied load using these values, along with the height and area of the specimen. Each point is plotted on a stress-strain curve and the slope of the best-fit line (corrected for friction and scale effects as discussed below) yields the constrained modulus (M_s) value.

Figure A-5 Upper Mobile Portion of Loading System Lowered by Hoist

Figure A-6 Completely Assembled Equipment for Constrained Modulus Test

Friction Effects

It is desired to have a uniform axial force throughout the test specimen. However, friction between the crushed rock and the test cylinder wall results in a net reduction in axial force. For a specimen tested in a rigid fix-bottomed cylinder, this effect accumulates with distance from the location of the applied loads.

The concept of the floating ring is discussed in most text on soil mechanics and is used primarily in the performance of one-dimensional consolidation tests (also see ASTM D 2435). In effect, the concept requires a cylindrically shaped specimen (crushed rock or gravel in this case) be tested in a rigid cylinder that is held in place only by the friction between the specimen and the cylinder. By doing this, the axially force applied to the top of the specimen is resisted by an equal and opposite force acting on the bottom of the specimen. In contrast, when a rigid container with an affixed base is used (no floating ring) the force at the bottom of the specimen would equal the force applied at the top minus cumulative frictional resistance between the specimen and the container wall.

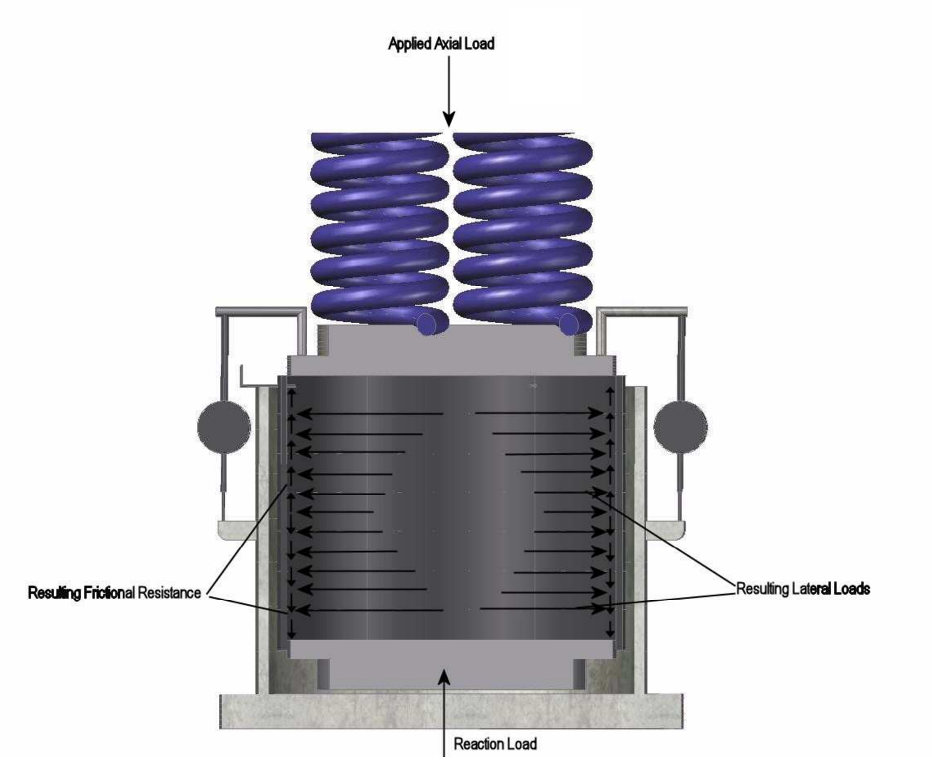


Figure A-7 Typical Equipment for USBR 5605 vs. Constrained Modulus Test Modified Loading System

Using the floating ring rather than a container with an affixed base, in effect, reduces friction on the specimen by about half. Since friction is not entirely eliminated, its effect on the measured value of Ms must be properly considered. Friction between the crushed rock and the side of the container progressively reduces the net axial force felt by the crushed rock as the center plane of the specimen is approached from the loaded ends. A correction factor to the measured Ms is derived by assuming the stress acting on the wall of the test cylinder is equal to the product of the axial stress and a constant (k). The constant, k, is approximated as 0.3, which is common for this type of material. The axial stress reduction with increasing depth towards the center within the specimen can then be calculated using a common value for the coefficient of friction (μ)

between Teflon and other material (0.2) and presuming the stress reduction is uniform on horizontal planes within the specimen. This can subsequently be used to derive the following friction correction factor (f2) to be applied by multiplication with the observed ratio of applied axial stress to measured axial strain.

$$f2 = \frac{D \cdot \left(1 - e^{-\frac{k \cdot \mu \cdot H}{D}}\right)}{k \cdot \mu \cdot H}$$

where:

e = 2.718
k = 0.3 (represents the ratio of radial to axial stress within the test specimen)
u = 0.2 (represents the coefficient of friction between the Teflon coated specimen container and crushed rock)

Scale Effects

Testing large particles requires large test specimens. As particles become larger, or the container smaller, the specimen will behave more rigidly. This is because the imposed rigid boundary of the container prevents movement of particles into voids that otherwise would be available for such movement had the boundary been absent. The distance from the boundary affected by imposing a rigid boundary is hypothesized to be approximately equal to half the hypothetical average specimen void diameter (Θ). A scale correction factor (f1) is consequently derived by assuming a layer of thickness Θ adjacent to the perimeter of the specimen is not available for compression and therefore not being tested. The following expression is used for the scale correction factor:

$$f1 = \frac{(D - \theta) \cdot (H - \theta)}{D \cdot H}$$

and:

$$\theta = \left[\frac{10 \cdot e_v (D30)^{\frac{13}{2}} \cdot (D60)^{\frac{13}{2}} \cdot \ln\left[\frac{(D60)}{(D30)}\right]}{\left(D60^{10} - D30^{10}\right)}\right]^{\frac{1}{3}}$$

where:

e_v = specimen placement void ratio calculated using the dry bulk specific gravity,
D30 = the particle diameters corresponding to 30 percent finer on the cumulative particle size distribution curve, and
D60 = the particle diameters corresponding to 60 percent finer on the cumulative particle size distribution curve.

Quality Control

The use of calibrated equipment is essential. Additionally, precision can be measured by periodically performing duplicate test. Accuracy can be measured by periodically testing a standard gravel used for just this purpose.

REFERENCES

BOR (1990) Earth Manual, A Water and Power Resources Technical Publication, Bureau of Reclamation, Denver, CO, Part 2, Third Edition, 1990.

BOR (1974) Earth Manual, A Water and Power Resources Technical Publication, Bureau of Reclamation, Denver, CO, Second Edition, 1974.

Appendix B: *Procedure for Determining the Constrained Modulus of Clean Gravel and Crushed Rock*

STANDARD TEST METHOD FOR DETERMINING CONSTRAINED MODULUS (M_s) OF CLEAN GRAVEL AND CRUSHED ROCK

1. Scope

1.1 This test method covers a procedure for determining the Constrained Modulus (M_s) of clean gravel and crushed rock. These soils are typically comprised of 100 percent passing a 1½ inch sieve and have a maximum of 10 percent passing the No. 4 sieve. Clean gravel and crushed rock have less than 10% passing the No. 200 sieve and are considered to be cohesionless, free-draining soils. Specimens having maximum particle size 1½ inch are placed in a cylindrical specimen container and axially loaded. During loading, lateral displacement is prevented by containment, and axial displacement is measured. M_s is calculated as the ratio of the applied vertical stress to the measured axial strain.

1.2 All observed and calculated values shall conform to the guidelines for significant digits and rounding established in ASTM Practice D 6026.

1.3 The method used to specify how data are collected, or recorded in this standard is not directly related to the accuracy with which the data can be applied in design or other uses, or both. How one applies the results obtained using this standard is beyond its scope.

1.4 This standard does not purport to address all of the safety concerns associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability if regulatory limitations prior to use.

2. Referenced Documents

2.1 ASTM Standards:

C 136 Test Method for Sieve Analysis of Fine and Coarse Aggregates

C 702 Practice for Reducing Samples of Aggregates to Testing Size

D 75 Practice for Sampling Aggregates

D 421 Practices for Dry Preparation of Soil Samples for Particle Size Analysis

D 4253 Test Method for Maximum Index Density and Unit Weight of Soils and Calculation of Relative Density

D 4254 Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density

D 653 Terminology for Soil Rock and Contained Fluids

D 854 Test Method for Specific Gravity of Soils

D 4753 Guide for Evaluating, Selecting, and Specifying Balances and Scales for Use in Soil, Rock, and Related Construction Materials Testing

D 6026 Practice for Using Significant Digits in Geotechnical Data

E 11 Specification for Wire Cloth and Sieves for Testing Purposes

3. Terminology

3.1 For Terminology used in this test method, refer to Terminology D 653.

3.2 Definitions of Terms Specific to this Standard:

3.2.1 Constrained Modulus, M_s – the ratio of stress to strain for a material under axial load and restrained laterally. The constrained modulus is numerically equal to the slope of a secant of a stress-strain curve.

3.2.2 Crushed rock – quarried rock, boulders, or cobbles that have been mechanically fragmented and then graded for use in construction.

3.2.2 Maximum index density - the reference dry density of a soil in the densest state of compactness that can be attained using a standard laboratory compaction procedure that minimizes particle segregation and breakdown

3.2.3 Minimum index density - the reference dry density of a soil in the loosest state of compactness at which it can be placed using a standard laboratory procedure which prevents bulking and minimizes particle segregation.

3.2.4 Relative density, D_d - the ratio, expressed as a percentage, of the difference between the maximum index void ratio and any given void ratio of a cohesionless, free-draining soil; to the difference between its maximum and minimum index void ratios. The equation is as follows:

$$D_d = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100$$

Or, in terms of corresponding dry densities

$$D_d = \frac{\rho_{d\max}(\rho - \rho_{d\min})}{\rho_d(\rho_{d\max} - \rho_{d\min})} \times 100$$

4. Summary of Test Method

4.1 A 14.5 inch-thick specimen is placed in a vertically positioned Teflon™ lined steel or aluminum pipe having a 17.25 inch nominal i.d. (inside diameter). The specimen and pipe is contained in a 19 inch nominal i.d. aluminum or steel test apparatus. Steel or aluminum platens on the top and bottom of the sample are used to apply a series of increasingly greater axial loads to the specimen creating axial stresses from approximately 1 lb/in^2 to a maximum of 150 lb/in^2. Axial deformation is measured following the application of each incremental pressure and cessation of axial displacement. M_s is calculated for each incremental pressure.

5. Significance and Use

5.1 Understanding the compression characteristics of clean gravels and crushed rock may significantly improve estimates of volume change in various field applications.

5.2 Clean gravels and crushed rock are preferred embedment material for buried pipe. The constrained modulus values for these materials are used in prediction of buried flexible pipe deflection and buckling. For deep installations, site-specific values of constrained modulus will be helpful to define the limits of the burial depth.

5.3 Establishing the M_s of clean gravel and crushed rock will enable determination of its suitability for some engineering purposes.

5.4 Test specimens may be placed at various densities to evaluate different construction placement conditions.

6. Apparatus

6.1 General Equipment:

6.1.1 Balances or Scales.- For determining the moisture content, a balance or scale having a minimum capacity of 1000 g and meeting the requirements of Specification D 4753 for a balance for 0.1 g readability. For the in-place density determination and development of the calibration equation, the balances or scales used must conform to the requirements and principles of Specification D 4753.

6.1.2 Moisture-proof Containers.- Large impenetrable bags or containers for storing material prior to testing.

6.1.3 Mixing Pans.- Metal mixing pans, approximately 3 by 2 feet by 4 inches deep.

6.1.4 Shovel.- D-handle, No. 4 scoop shovel (or equivalent).

6.1.5 Towels.- Large, cotton, bath towels for surface drying the apparatus and equipment.

6.1.6 Sieve.- U.S.A. Standard Series #4, 3/8-inch, ½-inch, ¾-inch, 1-inch, and 1½-inch sieves (with stands), conforming to requirements of Specification E11.

6.1.7 Yardstick.- A 36 inch yardstick having at least l/16 inch markings.

6.1.8 Tape.- A pocket tape graduated in inches.

6.1.9 Load Cell. – Minimum 1000 lb capacity.

6.1.10 Large Caliper.- Minimum length equal or greater to diameter of test cylinder; used to measure diameter of test cylinder.

6.2 Equipment Unique to This Procedure (see Figure 1):

6.2.1 Dial Indicator Reference Bracket.- Dial indicator reference bracket for 17.25 inch i.d. test apparatus.

6.2.2 Test chamber.- A 19 inch nominal i.d. steel or aluminum cylinder, a minimum of 16 inches deep able to completely encompass the test cylinder; must have a removable plug so that water can be contained/drained as required.

6.2.3 Load Platen.- Steel or aluminum plate, 17 inches diameter, ¾ inch minimum thickness, two required.

6.2.5 Test Cylinder.- A specimen thickness cylinder of approximately 17 inch inside diameter by 15.000+/-0.001 inches high, made from metal pipe.

6.2.6 Dial Indicator.- Dial indicator with holders, continuous dial, 0.00l inch graduations, 3.000 inch travel, two required.

6.2.7 Reaction Plate.- A steel or aluminum plate having six 13/16 inch diameter holes drilled for tension rods, three required.

6.2.8 Tension Rods.- Steel rods, 3/4 inch diameter by 45 inches long; top 18 inches threaded and bottom l 3/4 inches threaded, six required.

6.2.9 Loading System.- Hydraulic handheld jack with minimum 20 ton capacity.

6.2.10 Loading Spring.- Coiled steel loading spring meeting the following designation requirements (four required):

Coil o.d. +/- 1/4 in in	Free height +/- 1/4 in in	Static load at deflections of 2-1/4 in +/- 1/4 in lbf
6	12	15,000

6.2.11 Liner.- A Teflon™ liner having a coefficient of friction of approximately 0.2 and approximately 0.0003 inch thickness; applied to interior of test cylinder[1].

6.2.12 Hoist.- 1,500 lbf capacity hoist, for lifting load plates, reaction plates, etc., with yoke for lifting larger items.

6.2.13 Ball Bearing. – A 2 inch diameter steel ball bearing.

7. Precautions

7.1 Safety Precautions:

7.1.1 Care should be taken, when applying loads on the specimen, not to over stress the tension rods.

7.1.2 The entire threaded length of the tension rods must be screwed into the baseplate of the test chamber. Gloves should be worn to avoid cutting hands on the threads.

7.1.3 Safety glasses shall be worn from beginning to end of compaction phase. Safety boots shall be worn from start to end of testing process.

7.1.4 At least two testing personnel are required when assembling test apparatus.

7.1.5 Review safety and usage guidelines for all equipment used, especially hoist and hydraulic jack. Keep hands and arms away from hydraulic jack while in use

7.2 Technical Precautions:

7.2.1 Due to the effects of sampling, handling, processing, and testing on some materials, results of this test may not reflect the properties of the material after processing and placement during construction.

8. Sampling

8.1 The field sample should be representative of the source material. Since the soil to be tested is typically a clean gravel or crushed rock, the sampling procedures should follow the instructions in Practice D 75.

8.2 Each test specimen requires about 200 lbs. The field sample should be reduced to this amount by the procedures described in Practice C 702. If the material is uniformly graded, the field sample should be about twice the required mass. If segregation of the material is possible, then the field sample should be about four times the required mass.

8.3 If a suite of tests is required to evaluate the constrained modulus at various densities, then about four times the amount of material from 8.2 would be required.

9. Test Specimens

9.1 Process a test sample of approximately 200 lbm of material in accordance with Practice D 421.

9.2 Remove any particles larger than 1½ in by sieving the sample on a 1½ in sieve.

9.3 If desired, determine the maximum and minimum index density, in accordance with Test Methods D 4253 and D 4254, respectively.

[1] It is recommended that the Teflon™ liner be applied through a plating or equivalent technique by a surface finishing professional.

9.4 Air-dry the sample. Mix thoroughly to prevent segregation.

9.5 Determine the particle size distribution in accordance with Test Method C 136. Re-combine and thoroughly mix the material for use in the compression test.

9.6 Placement process (such as hand placement, hand compaction, vibration, fluviation, etc.) may be used to obtain a desired placement density. Record methods used, including lift heights and equipment, and include in report.

10. Preparation of Apparatus

10.1 Refer to Figure 1 to correctly assemble the fixed base section of the testing apparatus.

10.2 The placement of the test chamber, shims, and mobile upper section of the test apparatus are described in Section 14.

10.3 The interior of the test cylinder must be coated with Teflon™ in order to minimize the frictional effects on the sample specimen. Refer to 6.2.11 for Teflon™ liner specifics.

11. Calibration and Standardization

11.1 The following calibrations of test apparatus should be performed before initial use and at intervals not exceeding each 1000 tests, or annually, whichever occurs first.

11.1.1 Determine the average diameter of the test cylinder by measuring at six locations (including three at each end) and record the measurements to the nearest 0.01 inch.

11.1.2 Calculate the area of the test cylinder using the average diameter and record to the nearest 0.1 in^2.

11.1.3 Determine the average test cylinder height by measuring at six equidistant locations on the perimeter of the test cylinder and record to the nearest 0.01 inch.

11.1.4 Calculate the volume of the test cylinder using the area and height values obtained in the previous steps. Record to the nearest 0.1 in^3.

11.2 Load cells must have an NIST traceable calibration certificate.

11.3 Verify that equipment is clean and in good operating condition. If the calibrations are not current, perform the calibration before using the equipment for this procedure. Refer to equipment specific instruction manuals. Verify that Teflon™ liner is in good condition.

12. Conditioning

12.1 After the material is air-dried, place the material in a moisture-proof container for storage.

13. Procedure

13.1 Place the first loading platen in the center of the base of the test chamber.

13.2 Position three shims at equal intervals around the test cylinder and lower the test cylinder into the test chamber.

13.2.1 Place the test cylinder over the platen such that it is supported on the shims. The shims should be in a position that will allow them to be removed once the specimen has been prepared for testing (prior to the start of testing).

13.3 Split the sample into equal increments by repeated use of a quartering procedure such that each increment can be entirely contained in a scoop. Determine and record the mass of each increment. Place each increment one at a time into the test cylinder. Place the soil in the cylinder using the process that yields desired density.

13.4 When test cylinder is completely filled, place the second load platen (with attached dial indicator reference brackets and dial indicators) on top of the sample, centered in the test cylinder.

13.4.1 The dial indicator reference brackets holding the dial indicators are attached to this loading platen, and should be positioned so that the dial indicators on either side of the test chamber contact the fixed, flat arms protruding from the outside of the testing chamber.

13.5 Record the dial gauge readings from both dial indicators. This reading represents the initial specimen height of 15 inches.

13.5.1 Once this step is complete, care should be taken not to affect the zero setting on the dial gauges.

13.5.2 The difference between these readings and the test cylinder height readings will be used to determine the initial height of the test specimen.

13.6 Pour tap water into the tubing connected to the bottom of the test chamber until the sample is completely saturated; then seal the chamber.

13.7 Place the four loading springs on top of the loading platen, as shown in Figure 1.

NOTE - The size of load springs (see subparagraph 6.2.10) selected depends on the load required to provide the required pressure on the specimen. For example, if the soil represented by the test specimen is to be placed at a certain depth, the applied pressure should be equivalent to the estimated weight of overlying soil. The mass of the load plate with attached porous disk, four springs, and bottom reaction plate will apply about 1 lbf/in^2 pressure on the specimen.

13.7.1 Record the readings on dial gauges A and B as "Springs Loaded Reading."

13.8 Complete assembly of mobile upper portion of testing apparatus, as shown in Figure 1.

13.8.1 Thread tension rods into bottom reaction plate (with nuts finger tight)

13.8.2 Place the load cell in the center of bottom reaction plate, on top of the 2 inch ball bearing.

13.8.3 Position middle reaction plate on top of load cell.

13.8.4 Place the hydraulic jack on top of the middle reaction plate, in the center.

13.8.5 Place the top reaction plate on the hydraulic jack.

13.8.6 Use nuts as needed to maintain stability and level position.

13.9 Use the hoist to lower the entire mobile upper portion of testing apparatus onto the lower stable portion of testing apparatus.

