Engineering Properties and Maintenance of Golf Putting Greens

Annual Report
November 1999

James R. Crum, Thomas F. Wolff, and Leonard Becker
Michigan State University

1999 Objective: Correlation of Field and Lab Data

Introduction

The objective of the first phase of this research project was to apply engineering principles to the study of strength and stability in sand-textured root zones used for golf putting greens. In the second phase of this project, the primary objective was to evaluate existing golf putting greens and determine their basic geotechnical (soil) engineering properties. The final phase of this project is focused on developing an engineering model that will assist golf course personnel with the selection of sand base materials to be used for construction golf greens.

A deformation model was has been developed in the past year of study. It models the golf putting green as a soft spring layer (thatch) over a stiff elastic base (sand-based root zone). Given values representing the stiffness of the two layers and the size of the loaded area, the model will predict the vertical deformation of the soil as a function of load pressure. The required stiffness values can be estimated by field testing: values are selected by trial and error until the model prediction matches the observed pressure-displacement curve. The deformation model is an integral part of the analysis to date and will be the guiding factor as the recommendation model is completed.

Project Background

Literature Review

The first effort of the project was a literature review regarding the effects of sand particle size and gradation on strength. The results of sieve analysis for cohesionless soils may be presented as grain-size distribution curves. The diameter for which 10 percent of the sample by weight is finer (or sieve opening size for which 10 percent passes) is defined as the *effective grain size* D_{10}; the diameter for which 60 percent is finer is D_{60}, etc.. Then, the *uniformity coefficient* C_u is given as $C_u = D_{60} / D_{10}$. Larger values of C_u indicates the soil sample is *well-graded*, and contains a wider distribution of particle sizes. Previous studies provide conflicting results as to whether or not C_u has any impact on the strength of cohesionless soils.

Bishop (1948) tested a full range of cohesionless soils, ranging from sands to gravels and sandy gravels, in shear box tests. Only two samples are of interest here, breasted sand which is a well graded sand of the Folkeston bed ($C_u = 2.5$) and Ham River sand which is a uniform sieved fraction from the Thames Valley gravels ($C_u = 1.3$). It was observed that in the plot of porosity versus friction angle, the curves of two samples were almost parallel, with friction angle increasing with decreasing porosity. Due to lack of limiting porosities, the effect of C_u is not clear. Chen (1948) investigated the strength characteristics of cohesionless soils by using triaxial compression tests. He concluded that the friction angle of cohesionless soils increases with increasing uniformity coefficient.

Koerner (1970) studied the effect of gradation on the strength of cohesionless soils using three single mineral particles (quartz, feldspar and clacite). Gradation was evaluated by varying the uniformity coefficient (C_u) from 1.25 to 5. The conclusions from his study suggest that C_u has little effect on the strength of cohesionless soils.

Zelsko et al. (1975) performed triaxial tests using sand materials mainly consisting of quartz grains with a range of C_u values between 1.2 and 2.0. The conclusion was similar to Koerner's study, that varying gradation to increase Cu has little or no influence on ϕ.

Laboratory Testing

Laboratory testing focused on the effect of particle size, expressed as *median grain size* D_{50} and gradation, expressed as coefficient of uniformity C_u on friction angle ϕ and bearing capacity q.

Six gradations of sand were prepared; for each of three different D_{50} sizes (termed fine, medium and coarse), two gradations were prepared, a very uniform gradation with a low C_u and a more well-graded one with a higher C_u. In order to ensure consistency, these six sands were produced in the laboratory rather than directly using sands. These sands were made from a commonly available construction sand (MDOT 2NS) which has a wide range of particle sizes. To prepare the laboratory gradations, the 2NS sand was divided into a number of very narrow gradations by sieving; these were recombined to achieve the desired gradations for testing. As Figure 1 shows, all six of these test sands were designed to meet the USGA guidelines for golf putting greens.

Early strength testing was performed using a direct shear device; this has been reported on in earlier reports. As the loading of interest is vertical compression, a more direct measure of a soil's strength against failure under surface compressive load is its bearing capacity. This was directly tested in the lab by developing a modified California Bearing Ratio (CBR) testing device. This device has a circular plunger with a cross-sectional area of three square inches, which is forced into a sample volume of sand placed in a mold using a load frame. A load cell above the plunger displays the force pushing down on the soil sample. The depth the plunger has penetrated into the soil is measured with a dial gauge. Dividing the force by the piston area gives the applied pressure. Figure 2 indicates the developed pressure in psi as a function of plunger displacement in inches for a typical test. The bearing capacity, or ultimate pressure which the soil can withstand before it fails corresponds to the peak of the curve. Approximately 290 laboratory bearing capacity tests have been run on sand samples under a variety of test conditions.

The six experimental sands were tested under two different confining conditions: confined and unconfined. The confined samples were tested in the modified CBR device with a circular surcharge load plate above the surface of the sand. This donut-shaped plate has a center hole to permit the plunger to pass through. The unconfined samples were tested without the surcharge plate. The confined testing provides some indication of the effect of confinement such as that which the thatch layer provides for the root-zone sand. The thatch layer essentially acts as a membrane over the root-zone sand, which allows the root zone sand to undergo large deformations without a definite failure point. A comparison of the confined and unconfined bearing results are shown in Figure 3. As was expected, the confined lab bearing tests yielded higher ultimate bearing capacities than the unconfined lab bearing tests.

The bearing capacity tests show the benefits of sands with a high coefficient of uniformity (C_u). The confined lab bearing results given in Figure 4 shows that the well graded sands were capable of withstanding an ultimate pressure greater than those sustained by the uniform sands. For example the fine-well graded sand has an ultimate bearing capacity of approximately 265 pounds per square inch (psi), as compared to an ultimate bearing capacity of approximately 125 for the fine-uniform sand.

It should be reiterated that, although these sands display such a wide variety between their ultimate bearing capacities, they all fall within USGA gradation specifications and would be considered acceptable sands for golf putting green construction.

Field Testing

Field CBR Device

The field CBR device shown in Figure 5 is adapted from the original California Bearing Ratio (CBR) testing device. The CBR device can be pinned to the three-point hitch or clamped to loading bucket of most tractors. The device has a plunger which is pushed into the ground with a jack. A load cell with digital readout measures the force on the plunger. This force is recorded for a set of corresponding vertical displacements of the plunger into the ground, measured by a dial gauge clipped to plunger arm and measuring movement relative to a reference beam.

