Final Report
October 20, 2008

Comparison of the Soil Stress Gage with Performance Based Results using the Heavy Vehicle Simulator

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Disclaimer

The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the State of Florida Department of Transportation.

SI (MODERN METRIC) CONVERSION FACTORS (from FHWA)
Table 0-1. Approximate conversions to SI units

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	Mm
ft	feet	0.305	meters	M
yd	yards	0.914	meters	M
mi	miles	1.61	kilometers	Km
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	Square inches	645.2	square millimeters	mm^2
ft^2	Square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	Ha
mi^2	square miles	2.59	square kilometers	km^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	Ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	G
lb	pounds	0.454	kilograms	Kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	Lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
lbf	Pound force	4.45	newtons	N
lbf/in^2	Pound force per square inch	6.89	kilopascals	kPa

Table 0-3. Technical Report Documentation Page

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<tr><td colspan="5">16. Abstract
A limited field and laboratory study has been performed to quantify the effects of compaction on base course performance. The Heavy Vehicle Simulator was used in addition to buried soil stress gages to measure surface stress distributions. Numerical analysis was used in an attempt to predict performance based on under compaction. Results show that varying the modulus of the subbase layer(s) had a minimal effect on base course performance.</td></tr>
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EXECUTIVE SUMMARY

PROBLEM STATEMENT

Traditional pavement base course performance predictors include density and LBR. While density would at first inspection seem to provide a positive correlation to a well performing, i.e., stiff or rigid material, this premise is now subject to further assessment. The previous statement can be taken to the extreme using mercury as an example. While mercury is 13.6 times denser than water (or 6.5 times denser than a dense soil), its stiffness is virtually zero. However, the Soil Stiffness Gage or SSG, has recently been developed that measures rigidity of the soil rather than density to predict performance.

OBJECTIVES OF THE RESEARCH

This new stiffness technology may provide a very powerful tool for highway designers/constructors. In fact, currently the FDOT has several units in the field and lab attempting to demarcate the variability of both the device, the operator and material type effects. In addition, the University of Florida recently completed an FDOT sponsored project whose objective was to evaluate the instrument and suggest possible new design enhancements. While this effort indicates that the instrument can be operated at a confidence level necessary for successful field usage, it is critical to evaluate the SSG's data relative to actual pavement/base performance.

In addition, incorporation of mechanistic design (M.E.) methodologies call for modulus based input parameters. Measuring these parameters in the field would validate design values/assumptions.

Thus, the objective of this project was (and still is) is to develop a statistically valid mechanistic procedure, to predict base performance as a function of subsurface compactive stiffness/modulus - thereby improving an existing process.

FINDINGS

To conduct multiple tests for statistical analysis, the HVS or Heavy Vehicle Simulator was used to apply multiple wheel loads typical of heavy truck traffic. This concept is to load compacted material (typical subbase and base material) in the two large outdoors test pits to failure. The effect of degree of compaction, spatial variability of problematic zones and, since the water table can be raised or lowered during an actual test, the effect of moisture content on both SSG and performance criteria was to be evaluated. Since it is well documented that water affects the SSG results, the combination of the HVS, test pits and SSG devices provides an exceptional opportunity to evaluate the instrument's potential use for both roadway design and QC material acceptance.

While the project was terminated prematurely, preliminary results indicate that under-compaction of subbase material has a limited adverse effect on the deformation of the base course due to wheel loading.

CONCLUSIONS

Unfortunately, due to situations beyond the Principal Investigator and FDOT's control, the proposed project objectives could not be completed. This is due to a variety of issues involving the test pits - crucial to the successful meeting of the objectives. A enormous amount of effort was expended by FDOT to remedy the delays. Unfortunately, due to the uncertainty of future use of the test pits, the project objectives were altered and computer simulations performed in order to attempt to create a model that predicts field performance. While this was tentatively achieved, it is the intent of the PIs and Project Manager to continue studying this topic and hopefully be able to validate the proposed model sometime in the future when additional field testing is conducted.

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CHAPTER 1.
ORIGINAL PROPOSED WORK PLAN

DEVELOPMENT AND MANAGEMENT OF COMPACTION QUALITY CONTROL GUIDELINES THAT ACCOUNT FOR VARIABILITY IN PAVEMENT EMBANKMENTS IN FLORIDA

The original proposal is outlined below and provides an overview of the intent of the research.

Scope of Work

We propose the following tasks be implemented in order to achieve the stated research objectives. The tasks are subdivided into short-term and long-term or comprehensive goals.

Short Term Goals

TASK 1S. Begin immediate assessment of the Soil Stress Gage (SSG) under controlled conditions. Since another, unrelated FDOT project is investigating the capillary rise phenomena in A-2-4 material by varying the fines content, we propose to also conduct SSG tests in the 8 ft. by 8 ft. test pit. The 6-6inch lift sections (total depth 3 ft.) should provide ample depth to preclude boundary effects. A series of tests will be performed for the various moisture conditions, including: effects of surface preparation (e.g., sand layer versus scarified condition), plumb-ness of unit and test repeatability). Upon completion of these tests, a nuclear density (ND) test will be conducted for comparison. Additionally, a sand cone [SC] (or balloon) density test will be performed. For each series of capillary tests, the above methodology will be followed. It is evident that moisture content plays a significant role in the SSG interpretation. In fact, the FDOT has confirmed that the manufacturers intend to include some type of moisture content sensor with their SSG unit to increase its accuracy. Since this is not available as yet, we propose to purchase a sensor that will rapidly determine the soil moisture with depth. The details of the device are attached for your perusal. Finally, an FDOT sponsored Technical Report published in 1983, compared the sand cone versus nuclear density. It found a substantial variation within the sand cone results. We have requested a copy of this report and will examine its findings. It is relevant to point out that MDOT has adopted the sand cone test for its standardized density measuring procedure.

TASK 2S. Concurrent with the above, the data will be analyzed – specifically in terms of correlations between SSG and sand cone vis-à-vis sand cone and nuclear density. If a strong correlation exists between SSG/sand cone data, then a poor relationship between SSG and nuclear density indicates the ND may not be a creditable benchmark from which to assess SSG viability. For all data, variability within each test protocol will be examined, so as to confirm or refute its statistical viability.

TASK 3S. A design of a surface preparation tool that will assure consistent SSG test conditions will be produced. Conceptually, the device (shown below) will include a handle with spring assembly that will provide a constant downward force to a circular scarifying plate. Rotation of the device will prep the soil by smoothing the surface as well as leveling the surface.

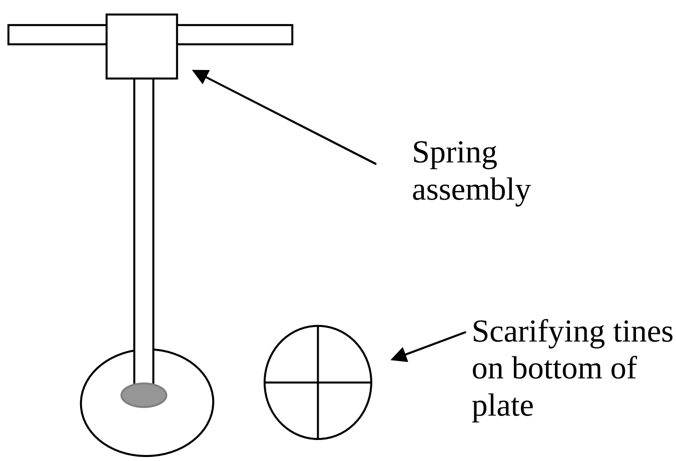


Figure 1-1. Surface Preparation Device

TASK 4S. Once the above tests are completed (or near completion), a tentative SOP will be produced for SSG operations. These suggestions will incorporate the Humboldt instructions and more standardized surface preparation procedures.

Long Term Goals

Based on the preliminary results of the above testing, further directed testing will be performed. During this phase, the SOP developed above will be continually examined and minor adjustments made. Specifically, the following tasks are envisioned.

TASK 1L. Using the test pit, proceed to place uniform soil layers (in 6" lifts) and conduct SSG, ND and SC tests. The goal of this task is to confirm the effects of surface preparation and to evaluate spatial variability. The lifts will be placed at or near optimum moisture content – thereby simulating actual field practice. Concurrently, at least 2 – 4 (depending on available staffing) plate load tests will be conducted. The rationale for these tests is to establish a correlation between SSG and soil moduli.

TASK 2L. Subsequent lift properties will be varied in terms of composition (A-3, A-2-4, etc.) - however, horizontal homogeneity will be preserved. For each lift, the tests outlined in TASK 1L. will be conducted. By repeating the above tests for each lift, the effects of soil type will be evaluated.

TASK 3L. Once TASK 2L is completed, additional tests will be performed to measure the effect that water has on the accuracy of the SSG results. This task may be canceled or reduced in scope depending on the short-term TASK 1S conclusions. If additional tests are to be conducted, 3 – 4 soil types (ranging from poor to good performers) will be placed in the test pit [in 8 ft. by 8 ft. sections] and three test conditions created; optimum, saturated and drained. SSG, ND, plate load tests and SC tests will refute or confirm moisture content effects on the stiffness and moduli. While it is implicit that moisture content will affect the SSG results, if a reliable trend can be determined, then we will provide the FDOT with a reduction factor (or factors) for the above conditions (i.e., soaked).

TASK 4L. Make recommendations to the FDOT so that they may make a decision on a best management practice for contractor conducted testing. (QC 2000 criteria).

As can be seen from the previous tasks, the scope was both extensive and comprehensive. However, due to a plethora of issues, the Project was delayed numerous times. This adversely impacted the anticipated progress and ultimately its termination. The following provides a timeline of the aforementioned project constraints.

A. A previous progress report cited the purchase of the embeddable soil stress gages. These were acquired and were ready for insertion into the test pits. However, prior to installation, the base course had to be constructed. Attempts to locate a contractor willing to perform this small project were problematic. Over twelve months were spent contacting several contractors, until Williston Concrete Inc. agreed to the limited scope task. **(Time frame: July 2002 – December 2003)**

B. The first task was to correlate the number of passes of the compaction roller with soil density. Hence, a typical scenario would be for the contractor to fill the pit, and conduct a limited number of passes. After this, both SSG and nuclear density tests were performed to measure the resulting stiffness and density. After numerous tests, it was determined that 11 passes were required to obtain 95% maximum stiffness and a baseline of 98% modified density was chosen. This was an important finding, since the objective of the research is to create base courses that are lower than the maximum stiffness (i.e., 50%, 70%, 80%) and then note their effect from actual wheel loading (using the HVS). These results would then be compared to conventional compaction control (i.e., meeting current specification standards; 98% of maximum density as determined by Modified Proctor.) The HVS was then used to observe the rutting progression. Two series of test pit tests were conducted, the first in October through December, 2004 and the second in June through July, 2005) These results are shown in the Power Point presentation later in this report. **(Time frame: January 2003 – June 2005)**

C. Based on the above test results, the first trial was set up. This involved applying a lift of soil (12”), inserting several stress cells, and compacting the material. This process was repeated until the proper level of the base course was achieved. It was determined that eleven passes produced 100% of the maximum stiffness from **B**. **(Time frame: March 2004 – December 2004)**

D. Since the Heavy Vehicle Simulator (HVS) is continuously used for other research, we had to wait until it was available. However, it was finally freed up and carefully positioned over the pit. The loading was initiated and the plan was to run it continuously, stopping only to measure the amount of wheel settlement (or soil rutting) into the base material. However, during the first night of operation, it began to rain and unfortunately, the HVS continued to operate. The combination of an ingress of water from both the actual rain as well as runoff from the adjacent pavement softened the base to such an extent that the HVS virtually destroyed the soil surface. This large amount of wheel settlement, in turn, resulted in higher contact stresses, thereby damaging a number of the embedded stress cells. The primary reason for this occurrence is due to the fact that the pit(s) are not covered and hence cannot be protected from rain. A second problem is that rain water runoff from the adjacent pavement, flows into the pit(s), exacerbating water infiltration. **(Time frame: January 2005 - March 2005)**

E. The contractor was contacted and requested to remove the soil and re-compact so that the test could be repeated. Unfortunately, this was a very frustrating period, since FDOT personnel continually contacted him and was given multiple assurances that he would complete it. After it became apparent that he was not able to comply, (which continued for 14 months) he defaulted. **(Time frame: April 2005 – April 2006)**

F. Based on the inability to properly control moisture content in the base material (a critical element in the research plan), a search was begun for an engineering firm to redesign the surrounding areas to allow for proper drainage and runoff. In addition, a cover was included in the specifications. Greiner Engineering was awarded the contract and completed the design. It includes a grate and ditch to channel water away from the pit and a removable cover. **(Time frame: May 2006 – September 2006)**

During the interim between September 2006 and September 2008, an ancillary task was implemented to look at the effects of laboratory compaction on different types of soils. Historical data showed that the modified compaction sometimes yielded lower dry densities compared to the standard Proctor test. Of course, this would affect the field compaction QA/QC documentation, since it might be possible for a contractor to achieve a specified density compared to a lab test that was inherently too low.

Due to delays in the test pits, the research team decided to conduct additional lab testing and model analysis to investigate the lab compaction test results and field stiffness/modulus on long term performance. This report is attached in Appendix A.

CHAPTER 2.
GENERATION OF LIMITED FIELD DATA

Due to the delay in completing the outside test pit drainage renovations, the PROJECT MANAGER suggested that the Principal Investigators attempt to utilize a software program to calibrate the limited field data available. This would then be used to predict base performance in future tests.

The first effort was to determine the optimum number of passes of a roller to yield the highest base stiffness. It is important to note that stiffness does not correlate to density and hence one of the major objectives in roadway construction is to reduce the dependence of density results on field performance. However, determining stiffness is not yet generally accepted by the practitioners - thus the reliance on nuclear density measurements.

As can be seen from the plot, 14 passes provided the greatest stiffness. Thus, the proposed testing would be to vary this and note the effect on rutting.

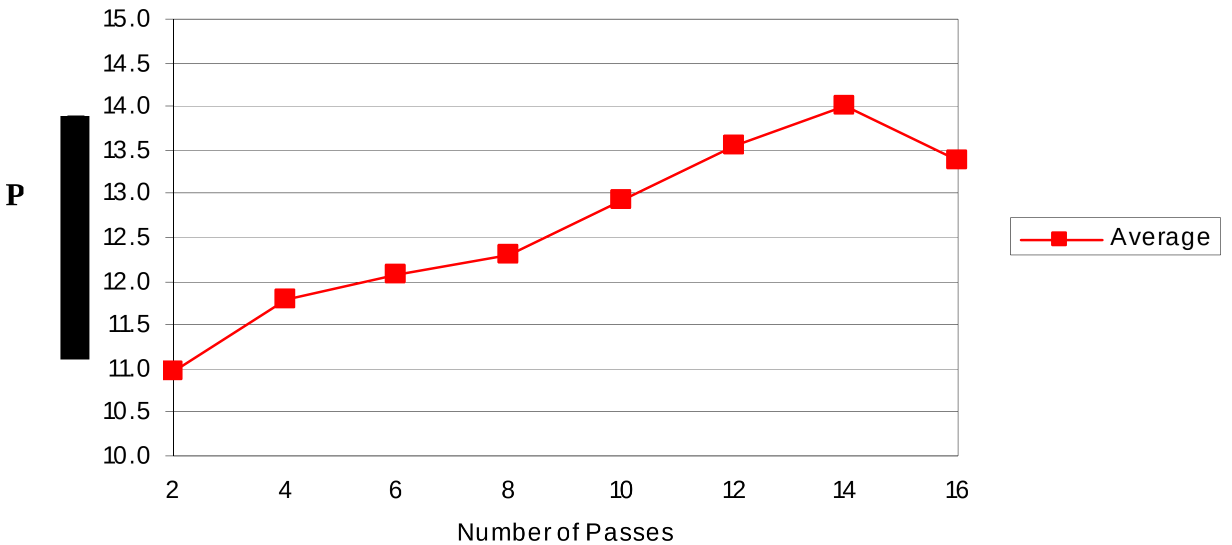


Figure 2-1. Field Test to Determine Maximum Stiffness as a Function of Compactive Effort

The figure below shows the layout for the first series of tests.

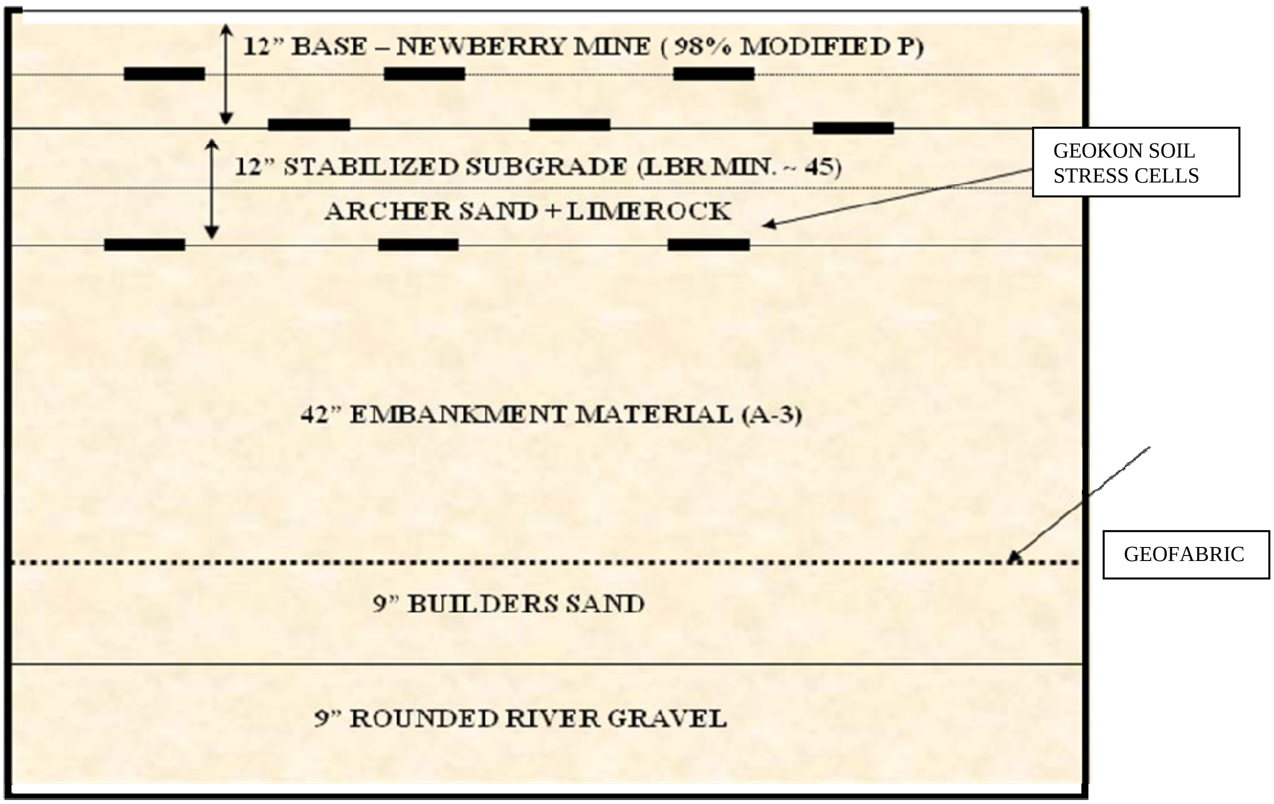


Figure 2-2. Test Pit Cross-section – Preliminary Trial Run Setup

After compacting the material, the HVS was placed on top of the test pit and run 700 passes. The resulting rutting depths were measured and are shown in the following figure.