13.10 The soil specimen will be loaded according to the calculated loads.

13.11 Using the hydraulic jack, increase the pressure on the load cell until the correct initial pressure (1 lb/in^2) is achieved.

13.12 Read dials A and B and record corresponding readings.

13.13 Repeat for additional remaining pressures (approximately 5, 10, 20, 40, 60, 80, 100 lb/in^2) in increments, and record the readings of dial gauges A and B for each.

13.14 Slowly remove the applied pressure, and return to the initial loading setting.

13.15 Unthread the tension rods and remove the upper mobile portion of the apparatus. Then remove the springs and loading platen with dial indicators attached. Drain the water.

13.16 Remove the entire specimen and thoroughly clean test cylinder and test chamber.

13.17 After drying, determine and record the mass of the entire specimen that was tested.

13.18 Adjust all Dial Gage A entries to account for the Dial Gage A Initial Reading by subtracting this value from each reading recorded, and recording. Repeat for Dial Gage B entries and record. Average these two values and record.

13.19 Determine the particle size distribution of the test specimen. Determine the particle breakdown by comparing the percent passing the sieve that retained all particles before the test. The percent passing is the particle breakdown.

14. Calculations[2]

14.1 *Specimen Area (A)* - The specimen area is calculated from the average of a minimum of six inside diameter measurements (D) of the test cylinder by the equation:

$$A = \pi \frac{D^2}{4}$$

14.2 *Specimen Volume (V)* - The specimen volume is calculated from multiplying the test specimen area (A) by the average of a minimum of six height measurements (H) of the test cylinder; expressed by the equation:

$$V = A \cdot H$$

14.3 *Applied Axial Load (P*)* – The applied axial load is the initial weight of the loading platen (W1), springs (W2), sum of reaction plates (W3), load cell (W4), and hydraulic jack (W5) plus the additional load applied by the jack as measured by the load cell (W6).

$$P_* = W1 + W2 + W3 + W4 + W5 + W6$$

14.4 *Seating load (P)* – The seating load is the initial weight of the loading platen (W1),

[2] English units used in all calculations. Length units are inches and force units are pounds.

springs (W2), sum of reaction plates (W3), load cell (W4), and hydraulic jack (W5).

$$P = W1 + W2 + W3 + W4 + W5$$

14.5 *Axial Stress (σ)* – The axial stress is calculated as the ratio of the applied axial load (P_*) to the specimen area (A).

$$\sigma = \frac{P_*}{A}$$

14.6 *Axial Strain (ε)* – The axial strain is calculated as the ratio of the ratio of the average of two axial displacement measurements (d) to the initial test specimen height (H). Zero displacement is the condition immediately following seating load application.

$$\varepsilon = \frac{d}{H}$$

14.7 *Constrained Modulus (M_s)* – The constrained modulus is calculated as the ratio of the change in axial stress (Δσ) to the change in axial strain (Δε) for a selected increment of stress or strain, corrected for the effects of limited specimen size [scale effect (f1)] and friction between the crushed rock and container [friction effect (f2)]. The constrained modulus must always be reported with the increment of stress or strain it represents.

$$Ms = \frac{\Delta\sigma}{\Delta\varepsilon}\cdot(f1)\cdot(f2)$$

14.8 *Scale Effect Factor (f1)* – The scale effect factor accounts for stiffness added due to the limited size of the test specimen and is calculated as follows[3]:

$$f1 = \frac{(D-\theta)\cdot(H-\theta)}{D\cdot H}$$

where:

θ = the average void diameter and is approximated by:

$$\theta = \left[\frac{10\cdot e_v (D30)^{\frac{13}{2}}\cdot(D60)^{\frac{13}{2}}\cdot\ln\left[\frac{(D60)}{(D30)}\right]}{\left(D60^{10} - D30^{10}\right)}\right]^{\frac{1}{3}}$$

and where:

e_v = specimen placement void ratio calculated using the dry bulk specific gravity,
D30 = the particle diameters corresponding to 30 percent finer on the cumulative particle size distribution curve, and
D60 = the particle diameters corresponding to 60 percent finer on the cumulative particle size distribution curve.

14.9 *Friction Effect Factor (f2)* – The friction effect factor accounts for loss of applied load within the test specimen due to frictional resistance between the specimen and container walls and is calculated as follows[3]:

$$f2 = \frac{D\cdot\left(1 - e^{-\frac{k\cdot\mu\cdot H}{D}}\right)}{k\cdot\mu\cdot H}$$

where:
e = 2.718
k = 0.3 (represents the ratio of radial to axial stress within the test specimen)
u = 0.2 (represents the coefficient of friction between the Teflon™ coated specimen container and crushed rock)

15. Report

15.1 Report the following information about the material tested:

15.1.1 Source, sample number, local or generic name, lithologic description

15.1.2 Description and classification

15.1.3 Minimum and maximum index densities, if performed

15.1.4 Particle size analysis

15.1.5 Any other testing performed (eg specific gravity, etc)

[3] Refer to Constrained Modulus of Crushed Rock, September 15, 2009 Appendix A – Procedure Development for development details

15.1.6 How field sample obtained

15.1.7 Percent and size of any oversized material removed before testing

15.2 Report the following information about the test:

15.2.1 Dates and personnel

15.2.2 How material placed in container

15.2.3 Density of test specimen

15.2.4 Seating load and load increments and time each load is held.

15.2.5 Wet or dry condition

15.2.6 Percent of particle breakdown and description of any acoustic emission during test

15.2.7 Constrained modulus for each load increment

15.2.8 Any difficulties or anomalies during testing

15.3 Report the following information about the test equipment:

15.3.1 Calibration dates

15.3.2 Make, model, and serial number

16. Precision and Bias

16.1 Precision **–** Test data on precision is not presented due to the nature of the material being tested. It is either not feasible or too costly at this time to have ten or more laboratories participate in a round-robin testing program.

16.1.1 The subcommittee is seeking any data from the users of this test method that might be used to make a limited statement on precision.

16.2 Bias **–** The procedure in this test method for measuring unit weight has no bias because the value for constrained modulus can be defined only in terms of a test method

17. Keywords

17.1 Compression test, constrained modulus, crushed rock, density, gravel, pipe, embedment, soil compaction, unit weight

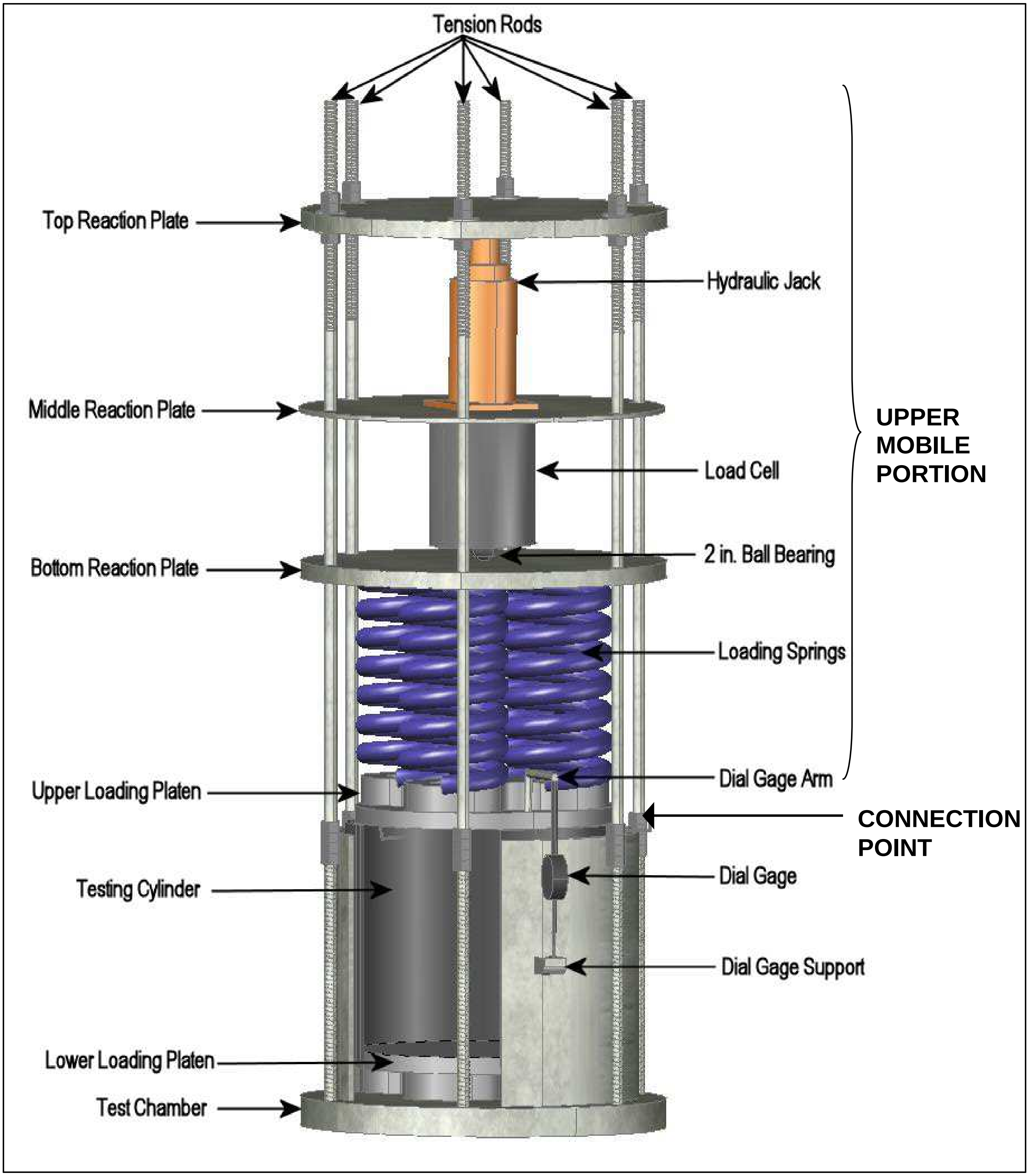


Figure 1. Constrained Modulus of Crushed Rock Testing Apparatus

Appendix C: *Constrained Modulus of Crushed Rock Test Results*

APPENDIX C. TEST RESULTS - CONSTRAINED MODULUS OF CRUSHED ROCK

INTRODUCTION

This appendix presents the results of a series of one-dimensional compression tests performed on the type of crushed rock typically used for buried pipe bedding material; these tests were performed for the purpose of determining constrained modulus of the material. The tests were completed on a series of five clean crushed rock samples obtained from four locations across the Colorado – Wyoming area; referred to as MS-1, MS-2, MS-3, MS-4, and MS-5. Standard property tests were performed on all samples and include: particle size analysis (gradation), specific gravity, angle of repose, minimum and maximum index densities, particle shape, and LA abrasion tests. All samples were classified using the Unified Soil Classification System (USCS). For each sample, a series of at least four one-dimensional compression tests was performed at varying placement densities in order to obtain a representative range of data. A procedure was developed for performing one dimensional compression tests on crushed rocks. The procedure development is discussed in Appendix A, and the procedure is presented in Appendix B. All standard property tests were performed in accordance with either America Society for Testing Materials (ASTM) or Bureau of Reclamation (BOR)[1] procedures. Complete details of all tests are discussed below. Standard property data is discussed first, and then constrained modulus test results are discussed. Test specimen physical properties are summarized in Table C1. Constrained modulus test data are summarized in Table C2 - M_s Test Results Summary Table and detailed in Attachment C.1 – Constrained Modulus Test Data Summary.

CRUSHED ROCK PROPERTIES

Source location, source name, geologic description and a summary of physical properties are included in the following section. These include particle size distribution (for each material, before and after compression testing), specific gravity, minimum and maximum index densities, shape, LA abrasion, and angle of repose. These values are summarized in Table C1. The grain size distributions for the five samples are presented on Figure C2. The following procedures were implemented to determine the test specimen properties:

- Particle Size Analysis (ASTM D422)
- Unified Soil Classification (ASTM D5878)
- Specific Gravity Test (USBR 5320)
- Relative Density Test (ASTM 4253, ASTM 4254)
- Particle Shape Determination (ASTM D 2488 and ASTM 4791)
- Angle of Repose Test (USBR 5380)
- LA Abrasion Test (ASTM 535 Grade B)

Unless otherwise noted, all testing procedures were performed in MCG Geotechnical Engineering Laboratory in Morrison, Colorado.

[1] Bureau of Reclamation procedures are prefixed by "USBR," however the agency's official acronym is BOR.

The tests were performed on crushed rock samples obtained from four locations across the Colorado – Wyoming area (See Figure C2). Sample MS-1 is a ½ -inch, blue-gray, mofic monzonite, from the Ralston Quarry, Golden Colorado. Sample MS-2 is a ¾ -inch gray Precambrian granite schist (with some biotite and pyrite) from the Morrison Quarry, Morrison Colorado. Sample MS-3 is from the same source as MS-2, but has 1 ½ -inch maximum particle size. Sample MS-4 is a ¾ -inch light-gray limestone from the Ingleside formation containing 92% $CaCO_3$, and was obtained from a quarry in Larimer County Colorado. Sample MS-5 is a ¾ -inch red quartzite from Hartville Uplift, Guernsey Wyoming.

Laboratory classification and gradations tests were performed, and all samples classified using the Unified Soil Classification system. All five crushed rock samples are classified as poorly graded gravel (GP), in accordance with ASTM D 2487.

The specific gravity was determined in accordance with ASTM D 854 for each sample. Apparent specific gravity, bulk specific gravity (SSD), bulk specific gravity (oven-dry), and percent absorption were calculated. Apparent specific gravity values ranged from 2.53 to 2.77, which represent typical specific gravity values for crushed rock.

The shapes of grab samples of each sample were measured in accordance with the provisions of ASTM D 2488 and ASTM 4791. The length, width, and thickness of each gravel particle in a representative sample are measured, implementing visualization a rectangular box around the specimen. The length is the largest dimension, the thickness is the smallest dimension, and the width is the intermediate dimension of the rectangular box. A particle with a width/thickness ratio of 3 or more is classified as “flat,” and a particle with a length/width ratio of 3 or more is an “elongate.” About 33 specimens of each sample were measured with a caliper. No flats were found in any of the samples, and two elongates were found in MS-1. Width to thickness ratio ranged from 1.2 to 1.6, and width to length ratios ranged from 1.4 to 1.7.

The minimum and maximum index densities were determined, and LA abrasion tests were performed on each sample, in accordance with ASTM D 4253, D 4254, and C 535 respectively. Terracon Consultants, Inc. in Fort Collins, Colorado performed these tests. Minimum densities ranged between 76.9 lb/ft^3 and 86.8 lb/ft^3; maximum densities ranged between 89.2 lb/ft^3 and 117.4 lb/ft^3. LA abrasion percent wear ranged between 13 percent and 30 percent.

Angle of Repose tests were performed on each crushed rock sample. These tests were performed in accordance with USBR 5380 and ranged in value between 37 degrees and 45 degrees.

The shapes of grab samples of each MS material were measured in accordance with the provisions of ASTM D 2488 and ASTM 4791. These ASTM standards provide for measuring shapes of gravel particles to see what percentage of “flats” and “elongates” are present. A significant portion of flats and/or elongate particles in a soil affects concrete mix design, concrete placement, and soil compaction. The length, width, and thickness of each gravel particle are measured visualizing a rectangular box around the specimen. The length is the largest dimension, the thickness is the smallest dimension, and the width is the intermediate dimension of the rectangular box. A particle with a width/thickness ratio of 3 or more is a “flat” and a particle with a length/width ratio of 3 or more is an “elongate.” About 33 specimens of each sample were measured with a caliper. No flats were found in any of the samples and two elongates were found in MS-1. The following table shows the average values for each sample:

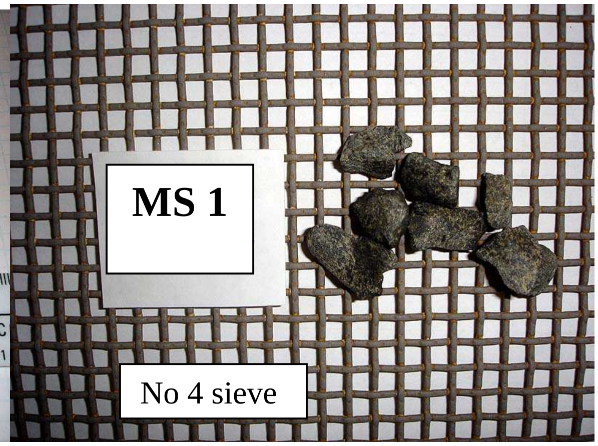
Crushed Rock	W/T	L/W	No. of Specimens with W/T or L/W greater than 2.0
MS-1	1.4	1.6	6
MS-2	1.2	1.4	5
MS-3	1.4	1.6	11
MS-4	1.5	1.6	8
MS-5	1.6	1.7	11

A W/T ratio and a L/W ratio of 1.0 would be indicative of an equi-dimensional particle. The measured ratios do not show a significant difference between samples as to shape.

The physical characteristics of five crushed rock samples, MS-1 through MS-5, were evaluated and found to have wide ranging properties. Tests on MS-1 and MS-5 yielded: (1) the extreme high and low values respectively for bulk specific gravity and maximum unit weight and (2) the extreme low and high values respectively for percentage wear in the LA abrasion test. MS-1 and MS-3 represented: (1) the extreme in particle sizes with MS-1 having the lowest maximum particle size and MS-3 having the largest maximum particle size and (2) the extreme in angle of repose measurements with MS-1 having the highest value and MS-3 having the lowest value. MS-2 and MS-4 properties are intermediate between extremes for maximum particle size, percentage wear in the LA abrasion test, bulk specific gravity, maximum unit weight, and angle of repose.

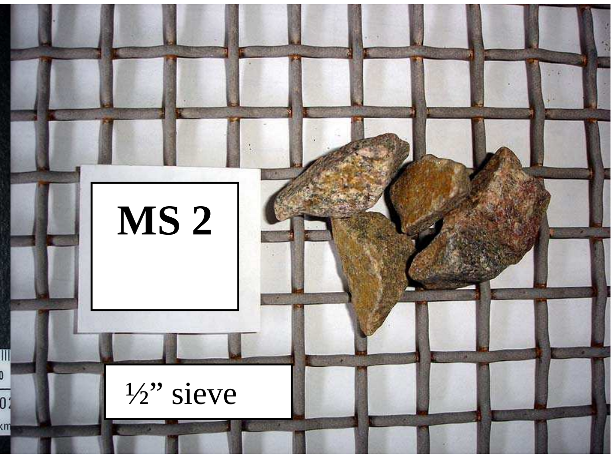
The following presents a summary of information pertinent to each sample. Additional photographs of samples are presented on Figures C.3 through C.7.