The force measured by the load cell is divided by the area of the load piston to obtain the pressure on the surface of the putting green. This calculation is performed for every increment of vertical displacement. Force is recorded at every 0.01 inch of displacement for consistency. The pressure at each 0.01 inch of displacement is plotted versus the vertical displacement as shown in Figure 6. The initial part of the curve, labeled A, represents the pressure causing initial deformation of the thatch layer. It is obvious from the graph and common sense that the thatch offers little resistance to deformation. The portion of the graph labeled B shows that increasing stresses are developed as the underlying sand-based root zone deforms under the thatch layer. The underlying sand requires significantly greater stresses to produce additional deformation.

As the putting green is loaded and then unloaded, some consolidation of the thatch and sand occurs. In the example in Figure 6, the sand and thatch deformed approximately 0.28 inches when subjected to a 17 psi load. When unloaded to 0 psi, a permanent deformation of about 0.20 inches remained. The permanent deformation can be estimated by taking the distance from the origin to the point where the tangent to the reload curve intersects the displacement axis. When reloaded, the stress – displacement curve will follow approximately the same line back to the 17 psi pressure since the thatch and sand have already "felt" that stress. Beyond the previous load of 17 psi, deforming the thatch and sand requires new, greater stresses, and will continue to consolidate until the sand begins to fail. If again unloaded, some elastic strain is recovered, and some permanent deformation remains. Engineers often refer to the load and reload curve as an elastic rebound curve.

Comparison of Field Bearing Tests and Laboratory Bearing Tests

The testing conditions in the lab are somewhat different than those in the field. In the lab there is no thatch layer covering the sand. Also in the lab, the sand is contained in a rigid mold that will not allow lateral deformation or strain of the sand. This leads to a well-defined peak stress at failure and a non-ambiguous bearing capacity. In the field, the thatch layer applies a tensile confinement that allow large magnitudes of deformation to occur at increasingly greater pressures on the sand without producing a well-defined peak stress at failure. Also, in the field, the sand-based root zone can strain or deform somewhat laterally, similarly reducing the tendency to exhibit a peak.

A comparison of typical lab bearing and field bearing results are shown in Figure 7. It is shown that the lab bearing results for both the confined (surcharge) and unconfined (no surcharge) bearing tests reach an ultimate strength and have a distinct peak stress and failure. The ultimate bearing capacity for the confined lab bearing test is approximately 198 psi, occurring at a vertical displacement of 0.15 inches. The field bearing test results shows no peak value failure and does not reach a specific ultimate strength. In fact, had the test been continued for larger pressures the soil would continue to deform at approximately the same rate and would not reach a distinct failure point. The sand-based root zone does not reach a distinct failure point because of the tensile confinement applied by the thatch layer. Also, the root zone material has the freedom to deform laterally and redistribute the pressure to the adjacent soil. Although the field and lab tests are not exactly equivalent, it is noted that the lab results with and without surcharge tend to act as upper and lower bounds, bracketing the field results.

It is also shown that the slope of the pressure-displacement curves, or rate at which the pressure increases with increasing displacement, is highest for the confined lab bearing test and lowest for the field bearing test. The high rate of increase in pressure due to increasing displacement for the confined lab test occurs because the sand is confined from both lateral deformation (due to the rigid mold) and vertical deformation (due to the applied surcharge). The root zone material is allowed to deform laterally, thus leading to its lower rate of increase in pressure due to increasing displacement.

Stiffness of Soils

An important characteristic of soils shown in the previous section is that soils can support loads and the magnitude of the peak supportable load, or ultimate bearing pressure, is determined by the physical properties of the soil and the degree of confinement. A second important property of soils often used by engineers is the *stiffness* of the soil. The stiffness of the soil is essentially a measure of how much pressure can be put on a soil at a certain limiting deformation. It is the rate of change in pressure due to increasing displacement. The soil modulus is described by the following relationship.

$$k = \Delta P/\Delta y$$

Here ΔP (psi) is the change in stress on the soil (in our case, the putting green) and Δy (inches) is the change in vertical displacement measured under the change in load. The units of k are pounds per inch cubed. The value of k depends on the elastic properties of the soil, but is dependent of the dimensions of the loaded area.

The typical bearing results represented in Figure 7 show that the confined lab bearing test yields the greatest stiffness and the field bearing test yields the lowest stiffness when comparing the three typical bearing test results.

Typically, geotechnical engineers study soils at their failure conditions, governed by local shear strength (ϕ) and by general shear failure under a loaded area. Engineers use a factor of safety and limit the allowable load to a third or a quarter of the ultimate bearing capacity to be conservative in their design. Although we are interested in what soil properties contribute to increased bearing capacity, we are more concerned about the behavior of the soil and golf putting green before failure occurs. An advantage to measuring or predicting the soil stiffness is that the deformation characteristics at pressures below the ultimate bearing capacity may be analyzed. Those greens with greater stiffness deflect less under load.

Deformation Model

To relate soil properties to deformation, a mathematical model was developed based on accepted principles of foundation engineering. The total deformation in the golf putting green can be taken as the sum of the elastic deformation in the thatch layer and the elastic and plastic deformation in the root-zone sand. Thus, the total deformation may be expressed as follows.

$$S_T = S_1 + S_2$$

where S_1 is the elastic deformation in the thatch layer and S_2 is the sum of the elastic and plastic deformation in the root-zone sand. The conceptual model of the golf putting green system is shown in Figure 8.

The elastic deformation in the thatch layer is determined by use of the deformation equation for a spring. The deformation of a spring is found by dividing the load applied to the spring by the spring stiffness. For our conceptual model the load must be converted from a pressure to a force and the stiffness is modeled by the elastic modulus of the thatch layer. The resulting equation for the elastic deformation in the thatch area takes the form:

$$S_1 = \frac{qB^2 \pi h_1}{4A'E_1}$$

where q is the pressure applied to the putting green surface, B is the diameter of the load source (i.e. testing plunger or mower tire contact area), h_1 is the undeformed thickness of thatch layer, A′ is the area upon which the load is applied, and E_1 is the modulus for the thatch layer. It is assumed that the stress is distributed through the thatch layer as shown in Figure 8, where the diagonal lines spreading the load are inclined at a slope of two vertical to one horizontal. Thus A′ is the area over which the load is applied at mid-height of the thatch layer.