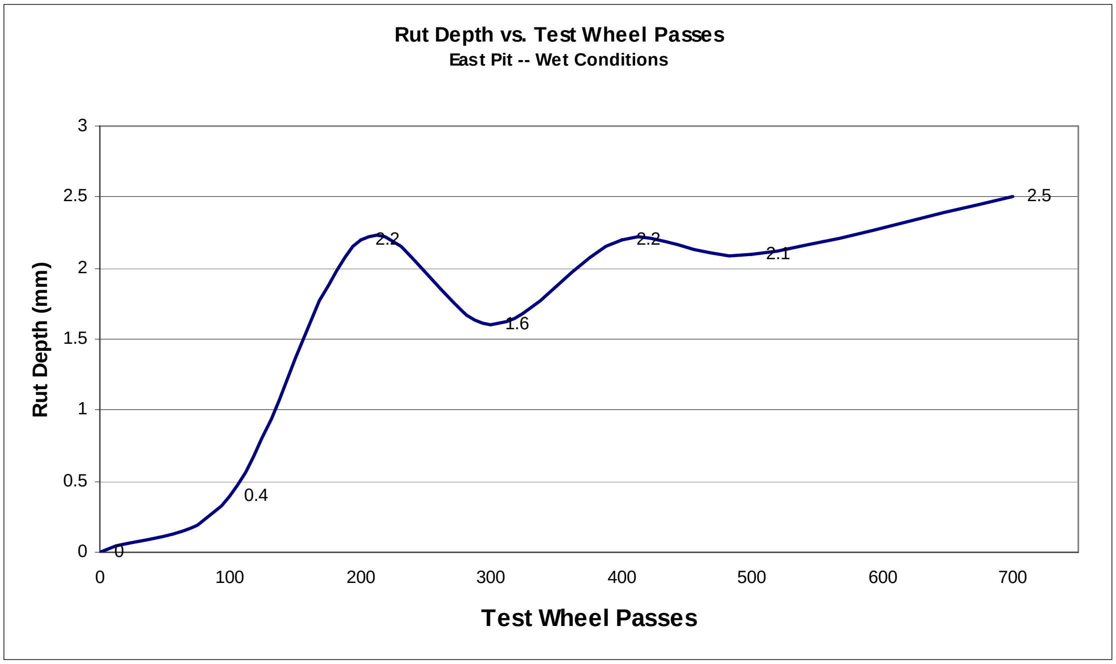


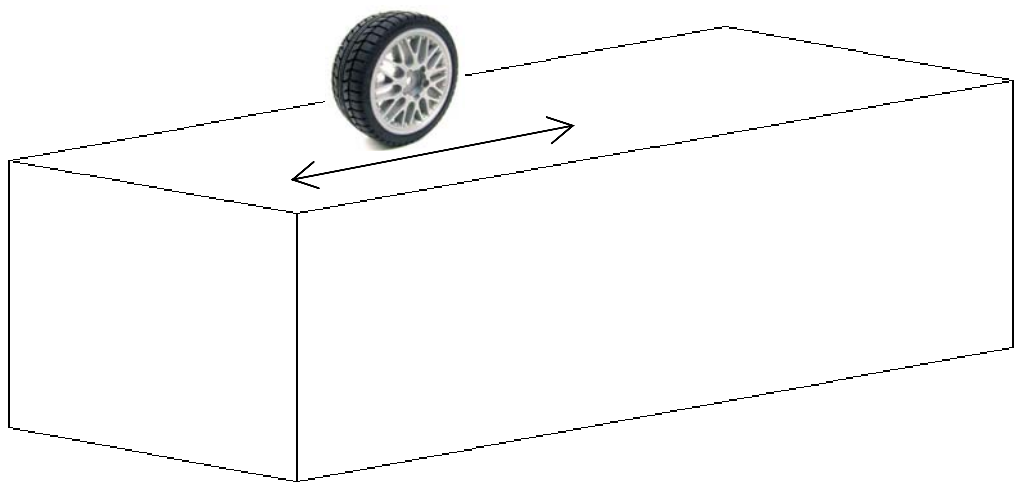
Figure 2-3. Typical Rut Depth for Wet Conditions

Further testing under dry conditions resulted in rut depths between 3 and 6 mm. This range is again within a narrow window and provided actual field results for comparison.

Armed with this limited information, the numerical analysis was attempted.

CHAPTER 3.
NUMERICAL ANALYSES

Plaxis software was used for the modeling. The input data is shown in the table following the set of figures.



Note: A Tire Pressure = $80\,psi = 550 kN/m^2$ was used in the analysis.

Figure 3-1. HVS Wheel Load Set Up on Base Material

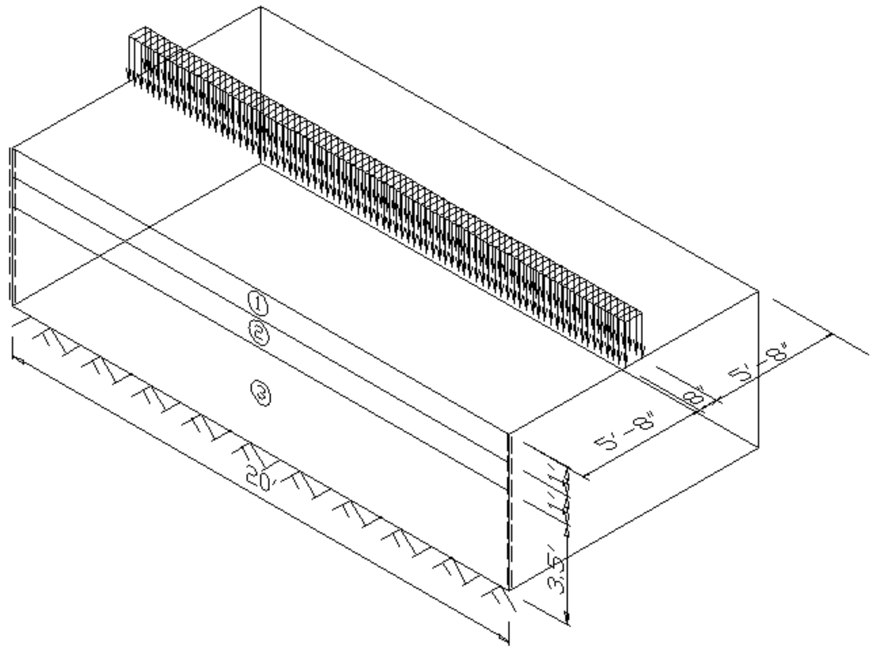


Figure 3-2. 3D Model of the Loading Configuration

The following soil properties, gleaned from the preliminary data were inputted into the program.

Table 3-1. Soil Properties (metric)

	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	E_{ref} (kN/m^2)	ν	Thickness (m.)	c_{ref} (kN/m^2)	φ $(^\circ)$	ψ $(^\circ)$
Base material	20	20	241316.5	0.25	0.3048			
Stabilized Subgrade	17	20	117210.8	0.30	0.3048	1.0	35.0	2.0
Embankment	16	18	62052.8	0.35	1.0668	5.0	30.0	0

Table 3-2. Soil Properties (US customary)

	γ_{unsat} (PCF)	γ_{sat} (PCF)	E_{ref} (ksi)	ν	Thickness (in.)	c_{ref} (psi)	φ ($^\circ$)	ψ ($^\circ$)
Base material	127	127	35	0.25	12			
Stabilized Subgrade	108	122	17	0.30	12	0.145	35.0	2.0
Embankment	102	115	9	0.35	42	0.725	30.0	0

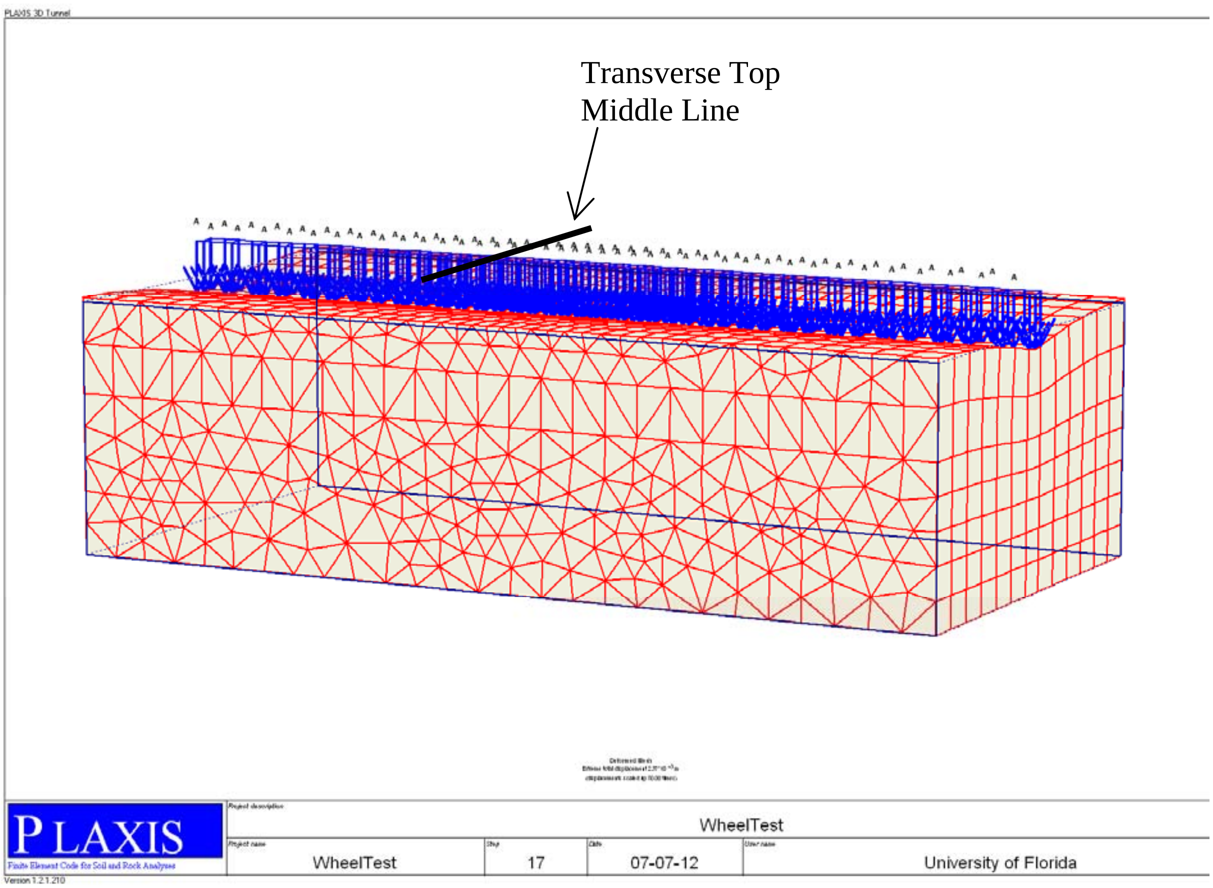


Figure 3-3. 3D Base Deflection

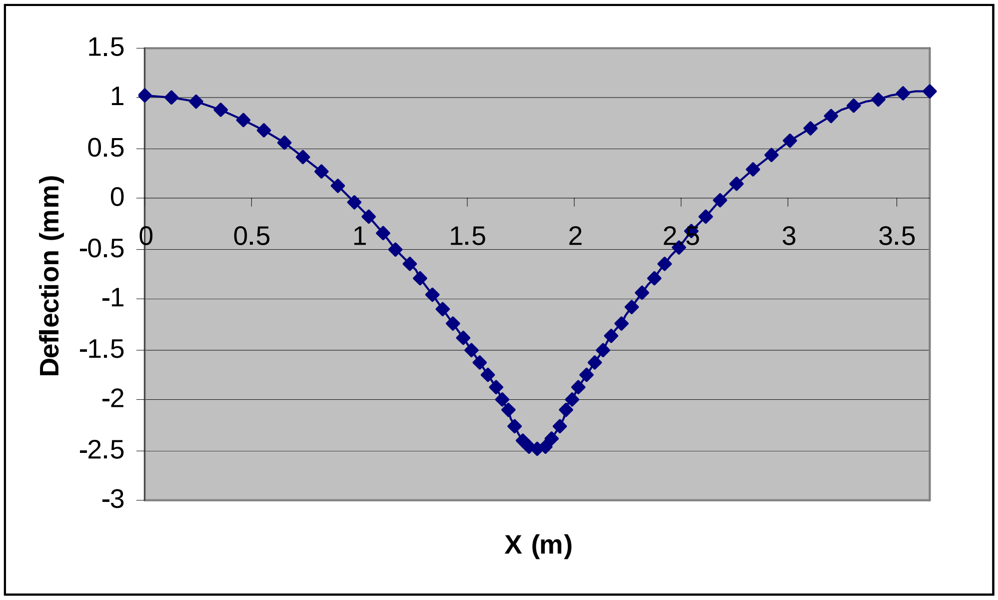


Figure 3-4. Transverse Deflection Using the 3D Model

As can be seen from the plot, a maximum of 2.5 mm was obtained from the simulation. This is in line with the field data, showing that the model provides reasonable results.

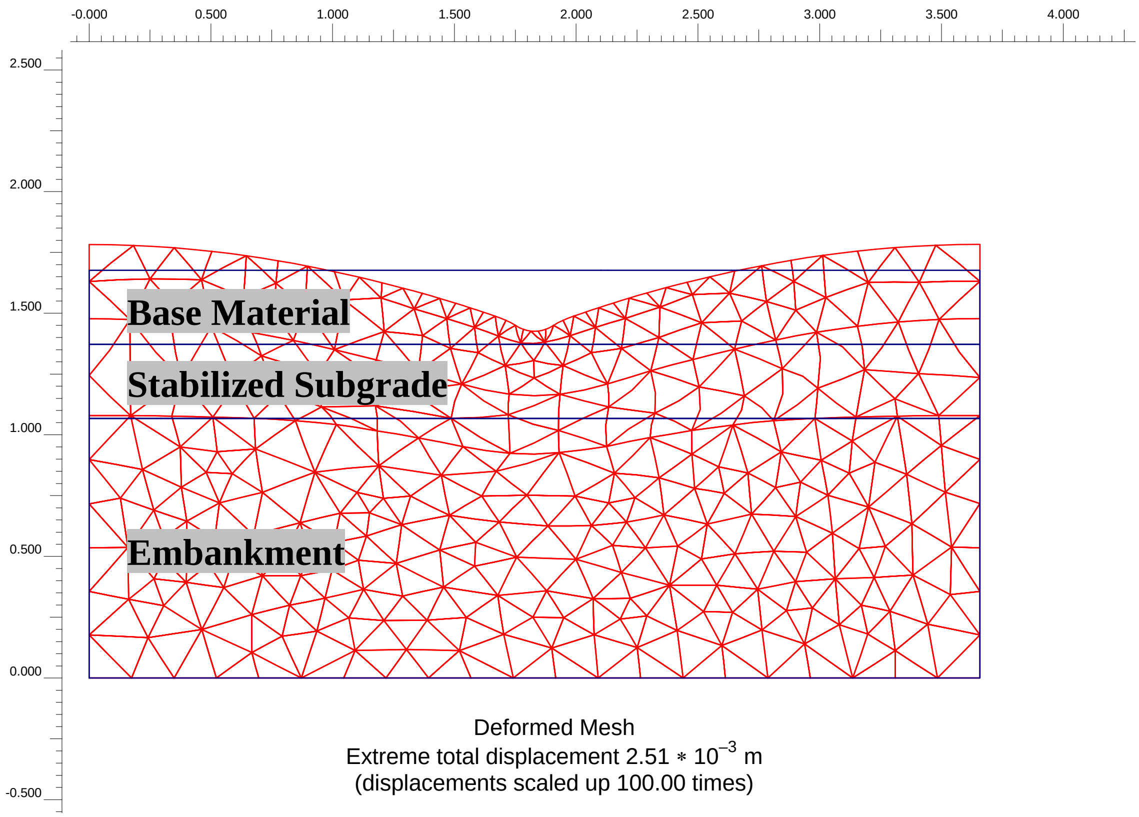


Figure 3-5. Deflection Plot Using 2D Model

From the figure above, it indicates that the three-dimensional model is similar to a plane strain condition. Hence to speed up the computation time, the two-dimensional model was used. The difference between the 3D model and it 2D counterpart is very small. For example the 3D maximum deflection is 2.49 mm, while the 2D model is 2.51 mm.

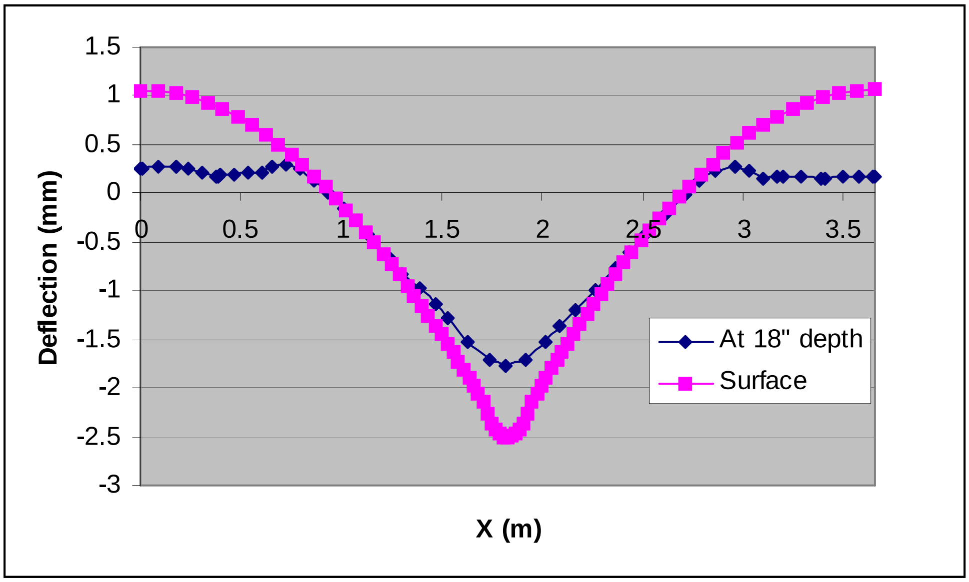


Figure 3-6. Deflection at Two Different Depths Using 2D Model

The above plot shows some interesting results. First, it correctly identifies the soil lifting up above the original datum, i.e., mounding from the rut generation. This is consistent with the field data which showed a 1.05 mm rise (see PPT presentation at the end of this report), whereas this model produced a 1 mm increase. Now it is obvious that one cannot depend on a single test to validate the model, but it is very reassuring that it does provide an excellent starting point from which predictions can be provided.

Another effort was expended looking at the sensitivity of the material properties on rut depth.

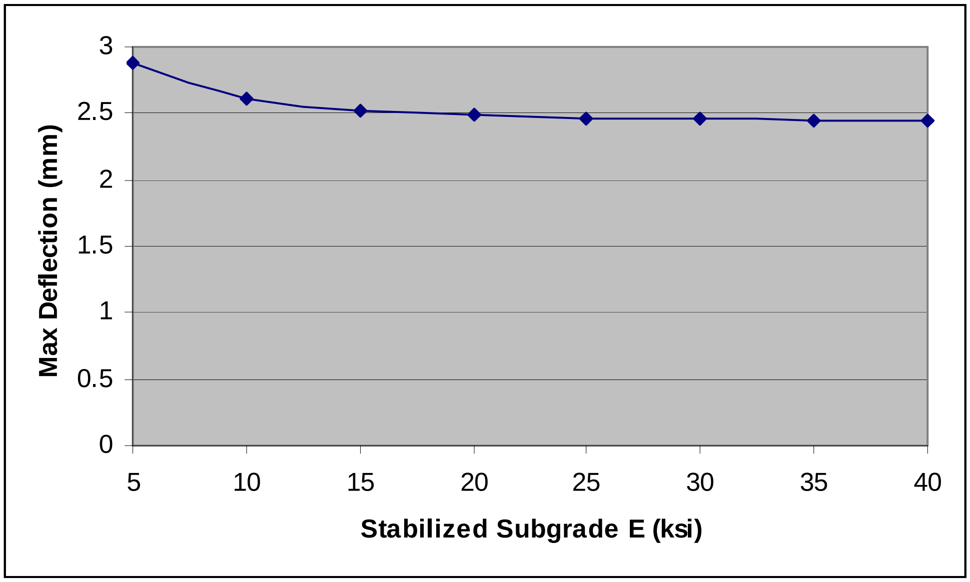


Figure 3-7. Maximum Deflection vs. Subbase's Modulus

The base material properties were maintained and the subbase. As seen above, the subgrade's modulus is relatively insensitive to surface base deflection – once 15 ksi is reached. This is a very interesting finding, suggesting that once this threshold is reached, the potential for excessive deflections are minimized.

However, the base material's properties are extremely sensitive to deflection – as would be expected. This is shown in the plot on the next page.

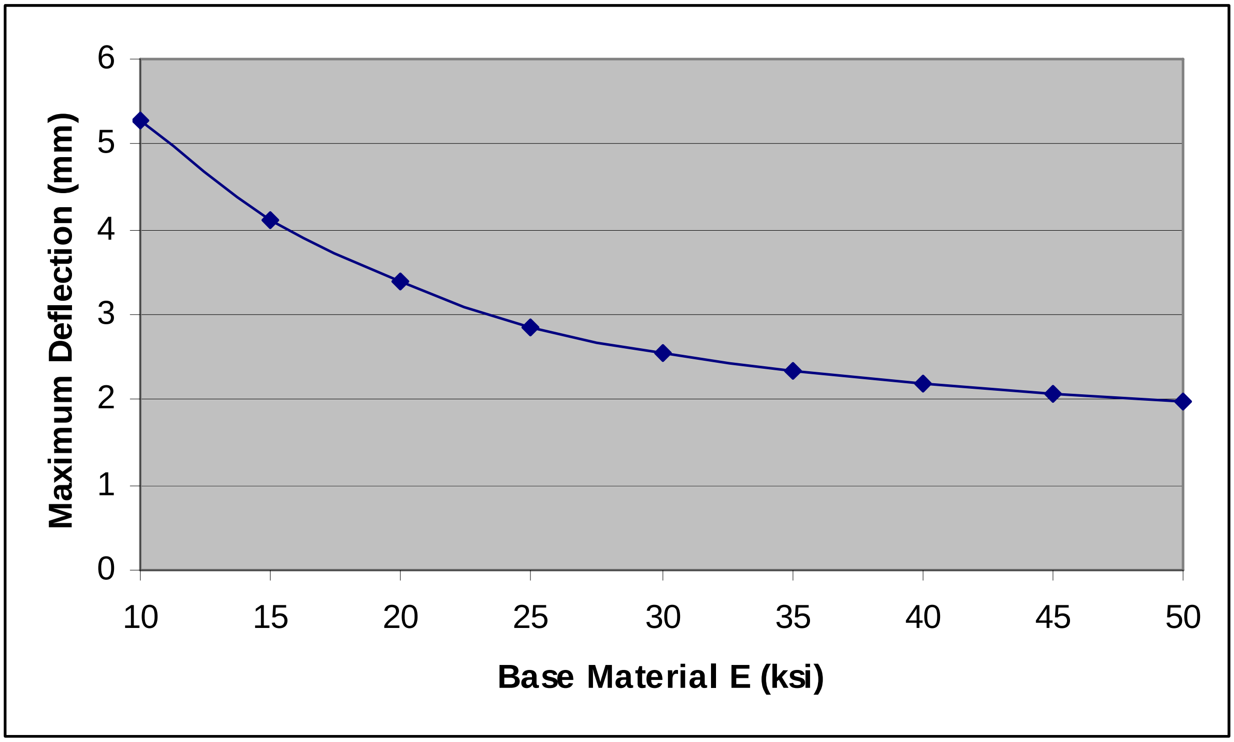


Figure 3-8. Surface Deflection versus Base Properties

Another series of simulations were conducted to further examine these variables as explained below.

