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# ANALYSIS OF TEST SECTION DATA: PHASE III OF VARIABLE TIRE PRESSURE STUDY

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**Letter Report**

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# PREFACE

This report documents analysis of data collected during one portion of a multi-year investigation into the effect of different tire pressures on performance of thaw- weakened pavement structures. The entire investigation has been referred to as the Variable Tire Pressure (VTP) project. The portion documented in this report is Phase III of the VTP project. The lower portion of the test section was constructed in 1996 (for Phase II testing) with the upper portion being reconstructed during the summer of 2000 for the Phase III work. The reconstructed test section used a different base course than the previous studies.

The VTP project was formulated and the test sections constructed and tested by Ms. Maureen Kestler while employed at the Cold Regions Research and Engineering Laboratory. Upon her departure, Dr. Marian P. Rollings was assigned analysis of the Phase III data and preparation of this report. Many individuals had worked on this multi-year project, but during analysis and report preparation the only person who had knowledge of the VTP Phase III project and remained at CRREL was Mr. Charles E. Smith. In this capacity, Mr. Smith provided an invaluable link to the completed work. He also conducted the forensic investigation after completion of trafficking, assisted with data reduction and analysis, and wrote summaries of the instrumentation used. Ms. Sherri Orchino conducted post-trafficking laboratory analyses for characterization of materials used in the test section and prepared summaries of the voluminous data.

This report was prepared by Dr. Marian P. Rollings under the general supervision of Dr. Justin B. Berman, Chief, Applied and Military Engineering Branch, and Mr. James L. Wuebben, Director, ERDC/CRREL. Financial support for this effort came from the Federal Highway Administration.

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# CHAPTER 1
# INTRODUCTION

Pavements in northern regions are subjected to annual cycles of freezing and thawing that can negatively affect pavement performance in one of several ways. Repeated freeze–thaw cycling in the presence of available water can result in heaving of the pavement structure if frost-susceptible materials were used in pavement construction. Proper design with non-frost-susceptible materials can eliminate this problem. However, the solution is often costly, and for many low-volume roads, the initial cost of high quality construction is not warranted or cannot be afforded. Another pavement performance problem introduced by freeze–thaw is that of rapid deterioration of pavements subjected to loading during spring thaw. In spring, as the weather warms, the surface of the frozen pavement structure begins to thaw. When high quality, free draining materials have not been used in the upper pavement structure, a layer of water-rich materials are underlain by frozen (hard, impermeable) material. When pavements in this condition are subjected to loads (particularly large loads), large, severe deformations of the pavement structure occur rapidly, leading to premature deterioration and failure. To prevent or minimize this rapid deterioration during the spring when pavements are in a thaw-weakened condition, many organizations prohibit or severely limit truck traffic on these roads. From the time of initial thawing through completion of thaw and draining of excess water from the pavement materials (known as recovery), many northern low volume roads are posted with lower weight limits, severely restricting truck traffic for a period of approximately 6 weeks.

By the late 1940s the increasing tire pressure used in ever larger aircraft had made pavement engineers critically aware of the role of tire pressure in structural calculations for pavement design and for the required quality of materials to be used in asphalt concrete surfaces and aggregate base courses (e.g., ASCE 1950, Waterways Experiment Station 1950, 1953). This became a crucial design and analysis topic for flexible pavements and is covered at length in many standard texts (e.g., Yoder and Witczak 1975). These fundamental concepts of the effect of tire pressure on flexible pavement response led to the concept that it may be possible to reduce damage to pavements by reducing the tire pressure of trucks using low-volume roads. The lower stresses that have to be supported within the pavement structure may allow traffic to continue on thaw weakened low-volume roads, thereby shortening the period when load restrictions have to be imposed. This observation has been the basis for a number of tests by various agencies of the effect of variable (lower) tire pressure on pavement performance, specifically subgrade performance (as the subgrade is usually the poorest quality material, most likely to fail in a thaw-weakened state).

To study the effects of reduced tire pressures on roads underlain by thawing subgrades, a low-volume road test section was constructed in the Frost Effects Research Facility (FERF) at the Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, New Hampshire. The test section was approximately 21 ft wide and 120 ft long; it was constructed of nominally 2 in. of hot mix asphalt, 8 in. of substandard base course, and 75 in. of Hanover silt subgrade. The test section was subjected to freezing to a depth of approximately 5 ft, after which the section was allowed to thaw from the top down. At various depths of thaw, portions of the test section (referred to as test items) were trafficked with high, medium, and low (35, 65, and 100 psi) tire pressures. A total of 12 test items were included in this test section.

Before freezing, during thaw (pre- and post-trafficking), and after recovery (pre- and post-trafficking), Falling Weight Deflectometer (FWD), profilometer, and rut depth probe testing were conducted on the test section. Moisture contents and temperatures were monitored in the test section throughout the test period. After completion of scheduled testing, a forensic investigation of the test section was conducted, and fundamental laboratory material characterization tests were conducted on the materials comprising the test section.

This report will document this test section and present the data collected. A direct data comparison of the effects of tire pressure on rutting (the standard failure method in low volume roads) will be presented. Layered elastic theory will be used to analyze the effect of tire pressure on the test items. The results from the failed sections will be used to back-calculate failure criterion for the base course or the subgrade, or both, for the thawing periods, if possible.

## *1.1 Purpose*

The purpose of this investigation was to determine the effect of three different tire pressures on an asphalt-surfaced, low-volume road section during a simulated spring thaw. To accomplish this, a road test section was constructed in the FERF. The section was artificially frozen from the surface down and then allowed to thaw. By monitoring the depth of thaw throughout the process, trafficking could be applied at selected depths of thaw. As trafficking progressed, surface rutting, subgrade rutting, and surface profilometer measurements were taken to quantify pavement deterioration.

## *1.2 Scope*

This test section was part of one in a series of tests conducted or planned for the Frost Effects Research Facility to investigate the effect of tire pressure on thaw-weakened pavements. This report will present and discuss the materials used, trafficking techniques, and test section data collected during trafficking of this VTP Phase III project. It will also present an analysis of this data as it relates to pavement performance.

## *1.3 VTP Project History*

During a period of years extending from 1996 through 2001, and envisioned to extend several years further into the future, numerous projects related to variable tire pressure, central tire inflation (CTI) vehicles, spring thaw, thaw-weakening properties of paving materials, and unsurfaced, gravel surfaced, and thin asphalt-surfaced low volume roads were undertaken by M. Kestler while employed at CRREL. Kestler envisioned utilizing these disparate projects to provide various pieces of information that, when combined together, would culminate in a new design procedure for low volume roads that would take into account use of lower tire pressures during thaw-weakened spring conditions to allow increased truck traffic during the usual period of posted roads. This would prove advantageous to numerous industries and communities as disruption of normal business, services and deliveries would be minimized each spring.

To accomplish this mission, Kestler had envisioned a four-part program to specifically lead to this new or revised design procedure. Phase I involved computer analysis of low volume road cross sections using thaw-weakened material properties and simulating truck traffic with various tire pressures. This work culminated in a report (Kestler and Berg 1996) that found reduced tire pressures did (analytically) result in reduced pavement deterioration. This report recommended that “field” tests be conducted to verify the findings of this work. Phase II involved evaluation of the performance of two test sections, one surfaced with a 2-in. thick hot-mix asphalt concrete and one surfaced with an asphalt chip seal. Each of these sections apparently included four test items that were subjected to some combination of thaw depth or tire pressure, or both, during trafficking. No published report summarizing this work has been located. Phase III encompasses the project reported in this document. The one test section was surfaced with a hot-mix asphalt and utilized a substandard base course material. Phase IV was envisioned to include finite element modeling of several aspects of low volume road design. At the end of all this work, a new or revised pavement design procedure was to be produced. For various reasons, aspects of these four phases were delayed, postponed, or left unfunded. With the departure of Kestler from CRREL, this Phase III report will complete work on this variable tire pressure project.

As a part of the Phase III test section evaluations, a study was commissioned to evaluate the current status of CRREL’s pavement design procedure and to use it for test section analysis (Berg 2004). This *Seasonal Layered Elastic Design* (SLED) procedure was developed at CRREL during the middle and late 1990’s and has been cited in a number of publications, among them Kestler and Berg (1996), Bigl and Berg (1996), Kestler et al. (1997), and Kestler et al. (1999). SLED is a mechanistic pavement design–evaluation model developed for use in seasonal frost areas. It is composed of four computer programs applied in series (Kestler and Berg 1996). The first program, FROST (or FROST B), is used to predict the amount of frost heave and thaw settlement as functions of time. The second program, TRANSFORM, then divides the pavement into sublayers and assigns resilient modulus values, Poisson’s ratios, and thicknesses. NELAPAV, a non-linear layered elastic program, developed at Cornell University (Irwin and Speck 1985), next simulates a load application and calculates stresses, strains, and deflections at specified points within the pavement structure. The final program, CUMDAM, calculates the cumulative pavement damage resulting from all load applications.

When the recent evaluation of SLED was undertaken by FROST Associates (Berg 2004) for use in the VTP Phase III work, they determined that there are several versions of FROSTB, TRANSFORM, and CUMDAM currently in existence, and no thorough documentation has been published or is available for the SLED procedure or for its constituent models. Berg (2004) determined that the SLED procedure can only be used for design; it cannot be used for evaluation and analysis of existing pavement sections owing to numerical limitations on frost heave values. Because no source code versions of the programs could be located, SLED could not be modified for evaluation and analysis of the VTP Phase III pavement test sections.

## 1.4 General Applicability of VTP Test Results

The VTP Phase III Test Section models a low-volume pavement that freezes deep into the subgrade. Traffic is then applied under four nominal test conditions: section fully frozen, thaw through the depth of the base course, thaw 1-ft into the subgrade, and thaw 2-ft into the subgrade. These represent four separate field conditions:

1. Winter, when the pavement is completely frozen and is in its stiffest condition.

2. Early thaw, where the upper portion of the pavement (surface and base course) have thawed. The pavement has a less stiff pavement structure than in the winter, but the frozen subgrade retains its winter characteristics. The quality of the base will have a major impact on how much deformation occurs in this weakened state.

3. Interim thawing, where the thawing has reached 1 ft into the subgrade. This condition has two potential failure zones: the base course and thawed subgrade. The underlying frozen subgrade would not be a source of deformation and remains stiff.

4. Extended thawing, where the thaw has reached several feet into the subgrade. This is a more severe case of condition 3 above and represents a near end-of-the-thaw period, when deep layers of soft thawed material are trapped over a relatively thin frozen layer.

The trafficking test conditions represent general structural conditions typical of winter and spring and do not try to mimic a specific pavement geometry or climate. Consequently, the modes of behavior observed in the VTP Phase III Test Section represent a stiff, frozen condition, a thawed base condition, a thaw into the subgrade, and a deep thaw into the subgrade with all conditions retaining an underlying stiff, frozen subgrade layer. The materials in the test represent pavements with a competent asphalt surface capable of withstanding loads, a frost-susceptible, low-grade base course, and a highly frost-susceptible, fine-grained subgrade. The observed behavior would be representative of any pavement experiencing the above conditions. Hence, the applicability of the observed behavior extends to a broader class of pavements than the precise geometry and frost–thaw conditions used in the test.

# CHAPTER 2
# TEST SECTION

The VTP Phase III Test Section is located within the U.S. Army Engineer Research and Development Center Cold Regions Research and Engineering Laboratory's (CRREL's) Frost Effects Research Facility (FERF). The test section was a flexible pavement nominally composed of 2 in. of asphalt concrete over 8 in. of granular base course on a sandy silt subgrade. The pavement was frozen to a nominal depth of 5 ft and then allowed to thaw. Simulated truck traffic was applied using CRREL's heavy-vehicle simulator (HVS) when frozen and at nominal thaw depths of 1, 2, and 3 ft corresponding to thaw in the base and 1 and 2 ft into the subgrade. This chapter provides a description of the FERF, the HVS, test section geometry, materials used in the test section, and the instrumentation incorporated into the test section.

## *2.1 Frost Effects Research Facility (FERF)*

The Frost Effects Research Facility is a 29,000-ft$^2$, climate-controlled building located beside the Connecticut River in Hanover, New Hampshire. The facility is dedicated to providing controlled environmental testing, especially cold-climate testing, of large-scale pavement and soil structures. The FERF building is a steel-framed structure with aluminized steel cladding; it is 182 ft long and 102 ft wide. Inside are 12 concrete-lined test basins separated by wood and concrete bulkheads. The test basins are all 21 ft wide; four of the test basins are 12 ft deep and 37 ft long and eight basins are 8 ft deep and 25 ft long. They can be used separately for smaller experiments, or combined to accommodate larger projects. Figure 2.1.1 shows the layout of the FERF and Figure 2.1.2 shows a photograph of the facility. Additional information is available at http://www.crrel.usace.army.mil .

The FERF is large enough to allow placement of soils and similar materials with full-sized construction equipment. It has the capability to simulate variable moisture conditions by introducing and then either maintaining, increasing, or decreasing a water table level in the test structure. Each individual test basin has a built-in temperature range of 155$^o$F; the temperatures can be as low as –35$^o$F and as high as 120$^o$F. Lower and higher temperatures can be obtained by using portable cooling or heating units. Each testing area can be controlled to within ±2$^o$F. The ambient air temperature can be changed at an average rate of 12$^o$F per hour, and soil can be frozen or thawed at a rate of about 1 in. per day, depending on moisture content and soil composition. Test basins are monitored by computer, and remote monitoring is available through telephone modem hookups. In two basins, refrigeration coils are cast within the concrete floor slabs, and freezing panels can be hung vertically along the side walls. By use of freezing panels on the surface and along the walls and the refrigeration coils in the floor, these basins can be deeply frozen and maintained at near isothermal conditions to simulate permafrost.

In most experiments, freezing panels are placed on the surface of the test area, and an artificial freezing regime is applied to the surface only of the section until the target depth of freezing is reached. The interconnected panels use a brine mixture of ethylene glycol as the cooling medium. The temperature in the panels can be varied to simulate winter weather. Typically, the panels are removed to permit the section to thaw. For the VTP Phase III test section, freezing from the top surface only was used; no water table was introduced into the section.

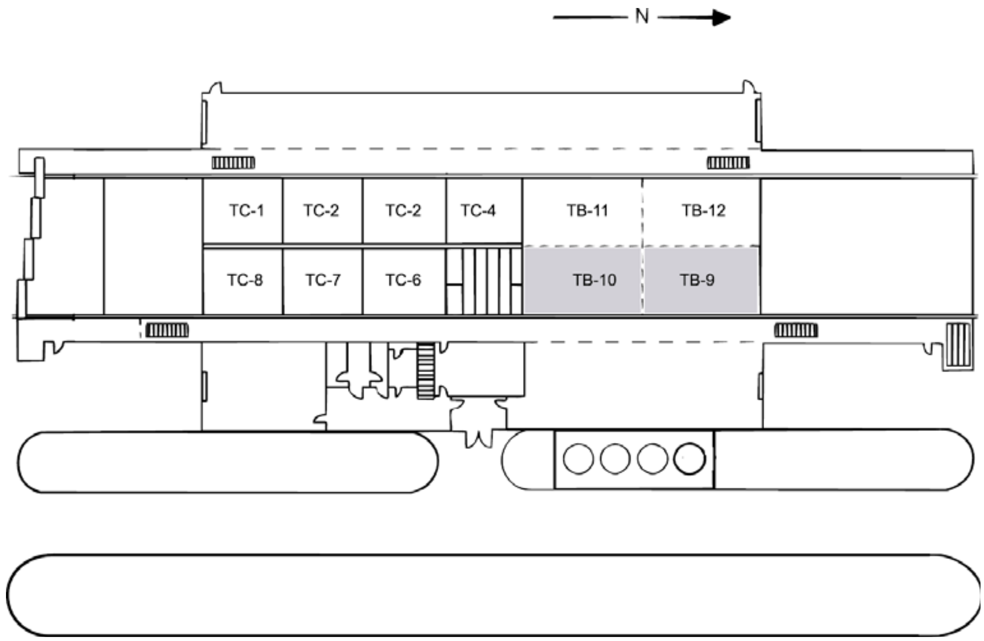


**Figure 2.1.1: Plan section of the Frost Effects Research Facility.**

**Figure 2.1.2: FERF with test sections under construction.**

## 2.2 Heavy Vehicle Simulator (HVS)

The Heavy Vehicle Simulator (HVS) is an accelerated trafficking device developed and manufactured in South Africa. The CRREL HVS is a Mark IV, a modified version of the South African Mark III vehicle. The Mark IV features increased speed capacity, automatic and manual controls, and an electric motor to drive the test carriage. The 75-ft long, 12-ft wide, and 13-ft high device has a mass of approximately 50 tons. It loads a full-sized tire or a pair of tires (dual tires) with up to 50,000 lb using an hydraulic loading system. Traffic can be applied in a single track or in a normally distributed pattern. Generally, distributed traffic seems to cause more rapid damage than traffic applied in a single wheel path, and a distributed pattern is more representative of actual traffic (Ahlvin 1991, Corps of Engineers 1942). The maximum lateral wander of the HVS test wheel is 3 ft. At present, the CRREL HVS is equipped to test dual truck tires, s super-single truck tire, or a C-141 aircraft tire. The HVS applies traffic at approximately 8 to 10 mph and at a rate up to 600 one-way passes per hour. Figure 2.2.1 shows the CRREL HVS in use. Features of the Mark IV are summarized in Table 2.2.1.

**Table 2.2.1: Performance Data for the Mark IV Heavy Vehicle Simulator**

| | |
|---|---|
| Wheel load | Typically 4,500-22,000 lb for roads; 25,000–62,500 lb for airfields |
| Test wheel | Single, dual, or aircraft |
| Tire pressure | Typically 80-100 psi on roads; up to 210 psi on airfields |
| Repetitions, per day | Approximately 18,000 (bi-directional) |
| Trafficked length | Approximately 23 ft |
| Trafficked width | Variable up to 5 ft |
| Trafficked pattern | Variable |


**Figure 2.2.1: CRREL's Heavy Vehicle Simulator in use.**

For the VTP Phase III test section, HVS traffic consisted of a single tire loaded to 9000 lb. When applying traffic, it takes a number of passes for the hydraulic system to equilibrate to a constant load. Actual loads applied varied. All analysis will be based on the nominal 9000 lb per tire. The test tire was 11Rx22.5 Michelin radial tire, and inflation was set at 35, 65, and 100 psi as a test variable. As traffic is applied to a section, there is a small increase in tire pressure from friction. Traffic was applied in a 3-ft wide wander pattern following a normal distribution and was applied in one direction only (north to south). Distributed traffic for accelerated traffic testing is more representative of actual traffic in the field, and previous accelerated traffic tests indicate traffic applied in three or more tire widths is more severe than traffic applied in a single tire width (Ahlvin 1991, Corps of Engineers 1942).

The term *passes* means the maximum number of times the load wheel was applied to the test item in a normal distribution. The term *coverages* represents the maximum number of load repetitions at a point in the test item. For normally distributed traffic, the pass-to-coverage ratio can be approximated as (Yoder and Witczak 1975):

$$\frac{P}{C} = \frac{12T}{0.75wN}$$

where
$P$ = passes
$C$ = coverages
$T$ = traffic wander, ft
$W$ = contact width of one tire, in.
$N$ = number of tires

For the VTP Phase III test section, there is only a single tire and the traffic wander is 3 ft. The tire width can be estimated by assuming the tire is a combination of a rectangle and two semicircles (Yoder and Witczak 1975). The width is approximately 0.6 $L$ where $L$ is the length of the tire calculated as:

$$L = \sqrt{\frac{A}{0.5227}}$$

where $A$ = tire contact area = $\dfrac{\textit{wheel load}}{\textit{tire pressure}}$

Table 2.2.2 shows the pass-to-coverage ratio calculations for the three test conditions used in the VTP Phase III Test Section.

**Table 2.2.2: VTP Phase III Test Section Traffic Pass-To-Coverage Ratios**

| Tire Pressure ($lb/in^2$) | Wheel Load (lb) | Contact Area ($in^2$) | Tire Length (in) | Tire Width (in) | Pass-to-Coverage Ratio |
|---|---|---|---|---|---|
| 35 | 9000 | 257 | 22.17 | 13.30 | 3.61 |
| 65 | 9000 | 138 | 16.25 | 9.75 | 4.92 |
| 100 | 9000 | 90 | 13.12 | 7.87 | 6.10 |
| The pass-to-coverage ratio estimates the maximum accumulation of traffic at a point in a normally distributed traffic pattern. For the 35 psi tire, 1,000 passes of distributed traffic will produce a maximum of approximately 277 repetitions at the center of the traffic pattern (1,000/3.61 = 277). | | | | | |

## *2.3 Test Section Geometry*

For the VTP Phase III project, the southwestern section of the FERF test basin area was used for the test section, which was 125 ft long, 21 ft wide, and 8 ft deep. The basin had a leveling layer of crushed stone at the bottom (to fill a subdrain that was not to be used in this study). Overlaying this was a geomembrane that provided an impermeable barrier. A silt material was then compacted in lifts to form a 6.25-ft subgrade.. A nominal 8-in.-thick, poor quality base course was constructed on the subgrade. The entire test section was paved with a nominal 2-in.-thick hot mix asphalt (HMA) layer. Actual measured thicknesses of the reconstructed portion of the subgrade, the base, and the asphalt concrete layer are given in Table 2.3.1. Mean values of thickness in the asphalt and base course layers were 2.16 and 7.53 in., respectively. The standard deviation from the means were 0.37 for the asphalt and 0.42 for the base course. A cross-section of the test section is shown in Figure 2.3.1.

It should be noted that the lower portion of the test section was built during the initial construction phase of the VTP Phase II project. This construction is reported to have occurred in 1996. Trafficking and testing of the original test section occurred during the period 1998 through early 2000. During the late spring and summer of 2000, the upper portion of the original test section was removed (to some depth within the subgrade). The same silt material was used to reconstruct an upper 7.2-in. portion of subgrade; then a (different) poor quality base course material was placed and a new HMA surface course was placed, resulting in the VTP Phase III test section as shown in Figure 2.3.1.

**Table 2.3.1: Measured Test Section Layer Thicknesses**

| Layer | Thickness, in. | |
|---|---|---|
| | West Side | East Side |
| **Asphalt** | 1.9 | 2.3 |
| | 2.4 | 2.3 |
| | 2.5 | 1.9 |
| | 2.2 | 1.4 |
| | 2.6 | 2.0 |
| | 2.6 | 1.8 |
| **Base** | 7.4 | 8.2 |
| | 7.3 | 7.5 |
| | 7.1 | 7.9 |
| | 7.1 | 8.2 |
| | 7.3 | 7.3 |
| | — | — |
| **Subgrade** | 7.2* | |

* This one value is the only information available for the reconstructed subgrade thickness. It was obtained by surveying after placement of the first lift and again after placement of the final lift. Total reconstructed subgrade thickness was reported to be 2 ft, but no documentation of this thickness was found.

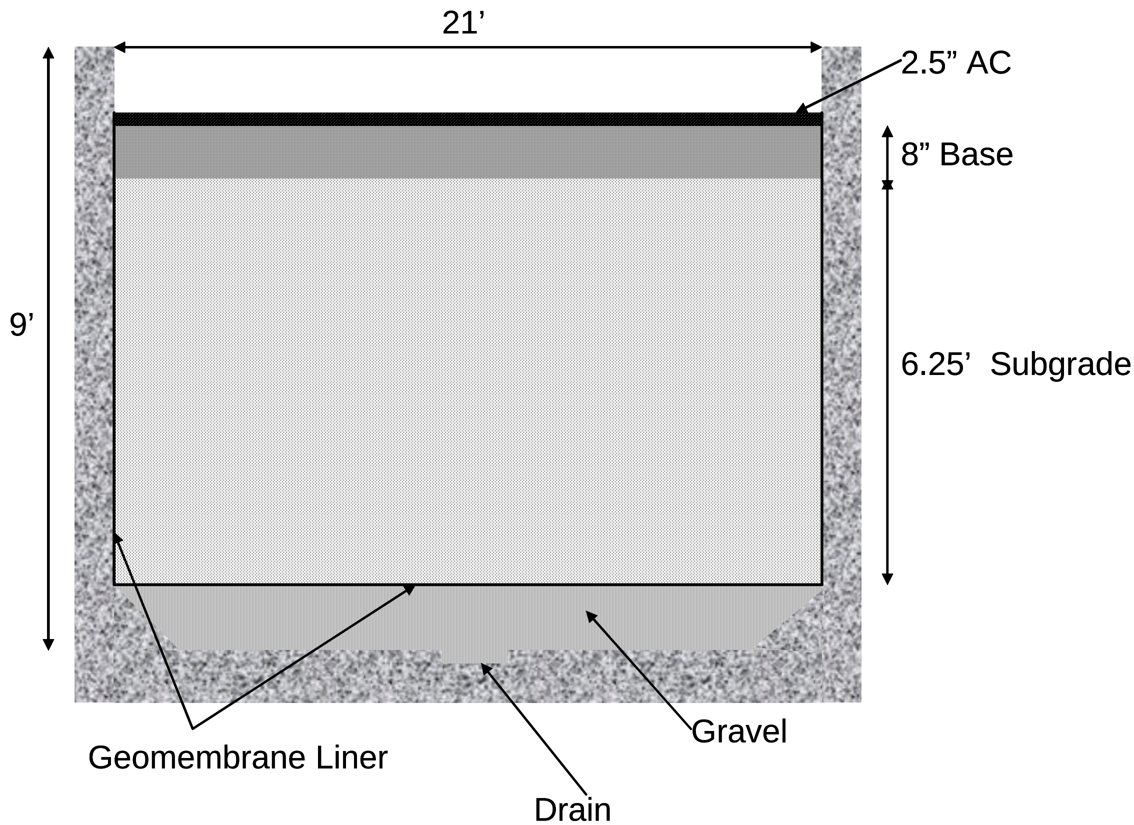


**Figure 2.3.1: Cross-Section of VTP Phase III Test Section in FERF.**

Within the footprint of the Phase III test section, 12 test items were designated; they were laid out three across the test section and four lengthwise, as shown in Figure 2.3.2. The test items were not centered laterally across the test basin. A sketch of the layout is shown in Figure 2.3.2. Each test item was 20 ft long by 3 ft wide to accommodate the footprint of the HVS trafficking window. These test items, with nominally the same pavement cross sections, were identified for HVS trafficking with various tire pressures and at different freeze–thaw conditions.