The equation for the deformation in the root-zone sand (stiff, plastic layer) is estimated using Schmertmann's Method, a common geotechnical engineering approach for . The equation for deformation in the root-zone sand is as follows.

$$S_2 = \frac{q'}{E_2}\left(\sum I_z \Delta z\right)$$

Where q′ is the load applied at the thatch-soil interface, I_z is the is a strain influence factor, Δz is the thickness of sublayers used in the calculation and E_2 is the modulus for the root-zone sand.

Use of the Deformation Model

The deformation model was developed for a two layer system, a soft spring over a stiff layer, which is a good representation of a golf putting green. Although the model was developed for a two-layer system it can be applied to a one-layer system (i.e. lab bearing test). In this case the modulus, E_1, represents the apparent deformation that occurs due to play (error) in the testing device.

The deformation model was applied to both the field bearing results and the lab bearing results. The application of the deformation model to a typical field bearing test is shown in Figure 9. It is shown that the model fits the field bearing curve very well through the initial loading and reloading cycles. The initial loading and reloading cycles are the areas of most interest because they occur at pressures in the range in which we are interested (10 – 30 psi).

The application of the deformation model to a typical lab bearing test is shown in Figure 10. It is shown that the model effectively predicts the deformation of the lab sand under the applied load. The model begins to deviate from the lab bearing curve near the peak. This deviation is due to the fact that the model is predicting deformation of a plastic material (no definite failure point) and the lab sand is elastic in behavior. The deviation near the peak is not of great concern because we are only interested in the modulus of the soil (the slope of the bearing curve).

By trial and error matching, the deformation model was used to estimate modulus values for the lab tests. It is shown in Figures 11 that the modulus increases as the coefficient of uniformity increases for lab sands tested with and without vertical confinement. This shows that well-graded sands have a higher modulus than uniform sands. Therefore, well-graded sands will have less permanent deformation than uniform sands.

Unfortunately, the field data did not give as clear a trend for the relationship between coefficient of uniformity and the modulus. The scatter shown in Figure 12 is likely due to the variability in constituents that make up the base soils for the various putting greens that were tested. It should be noted that scatter is expected when testing is being performed in the field because it is impossible to control all the variables that contribute to the result. However, the general order of magnitude of the field moduli are reasonable in light of the lab test results and published typical values for sand.

Findings

Initial findings suggest that golf putting greens can be modeled as a soft spring over a stiff base that has some modulus, *E*. The modulus of the root-zone sand increases with higher coefficient of uniformity, C_u. Field tests show that the stiffness of the green is dependent on soil properties but it also has increased ductility due to tensile confinement applied by the thatch layer (i.e. the sand base can undergo large deformations with no defined failure).

Remaining Work

Over the next few months, the data collected from field and lab testing will be synthesized to develop a recommended model to assist superintendents in choosing appropriate base materials for newly constructed golf greens. It has been found that well-graded sands are stiffer than uniform sands, which means they will have less permanent deformation under loads that are typically applied to golf putting greens.

The recommended model will likely show that an adjustment in the USGA gradation specifications that will allow more particles in the range of .05 to .25 mm (material that would be retained on a No. 70 sieve) would increase stiffness and reduce deformation potential. In developing the final recommendations, the effects of such a gradation adjustment on the hydraulic conductivity of the base sands will be considered to ensure the drainage characteristics of the base material are not jeopardized.