General average moduli conditions were set for the subgrade and subbase. The Plaxis input data were adjusted only for the base with a range of moduli from 10 - 60 ksi in increments of 5 ksi.

The output contained six vertical points for vertical displacements as well as effective stresses throughout the base course. The average vertical base displacement and moduli are presented in the graph as well as the average effective stresses versus moduli and are shown on the following page.

A graph of the known data moduli versus the known average displacements was developed from the given data of the east pit. This basic shape was compared to the shape determined from the Plaxis output. The two graphs experience similar curve and shape appeal with Plaxis reporting a displacement of 1 to 5 mm while the pits result in 2 to 7 mm.

Another output of surface stresses within the base was developed in order to see how they vary with moduli. These surface stresses were directly compared to the moduli via the graph shown below. (note: the minus sign in front of the stresses indicates compression)

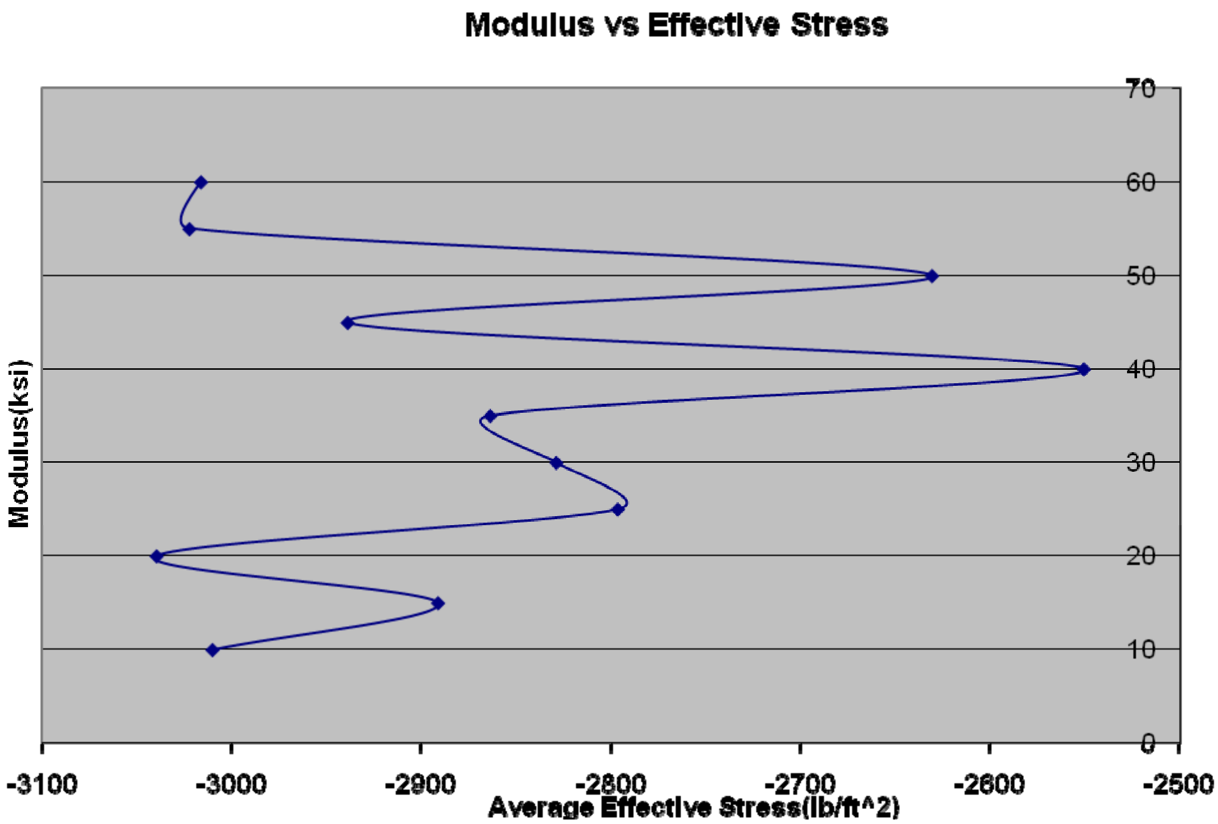


Figure 3-9. Base Modulus versus Effective Stress

This plot shows that as the base moduli reduces, the induced stresses increases correspondily. While more simulations are needed, the preliminary inference is that increasing the modulus of the base course, significantly reduces the induced stresses within the layer. Thus, as was noted previously, stiffness/modulus of the embankment and subgrade does not adversely impact the overall stability of the base course, provided that a minimum acceptable value is obtained. Howeve, it is critical that the base be compacted as well as can be done and that the quality assurance/quality control of this layer monitored consistently.

In addition, the purpose of embankment and subgrade layers are to provide a satisfactory construction platform in order to obtain adequate density/stiffness of the base.

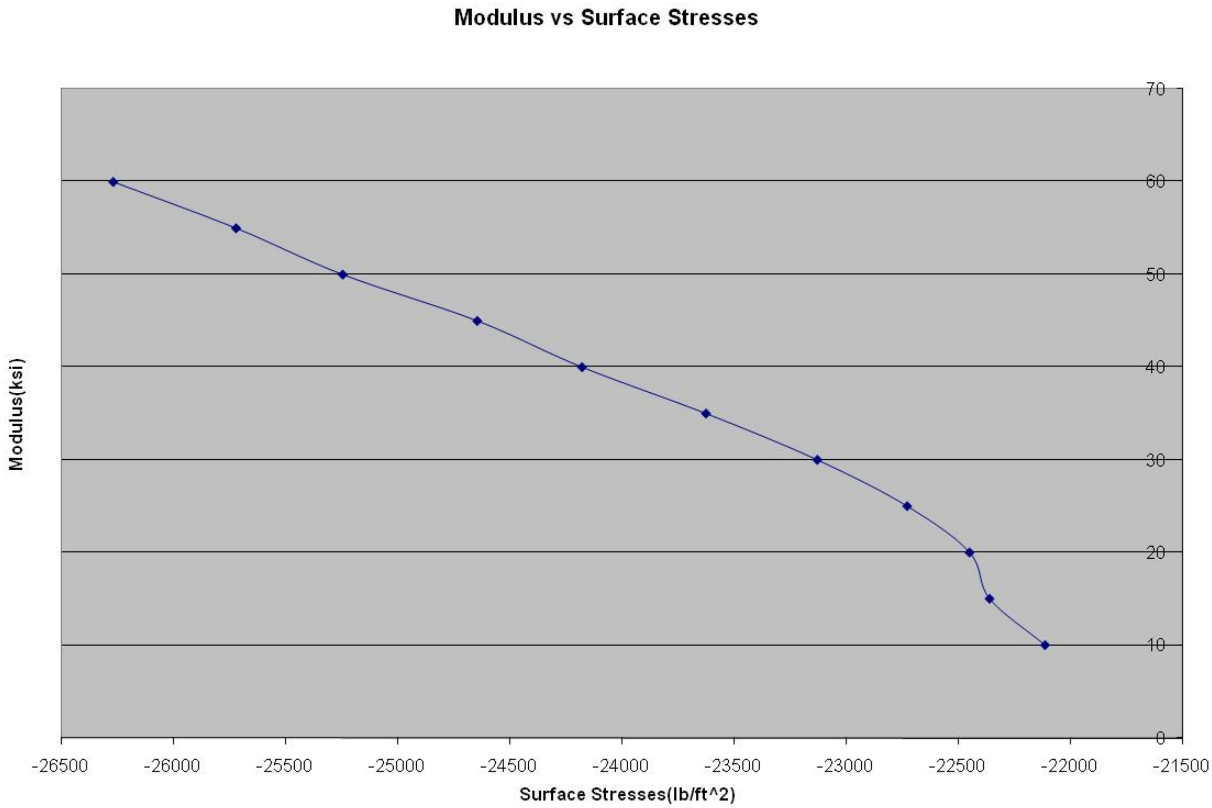


Figure 3-10. Contact Surface Stress versus Base Moduli

The above plot shows the effect of contact surface stresses (wheel load stress) versus base moduli. From this plot it appears that as the modulus decreases, it allows the wheel to settle, thereby increasing the contact area and hence reducing the applied stress to the base course. Again, this is intuitive and corresponds to the previous plot that shows the same trend, i.e., the reduction in surface stresses reduces the internal effective stresses as well. Of course, there is a point in which the rutting becomes problematic - akin to the analysis of a shallow foundation. That is to say, while bearing capacity and settlement are the two criteria for a stable footing, settlement virtually always governs the design. This is because the settlement needed to induce a bearing capacity failure is much greater than the allowable settlement, the simulation shows a similar trend. That is to say, if one reduces the modulus, the stresses (both surface and internal) decrease. However, this comes at the expense of rutting or excessive settlement due to wheel loading.

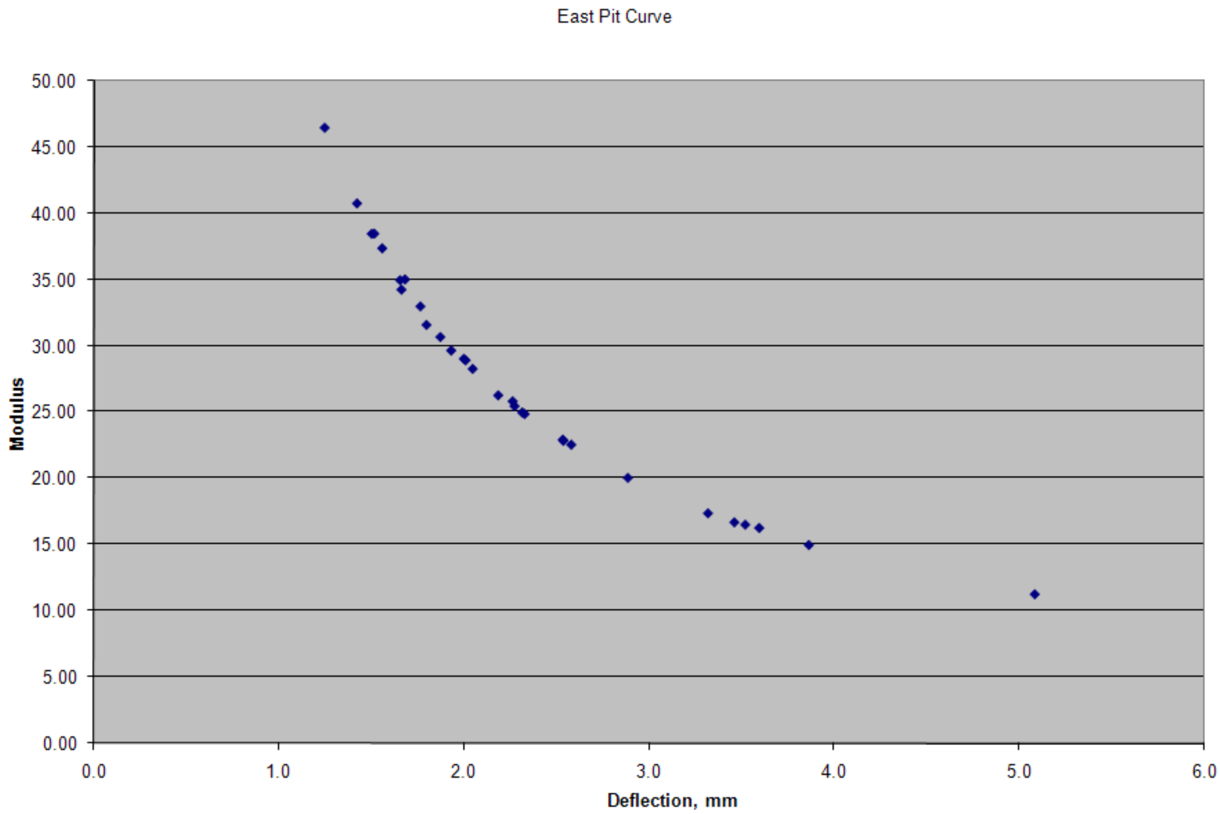


Figure 3-11. Plot of Settlement versus Modulus - Plaxis Simulation

Table 3-3. Surface Rut Generation versus Moduli

Surface Rut Depth (Varying Subbase Moduli)		
Modulus (ksi)	**Surface Rut Depth (in)**	**Surface Rut Depth (mm)**
2	0.285	7.25
10	0.234	5.95
12	0.231	5.87
14	0.229	5.82
16	0.227	5.78
18	0.226	5.75
20	0.225	5.73
22	0.224	5.71
24	0.224	5.69
26	0.223	5.68
50	0.220	5.59

The following plots show examples of the PLAXIS output - noting the versatility of the program in observing stress distribution within a soil mass. Future work will attempt to fine tune the model and ultimately provide an enhancement to the Florida Method Specifications.

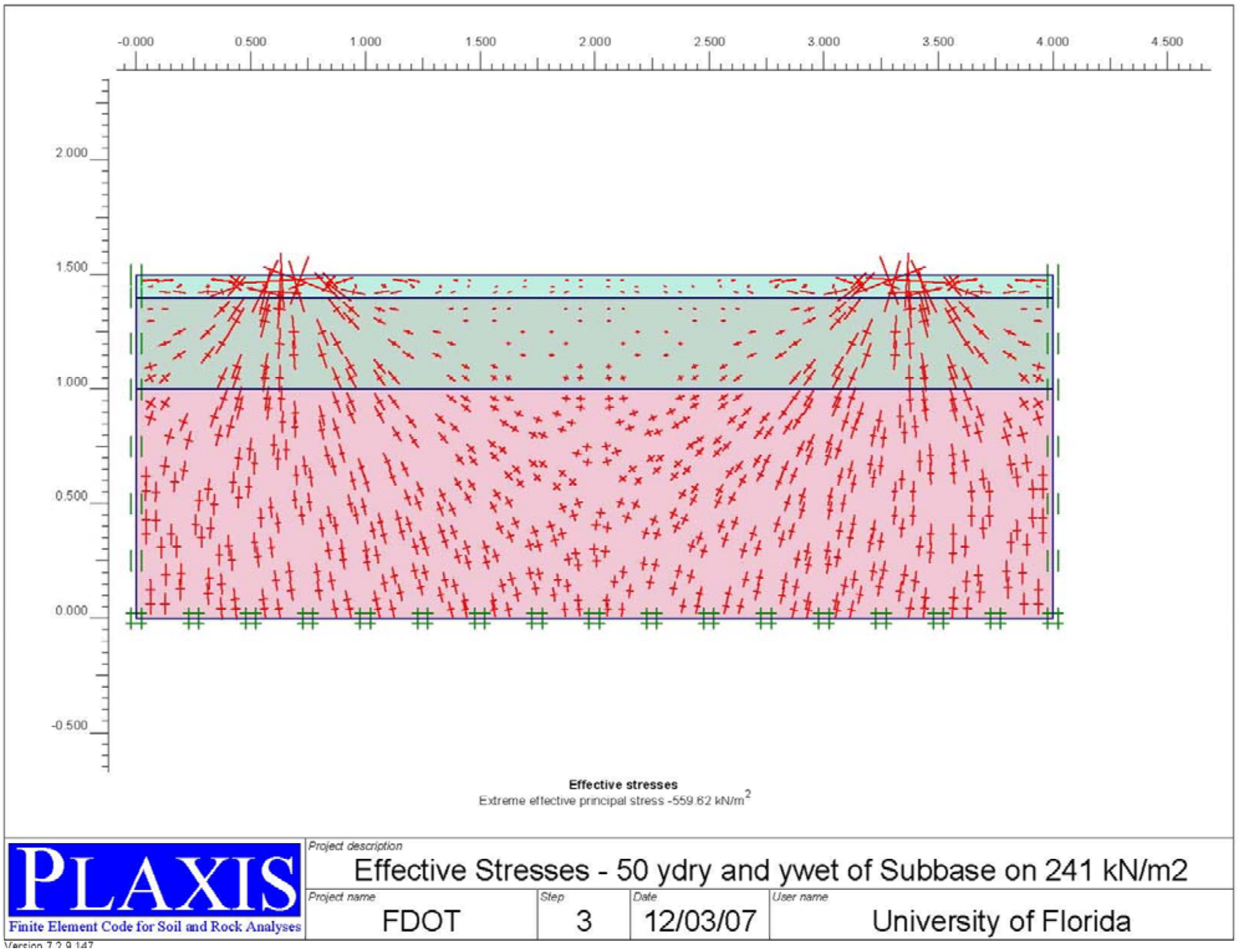


Figure 3-12. Effective Stress Distribution due to Contact Surface Stresses

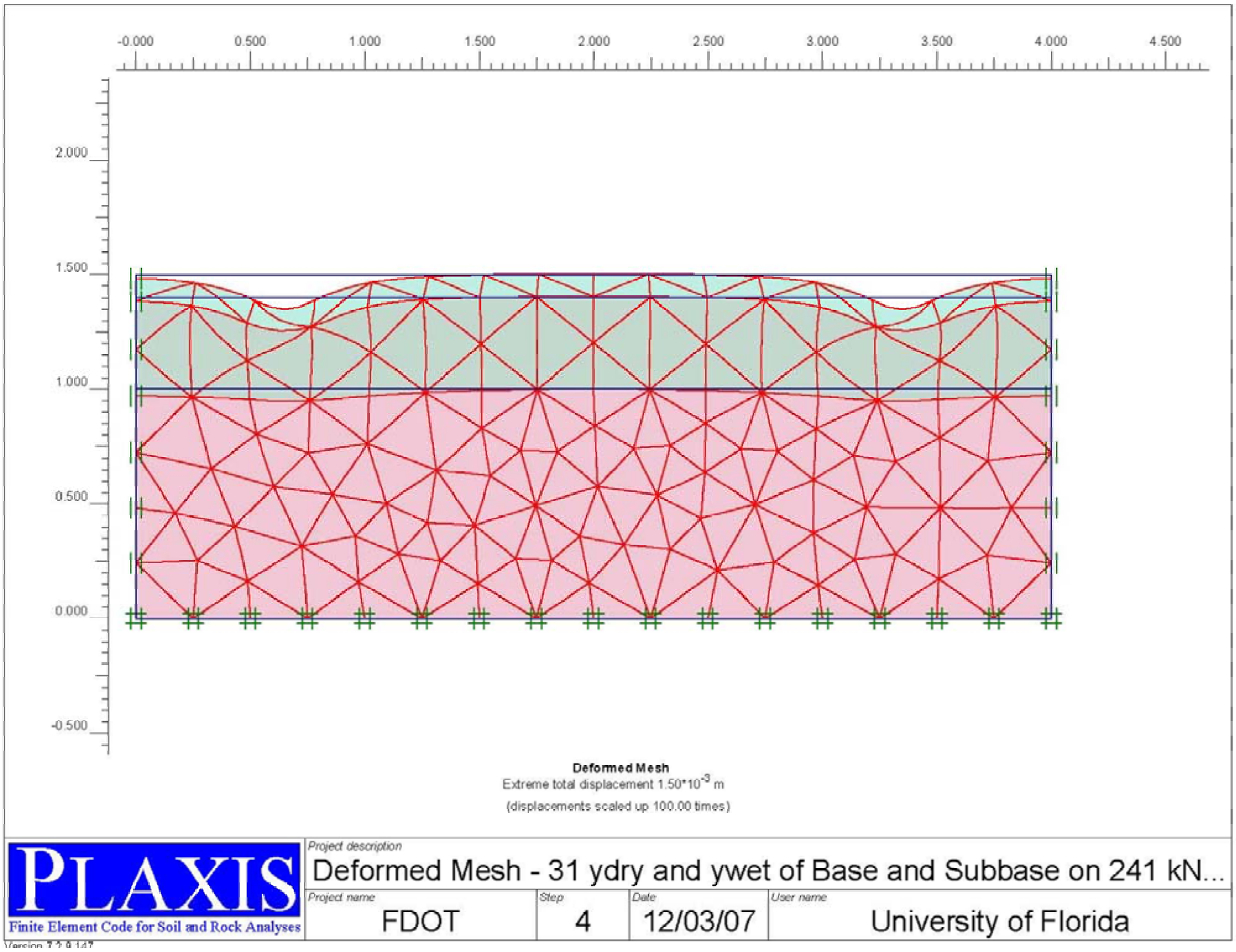


Figure 3-13. Subsurface Deformations due to Surface Loading (31 ksi)