CRUSHED ROCK PROPERTIES
MS-1
½-inch granite



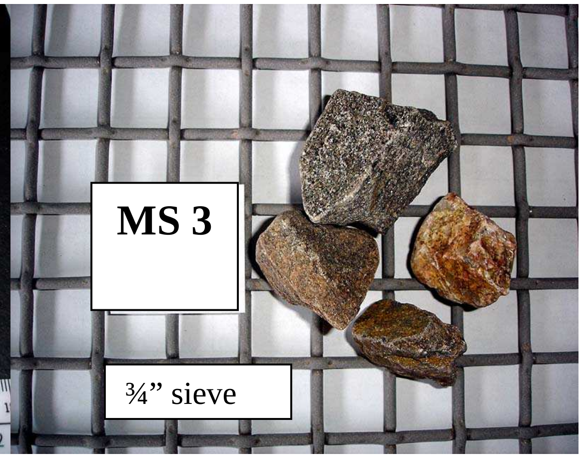
Test name:	½-inch granite	
Purchased from:	Santa Fe Sand and Gravel, Littleton CO	
Purchase name:	½” TT blue-gray	
Source:	Ralston Quarry Golden CO	
	Asphalt Paving Co Golden CO	
Geologic description:	Tertiary age hard volcanic rock	
USCS Classification	“GP” Poorly Graded Gravel	
Gradation before test:	100% passing ½-inch sieve	
	76% passing 7/16-inch sieve	
	34% passing 3/8-inch sieves	
	10% passing 5/16-inch sieve	
	0% passing No. 4 sieve	
Gradation after test:	100% passing ½-inch sieve	
	5% passing No. 4 sieve	
Index Densities:	Minimum = 86.8 lb/ft^3	ASTM D 4254
	Maximum = 117.4 lb/ft^3	ASTM D 4253
LA Abrasion:	13%	ASTM C 535
Angle of Repose:	45 degrees	USBR 5380
Specific Gravity (S.G.):		
Apparent S.G.:	2.77	
Bulk S.G. (SSD):	2.72	
Bulk S.G. (ovendry):	2.69	

CRUSHED ROCK PROPERTIES
MS-2
3/4-inch granite



Test name:	3/4-inch granite	
Purchased from:	Santa Fe Sand and Gravel, Littleton CO	
Purchase name:	¾” granite	
Source:	Morrison Quarry, Morrison CO	
	Aggregate Industries, Inc., Morrison CO	
Geologic description:	Precambrian granite gneiss with biotite shear planes	
USCS Classification	“GP” Poorly Graded Gravel	
Gradation before test:	100% passing 7/8-inch sieve	
	86% passing ¾-inch sieve	
	13% passing ½-inch sieve	
	0% passing 3/8-inch sieve	
Gradation after test:	100% passing 7/8-inch sieve	
	4% passing No. 4 sieve	
Index Densities:	Minimum = 87.8 lb/ft^3	ASTM D 4254
	Maximum = 104.6 lb/ft^3	ASTM D 4253
LA Abrasion:	27%	ASTM C 535
Angle of Repose:	39 degrees	USBR 5380
Specific Gravity (S.G.):		
Apparent S.G.:	2.67	
Bulk S.G. (SSD):	2.64	
Bulk S.G. (ovendry):	2.62	

CRUSHED ROCK PROPERTIES
MS-3
1-½-inch granite



Test name:	1-1/2-inch granite	
Purchased from:	Santa Fe Sand and Gravel, Littleton CO	
Purchase name:	¾” granite	
Source:	Morrison Quarry, Morrison CO	
	Aggregate Industries, Inc., Morrison CO	
Geologic description:	Precambrian granite gneiss with biotite shear planes	
USCS Classification	“GP” Poorly Graded Gravel	
Gradation before test:	100% passing 1-½-inch sieve	
	32% passing 1-inch sieve	
	20% passing 7/8-inch sieve	
	0% passing ¾-inch sieve	
Gradation after test:	100% passing 1-½-inch sieve	
	8% passing No. 4 sieve	
Index Densities:	Minimum = 85.0 lb/ft^3	ASTM D 4254
	Maximum = 98.3 lb/ft^3	ASTM D 4253
LA Abrasion:	30%	ASTM C 535
Angle of Repose:	37 degrees	USBR 5380
Specific Gravity (S.G.):		
Apparent S.G.:	2.61	
Bulk S.G. (SSD):	2.58	
Bulk S.G. (ovendry):	2.57	

CRUSHED ROCK PROPERTIES
MS-4
¾-inch limestone

Test name:	¾-inch limestone	
Purchased from:	Pete Lien, Ft Collins CO	
Purchase name:	¾-inch gray	
Source:	Rex Quarry, Livermore CO	
	Pete Lien & Sons, Ft Collins CO	
Geologic description:	limestone from Ingleside formation, 98% calcium carbonate	
USCS Classification	“GP” Poorly Graded Gravel	
Gradation before test:	100% passing 7/8-inch sieve	
	81% passing ¾-inch sieve	
	0% passing ½-inch sieve	
Gradation after test:	100% passing 7/8-inch sieve	
	6% passing No. 4 sieve	
Index Densities::	Minimum = 86.8 lb/ft^3	ASTM D 4254
	Maximum = 103.6 lb/ft^3	ASTM D 4253
LA Abrasion:	28%	ASTM C 535
Angle of Repose:	40 degrees	USBR 5380
Specific Gravity (S.G.):		
Apparent S.G.:	2.65	
Bulk S.G. (SSD):	2.62	
Bulk S.G. (ovendry):	2.61	

CRUSHED ROCK PROPERTIES
MS-5
¾-inch dolomite

Test name:	¾-inch quartzite	
Purchased from:	Santa Fe Sand & Gravel, Littleton CO	
Purchase name:	¾” Wyoming Red	
Source:	Guernsey Quarry, Cemex, Guernsey WY	
Geologic description:	quartzite	
USCS Classification	“GP” Poorly Graded Gravel	
Gradation before test:	100% passing 7/8-inch sieve	
	55% passing ¾-inch sieve	
	0% passing ½-inch sieve	
Gradation after test:	100% passing 7/8-inch sieve	
	13% passing No. 4 sieve	
Relative Density:	Minimum = 76.9 lb/ft^3	ASTM D 4254
	Maximum = 89.2 lb/ft^3	ASTM D 4253
LA Abrasion:	25%	ASTM C 535
Angle of Repose:	40 degrees	USBR 5380
Specific Gravity (S.G.):		
Apparent S.G.:	2.53	
Bulk S.G. (SSD):	2.40	
Bulk S.G. (ovendry):	2.31	

CONSTRAINED MODULUS TESTS

One-dimensional compression tests were performed on several test specimens representing each crushed rock sample, and constrained modulus was calculated for two different stress ranges. Tests were performed in accordance with the test procedure presented in Appendix B – Constrained Modulus of Crushed Rock and Gravel Procedure. This procedure involves axially loading crushed rock specimens (approximately 17.3 inch diameter by 13.6 inch high) in a Teflon lined, steel test cylinder while measuring axial displacement. The steel test cylinder prevents radial displacement, i.e. provides lateral constraint. Constrained modulus is calculated as the ratio of the applied axial stress to the measured axial strain (corrected for friction and scale effect, as discussed in the next paragraph) for the stress ranges 2.3 lb/in^2 to 87.5 lb/in^2 and 2.3 lb/in^2 to 151.5 lb/in^2. Calculated values for each crushed rock specimen are summarized in Table C2. Data sheets and plots of stress verses strain and void ratio verses stress for each specimen are provided on summary plots contained in Attachment C1 – Constrained Modulus Test Data.

Scale effect and friction effect factors were applied when calculating constrained modulus values. The development of these factors and their application are presented in Appendix A – Constrained Modulus Procedure Development. The scale effect factor (f1) accounts for stiffness added to the test specimen due to the limited size of the test specimen, and depends on the particle size distribution of the sample tested. Values range from 0.96 to 0.88, depending on the particle size distribution of the sample tested. The friction effect factor (f2) accounts for the loss of axial load within the test specimen due to frictional resistance between the specimen and container walls, and was calculated to be 0.98.

Crushed rock samples were carefully and consistently processed for testing to create 16 buckets of representative material. The process proceeded as follows. Approximately 800 lbs of each sample was purchased and sieved to remove oversized and undersized particles. The sieved material was then placed in 16 five-gallon buckets. These 16 buckets were systematically split and recombined in a process designed to result in 1/16 of the material in each of the original buckets being represented in each of 16 resulting buckets of crushed rock sample. Sometimes, crushed rock used in testing was reused for subsequent tests. However, it was first sieved to remove a small fraction of undersize particles that resulted from particles breaking during previous testing. Crushed rock comprising test specimens was not reused more than once.

Three different specimen placement methods were applied to create test specimens of each sample. These consisted of: 1) placing crushed rock in shallow layers in an attempt to achieve minimum placement density; 2) pouring from a height two feet above the specimen surface while filling the container uniformly and; 3) hand compacting the crushed rock into the specimen container in approximately two inch lifts in an attempt to achieve maximum placement density. Both wet and dry placement moisture conditions were used for different test specimen preparations. One MS-1 specimen was placed wet using a concrete vibrator in an attempt to achieve maximum placement density. No advantage in its use was evident and vibrating to compact specimens was not used again.

Additionally two tests were performed on specimens of Sample MS-5 using a ¼ inch steel plate insert in order to investigate scale effect. These test results are identified on Table C2. These tests consisted of placing a ¼ inch thick steel plate horizontally at the center of the test specimen thereby effectively creating two test specimens, one being the upper half the original test specimen and the other being the lower half of the original test specimen. This, in effect, doubled the ratio of particle size to specimen thickness. The usefulness of these tests for the intended purpose is diminished by the fact that placement densities for the two test specimens having the plates inserted were significantly lower than the placement densities of any other specimens of Sample MS-5 making direct comparison to tests in which the plate was not used impossible. Consequently, a meaningful interpretation of the effect of scale by these tests is not possible.

The dry density and void ratio were calculated for each test for the conditions representing: specimen placement; following seating load application; and maximum load application. Relative density and the specimen moisture condition at placement were also calculated and recorded. Calculated values for each crushed rock specimen are summarized in Table C2.

Tables & Figures

Figure C.1 Quarry Location Map

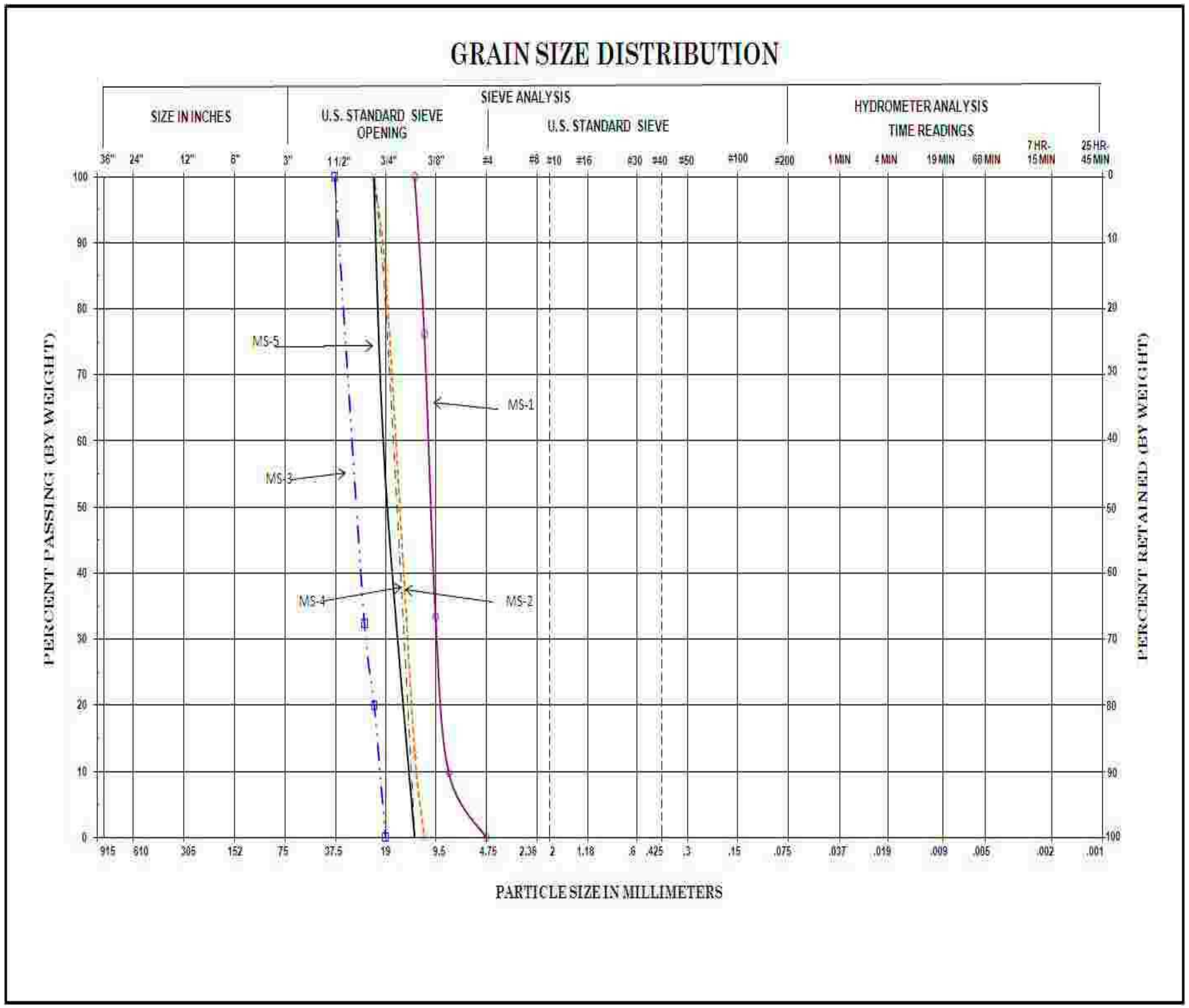


Figure C.2 Grain Size Distribution Graph

Figure C.3 Sample MS-1

Figure C.4 Sample MS-2

Figure C.5 Sample MS-3

Figure C.6 Sample MS-4

Figure C.7 Sample MS-5

Identification			Particle Size Distribution (Percent Passing)									Particle Size Statistics					Specific Gravity (USBR 5320)				Relative Density Test		Angle of Repose (USBR 5380)	LA Abrasion	Shape Data		
Laboratory Sample No. MS-	Type	Unified Soil Classification (ASTM D 2487)	1 1/2 -in.	1 -in.	7/8 -in.	3/4 -in.	1/2 -in.	7/16 -in.	3/8 -in.	5/16 -in.	No. 4	Maximum particle size (mm)	Minimum particle size (mm)	D_{60} (mm)	D_{30} (mm)	D_{10} (mm)	Apparent specific gravity	Bulk specific gravity (SSD)	Bulk specific gravity (oven-dry)	Absorption (%)	Maximum Unit Weight (lb/ft^3) (ASTM D 4253)	Minimum Unit Weight (lb/ft^3) (ASTM D 4254)	Angle of Repose (degrees)	LA Abrasion (% Wear) (ASTM C 535) Grade B	W/T	L/W	No. of Specimens with W/T or L/T greater than 2.0
1	1/2 -in. Granite	GP[A]					100	76.3	33.5	10	0	12.7	4.8	10.5	9.3	8.0	2.77	2.72	2.69	1.10	117.4	86.8	45	13	1.4	1.6	6
2	3/4 -in. Granite	GP[A]			100	86.1	12.5	0				22.2	11.1	16.5	14.0	12.4	2.67	2.64	2.62	0.65	104.6	87.8	39	27	1.2	1.4	5
3	1- 1/2 -in. Granite	GP[A]	100	32	20	0						38.1	19.1	30.0	24.9	20.8	2.61	2.58	2.57	0.71	98.3	85.0	37	30	1.4	1.6	11
4	3/4 -in. Lime-stone	GP[A]			100	80.9	0					22.2	12.7	17.2	14.8	13.4	2.65	2.62	2.61	0.56	103.6	86.8	39	28	1.5	1.6	8
5	3/4 -in Quart-zite	GP[A]			100	55.1	0					22.2	12.7	19.4	15.8	16.3	2.53	2.40	2.31	3.50	89.2	76.9	40	25	1.6	1.7	11

Table C.1 Test Specimen Physical Properties Table

Notes:
[A] = "GP" is the USCS symbol for "Poorly Graded Gravel"

Identification			Placement Conditions					Seating Load Conditions[D]		Final Load Conditions[E]		Effect Factors		Test Results		
Sample ID	Vendor Description	Apparent Specific Gravity	Moisture Condition	Compaction Effort/ Method[B]	Relative Density	Dry Density (lb/ft^3)	Void Ratio	Dry Density (lb/ft^3)	Void Ratio	Dry Density (lb/ft^3)	Void Ratio	Scale Effect Factor f1	Friction Effect Factor f2	Ms[C] (Stress Range: 2.17 - 151.49 lb/in^2)	Ms[C] (Stress Range: 2.17 - 87.5 lb/in^2)	Percent Particle Break-down[F]
MS-1	1/2 in granite	2.77	Dry	Low	14%	90.18	0.917	90.42	0.912	94.96	0.821	0.95	0.98	5059.29	5388.40	5%
			Wet	Low	18%	90.98	0.900	91.21	0.896	94.55	0.829	0.95		3984.76	4230.20	
			Wet	Low	19%	91.35	0.893	91.72	0.885	95.08	0.819	0.95		3968.80	4073.80	
			Dry	Low	23%	92.36	0.872	92.66	0.866	95.18	0.817	0.96		5287.42	5367.10	
			Wet	Low	31%	94.43	0.831	95.25	0.815	98.08	0.763	0.96		4938.51	5221.90	
			Wet	Medium	44%	98.08	0.763	98.36	0.758	100.44	0.721	0.96		6890.44	7493.80	
			Wet	High	69%	105.91	0.633	106.12	0.629	107.66	0.606	0.96		9772.19	9594.60	
			Wet	High	74%	107.65	0.606	108.27	0.597	109.80	0.575	0.96		9811.01	9437.40	
MS-2	3/4 in granite	2.67	Wet	Low	12%	89.48	0.863	89.76	0.857	92.97	0.793	0.93	0.98	3339.23	4051.50	4%
			Dry	Medium	39%	93.39	0.785	93.65	0.780	95.31	0.749	0.93		6949.94	6736.70	
			Wet	Medium	62%	97.08	0.717	97.26	0.714	98.89	0.685	0.93		7171.10	7709.40	
			Wet	High	97%	103.45	0.611	103.63	0.608	105.12	0.585	0.94		8483.64	8255.70	
MS-3	1-1/2 in granite	2.61	Wet	Low	15%	86.61	0.881	86.95	0.874	93.02	0.751	0.88	0.98	1726.85	2215.20	8%
			Wet	Medium	82%	94.91	0.717	95.43	0.707	98.51	0.654	0.88		3515.21	4726.00	
			Dry	High	94%	96.52	0.688	96.76	0.684	98.58	0.653	0.89		6190.41	5910.10	
			Wet	High	101%	97.53	0.670	97.77	0.666	100.03	0.629	0.89		4929.66	6219.80	
MS-4	3/4 in limestone	2.65	Wet	Low	-29%	84.89	0.949	85.25	0.940	89.90	0.840	0.92	0.98	2086.76	2212.70	6%
			Dry	Low	-3%	88.11	0.877	88.30	0.873	91.22	0.813	0.92		3564.15	4277.90	
			Wet	Medium	33%	92.91	0.780	93.19	0.775	96.22	0.719	0.93		3674.63	4274.20	
			Wet	High	64%	97.53	0.696	97.90	0.690	100.04	0.653	0.93		5427.79	7540.80	
MS-5	3/4 in	2.53	Wet, Plate	Low	-47%	72.16	1.189	72.79	1.170	74.94	1.107	0.83	0.98	3835.02	4097.10	13%
			Wet, Plate	Low	-38%	73.03	1.162	74.37	1.123	76.90	1.054	0.83		3332.34	3842.70	
			Wet	Low	-9%	75.93	1.080	76.23	1.072	79.33	0.991	0.92		3011.09	3601.30	
			Wet	Low	9%	77.86	1.028	78.09	1.022	79.99	0.974	0.92		3815.73	4607.10	
			Wet	Medium	38%	81.11	0.947	81.66	0.934	83.45	0.892	0.92		5413.78	6412.60	
			Dry	Medium	61%	83.97	0.881	84.28	0.874	85.92	0.838	0.92		6350.93	6660.10	
			Wet	High	93%	88.27	0.789	88.82	0.778	90.30	0.749	0.93		7306.28	7721.80	

Table C.2 Ms Test Results Summary Table (notes on next page)