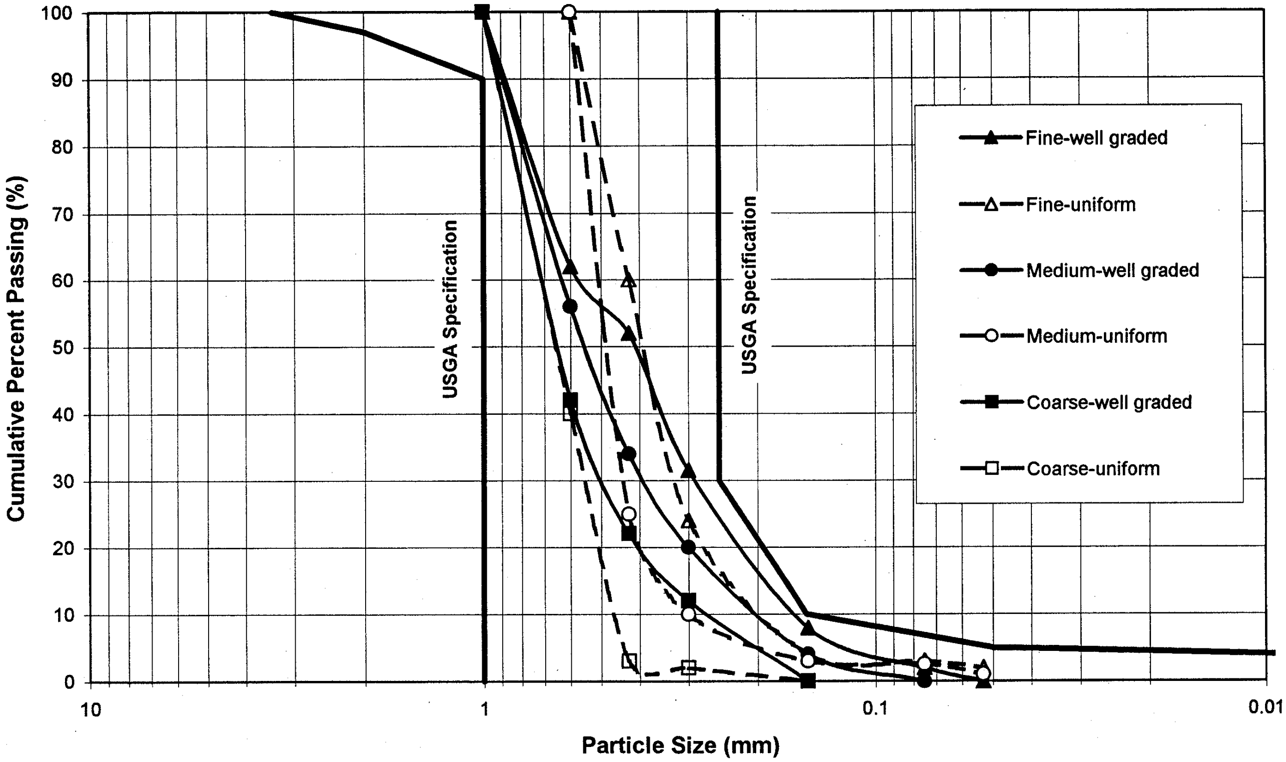

Figure 1: USGA Gradation Specification Bands and Experimental Sand Gradations

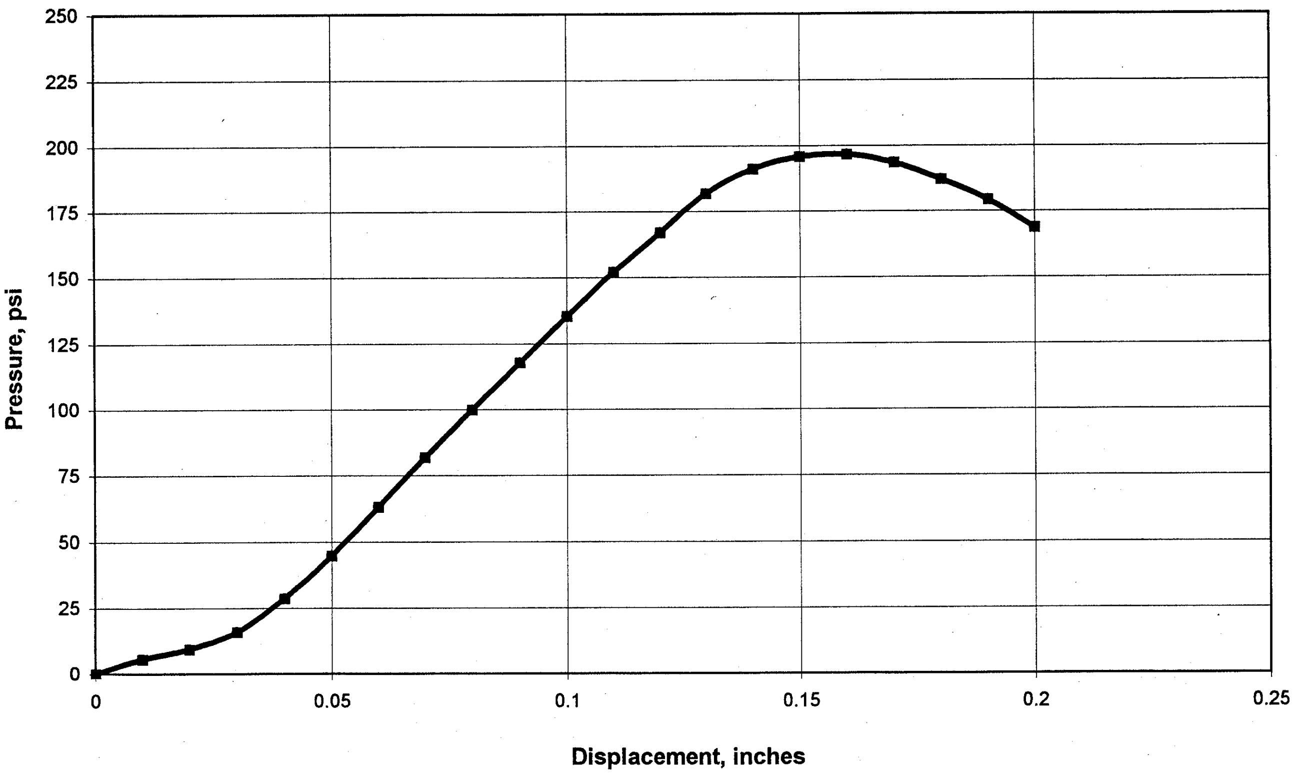

Figure 2: Lab Bearing Test Results