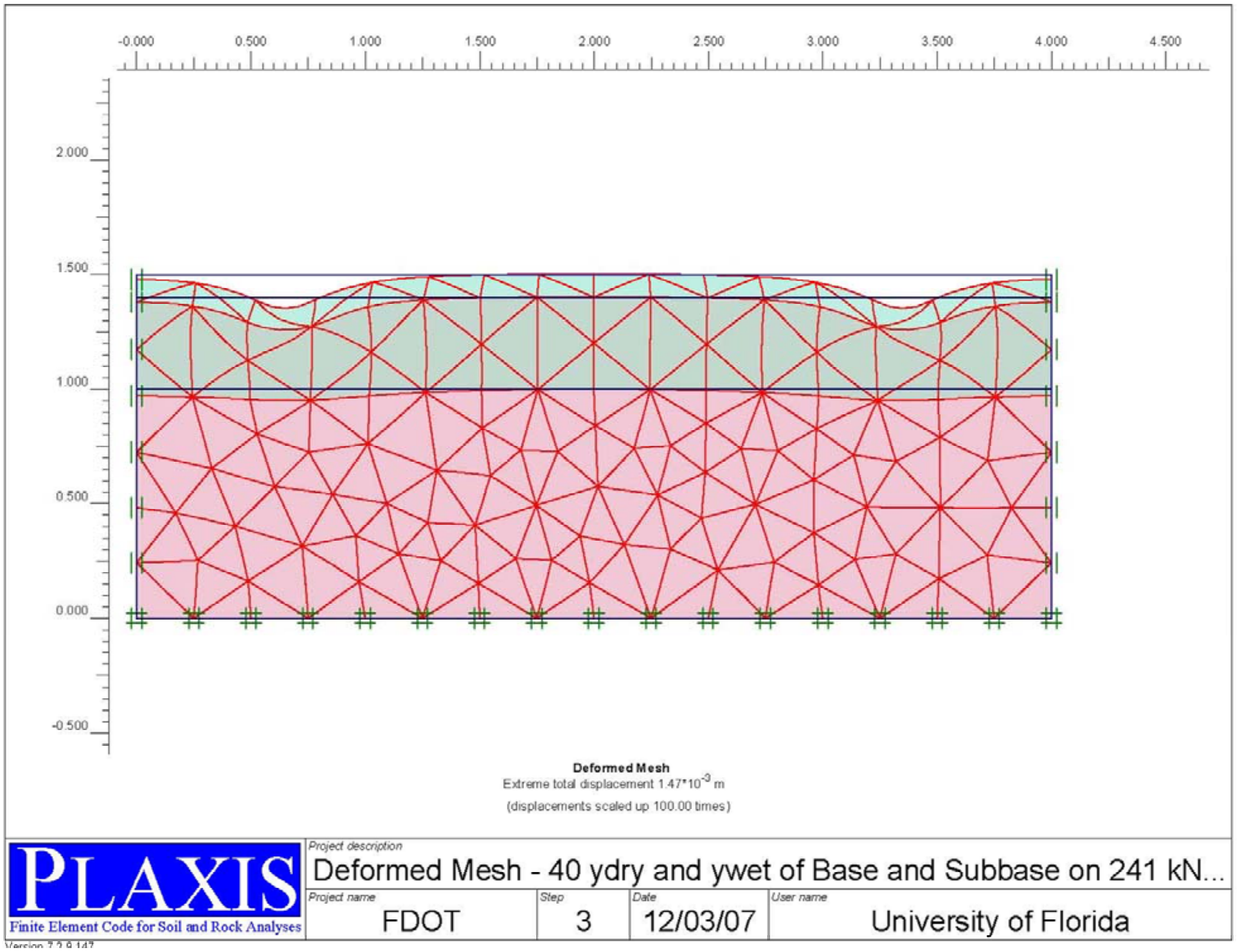


Figure 3-14. Subsurface Deformations due to Surface Loading (40 ksi)

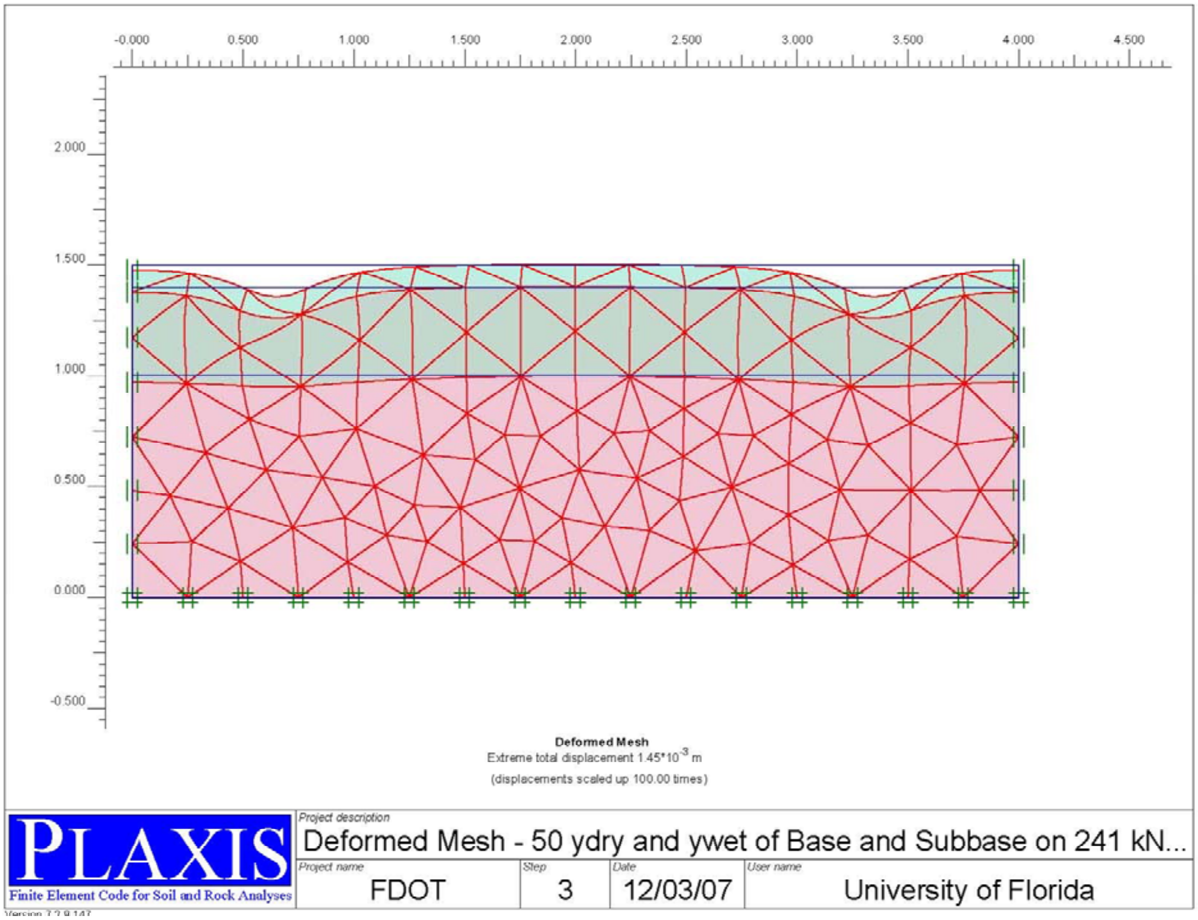


Figure 3-15. Subsurface Deformations due to Surface Loading (50 ksi)

In determining the relation between the base and subbase moduli and rut depths, some interesting trends were observed. The subbase modulus does not seem to have a large effect on the surface rut depths. That is to say, when the base modulus was kept constant and the subbase modulus manipulated, the rut depth only changed in small increments. When the base modulus was manipulated and the subbase modulus kept constant, there was a significant change in the surface rut depths. These relationships could be seen in the graphs comparing the moduli to the rut depths. Thus, in conclusion there would be virtually no effect on the surface rut depth by manipulating the subgrade modulus - within a reasonable range - (keeping the other moduli constant).

Table 3-4. Surface Deflection versus Subbase Modulus

AVERAGE	
Deflection (mm)	Modulus (ksi)
2.0	28.28
3.5	16.70
1.4	40.77
1.8	33.00
1.8	31.60
3.9	14.98
1.5	38.49
1.9	29.66
2.0	28.93
3.6	16.27
3.3	17.38
2.2	26.28
1.6	37.38
1.7	34.26
1.9	30.68
2.3	25.47
2.0	29.04
2.6	22.56
2.3	24.99
1.2	46.48
2.3	25.83
2.9	20.05
2.5	22.85
1.7	35.03
2.5	22.93
2.3	24.87
1.7	34.98
1.5	38.47
3.5	16.52
5.1	11.26

CHAPTER 4.
DISCUSSION OF THE RELEVANCE OF THE TESTING AND SIMULATION RESULTS TO ACCEPTANCE CRITERIA

The Relationship between Base and Subgrade Density and Pavement Structure Performance

Because of problems with the availability of the test pits, we were not able to obtain sufficient data to define this relationship. However, some informed observations are possible. In some engineering materials such as structural concrete, current quality acceptance thinking is that a small percentage below the design target may be accepted because it is likely that an equal amount will be above the design target. With structures this is acceptable because of the existence of load transfer. However, in the case of flexible pavements, it appears likely that any small, localized element of the base/subgrade structure with less than the required stiffness would result in localized pavement failure. This means that for a flexible pavement system, all elements of the base/subgrade system must have a minimum stiffness (density). We cannot accept even a small portion with less than adequate density or better yet, stiffness.

Soil Material Compaction Results are Inherently Variable

Soil materials used in support of pavement structures are a natural product and have a certain degree of variability. Even with reasonable production controls and sampling, they are not likely to reach the uniformity of manufactured products such as HMA or Portland cement concrete. Therefore, acceptance limits (densities/stiffness/moduli) must be set sufficiently high so that there is a very small percentage below the acceptable limit.

The placement of material in the test pits for the initial tests occurred for an optimum situation: relatively small quantity of material (less likely to be variable), placed under closely controlled conditions and constrained by the concrete walls of the pits. The figure below presents the density results from the East Test Pit.

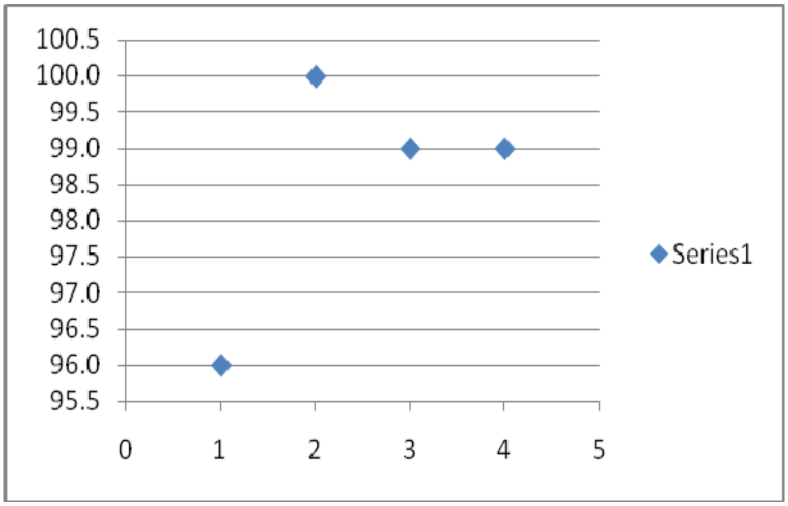


Figure 4-1. Subgrade Density Values - East Pit

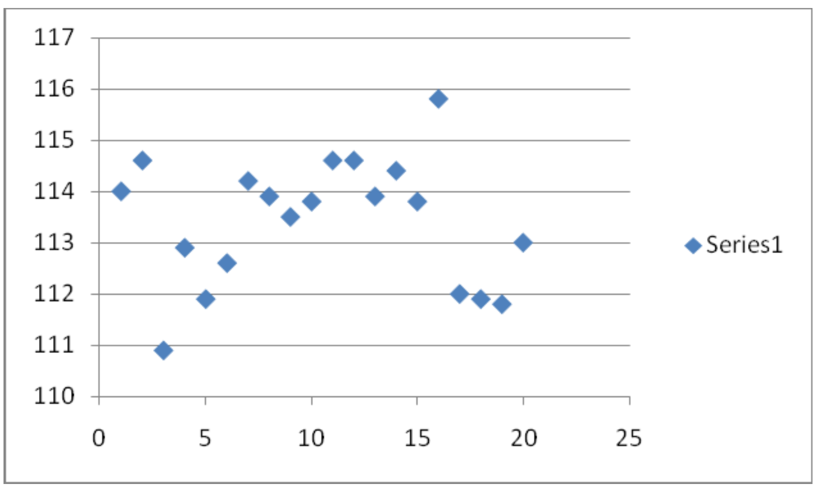


Figure 4-2. Base Density Values - East Pit

These test values were obtained under optimum controlled conditions. The implication is that under normal field conditions we would expect considerable more variability. Under the current acceptance procedure (one passing density per lot), it is likely that a portion of the population is below the specified minimum density. However, pavement failures due to base or subgrade failures appear to be rare. The explanation is that the current density requirement is sufficiently high enough to ensure that the portion below the specified value is still above an unacceptable limit.

Given the single sample acceptance procedure, statistical acceptance criteria is not possible. There are no statistics. However, as stated above, the current acceptance criteria is apparently delivering the desired outcome.

Contribution to Design Strength (Stiffness) Decreases Rapidly with Depth

The results of the simulation analysis clearly indicates the relative contribution of the soil structure layers to pavement stiffness. Intuitively we would expect surface layers to be the most active in resisting loading. This was confirmed by simulation analysis. The figure below presents a graphical representation of the contribution to design strength of the different soil structure layers. The blue series is yet to be identified - it will depend on additional tests.

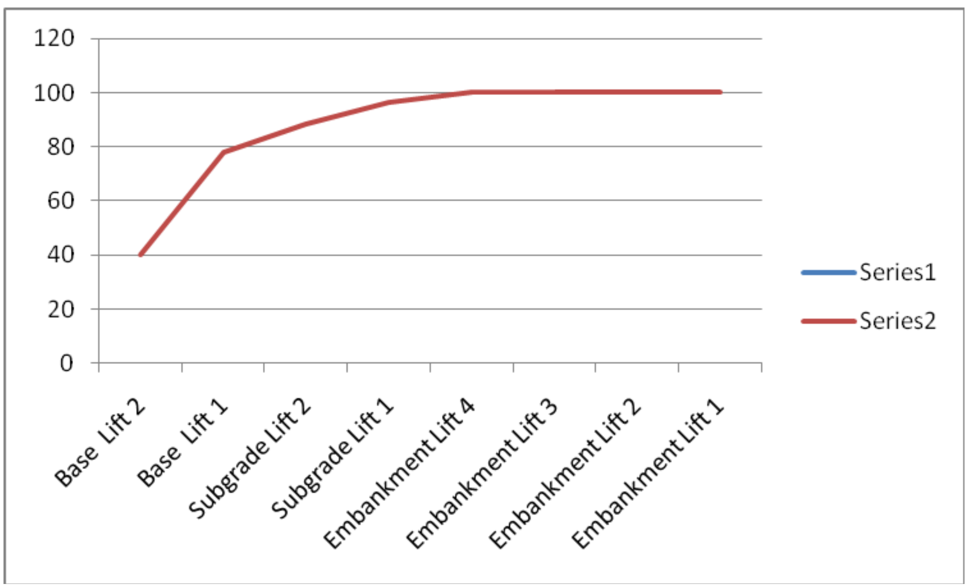


Figure 4-3. Percent Contribution to Design Strength (Resistance to Load)

Resistance to loading is essentially mobilized in the base and subbase layers. Below the top layer of the embankment there is little contribution to resistance to loading.

From an acceptance criteria point of view, this means that the base and subbase structures are critical. Below the top lift of the embankment, densities are less critical. Quality management efforts should be focused on the surface layers. This is a concept that should be understood by field personnel.

CHAPTER 5.
SUGGESTIONS FOR FUTURE STUDIES

Determine from controlled field testing the likely variance in densities achieve under various filed project conditions. This will provide a better understanding of the characteristics of the product that is now being accepted.

Continue a controlled testing program using the test pits with the objective of determining minimum acceptable density/stiffness values for different materials and pavement systems. Additional simulation analysis should also be done concurrently.

These two essential benchmarking studies are a necessary prerequisite to moving forward with base/soil acceptance criteria development.

APPENDIX A.

SUPPLEMENTAL REPORT

A COMPARISON OF STANDARD PROCTOR AND MODIFIED PROCTOR TESTS

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This study was conducted in order to identify a problem with the density relationship between the Standard and Modified Proctor tests. Specifically, that the Modified Proctor density was consistently lower than the Standard Proctor density when performed on certain soils. Once identified, the problem was researched in order to diagnose the soil types most prone to experience this problem. Finally, the issue was explored to discover possible explanations and viable solutions. It was shown that there is a dissipation of energy during compaction by the Modified Proctor method, specifically when using the mechanical machine and when performed on A-3 and A-1-b samples, which have exceptionally low clay percentage. It is proposed that this energy dissipation is due to a lack of cohesion in the soil combined with the sector shaped rammer head impacting with a large amount of energy. Therefore, when performing a 6 inch Modified Proctor density test on these specific soil types, energy dissipation will occur and an unreasonably low density will result.

CHAPTER A-1
INTRODUCTION

The Proctor Density Test is a widely used laboratory density test used to determine the relationship between water content and dry density of soils, as well as to identify the optimum water content of soils (Means & Parcher, 1963). The Proctor Density Test was created by R.R. Proctor of the Bureau of Waterworks and Supply of Los Angeles, California (Means & Parcher, 1963). In 1933, Proctor published a series of articles in the *Engineering News – Record* in which he introduced the theory that density is directly related to water content. Specifically, when the water content of a soil is increased, the moisture lubricates the soil and reduces surface tension allowing the particles to move over each other and compact more efficiently. Eventually, as more water is added, a lubrication limit is reached in which the water acts to separate the soil particles resulting in a decrease in density (*Soils manual*, n.d.). Thus, a maximum density at optimum water content can be determined when the soil is compacted at a constant effort. This source of constant effort is known as the Proctor Density Test and is given the American Association of State Highway and Transportation Officials (AASHTO) designation: T 99 and T 180 (Standard specifications, 2006).

There are two versions of the Proctor Density Test, the Standard (T 99) and the Modified (T 180), which differ in the amount of compaction effort. The Standard Proctor Density Test is performed with a 4 or 6 inch mold using a 5.5 pound metal rammer dropped from a height of 12 inches (Standard specifications, 2006). For the 4 inch mold, the soil is compacted in 3 equal lifts with 25 uniformly distributed blows per lift. For the 6 inch mold, the soil is compacted in 3 equal lifts with 56 uniformly distributed blows per lift. The Modified Proctor Density Test is performed with a 4 or 6 inch mold using a 10 pound metal rammer dropped from a height of 18 inches (Standard specifications, 2006). For the 4 inch mold, the soil is compacted in 5 equal lifts with 25 uniformly distributed blows per lift. For the 6 inch mold, the soil is compacted in 5 equal lifts with 56 uniformly distributed blows per lift (Standard specifications, 2006).

The Modified Proctor test was originally created to mimic the greater compaction effort generated by heavy equiProject Managerent for the construction of large highways and airfields (Means & Parcher, 1963). It is suggested from the AASHTO standards that the greater drop height and increased blow count will result in a large increase in the compaction effort. In the Standard Proctor test the ideal energy of compaction is equal to 12,375 ft-lb/ft^3 and for the Modified Proctor test it is equal to 56,250 ft-lb/ft^3 (Das, 1989). This increase in compaction effort will generally result in an increase in maximum density and a decrease in the optimum moisture point (*Soils Manual*, n.d.).

The rammer for both methods is specified to have a 2 inch diameter circular face, however a sector shaped face may be used given it has the same surface area. Many geotechnical laboratories including the Florida State Materials Research Laboratory use this sector head for the mechanical compaction of the 6 inch mold for both the Standard and Modified methods. The use of the mechanical rammer is accepted in the standards. However, it must be calibrated to give the same moisture-density results as the manually operated rammer (Standard specifications, 2006).

This moisture-density relationship produced by the Standard and Modified Proctor tests is extremely important to determine compaction requirements in the field. Greater density will typically result in greater strength and less compressibility making it vital to pavement design (*Soils manual*, n.d.). Therefore, the engineer will need to know the maximum density as well as the optimum moisture point in order to design the project and monitor field samples (Means &

Parcher, 1963). This reliance on the Proctor Density test for design and field inspection makes receiving accurate and consistent results of the utmost importance.