Notes for Table C.2

Density of Water at 4^{o} C = 62.42 lb/ft^3

[A] = MS-1 was also evaluated for stress range 2.31 psi to 258.28 psi; this range yielded an Ms value of 5427

[B] = Compaction Effort/Method Terminology: Low=>Hand Placement; Medium=>Fluviated; High=>Hand Tamping

[C] = Both a scale effect factor, f1, and a friction effect factor, f2, have been applied to this value

[D] = Seating Load Conditions = 2.17 lb/in^2, except for MS-1: 14%, 23%, 31%, 44%, 69%, and 74% RD's, where it is 2.31 lb/in^2

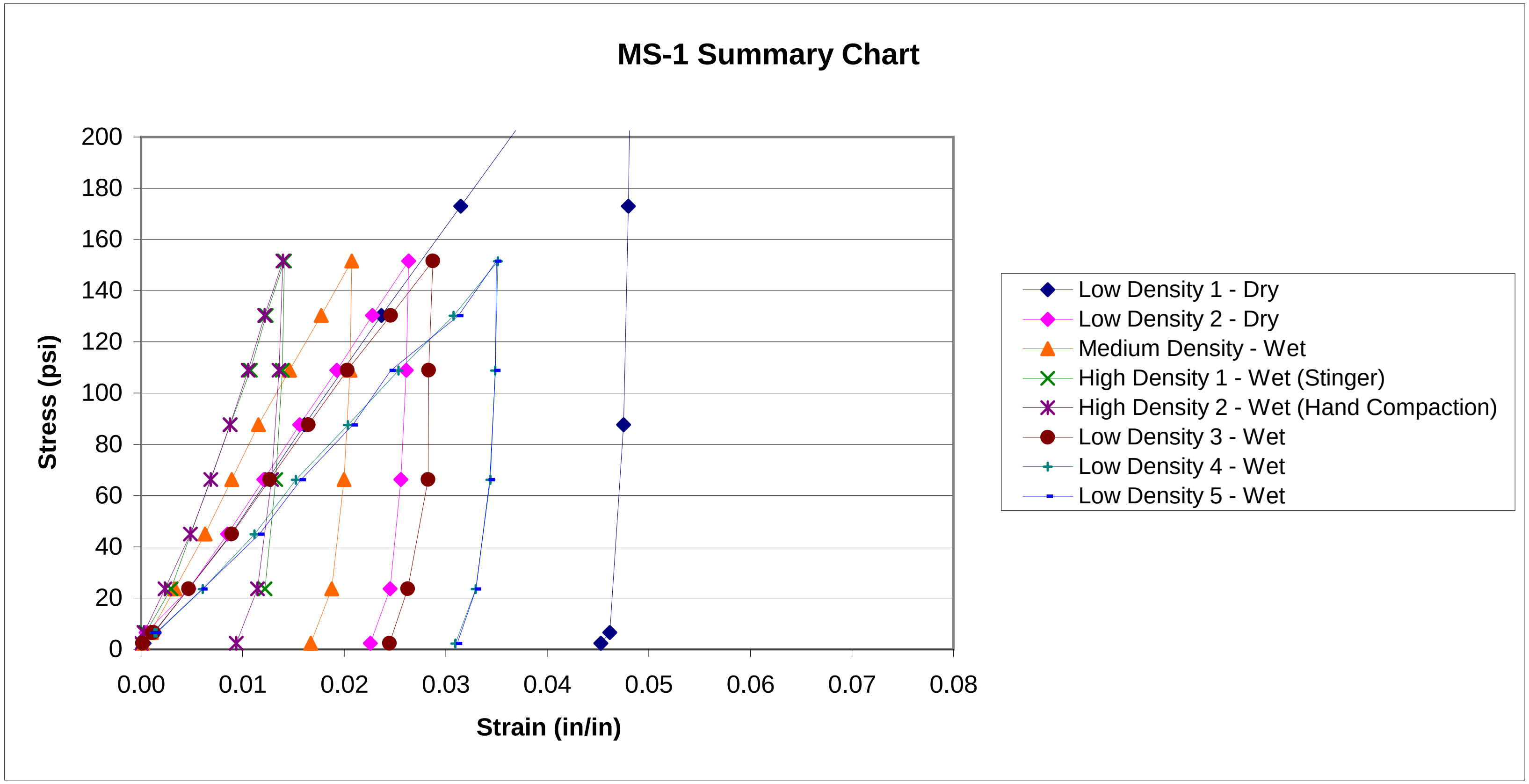
[E] = Final Load Conditions = 2.17 lb/in^2, except for MS-1: 14%, 23%, 31%, 44%, 69%, and 74% RD's, where it is 152 lb/in^2

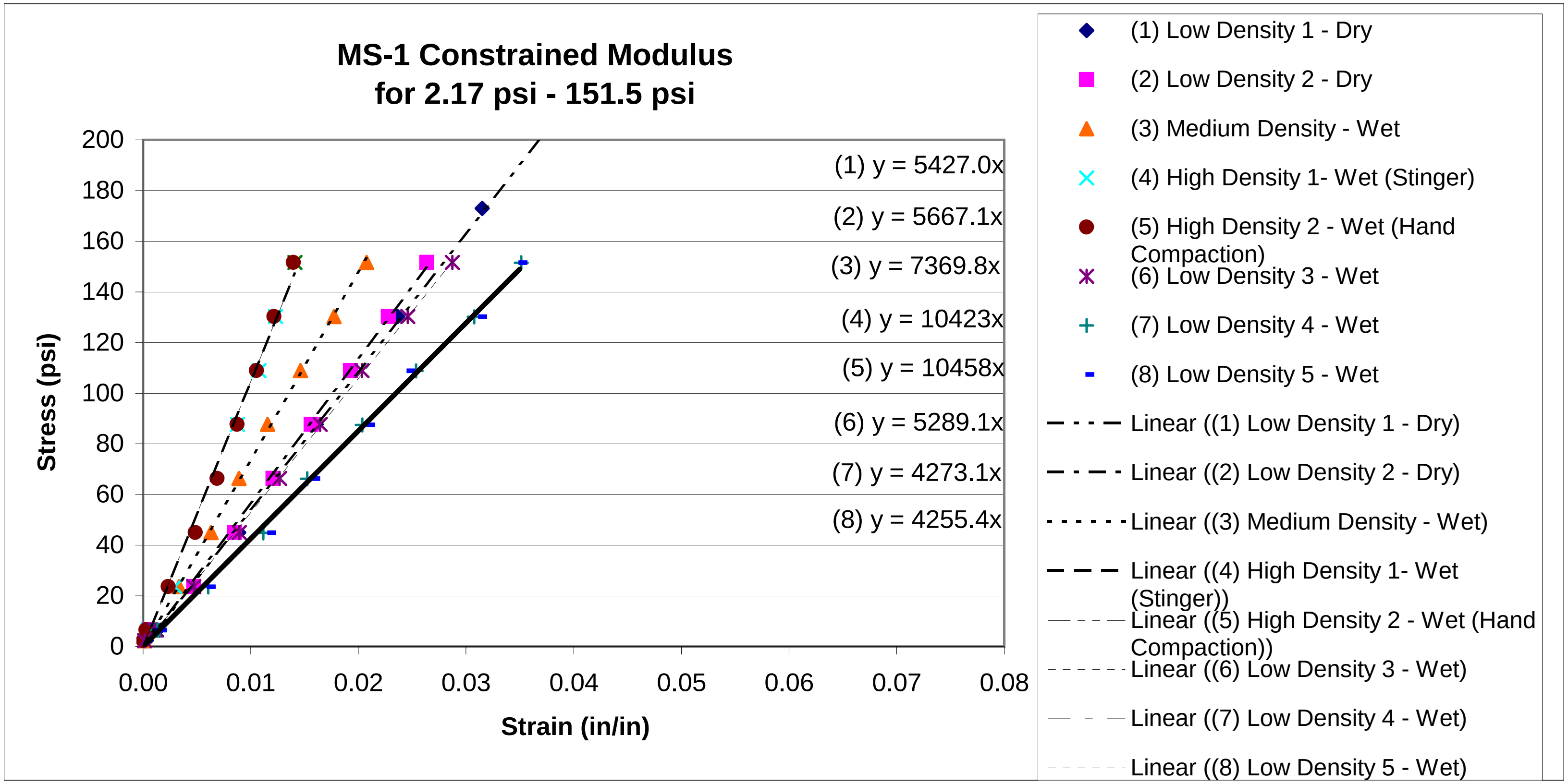
[F] = Percent Particle Breakdown is a term used to describe the particles that passed the largest sieve opening that retained all particles prior to testing

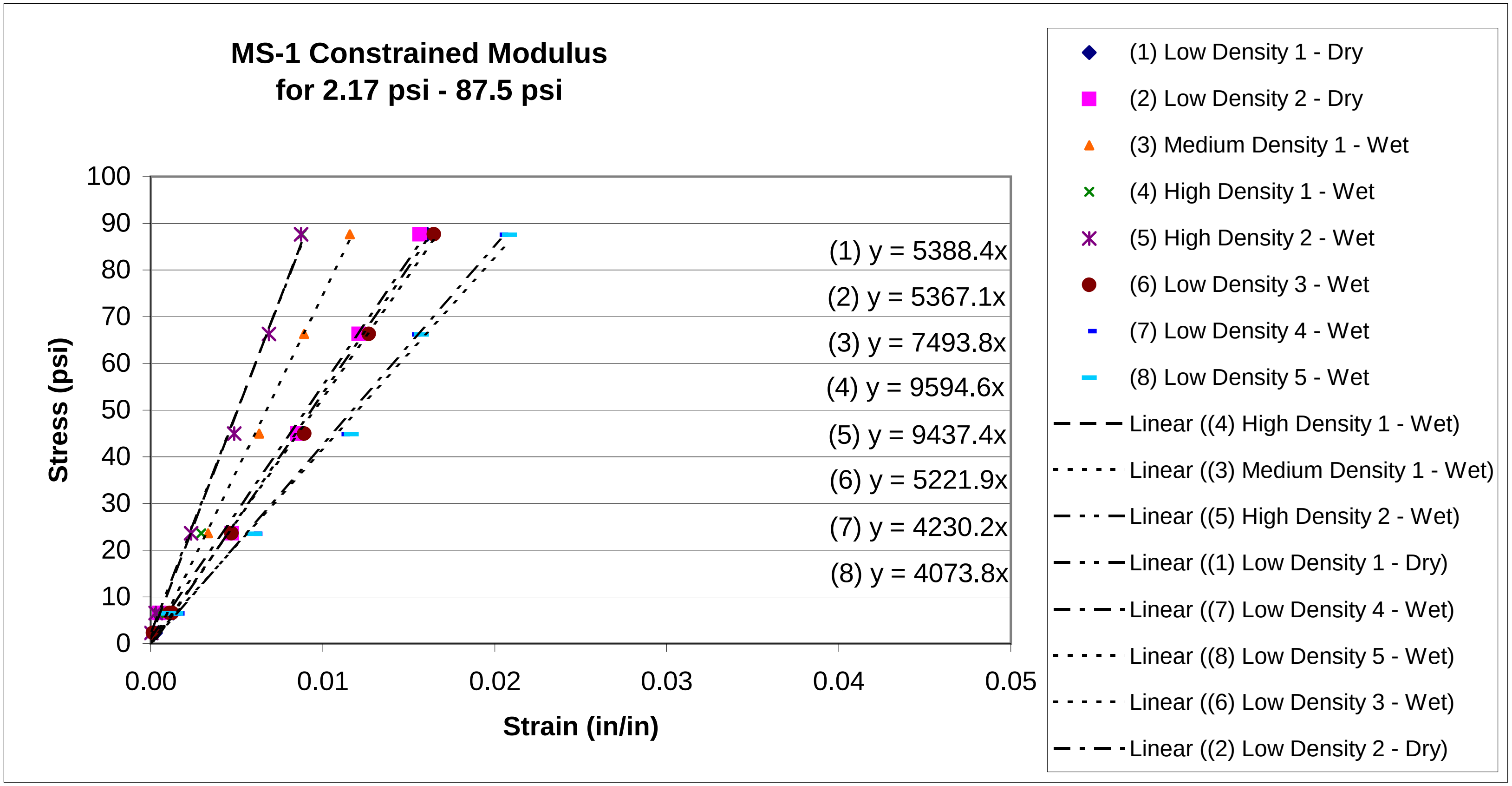
Attachment C1: *Constrained Modulus of Crushed Rock Test Data Summary*

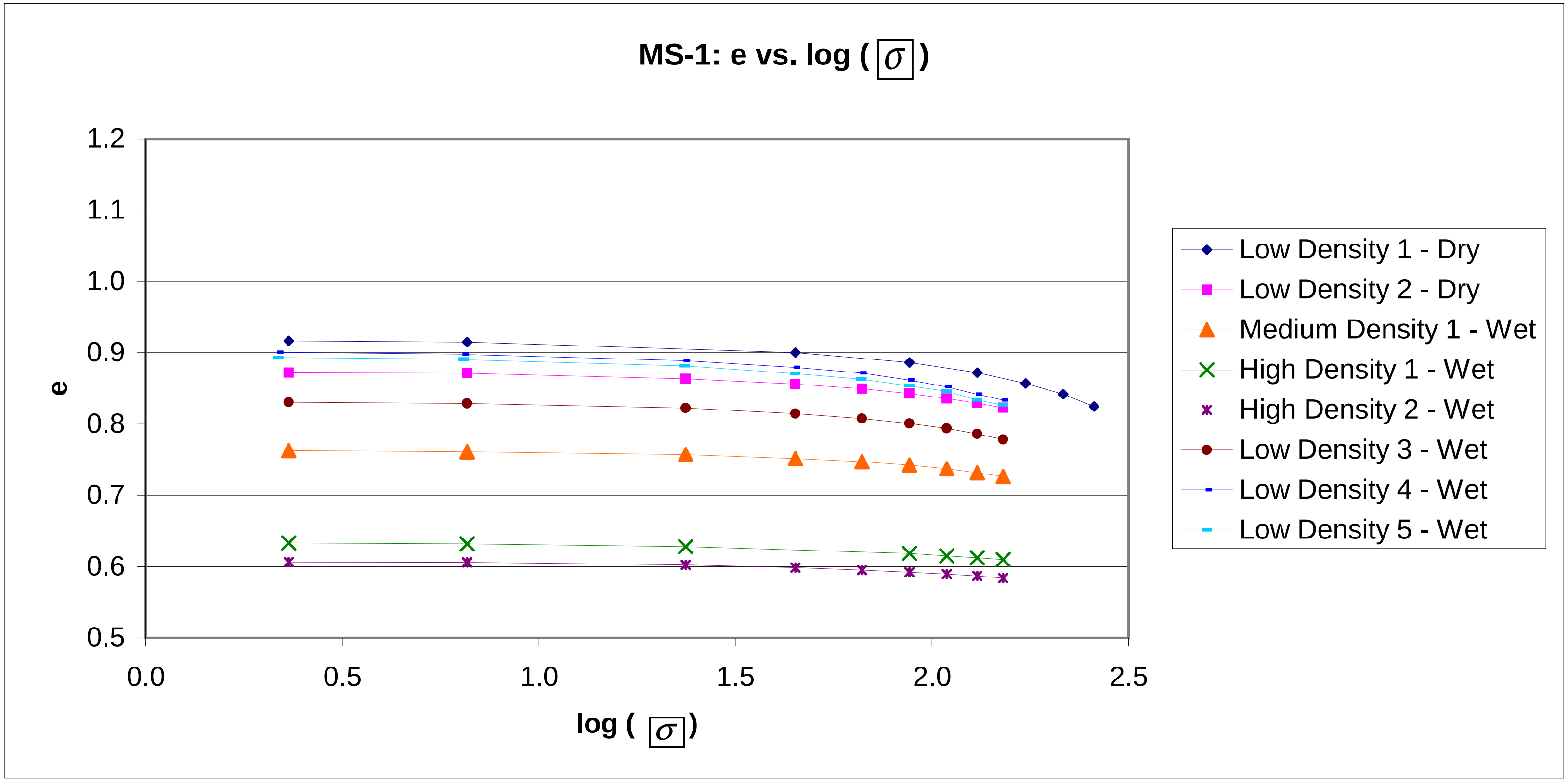
MS-1 Summary Table

Test	Unfactored Constrained Modulus (psi)	Ms (Stress Range: 2.17 -151.49 lb/in^2)	Ms (Stress Range: 2.17 -87.5 lb/in^2)	Initial Dry Density (lb/ft3)	Seating Load Dry Density (lb/ft3)	Final Load Dry Density (lb/ft3)	Initial RD	Final RD
Low Density 1 - Dry	5427	**5059**	**5023**	90.2	90.4	95.0	14.4%	33.0%
Low Density 2 - Dry	5667	**5287**	**5008**	92.4	92.7	95.2	23.1%	33.8%
Medium Density 1 - Wet	7370	**6890**	**7006**	98.1	98.4	100.4	44.1%	52.1%
High Density 1 - Wet	10423	**9772**	**8996**	105.9	106.1	107.7	69.2%	74.3%
High Density 2 - Wet	10458	**9811**	**8854**	107.6	108.3	109.8	74.3%	80.4%
Low Density 3 - Wet	5289	**4939**	**4876**	94.4	95.3	98.1	31.0%	44.1%
Low Density 4 - Wet	4273	**3985**	**3945**	91.0	91.2	94.5	17.6%	31.4%
Low Density 5 - Wet	4255	**3969**	**3799**	91.3	91.7	95.1	19.1%	33.4%


MS-1 Summary Chart
Stress (psi)
Strain (in/in)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Low Density 1 - Dry
Low Density 2 - Dry
Medium Density - Wet
High Density 1 - Wet (Stinger)
High Density 2 - Wet (Hand Compaction)
Low Density 3 - Wet
Low Density 4 - Wet
Low Density 5 - Wet