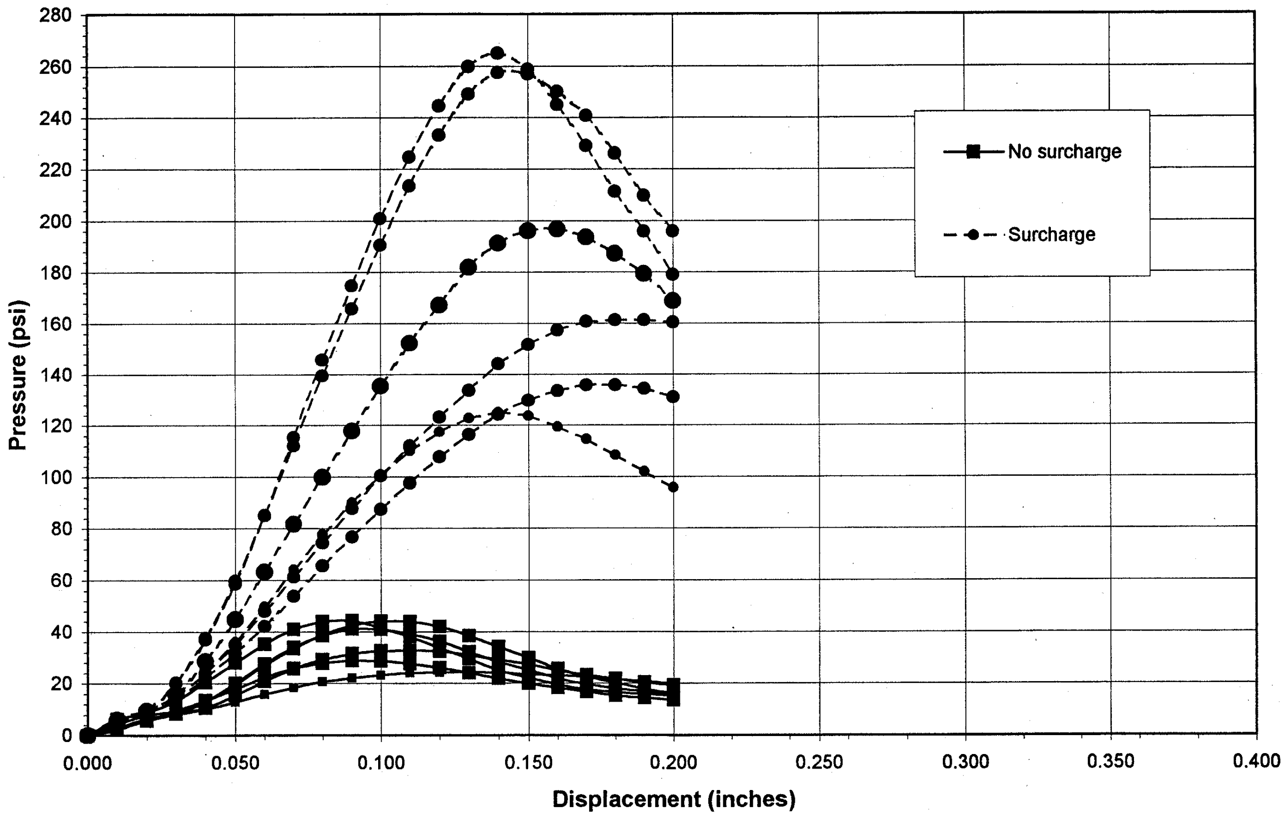

Figure 3: Comparison of Lab Bearing with Surcharge and without Surcharge

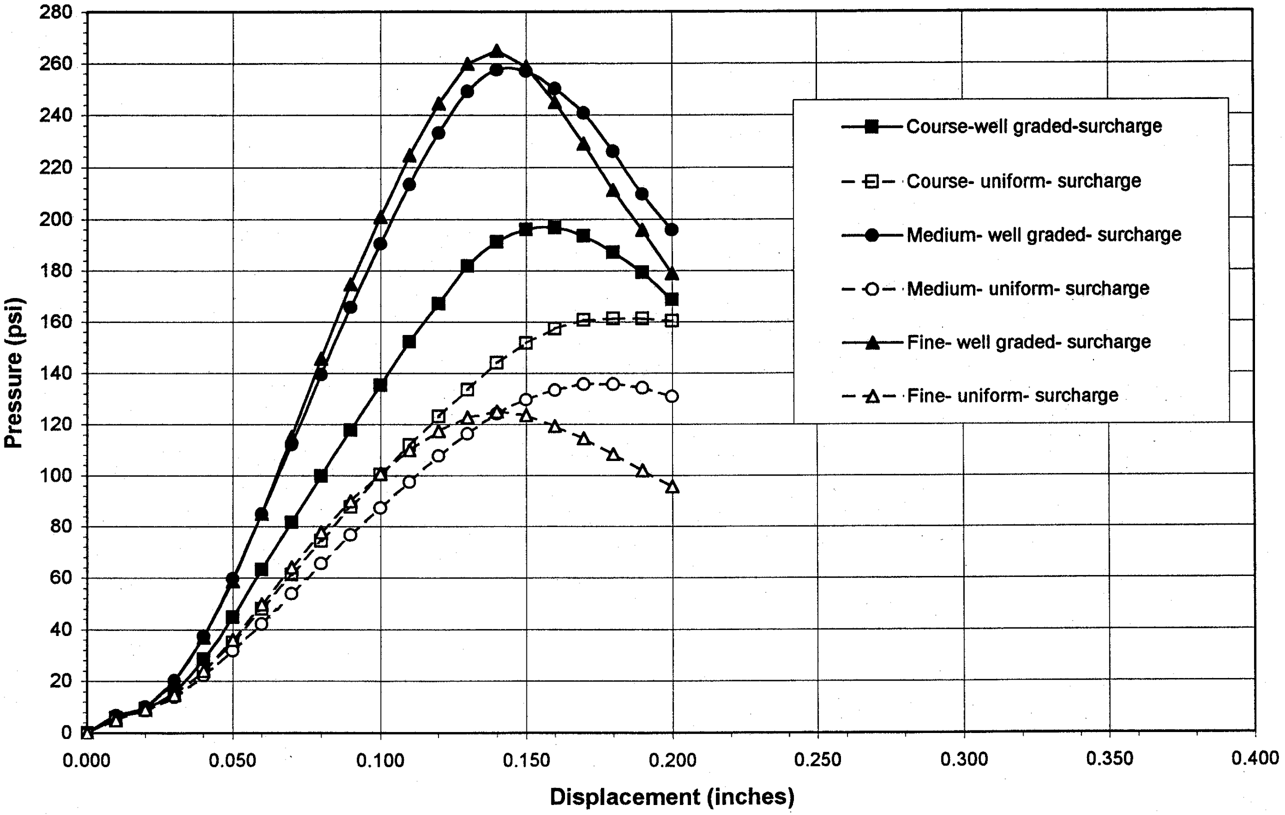

Figure 4: Laboratory Bearing Results with an Applied Surcharge