Purpose

The following are the aims of this study: to prove that for certain soil types there is a serious issue regarding the density relationships of the Standard Proctor and Modified Proctor Tests; to research specifically which soil types display this behavior and which are unaffected; and to research and explain possible causes of these density differences.

CHAPTER A-2
LITERATURE REVIEW

Laboratory Density Tests

It has always been difficult to simulate the field state of a soil in a laboratory setting. Several laboratory compaction methods are currently used to recreate the field compaction states and to determine the maximum densities of soils.

Static Compaction utilizes a slowly increasing load applied to a portion or the total cross sectional area of the soil sample for a given amount of time. This method of compaction is used in the Florida State Materials Laboratory for the sample preparation of Resilient Modulus (AASHTO designation T 307), Direct Shear (American Society for Testing and Materials (ASTM) designation D 3080), and Triaxial Compression Test (AASHTO designation T-297). Some laboratories use this method to determine maximum unit weight and optimum water content (Wahls, Fisher, & Langfelder, 1966).

Kneading Type Compaction is meant to more closely simulate field compaction methods because sheepsfoot rollers and rubber tired rollers are not static or impact forms of compaction. Kneading Type Compaction is performed by gradually building up the pressure on the sample, applying the pressure for a given period of time, and then gradually releasing the pressure (Wahls et al., 1966).

Vibratory Type Compaction is performed on predominantly cohesionless soils due to their physical properties. These soils will deform very little under a heavy static load. However, when vibrated it allows the particles to fit together and compact with ease (Means & Parcher, 1963). The ASTM test for determining relative density of sand (ASTM designation D 4253) is a vibratory type compaction method. It is performed by exposing the sample to a specific frequency vibration for a given amount of time and applying a given amount of surcharge (Standard specifications, 2006).

Gyratory Compaction is another method used to simulate field conditions. Its principles are based on the kneading type compaction and was originally devised by the Texas Highway department and developed by the Army Corps of Engineers (Wahls et al., 1966). It is performed by exposing samples to a given gyratory angle, vertical pressure, and number of cycles (Hoff, Baklokk, & Aurstad, 2005).

Impact Type Compaction utilizes the force of a hammer of specific weight and drop height to rapidly strike the sample (Standard specifications, 2006). Several methods of impact compaction are used, and they vary depending on hammer weight, drop height, mold dimensions, number of soil layers, number of blows, and maximum material size. The many different impact compaction tests are all modifications of Proctor's original method (Wahls et al., 1966). Impact compaction tests can be performed manually or by mechanical means. However, the mechanical method must be designed to replicate the manual method (Standard specifications, 2006). Impact compaction is the most widely used compaction method for determining the maximum density and optimum water content in design today (*Soils manual*, n.d.).

Research Questions

When performing the Standard Proctor Density Test and the Modified Proctor Density Test on certain sandy soils, counterintuitive results have been noticed. Standard Proctor density results should be approximately 90-92% lower than the Modified Proctor results due to the lower compaction effort ("Soil density," 2003). The Modified Proctor test applies approximately 4.5 times the amount of compaction energy of standard Proctor test, thus giving the Modified Proctor sample a greater maximum density (Das, 1989). At the Florida Department of

Transportation State Materials Research Park, many soil samples which are tested by both these methods yield a Modified Proctor density that is actually lower than the Standard Proctor density. These results were initially overlooked as operator error, but have since spurred the need for further research.

- RQ1: Is there a reoccurring issue in which the Modified Proctor Density is not sufficiently greater than the Standard Proctor Density?
- RQ2: If there is a reoccurring issue, than why is compaction energy being dissipated in the Modified Proctor Density test?
- RQ3: How is the connection between Standard and Modified Proctor Densities related to or dependant on the type of soil?

CHAPTER A-3
MATERIALS AND METHODS

Sample

A total of 40 soil samples were chosen for this study. All samples were classified according to the AASHTO Classification of Highway Subgrade Material (Standard specifications, 2006). Twenty samples were classified as A-3 fine sand and 2 samples as A-1-b sand. These samples are characterized as having less than or equal to 10% passing the No. 200 sieve and were all non-plastic (Standard specifications, 2006). The final 18 samples were classified as A-2-4, silty or clayey sand. These samples are characterized by having greater than 10% passing the No. 200 sieve and were all non-plastic. The selection of these materials was based on the observation that sandy soils were those most often experiencing the problem at hand. However, the influence of fine particle percentage was unknown. Also, these soils are typical embankment samples with which the Proctor test would normally be utilized. The inclusion of A-1-b, A-3, and A-2-4 will create fine particle percentage as the main variable between these relatively similar materials.

Design

All tests were performed at the State of Florida Materials Research Park. All Standard Proctor tests were performed according to AASHTO designation: T 99 Method C and D. All Modified Proctor tests were performed according to AASHTO designation T-180 Method C and D. All Hydrometer tests were performed by AASHTO designation T-227. All Particle Size Analysis tests were performed by AASHTO designation T 88. All Limerock Bearing Ratio (LBR) tests were performed by the AASHTO designation FM 5-515 (Standard specifications, 2006).

The general procedure was to initially run the 6 inch Standard and Modified Proctor Tests on the samples. These tests were performed using the mechanically operated compaction machines which utilize the sector shaped rammer head. The 6 inch samples were then tested for the Limerock Bearing Ratio. The 6 inch Modified Proctor test was rerun on 23 of the samples using the mechanically operated rammer for the first 3 lifts and the manually operated rammer for the final two lifts. The manual rammer was only used on the final two lifts, and in order to facilitate the research. The Limerock Bearing Ratio was then determined for these samples.

The resulting data was organized into tabular format along with the results of the soil classification and particle size analysis. This became the base model for data analysis and manipulation.

CHAPTER A-4
RESULTS

General Results

Tables 4-1 and 4-2 display all of the data compiled on the 40 samples. Included are the mechanically operated Standard and Modified Proctor results, Modified Proctor with the 2 manual lift results, LBR results, soil classification, and particle analysis. The Modified Proctor with 2 manual lifts was not performed on all the samples due to the time constraints of compiling such data. However, enough tests were performed to make viable conclusions. All figures are based on the results displayed in Tables 4-1 and 4-2.

Reoccurring Issue

Table 4-3, column 1 displays the additional density (pcf) achieved when mechanically compacted by the Modified Proctor method for each of the 40 samples. This is the maximum Modified Proctor density of the sample, minus the maximum Standard Proctor density. Values in the negative range signify a decrease in the maximum density when compacted by the Modified Proctor method. Table 4-3, column 1 shows that 16 of the samples yielded a lower maximum density when compacted by the Modified Proctor method. The average density increase from the Standard Proctor to the Modified Proctor was 1.12 pcf.

Relation to Soil Type

Tables 4-3, column 1 shows the average density increase for the A-3/A-1-b and the A-2-4 samples when mechanically compacted by the Modified Proctor method. The A-3 and A-1-b samples experienced an average of a 0.16 pcf density increase while A-2-4 samples experienced an average of a 2.28 pcf density increase.

Figure 4-1 relates the Modified Proctor density increase to the percentage of particles passing the No. 200 sieve. This figure shows a trend that as the percentage of particles passing the No. 200 sieve increases, the density difference from Standard to Modified Proctor also increases. In other words, the samples which contained a greater amount of fine particles, those passing the No. 200 sieve, showed an increase in maximum density when compacted by the Modified Proctor method. However, the samples which contained less amounts of fine particles showed much less of an increase in density when compacted by the Modified Proctor method and in some cases an actual decrease in density.

Figure 4-2 relates the Modified Proctor density increase to the percentage of sand in each sample. This figure shows that as the percentage of sand particles was increased, the density difference between the Standard and Modified Proctor tests decreased.

Figure 4-3 relates the Modified Proctor density increase to the percentage of clay in each sample. This figure shows that as the percentage of clay particles was increased, the density difference between the Standard and Modified Proctor tests increased.

Figure 4-4 relates the Modified Proctor density increase to the percentage of silt in each sample. This figure shows a weaker trend that when the percentage of silt particles was increased, the density difference between the Standard and Modified Proctor tests increased.

Effect of Manual Compaction

Columns 2 and 3 of Table 4-3 show the effect of manual compaction of the final two lifts during the Modified Proctor test. Column 2 displays the density difference between the manual compaction for two lifts of the Modified Proctor and the mechanically compacted Standard Proctor. The average density increase from the Standard Proctor density for the A-3/A-1-b material was 3.55 pcf and for the A-2-4 material was 3.48 pcf. The average density increase overall was equal to 3.52 pcf. Column 3 displays the density difference between the maximum Modified Proctor density performed mechanically and the maximum Modified Proctor density with the final two lifts performed manually. This column shows the increase in density when samples were compacted with equivalent energy but different testing methods. The average density increase when the final two lifts of the Modified Proctor test were performed manually is 3.08 pcf for the A-3/A-1-b material and is 1.02 pcf for the A-2-4 material. The average density increase overall was equal to 2.18 pcf.

Figure 4-5 shows the relationship between the values in column 3 of Table 4-2 and percentage of particles passing the No. 200 sieve. This figure shows that as the percentage of fine material increases, the difference in Modified Proctor performed manually for two lifts decreases.

Figure 4-6 shows the relationship between the values in column 3 of Table 4-2 and the percentage of sand particles. This figure shows that as the percentage of sand increases, the difference in Modified Proctor performed manually for two lifts increases.

Figure 4-7 shows the relationship between the values in column 3 of Table 4-2 and the percentage of clay particles. This figure shows that as the percentage of clay increases, the difference in Modified Proctor performed manually for two lifts decreases.

Figure 4-8 shows the relationship between the values in column 3 of Table 4-2 and the percentage of silt particles. This figure shows that as the percentage of silt particles increases the difference in Modified Proctor remains relatively unchanged.

Figure 4-9 shows the relationship between the values in column 2 of Table 4-2 and the percentage of particles passing the No. 200 sieve. This figure shows that when the manual Modified Proctor was performed on two layers, the densities were all but one brought up into the positive level. It also shows that there is no longer a correlation between the density increase and the fine particle percentage.

Table 4-1. General results for 6" Standard and Modified Proctor tests

	Standard & Modified Proctor						LBR Values		
Soil #	6" Den. (pcf)	T-99* Moist. (%)	6" Den. (pcf)	T-180* Moist. (%)	6" T-180 Den. (pcf)	Man.** Moist. (%)	6" T-99*	6" T-180*	6" T-180 Man.**
1	111.5	13.0	109.9	11.2	113.3	11.1	56.0	55.0	55.0
2	109.2	12.2	107.8	11.8	112.5	11.1	43.0	46.0	55.0
3	115.8	10.4	118.2	7.8	118.5	8.4	43.0	40.0	50.0
4	111.0	11.5	116.0	10.4	116.9	11.1	20.0	32.0	31.0
5	116.5	10.3	116.3	8.3			43.0	40.0	
6	112.7	11.3	112.1	7.5			32.0	28.0	
7	114.1	9.3	113.7	11.1			31.0	29.0	
8	117.7	9.3	118.8	7.5			50.0	48.0	
9	114.4	10.2	111.9	12.3			42.0	36.0	
10	111.8	10.2	110.2	8.3			35.0	23.0	
11	112.0	10.2	112.6	10.4			37.0	61.0	
12	112.7	10.8	113.4	11.1			52.0	61.0	
13	112.0	10.2	112.4	11.3			45.0	57.0	
14	104.5	14.2	103.6	13.6	107.3	12.3	43.0	37.0	45.0
15	104.5	14.5	106.1	13.9	109.5	12.8	31.0	32.0	28.0
16	106.2	13.3	105.6	11.3	110.8	10.9	44.0	32.0	34.0
17	103.7	15.2	105.8	13.6	107.8	12.6	34.0	43.0	45.0
18	104.6	14.1	105.3	12.6	107.4	12.6	59.0	61.0	56.0
19	104.7	14.2	104.2	13.5	106.6	13.1	37.0	52.0	43.0
20	106.9	13.7	109.3	13.4	112.6	12.6	24.0	29.0	40.0
21	113.0	10.4	112.1	8.8	115.8	9.1	36.0	25.0	41.0
22	112.9	10.2	110.7	8.1	115.6	9.3	40.0	31.0	41.0
23	114.7	10.9	115.6	7.0	118.6	9.3	43.0	43.0	56.0
24	113.6	10.6	115.1	9.2	117.3	9.2	49.0	48.0	71.0
25	113.0	11.2	111.9	10.1	114.6	9.3	52.0	43.0	65.0
26	119.9	9.4	125.7	8.0	125.5	8.8	36.0	79.0	83.0
27	118.4	9.5	124.0	7.6	124.5	9.0	39.0	85.0	88.0
28	115.6	11.2	113.5	10.3	115.4	10.4	76.0	64.0	66.0
29	113.0	10.4	117.5	9.5			36.0	48.0	
30	117.2	10.2	115.4	9.1			56.0	43.0	
31	115.0	10.5	117.0	9.6			49.0	51.0	
32	115.9	10.7	118.9	9.7			42.0	61.0	
33	115.5	10.5	117.5	9.5			46.0	57.0	
34	115.1	10.6	118.3	9.7			40.0	57.0	
35	116.1	10.8	119.0	9.9			46.0	58.0	
36	116.0	10.3	116.6	9.3			61.0	67.0	
37	115.5	11.5	121.1	8.7	118.5	11.1	35.0	125.0	42.0
38	118.9	10.1	125.4	8.9	121.7	10.6	37.0	102.0	32.0
39	111.4	11.2	111.0	10.4	113.9	10.9	20.0	30.0	33.0
40	108.6	11.5	110.9	11.6	114.4	10.8	37.0	46.0	63.0

* Performed with mechanical compaction machine; ** Performed last two lifts with manual rammer

Table 4-2. Soil class and particle analysis

General Class				Particle Analysis									
Soil #	Liquid limit	Plastic index	Soil class	Pass 3/4 sieve	Pass 4 sieve	Pass 10 sieve	Pass 40 sieve	Pass 60 sieve	Pass 100 sieve	Pass 200 sieve	% silt	% clay	% sand
1	NP	NP	A-3	100	100	100	77.5	46.6	22.6	8.4	6	2	92
2	NP	NP	A-3	100	100	100	75.4	42.2	18.1	6.2	4	2	94
3	NP	NP	A-3	100	86.9	80.4	57.3	35.9	18.4	7.1			
4	NP	NP	A-3	100	100	96	57.8	31.8	17.8	9.2			
5	NP	NP	A-3	100	100	99	55.8	25	14.4	9.4			
6	NP	NP	A-3	100	100	99.2	61.8	27.2	13.8	8.5			
7	NP	NP	A-3	100	100	98.7	55.3	23.9	12	7			
8	NP	NP	A-3	100	94.2	88.6	61.8	35.4	17.3	6.3			
9	NP	NP	A-3	100	100	99.7	66.1	32	16.9	7.4			
10	NP	NP	A-3	100	100	99.5	67.7	33.5	14.2	7			
11	NP	NP	A-3	100	100	99.9	82	59	29.9	6.3	4	2	94
12	NP	NP	A-3	100	100	99.8	77.8	54.7	27.2	7	4	3	93
13	NP	NP	A-3	100	100	99.6	84.7	62.8	31.3	7.1	4	3	93
14	NP	NP	A-3	100	100	100	98.6	90.9	63.4	9.9	8	2	90
15	NP	NP	A-3	100	100	99	98	90	61	9	8	1	91
16	NP	NP	A-3	100	100	100	99	91	60	9	8	1	91
17	NP	NP	A-3	100	100	100	97	85.7	34.5	7.1	3	4	93
18	NP	NP	A-3	100	100	100	98.3	87.4	25.8	6.9	5	2	93
19	NP	NP	A-3	100	100	100	98.7	86.3	19.3	5.2	4	1	95
20	NP	NP	A-3	100	100	99.5	96.7	84.4	29.6	10.4	6	4	90
21	NP	NP	A-1-b	100	100	98	49.4	21.6	11.1	6.9			
22	NP	NP	A-1-b	100	100	98.3	49.2	20.4	11.2	7.1			
23	NP	NP	A-2-4	100	100	98.7	75.3	52.4	28.8	12.8	6	7	87
24	NP	NP	A-2-4	100	100	97.9	77.8	52.3	28.9	12	7	5	88
25	NP	NP	A-2-4	100	100	100	87.6	66.8	38.9	14.1	13	1	86
26	NP	NP	A-2-4	100	100	97.1	81	62.5	44.1	22.1	11	11	78
27	NP	NP	A-2-4	100	100	96.1	83	65.7	48.2	22.8	18	5	77
28	NP	NP	A-2-4	100	100	100	79.2	52.4	30.1	12.8	8	5	87
29	NP	NP	A-2-4	98.5	93.3	88.2	63.9	39.9	24.6	11.2			
30	NP	NP	A-2-4	100	100	99.6	72	39.4	20.5	10.6			
31	NP	NP	A-2-4	100	100	98.7	84.1	65	43.4	16.1	16	0	84
32	NP	NP	A-2-4	100	100	98.5	80.6	60.1	38	14.1	13	1	86
33	NP	NP	A-2-4	100	100	98.1	81.7	61	38.2	12.7	12	1	87
34	NP	NP	A-2-4	100	100	96.7	79	58.5	38.5	12.4	11	1	88
35	NP	NP	A-2-4	100	100	98.5	78.7	55.9	35.1	13.6	11	3	86
36	NP	NP	A-2-4	100	100	99.8	79.5	56.4	38.8	12.8	12	1	87
37	NP	NP	A-2-4	100	100	99	97	91	72	18	11	7	82
38	NP	NP	A-2-4	100	100	99	80	56	35	14	4	10	86
39	NP	NP	A-2-4	100	100	100	99	91	67	11	8	3	89
40	NP	NP	A-2-4	100	100	96.7	92.4	82.5	35.1	13.6	10	4	86

Table 4-3. Density increases

		1	2	3
		Mechanical	T180 Manual two lifts	Manual - Mechanical
Soil #	Soil Class	T-180* Max - T-99* Max Density Increase	T-180 Max** - T-99 Max* Density Increase	T-180 Max** - T-180 Max* Density Increase
1	A-3	-1.6	1.8	3.4
2	A-3	-1.4	3.3	4.7
3	A-3	2.4	2.7	0.3
4	A-3	5	5.9	0.9
5	A-3	-0.2		
6	A-3	-0.6		
7	A-3	-0.4		
8	A-3	1.1		
9	A-3	-2.5		
10	A-3	-1.6		
11	A-3	0.6		
12	A-3	0.7		
13	A-3	0.4		
14	A-3	-0.9	2.8	3.7
15	A-3	1.6	5	3.4
16	A-3	-0.6	4.6	5.2
17	A-3	2.1	4.1	2
18	A-3	0.7	2.8	2.1
19	A-3	-0.5	1.9	2.4
20	A-3	2.4	5.7	3.3
21	A-1-b	-0.9	2.8	3.7
22	A-1-b	-2.2	2.7	4.9
A-3/A-1-b Average:		0.16	3.55	3.08
23	A-2-4	0.9	3.9	3
24	A-2-4	1.5	3.7	2.2
25	A-2-4	-1.1	1.6	2.7
26	A-2-4	5.8	5.6	-0.2
27	A-2-4	5.6	6.1	0.5
28	A-2-4	-2.1	-0.2	1.9
29	A-2-4	4.5		
30	A-2-4	-1.8		
31	A-2-4	2		
32	A-2-4	3		
33	A-2-4	2		
34	A-2-4	3.2		
35	A-2-4	2.9		
36	A-2-4	0.6		
37	A-2-4	5.6	3	-2.6
38	A-2-4	6.5	2.8	-3.7
39	A-2-4	-0.4	2.5	2.9
40	A-2-4	2.3	5.8	3.5
A-2-4 Average:		2.28	3.48	1.02
TOTAL AVERAGE:		1.12	3.52	2.18