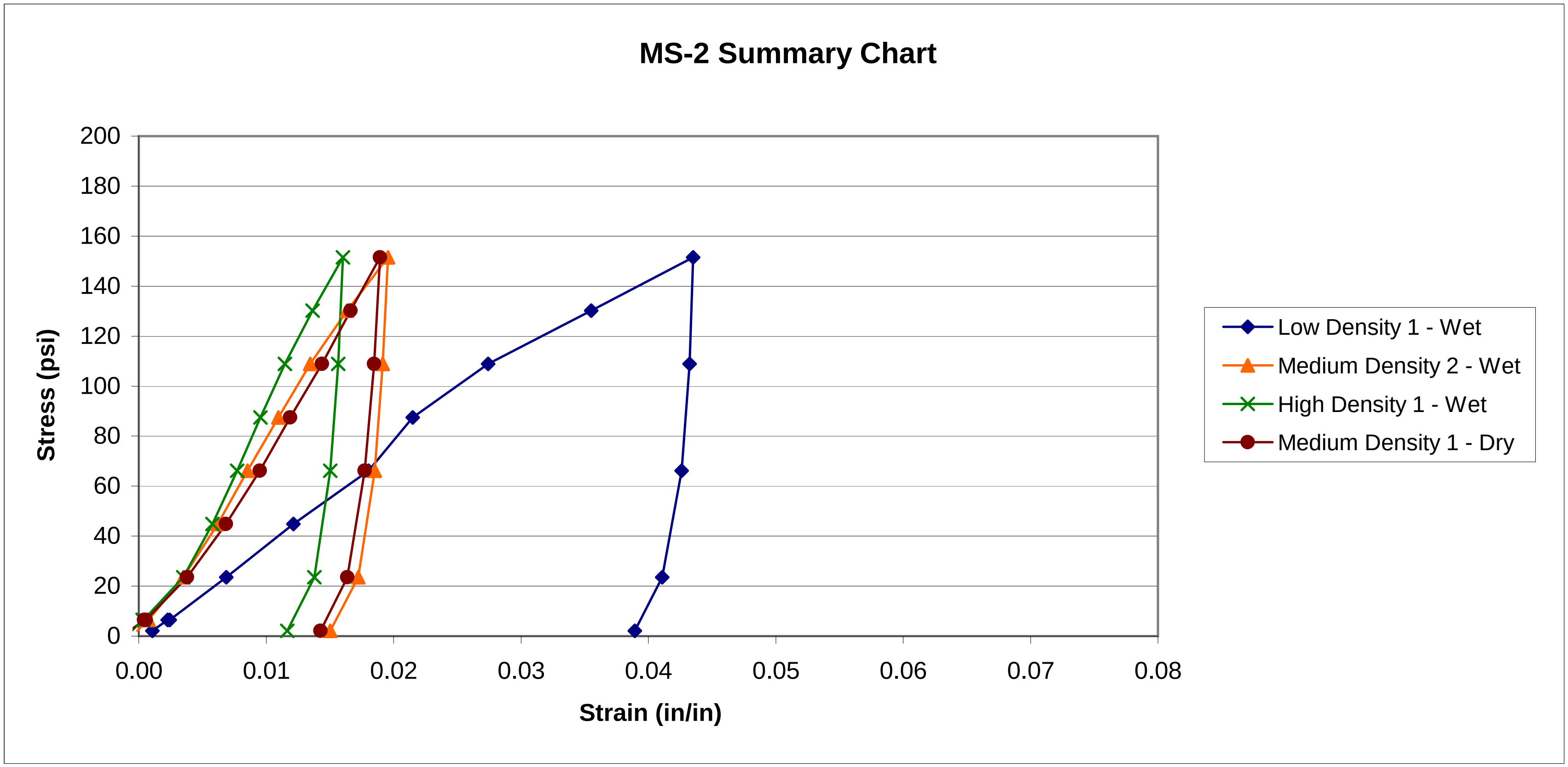

MS-1 Constrained Modulus
for 2.17 psi - 151.5 psi
Stress (psi)
Strain (in/in)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
(1) y = 5427.0x
(2) y = 5667.1x
(3) y = 7369.8x
(4) y = 10423x
(5) y = 10458x
(6) y = 5289.1x
(7) y = 4273.1x
(8) y = 4255.4x
(1) Low Density 1 - Dry
(2) Low Density 2 - Dry
(3) Medium Density - Wet
(4) High Density 1- Wet (Stinger)
(5) High Density 2 - Wet (Hand Compaction)
(6) Low Density 3 - Wet
(7) Low Density 4 - Wet
(8) Low Density 5 - Wet
Linear ((1) Low Density 1 - Dry)
Linear ((2) Low Density 2 - Dry)
Linear ((3) Medium Density - Wet)
Linear ((4) High Density 1- Wet (Stinger))
Linear ((5) High Density 2 - Wet (Hand Compaction))
Linear ((6) Low Density 3 - Wet)
Linear ((7) Low Density 4 - Wet)
Linear ((8) Low Density 5 - Wet)

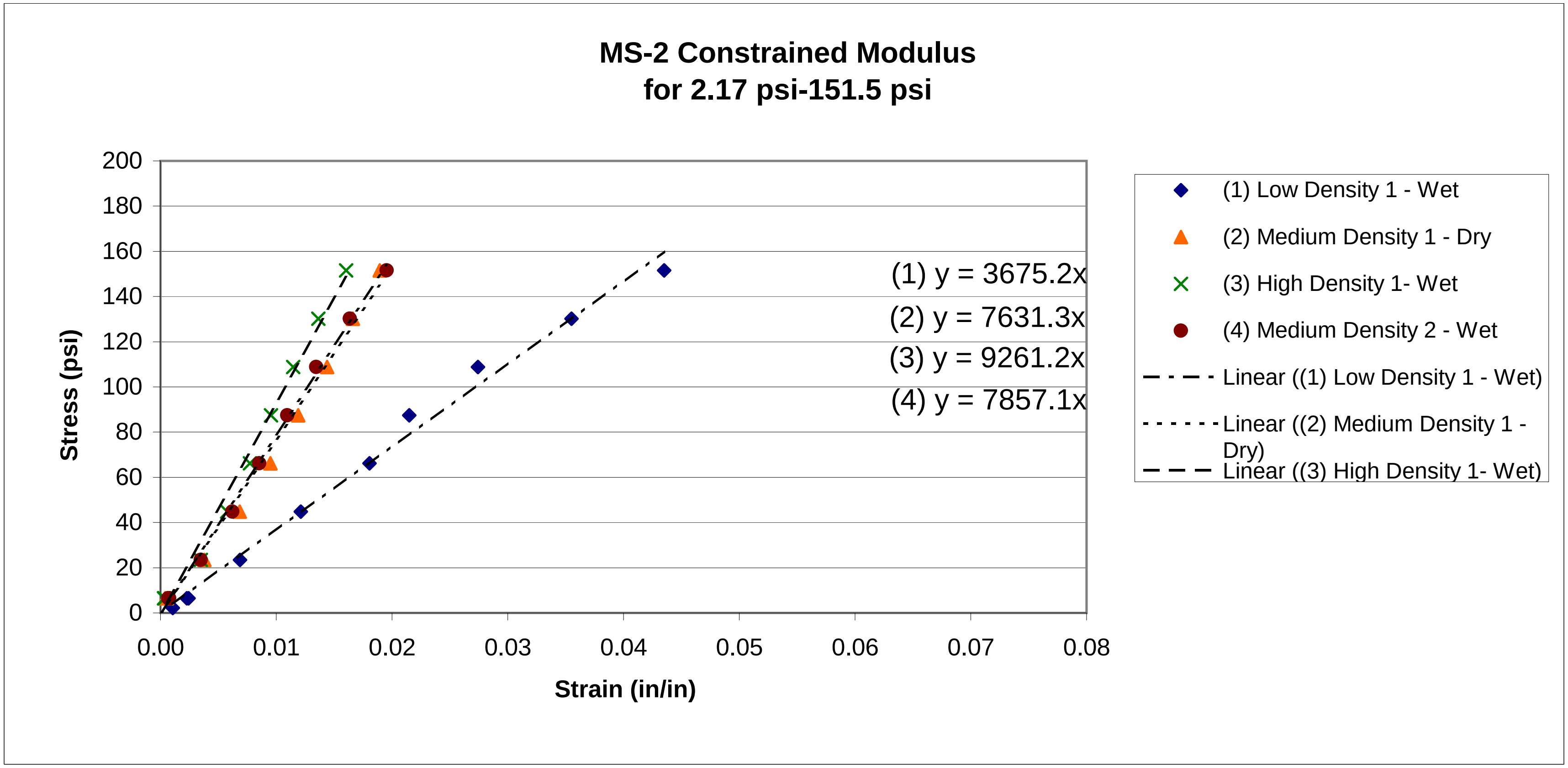

MS-1 Constrained Modulus
for 2.17 psi - 87.5 psi
Stress (psi)
Strain (in/in)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.01
0.02
0.03
0.04
0.05
(1) y = 5388.4x
(2) y = 5367.1x
(3) y = 7493.8x
(4) y = 9594.6x
(5) y = 9437.4x
(6) y = 5221.9x
(7) y = 4230.2x
(8) y = 4073.8x
(1) Low Density 1 - Dry
(2) Low Density 2 - Dry
(3) Medium Density 1 - Wet
(4) High Density 1 - Wet
(5) High Density 2 - Wet
(6) Low Density 3 - Wet
(7) Low Density 4 - Wet
(8) Low Density 5 - Wet
Linear ((4) High Density 1 - Wet)
Linear ((3) Medium Density 1 - Wet)
Linear ((5) High Density 2 - Wet)
Linear ((1) Low Density 1 - Dry)
Linear ((7) Low Density 4 - Wet)
Linear ((8) Low Density 5 - Wet)
Linear ((6) Low Density 3 - Wet)
Linear ((2) Low Density 2 - Dry)

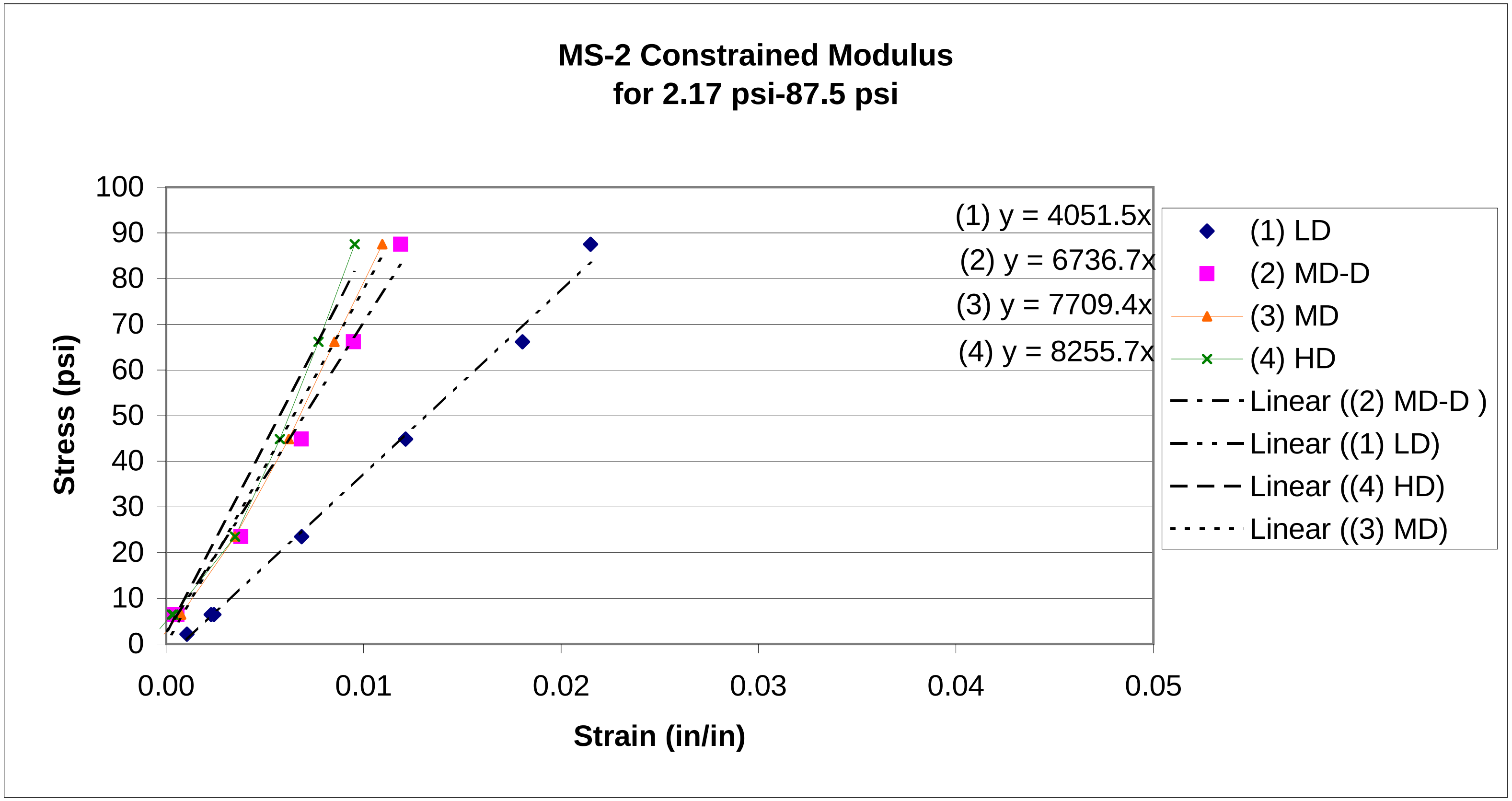

MS-1: e vs. log ($\boxed{\sigma}$)
e
log ($\boxed{\sigma}$)
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.0
0.5
1.0
1.5
2.0
2.5
Low Density 1 - Dry
Low Density 2 - Dry
Medium Density 1 - Wet
High Density 1 - Wet
High Density 2 - Wet
Low Density 3 - Wet
Low Density 4 - Wet
Low Density 5 - Wet

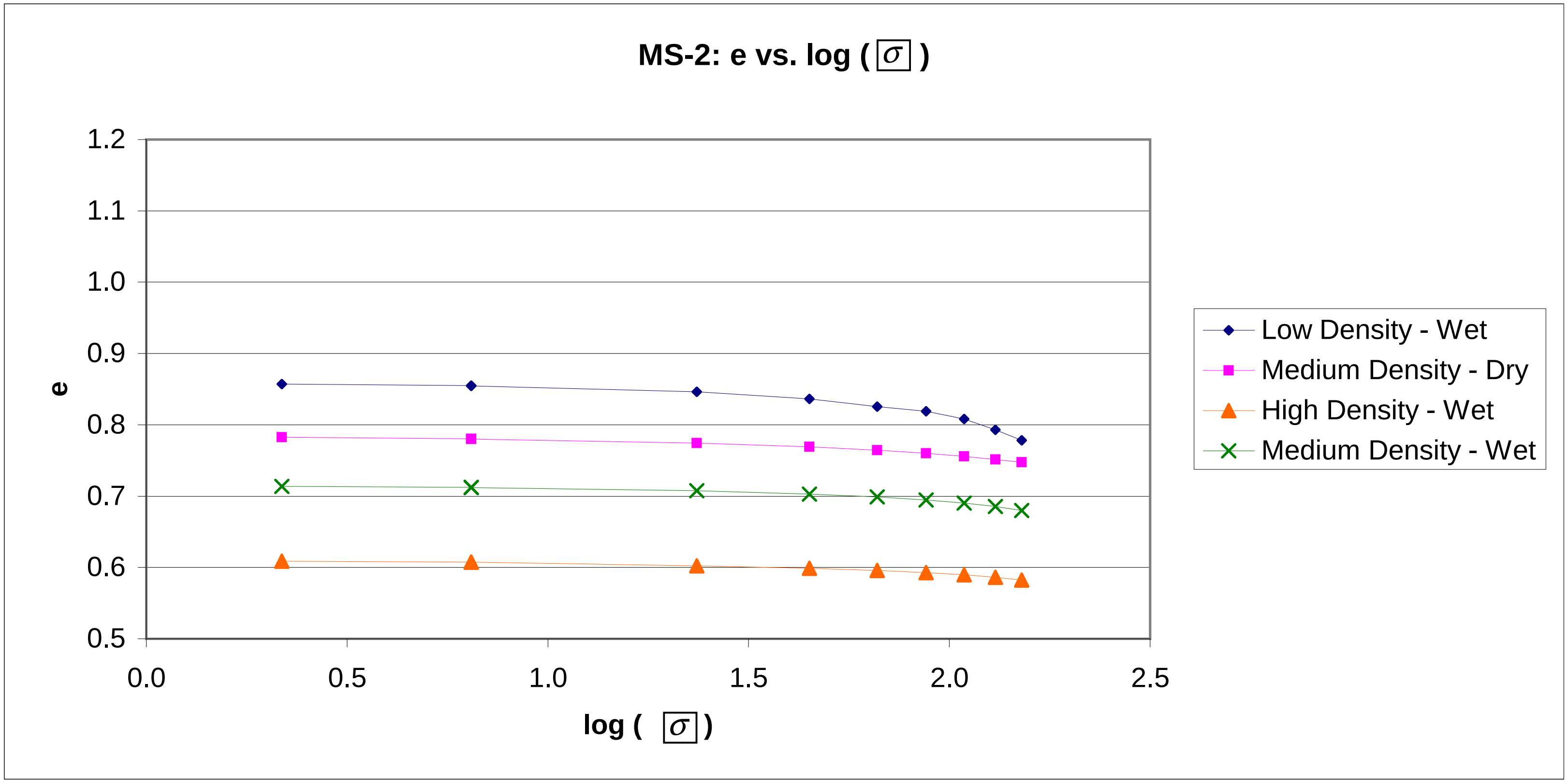
MS-2 Summary Table

Test	Unfactored Constrained Modulus (psi)	Ms (Stress Range: 2.17 -151.49 lb/in^2)	Ms (Stress Range: 2.17 -87.5 lb/in^2)	Initial Dry Density (lb/ft3)	Seating Load Dry Density (lb/ft3)	Final Load Dry Density (lb/ft3)	Initial RD	Final RD
Low Density 1 - Wet	3675	**3339**	**3681**	89.5	89.8	93.0	12.1%	35.8%
Medium Density 1 - Dry	7631	**6950**	**6135**	93.4	93.6	95.3	38.6%	50.7%
Medium Density 2 - Wet	7857	**7171**	**7062**	97.1	97.3	98.9	61.6%	72.2%
High Density 1 - Wet	9261	**8484**	**7535**	103.4	103.6	105.1	97.3%	106.0%


MS-2 Summary Chart
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Strain (in/in)
Low Density 1 - Wet
Medium Density 2 - Wet
High Density 1 - Wet
Medium Density 1 - Dry


MS-2 Constrained Modulus
for 2.17 psi-151.5 psi
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Strain (in/in)
(1) y = 3675.2x
(2) y = 7631.3x
(3) y = 9261.2x
(4) y = 7857.1x
(1) Low Density 1 - Wet
(2) Medium Density 1 - Dry
(3) High Density 1- Wet
(4) Medium Density 2 - Wet
Linear ((1) Low Density 1 - Wet)
Linear ((2) Medium Density 1 - Dry)
Linear ((3) High Density 1- Wet)

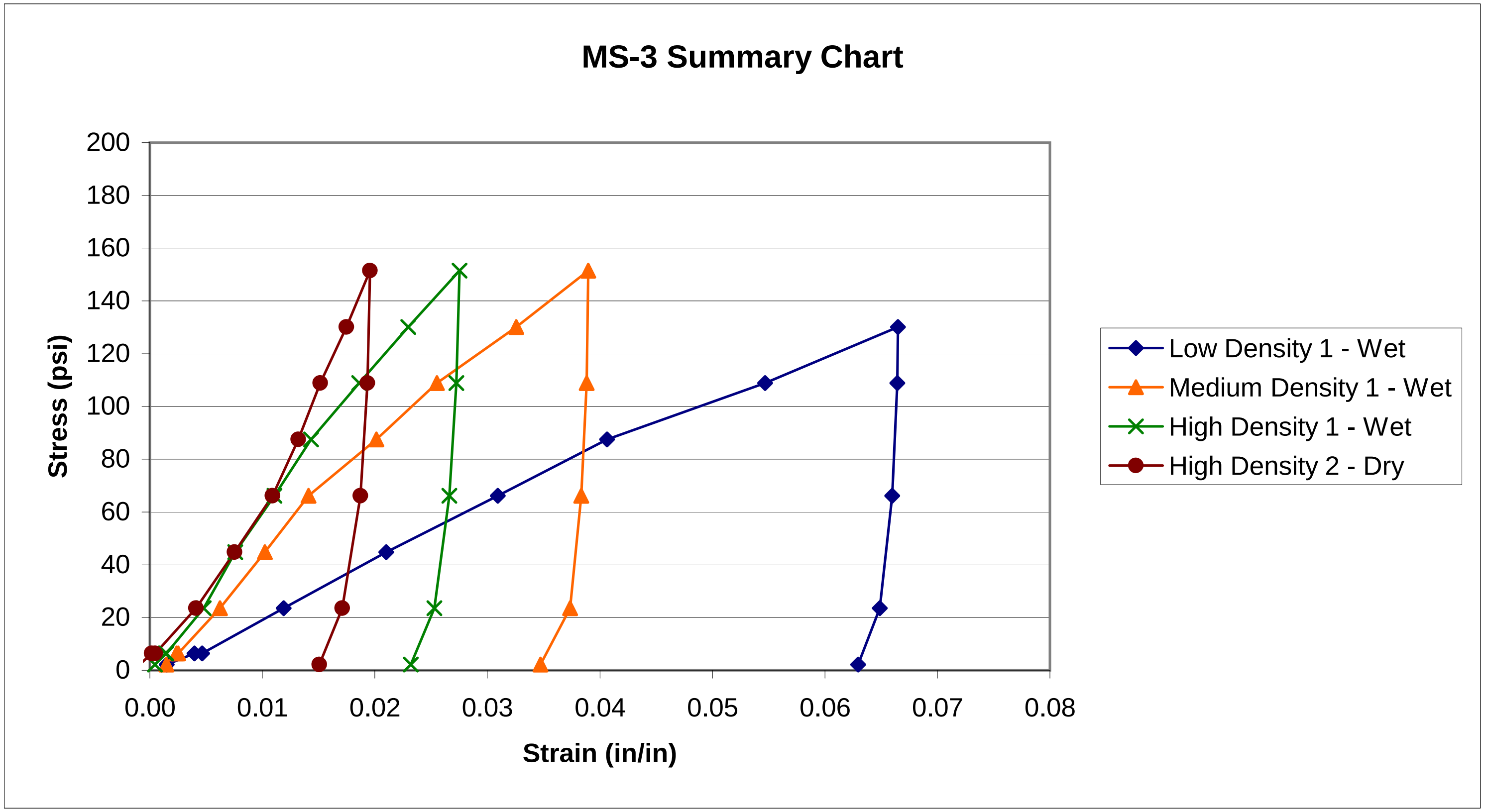

MS-2 Constrained Modulus
for 2.17 psi-87.5 psi
Stress (psi)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.01
0.02
0.03
0.04
0.05
Strain (in/in)
(1) y = 4051.5x
(2) y = 6736.7x
(3) y = 7709.4x
(4) y = 8255.7x
(1) LD
(2) MD-D
(3) MD
(4) HD
Linear ((2) MD-D)
Linear ((1) LD)
Linear ((4) HD)
Linear ((3) MD)