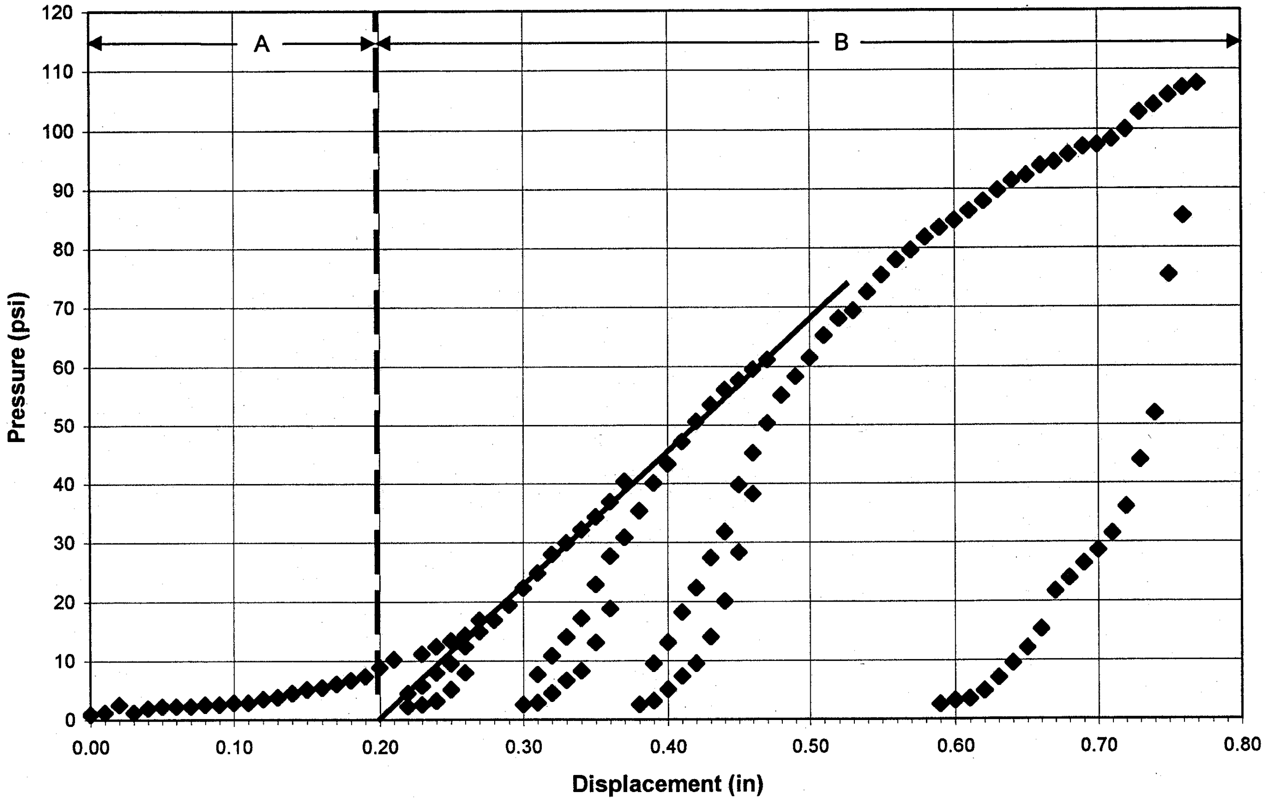

Figure 6: Field Bearing Test Results

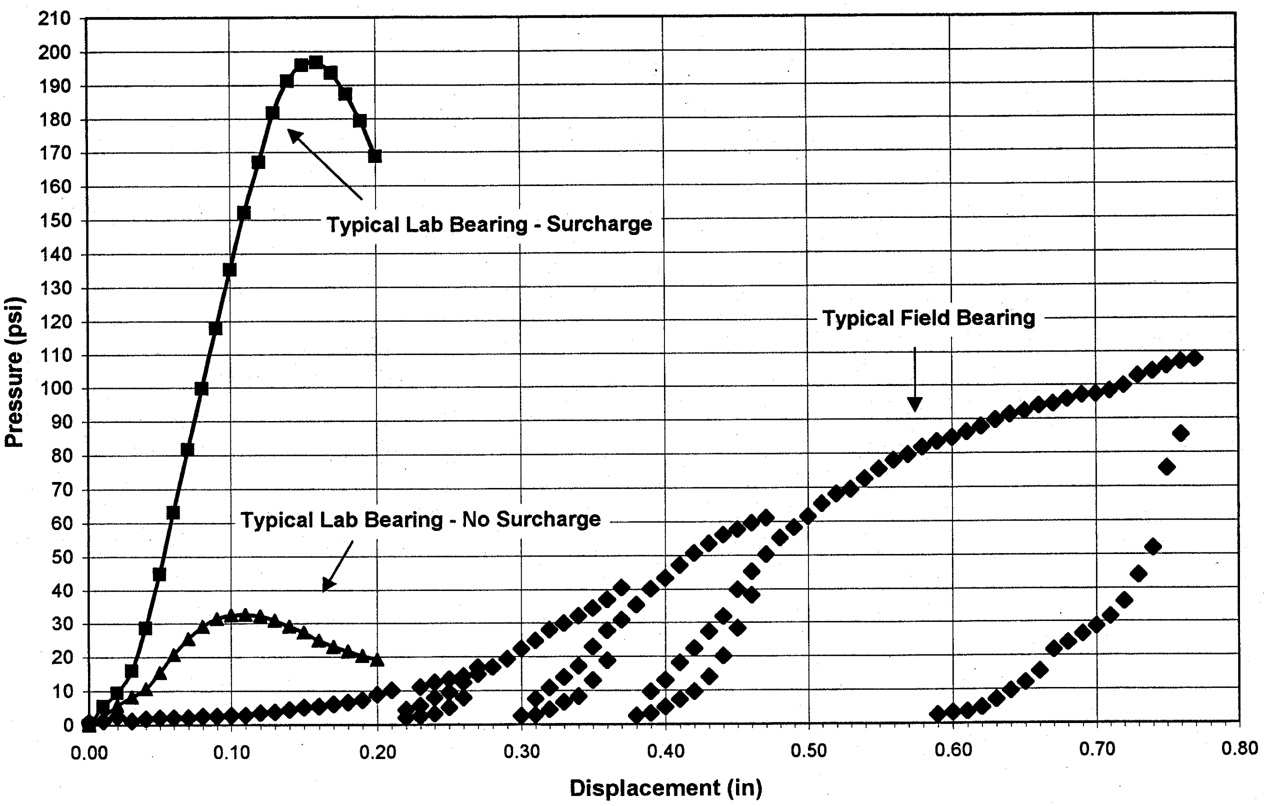

Figure 7: Comparison of Lab Bearing and Field Bearing Test Results

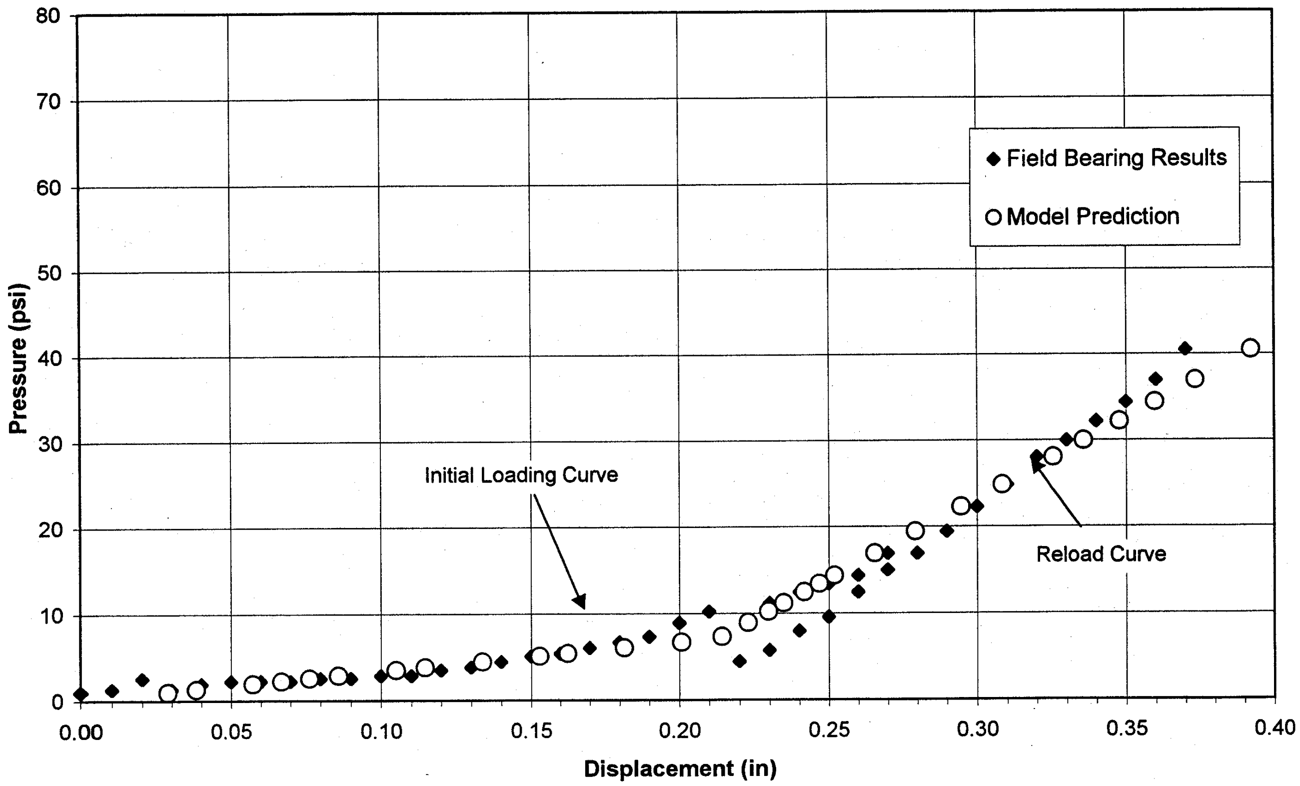