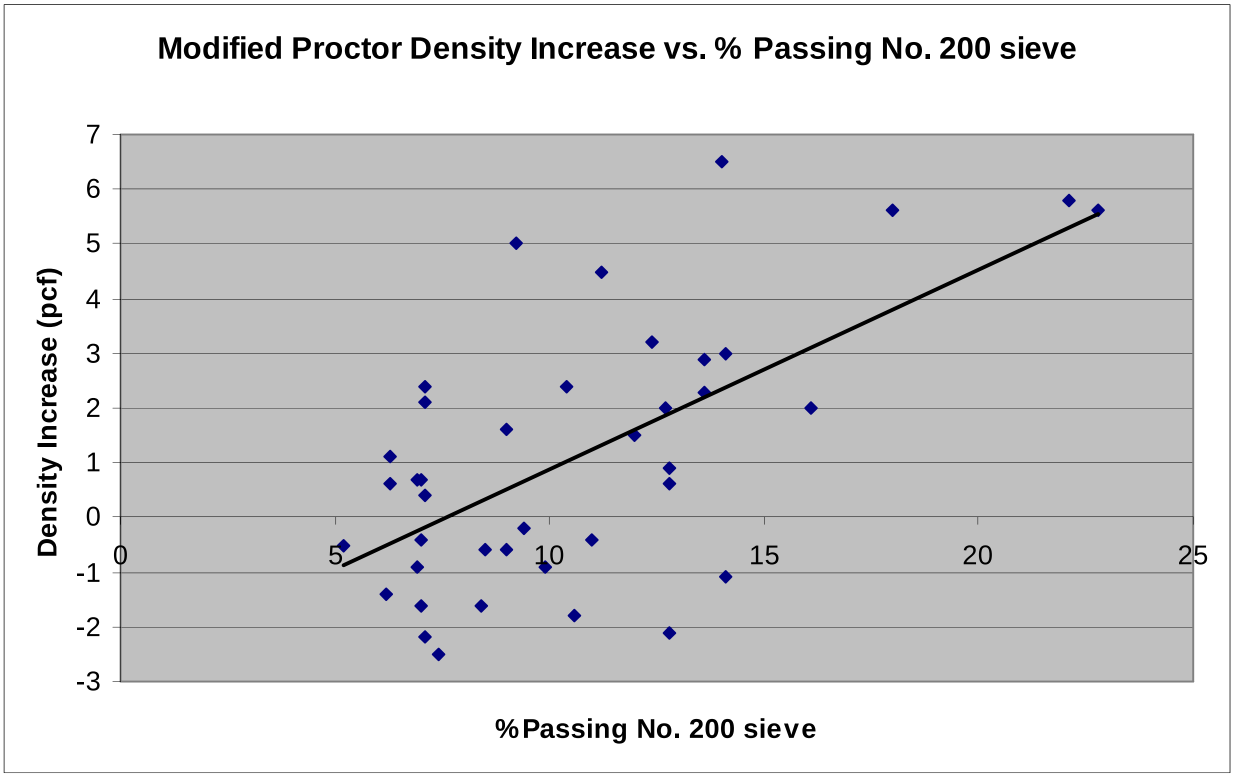

Figure 4-1. Density increase vs. % passing No. 200 sieve (mechanical T180 – mechanical T99)

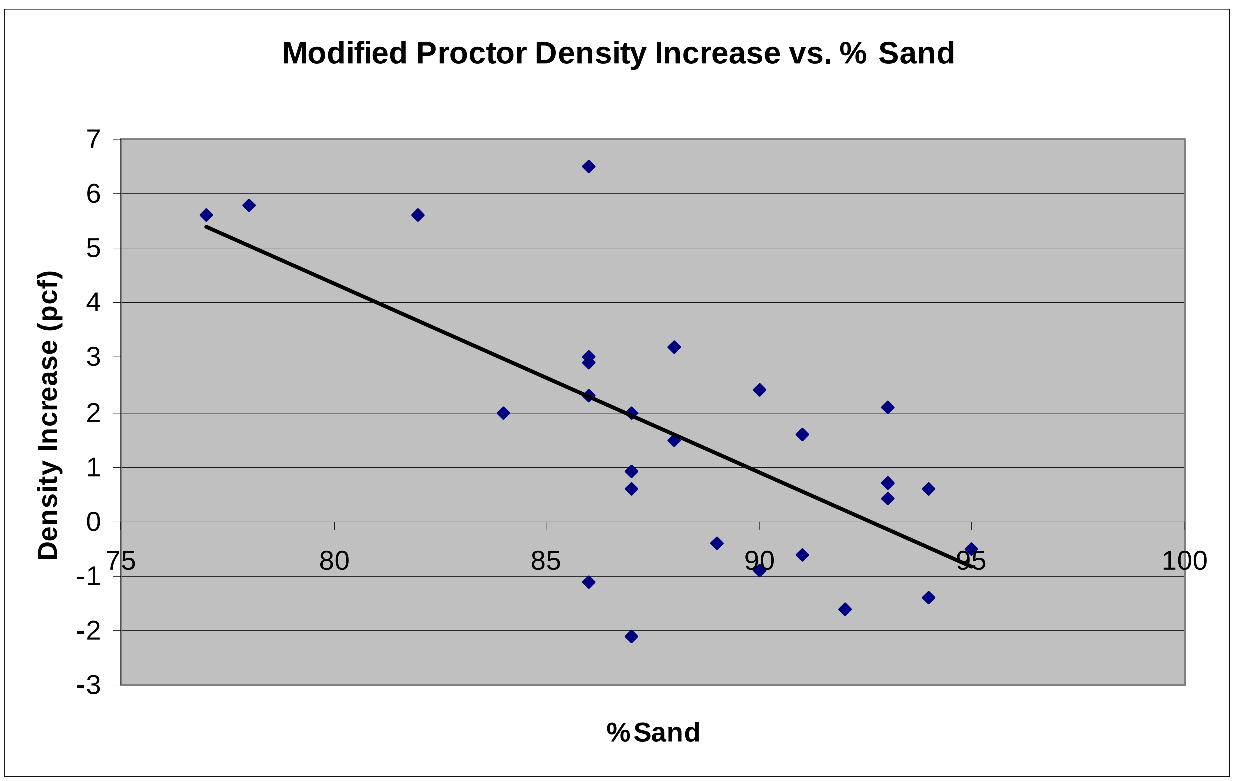

Figure 4-2. Density increase versus % sand (mechanical T180 – mechanical T99)

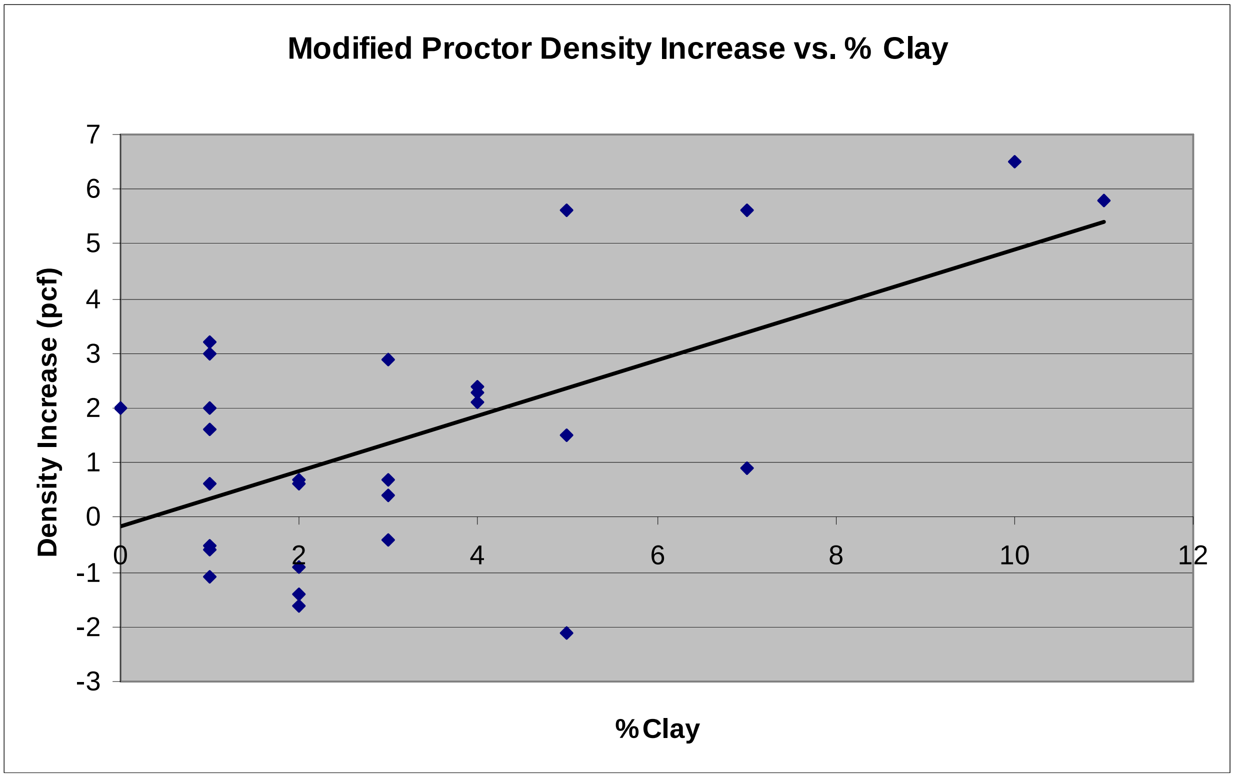

Figure 4-3. Density increase versus % clay (mechanical T180 – mechanical T99)

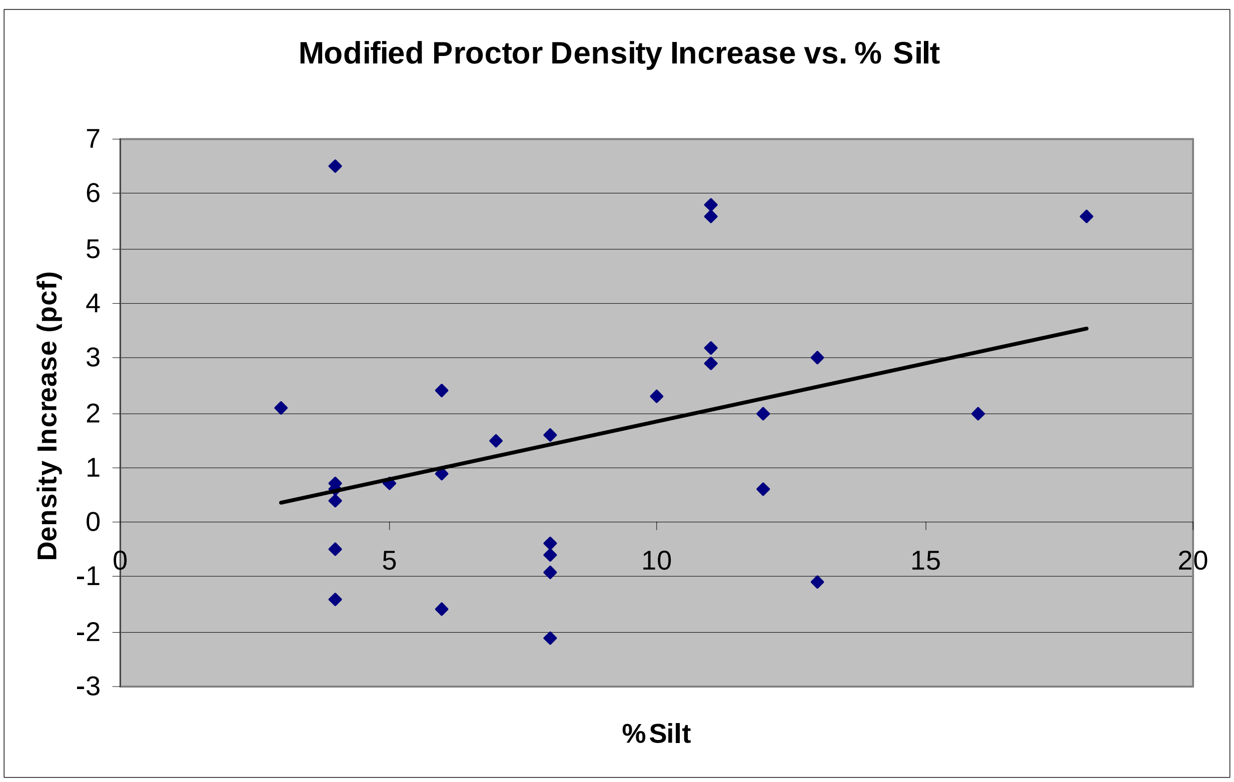

Figure 4-4. Density increase versus % silt (mechanical T180 – mechanical T99)

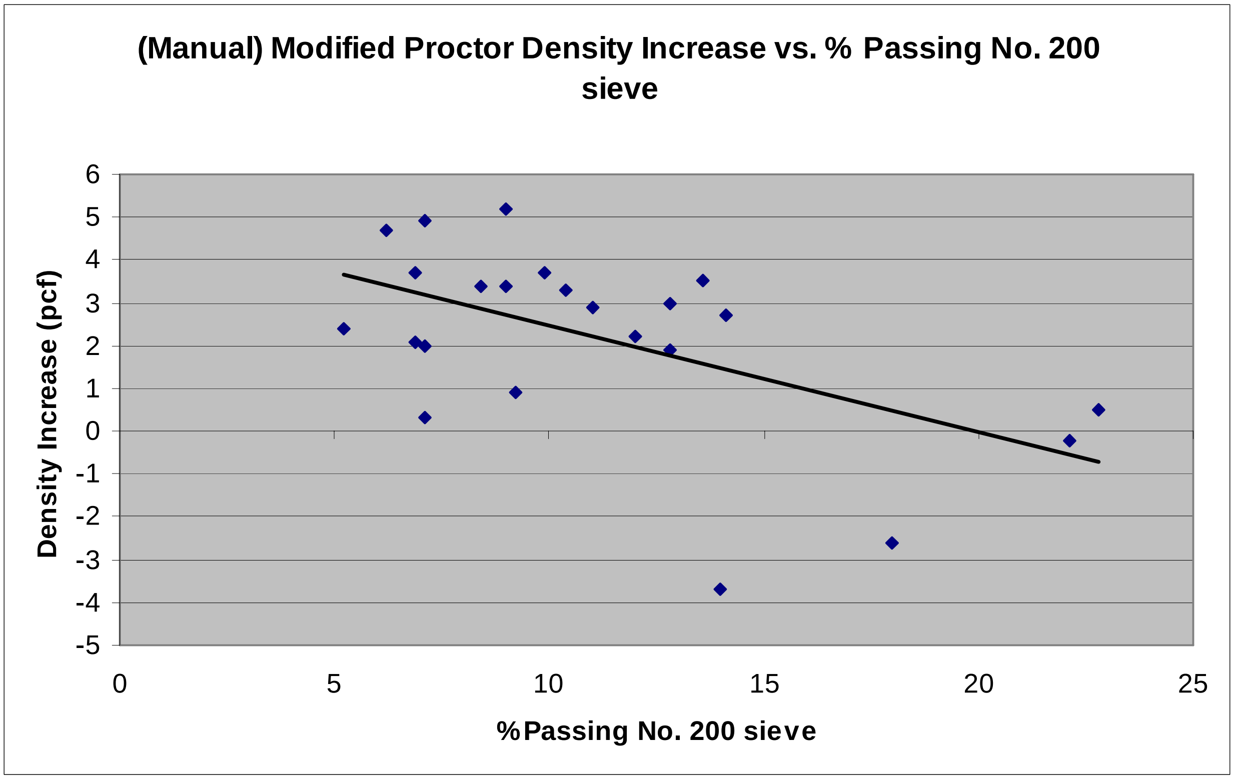

Figure 4-5. Density increase versus % passing No. 200 sieve (manual T180 – mechanical T180)

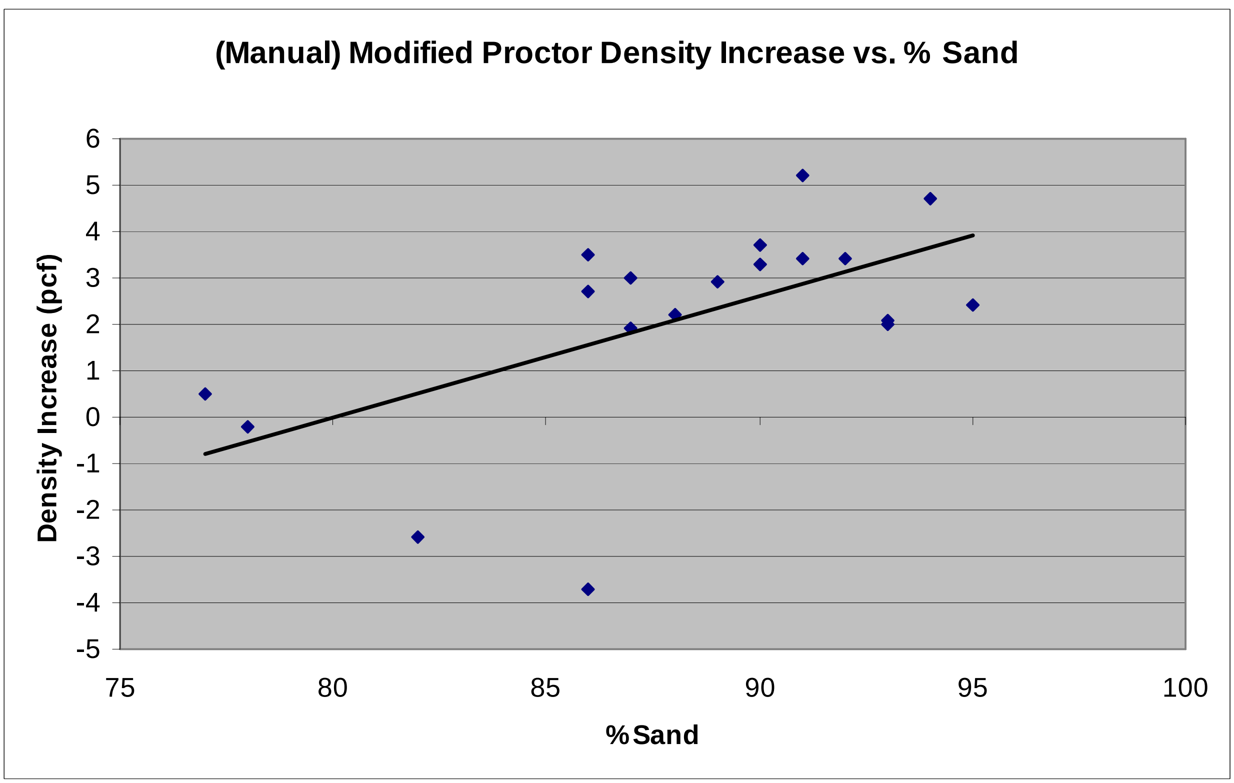

Figure 4-6. Density versus % sand manual (manual T180 – mechanical T180)

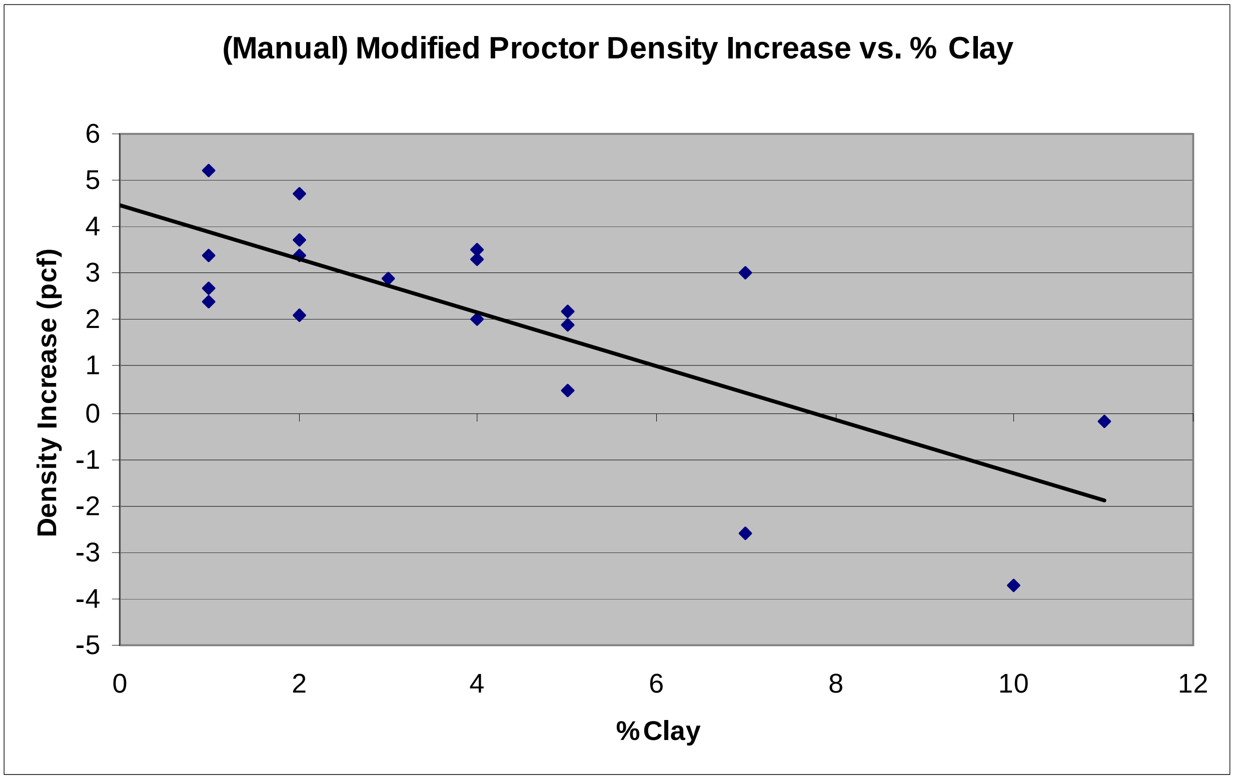

Figure 4-7. Density increase versus % clay manual (manual T180 – mechanical T180)