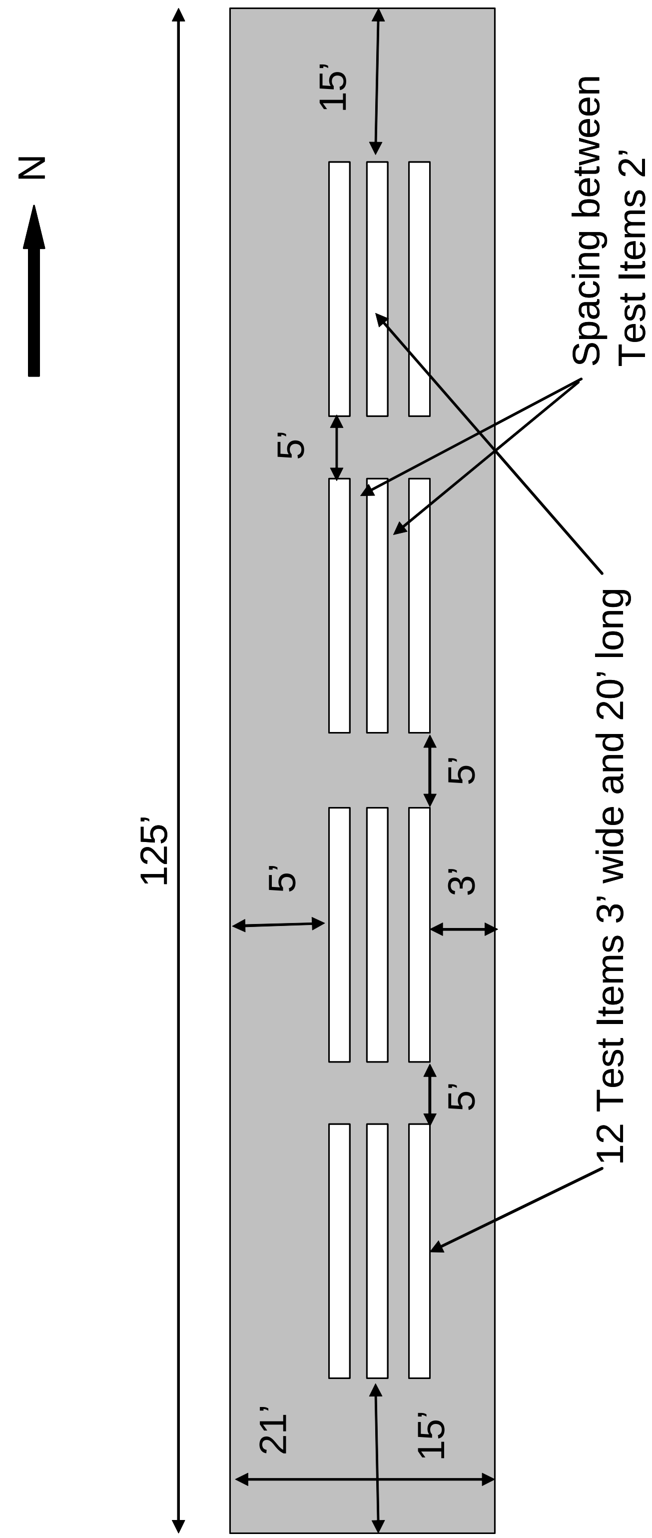


**Figure 2.3.2: Layout of Test Items within FERF Test Section.**

## 2.4 Materials

The VTP Phase III Test Section was built of three materials: an asphalt concrete surface, a granular base course aggregate, and a fine-grained, nonplastic, subgrade soil. The surface asphalt was procured using a New Hampshire State Department of Transportation standard roadway designation. No testing was done on this material nor are there any details available of its composition or properties. It was stable under traffic and was not a contributor to the performance of the test section.

The subgrade soil and base course are local natural materials procured from borrow sources. Representative gradations of the two materials are shown in Figure 2.4.1, and additional information on the materials is provided in Tables 2.4.1 and 2.4.2. The subgrade soil is a fine-grained, nonplastic ML silt (locally called a Hanover silt) under the Unified Soil Classification System (USCS) and is an F-4 highly frost-susceptible soil according to the Army Corps of Engineers frost-susceptibility rating system (Rollings and Rollings 1996). Subgrade soils of this type are notoriously difficult to work, are weak when wet, have significant capillary rise capability, and are capable of supporting large amounts of frost heave when a source of moisture and freezing temperatures are available. This soil creates a difficult and adverse subgrade condition.

The base course is a fairly well-graded, nonplastic, gravelly, silty sand, classifying as an SM soil under the Unified Soil Classification System. Because of its high fines content with two of three samples in Table 2.4.2 exceeding 20%, this base material would classify as an F-3 medium to very high frost-susceptibility material. The high fines content would generally preclude use of this aggregate as a base course, and it would barely qualify as a select fill material with a design CBR of 20 (Ahlvin 1991, Rollings and Rollings 1996). The implications of this for analysis are examined further in Section 4.2 and Table 4.2.2. As this is a natural material, particles are rounded and will not develop the interlock and shear resistance of a crushed material. If well compacted, this material would be expected to show moderate strength when dry, but it would be expected to lose strength rapidly when wet because of the high fines content. It will also be prone to frost heaves and formation of ice lenses. This material represents a low-quality base that would be expected to perform poorly under heavy loads, wet conditions, or on exposure to frost.

Figures 2.4.2 and 2.4.3 provide CE-12 (standard compaction), CE-26 (intermediate level of compaction), and CE-55 (modified compaction) density–moisture content curves, along with curves for soaked and unsoaked CBR for the silt subgrade and the base course material. A comparison of the ML silt subgrade soil’s soaked and unsoaked CBR curves shows that a rise in moisture content will lead to rapid loss in strength. The CBR curves for the SM base material suggest that if the material is well-compacted to or near 100% compaction, as is often the target for granular pavement bases, unsoaked CBR values may be in the 60 to 80 range and the loss on soaking is modest. However, if poor density is achieved, strength will be appreciably less. Some caution is needed in interpreting laboratory CBR values on aggregate materials. The confinement of the mold tends to provide artificially high CBR values in such materials, so one should realize that these are optimistic values. Experience with high fines granular materials such as this suggests they are more likely to behave like a CBR 20 material in the field (Ahlvin 1991).

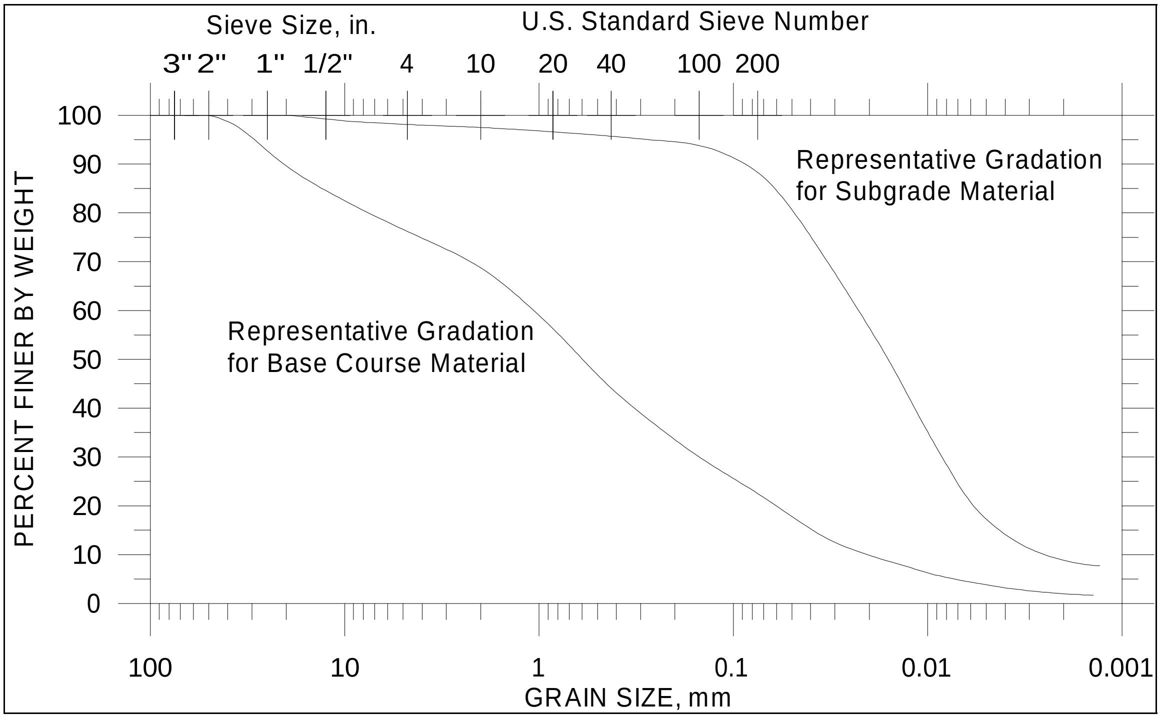


**Figure 2.4.1: Representative gradations of base course and subgrade materials.**

**Table 2.4.1: Subgrade soil characteristics**

| Characteristics | Sample 1 | Sample 2 | Sample 3 | Mean |
|---|---|---|---|---|
| Particle Size Characteristics | | | | |
| Gravel,% | 5.3 | 1.9 | 1.3 | 2.8 |
| Sand,% | 8.9 | 9.8 | 9.8 | 9.5 |
| Fines, pass No. 200, % | 85.8 | 88.3 | 88.9 | 87.5 |
| Silt,% | 67.5 | 71.2 | 71.2 | 70.0 |
| Clay,% | 18.3 | 17.1 | 17.7 | 17.7 |
| Atterberg Limits | | | | |
| Liquid Limit | 0 | 0 | 0 | 0 |
| Plastic Limit | 25 | 25 | 25 | 25 |
| Plasticity Index | 0 | 0 | 0 | 0 |
| Specific Gravity | 2.73 | 2.73 | 2.71 | 2.72 |
| Soil Classification | | | | |
| Unified Soil Classification System | ML | ML | ML | ML |
| AASHTO Classification System | A-4(0) | A-4(0) | A-4(0) | A-4(0) |
| Modified Laboratory Compaction Test | | | | |
| Maximum Density, lb/ft$^3$ | — | — | — | 115.5 |
| Optimum Moisture Content, % | — | — | — | 15.5 |

**Table 2.4.2: Base Course material characteristics**

| Characteristics | Sample 1 | Sample 2 | Sample 3 | Mean |
|---|---|---|---|---|
| Particle Size Characteristics | | | | |
| Gravel,% | 20.7 | 23.8 | 24.5 | 23.0 |
| Sand,% | 57.9 | 53.7 | 57.3 | 56.3 |
| Fines, pass No. 200, % | 21.4 | 22.5 | 18.2 | 20.7 |
| Silt,% | 17.6 | 19.1 | 14.4 | 17.0 |
| Clay,% | 3.8 | 3.4 | 3.8 | 3.7 |
| Atterberg Limits | | | | |
| Liquid Limit | 0 | 0 | 0 | 0 |
| Plastic Limit | 0 | 0 | 0 | 0 |
| Plasticity Index | 0 | 0 | 0 | 0 |
| Specific Gravity | 2.71 | 2.74 | 2.70 | 2.72 |
| Soil Classification | | | | |
| Unified Soil Classification System | SM | SM | SM | SM |
| AASHTO Classification System | A-1-b | A-1-b | A-1-b | A-1-b |
| Modified Laboratory Compaction Test | | | | |
| Maximum Density, lb/ft$^3$ | — | — | — | 134.5 |
| Optimum Moisture Content, % | — | — | — | 6.5 |

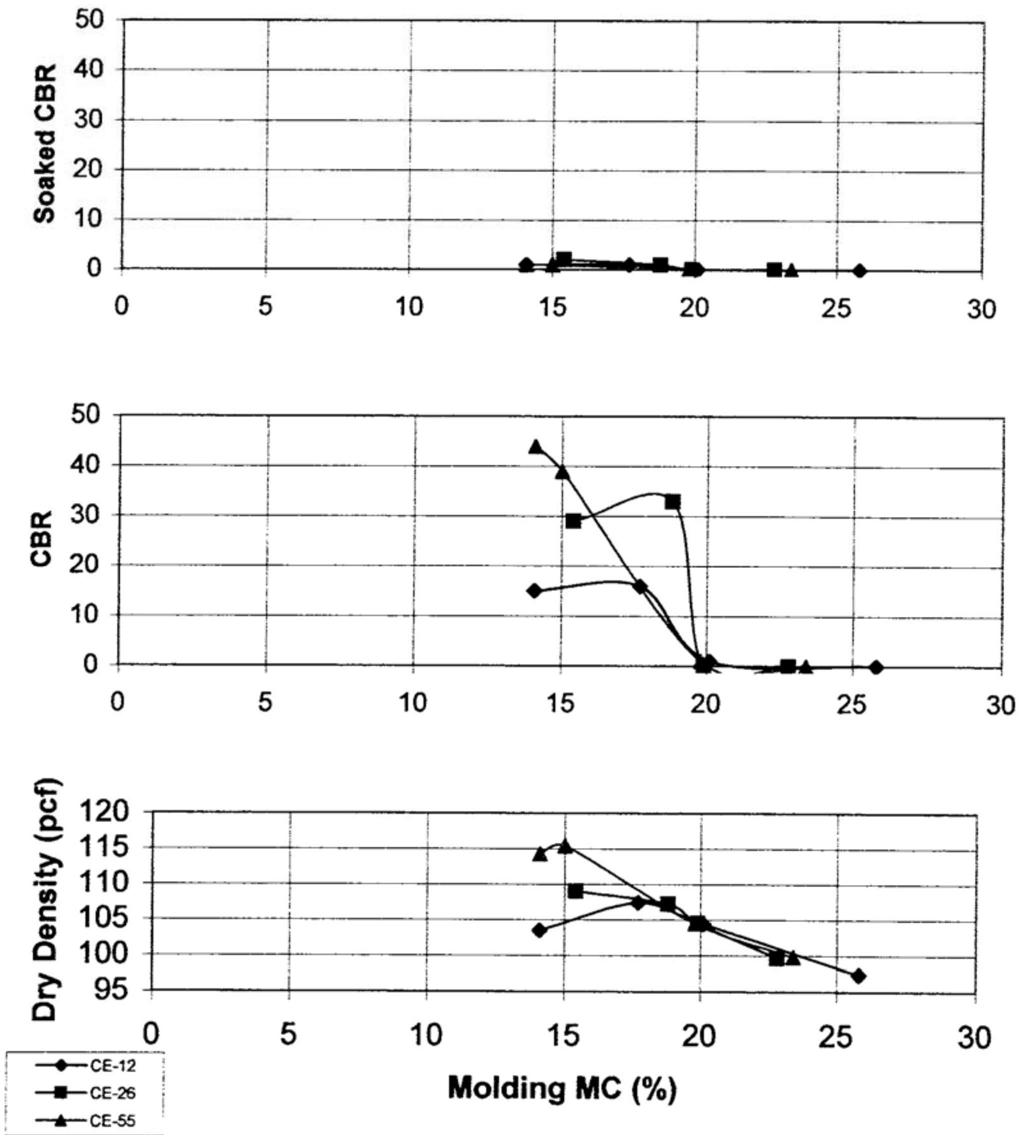


**Figure 2.4.2: Compaction and CBR curves for ML-silt subgrade.**

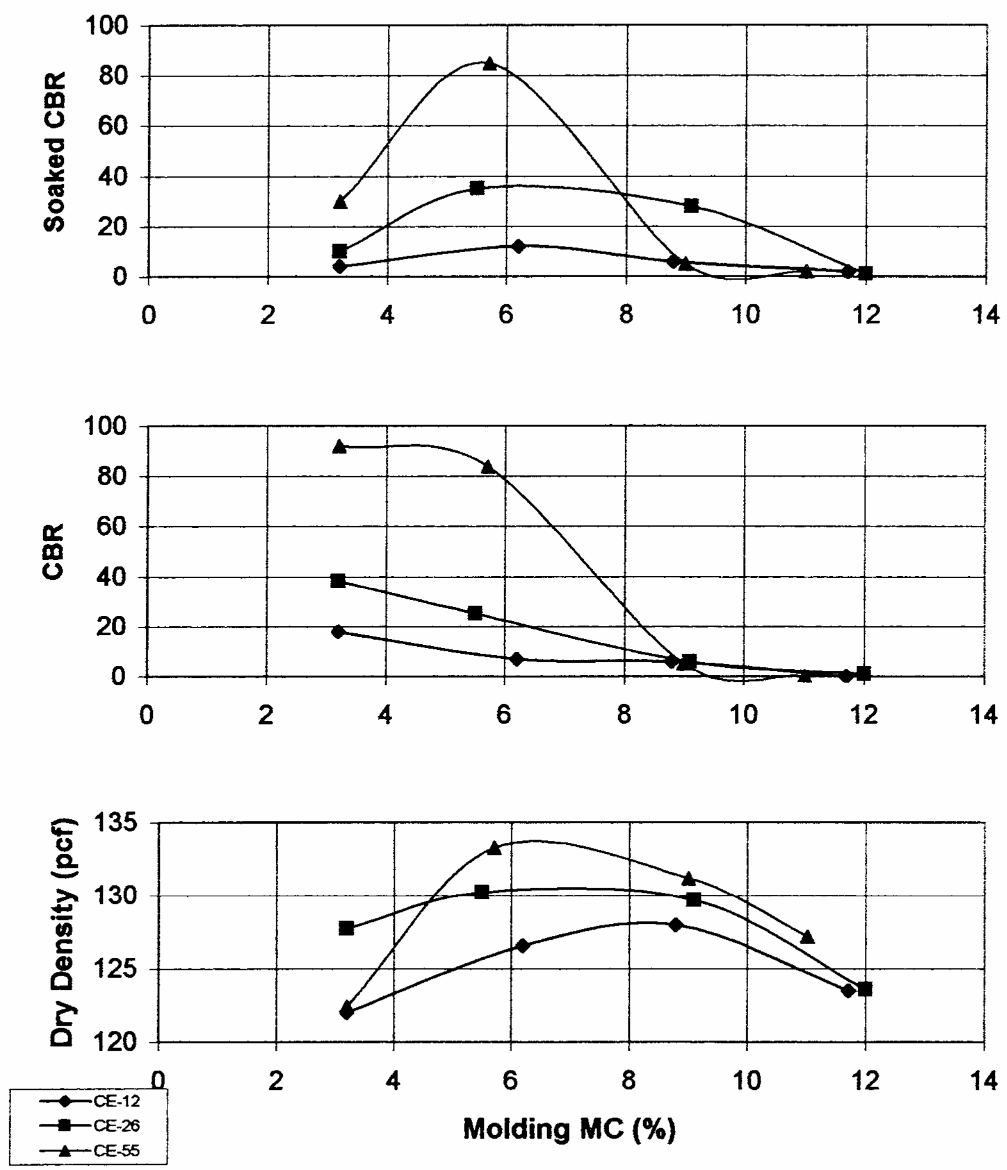


**Figure 2.4.3: Compaction and CBR curves for SM base course material.**

Table 2.4.3 summarizes the available information on placement and post-trafficking conditions for the subgrade and base course materials. The subgrade silt was placed at an average density of 89.7% of the modified compaction laboratory density with variation from 83.8 to 96.1%. This is quite low. It is well below that shown in the compaction and CBR curves in Figure 2.4.2, so they are of little help in estimating possible field strengths. The density measured after all testing was complete shows a significant increase in the average density to 95.9% of the modified laboratory maximum value. The range in percent laboratory compaction has also shown a consistent increase to 90.1 to 101.6%. This strongly suggests that the subgrade will compact under traffic. Six in-situ field CBR values were run on the subgrade after testing. These averaged a CBR value of 6; but 5 of the 6 tests were in the range of CBR 4 to 6. During the actual trafficking of the test items, one would anticipate that the subgrade soil would be at a lower density than the post-trafficking results and that it would be wetter. When the freezing occurred, it drew water from lower unfrozen portions upward to the freezing front, increasing the moisture content in the area where freezing was occurring. The freezing also results in a volume increase, which results in decreasing the material density. Later, when the test items were thawed, the water was trapped in the thawed layer and was unable to drain because of the lower frozen material. Once the entire section was thawed, the water could drain back to equilibrium conditions. The nature of the test would tend to have the thawed sections wetter and less dense (volume expansion when ice forms as well as low initial density) than the post-test conditions when CBRs were measured. Consequently, if the post-test CBR values were in the range of 4 to 6, then the thawed subgrade materials would be extremely weak.

Only pre-test information is available on the base course material. This shows that the base was placed at an average of 93.2% of the modified maximum laboratory density, with values ranging from 92.0 to 95.0. These are relatively low, and if the subgrade compacted under the test traffic, one suspects that the base did also. These data suggest that the base material was placed at between 5.0 and 6.5% moisture. The compaction and CBR curves in Figure 2.4.2 were used to construct the density–CBR relations at lines of constant 5, 6, and 7% compaction moisture content in Figure 2.4.4. The placed density and moisture conditions are also shown on this figure. Recalling that these laboratory CBR values tend to be optimistic for these types of materials, one still observed that the placement conditions of the base would lead one to expect CBR values of less than 10 to perhaps 20. Trafficking damage and further damage to the base when the asphalt concrete surface was removed precluded any meaningful post-traffic measurements of CBR or density. Dynamic cone penetration (DCP) tests suggested base course CBR values in the range of 15 to 30, but there is only a very crude correlation between DCP tests and CBR values. The DCP test results are included in the appendix. The same reasons that we anticipated that the subgrade during thaw would be less dense and wetter than when tested after trafficking apply to the base also. Consequently, the base course during thaw would also be quite weak. Past experience with these gradations suggest that when well-compacted they have a reliable design CBR value of no more than 20; the placement density and moisture content suggest CBR values of 10 or less were possible, and the DCP results when the base was drier and denser than when thawed gave results as low as 15. This base course, which must carry much of the load in a flexible pavement, is a weak material and provides a potential major failure zone.

**Table 2.4.3: Field properties of subgrade and base materials**

| | Pre-Trafficking | | | Post-Trafficking | | | |
|---|---|---|---|---|---|---|---|
| | Dry Density, lb/ft$^3$ | Percent Modified Lab Test | Moisture Content, % | Dry Density lb/ft$^3$ | Percent Modified Lab Test | Moisture Content, % | In-Situ Field CBR |
| **ML Silt Subgrade** | | | | | | | |
| mean | 103.6 | 89.7 | 15.3 | 110.8 | 95.9 | 16.5 | 6 |
| standard deviation | 4.08 | 3.53 | 0.77 | 4.82 | 4.82 | 1.72 | 3.0 |
| coef. of variation | 3.9% | 3.9% | 5.0% | 4.4% | 4.4% | 10.4% | 50.6% |
| number measured | 27 | 27 | 27 | 16 | 16 | 16 | 6 |
| max value | 111.0 | 96.1 | 16.6 | 117.4 | 101.6 | 19.2 | 12 |
| min value | 96.8 | 83.8 | 13.9 | 104.1 | 90.1 | 14.3 | 4 |
| **SM Base Course Material** | | | | | | | |
| mean | 125.3 | 93.2 | 5.7 | — | — | — | — |
| standard deviation | 1.38 | 1.02 | 0.41 | — | — | — | — |
| coef. of variation | 1.1% | 1.1% | 7.31% | — | — | — | — |
| number measured | 12 | 12 | 12 | — | — | — | — |
| max value | 127.8 | 95.0 | 6.4 | — | — | — | — |
| min value | 123.7 | 92.0 | 5.0 | — | — | — | — |

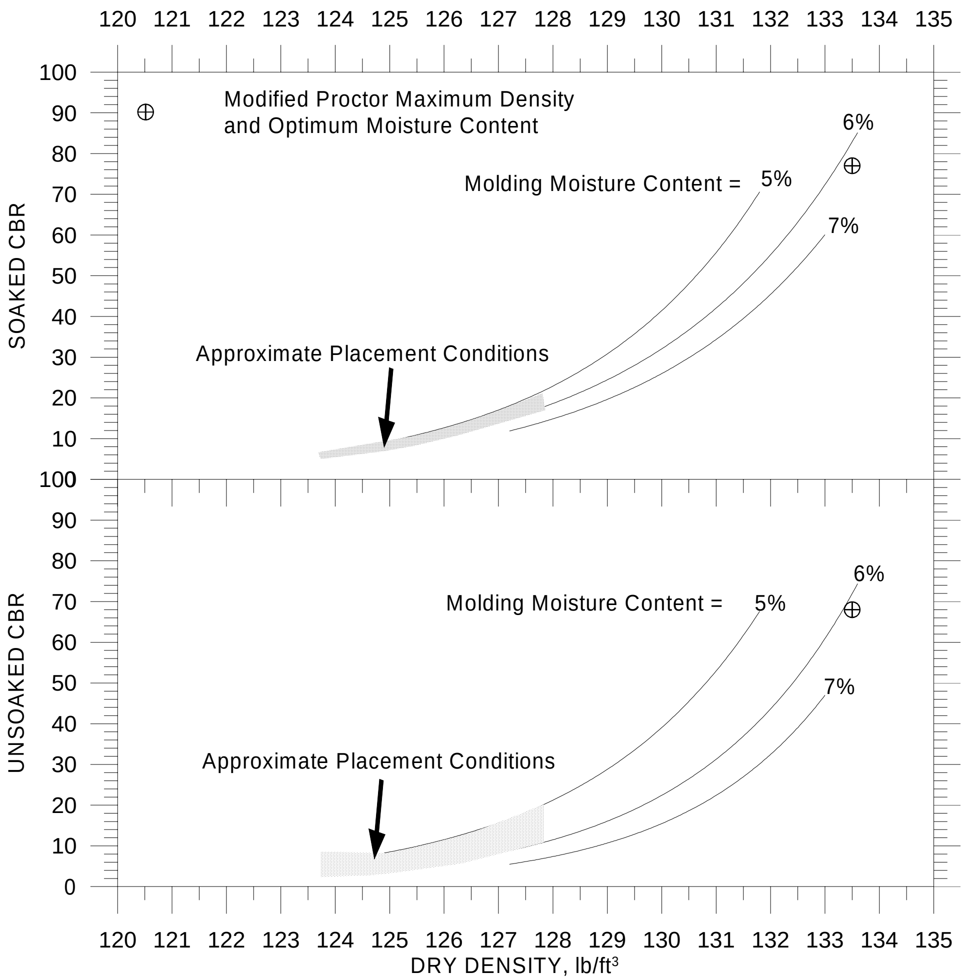


**Figure 2.4.4: Density-strength estimates for base course material.**

## *2.5 Instrumentation and Equipment*

Instrumentation was installed in the test section during its construction. Types of instrumentation included rut (or road) depth probes (RDPs) for measuring rutting in the subgrade, time domain reflectometry (TDR) probes for collecting moisture data, strain gages for measuring horizontal strain at the base of the asphalt layer, and thermisters and thermocouples for temperature measurements. Additional data were collected periodically throughout freezing–thawing and trafficking. Data collected in this manner included falling weight deflectometer (FWD), profilometer, and elevation surveys. Figure 2.5.1 (under revision) shows the locations of instrumentation and surface measurement locations in relation to the test item locations. Some portable FWD (PFWD) and Clegg hammer measurements were taken for possible correlation with the FWD, but that analysis is beyond the scope of this report.

## 2.5.1 Rut Depth Probe

The rut depth probe consists of a fluid-filled tube that is placed transversely on the surface of the subgrade prior to placement of the base course. This liquid level gage measures elevation differences with a pressure transducer that is passed through the fluid-filled tube. A schematic of the rut depth probe is shown in Figure 2.5.1.1. By taking readings at specified locations along the tube at different times during trafficking, any changes in elevation of the tube (indicating rutting of the subgrade soil) can be documented. Then, using these data in conjunction with profilometer data on the surface, one can determine where in the pavement structure deformations occurred as a result of the applied traffic.

The RDPs used in the VTP Phase III project consisted of four components: 1) the glycol reservoir, 2) 60-ft hydraulic hose filled with glycol, 3) the probe (pressure transducer), and 4) the "tattle tale" control box. To take readings, the probe was inserted into the glycol reservoir and then snaked through the hydraulic hose to obtain readings. Readings were usually taken at a stationary bench mark in the reservoir and then every 6 in. through the section of interest.