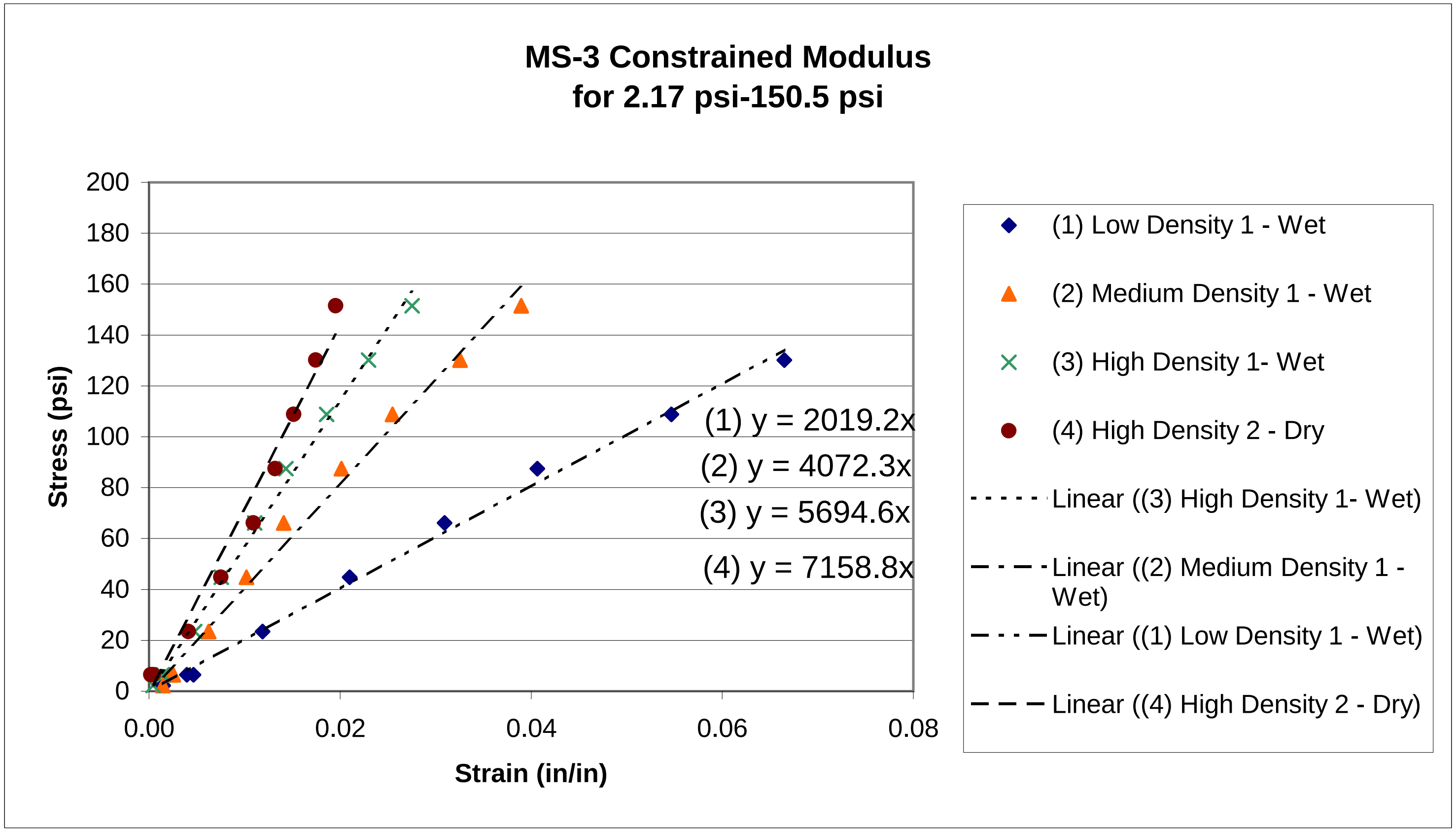
MS-2: e vs. log (σ)
e
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.0
0.5
1.0
1.5
2.0
2.5
log (σ)
Low Density - Wet
Medium Density - Dry
High Density - Wet
Medium Density - Wet

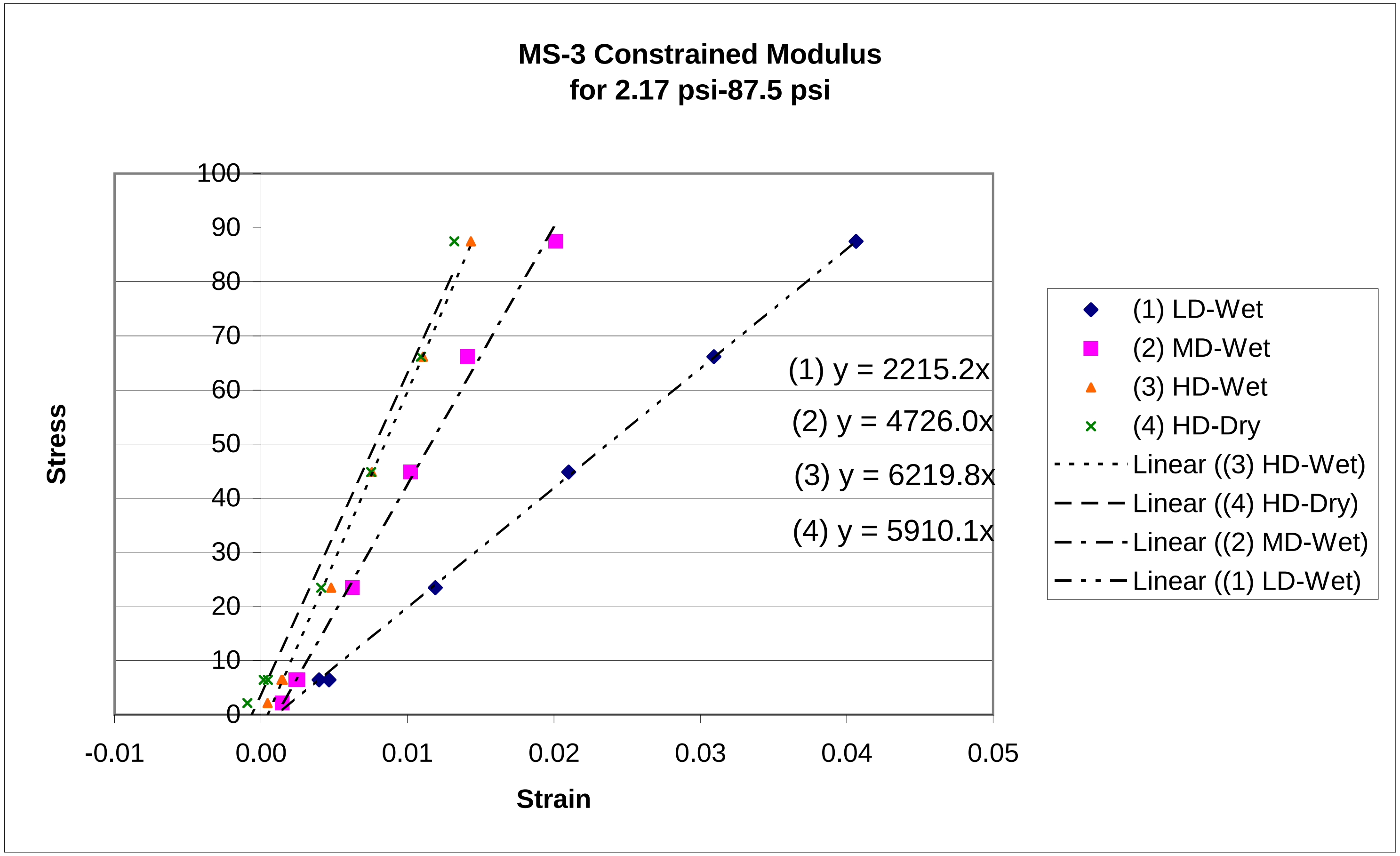
MS-3 Summary Table

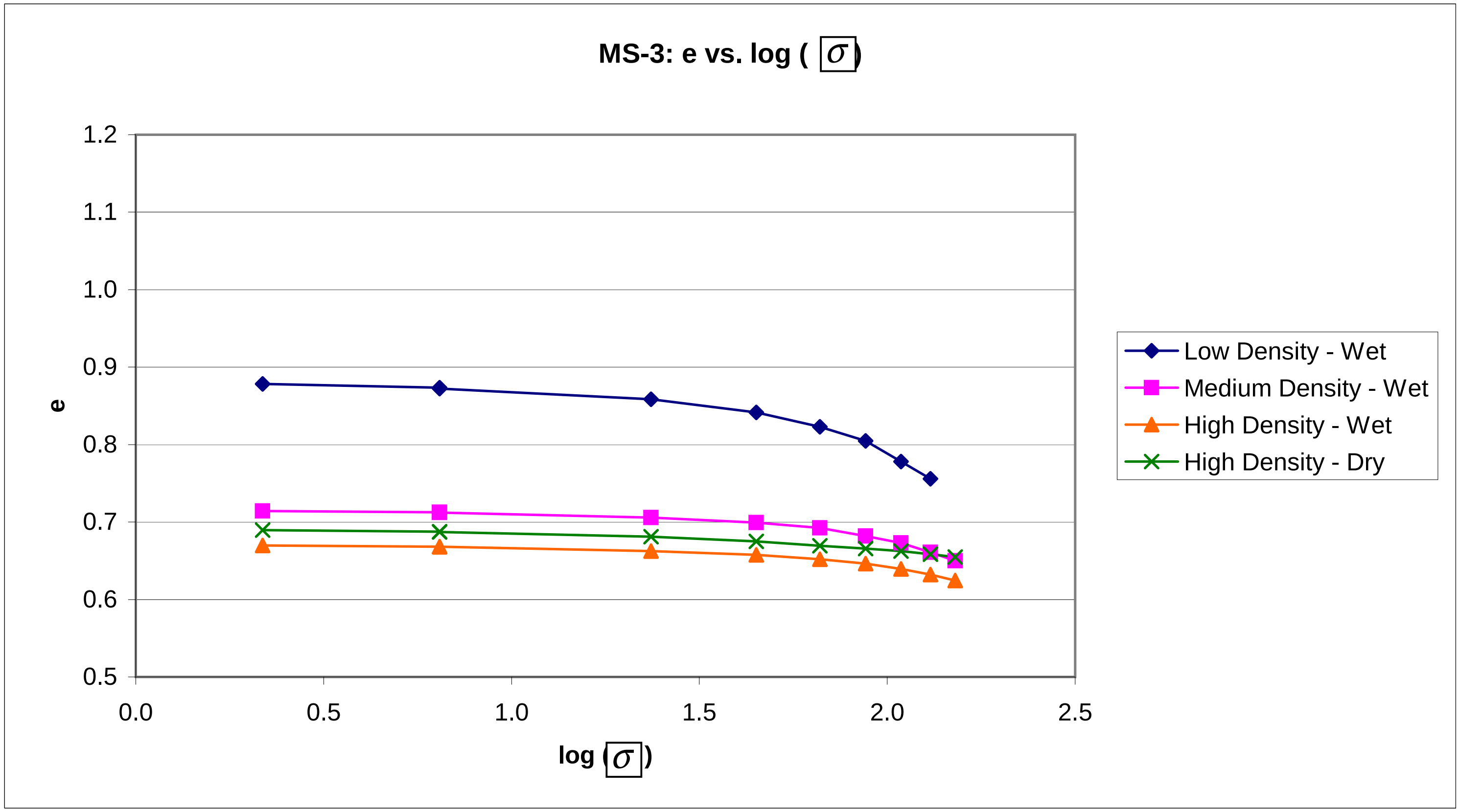
Test	Unfactored Constrained Modulus (psi)	Ms (Stress Range: 2.17 -151.49 lb/in^2)	Ms (Stress Range: 2.17 - 87.5 lb/in^2)	Initial Dry Density (lb/ft3)	Seating Load Dry Density (lb/ft3)	Final Load Dry Density (lb/ft3)	Initial RD	Final RD
Low Density 1 - Wet	2019	**1727**	**1894**	86.6	86.9	93.0	14.6%	67.7%
Medium Density 1 - Wet	4072	**3515**	**4079**	94.9	95.4	98.5	82.0%	107.7%
High Density 1 - Wet	5695	**4930**	**5384**	97.5	97.8	100.0	100.9%	118.0%
High Density 2 - Dry	7159	**6190**	**5111**	96.5	96.8	98.6	93.8%	108.2%



MS-3 Summary Chart
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Strain (in/in)
Low Density 1 - Wet
Medium Density 1 - Wet
High Density 1 - Wet
High Density 2 - Dry

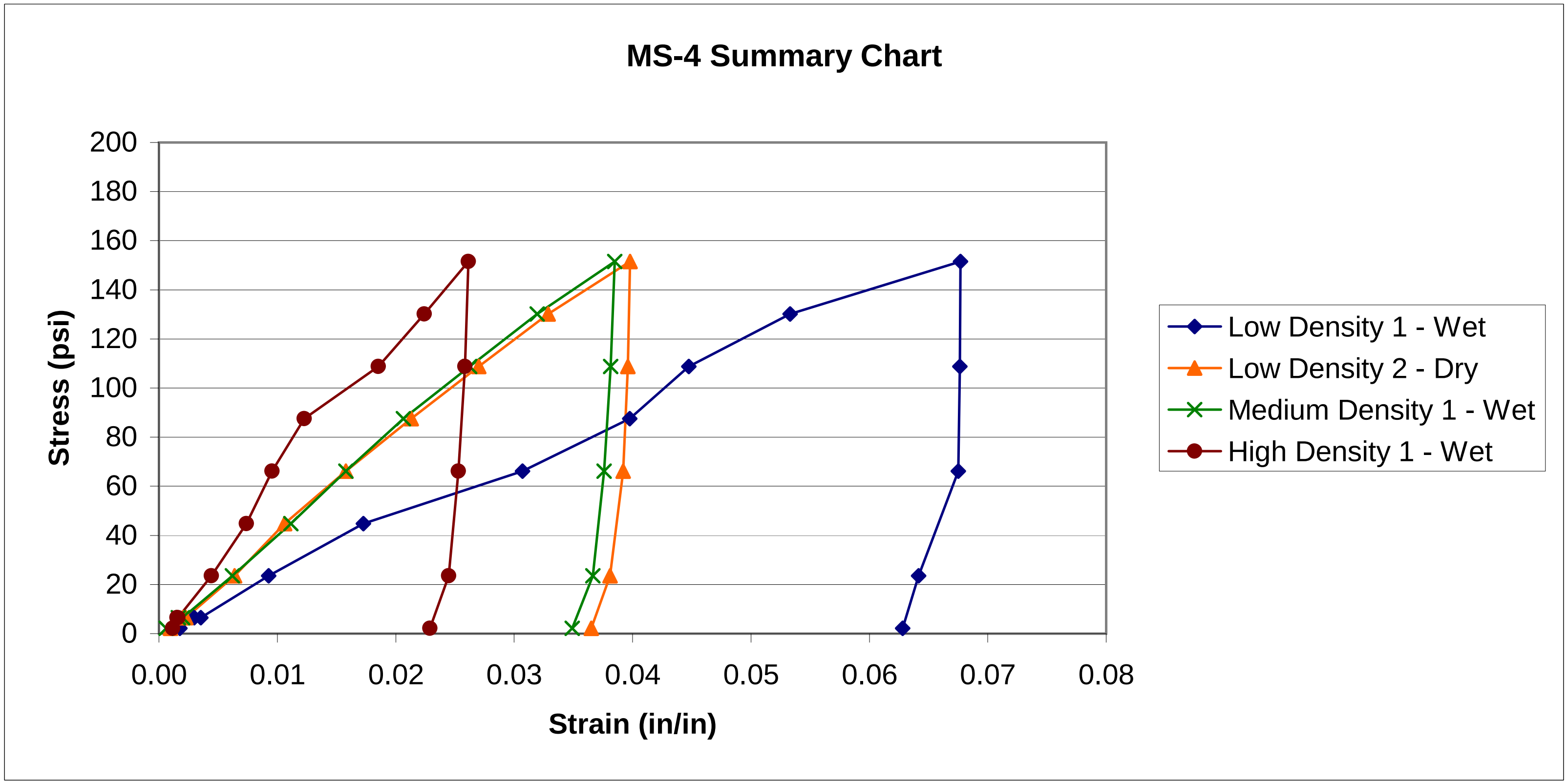

MS-3 Constrained Modulus
for 2.17 psi-150.5 psi
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.02
0.04
0.06
0.08
Strain (in/in)
(1) y = 2019.2x
(2) y = 4072.3x
(3) y = 5694.6x
(4) y = 7158.8x
(1) Low Density 1 - Wet
(2) Medium Density 1 - Wet
(3) High Density 1- Wet
(4) High Density 2 - Dry
Linear ((3) High Density 1- Wet)
Linear ((2) Medium Density 1 - Wet)
Linear ((1) Low Density 1 - Wet)
Linear ((4) High Density 2 - Dry)