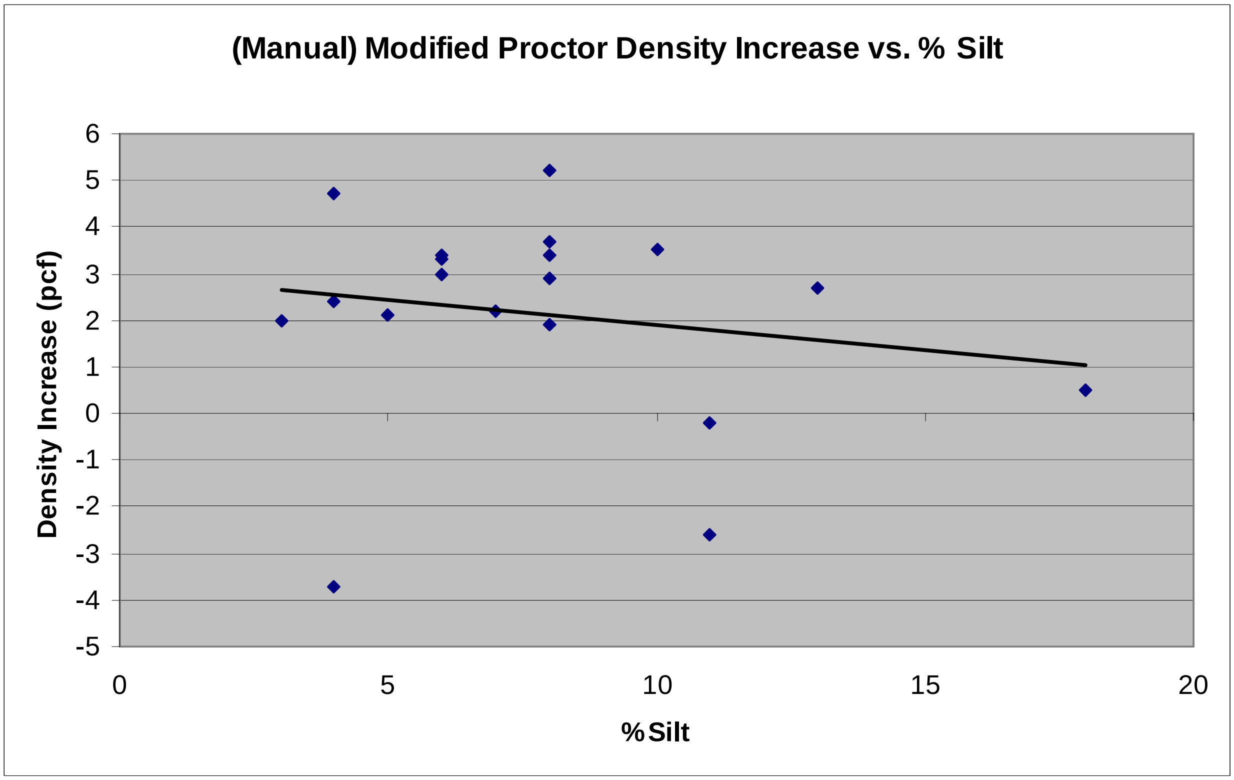

Figure 4-8. Density increase versus % silt manual (manual T180 – mechanical T180)

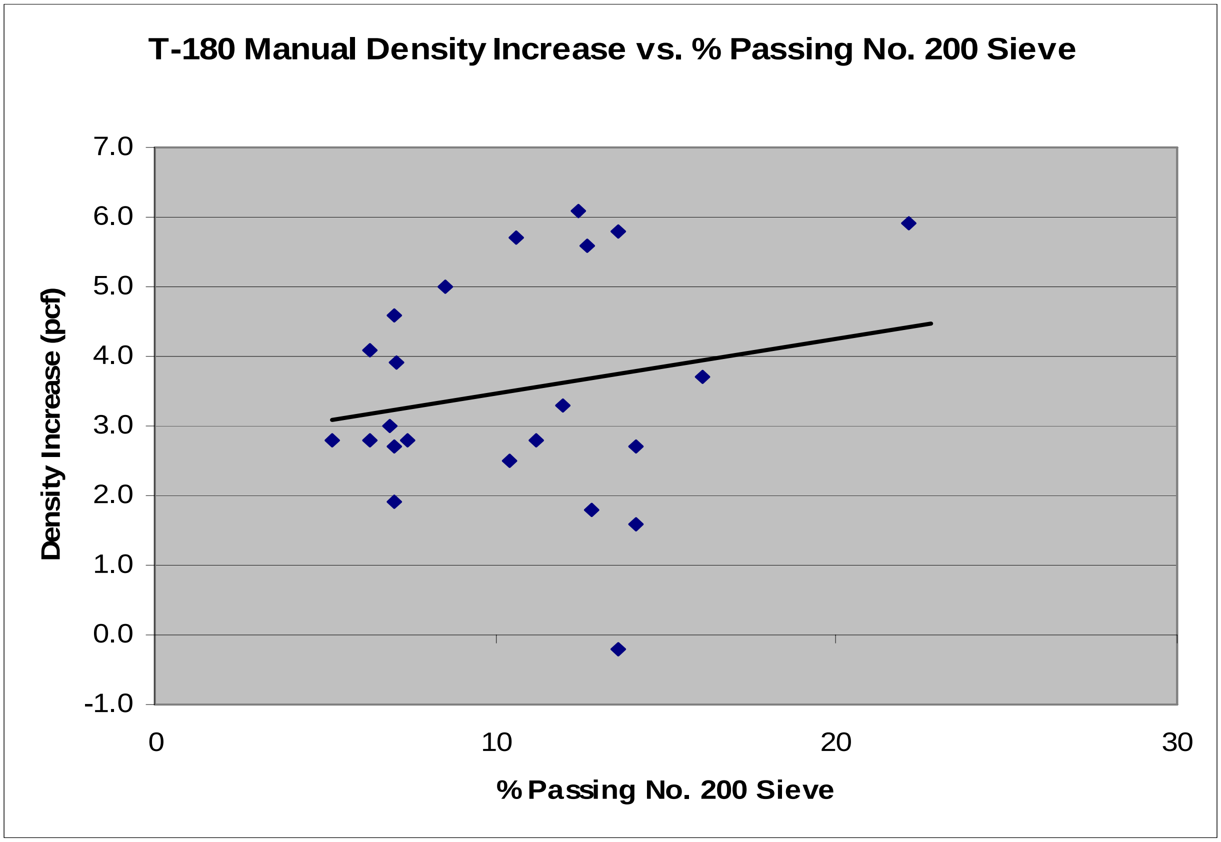

Figure 4-9. Density increase versus % passing No. 200 sieve (manual T180 – mechanical T99)

CHAPTER A-5
DISCUSSION AND CONCLUSIONS

Reoccurring Issue

It is clear from Table 4-3, column 1 that there is a serious issue regarding the Modified Proctor density not being sufficiently greater than the Standard Proctor density. The fact that 16 of 40 samples experienced a decrease in density when exposed to a compaction effort that delivers over 4.5 times the energy definitely qualifies as a reoccurring issue in need of investigation (Das, 1989).

Soil Type

Soil type and particle composition play an influential role as to what degree this problem has an effect. The A-3 and A-1-b samples had a considerably lower average density increase from Standard Proctor to Modified Proctor when compared to the A-2-4 samples (Table 4-3, column 1). Also, Figures 4-1 through 4-4 all show that the lack of fine particles prevents the Modified Proctor density from appropriately increasing.

This lack of fine particles, most importantly clay, truly contributed to the A-3 and A-1-b samples not receiving a reasonable density increase when exposed to a greater compaction effort. These soils, which are practically cohesionless, tend to become loosened when exposed to the higher compaction effort of the Modified Proctor test. Figures 5-1 and 5-2 show the before and after pictures of the final lift of a Modified Proctor test on A-3 material number 16 (Table 4-1). The before picture (Figure 5-1) shows the soil as lightly tamped and smooth, while the after picture (Figure 5-2) shows how the soil became loosened instead of compacted. This loosening is due to the sector head cutting or digging into the soil and certainly prevents the rammer from delivering its full compaction energy. This cutting and digging was prevalent among samples which lacked fine materials and is a major source of energy dissipation during the Modified Proctor test.

The A-2-4 samples, on the other hand, had a much higher average density increase in Table 4-3, column 1, due primarily to their cohesive properties. The A-2-4 materials did not experience this extreme loosening or digging and therefore showed a more reasonable density increase when compacted by the Modified Proctor test.

In the A-2-4 samples, the higher compaction energy was delivered more efficiently and therefore created densities that were more appropriate. Using the Modified Proctor test with the sector shaped head may simply be too much energy for a soil of A-3 or A-1-b characteristics to withstand and should be seriously considered when interpreting their test results.

Manual Compaction

It is clear from Table 4-3, column 2 and Figure 4-9, that the use of the manual rammer with a round shaped head for the final two lifts brought most of the densities and the averages back into an appropriate range. It raised all but one Modified Proctor density back above the Standard Proctor density, where it should be. Figure 4-9 shows there is no longer a correlation between density increase and fine particle percentage. This means the manual compaction brought the densities up almost equally and independent of the soil makeup. This is a more desirable result considering the Proctor test should be consistent, regardless of the soil type.

These results were very surprising because although the manual compaction was only performed on the final two lifts, it still had a very noticeable effect on the compaction. The mechanical machines are calibrated yearly to simulate the manual method, but with these samples they certainly did not deliver equivalent results. This is not acceptable by AASHTO standard T-180, which requires the mechanical and manual results to be equal (Standard specifications, 2006).

The main difference between the manual and mechanical methods is the head shape. Although the round head still had digging issues, it was not as extreme as the sector head. The head shapes do share the same surface area, but the round head is less prone to cut the soil than the sector head and would therefore deliver more compaction energy to the sample.

Table 4-3, column 3 and Figures 4-5 through 4-8 show how the A-2-4 soils did not experience as high of a density gain when manually compacted for two lifts. The higher percentage of sand and lower percentage of clay samples experienced the greatest increases when compacted manually for two lifts. The silt content, however, showed little effect on the compaction increase. This is further proof that the A-3 and A-1-b soils, due to their composition, experience the greatest energy dissipation when compacted by the mechanical Modified Proctor machine. These compaction energy losses must be accounted for if results from cohesionless soils are expected to be accurate.

Conclusions

This study was conducted in order to identify a problem with the density relationship between the Standard and Modified Proctor tests. Once identified, the problem was researched in order to diagnose the soil types most prone to experience this problem. Finally, the issue was explored to discover possible explanations and viable solutions. It was shown that there is a dissipation of energy during compaction by the Modified Proctor method, specifically when using the mechanical machine and when performed on A-3 and A-1-b samples, which have exceptionally low clay percentage. It is proposed that this energy dissipation is due to a lack of cohesion in the soil combined with the sector shaped rammer head impacting with a large amount of energy. Therefore, when performing a 6 inch Modified Proctor density test on these specific soil types, energy dissipation will occur and an unreasonably low density will result. The Proctor density test was developed decades ago on the other side of the country and it is not surprising that it contains limitations when exposed to the distinctly unique soil types found in Florida. It is very important, however, that these limitations be identified and considered during design.

Limitations

Several of the tests were performed by two separate operators, and although AASHTO standards were followed, slight variations in results could occur. Operator error is possible when performing the manual compaction of the 2 layers due to lack of mechanical precision. The results found in this study could certainly use more data in the future to reinforce the proposed trends.

Suggestions for Future Research

Future research is highly encouraged on this subject. Specifically, by pinpointing the source of energy dissipation by precise measurement, it would become clear how to resolve the issue. Strain gauges and accelerometers could be attached to the rods of both the mechanical and manual rammers to record the energy at impact. This equiProject Managerent can be used to compare the energy delivery differences between the mechanical and manual rammers as well as different soil types. Soils that experienced the digging during compaction should show a slower energy delivery due to the dissipation of energy while cutting into the soil. Soils that compact easily and do not experience digging should deliver the energy faster in a more solid manner. There is no question it is the same amount of energy regardless of soil type, however the manner in which it is delivered will certainly change. Wireless equiProject Managerent should be used due to the rotation of the rammer.

Since the Proctor test may not be the most appropriate method to compact cohesionless soils, research could be performed to compare other forms of laboratory compaction methods. Static compaction and Vibratory compaction methods may yield more reliable results.

The sector head could be replaced with a shape less prone to cutting and digging. This would be difficult because it would have to match the surface area of the sector head yet still impact all areas of the sample evenly. Perhaps a physical modification which will yield more accurate results is necessary.

Figure 5-1. Modified Proctor prior to compaction


Figure 5-2. Modified Proctor after compaction

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APPENDIX B.

POWERPOINT PRESENTATION ON PROGRESS TO DATE

Comparison of the Soil Stress Gage (et. al.) with Performance Based Results Using The Heavy Vehicle Simulator

Dave Bloomquist, UF

Dr. Ralph Ellis, UF

Dr. David Horhota, Project Manager, FDOT

HVS

Assessment of Pavement Base Performance Predictors

- Traditionally
  - Density
  - LBR
- More recently emerging tools
  - SSG
  - SPA

Project Objectives

- **Follow-up study evaluating the SSG as a predictor of long-term base performance – stiffness vs. deformation.**
- **To develop a more rational understanding of the relationship between pavement base performance and its measured rigidity (as opposed to density) using the SSG.**
- **Concomitantly to better understand the effects of soil moisture, time effects, and multiple wheel loadings on the both the base performance and the measurement (data acquisition) process.**
- **Utilize FDOT's unique HVS to apply "real world", consistent, loading to the pavement support system**

GATOR Engineering

Research Procedure

- **The two outside test pits at the FDOT State Materials Research facility (SMO) is filled (6 to 12 inch lifts) with approved sub-grade and base materials.**
- **Material properties acquired:**
  - **Multiple density, stiffness, moisture content, seismic pavement analyzer data, hand cone penetrometer data taken. (Note: time is also a variable – hence its effect on the above parameters will be evaluated.)**
- **Additionally, soil stress cells are placed in the soil strata at various levels as the pits are filled.**

Research Procedure

- **Wheel load testing of the pits then commences utilizing the HVS.**
- **Various loading patterns (number of passes, wander and tire pressure values = currently 80 psi) is specified.**
- **Preliminary results evaluated and based on the initial findings numerical modeling to ultimately predict future performance for different soil conditions.**
  - **Varying base stiffness (100, 90, 80%)**
  - **Densities: 98% Modified Proctor (control section)**
  - **Moisture contents: optimum (construction), drained (long-term) & high DHW (worse case)**

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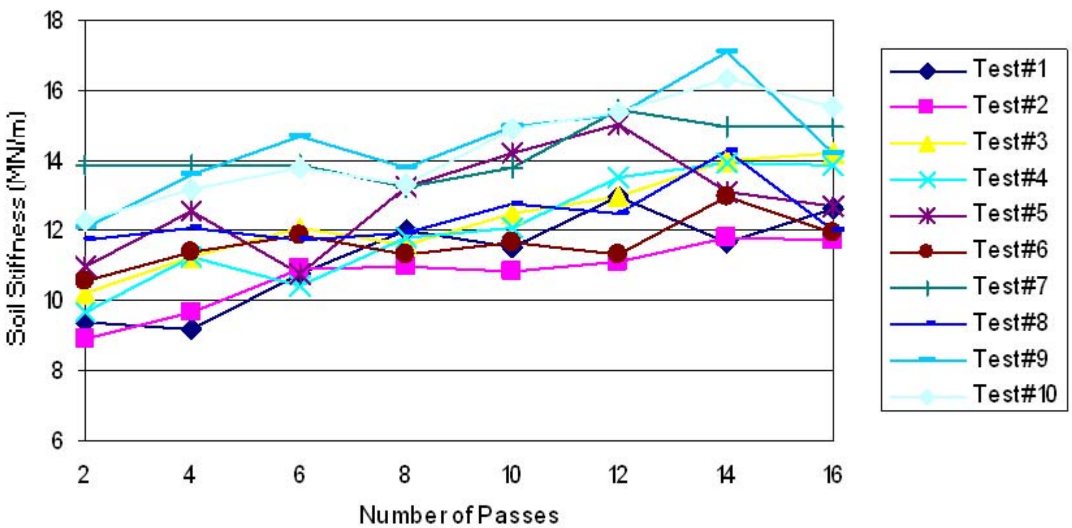
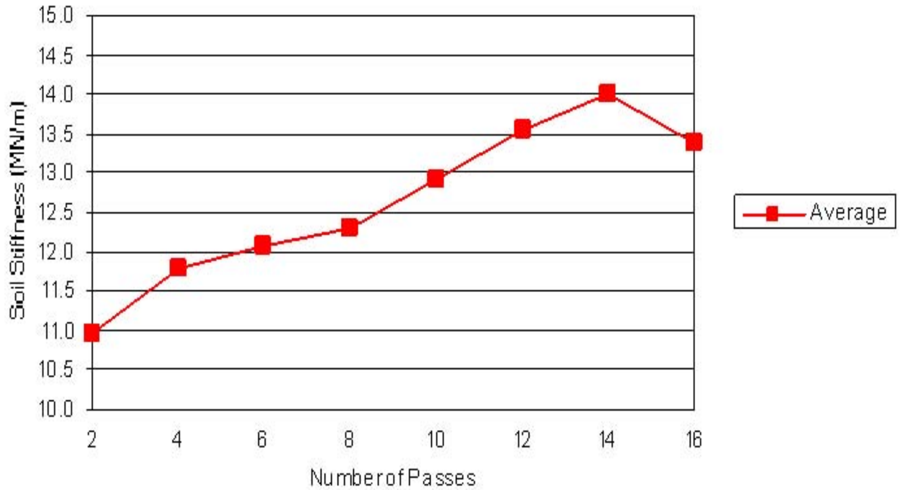
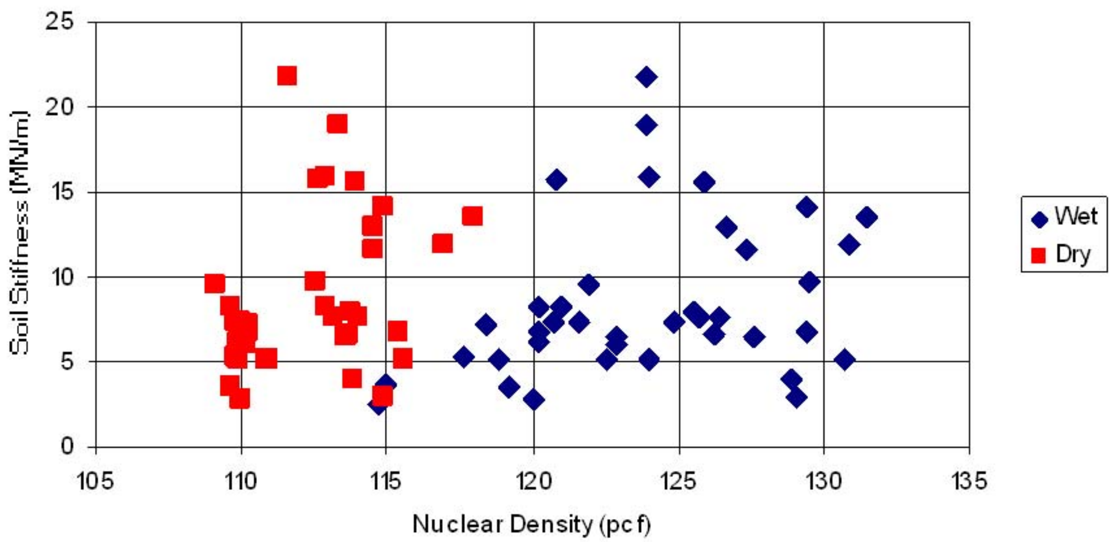


18
16
14
12
10
8
6
2
4
6
8
10
12
14
16
Soil Stiffness (MN/m)
Number of Passes
Test#1
Test#2
Test#3
Test#4
Test#5
Test#6
Test#7
Test#8
Test#9
Test#10

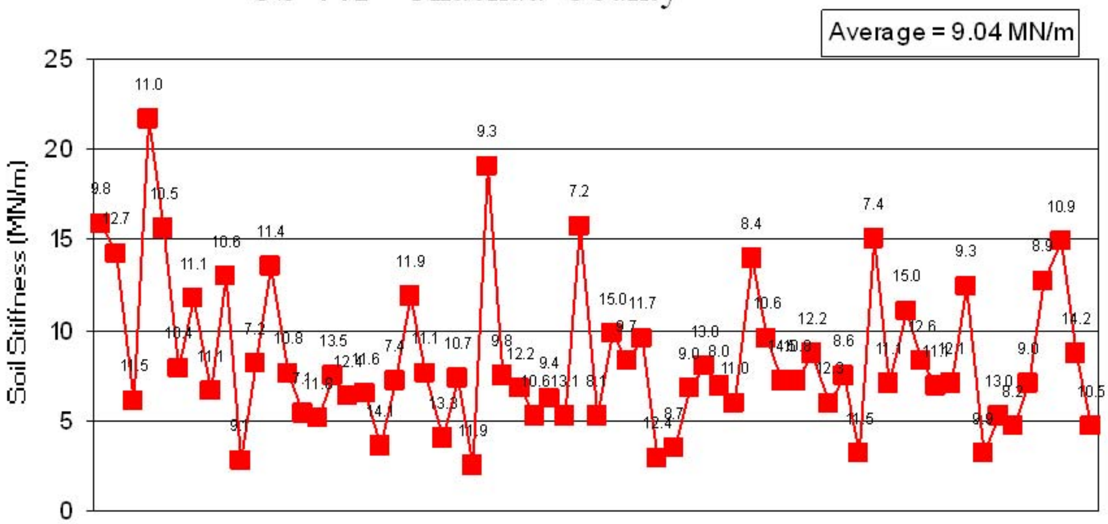
Field Test to Determine Maximum Stiffness as a Function of Compactive Effort





Soil Stiffness (MN/m)
Nuclear Density (pcf)
Wet
Dry
0
5
10
15
20
25
105
110
115
120
125
130
135

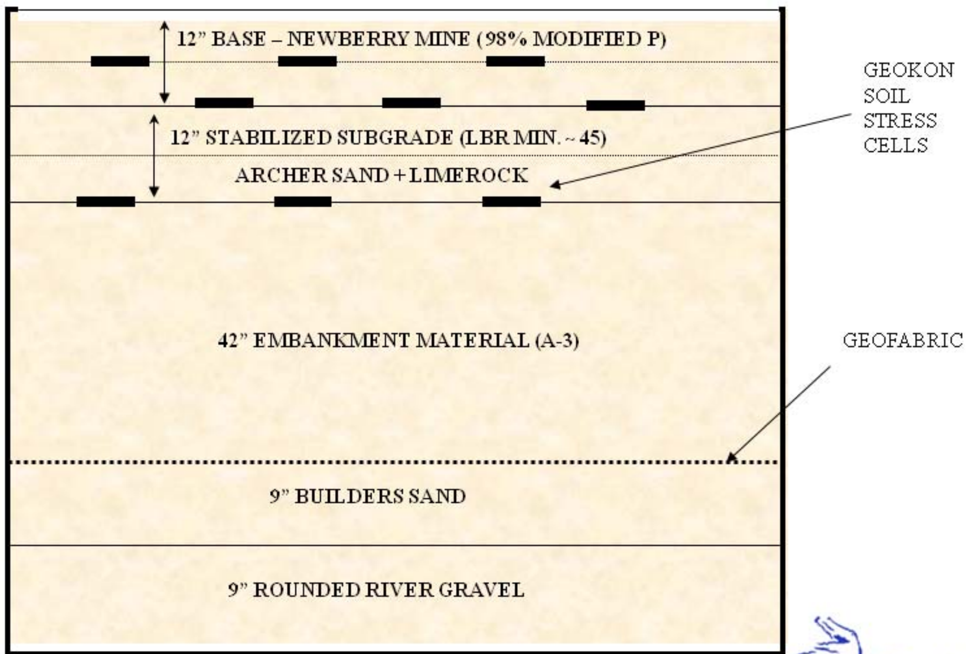
US 441 – Alachua County



Test Pits before alterations/filling.