Eight RDPs were installed on the surface of the reconstructed subgrade in the VTP Phase III test section. At four locations, a pair of RDPs were placed transversely across the test section. Figure 2.3.2 shows the locations of all RDPs. Each pair of RDPs passed through three test items (trafficking areas). Before, during, and after trafficking of a particular test item, RDP readings were taken to document deformation caused by that trafficking. Intermittent readings were usually taken after 5, 10, 25, 50, 100, and 250 passes of the HVS wheel. Readings were usually taken only in the portions of the RDP tubes underlying the trafficked test item.

## 2.5.2 Time Domain Reflectometry Probes

Time domain reflectivity is an electromagnetic technique that can be used for determining in situ moisture content of soils. A probe, attached to a cable and a TDR instrument, is buried or inserted into the soil mass. When a signal is passed through the cable to the probe, a reflected signal is returned to the TDR instrument; from this returned signal, the propagation velocity can be calculated. As the propagation velocities of air, soil particles, and water are very different (1, 2 to 5, and 80 respectively), TDR probes can distinguish various combinations of these materials. Topp et al. (1980) were among the first to use TDR for soil moisture measurements. They introduced the concept of *apparent* dielectric constant, $K_a$, which is related to the propagation velocity of air, soil, and water. Because of the great difference in dielectric constant of water, the measured bulk $K_a$ is highly dependent on the water content of the soil mass.

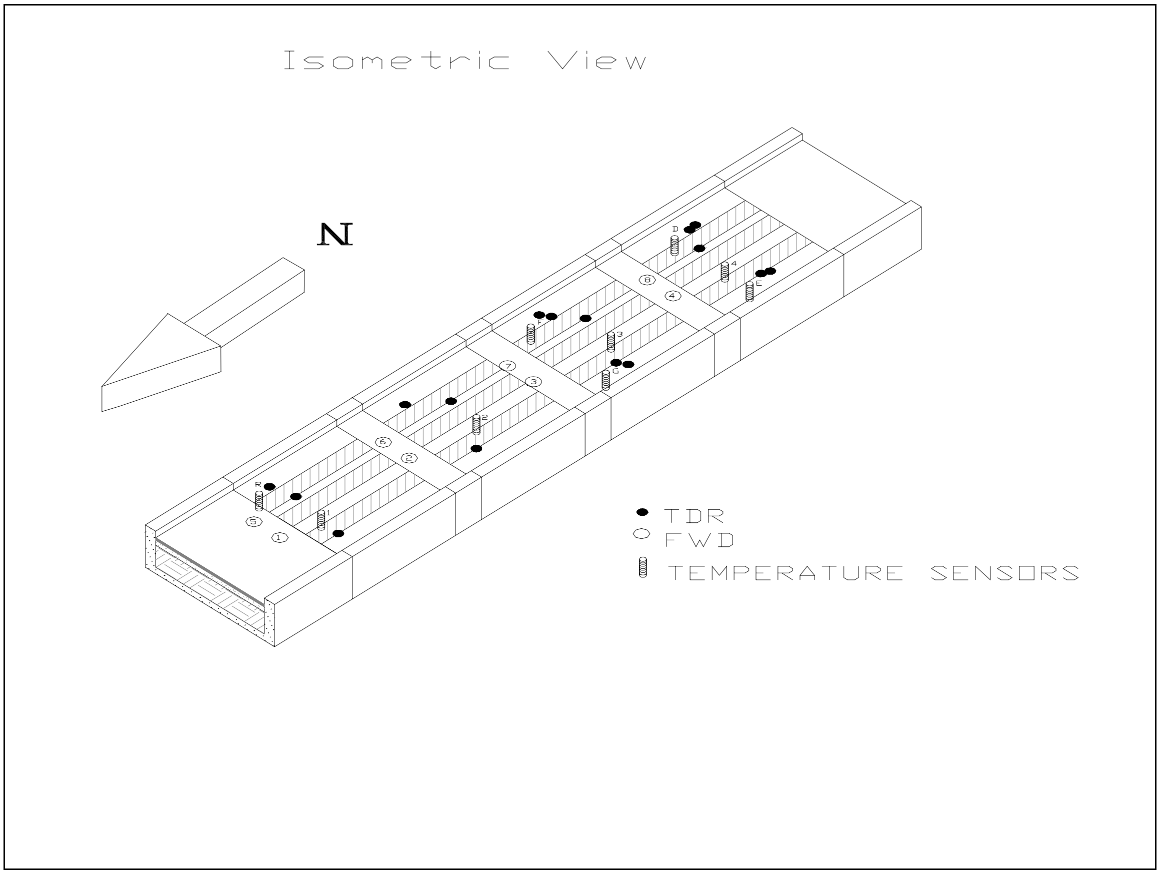


**Figure 2.5.1: 3-D Schematic of test section showing instrumentation and surface measurement locations.**

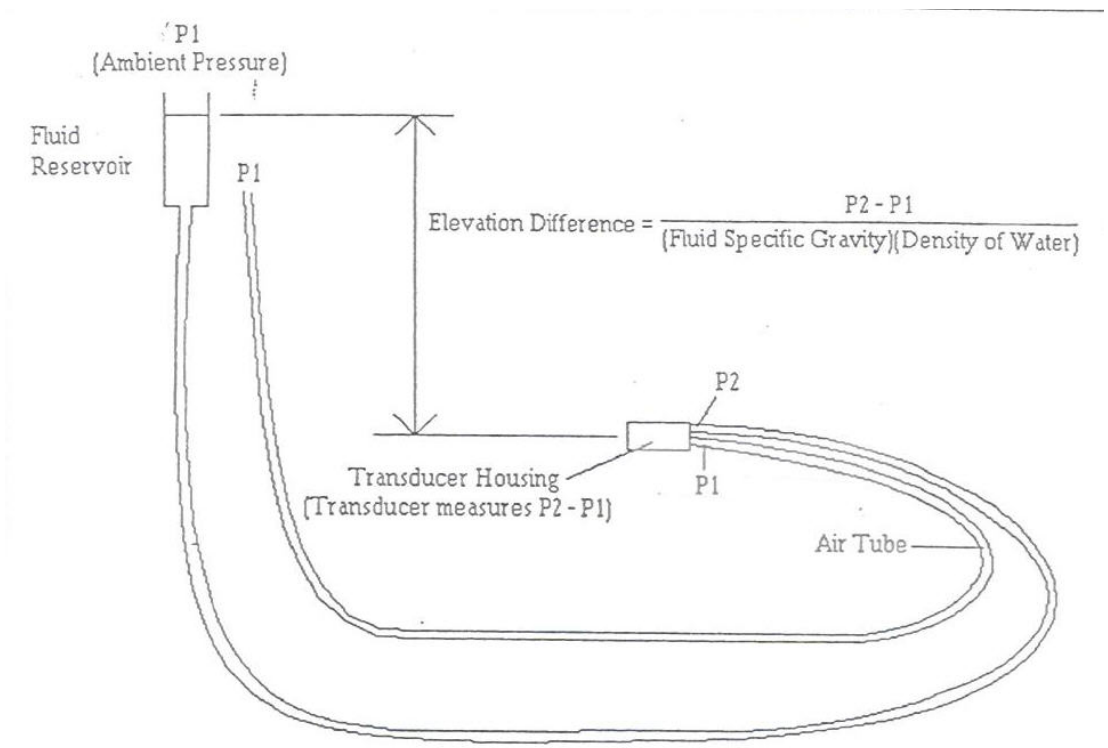


**Figure 2.5.1.1: Schematic of rut depth probe.**

For the VTP Phase III project, the Trace System I Model 6050 was used with a 6020 multiplexer to collect data from the soil moisture probes. These probes were Buriable Wave Guides, model 6005. The probes consist of three 1/8-in.-diameter stainless steel wave guide rods that are about 8 in. long; they are shown in Figure 2.5.2.1. They are designed to be installed permanently in the soil; measurements can be made at any depth in the soil profile. The moisture content measurement is made by taking the dielectric constant between the outer probes and the inner probe. This moisture content measurement is reported as the volumetric moisture content; for most geotechnical engineering work, this must be converted to gravimetric moisture content.

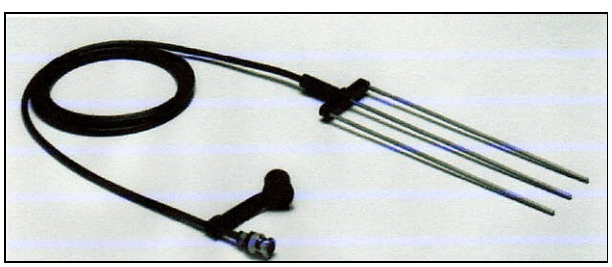

**Figure 2.5.2.1: TDR probe used in Phase III testing.**

## 2.5.3 Strain Gages

Standard SR-4 strain gages, Type FAE2-300-35-PEWL, were installed in the bottom surface of the hot mix asphalt layer. Two strain gages were placed in each test item. One was placed longitudinally and one transversely to measure horizontal strains in the bottom of the asphalt layer. Figure 2.5.3.1 shows a schematic diagram of the strain gages. Figure 2.5.3.2 shows strain gages being placed on the base course surface; prior to paving, the strain gages were covered with a thin layer of asphalt concrete to prevent damage during paving.

Many of the strain gages were over-ranged during initial trafficking of the test items. No useable data has been located for these strain gages.

**Type – FAE2-300-35-PEWL**
**P/N – 285970**
**Gage Factor – 2.10 ± 0.5%**
**Ohms – 350.0 ± 0.2%**
**S/N – 961507 – 1211 – AJK**
**L/N – 199 – 5 – CX**
**Gage Length – 76.20 mm**

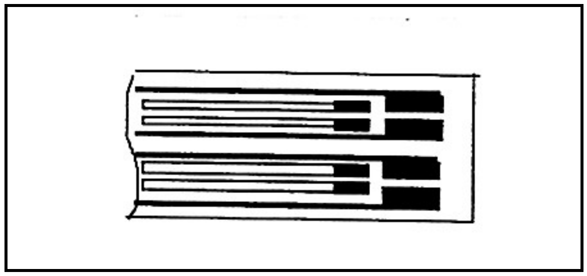

**Figure 2.5.3.1: Schematic diagram of strain gages.**

**Figure 2.5.3.2: Strain gage being installed prior to paving.**

## 2.5.4 Thermocouple–Thermister Probes

In the initial VTP test sections, thermister probes were used for all temperature measurements. Several strings of thermisters were installed in the original Phase II test sections; some were located in the lower subgrade soil, while other strings were located in the upper subgrade and the base course. During reconstruction for Phase III, several of the upper thermister strings were damaged during removal and were replace with thermocouple strings.

Also, because there were only two parallel trafficking windows in the Phase II project, while there were three parallel test items in Phase III, the replaced thermister–thermocouple strings are not located vertically above the strings in the lower subgrade. Positions of all thermister and thermocouple strings are shown in Figure 2.5.1.

## 2.5.5 Laser Profilometer

A profilometer measures changes in elevation at points along a cross-sectional line on a pavement. It is typically used to measure the depth of rutting that occurs due to traffic loads on a pavement. The CRREL profilometer is shown in Figure 2.5.5.1.

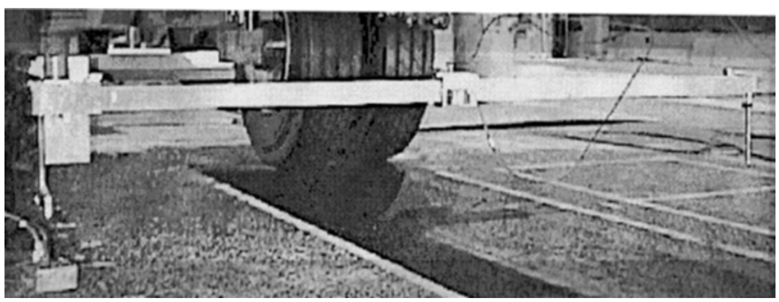

**Figure 2.5.5.1: CRREL Laser Profilometer.**

This laser profilometer was developed in South Africa by their Commonwealth Scientific and Industrial Research (CSIR) organization. It consists of a 10-ft-long beam with a stepper motor–cable system that moves a small carriage along the beam. The beam rests on three feet about 18 in. above the pavement surface. Mounted on the carriage is an infrared laser range finder. A computer program controls the speed and location of the carriage and collects 256 data points (at about ½-in. intervals) as it moves along the 10-ft beam. Measurements were made prior to trafficking, after specified numbers of passes, and after completion of trafficking. The initial measurements were used to normalize all other measurements.

## 2.5.6 Falling Weight Deflectometer

The falling weight deflectometer is a non-destructive test method used to obtain data from a layered pavement system to allow back-calculation of modulus of elasticity for each of the pavement layers. It consists of a specialty towed trailer with a set of seven geophones that can be lowered to the pavement surface at each testing location and a loading head that can apply a variety of loads to the pavement surface. The geophones are located at 0, 12, 24, 36, 48, 60, and 72 in. from the center of the 18-in. diameter loading plate. When a load is applied to the loading plate, a deflection basin is formed. A photograph of the CRREL falling weight deflectometer is shown in Figure 2.5.6.1.

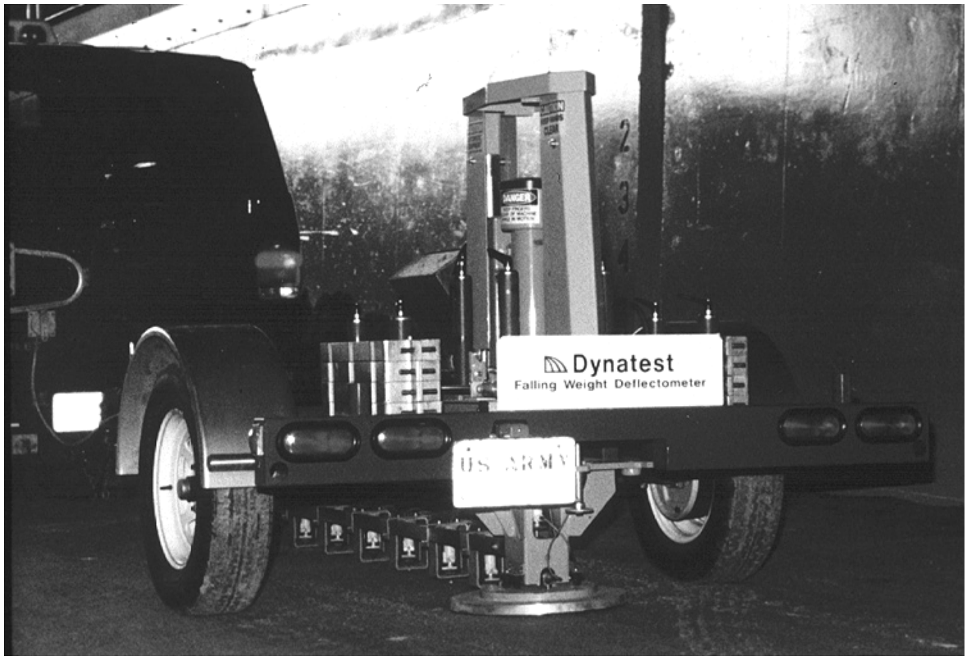


**Figure 2.5.6.1: CRREL Falling Weight Deflectometer.**

Deflection data used in back-calculation procedures must not exceed 80 mils of deflection and must exhibit a decreasing deflection from the center of the basin to the outermost sensor. Few of the numerous FWD data sets collected in the Phase III project met both of these criteria. Impulse stiffness modulus values were calculated for each useable deflection basin. The evaluation module of PCASE was used for the back-calculation. The pavement layer thicknesses used for the back calculation procedures were the average values reported in the construction field book. The total thickness of the pavement structure used was 7 ft, with 2.4 in. of asphalt concrete, 7.4 in. of base, and 75 in. of subgrade soil.

# Chapter 3
# Testing and Results

The VTP Phase III Test Section was constructed during the summer 2000 by reconstructing the upper portion of a test section originally built in 1996. This new test section was to examine the performance of a relatively thin pavement with a substandard base course after it was frozen and allowed to thaw to various depths from the surface. Performance was to be evaluated by response of various test items to traffic from a 9000-lb single tire inflated at 35, 65, and 100 psi. Figure 3.1 and Table 3.1 show the layout of the individual test items in the test section and the specific test item objectives in terms of thaw condition and tire pressure loading. Test items 2 and 5 were used twice for trafficking. Negligible deformation occurred during trafficking while frozen, and they were reused for trafficking when the thaw had penetrated approximately 2 ft into the subgrade.

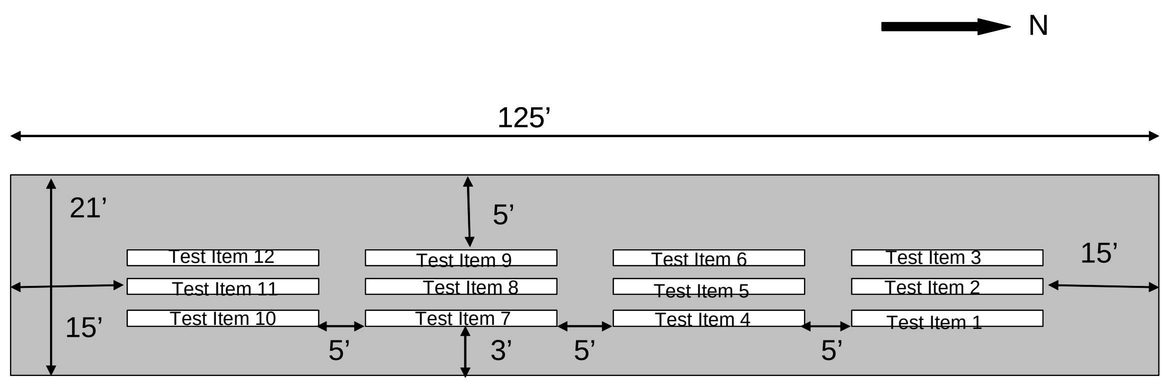


**Figure 3.1: Designation of test items in VTP Phase III test section.**

Freeze panels were installed on 9 August 2000. Freezing began on 16 August and was discontinued on 20 November when the total depth of freezing to 5 ft was met or exceeded on all instrumentation. The test section was allowed to thaw, and trafficking tests were conducted with the HVS when instrumentation suggested the desired nominal thaw conditions were approximated by the test section condition. Falling Weight Deflectometer (FWD) data and elevations were collected periodically between HVS tests. Surface profiles, rut depth probe, and elevation data were also collected periodically during HVS tests. Temperature and moisture data were collected continuously during the test period. Collected data are recorded in the Appendices and will be analyzed in Chapter 4. The first HVS test was conducted on 20 November 2000 and the final one was conducted on 11 February 2001. During March and April, field tests were conducted on the in-situ test section materials, the site was excavated, and instrumentation was recovered. Samples of the test section materials were then tested in the laboratory in May through July 2001. Table 3.2 shows a detailed chronology of testing of the VTP Phase III Test Section.

**Table 3.1: Test conditions for each test item in the VTP Phase III test section**

| Test Item | Test Condition | Tire Pressure, psi | Date of HVS Tests |
|---|---|---|---|
| 1 | Thaw 1-ft into subgrade | 100 | 17 Dec. 2000 |
| 2 | Frozen | 100 | 20 Nov. 2000 |
| | Thaw 2-ft into subgrade | 100 | 12 Jan. 2001 |
| 3 | Thaw through base | 100 | 7 Dec. 2000 |
| 4 | Thaw 1-ft into subgrade | 65 | 17 Dec. 2000 |
| 5 | Frozen | 65 | 22 Nov. 2000 |
| | Thaw 2-ft into subgrade | 35 | 12 Jan. 2001 |
| 6 | Thaw through base | 65 | 6 Dec. 2000 |
| 7 | Thaw 1-ft into subgrade | 35 | 14 Dec. 2000 |
| 8 | Frozen | 35 | 21 Nov. 2000 |
| 9 | Thaw through base | 35 | 5 Dec. 2000 |
| 10 | Recovered | 35 psi | 10 Feb. 2001 |
| 11 | Recovered | 65 | 11 Feb. 2001 |
| 12 | Recovered | 100 | 11 feb. 2001 |

Five potential thawing conditions were evaluated during this test. Nominally, these were:

1. Test items were trafficked while fully frozen to a depth of 5 ft. Test items 2, 5, and 8 used differing tire pressures to evaluate this condition. Negligible damage occurred under these conditions.
2. Test items were trafficked when shallow thaw had penetrated through asphalt and base layer, nominally 10 in. from the surface. Test items 3, 6, and 9 used varied tire pressure on the HVS to evaluate this condition. Failure was rapid and showed major shearing within the pavement material (see Figure 3.2).
3. Test items were trafficked when thaw had penetrated approximately 1 ft into subgrade. Test items 1, 4, and 7 used varied tire pressure to evaluate this

condition. Failure was rapid and showed major shearing within the pavement materials (see Figure 3.2.

4. Test items were trafficked when thaw had penetrated approximately 2 ft into subgrade. Test items 2 and 5 were reused for this test and used 100 and 35 psi tire pressures. Failure was rapid and showed major shearing within the pavement materials.
5. Test items were trafficked several weeks after all of the test section had thawed. Test items 10, 11, and 12 used varied tire pressure to accomplish this objective. Performance was better than with the thawing test items, but failure was still rapid and demonstrated major shear deformations.

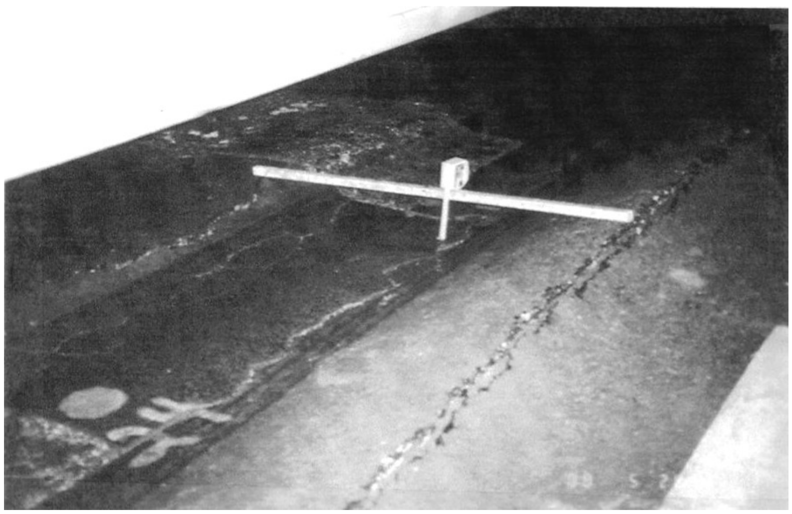

**Figure 3.2: Example of massive shear failure in VTP Phase III test items.**

**Table 3.2: VTP Phase III test section testing chronology**

| **Date** | **Julian Date for Data Records** | **Nominal Freezing Regime and Activities** | **HVS Traffic Tests** | **Measurements** | | | |
|---|---|---|---|---|---|---|---|
| | | | | Profil-ometer | RDP | Elev | FWD |
| 2000 | | | | | | | |
| 9 Aug | 222 | panels installed | | | | | |
| 16 Aug | 229 | freezing started | | | | | |
| 15 Nov | 320 | freezing ends,<br>5 ft depth of freeze | | | | | |
| 16 Nov | 321 | panels removed | | | | | |
| 17 Nov | 322 | | | X | | X | X |

| Date | Julian Date for Data Records | Nominal Freezing Regime and Activities | HVS Traffic Tests | Measurements | | | |
|---|---|---|---|---|---|---|---|
| | | | | Profil-ometer | RDP | Elev | FWD |
| 18 Nov | 323 | | HVS moved on section | | | | |
| 20 Nov | 325 | Fully Frozen | Test Item 2 | X | X | X | |
| 21 Nov | 326 | | Test Item 8 | X | X | X | |
| 22 Nov | 327 | | Test Item 5 | X | X | X | |
| 23 Nov | 328 | | HVS moved off section | | | | |
| 25 Nov | 330 | | | | | | X |
| 1 Dec | 336 | | | | | | X |
| 4 Dec | 339 | | | | | | X |
| 5 Dec | 340 | Thawed base | HVS moved on, Test Item 9 | X | X | X | |
| 6 Dec | 341 | Thawed base | Test Item 6 | X | X | X | |
| 7 Dec | 342 | Thawed base | Test Item 3,<br>HVS moved off | X | X | X | |
| 13 Dec | 349 | Thaw 1-ft into subgrade | HVS moved on section | | | | |
| 2000 | | | | | | | |
| 14 Dec | 349 | Thaw 1-ft into subgrade | Test Item 7 | X | X | | |
| 17 Dec | 352 | Thaw 1-ft into subgrade | Test Item 1<br>Test Item 4 | X | X | | |
| 18 Dec | 353 | | | | | | X |
| 19 Dec | 354 | | | | | X | |
| 22 Dec | 357 | | | | | | X |
| 27 Dec | 362 | | | | | | X |
| 29 Dec | 364 | | | | | | X |

| Date | Julian Date for Data Records | Nominal Freezing Regime and Activities | HVS Traffic Tests | Measurements | | | |
|---|---|---|---|---|---|---|---|
| | | | | Profil-ometer | RDP | Elev | FWD |
| 2001 | | | | | | | |
| 2 Jan | 368 | | | | | | X |
| 5 Jan | 371 | | | | | | X |
| 9 Jan | 375 | | | | | X | X |
| 12 Jan | 378 | Thaw 2-ft into subgrade | Test Item 5 and Test Item 6 | X | X | | |
| 13 Jan | 379 | | | | | X | X |
| 19 Jan | 385 | | | | | | X |
| 24 Jan | 390 | | | | | | X |
| 31 Jan | 397 | | | | | | X |
| 1 Feb | 398 | | | | | X | |
| 7 Feb | 404 | | | | | X | X |
| 8 Feb | 405 | | | | | X | |
| 2001 | | | | | | | |
| 10 Feb | 407 | Fully Thawed and Recovered | Test Item 10 | X | X | | |
| 11 Feb | 408 | Fully Thawed and Recovered | Test Item 12 and Test Item 11 | X | X | X | |
| 15 Feb | 412 | | | | | | X |
| 12 Mar | 437 | | | | | | X |
| 27 Mar -1 May | 452 - 486 | Post trafficking field tests, sampling, excavation, and instrumentation recovery | | | | | |
| 1 May - 12 July | 486 - 557 | Laboratory testing of materials used in test section | | | | | |

Notes: RDP is rut depth probe, elevations are rod and level survey, FWD is falling weight deflectometer.

Table 3.3 compares the nominal test conditions with the best estimate of actual test conditions. In general, desired test conditions were fairly closely replicated with differences as noted in Table 3.3. The differences are sufficiently small that analysis in the following section will be based on the nominal thaw conditions. It is impractical to model small divergences with fairly crude theoretical models. For instance, there appears to be perhaps 2 in. of the thawed subgrade included with the thawed base course. It is doubtful that layered elastic theory could adequately model this thin soft layer. In addition there are variations in base and asphalt thickness and thaw rates that further complicate the picture. Given these issues, the nominal thaw conditions will be used for analysis in Chapter 4.