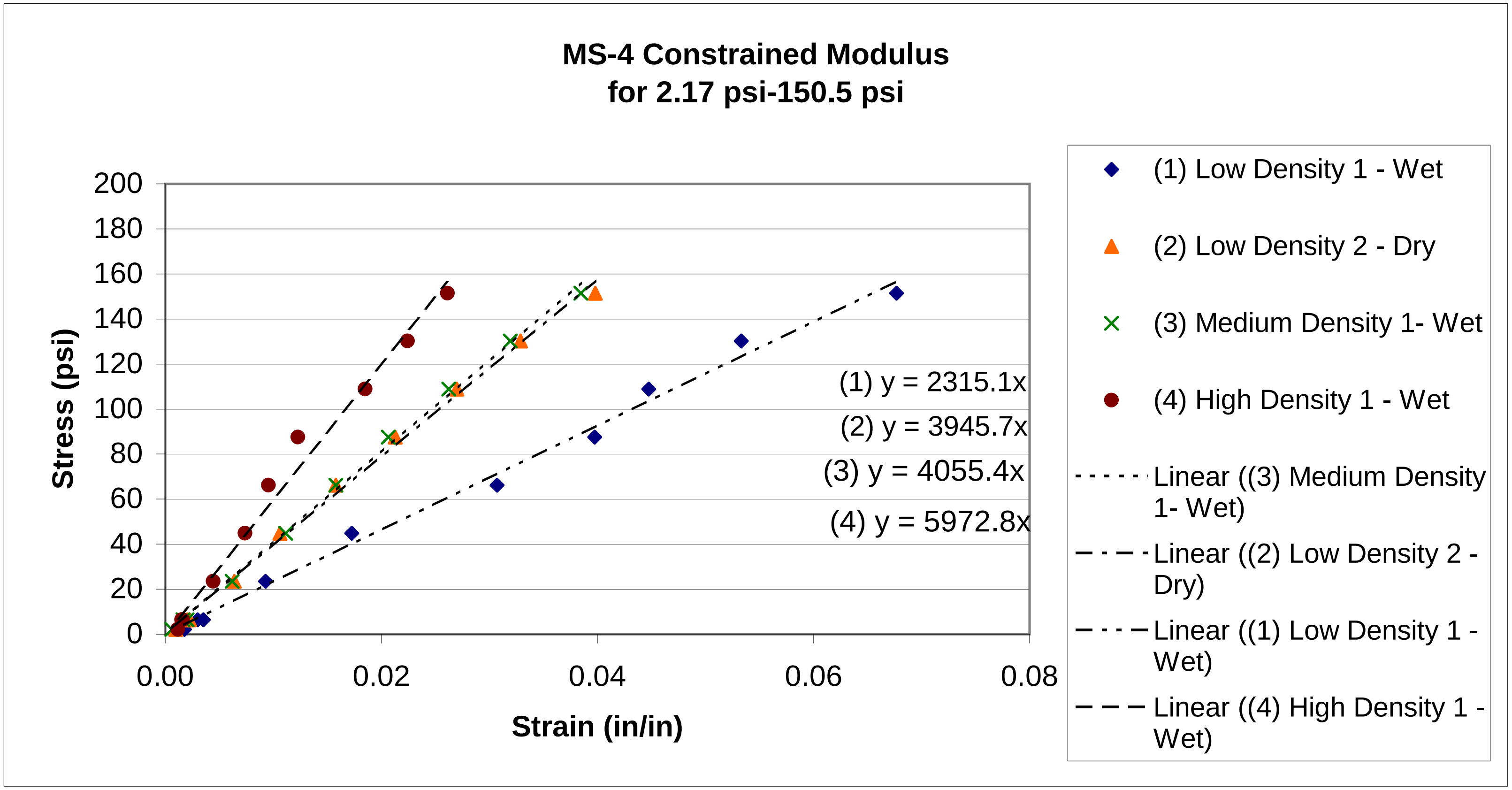

MS-3 Constrained Modulus
for 2.17 psi-87.5 psi
Stress
Strain
0
10
20
30
40
50
60
70
80
90
100
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
(1) y = 2215.2x
(2) y = 4726.0x
(3) y = 6219.8x
(4) y = 5910.1x
(1) LD-Wet
(2) MD-Wet
(3) HD-Wet
(4) HD-Dry
Linear ((3) HD-Wet)
Linear ((4) HD-Dry)
Linear ((2) MD-Wet)
Linear ((1) LD-Wet)

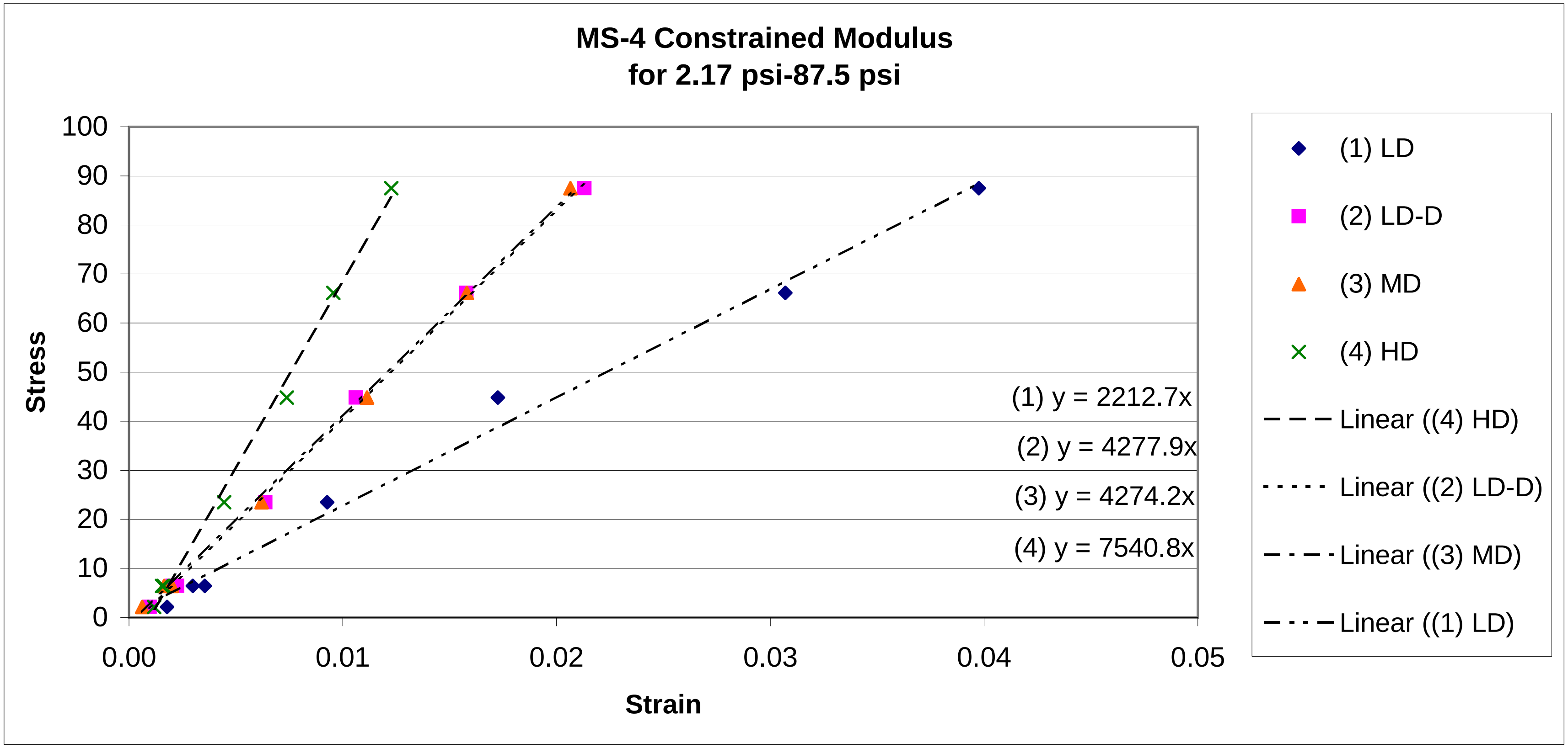

MS-3: e vs. log (σ)
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
e
0.0
0.5
1.0
1.5
2.0
2.5
log (σ)
Low Density - Wet
Medium Density - Wet
High Density - Wet
High Density - Dry

MS-4 Summary Table

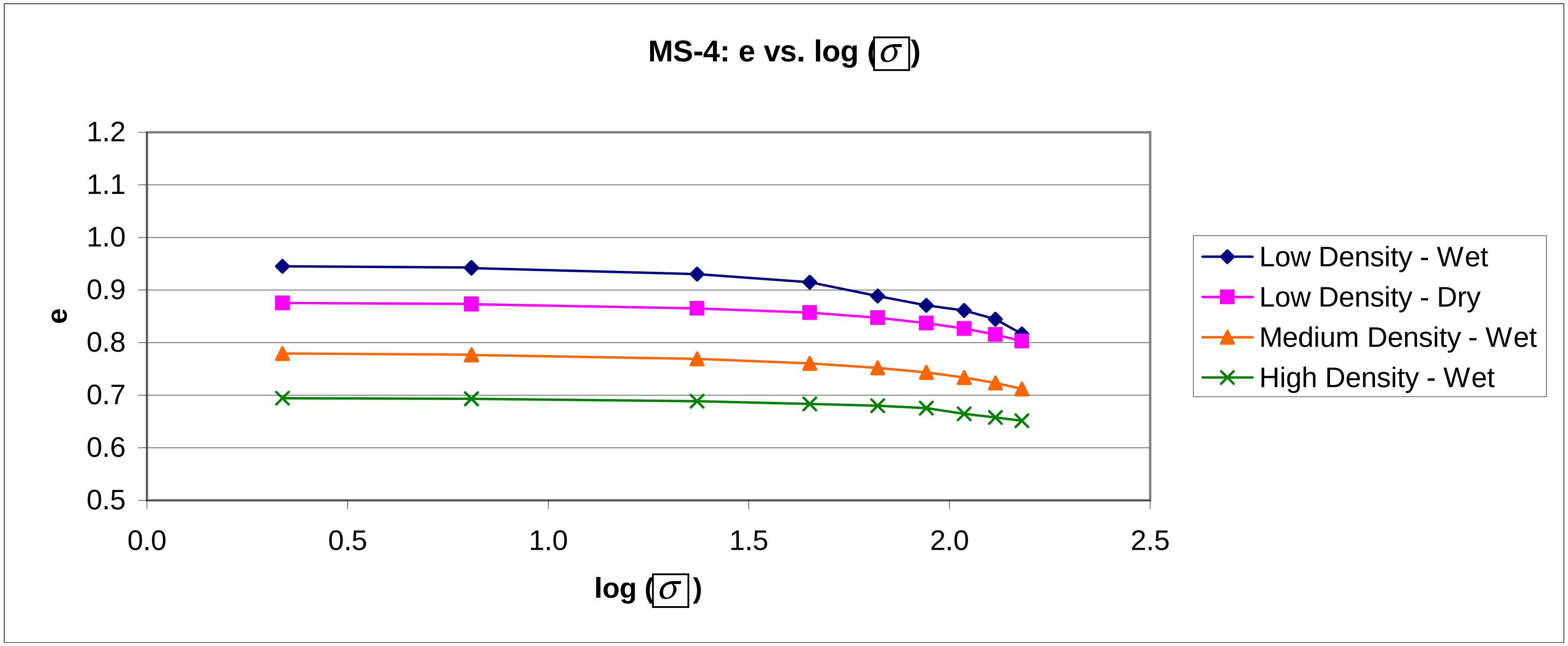
Test	Unfactored Constrained Modulus (psi)	Ms (Stress Range: 2.17 -151.49 lb/in^2)	Ms (Stress Range: 2.17 - 87.5 lb/in^2)	Initial Dry Density (lb/ft3)	Seating Load Dry Density (lb/ft3)	Final Load Dry Density (lb/ft3)	Initial RD	Final RD
Low Density 1 - Wet	2315	**2087**	**1994**	84.9	85.2	89.9	-28.8%	11.0%
Low Density 2 - Dry	3946	**3564**	**3864**	88.1	88.3	91.2	-2.7%	20.7%
Medium Density 1 - Wet	4055	**3675**	**3873**	92.9	93.2	96.2	32.8%	55.1%
High Density 1 - Wet	5973	**5428**	**6853**	97.5	97.9	100.0	63.6%	79.2%


MS-4 Summary Chart
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Strain (in/in)
Low Density 1 - Wet
Low Density 2 - Dry
Medium Density 1 - Wet
High Density 1 - Wet


MS-4 Constrained Modulus
for 2.17 psi-150.5 psi
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.02
0.04
0.06
0.08
Strain (in/in)
(1) y = 2315.1x
(2) y = 3945.7x
(3) y = 4055.4x
(4) y = 5972.8x
(1) Low Density 1 - Wet
(2) Low Density 2 - Dry
(3) Medium Density 1- Wet
(4) High Density 1 - Wet
Linear ((3) Medium Density 1- Wet)
Linear ((2) Low Density 2 - Dry)
Linear ((1) Low Density 1 - Wet)
Linear ((4) High Density 1 - Wet)

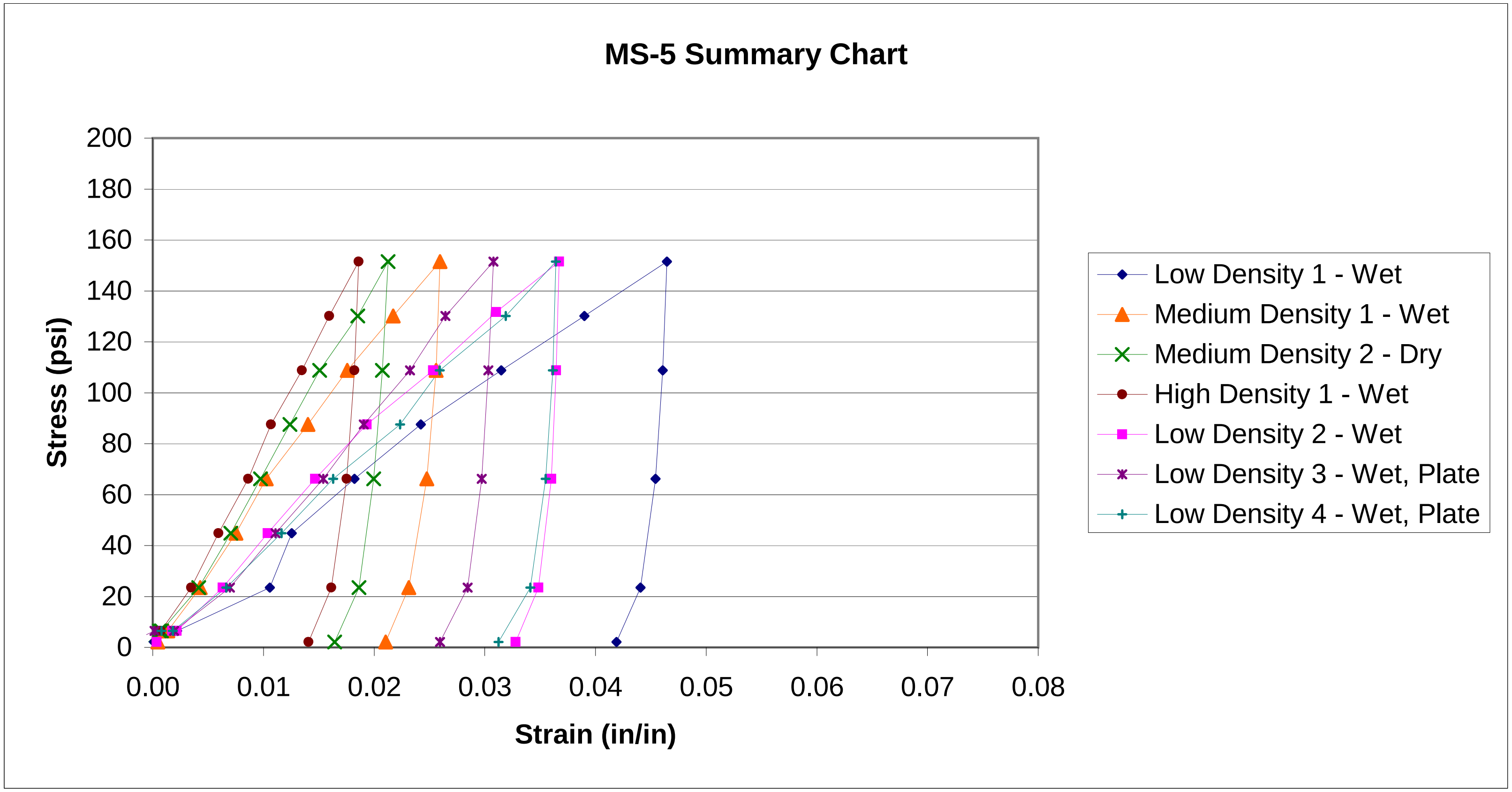


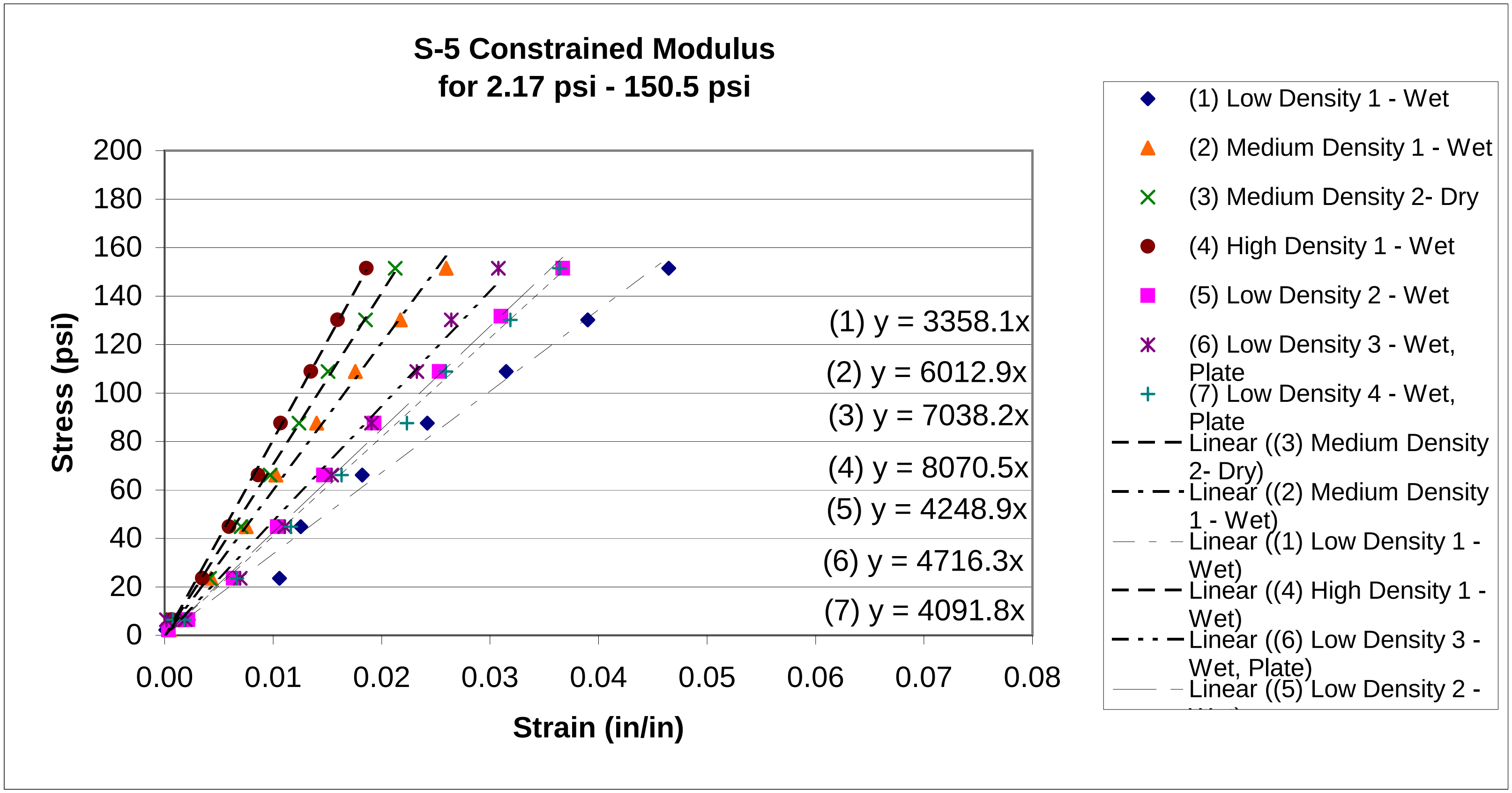
MS-4 Constrained Modulus
for 2.17 psi-87.5 psi
Stress
Strain
100
90
80
70
60
50
40
30
20
10
0
0.00
0.01
0.02
0.03
0.04
0.05
(1) y = 2212.7x
(2) y = 4277.9x
(3) y = 4274.2x
(4) y = 7540.8x
(1) LD
(2) LD-D
(3) MD
(4) HD
Linear ((4) HD)
Linear ((2) LD-D)
Linear ((3) MD)
Linear ((1) LD)