Figure 2.15: Application of the Deformation Model to Field Bearing Results

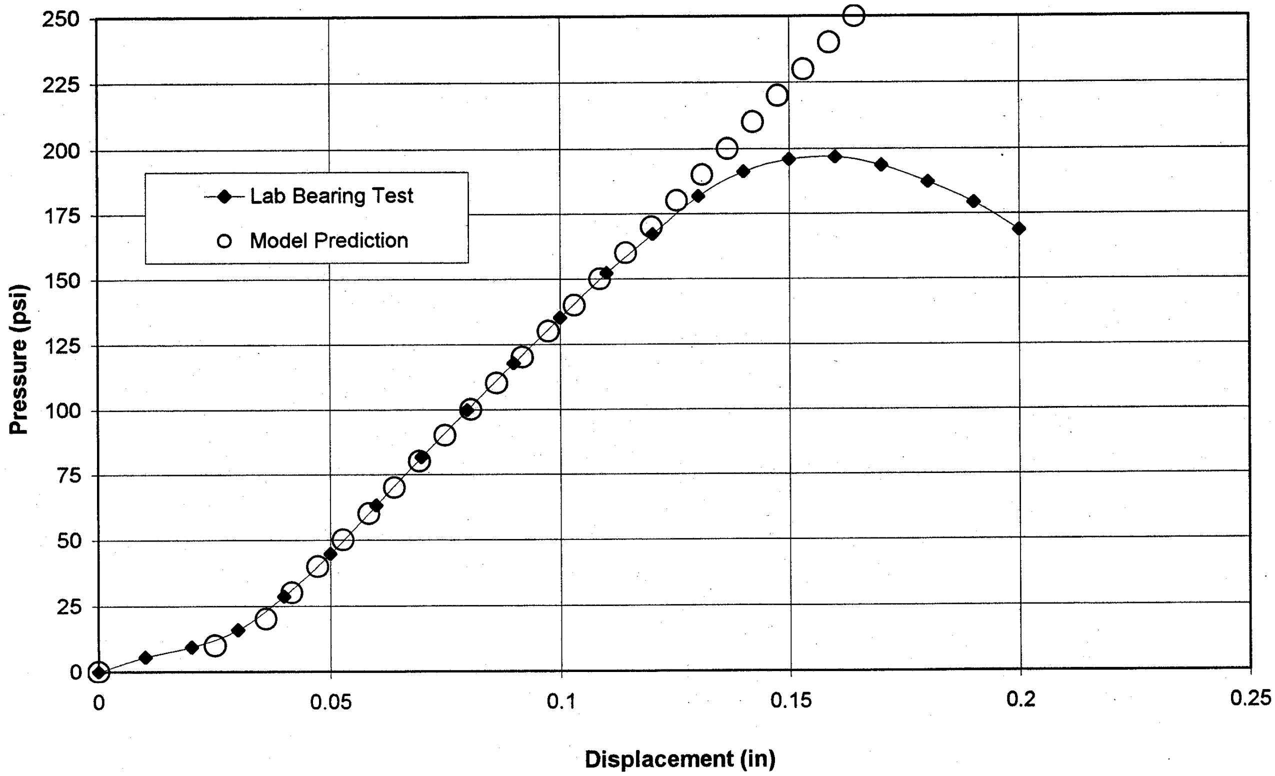

Figure 10: Application of Deformation Model to Lab Bearing Results