**Table 3.3: Comparison of Nominal Test Conditions and Actual Conditions**

| Nominal Test Condition | Actual Test Conditions | Impact |
|---|---|---|
| Frozen 60 in. | Instrumentation shows frozen to slightly more than 60 in. | Negligible |
| Trafficked Frozen | From 0 to 5 in. of surface thawed during traffic | Slightly less stiff when thawed. Negligible effect. |
| Trafficked when AC and Base Course thawed (10-in. depth) | Actual thaw depths were right at 12 in. and were 15 in. on item 6 | Several in. thawed subgrade included with the thawed base. Item 6 had 5 in. thawed material which would make it appear to perform worse than expected. |
| Thaw 1-Ft into Subgrade (22-in. depth) | Testing began with about 19 in. depth of thaw with no more than 23 in. depth thawed | Close to nominal test condition. Differences should be relatively minor |
| Thaw 2-Ft into Subgrade (32-in. depth) | Instrumentation problems make this less clear. TDR data suggests thawing at 2.5 ft occurred 16 to 25 days prior to these test items being tested | Likely thaw deeper than nominal conditions. Thawed soils so soft that increased depth of soft material had relatively minor effect. |
| Recovered , item fully thawed | Instrumentation shows fully thawed by time tested | No impact |

# CHAPTER 4
# ANALYSIS

Analysis of the test section will include theoretical and empirical assessments of the test results. The theoretical analysis will be based on continuum mechanics, although the actual materials are discrete particles. The classic pavement analysis assumptions of linear elasticity, homogeneity, and isotropy will be invoked. The assumption of linear elasticity allows linear superposition to be used, which allows the combined effect of each of multiple loads to be calculated as the sum of the individual tire effects at any given point in space. The actual pavement materials show distinct viscoelasticity (asphalt concrete) and nonlinear stress dependancy and significant plastic deformations (soils and aggregates). Despite these limitations, linear elastic theory remains the most widely used theoretical tool for analysis of flexible pavement structures. Design methodologies are increasingly based on linear elastic theoretical models while researchers continue to study more powerful numerical models to better capture the behavior of the complex materials in pavements.

Two specific linearly elastic models will be used in this analysis. The first is Boussinesq theory, which will be used to examine tire pressure effects under a single tire on the surface of an elastic half-space. This analysis neglects the different stiffness values of each layer in the pavement but allows a simple picture of the interaction of tire pressure and depth below the surface on theoretical stress levels in the pavement. Most of the remaining analysis will be based on more comprehensive layered elastic theory. This allows the representation of each layer in the pavement with a modulus of elasticity and a Poisson's ratio. Shell's BISAR computer program will be used for the layered elastic calculations (De Jong et al. 1973). It has been the analysis tool for much of the Corps of Engineers' work developing layered elastic theory for pavements (e.g., Barker and Brabston 1975, Parker et al. 1979, Rollings 1987) Other computer programs are available to do the same analysis (e.g., JULEA, WESLEA, CHEVRON, ELSYM5). In the formulation of the layered elastic problem, one encounters a double integral of a Bessel function that most be solved numerically rather than in a closed form. Most of these layered elastic programs differ in how the double integral is approximated and in how the interface between layers is modeled.

All tire loads are assumed to be circular with uniform constant stress levels acting normal to the pavement surface. Actual tire contact areas tend to be elliptical, and contact stress varies, depending on tire characteristics. Saint-Venant's Principle states that the exact intensity and distribution of load on the surface of a body has progressively less influence on the calculated stress as one moves further from the point of application of the load as long as the total load is correct (i.e., both the resultant and moment must be the same). In pavements, we often tend to examine load effects at some depth in the pavement. Saint-Venant's Principle provides the rationale for the circular area and constant load approximation commonly used in pavement analysis. This assumption is most accurate for subgrade analysis and least correct for analyzing load effects within thin asphalt concrete surface layers.

Furthermore, we will accept the widely used approximation that the wheel load ($P$), tire pressure ($t_p$), and circular tire contact area ($A$) are interrelated by the expression:

$$P = t_p A$$

## 4.1 Theoretical Assessment of Variable Tire Pressure—Boussinesq Theory

All other things being equal, as stress increases at a critical point in a pavement, the fewer the load repetitions that can be sustained before failure. This leads to the premise that the distribution of stress within the pavement is an indicator of how severely the pavement is being loaded and should indicate expected pavement life if proper criteria are available for the pavement materials. Hence, if lowering the tire pressure reduces stresses within the pavement structure, particularly in the weakest areas of the pavement, one would presume pavement damage would be reduced.

The distribution of vertical stress under the center of a circular load applied on a flexible plate on an elastic half-space may be calculated based on Boussinesq theory by the equation (Van Cauwelaert 2003, Huang 1993):

$$\sigma_z = q\left(1 - \frac{z^3}{\left(a^2 + z^2\right)^{1.5}}\right)$$

where
$\sigma_z$ = vertical stress
$q$ = applied pressure (tire pressure)
$z$ = depth below surface
$a$ = radius of loaded area.

Figure 4.1.1 shows the distribution of stress calculated with the above equation for the VTP Phase III test section loading conditions (9000-lb wheel load, 35-, 65-, and 100-psi tire pressures). The nominal cross section of the VTP Phase III Test Section is shown on the figure for reference, but all calculations are made for an elastic half-space without differentiating between the stiffness of the different materials. Inclusion of the effect of layers of varying stiffness will be addressed in the following section of this chapter.

While initial stress levels at the surface are quite different, they close asymptotically with depth. Table 4.1.1 shows that although 65 psi separate the calculated stresses at the surface, by the bottom of the base course the difference between calculated stresses for the different tire pressures has dropped to 10.7 psi. One foot into the subgrade the difference is only 1 psi and only 0.3 psi at 2 ft into the subgrade. Figure 4.1.2 shows another way of viewing the asymptotic relation between stresses. In this figure, the calculated stress at each depth under the 100-psi tire is taken as the base reference. With depth, the difference between the stresses for different tire pressures steadily decreases.

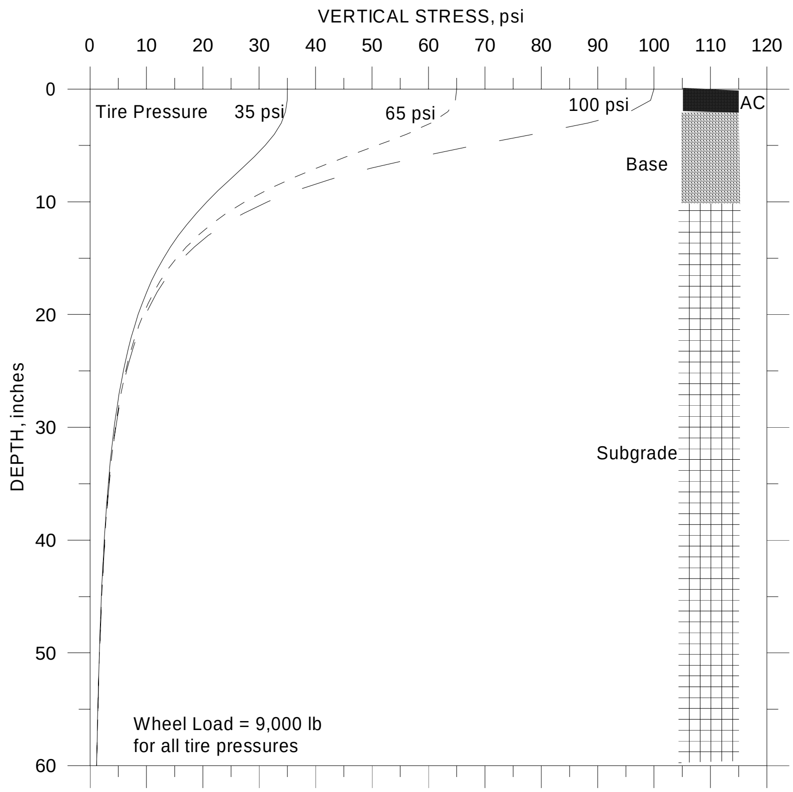


**Figure 4.1.1: Vertical stress distribution under the VTP Phase III loading conditions using Boussinesq elastic theory.**

**Table 4.1.1: Calculated Boussinesq stresses for VTP loadings**

| **Nominal Depth Below Surface, in.** | **Location of Stress Calculation** | **Calculated Stress for Varying Tire Pressure (psi)** | | |
|---|---|---|---|---|
| | | 35 | 65 | 100 |
| 0 | surface | 35.0 | 65.0 | 100.0 |
| 2 | top of base | 34.6 | 63.4 | 95.7 |
| 6 | midpoint of base | 29.1 | 45.4 | 58.4 |
| 10 | bottom of base/top of subgrade | 20.7 | 27.4 | 31.4 |
| 22 | 1-ft into subgrade | 7.3 | 8.0 | 8.3 |
| 34 | 2-ft into subgrade | 3.8 | 4.0 | 4.1 |

Boussinesq theory, while simple, illustrates a fundamental fact. As tire pressure increases, stresses in the upper portion of the pavement increase dramatically, but stresses at depth are little affected by such tire pressure increases. This is a widely recognized fact in pavement engineering and is discussed in standard texts (e.g., Yoder and Witczak 1975). In Figure 4.1.1 and Table 4.1.1, one observes that over two-thirds of the difference in stress levels between the 35- and 100-psi tires is accounted for within the asphalt concrete and base course layers. Hence, Boussinesq theory suggests that lowering tire pressure will primarily affect the quality of asphalt concrete and base-course aggregate one needs in the pavement structure.

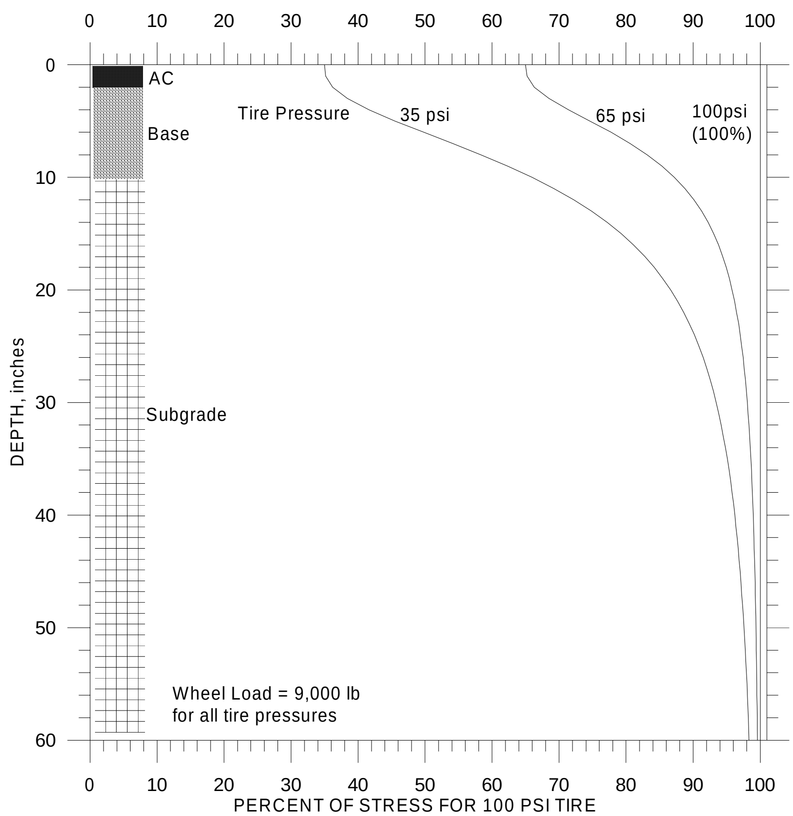


**Figure 4.1.2: Asymptotic closure of Boussinesq calculated stresses with 100-psi tire pressure stresses taken as the basis for normalization of data.**

## *4.2 Theoretical Assessment of Variable Tire Pressure—Layered Theory*

Burmeister (1943, 1945) developed the solution to two- and three-layered elastic systems subject to uniform vertical loads. Computerization made it practical to extend Burmeister's layered elastic theory to solutions for a system with multiple layers (e.g., Schiffman 1967, Huang 1967, De Jong et al. 1973, Barker and Brabston 1975). A full discussion of the solution techniques for the layered elastic pavement problem is provided by Van Cauwelaert (2003).

In the layered elastic problem, each layer is characterized by a thickness, modulus of elasticity, and Poisson's ratio. The last layer in the system is semi-infinite. Depending on the problem formulation and assumptions used in the computer program, the interface condition between layers may be fixed, free, or some intermediate slip condition. For the VTP Phase III Test Section analysis, the interface between layers is treated as fixed, as is common for typical flexible pavement analyses. Loads are all circular, uniform, and vertical. Some programs allow horizontal loads as well as vertical but these are not used in most pavement analysis. Multiple load effects are accommodated by linear superposition.

Flexible pavement design traditionally centers on protecting the subgrade by providing sufficient pavement thickness above the subgrade to reduce the load-induced stresses on the subgrade to prevent rutting. This was the original concept in the first structurally oriented flexible pavement design system: the Corps of Engineers CBR method, inspired by World War II (Ahlvin 1991). This criterion of protecting the subgrade from rutting was incorporated in layered elastic design concepts by developing limits on allowable vertical subgrade strain calculated using layered elastic theory. A number of different criteria have been used for allowable subgrade strain. This analysis will use the Corps of Engineers criterion (Barker and Brabston 1975), which is expressed:

$$N = 10,000\left(\frac{A}{\varepsilon_{\mathrm{v}}}\right)^{B}$$

where
$N$ = allowable repetitions
$A$ = 0.000247 + 0.000245 Log ($E_{\mathrm{s}}$)
$\varepsilon_{\mathrm{v}}$ = calculated vertical subgrade strain
$B = 0.0658E_{\mathrm{s}}^{0.559}$
$E_{\mathrm{s}}$ = subgrade modulus, psi.

This criterion is similar to most other published criteria but differs from these others in one important factor. It includes the modulus of elasticity of the subgrade within one term. Most other criteria present allowable repetitions of load as a function of calculated subgrade vertical strain alone. This Corps of Engineers criterion was back-calculated from traditional CBR designs, which then makes allowable repetitions of load and a function of subgrade strength or stiffness (Barker and Brabston 1975) a factor in the calculation. The CBR method is particularly effective for relatively large wheel loads, relatively thin asphalt surfaces, granular base and subbase layers, and relatively weak, fine-grained subgrades. The VTP Phase III Test Section fits this well. Consequently, the Corps of Engineers subgrade strain criterion was selected as particularly appropriate for this specific test problem.

Layered elastic theory allowed the pavement engineer to add analysis of fatigue cracking in the asphalt layer to the design. Like allowable subgrade vertical strain criteria, there are a variety of published asphalt fatigue criteria to use with layered elastic theory (e.g., Barker and Brabston 1975, Yoder and Witczak 1975, Huang 1993). The following relation is the one proposed by Barker and Brabston (1975) and is currently used by the Corps of Engineers for assessment of fatigue cracking in asphalt layers using layered elastic theory.

$$N = 10^{\left(2.68 - 5\log(\varepsilon_a) - 2.665\log(E)\right)}$$

where
$\varepsilon_a$ = computed asphalt tensile strain
$E$ = modulus of asphalt

Fatigue cracking of asphalt concrete has been the topic of intensive research and debate in the pavements community in the last decade. It is an evolving area that is advancing rapidly. However, asphalt fatigue is not normally an issue with relatively thin asphalt layers subject to relatively few load repetitions as is the test condition in the VTP Phase III Test Section. Indeed, all failures observed in the test section were attributable to rutting and not fatigue cracking. Consequently, while it is possible to make calculations to check asphalt fatigue, the point is moot, as this is not the observed distress mode in the actual test section.

Layered elastic pavement design is based on selecting material properties and thicknesses that meet the load and repetitions requirements without failing the subgrade strain and asphalt fatigue cracking criteria. There are no checks on the granular base (or subbase if included). Despite decades of research, no satisfactory analysis method exists for determining suitable criteria for these granular layers between the subgrade and asphalt surface. Inherent in the design process is the requirement that the intermediate granular layers be sufficiently strong to prevent rutting from shear and sufficiently well compacted to prevent rutting from densification. As was seen in Figure 4.1.1, these layers can be subject to significant stresses, and these stresses in the base increase rapidly as tire pressure increases. The requirements for granular base and subbase materials are usually based on agency experience, empirical trials, or a combination of these. Typical base course requirements include specifications on allowable gradation, Atterberg limits, particle shape, and durability (e.g., sulfate soundness or LA abrasion), and differ significantly among organizations, reflecting their experiences, climate, local materials, and load conditions (Rollings and Rollings 1996). Placing granular layers of inadequate strength in a pavement will cause a failure, even if the design calculations for subgrade and asphalt strain show the system to be adequate. This is a very important factor: classical pavement design calculations do not address all failure modes, and these may require separate requirements and analysis.

Layered elastic theory will next be used to analyze the vertical stress distribution in the VTP Phase III test section, accounting for the effect of stiffness in the various layers. Table 4.2.1 shows the parameters used in this analysis. The asphalt was assigned an intermediate quality modulus of 500,000 psi under cool weather conditions. Depending on their specific gradations, plasticity characteristics, density, and moisture content, ML silts will often have design CBR ranges of 5 to 15, which would correspond to modulus values on the order of 7500 to 22,500 psi (Rollings and Rollings 1996). For this analysis, the subgrade was assigned a modulus of 8000 psi. In the earlier Boussinesq analysis, it was the base that carried the bulk of the increase in stress as tire pressure increased. Now, the impact of the base course stiffness on stress distribution will be examined.

In a layered pavement analysis, the stiffness that one layer can develop depends to some extent on the stiffness of the layer beneath it. In addition, granular materials within the structure are highly nonlinear and stress dependent—i.e., the material stiffness (modulus) and deformation characteristics vary, depending on what stress the granular material is under and the relation is complex. Hence. some simplifying assumptions must be used to convert a complex real world pavement structure into an analytically tractable form. For this analysis, we will use the procedure proposed by Barker and Brabston (1975) and currently adopted by the Corps of Engineers and U.S. Air Force for pavement layered elastic design calculations. This states that the stiffness of a granular layer over another layer is a function of the stiffness of the underlying layer and the quality of the granular layer. Thick layers may be subdivided into multiple layers of 4 to 8 in. and the stiffness of each layer may be iteratively calculated as a function of the underlying layer. The functions for the stiffness of layer *n* based upon the stiffness of underlying layer *n+1* are (Barker and Brabston 1975) as follows. For subbase-quality materials (design CBR 30 to 50, typically natural sandy and gravelly materials with plasticity index less than 5):

$$E_{\mathrm{n}} = \left(E_{\mathrm{n+1}}\right)\left(1 + 7.18\log(t) - 1.56\log\left(E_{\mathrm{n+1}}\right)\log(t)\right)$$

For base-course-quality materials (design CBR 80 to 100, typically well-graded crushed stone with low fines and plasticity index less than 5):

$$E_{n=} \left(E_{\mathrm{n+1}}\right)\left(1 + 10.52\log(t) - 2.10\log\left(E_{\mathrm{n+1}}\right)\log(t)\right)$$

where
$E_{\mathrm{n}}$ = modulus of elasticity of layer *n*
$E_{\mathrm{n+}1}$ = modulus of elasticity of below layer *n*
$t$ = thickness of layer *n* (moduli in psi, *t* in inches).

**Table 4.2.1: Parameters used for assessing base stiffness effects with layered elastic theory**

| **Material Properties** | | |
|---|---|---|
| **Material** | **Modulus, psi** | **Poisson's Ratio** |
| Asphalt Concrete | 500,000 | 0.35 |
| Base Course | Varies | 0.35 |
| Subgrade | 8000 | 0.4 |
| **Loading Conditions** | | |
| **Tire Pressure, psi** | **Contact Area, in.$^2$** | **Load Radius, in.** |
| 35 | 257 | 9.05 |
| 65 | 138 | 6.64 |
| 100 | 90 | 5.35 |

The base course used in the VTP Phase III test section was a low-quality material that would fail to meet the requirements for a Corps of Engineers 30-, 40-, or 50-design CBR subbase, but would be assigned a select fill designation with a design CBR of 20 (Table 4.2.2). By use of the relation for estimating modulus above, the VTP Phase III test section base course on a subgrade modulus of 8000 psi would have an estimated modulus of 15,883 psi if is treated as a subbase-quality material and 24,787 psi if treated as a base-course-quality material. As seen in Table 4.2.1, the VTP Phase III base material really only qualifies as select fill. Hence, an estimated modulus on the order of 16,000 psi is probably representative and possibly optimistic for the material. Figure 4.2.1 shows a stress distribution calculated for a 9000-lb wheel load at 35- and 100-psi tire pressures using a relatively optimistic 16,000-psi modulus value for the base, 8000 psi for the subgrade, and 500,000 psi for the asphalt concrete. The results are consistent with those of the Boussinesq analysis: the major difference between stress distribution due to tire pressure is in the upper base and asphalt layers. However, now the relative stiffness of each layer is included in the analysis. Figures 4.2.2 and 4.2.3 show that if the base course stiffness is progressively increased from the essentially Boussinesq condition ($E_{base}/E_{subgrade}$ = 1.0 except the effect of the thin stiffer asphalt layer is included in the figures) to a ratio of the base being five times greater than the subgrade, the stresses in the upper base and asphalt layers increase and the stresses decrease in the upper subgrade. Essentially, as the quality and stiffness of the base are increased, the proportion of load carried in these upper materials also increases. As the upper layers become more stiff, they will be subjected to more stress and their quality must be adequate for this role.

**Table 4.2.2: Comparison of VTP Phase III base course and standard subbase requirements**

| Material | Maximum Design CBR | Maximum Permissible Values | | | | |
|---|---|---|---|---|---|---|
| | | Max. Particle Size | Percent Passing, % | | Liquid Limit | Plasticity Index |
| | | | No. 10 | No. 200 | | |
| Subbase | 50 | 3-in. | 50 | 15 | 25 | 5 |
| Subbase | 40 | 3-in. | 80 | 15 | 25 | 5 |
| Subbase | 30 | 3-in. | 100 | 15 | 25 | 5 |
| Select Fill | 20 | 3-in. | — | 25 | 35 | 12 |
| Actual VTP Base Course | 20 | 3-in. | 69.4 | 20.7 | Nonplastic | |

Note: Corps of Engineers work found that laboratory compacted samples gave artificially high CBR values because of the confining effect of the mold. The above criteria were found to give a more accurate assessment of the performance of granular materials in pavement subbases in accelerated traffic tests and in the field. They were adopted as default design values for materials meeting all of the criteria and have been in use now for almost 50 years (Ahlvin 1991).

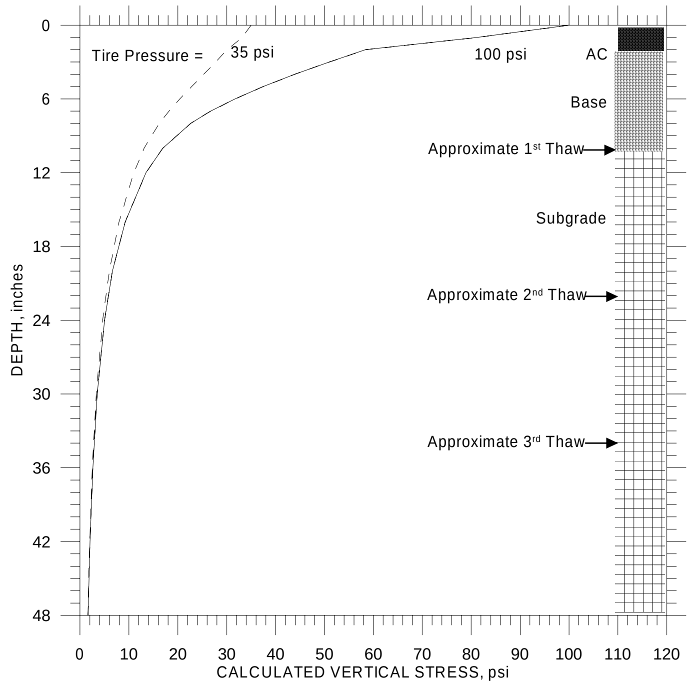


**Figure 4.2.1: Effect of tire pressure on stress distribution in a layered elastic analysis of the VTP Phase III test section.**

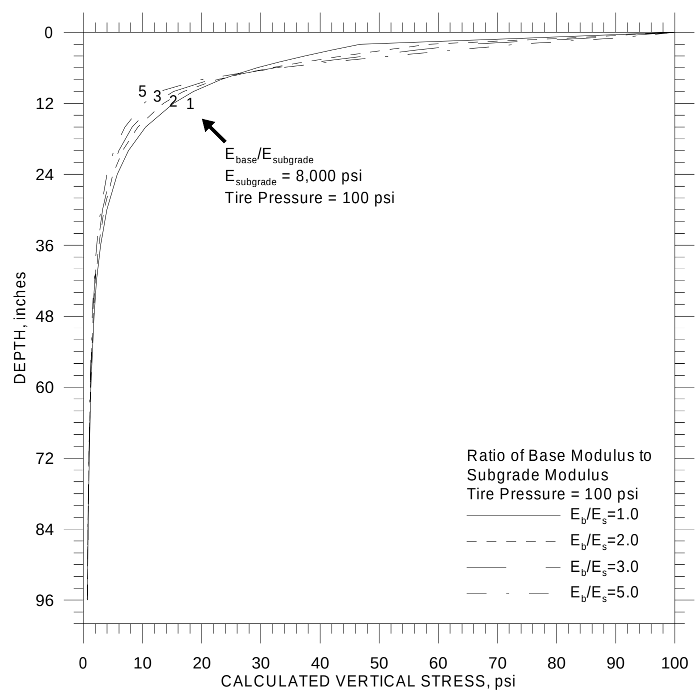


**Figure 4.2.2: Effect of increasing base stiffness for 100 psi tire on VTP Phase III test section.**

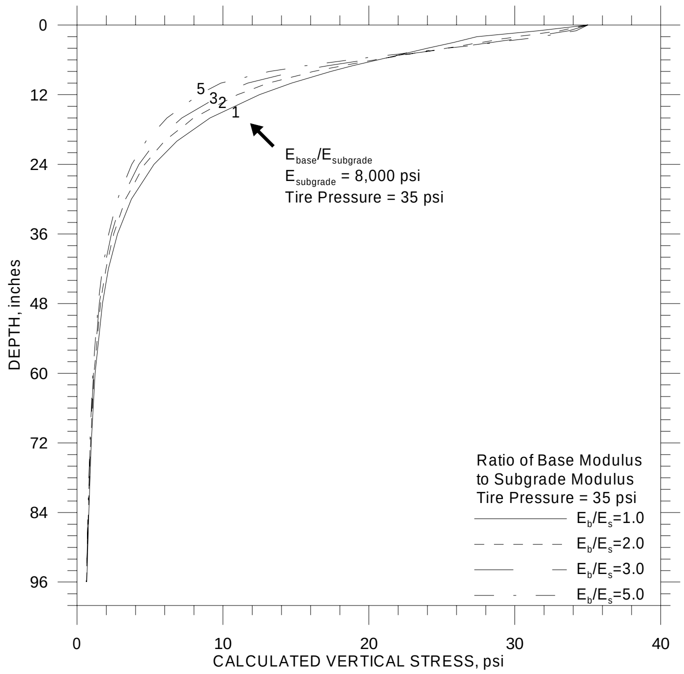


**Figure 4.2.3: Effect of increasing base stiffness for 35 psi tire on VTP Phase III test section.**

Previous investigations of the effects of variable tire pressures on low-volume pavements have indicated that as the asphalt layer thickness increases, the beneficial effects of lowered tire pressure may be decreased (Kestler and Berg 1996, Kestler et al. 1997). In Figure 4.2.4, the asphalt thickness is increased from 2 to 4 to 6 in. while keeping the combined base and asphalt thickness a constant 10 in. As the thicker, stiffer layer is added to the VTP Phase III Test section, an increase in stress in the upper layer and a decrease in stress in the subgrade regions was observed. This lends support to the observation by Kestler and Berg (1996) and Kestler et al. (1997) that the potential benefit of reduced tire pressure on reducing damage to thaw-weakened subgrades decreases as the upper asphalt layer increases in thickness.