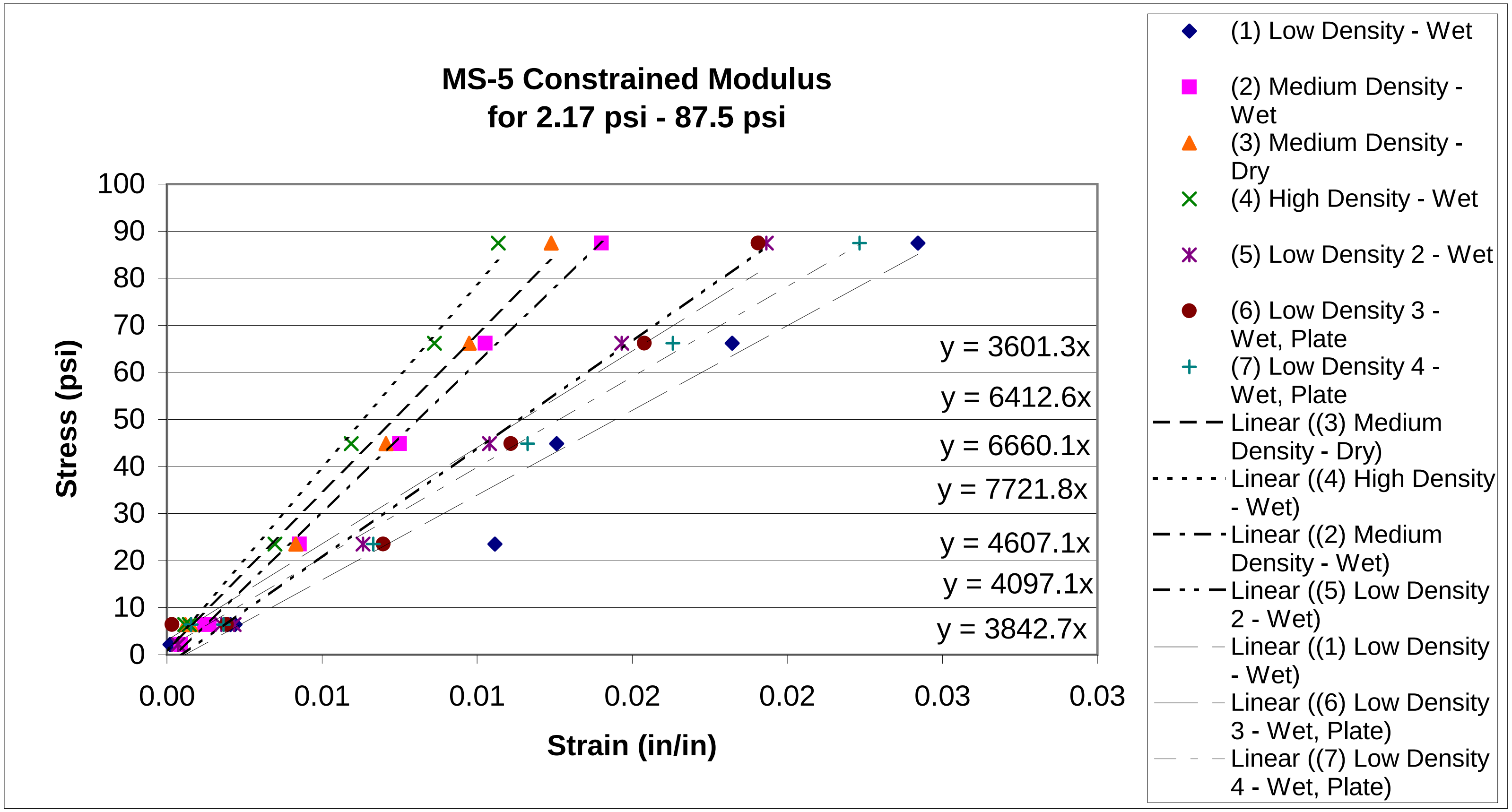

MS-4: e vs. log (σ)
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
e
0.0
0.5
1.0
1.5
2.0
2.5
log (σ)
Low Density - Wet
Low Density - Dry
Medium Density - Wet
High Density - Wet

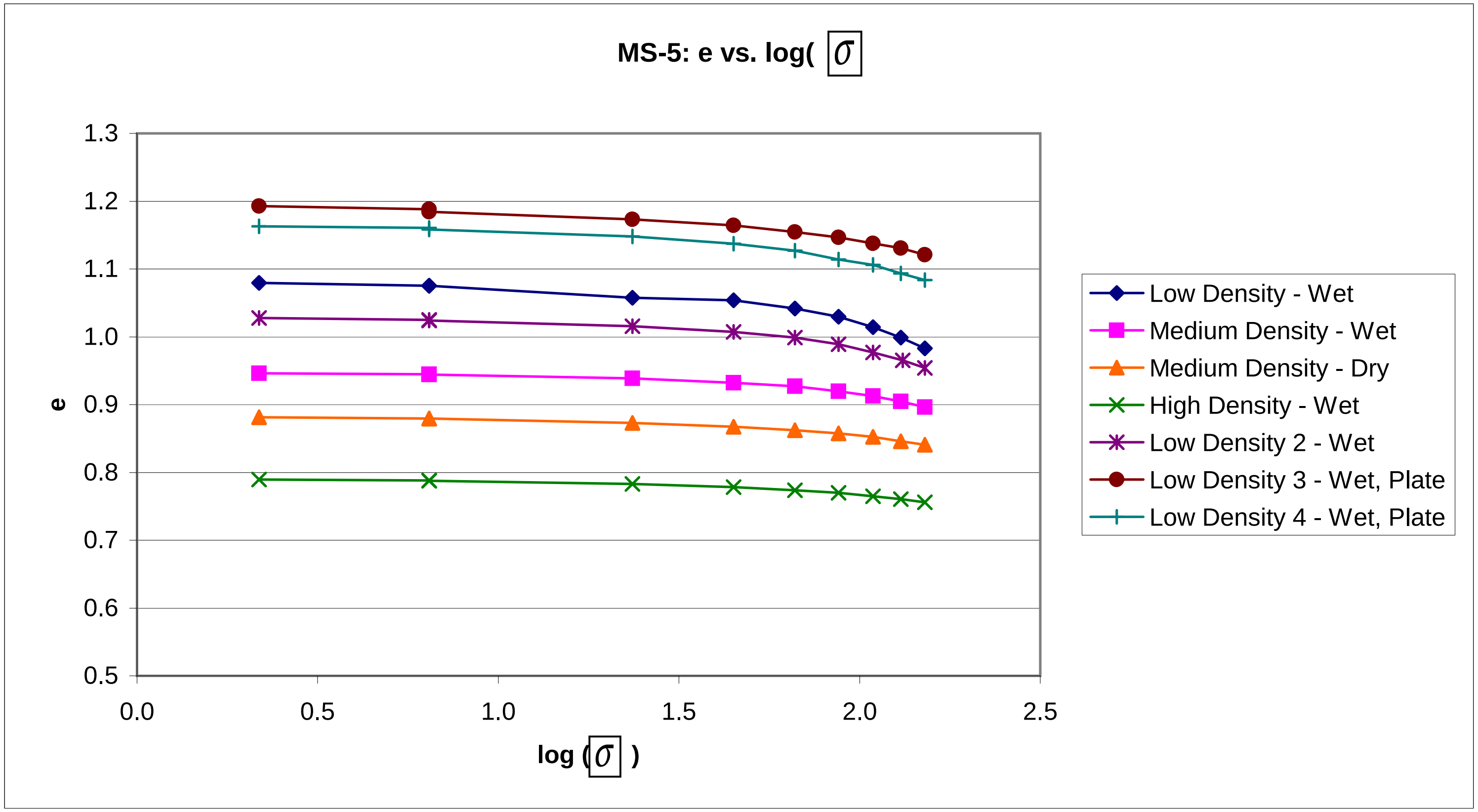
MS-5 Summary Table

Test	Unfactored Constrained Modulus (psi)	Ms (Stress Range: 2.17 - 151.49 lb/in^2)	Ms (Stress Range: 2.17 - 87.5 lb/in^2)	Initial Dry Density (lb/ft3)	Seating Load Dry Density (lb/ft3)	Final Load Dry Density (lb/ft3)	Initial RD	Final RD
Low Density 1 - Wet	3358	**3011**	**3229**	75.9	76.2	79.3	-9.0%	22.3%
Medium Density 1 - Wet	6013	**5414**	**5774**	81.1	81.7	83.5	37.8%	57.0%
Medium Density 2 - Dry	7038	**6351**	**6010**	84.0	84.3	85.9	61.1%	76.1%
High Density 2 - Wet	8071	**7306**	**6991**	88.3	88.8	90.3	93.4%	107.6%
Low Density 2 - Wet	4249	**3816**	**4137**	77.9	78.1	80.0	9.1%	28.2%
Low Density 3 - Wet, Plate	4716	**3835**	**3332**	72.2	72.8	74.9	-47.4%	-18.7%
Low Density 4 - Wet, Plate	4092	**3332**	**3129**	73.0	74.4	76.9	-38.2%	0.2%


MS-5 Summary Chart
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Strain (in/in)
Low Density 1 - Wet
Medium Density 1 - Wet
Medium Density 2 - Dry
High Density 1 - Wet
Low Density 2 - Wet
Low Density 3 - Wet, Plate
Low Density 4 - Wet, Plate


S-5 Constrained Modulus
for 2.17 psi - 150.5 psi
Stress (psi)
200
180
160
140
120
100
80
60
40
20
0
Strain (in/in)
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
(1) y = 3358.1x
(2) y = 6012.9x
(3) y = 7038.2x
(4) y = 8070.5x
(5) y = 4248.9x
(6) y = 4716.3x
(7) y = 4091.8x
(1) Low Density 1 - Wet
(2) Medium Density 1 - Wet
(3) Medium Density 2- Dry
(4) High Density 1 - Wet
(5) Low Density 2 - Wet
(6) Low Density 3 - Wet, Plate
(7) Low Density 4 - Wet, Plate
Linear ((3) Medium Density 2- Dry)
Linear ((2) Medium Density 1 - Wet)
Linear ((1) Low Density 1 - Wet)
Linear ((4) High Density 1 - Wet)
Linear ((6) Low Density 3 - Wet, Plate)
Linear ((5) Low Density 2 -


MS-5 Constrained Modulus
for 2.17 psi - 87.5 psi
Stress (psi)
Strain (in/in)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.01
0.01
0.02
0.02
0.03
0.03
y = 3601.3x
y = 6412.6x
y = 6660.1x
y = 7721.8x
y = 4607.1x
y = 4097.1x
y = 3842.7x
(1) Low Density - Wet
(2) Medium Density - Wet
(3) Medium Density - Dry
(4) High Density - Wet
(5) Low Density 2 - Wet
(6) Low Density 3 - Wet, Plate
(7) Low Density 4 - Wet, Plate
Linear ((3) Medium Density - Dry)
Linear ((4) High Density - Wet)
Linear ((2) Medium Density - Wet)
Linear ((5) Low Density 2 - Wet)
Linear ((1) Low Density - Wet)
Linear ((6) Low Density 3 - Wet, Plate)
Linear ((7) Low Density 4 - Wet, Plate)



MS-5: e vs. log(σ
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
e
0.0
0.5
1.0
1.5
2.0
2.5
log (σ)
Low Density - Wet
Medium Density - Wet
Medium Density - Dry
High Density - Wet
Low Density 2 - Wet
Low Density 3 - Wet, Plate
Low Density 4 - Wet, Plate

Appendix D: *Bureau of Reclamation Data*

EARTH MATERIALS REPORT NUMBER (EM-)	DATE	SAMPLE INDEX				CLASS	FINES	SAND	GRAVEL	APPARENT SPECIFIC GRAVITY	MAX DENSITY (lb/ft^3)	MIN DENSITY (lb/ft^3)	PLACEMENT DENSITY (lb/ft^3)	PLACEMENT MOISTURE CONTENT (%)	LOAD (psi)	SETTLMNT % DRY	Ms Dry	SETTLMNT % Wet	Ms Wet
584	JUL 1960	29F	-	X	239	GW		21	79	2.65	145.5	108.7	132.0	0.0	0.7	2.7		2.7	
702	NOV 1964	37Q	-		60	GP-GM	7	14	79	2.45	142.6	114.4	128.5		0.7	2.7		2.8	
584	JUL 1960	29F	-		230	GW	2	22	76	2.64	141.3	113.2	131.5	0.0	0.7	3.0		3.0	
640	FEB 1962	18T	-		129	GP	4	21	75	2.56	142.2	114.7	133.0		0.1	0.5		0.6	
584	JUL 1960	29F	-		219	GW	1	25	74	2.65	140.5	110.6	129.8	0.0	0.7	5.6		5.8	
628	JUN 1961	36B	-	X	64	GW	5	23	72	2.53			128.8	0.0	2.1	3.5		3.6	
754	MAR 1968	49A	-		7	GP	4	26	70	2.49	137.4	115.5	129.4		0.7	2.0		2.2	
683	SEP 1963	39Z	-		9	GP-GM	11	22	67	2.52	139.4	118.1	134.5		100.0	4.2	2404	4.3	2315
615	MAR 1961	34Q	-		6	GM	14	20	66	2.55	135.1	107.8	134.9	5.4	100.0			2.8	3571
584	JUL 1960	29F	-	X	234	GW	2	33	65	2.67	133.7	109.1	125.2	0.0	0.7	6.5		6.7	
754	MAR 1968	49A	-		10	GW-GM	9	26	65		135.3	114.6	126.8	0.0	0.7	1.8		1.9	
594	OCT 1960	33T	-	X	8	GM-GC	14	24	62				131.2	10.7	20.0			1.0	2020
594	OCT 1960	33T	-	X	7	GP-GC	11	22	67				126.4	12.3	20.0			0.9	2222
700	OCT 1964	42K	-	X	17	GC	22	19	59				122.0	9.6	20.0	1.2	1724	1.2	1626
700	OCT 1964	42K	-	X	18	GC	20	21	59				114.0	10.4	20.0	1.6	1250	2.4	847
700	OCT 1964	42K	-	X	18	GC	20	21	59				117.0	10.8	20.0	1.7	1198	1.7	1156
700	OCT 1964	42K	-	X	18	GC	20	21	59				122.0	10.8	20.0	1.9	1070	1.9	1070
615	MAR 1961	34Q	-		7	GP-GM	10	32	58	2.56	135.8	112.1	132.1	5.8	100.0			2.1	4785
700	OCT 1964	42K	-	X	19	GC	26	16	58				97.0	11.6	20.0	2.3	866	2.4	840
602	NOV 1960	26R	-	X	31	GP-GM	23	20	57				134.4	6.9	1.0			0.3	
641	JAN 1962	20A	-		13	GP-GM	11	32	57		140.2	117.2	132.7		0.7	1.9		2.3	
637	MAR 1962	33G	-	X	48	GM-GC	31	13	56				111.7	16.1	1.0	0.1		0.1	
749	NOV 1967	45U	-	X	116	GM	17	28	55	2.19			113.4	14.9	20.0	3.5	571		
624	JUN 1961	33G	-		35	GC	21	25	54				113.6	12.8	1.0	0.8		1.3	
624	JUN 1961	33G	-		26	GM-GC	42	6	52				112.8	15.3	1.0	0.4		0.5	
754	MAR 1968	49A	-		13	GP	5	45	50	2.49	132.3	114.9	128.8		0.7	1.6		1.8	
740	SEP 1966	46E	-		11	GM-GC	26	27	47	2.40			126.0	9.1	100.0	2.9	3448	3.1	3226
693	JUL 1964	41P	-	X	57	GM-GC	21	36	43	2.68			134.8	5.7	100.0	3.1	3226	3.4	2941
749	NOV 1967	45U	-	X	120	GM-GC	33	24	43	2.41			120.8	14.2	25.0	5.3	472	5.4	463
615	MAR 1961	34Q	-		3	GC	21	38	41	2.38			128.3	7.5	100.0			2.4	4237
749	NOV 1967	45U	-	X	129	GC	25	35	40	2.44			122.3	12.2	20.0	1.8	1111	1.9	1053

EARTH MATERIALS REPORT NUMBER (EM-)	DATE	SAMPLE INDEX				CLASS	FINES	SAND	GRAVEL	APPARENT SPECIFIC GRAVITY	MAX DENSITY (lb/ft^3)	MIN DENSITY (lb/ft^3)	PLACEMENT DENSITY (lb/ft^3)	PLACEMENT MOISTURE CONTENT (%)	LOAD (psi)	SETTLMNT % DRY	Ms Dry	SETTLMNT % Wet	Ms Wet
733	FEB 1968	43G	-		1	SM	18	46	36				137.7	5.3	20.0	2.1	952	2.2	909
591	SEP 1960	29X	-		13	GC	39	27	34				118.1	13.8	20.0	0.4	5000	0.4	5000
749	NOV 1967	45U	-	X	121	GM	43	27	30	2.24			101.5	22.4	20.0	3.6	556	3.7	541
624	JUN 1961	33G	-		37	SM	34	47	19				112.6	16.9	1.0	0.1		0.1	
733	FEB 1968	43G	-		2	SC	46	35	19				127.4	9.9	100.0	2.5	4000	2.6	3846
640	FEB 1962	18T	-		130	SP-SM	11	70	19	2.52	118.8	94.7	112.9	0.0	0.1	3.6		3.6	
701	DEC 1964	37N	-	X	53	SM-ML	51	29	17				115.5	12.0	20.0	0.5	4000	0.5	4000
591	SEP 1960	29X	-		14	GC	29	55	16				112.6	16.2	20.0	0.5	4000	0.5	4000
582	JUL 1960	26W	-	X	76	SM	33	60	7		122.9	88.0	115.9	10.1	20.0			0.8	2500
640	FEB 1962	18T	-		78	SM	35	61	4				22.0	10.5	20.0	0.4	5000	0.5	4000
596	OCT 1960	18D	-		228	SM	37	61	2				118.6	8.2	1.0	0.1		0.1	
596	OCT 1960	18D	-		229	SM-ML	50	48	2				119.0	9.9	1.0	0.2		0.2	
743	FEB 1967	38N	-	X	68	SM	42	57	1				98.5	17.0	1.0	0.3		0.3	
743	FEB 1967	38N	-	X	68	SM	42	57	1				98.5	17.0	5.0			0.9	556
743	FEB 1967	38N	-	X	68	SM	42	57	1				98.5	17.0	20.0	0.3		1.4	1429
743	FEB 1967	38N	-	X	68	SM	42	57	1				98.5	17.0	40.0	0.3		1.9	2105
596	OCT 1960	18D	-		227	SM	39	60	1				110.1	11.0	1.0	0.1		0.2	
590	AUG 1960	33R	-		2	SM-ML	50	50	0				125.7	11.0	20.0	1.7		1.9	1053
596	OCT 1960	18D	-		230	SM	41	59	0				124.9	8.0	1.0	0.2		0.3	
596	OCT 1960	18D	-		231	SM	43	57	0				109.4	12.4	1.0	0.3		0.4	
597	OCT 1960	33G	-		8	SM	17	83	0				108.5	14.4	1.1	0.9		0.9	
702	NOV 1964	37Q	-	X	63	SM	26	74	0				115.3	9.2	20.0	0.4	5000	0.5	4444
754	MAR 1968	49A	-		3	SM	48	52	0				119.8	8.2	100.0	2.5	4000	2.7	3704
604	MAR 1961	15R	-		66	SM	14	86	0				113.0	13.6	20.0	0.4	5000	0.4	5000
604	MAR 1961	15R	-		71	SM	36	64	0				116.7	11.4	20.0	0.3	6667	0.3	6667
700	OCT 1964	42K	-	X	18								121.7	10.8	20.0	1.9	1070	1.9	1070
700	OCT 1964	42K	-	X	18								116.9	10.8	20.0	1.7	1198	1.7	1156
Portions of this Table were provided by Richard Young. Former BOR employee.																			