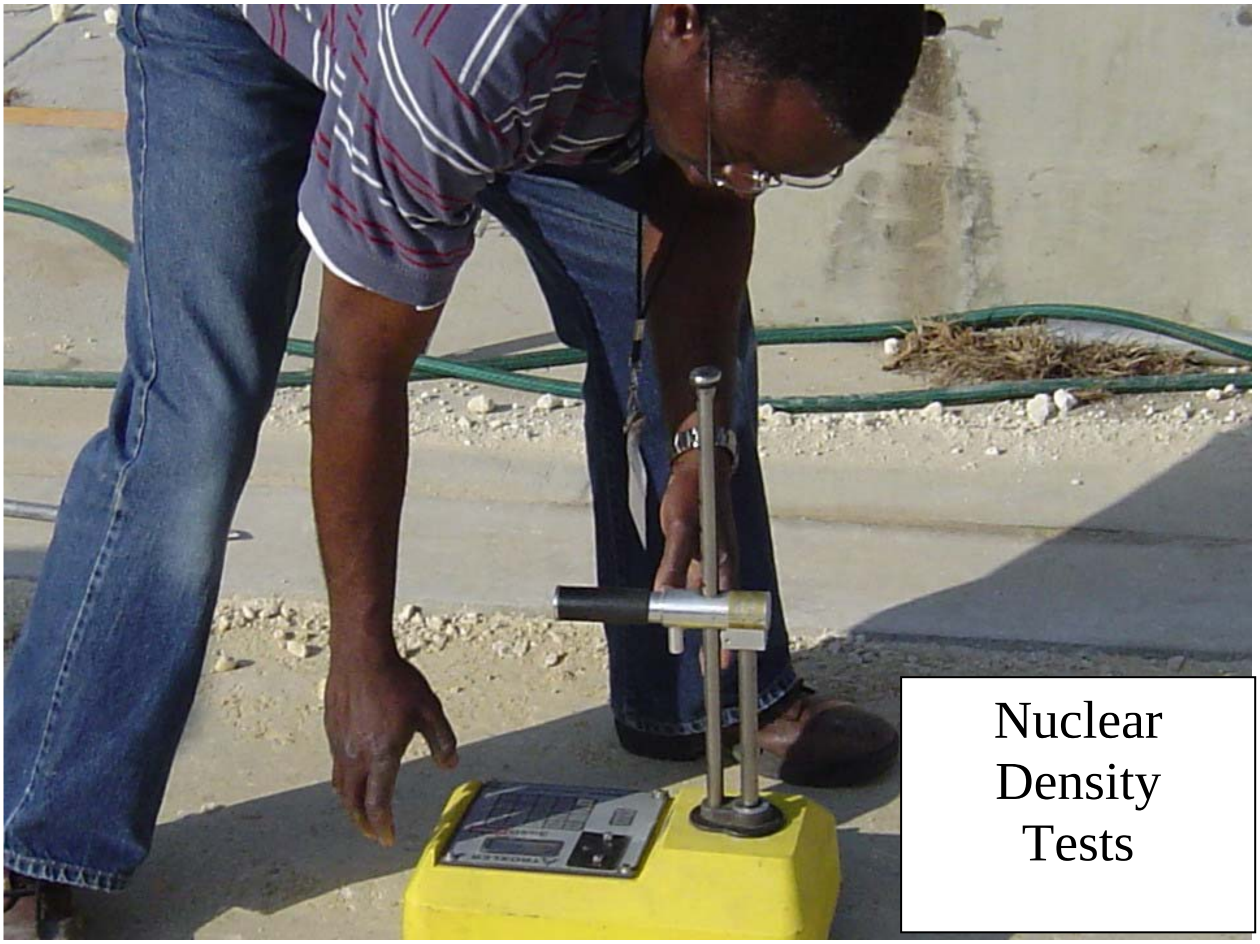
12' x "44'" x 6.5/7.5' **18' x 44' x 6.5/7.5'**

TEST PIT CROSS-SECTION – PRELIMINARY TRIAL RUN



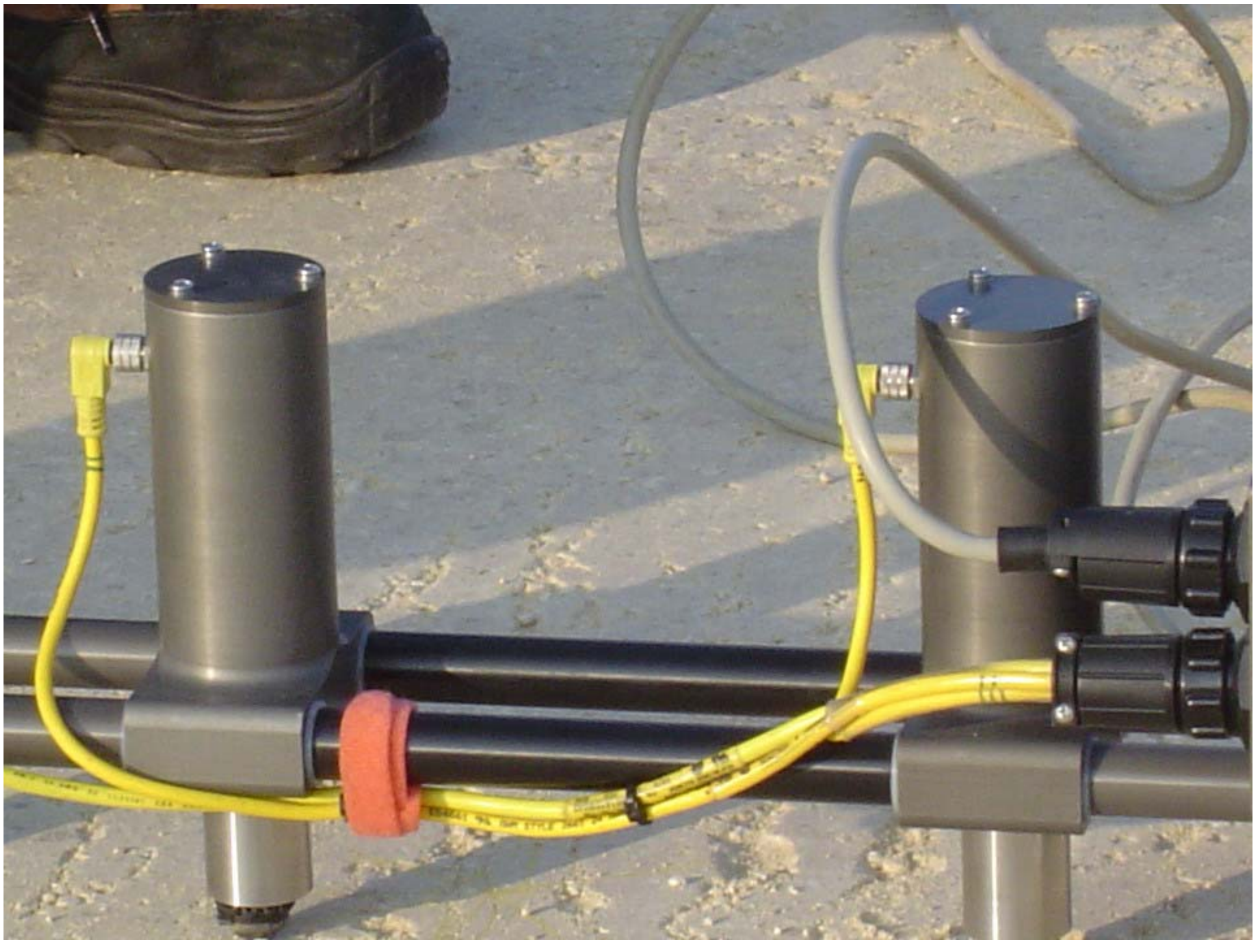
ORIGINAL STRESS CELL AND NEW AGGREGATE STRESS CELL

SSG Measurement Taken on Test Pit



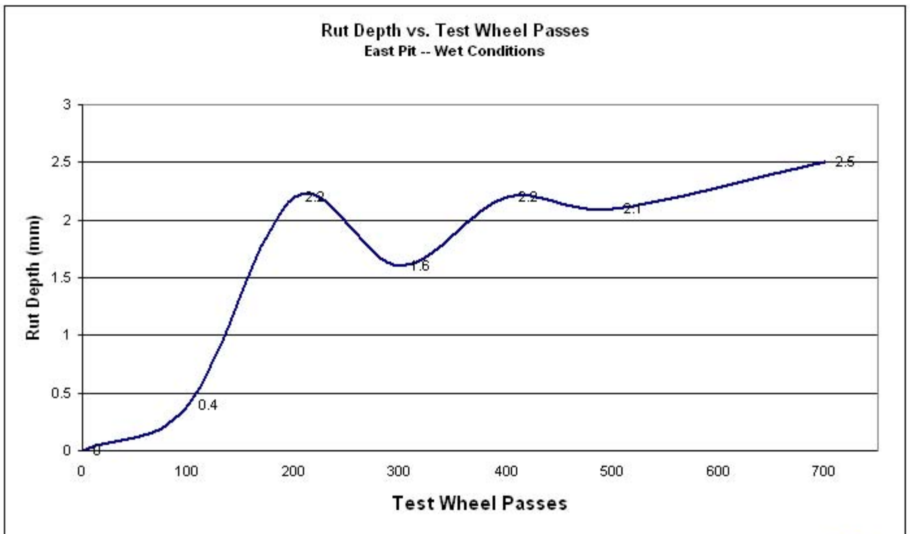
Nuclear Density Tests

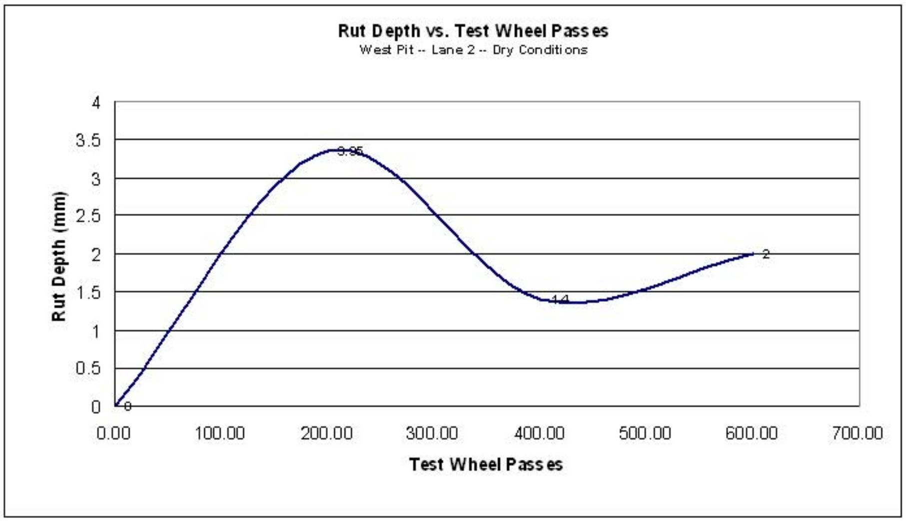

Geophone Testing



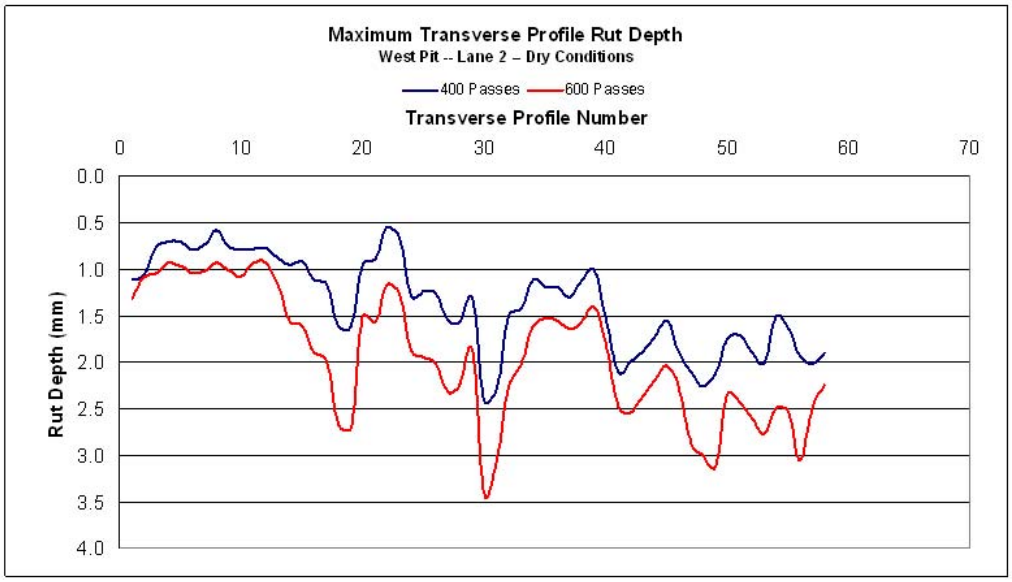
HVS ready to be positioned
Over pit.

'Soaked' Moisture Condition (base clearance @ 18 inches):

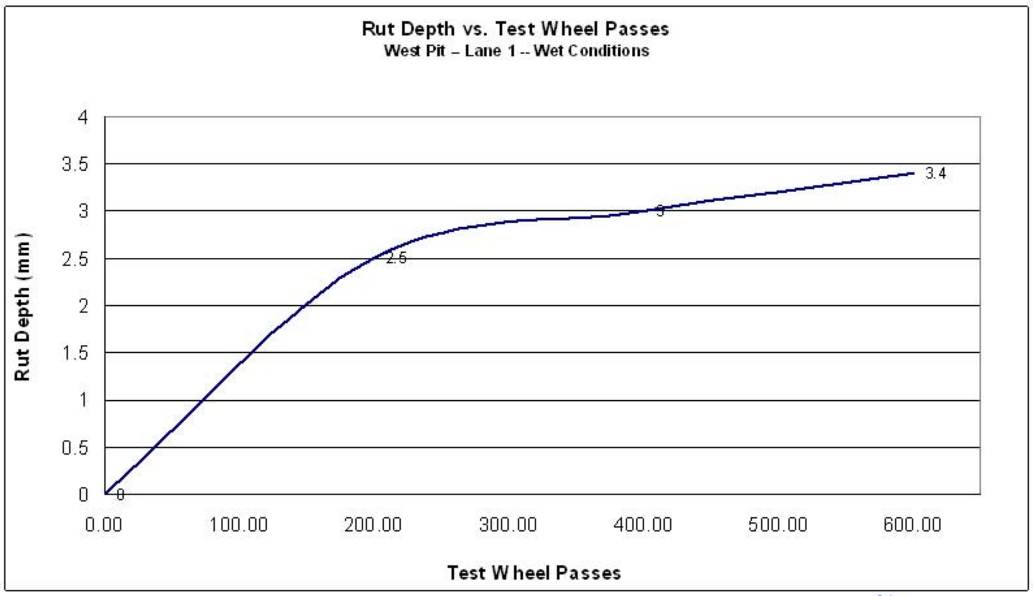


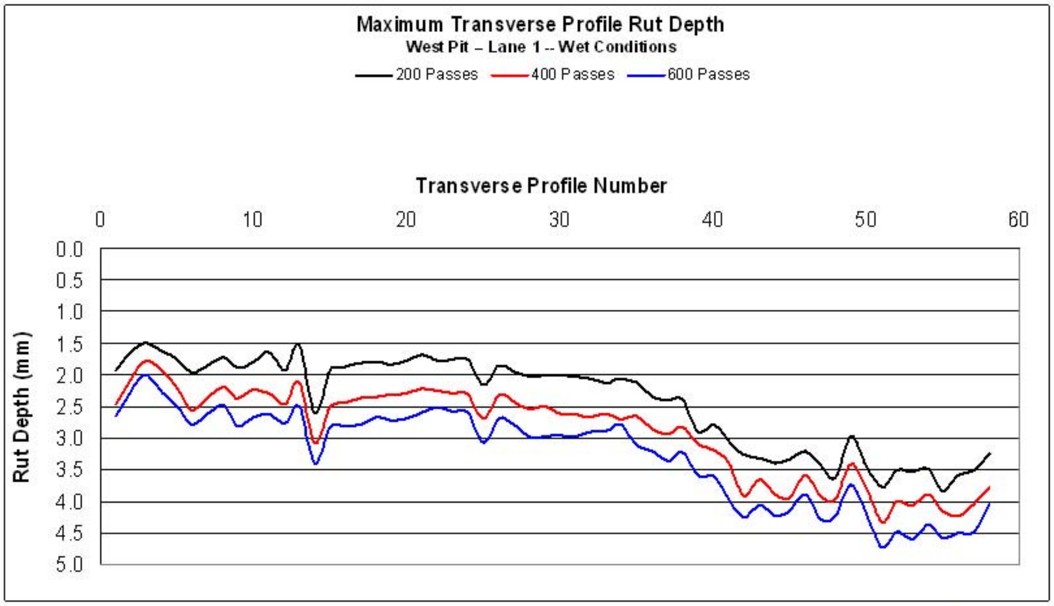


Rut Depth vs. Test Wheel Passes
West Pit -- Lane 2 -- Dry Conditions
Rut Depth (mm)
4
3.5
3
2.5
2
1.5
1
0.5
0
0.00
100.00
200.00
300.00
400.00
500.00
600.00
700.00
Test Wheel Passes
0
3.35
1.4
2



Maximum Transverse Profile Rut Depth
West Pit -- Lane 2 -- Dry Conditions
400 Passes
600 Passes
Transverse Profile Number
Rut Depth (mm)
0
10
20
30
40
50
60
70
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0


Rut Depth vs. Test Wheel Passes
West Pit – Lane 1 -- Wet Conditions
Rut Depth (mm)
Test Wheel Passes
0
0.5
1
1.5
2
2.5
3
3.5
4
0.00
100.00
200.00
300.00
400.00
500.00
600.00
0
2.5
3
3.4



Maximum Transverse Profile Rut Depth
West Pit -- Lane 1 -- Wet Conditions
200 Passes
400 Passes
600 Passes
Transverse Profile Number
0
10
20
30
40
50
60
Rut Depth (mm)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0

Project Status

- Completed one test cycle – iron out “bugs”
- Data collected and analysed – to establish a SOP for HVS base testing
- Address drainage issues with pits
- Prepare for next test – density based control section/stiffness improvement curves – verify numerical model
- Statistically Based Earthwork Compaction QC

GATOR Engineering