Finally, the impact of reduced tire pressure on the nominal cross section used in the VTP Phase III will be assessed using conventional layered elastic design. The predicted traffic for failure by rutting in the subgrade based on vertical compressive strain and for asphalt concrete fatigue cracking based on asphalt tensile strain will be computed using layered elastic theory and the criteria of Barker and Brabston (1975) given earlier. Three conditions will be considered: 1) the nominal cross section and the cross section with 2) 1 ft and 3) 2 ft of thawed soft material in the top of the subgrade, following freezing to a total depth of 5 ft. Table 4.2.3 shows the material properties used for the assessment. The thawed layer is represented by a soft layer with a modulus of 2,000 psi or roughly a CBR of approximately 1 or 2. Recent tests at CRREL have found that frozen silts of the type used in this test section can have modulus values of 80,000 to over 200,000 psi, depending on temperatures and other conditions. A lower value was selected for this analysis.

The results of the calculation and application of asphalt cracking and subgrade rutting criteria given earlier are shown in Table 4.2.4. Figure 4.2.5 shows these results graphically. The beneficial impact of lowering tire pressure on predicted traffic is clear. In Table 4.2.4, the unfrozen nominal cross section shows a better than 16-fold increase in predicted coverages to subgrade failure for a decrease in tire pressure from 100 to 35 psi. For the two thawing conditions, the increase in predicted traffic is about 2.8 times for the same drop in tire pressure. The asphalt fatigue predictions show an even more dramatic impact with reduction in tire pressure leading to 80- to 160-fold increase in predicted life for fatigue cracking as tire pressure is reduced.

The layered elastic analysis of the VTP Phase III test section suggests that subgrade shear rutting should be the dominant failure mode and should appear very rapidly (less than 1000 coverages for all test conditions). The test section would be expected to suffer massive subgrade failure long before asphalt fatigue becomes an issue. However, nothing in this analysis considers possible base failures. This is a low quality base material and there is no accepted criterion to evaluate its possible life. The analysis assumed a fairly optimistic subbase quality for this material. Under conditions where additional moisture could be introduced into the base during the freezing process (a condition often found in the field but not modeled in the VTP Phase III Test Section), this base material could be a major contributor to failure. In such a scenario, reduced tire pressure, as discussed earlier, would have a major impact on the performance of the base materials.

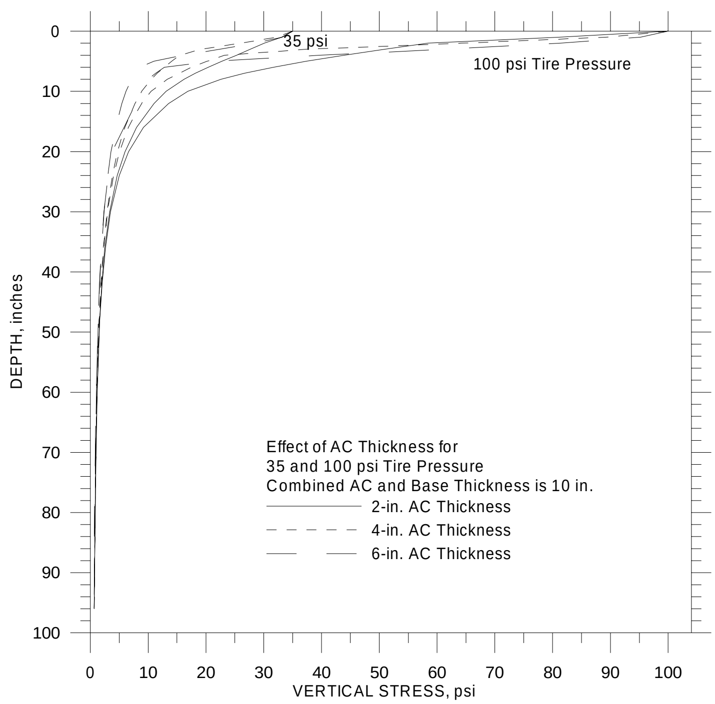


**Figure 4.2.4: Effect of increasing asphalt concrete thickness on stress distribution.**

**Table 4.2.3 Material properties used for layered elastic analysis**

| Layer | Thickness, in. | Modulus, psi | Poisson's Ratio |
|---|---|---|---|
| **Nominal Cross Section** | | | |
| Asphalt Concrete | 2 | 500,000 | 0.35 |
| Base Course | 8 | 16,000 | 0.35 |
| Subgrade | - | 8,000 | 0.40 |
| **1 Foot of Thawed Subgrade** | | | |
| Asphalt Concrete | 2 | 500,000 | 0.35 |
| Base Course | 8 | 16,000 | 0.35 |
| Thawed Subgrade | 12 | 2,000 | 0.4 |
| Frozen Subgrade | 38 | 100,000 | 0.4 |
| Original Subgrade | — | 8,000 | 0.4 |
| For 2 ft of thawed subgrade the thawed subgrade thickness is 24 in. and the frozen subgrade thickness is 26 in. | | | |

**Table 4.2.4: Results of strain calculations for various thawed and unfrozen conditions**

| Test Condition | Parameter | Tire Pressure, psi | | |
|---|---|---|---|---|
| | | 35 | 65 | 100 |
| Unfrozen, nominal section | Asphalt Tensile Strain | 0.000247 | 0.000470 | 0.000681 |
| | Predicted Repetitions to Asphalt Fatigue Cracking | 337,878 | 13,544 | 2,121 |
| | Subgrade Vertical Strain | 0.00156 | 0.0019 | 0.00209 |
| | Predicted Repetitions to Subgrade Rutting | 745 | 104 | 46 |
| Thaw 1 ft into subgrade | Asphalt Tensile Strain | 0.000302 | 0.000533 | 0.000747 |
| | Predicted Repetitions to Asphalt Fatigue Cracking | 123,655 | 7,221 | 1,335 |
| | Subgrade Vertical Strain | 0.00359 | 0.00416 | 0.00446 |
| | Predicted Repetitions to Subgrade Rutting | 36 | 18 | 13 |
| Thaw 2 ft into subgrade | Asphalt Tensile Strain | 0.000314 | 0.000546 | 0.000760 |
| | Predicted Repetitions to Asphalt Fatigue Cracking | 101,765 | 6,401 | 1,225 |
| | Subgrade Vertical Strain | 0.00337 | 0.00392 | 0.00421 |
| | Predicted Repetitions to Subgrade Rutting | 48 | 24 | 17 |

Notes: 1. Repetitions are at a point and represent maximum repetitions or coverages in the traffic pattern. To get the equivalent number of passes in the test items multiply the repetitions (coverages) by the pass-to-coverage ratio, which for the VTP Phase III test section is 3.61 for the 35-psi tire, 4.92 for the 65-psi tire, and 6.10 for the 100-psi tire.
2. These are predicted traffic repetitions by conventional layered elastic theory developed for conventional conditions. Applying to soft thawed soils is an extrapolation.
3. Conventional layered elastic design approaches address only subgrade shear rutting and fatigue cracking of the asphalt layer. Other failure modes such as shear rutting in the base course, shear rutting in the asphalt layer, or densification in any layer are not addressed.
4. The subgrade rutting criteria equates to approximately 1-in. of shear induced rutting in the subgrade.

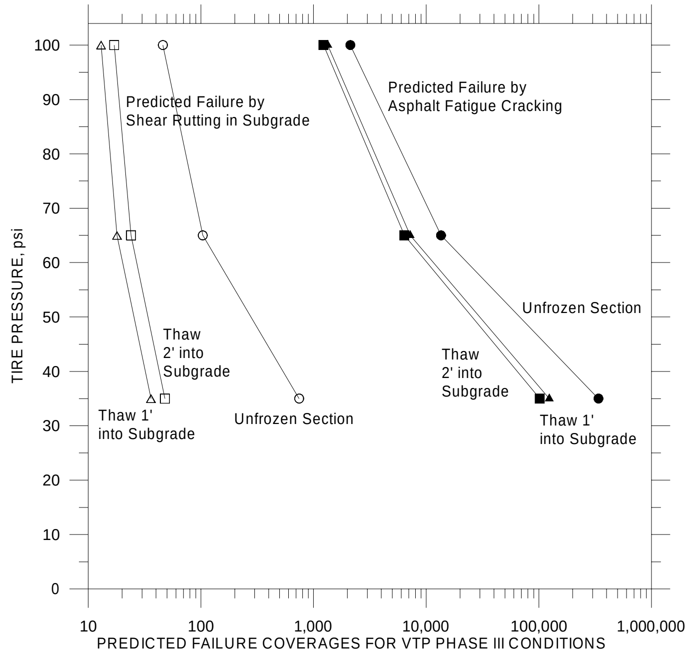


**Figure 4.2.5. Predicted performance of VTP Phase III test section using layered elastic theory.**

## *4.3 Falling Weight Deflectometer Results*

The primary assessment of the structural condition of the test items during freezing and thawing was the falling weight deflectometer (FWD). The quality of these data collected during testing has proven extremely disappointing. For instance, all of the FWD deflection basins collected during the thawed-base-course portions of the test exceeded the range of the sensors so the data cannot be used for back-calculation analysis. In other test series, apparently faulty sensors provided anomalous deflection basins. Even when reasonable basins or when suspected faulty sensor data were removed to provide usable basins, back-calculated moduli values using the Corps of Engineers WESDEF procedure had unacceptable errors. This is a robust back-calculation procedure that has been used by the military in standard evaluations for over 20 years. The standard back-calculation procedures such as WESDEF use known thicknesses, assumed Poisson's ratio values, and then vary the modulus of the layers until an acceptable match between a layered-elastic calculated basin and the measured deflection basin is achieved. With the VTP Phase III basins, errors between the back-calculated basin and the measured FWD basin are often ranging from 20 to over 6 % which makes the back-calculated moduli layers highly suspect.

A number of problems may be contributing to the poor results achieved with the FWD data. First, the calculation is very sensitive to the thickness used in the back-calculation. The VTP Phase III Test Section had variation in thicknesses of the different layers. The back-calculation had to use average thickness values, but there are point to point variations that could impact the results. Periodically, there are problems with sensor readings. This suggests a possible consistent equipment problem that may have provided inaccurate measured basins. Finally, many of the physical test section conditions that existed are simply hard to monitor well with layered elastic theory. In particular, soft thawed layers between a stiff asphalt surface and a lower stiff frozen layers are computationally difficult with layered elastic theory and are far different from pavement conditions normally assessed with FWD technology and back-calculation.

The original VTP Phase III Test Section plan relied heavily on FWD to determine structural conditions in the various freezing and thawing phases of the test. All efforts to date have failed to reduce these data into a usable, reliable set of results. Work continues with these data, but conventional back-calculation is not adequate to properly interpret the data collected during the test.

Of the hundreds of individual FWD data sets that have been examined and analyzed, only one seems to give reasonable results with acceptable error in matching the calculated basin and the measured deflection basin. This data set is from the nominal 2-ft thaw in the subgrade and the information is summarized in Table 4.3.1. Table 4.2.3 presented estimated modulus values for these materials based on their characteristics described in section 2.4 and analysis requirements discussed in section 4.2 . In general, the back-calculated modulus values show general agreement with the estimated values used for the earlier for a-priori estimates of test item performance. The back-calculated asphalt modulus of 496,000 psi is consistent with the 500,000 psi estimated as appropriate for a medium quality asphalt concrete at relatively low temperatures. The back-calculated thawed subgrade values of 2600 psi and the 2000 psi for Table 4.2.3 are both consistent with a weak CBR 1 or 2, wet, poorly compacted ML silt. Similarly, the frozen layer modulus values of 118,000 and 100,000 psi are remarkably close. This requires some consideration. The back-calculated values had a large range from 26,329 to 281,427 psi. Frozen wet materials have the capability to achieve quite high stiffness, but the stiffness depends highly on temperature and ice content. In the test section, the frozen layer is thawing from the top and bottom, so the actual structure is stiffest in the middle where the temperature is the lowest and then a gradient exists to quite soft at the thawing front. Thawing is a natural process, so the thawing front will be irregular and not the precise line required in analysis. In examining the back-calculated values, one sees that those three results over 200,000 are associated with matching errors greater than 4%. The two over 100,000 psi have 3 to 4% error, those in the range of 70,000 to 1 to 2% errors, and those below 70,000 have errors less than 2%. Considering the rough approximation of representing the varying stiffness and thickness frozen layer, probably a representation of the modulus in the 70,000 to 100,000 psi range might be more reasonable than the back-calculated average value of 118,000 psi. The original average back-calculated modulus for the original subgrade that was never frozen was 9900, which is similar to the 8000 psi value used in Table 4.2.3. This back-calculated value would be consistent with a CBR of 6 or 7, which is consistent with what one might expect from a ML silt at moderate density and moisture content. It is also reasonably consistent with the CBR values measured after the test and presented in Table 2.4.3. This bottom subgrade silt had originally been placed and tested earlier. Only the upper portion of the silt layer was reworked and replaced for the VTP Phase III Test Section. Hence, one might expect this lower subgrade to be somewhat denser and stronger than the poorly compacted upper subgrade in the initial portions of the test.

The biggest discrepancy in estimated and back-calculated modulus values is in the base course. As discussed in section 4.2, the 16,000 psi modulus was an optimistic value because of the characteristics of the base and the low density of placement and potential wet thawed conditions. Hence, a lower value as suggested by the back-calculation would certainly be reasonable. However, 11 of the 14 back-calculations assigned the base a minimal modulus of 5000 psi, and the actual value might be lower. If one uses an average 7.5-in.-thick layer of subbase material and underlying layers of 2336 to 3153 psi modulus consistent with the range seen in the back-calculated results in Table 4.3.1 with the modulus relation of Barker and Brabston (1975) given earlier in section 4.2, one would estimate base modulus to have values of 6271 to 7904 psi. Considering that the base material does not actually meet subase-quality requirements used as a basis in the work by Barker and Braston (1975), the placed densities and volume expansion on freezing, and wet conditions of the thawed base (see Figure 4.5.9 and associated discussion later) , the back-calculated modulus of about 5000 psi seems quite reasonable.

Based on the best estimates of laboratory characteristics, placement conditions, and one successful FWD measurement, the materials in the VTP Phase III test Section seem most reasonably modeled in layered elastic theory by modulus values of 496,000 psi for the asphalt layer, 5000 psi for the thawed base, 2600 psi for the thawed subgrade, 100,000 psi for the frozen subgrade, and 9900 psi for the original unfrozen or recovered subgrade. The base course should recover to be stiffer than when in the initial wet thawed state. No reliable FWD data are available to estimate this. Barker and Brabston's (1975) procedure is probably the best available estimate at this point. For a 7.5-in. base composed of near subbase-quality material on an original or recovered modulus of 9900 psi, this would equate to an original or recovered modulus of about 18,000 psi or somewhat less. In-situ DCP measurements had suggested this base was on the order of 15 to 30 CBR (see section 2.4) and Figure 2.4.4 suggested placement CBR values of less than 10 to 20. A CBR value of 12 or so would be consistent with a modulus of about 18,000 psi. With the lack of viable better measurements or estimates, an original or recovered modulus of 18,000 psi seems to be a good estimate.

**Table 4.3.1 Back-calculated Modulus Values, 2-ft Thaw in Subgrade**

| | **Asphalt Surface E, psi** | **Thawed Base E, psi[1]** | **Thawed Subgrade E, psi** | **Frozen Subgrade E, psi** | **Original Subgrade, E, psi** | **Error Matching Basin, %** |
|---|---|---|---|---|---|---|
| Mean | 496,296 | 5,034 | 2,641 | 117,635 | 9,924 | 2.5 |
| Standard Deviation | 49,210 | 84 | 256 | 91,558 | 293 | 1.30 |
| Coefficient of Variation, % | 9.9% | 1.7% | 9.7% | 77.9% | 3.0% | 51.4% |
| Max Value | 558,879 | 5,302 | 3,153 | 281,427 | 10,251 | 4.6 |
| Min. Value | 403,475 | 5,000 | 2,336 | 26,369 | 9,039 | 0.6 |
| Usable Measurements | 14 | 14 | 14 | 14 | 14 | 14 |

Notes: 1. The back-calculation set a minimum 5,000 psi modulus for this layer and 11 of the 14 back-calculations hit the minimum for the base modulus.
2. Testes January 2, 2001, Feature: Station 2 - Section 01, Station 4, Drops 1 through 16.

## 4.4 Assessment of Load Interactions in the Test Section

The individual test items of the VTP Phase III Test Section were located relatively close together. However, examination of the stress distribution of adjacent loads in pavements, shows that as one proceeds deeper. the overlapping stresses become additive and stress distribution becomes quite different than with a single load. This is illustrated in Figure 4.4.1 where two hypothetical loads the size of the VTP Phase III Test Section loads with 100-psi tire pressure are imposed on the surface. The interaction at shallow depths such as the top of the basic course (2-in. depth) are negligible, but as one reaches the top of the subgrade (10-in. depth) or 1 ft into the subgrade (22-in. depth), the differences between single and multiple wheels become significant. Both stress and damage are treated as linear functions in linear elasticity theory for pavements.

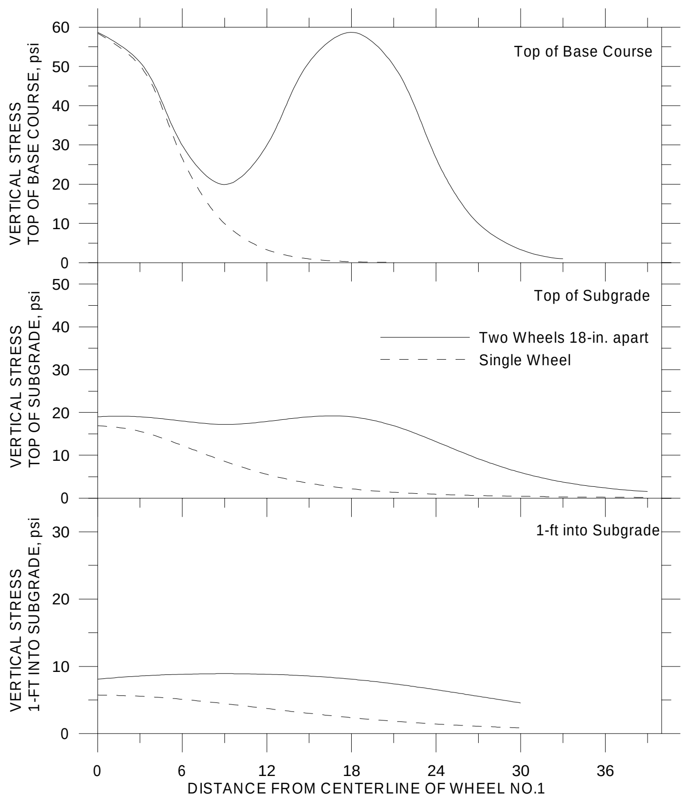


**Figure 4.4.1 Potential impact of adjacent test items on one another: Theoretical effect of adjacent loads on stress distribution.**

Hence, the nearness of adjacent test items would be expected to have little impact on trafficking for the items tested when the base was thawed but for the test items with the top foot of the subgrade thawed and top 2 ft thawed, the adjacent traffic may have affected the results. Some upheaval was observed in areas immediately adjacent untrafficked test items following trafficking, but records were not adequate to ascertain how major a problem this may have been. The center lane of the test items were trafficked frozen, then the outside lanes were trafficked when the base was thawed and then the top foot of subgrade had thawed. The top 2 ft of subgrade thawed test items were then run on the same center test items as the frozen tests. If there was collateral influence from adjacent testing, it is most likely in these test items tested when the top 2 ft of the subgrade were thawed.

## 4.5 Rutting Performance of the Test Sections

The performance of the test section will primarily be assessed by surface rutting as measured by the surface profilometer. Summaries of these data will be used in this section and the complete surface data are in the Appendix D of the report. The road depth probe is an experimental device that was placed at the subgrade and base course interface to determine quantitative estimates of rutting at depth. The road depth probe data are reported in the appendix, but there was sufficient variability and anomalies in the data that they were not incorporated into the analysis. It is promising technology, but more work is needed to develop it further.

In Figures 4.5.1 through 4.5.5 the average maximum surface deformation is plotted with the total number of HVS load passes applied in the 3-ft wander width. At various levels of trafficking, three transverse profiles were measured for each test item, and these profiles are in the appendix. The maximum deformation on each transverse profile was determined, and the average of these three maximum deformations at each traffic level is the data shown in Figures 4.5.1 through 4.5.5. Deformations are determined as deviations from the original untrafficked profile. Note that this deformation is different than if one placed a straightedge across the tops of the upheaved areas and measured down to the bottom of the deformation, as is sometimes done.

The frozen section showed negligible rutting through 5010 passes. However, the test items for the thawed base, thaw 1 ft into the ground, and 2 ft into the ground, all failed rapidly under traffic. The last three test items that were allowed to completely thaw and recover showed some slight improvement in their ability to support load.

Surface deformation may arise from densification, which is simply the result of the pavement materials being compacted by traffic without shearing, or it may arise from plastic shear deformations within the paving materials. Densification generally causes a depression in the trafficked area while shearing causes a depression in the trafficked width with adjacent upheaved portions, as illustrated in Figure 4.5.6. In the field, these modes may appear singularly or together and as the zone of shearing is deeper in the pavement, the upheaval height may be smaller and the area of upheaval may be wider. This may make the upheaved portion more subtle and harder to detect. Other than the frozen test items, the surface profile data plots in the appendix almost invariably show distinct upheaved portions adjacent to the trafficked zone. This suggests the failure mode was shearing within the pavement structure. The test items with the thawed base and frozen subgrade failed rapidly and dramatically with shear upheaval adjacent to the traffic lane, regardless of tire pressure. This clearly shows the base material was not stable under the test loads. The rapid failure of the recovered test items may also simply reflect the fact that the base course material was inadequate for the loads. With the base unstable under the test traffic loads, the additional thawing of the subgrade simply sped up the failure, and no test item was capable of supporting the test traffic under any condition of thawing. The recovered test items did only marginally better than the frozen items with various degrees of thawing.

Figure 4.5.7 illustrates different modes of deformation that may develop with traffic. In the simplest case, there is some minor densification (perhaps combined with minor shear deformation), and the pavement system then becomes stable for an extended period of time. Alternatively, there may be some initial densification followed by a slow progressive accumulation of shear deformations. Finally, there may be rapid shear failure. Almost all the failures in the thawed and recovered test items show deformation patterns consistent with rapid shearing.