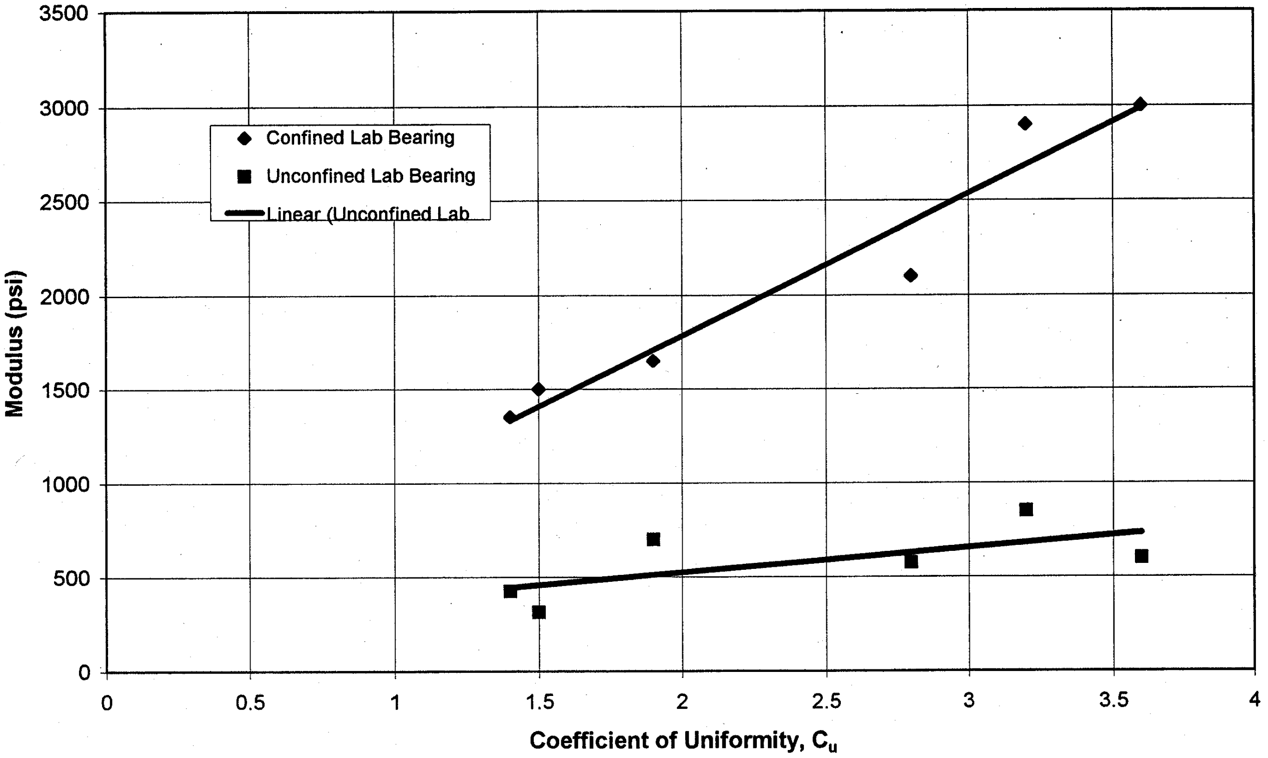

Figure 11: Modulus as a Function of Coefficient of Uniformity for Lab Data

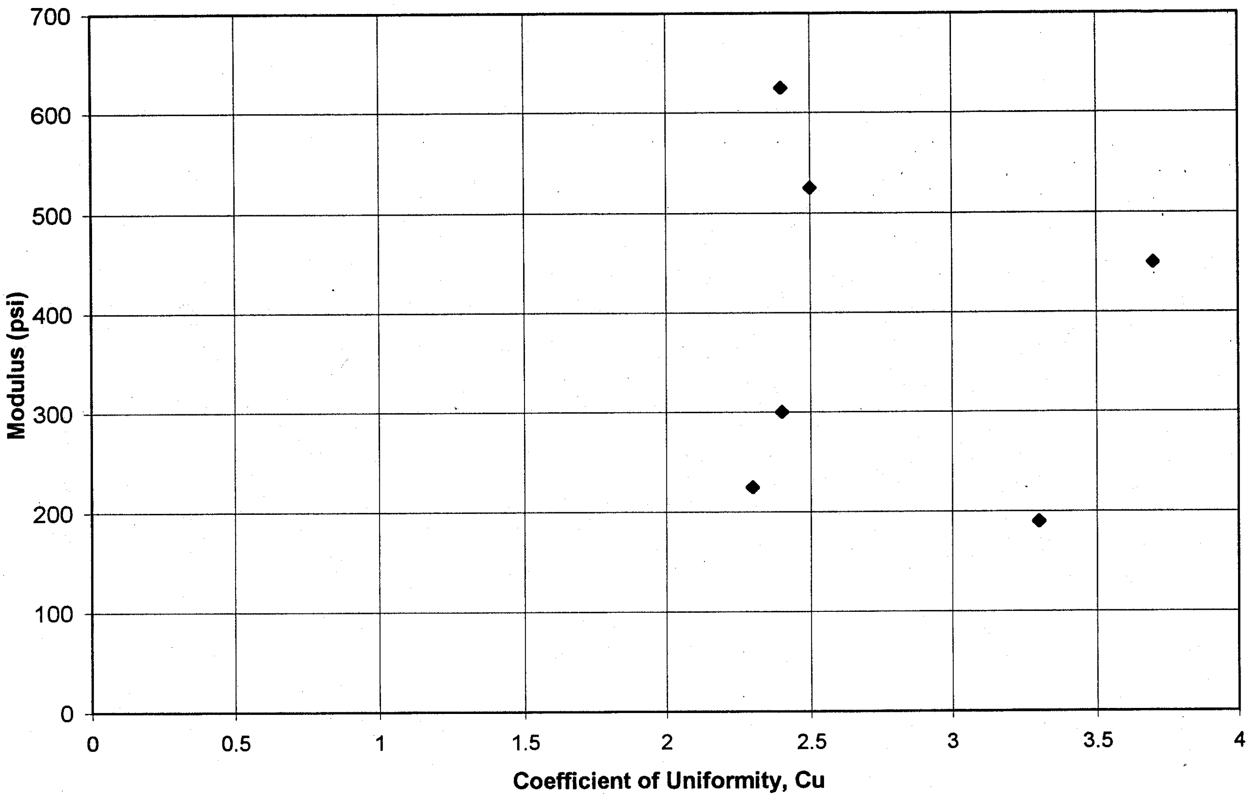

Figure 12: Modulus as a Function of Coefficient of Uniformity for Field Data