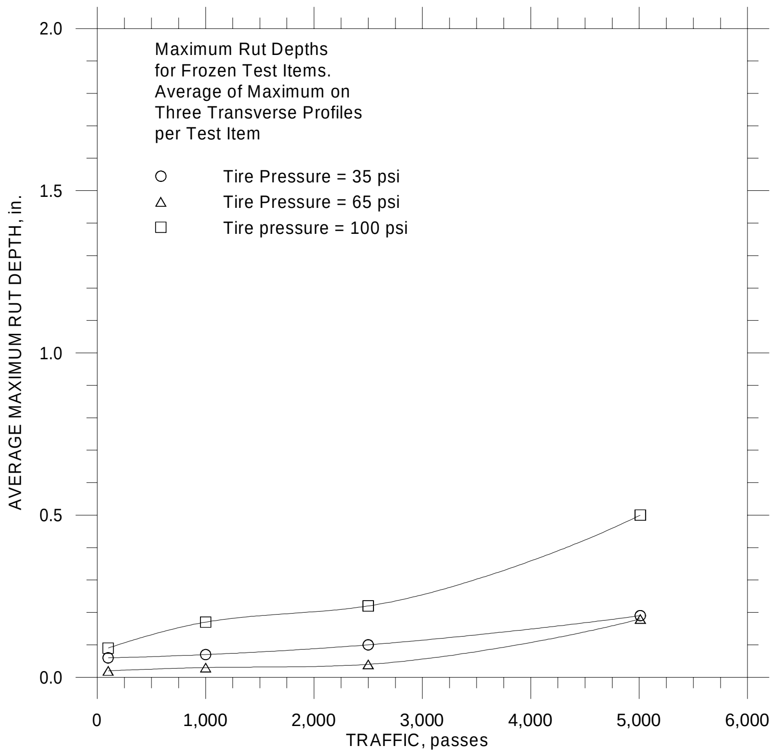


**Figure 4.5.1: Rutting performance of frozen test items.**

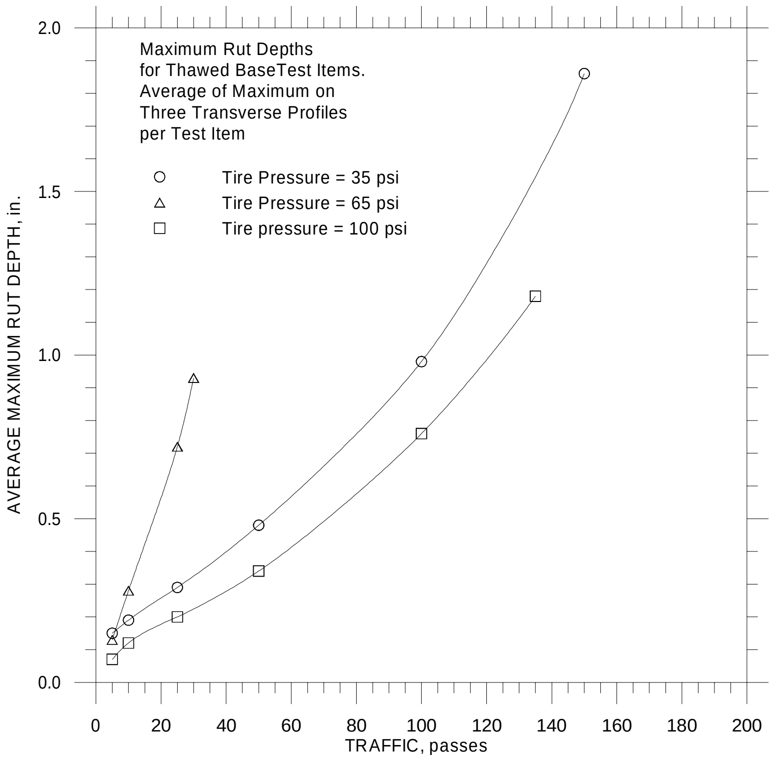


**Figure 4.5.2: Rutting performance of thawed-base test items.**

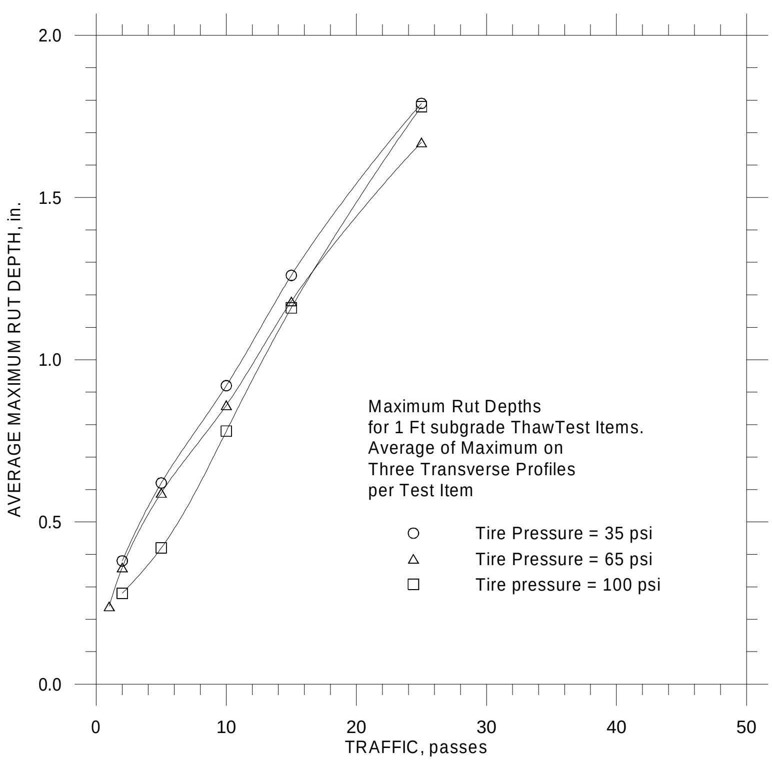


**Figure 4.5.3: Rutting performance of 1-ft thaw into subgrade test items.**

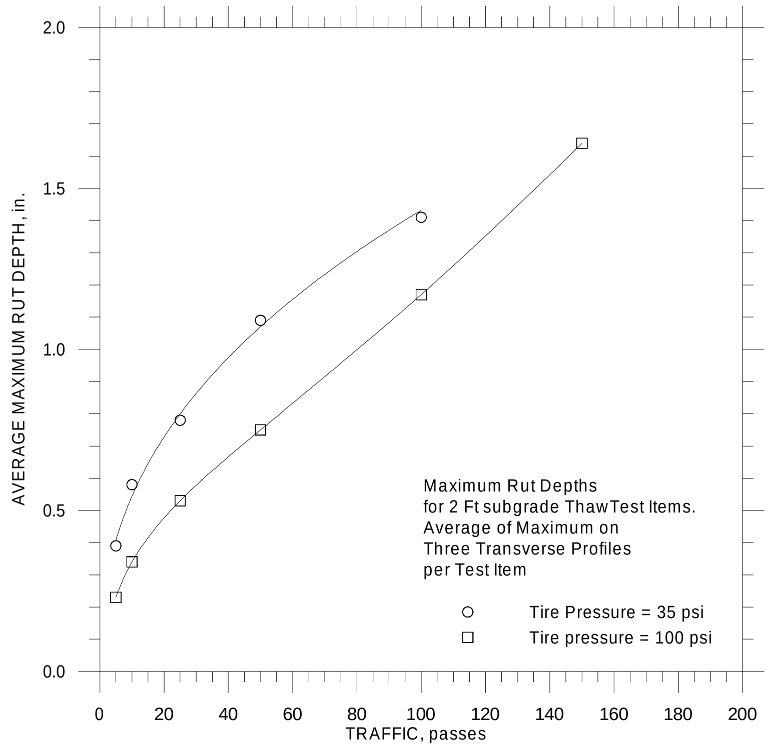


**Figure 4.5.4: Rutting performance of 2-ft thaw into subgrade test items.**

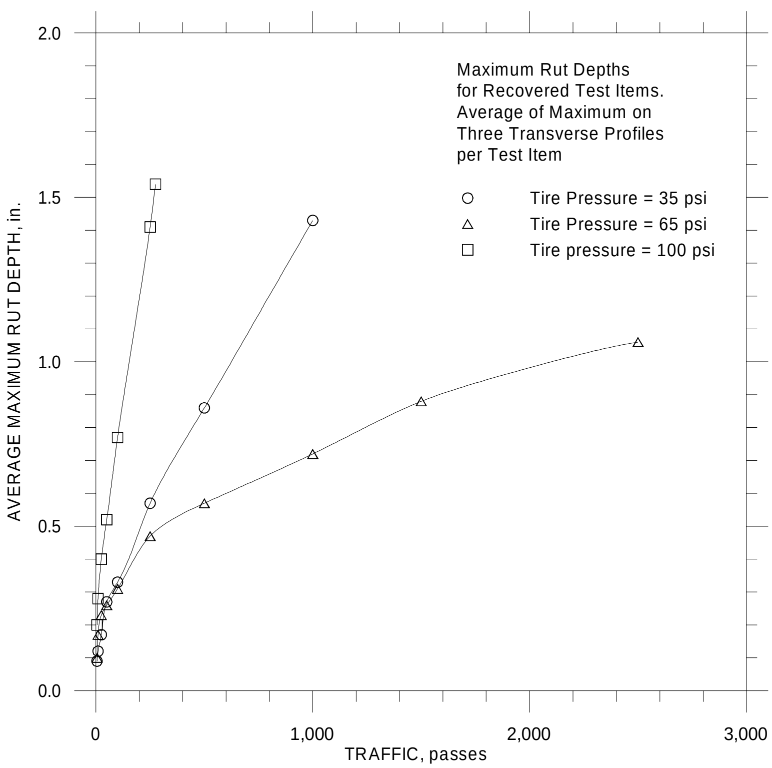


**Figure 4.5.5: Rutting performance of recovered test items.**

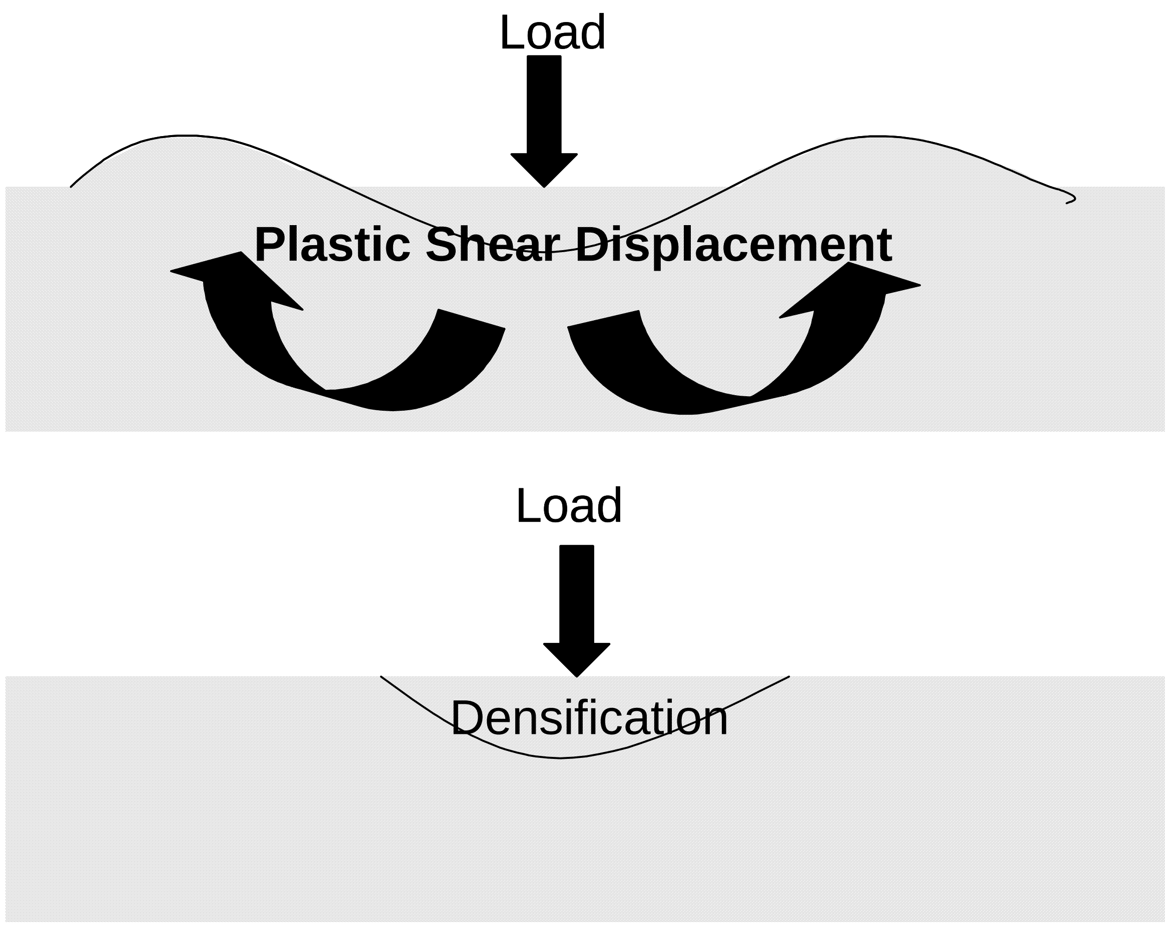


**Figure 4.5.6: Conceptual illustration of rutting from shear deformation and from densification.**

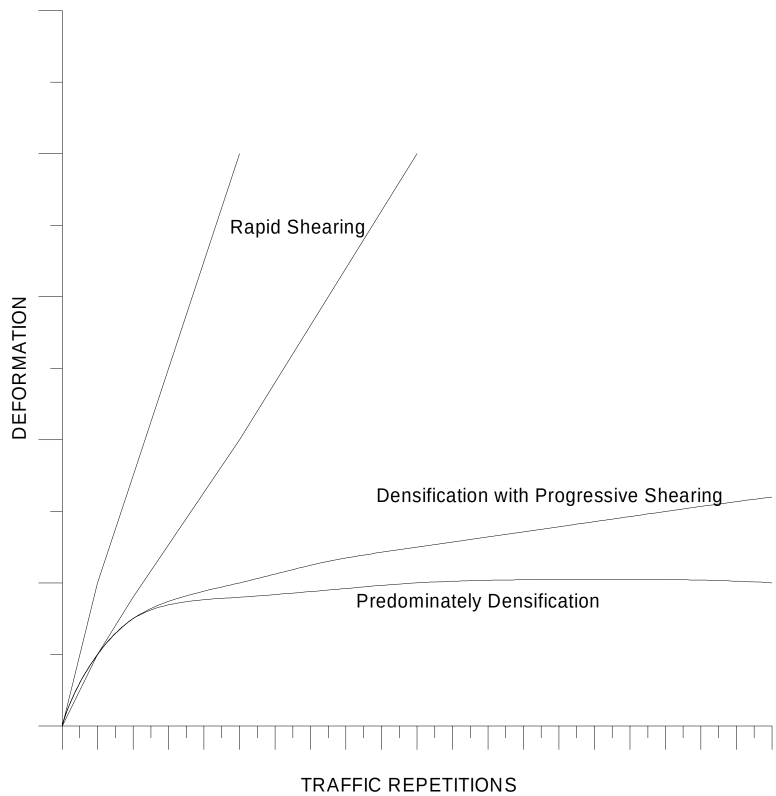


**Figure 4.5.7: Deformation patterns for different types of failure modes.**

The maximum number of stress repetitions at a point within a traffic lane subjected to a normal distribution of traffic is a function of the tire width of the trafficking vehicle. As the tire pressure is decreased at constant wheel load, both tire contact area and width increase. Table 2.2.1 provided approximate calculations to estimate the pass-to-coverage ratio so that passes and coverages could be interrelated for the VTP Phase III Test Section traffic.

In Table 4.5.1, the number of traffic passes at failure for each test item is shown, these are converted to coverages at failure, and the mode of failure of each test item is noted. Failure for this analysis was taken as 1-in. average maximum deformation in Figures 4.5.1 through 4.5.5. Figure 4.2.5 had made some preliminary predictions of performance, based on estimated material properties, and had predicted the unfrozen (recovered) test items might support between 50 and 750 coverages of traffic, depending on tire pressure, but when the subgrade was allowed to thaw, these estimates plummeted to 13 to 48 coverages. The material properties used in this analysis were more favorable than those suggested in Table 4.3.2. This predicted behavior is consistent with the observed behavior. The recovered sections supported 25 to 422 coverages but the thawed subgrade sections saw dramatic rapid failure in 2 to 13 coverages. The earlier analysis could not account for possible base course failure, as there remains no accepted criteria for analyzing these portions of the pavement analytically. In the VTP Phase III Test Section, the thawed base course on frozen subgrade alone led to failure in 6 to 28 coverages.

As the VTP Phase III Test Section was frozen from the surface downward, water deeper in the section is drawn upwards to the freezing front. This can be seen in Figure 4.5.8 where the TDR measured moisture content begins to decrease until the vertical portion of the curve indicates freezing. Note that moisture content has been converted from volumetric to gravimetric moisture content and that when the water freezes there is a dramatic change in dielectric constant, causing a sudden vertical drop in the curve, followed by a horizontal portion until the ice thaws with a resulting vertical rise. In Figure 4.5.9 and 4.5.10 the impact of this upwards draw of water can be seen in the base course and upper subgrade. When these thaw, their moisture content is higher after thawing than when they originally froze. For the base course, this is an increase from approximately 8% to 14 or 15% (Figure 4.5.9). For the subgrade about 8 in. below the base, it is an increase from around 21 to about 24 or 25% (Figure 4.5.10). Both the silty base and subgrade material would see a drop in strength with an increase in moisture. In both cases, water within the soil and aggregate skeleton expands 9% when it freezes. This would lower the material density with a further adverse impact on strength. Hence, given the nature of the base and subgrade materials, the loss of density upon freezing, and the increase in moisture content when thawed, the poor performance of the materials under traffic is not surprising.

**Table 4.5.1: Determination of failure coverages**

| Test Condition | Tire Pressure (psi) | P/C Ratio | Passes at Failure | Coverages at Failure | Dominant Deformation And Failure Mode |
|---|---|---|---|---|---|
| Frozen | 35 | 3.61 | 5,010+ | 1,388+ | Unfailed, some densification |
| | 65 | 4.92 | 5,010+ | 1,018+ | Unfailed, some densification |
| | 100 | 6.10 | 5,010+ | 821+ | Unfailed, some densification |
| Base Thawed | 35 | 3.61 | 102 | 28 | Rapid shear |
| | 65 | 4.92 | 32 | 6 | Rapid shear |
| | 100 | 6.10 | 120 | 20 | Rapid shear |
| 1-ft Thaw into Subgrade | 35 | 3.61 | 11 | 3 | Rapid shear |
| | 65 | 4.92 | 12 | 2 | Rapid shear |
| | 100 | 6.10 | 31 | 5 | Rapid shear |
| 2-ft Thaw into Subgrade | 35 | 3.61 | 42 | 12 | Rapid shear |
| | 65 | 4.92 | | | Not Tested |
| | 100 | 6.10 | 80 | 13 | Rapid shear |
| Recovered | 35 | 3.61 | 625 | 173 | Rapid shear |
| | 65 | 4.92 | 2,075 | 422 | Progressive shear |
| | 100 | 6.10 | 150 | 25 | Rapid shear |

Notes: 1. Pass-to-coverage ratio from Table 2.2.1.

2. Passes to failure determined from Figures 4.5.1 through 4.5.5 with failure defined as 1 in. Rut depth.

3. Shear and deformation modes defined in Figure 4.5.6 and 4.5.7. Judgement based on Figures 4.5.1 through 4.5.5 and profilometer cross sections in appendix.

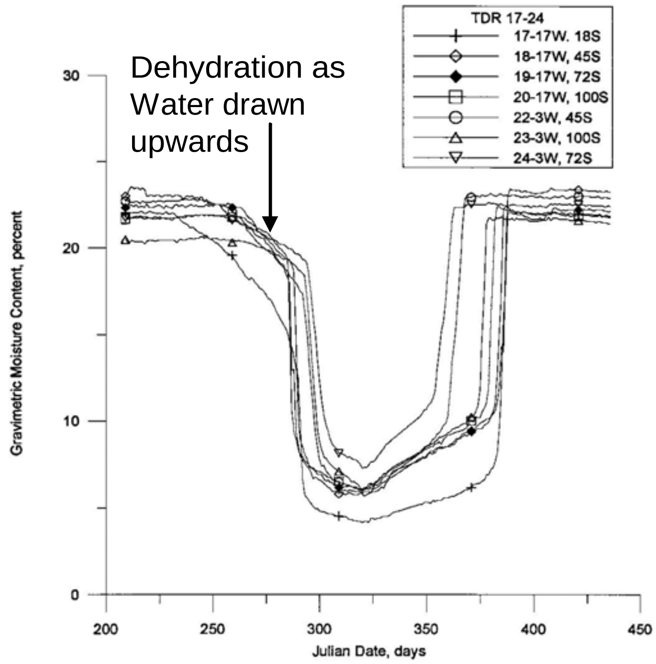


**Figure 4.5.8 Dehydration of subgrade at 4-ft depth as water drawn upwards to freezing front.**

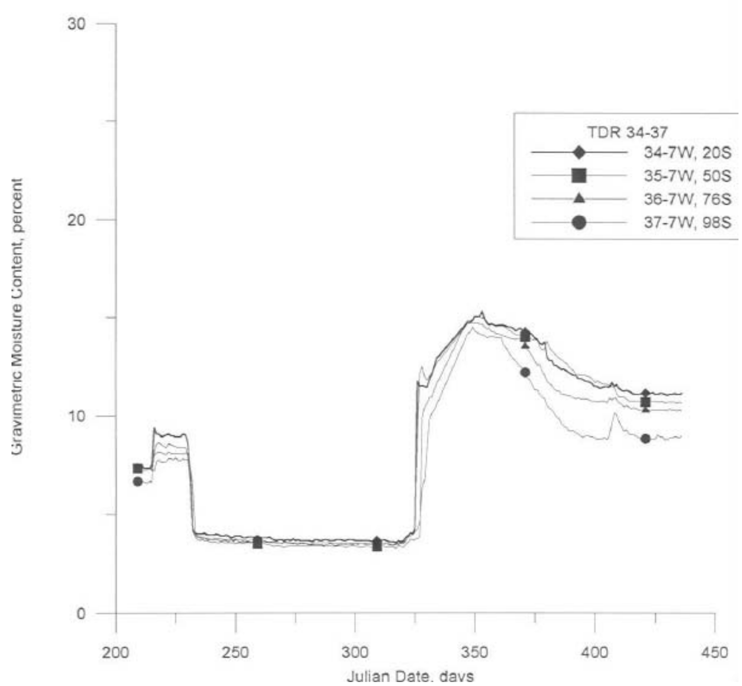


**Figure 4.5.9 Base moisture content initially, freezing period (horizontal because of change in dielectric constant), and increase in moisture content upon thawing.**

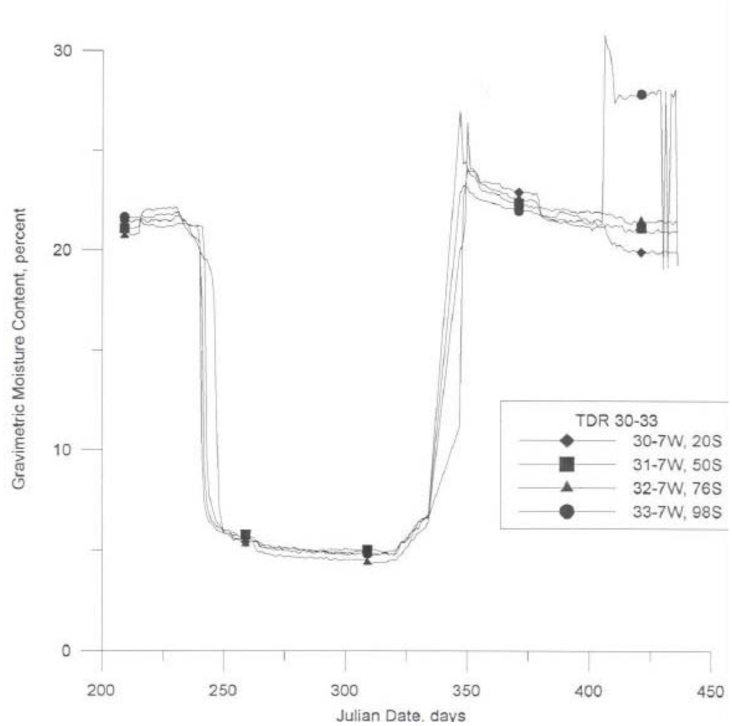


**Figure 4.5.10 Moisture content in subgrade before and after freezing approximately 8 in. below base and subgrade interface.**

## *4.6 Assessment of Results*

Test items 3, 6, and 9 were tested when essentially only the base was thawed. Because the subgrade remained frozen, the failure mode was dominated by shear failures in the base course, which all appeared rapidly. From the earlier discussions in section 2.4, it is likely that the base was placed at CBRs of something less than 10 to 20, after recovery DCP tests suggested CBRs of 15 to 30, and assessment of the material characteristics suggest performance of well compacted base material of this type should not be expected to be above 20. In Table 4.6.1 the effective CBR was back-calculated from the Corps of Engineers' CBR equation (Ahlvin 1991, Pereira 1977). To accomplish this, the failure coverages from Table 4.5.1 were used to determine the appropriate alpha traffic factor for use with the CBR equation. The tire pressure and contact pressure for each of the loading conditions was used with the alpha factor to calculate the CBR value for which the required thickness was 2.4—the average asphalt thickness above the base course in the test. This is the CBR for which the given traffic and load conditions would require the thickness of asphalt actually present. Essentially, the base failures in these three items occurred as though the base had a CBR of 15, 13, and 24 for the 35-, 65-, and 100-psi tire loads. These are all CBR values well within the range of what one would expect the in-situ base course based on previous discussions. Hence, the rapid failure of this base course material under these test conditions is just as existing pavement theory would predict. The variation in back-calculated CBR values is consistent with the variation in asphalt thickness and base density noted in earlier sections. Under these test conditions, material properties dominate pavement behavior, and tire pressure effects are secondary factors.

Back-calculation of effective CBR values were then made for the 1-ft thaw into the subgrade and the recovered test items. This was done as above, except one set of calculations was made for the CBR for the base course and one for the top of the subgrade (i.e., determine the CBR to predict failure coverages for which overlying pavement is 9.9 in. [asphalt plus base thickness]). These results are also shown in Table 4.6.1. For the 1-ft thaw into the subgrade, the base CBRs would have to be 8, 11, and 15 to explain the failure for the 35-, 65-, and 100-psi loadings. This remains within the estimated range of the in-situ base course developed in earlier sections. The subgrade would have to be in the 0.7 to 1.2 CBR range to explain the failure. Very wet loose silts can reach these low values. In this case, the low density is present but the only additional water present comes from the upward draw of water from lower in the test section without a continuous feed of water (as from a ground table). Figure 4.5.10 shows an increase in moisture in the thawed subgrade, but it is not terribly dramatic. The FWD results in Table 4.3.1 suggested a CBR of 2, but these are all close. Probably both the subgrade and the base contributed to the rapid failure seen here as both are in or very near the estimated in-situ values.

The back-calculation of the CBRs for the recovered sections found the failure could be explained by base CBR values of 21, 18, and 26 or by subgrade CBR values of 5, 2, and 3 for the 35-, 65-, and 100-psi loadings. The base values are once again in the range expected of this base material. The subgrade CBR values are slightly lower than the 4- to 6-CBR values measured in situ after testing. Once again, the low-quality base material appears the most likely cause of failure.

**Table 4.6.1: Determination of effective CBR values for observed failures**

| Test Condition | Tire Pressure | Failure Coverage (from Table 4.5.1) | Effective CBR if Failure is in: | |
|---|---|---|---|---|
| | | | Base Course | Top of Subgrade |
| Base Thawed | 35 | 28 | 15 | Frozen |
| | 65 | 6 | 13 | Frozen |
| | 100 | 20 | 24 | Frozen |
| 1-ft Thaw into Subgrade | 35 | 3 | 8 | 0.7 |
| | 65 | 2 | 11 | 0.9 |
| | 100 | 5 | 15 | 1.2 |
| Recovery | 35 | 173 | 21 | 5 |
| | 65 | 422 | 18 | 2 |
| | 100 | 25 | 26 | 3 |

Analysis of the test results using very conventional CBR pavement theory is adequate to explain the observed failures in light of the loads and materials used in this pavement test section.

The assessment of pavements with thawed and frozen layers is computationally difficult and ascertaining appropriate material properties is similarly challenging. In section 4.3, the best estimate of material properties available was developed. In Table 4.6.2, the results of layered elastic computations and the subgrade strain criteria of Barker and Brabston (1975) are used to estimate the traffic to subgrade failure for the 1-ft thaw into the subgrade cases. The theoretical predictions are quite close to the observed failures and predicted the rapid early failure observed. As noted earlier, layered elastic theory has yet to develop a usable failure criterion for granular bases so a base course assessment could not be carried out as could be done with the older CBR method. As also noted earlier, the CBR assessment suggested that the base course was a major contributor to rutting, which would lead one to expect more rapid failure than layered elastic predictions based solely on subgrade failure.

Overall conventional CBR and layered elastic theory show themselves to be useful tools for assessing behavior of thaw-weakened pavement sections such as seen in the VTP Phase III Test Section. Both are able to account for the effect of varying tire pressure. The layered elastic approach can more easily assess varying stiffness in layers but has no criterion for checking base course adequacy as the CBR does.

Conventional pavement analytical theory would predict that reduced tire pressure can lead to reduced damage in pavements. Because of the nature of stress distribution within the pavement, the beneficial effects of reduced tire pressure are most pronounced in the upper regions of the pavement, notably the asphalt and base course layers. Depending on the specific loads and geometry involved, reduced tire pressure can also significantly impact subgrade rutting (e.g., Figure 4.2.5).

The test objectives of the VTP Phase III Test Section were to ascertain if reduced tire pressure would enhance the behavior of a low-quality base and weak subgrade during thawing conditions. Figure 4.6.1 summarizes the trafficking results for the test conditions. In this case, the base and subgrade materials, whether in thawed or recovered condition, were inadequate for the applied loads, and rapid shear failures occurred regardless of tire pressure. Only when frozen, were the test section materials adequate to support the traffic loads. The poor quality base course in particular seems to have been a major cause of the rapid failure of the test items.

**Table 4.6.2: Comparison of Calculated and Actual Coverages to Failure for the 1-ft Thaw into Subgrade Case**

| Material | Thickness | Modulus, psi | Poisson's ratio |
|---|---|---|---|
| Asphalt Layer | 2.4" | 496,000 | 0.35 |
| Base | 7.5" | 5,000 | 0.35 |
| Thawed Silt | 12" | 2,600 | 0.4 |
| Frozen Silt | 38" | 100,000 | 0.4 |
| Unfrozen Silt | 16" | 9,900 | 0.4 |
| Rigid Base | - | 1,000,000 | 0.5 |
| **Traffic Predictions for 1-ft thaw into subgrade** | | | |
| **Tire Pressure** | **Layered Elastic predictions based on subgrade strain** | | **Actual coverages to failure** |
| 35 | 15 | | 3 |
| 65 | 7 | | 2 |
| 100 | 5 | | 5 |

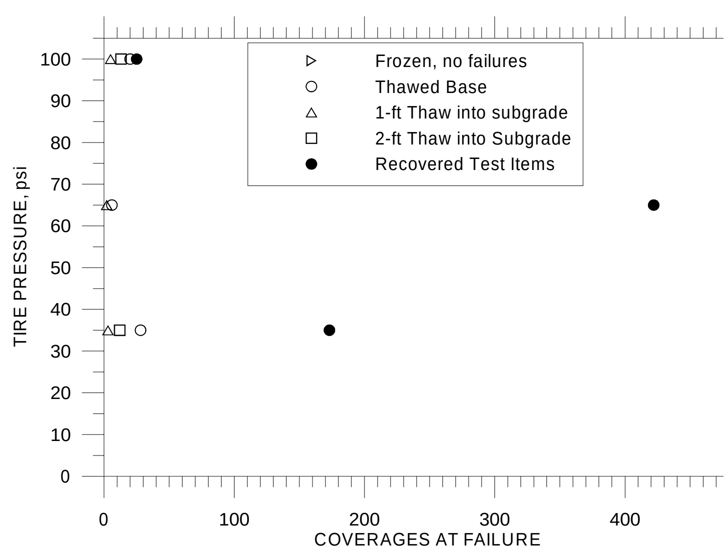


**Figure 4.6.1 Failure coverages for different test conditions.**

# CHAPTER 5
# CONCLUSIONS

This report has gathered the available information on the VTP Phase III Test Section built and tested in the summer of 2000 through the summer of 2001. This information has been documented and analyzed in this report.

The objective of the VTP Phase III Test Section was to evaluate the effectiveness of reduced tire pressure for enhancing the performance of a relatively weak pavement subjected to freezing and thawing. The pavement subgrade was a weak, highly frost-susceptible ML silt and the base was highly frost-susceptible gravelly, silty sand that did not meet conventional subbase criteria. The test section was surfaced with asphalt concrete. Traffic was applied with the CRREL HVS with a 9000-lb wheel load at tire pressures of 35, 65, and 100 psi. Nominal test conditions included fully frozen, thawed through the depth of the base, thawed 1 ft into the subgrade, thawed 2 ft into the subgrade, and fully thawed (recovered).

Only when the test section was fully frozen were any test items capable of withstanding substantial traffic. Under essentially all other test conditions rapid shear failures developed in 2 to 28 coverages of traffic. In two of the recovered sections, the test items lasted slightly longer: 173 and 422 coverages before failing in shear. Various analysis techniques were used to examine the data, and the substandard base course material appears to be the dominant failure medium in all cases. The weak silt subgrade was probably also a contributor, especially in thawed states, but the quality of the base course alone is sufficient to explain all observed rapid failures. The combination of weak base and subgrade materials dominated the behavior of the test section and tire pressure was relegated to a secondary effect. Consequently, the test section was unsuccesful in providing insight into the effectiveness of reduced tire pressure on thaw-weakened low-volume pavements.

Both layered elastic and classical CBR analysis techniques proved adequate to describe the observed behavior of the test items and to account for tire pressure effects and material quality. Each has certain advantages and limitations and are powerful tools when used together. Conventional pavement analysis tools are effective for addressing tire pressure and thaw-weakened materials if the materials can be adequately defined in engineering terms. This is a major challenge and an area of further work. More advanced analytical tools are certainly available, but the inability to describe properties of thaw-weakened pavement materials seems a greater hurdle than computational sophistication.

Boussinesq and layered elastic theory were also used to demonstrate the potential effectiveness of reduced tire pressure for pavements. The effect is most pronounced for the upper base and asphalt layers but, particularly for thin pavements, there may be advantages for the subgrade also. This is a matter that requires site-specific analysis for which existing pavement analysis tools such as CBR or layered elastic theory are adequate.

There is not a general rule or caveat about whether reduced tire pressure will be effective for low-volume, under-designed roads. As with this test section, the specific loads, geometry, and materials must be evaluated before any conclusion on the effectiveness of reduced tire pressure is possible.

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# Appendix A:
# VTP Soils Classification and Test Results

***Soils Classification***
***VTP Test Section***
***Testing Results***

24 July 2001

U.S. Army Engineering Research & Development Center

COLD REGIONS RESEARCH AND ENGINEERING LABORATORY
Civil Engineering Materials Laboratory
July 2001

**General:** The materials tested were obtained during removal of the VTP test sections in FERF. Testing to be performed on the material included grain size analysis (ASTM D422-63), specific gravity (ASTM D854-92), liquid and plastic limits (ASTM D4318-84), proctors (MIL STD 621A CE-12, CE-26, and CE-55), and both unsoaked and soaked CBR's (ASTM D1883-92). Two materials were collected for testing. One material was a coarse-grained material the second material was fine-grained.

**Material/Sample Prep:** Material for testing was collected from the VTP test sections in FERF and brought to the Civil Engineering Materials Lab (CEML) in 5–gallon pails.  The material was placed on pans and oven-dried at 110 ±5ºC.

Material for grain size analysis, liquid and plastic limits, and specific gravity was prepared for testing by separating the soil on a #10 (2.00 mm) sieve. The +10 material was used for mechanical analysis while the –10 material was used for hydrometers and specific gravity. The liquid and plastic limits were performed on material passing the #40 (0.42 mm) sieve.

Individual samples for proctor tests were obtained by splitting the material with a sample splitter. A sample was taken to obtain the moisture content of the soil. After the moisture content was determined, water was added to each individual sample to obtain a range of water contents by weight. These samples were then allowed to sit overnight to allow an even distribution of water.

**Testing:** The same testing procedures were performed on both soils. A general soil classification consisting of mechanical analysis and hydrometers were performed. Specific gravity was determined and the liquid and plastic limits were defined.

Proctor testing followed MIL STD 621A.  All compaction efforts, CE-12, CE-26, and CE-55 were performed.  A mechanical rammer was used for compacting the samples. After each sample was compacted and the weight of the wet soil was obtained, a CBR test was performed. This testing followed ASTM D1883-92. A Boart Longyear automated load frame controlled by Labview was used for testing and data collection.

After completion of the CBR test, the moisture samples were taken from the top and bottom of each mold, weighed and placed in an oven at 110 ±5°C overnight. The samples were then removed and allowed to stand for approximately 30 minutes before the dry weights were obtained.

Soaked CBR's were also performed on both soils at each compaction effort. The samples were compacted in the same fashion as the unsoaked samples but were then placed a water bath and allowed to soak for 5 days. The samples were then removed and allowed to drain for 30 minutes before CBR tests were performed.
**Results:** Three replication of each soil type were run for soil classification. A summary chart of each soil type is below.

| Coarse-Grained Soil | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| % Cobbles | % Gravel | % Sand | % Silt | % Clay | USCS | AASHTO | PL | LL |
| 0 | 20.7 | 57.9 | 17.6 | 3.8 | SM | A-1-b | 0 | 0 |
| 0 | 23.8 | 53.7 | 19.1 | 3.4 | SM | A-1-b | 0 | 0 |
| 0 | 24.5 | 57.3 | 14.4 | 3.8 | SM | A-1-b | 0 | 0 |

| Fine-Grained Soil | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| % Cobbles | % Gravel | % Sand | % Silt | % Clay | USCS | AASHTO | PL | LL |
| 0 | 5.3 | 8.9 | 67.5 | 18.3 | ML | A-4 | 25 | 0 |
| 0 | 1.9 | 9.8 | 71.2 | 17.1 | ML | A-4 | 25 | 0 |
| 0 | 1.3 | 9.8 | 71.2 | 17.7 | ML | A-4 | 25 | 0 |

Proctor tests for each soil type were performed at 3 compaction efforts. A summary chart for each soil is below.

| Coarse-Grained Soil | | |
|---|---|---|
| Compaction Effort | Maximum Dry Density | Optimum Moisture |
| CE-12 | 128.0 pcf | 8.5% |
| CE-26 | 131.0 pcf | 7.0% |
| CE-55 | 134.5 pcf | 6.5% |

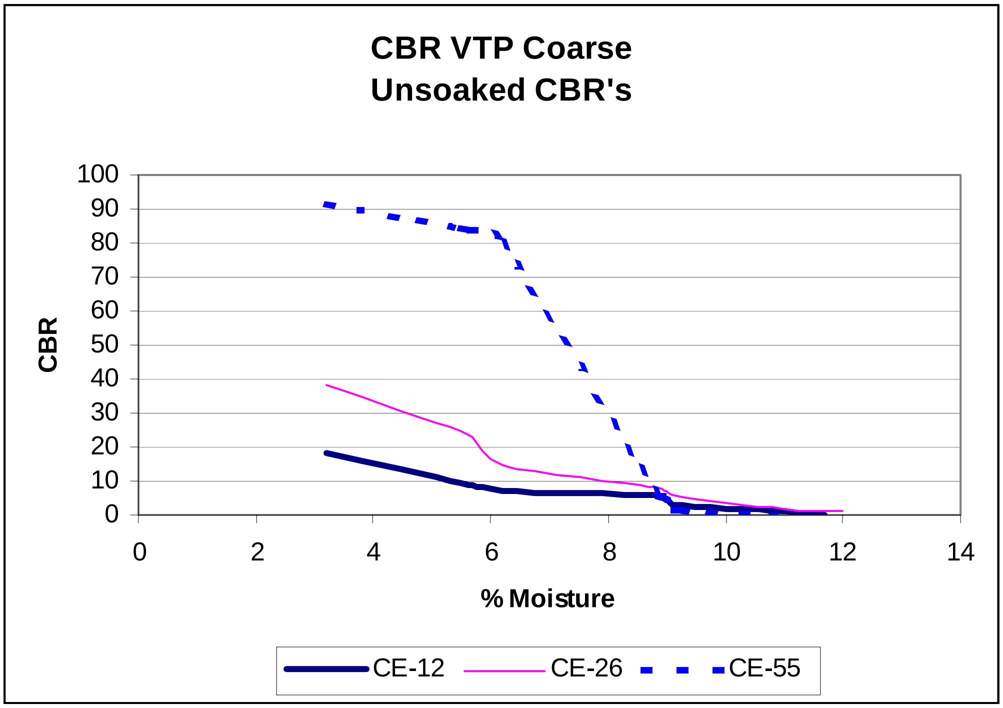
| Fine-Grained Soil | | |
|---|---|---|
| Compaction Effort | Maximum Dry Density | Optimum Moisture |
| CE-12 | 107.5 pcf | 17.5% |
| CE-26 | 109.5 pcf | 16.5% |
| CE-55 | 115.5 pcf | 15.5% |

CBR testing was done on both soils. Both unsoaked and soaked CBR's were done for all compaction efforts. Summary charts of CBR testing are below.

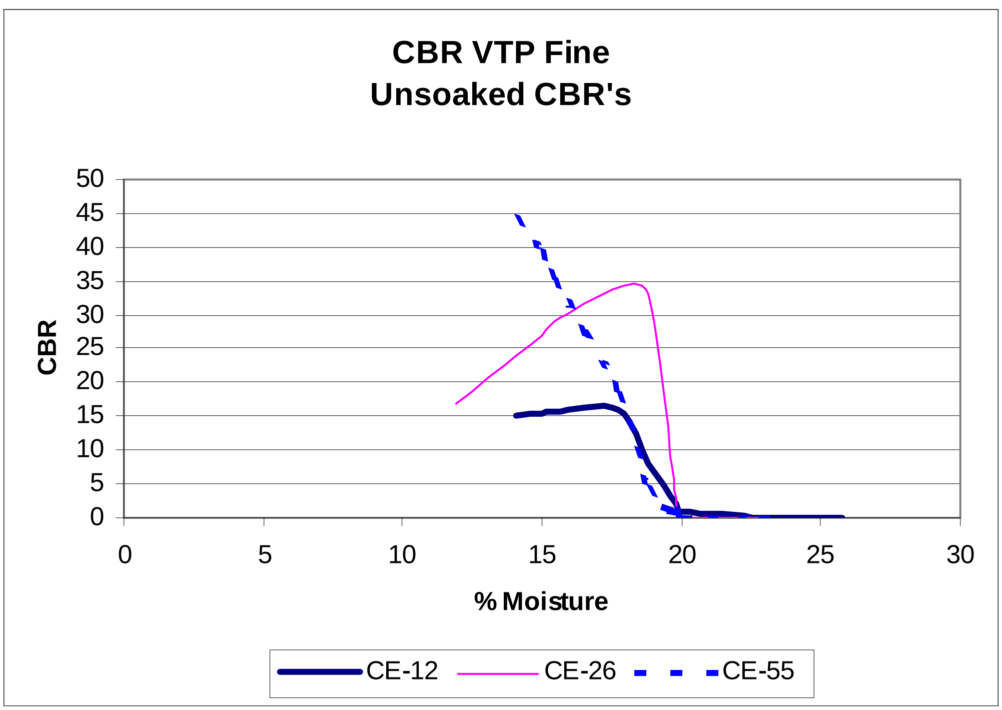
| Coarse-Grained Soil | | | | |
|---|---|---|---|---|
| Compaction Effort | Dry Density (pcf) | Moisture Content | CBR Unsoaked | CBR Soaked |
| CE-12 | 122.0 | 3.2 | 18 | 4 |

| | | | | |
|---|---|---|---|---|
| | 126.6 | 6.2 | 7 | 12 |
| | 128.0 | 8.8 | 6 | 6 |
| | 123.5 | 11.7 | <1 | 2 |
| CE-26 | 127.7 | 3.2 | 38 | 10 |
| | 130.2 | 5.5 | 25 | 35 |
| | 129.7 | 9.1 | 6 | 28 |
| | 123.6 | 12.0 | 1 | 1 |
| CE-55 | 122.4 | 3.2 | 92 | 30 |
| | 133.3 | 5.7 | 84 | 85 |
| | 131.2 | 9.0 | 5 | 5 |
| | 127.2 | 11.0 | <1 | 2 |

| Fine-Grained Soil | | | | |
|---|---|---|---|---|
| Compaction Effort | Dry Density (pcf) | Moisture Content | CBR Unsoaked | CBR Soaked |
| CE-12 | 103.5 | 14.1 | 15 | 1 |
| | 107.4 | 17.7 | 16 | 1 |
| | 104.4 | 20.1 | 1 | <1 |
| | 97.4 | 25.8 | <1 | <1 |
| CE-26 | 103.4 | 11.9 | 17 | |
| | 109.0 | 15.4 | 29 | 2 |
| | 107.2 | 18.8 | 33 | 1 |
| | 104.5 | 19.9 | <1 | <1 |
| | 99.7 | 22.8 | <1 | <1 |
| CE-55 | 114.3 | 14.1 | 44 | 1 |
| | 115.4 | 15.0 | 39 | 1 |
| | 104.5 | 19.8 | 1 | <1 |
| | 99.8 | 23.4 | <1 | <1 |




The above graph shows the relationship between the moisture content and the unsoaked CBR values of the coarse-grained soil.

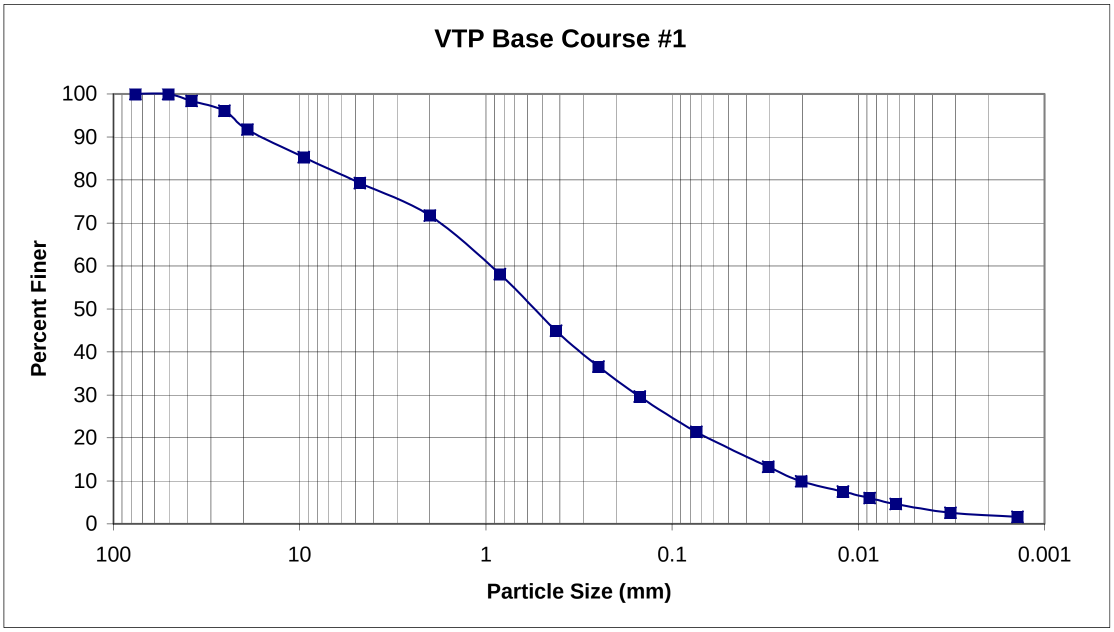


The above graph shows the relationship between the moisture content and the unsoaked CBR values of the fine-grained soil.

No unusual problems occurred during testing.  Copies of the testing data and results are attached. The original copies are maintained in the Civil Engineering Materials Lab.

SHERRI A. ORCHINO
Civil Engineering Technician

**PARTICLE SIZE ANALYSIS**




**Mechanical Analysis Data**

| Seive opening in | Seive opening mm | Percent Passing |
|---|---|---|
| 3 | 76.2 | 100 |
| 2 | 50.8 | 100 |
| 1.5 | 38.1 | 98.4 |
| 1 | 25.4 | 96.1 |
| 0.75 | 19.1 | 91.7 |
| 0.375 | 9.52 | 85.3 |
| # 4 | 4.76 | 79.3 |
| # 10 | 2 | 71.7 |
| # 20 | 0.84 | 58.1 |
| # 40 | 0.42 | 44.9 |
| # 60 | 0.249 | 36.6 |
| # 100 | 0.149 | 29.6 |
| # 200 | 0.074 | 21.4 |
| Hydrometer | 0.0305 | 13.3 |
| | 0.0203 | 9.9 |
| | 0.0121 | 7.5 |
| | 0.0087 | 6 |
| | 0.0063 | 4.6 |
| | 0.0032 | 2.6 |
| | 0.0014 | 1.6 |

**Sample Data**

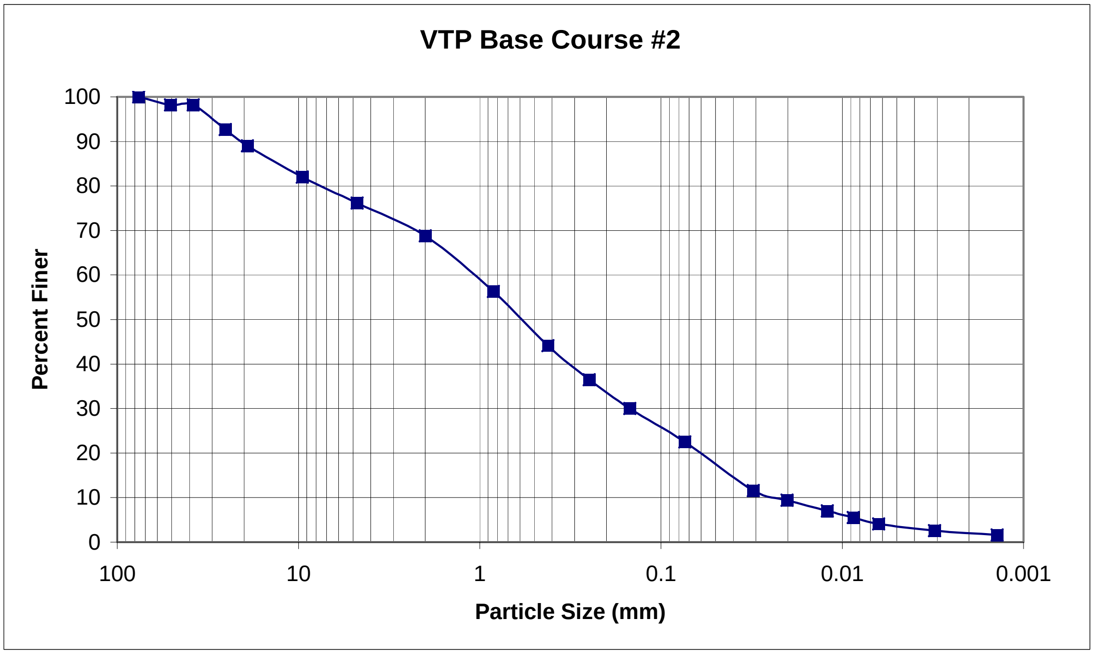
Location: VTP Base Course #1
Elev or Depth:
Description: Silty Sand with Gravel
USCS Classification: SM
AASHTO Classification:A-1-b
Plastic limit: 0
Liquid limit:0
Plastic index: NP
Remarks:

**Fractional Components**

Gravel/sand based on: # 4
Sand/fines based on:# 200
% Cobbels = 0.0
% Gravel = 20.7
% Sand = 57.9
% fines = 21.4 (silt =17.6 clay =3.8)

| | |
|---|---|
| D85 = 9.16 | D60 = 0.94 |
| D50 = 0.56 | D30 = 0.15 |
| D15 = 0.04 | D10 = 0.02 |
| Cc = 1.2359 | Cu = 45.72 |

## PARTICLE SIZE ANALYSIS




**Mechanical Analysis Data**

| Seive opening in | Seive opening mm | Percent Passing |
|---|---|---|
| 3 | 76.2 | 100 |
| 2 | 50.8 | 98.2 |
| 1.5 | 38.1 | 98.2 |
| 1 | 25.4 | 92.7 |
| 0.75 | 19.1 | 89 |
| 0.375 | 9.52 | 82 |
| # 4 | 4.76 | 76.2 |
| # 10 | 2 | 68.8 |
| # 20 | 0.84 | 56.3 |
| # 40 | 0.42 | 44.1 |
| # 60 | 0.249 | 36.5 |
| # 100 | 0.149 | 30 |
| # 200 | 0.074 | 22.5 |
| Hydrometer | 0.031 | 11.6 |
| | 0.0202 | 9.4 |
| | 0.0121 | 7 |
| | 0.0087 | 5.5 |
| | 0.0063 | 4.1 |
| | 0.0031 | 2.6 |
| | 0.0014 | 1.6 |

**Sample Data**

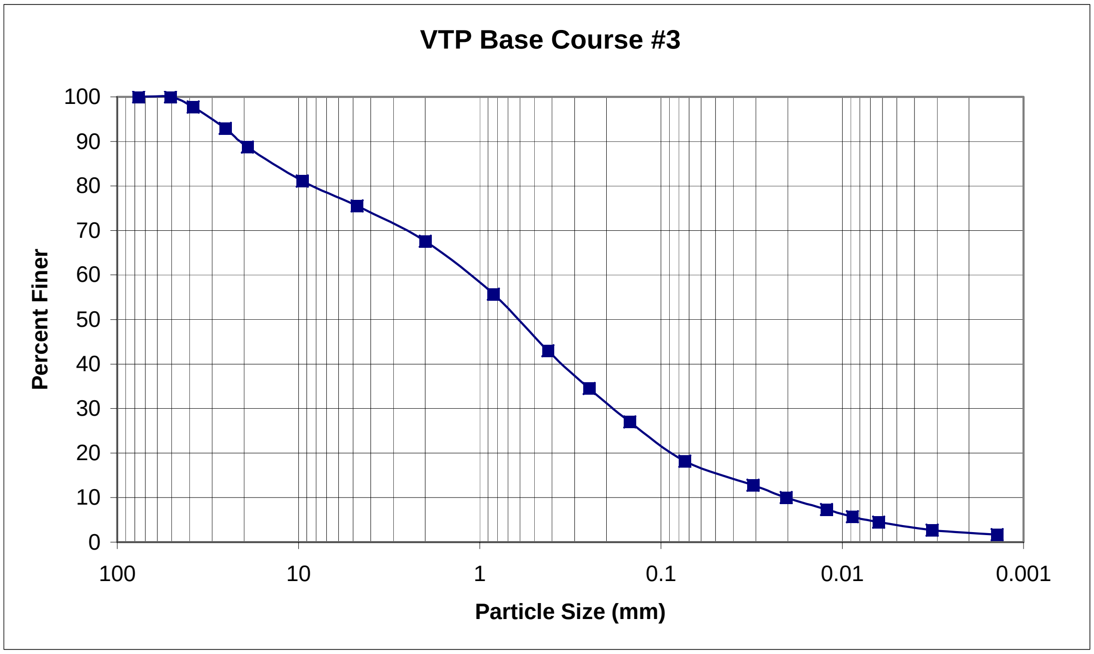
Location: VTP Base Course #2
Elev or Depth:
Description: Silty Sand with Gravel
USCS Classification: SM
AASHTO Classification:A-1-b
Plastic limit: 0
Liquid limit:0
Plastic index: NP
Remarks:

**Fractional Components**

Gravel/sand based on: # 4
Sand/fines based on:# 200
% Cobbels = 0.0
% Gravel = 23.8
% Sand = 53.7
% fines = 22.5 (silt =19.1 clay =3.4)

| | |
|---|---|
| D85 = 13.17 | D60 = 1.06 |
| D50 = 0.60 | D30 = 0.15 |
| D15 = 0.04 | D10 = 0.02 |
| Cc = 0.9084 | Cu = 45.2299 |

## PARTICLE SIZE ANALYSIS




**Mechanical Analysis Data**

| Seive opening in | Seive opening mm | Percent Passing |
|---|---|---|
| 3 | 76.2 | 100 |
| 2 | 50.8 | 100 |
| 1.5 | 38.1 | 97.7 |
| 1 | 25.4 | 92.9 |
| 0.75 | 19.1 | 88.7 |
| 0.375 | 9.52 | 81.1 |
| # 4 | 4.76 | 75.5 |
| # 10 | 2 | 67.6 |
| # 20 | 0.84 | 55.7 |
| # 40 | 0.42 | 43 |
| # 60 | 0.249 | 34.5 |
| # 100 | 0.149 | 27 |
| # 200 | 0.074 | 18.2 |
| Hydrometer | 0.0311 | 12.8 |
| | 0.0204 | 10 |
| | 0.0122 | 7.3 |
| | 0.0088 | 5.7 |
| | 0.0063 | 4.5 |
| | 0.0032 | 2.7 |
| | 0.0014 | 1.7 |

**Sample Data**

Location: VTP Base Course #3
Elev or Depth:
Description: Silty Sand with Gravel
USCS Classification: SM
AASHTO Classification:A-1-b
Plastic limit: 0
Liquid limit:0
Plastic index: NP
Remarks:

**Fractional Components**

Gravel/sand based on: # 4
Sand/fines based on:# 200
% Cobbels = 0.0
% Gravel = 24.5
% Sand = 57.3
% fines = 18.2 (silt =14.4 clay =3.8)

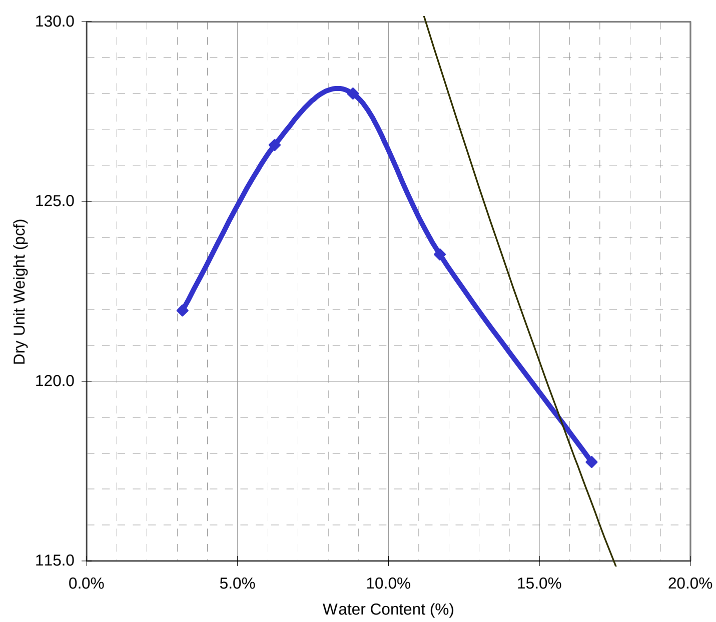
| | |
|---|---|
| D85 = 14.17 | D60 = 1.11 |
| D50 = 0.62 | D30 = 0.18 |
| D15 = 0.05 | D10 = 0.02 |
| Cc = 1.4999 | Cu = 54.368 |

**SPECIFIC GRAVITY**

| | | | |
|---|---|---|---|
| **Project:** | **VTP** | **Date:** | **4-May-01** |
| **Remarks:** | **Coarse Aggregate** | **Technician:** | **S. Orchino** |

| | | | | | | |
|---|---|---|---|---|---|---|
| **Sample Number** | 1 | 2 | 3 | | | |
| **Flask** | S-5 | SPG-5 | P-5 | | | |
| **Temp. of water and soil T** | 19.5 | 20 | 20.5 | | | |
| **Dish No.** | 1 | 2 | 3 | | | |
| **Dish + Dry Soil (g)** | 142.75 | 144.18 | 137.69 | | | |
| **Dish (g)** | 0 | 0 | 0 | | | |
| **Dry Soil (g) Ws** | 142.75 | 144.18 | 137.69 | 0 | 0 | 0 |
| **Flask + water at T Wbw** | 696.1 | 680.11 | 681.15 | | | |
| **Ws + Wbw** | 838.85 | 824.29 | 818.84 | 0 | 0 | 0 |
| **Flask + water + soil Wbws** | 786.16 | 771.64 | 767.87 | | | |
| **Displaced Water Ws+Wbw-Wbws** | 52.69 | 52.65 | 50.97 | 0 | 0 | 0 |
| **Correction Factor K** | 1 | 1 | 0.9998 | | | |
| **(WsK)/(Ws+Wbw-Wbws) Gs** | 2.71 | 2.74 | 2.70 | #DIV/0! | #DIV/0! | #DIV/0! |

## COMPACTION TEST REPORT




**MATERIAL**
VTP Base Course

**Test Specification:**
MIL-STD-621A, Method A, CE-12

**Hammer Wt.:** 10 lbs
**Hammer Drop:** 18 in
**Number of Layers:** 5
**Blows per Layer:** 12
**Mold Size:** .075 cu.ft.

**Test Performed on Material**
**Passing** 3/4 in. **Sieve**

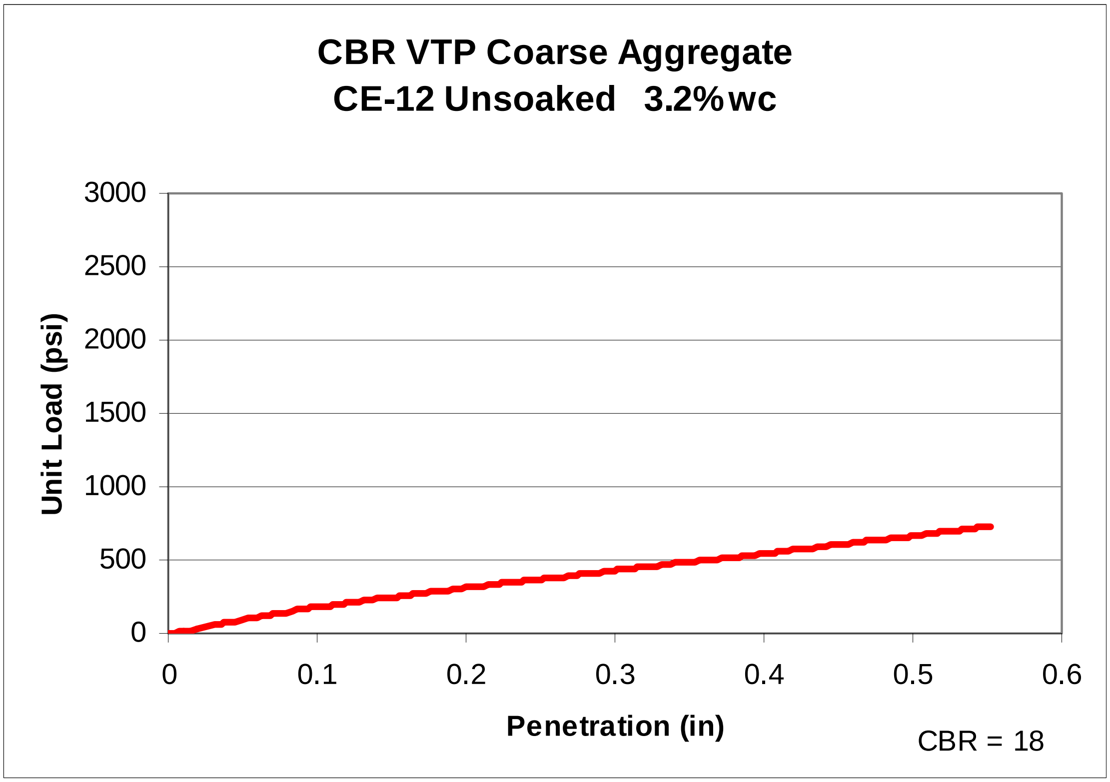
**SOIL DATA**
**SpG:** 2.72
**LL:** 0
**PI:** 0
**% >3/4 in.:** 11
**% <#200:** 20

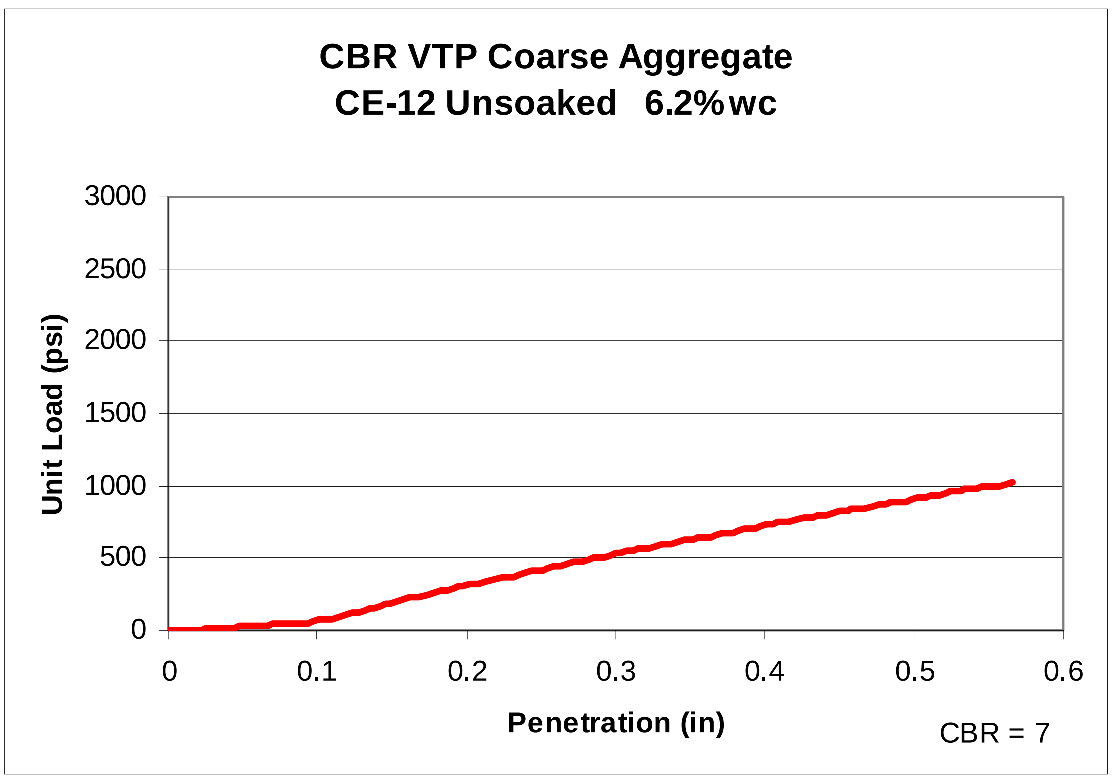
**CLASSIFICATION**
**USCS:** SM
**AASHTO:** A-1-b

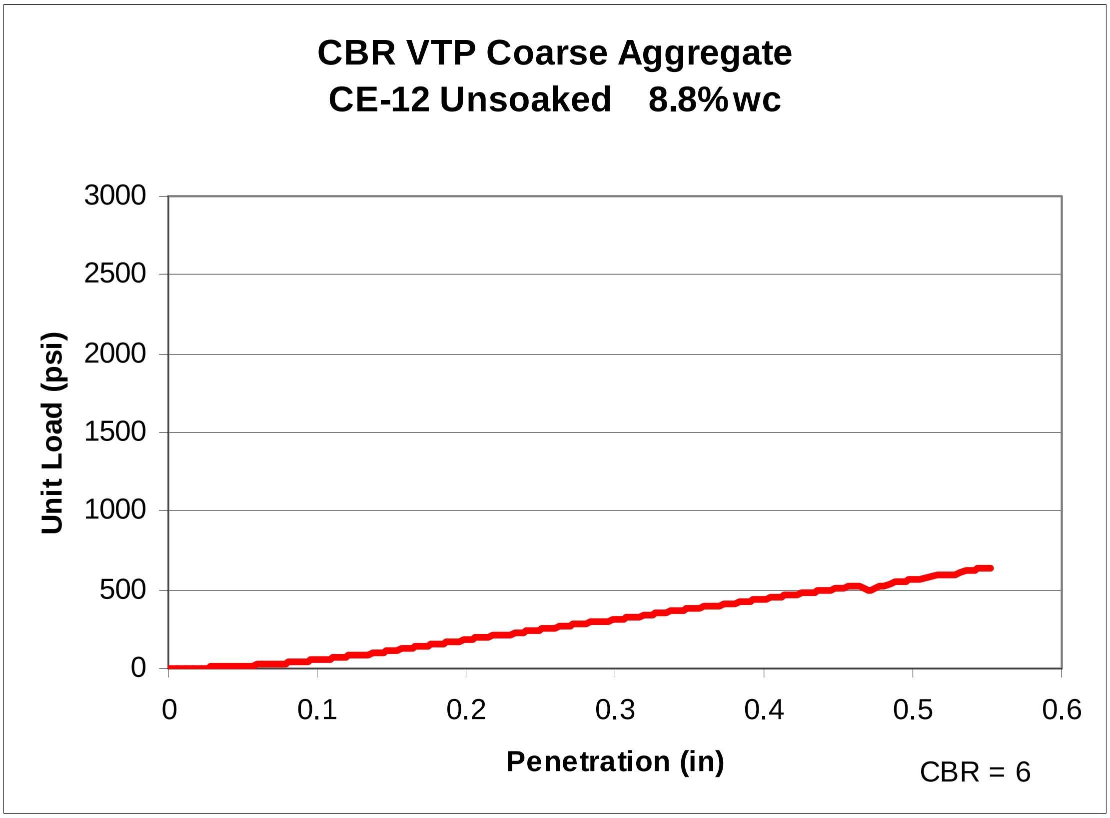
| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| **Wet Soil + Mold** | 20038.00 | 20254.00 | 20491.00 | 20453.00 | 20357.00 |
| **Mold** | 15757.00 | 15680.00 | 15752.00 | 15759.00 | 15681.00 |
| **Wet Density** | 125.84 | 134.45 | 139.30 | 137.98 | 137.45 |
| **Wet Soil + Tare 1** | 282.63 | 299.45 | 250.72 | 346.59 | 505.60 |
| **Dry Soil + Tare 1** | 273.97 | 282.62 | 231.08 | 314.81 | 431.62 |
| **Tare 1** | 7.22 | 7.13 | 7.18 | 7.19 | 7.13 |
| **Moisture 1** | 3.25% | 6.11% | 8.77% | 10.33% | 17.43% |
| **Wet Soil + Tare 2** | 306.40 | 281.54 | 289.27 | 429.19 | 577.93 |
| **Dry Soil + Tare 2** | 297.39 | 265.18 | 266.27 | 380.43 | 499.09 |
| **Tare 2** | 7.13 | 7.13 | 7.18 | 7.15 | 7.17 |
| **Moisture 2** | 3.10% | 6.34% | 8.88% | 13.06% | 16.03% |
| **Moisture** | 3.2% | 6.2% | 8.8% | 11.7% | 16.7% |
| **Dry Density** | 122.0 | 126.6 | 128.0 | 123.5 | 117.8 |

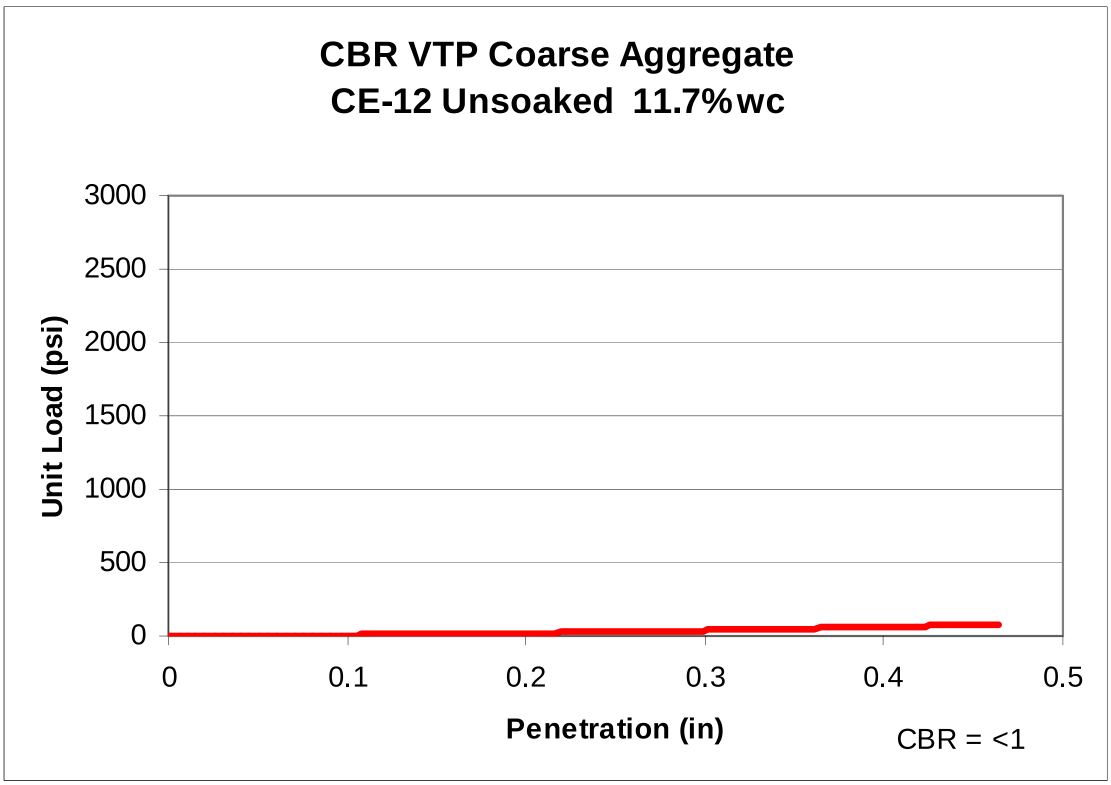
**MAXIMUM DRY DENSITY =** **128 pcf**

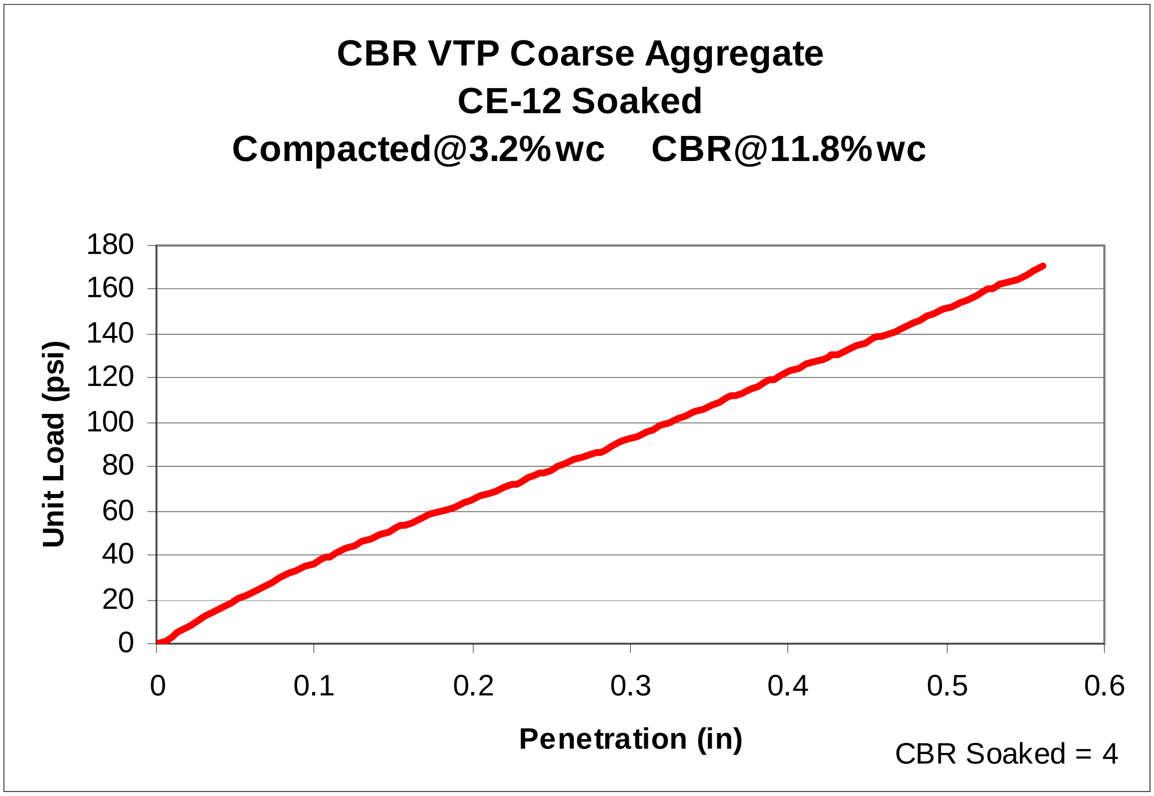
**OPTIMUM MOISTURE =** **8.50%**

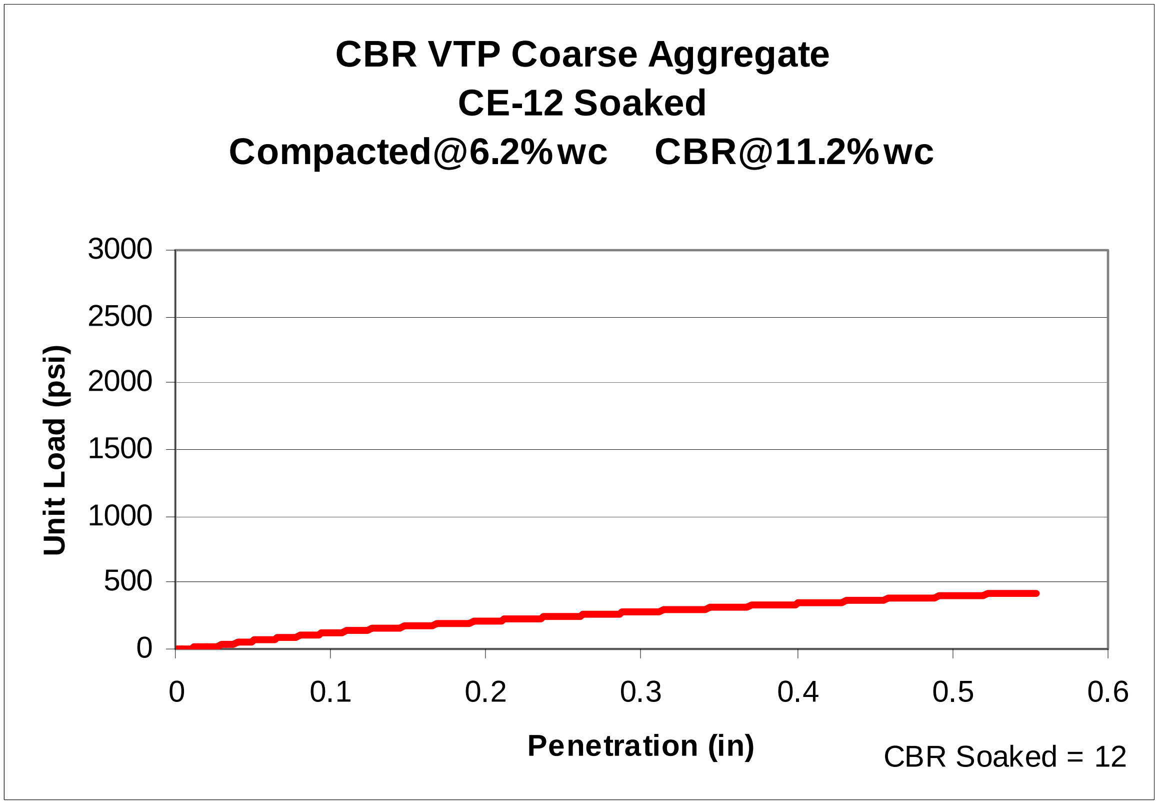

CBR VTP Coarse Aggregate
CE-12 Unsoaked 3.2% wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 18

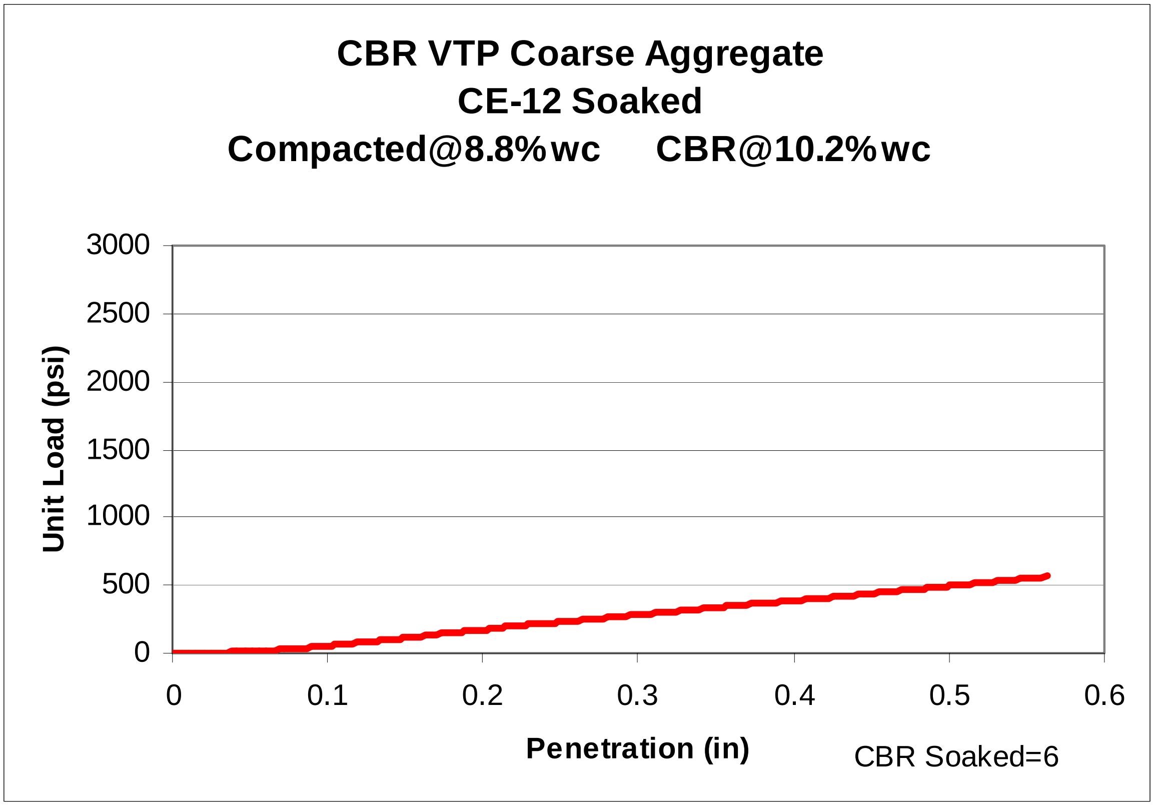

CBR VTP Coarse Aggregate
CE-12 Unsoaked 6.2% wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 7

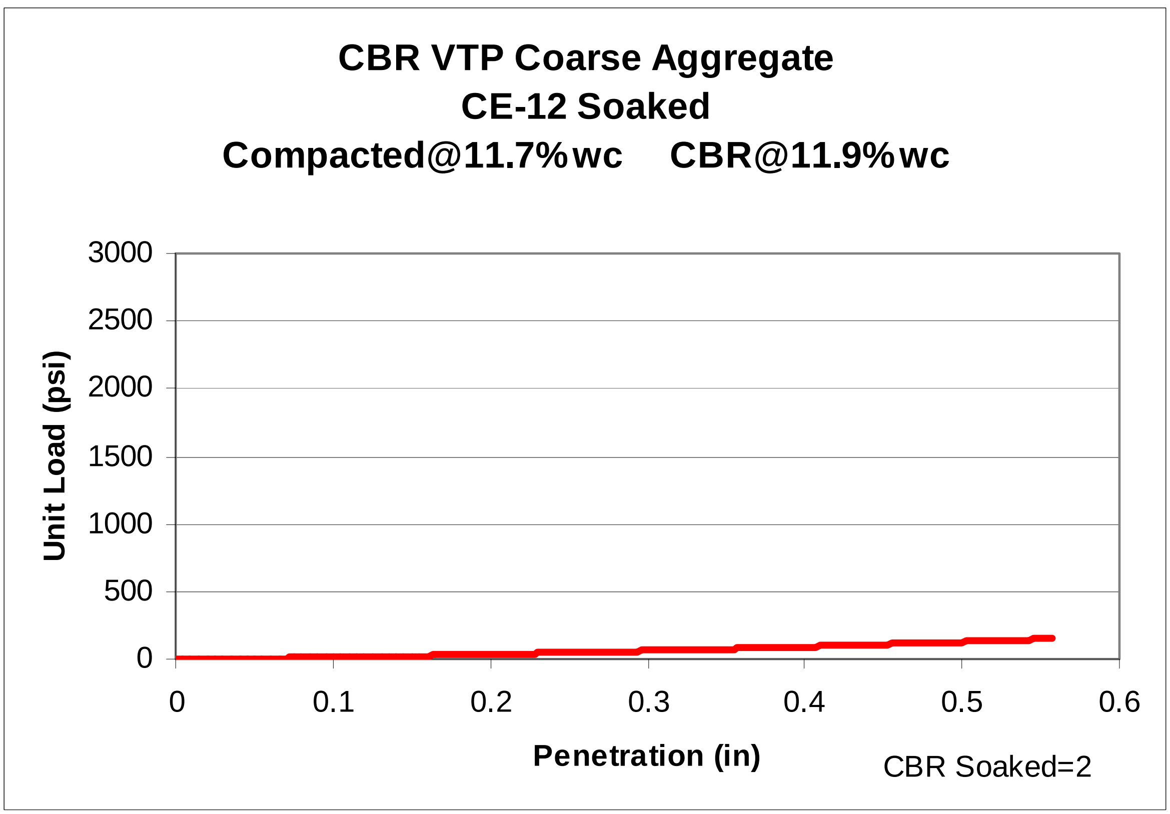

CBR VTP Coarse Aggregate
CE-12 Unsoaked 8.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 6


CBR VTP Coarse Aggregate
CE-12 Unsoaked 11.7%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
Penetration (in)
CBR = <1

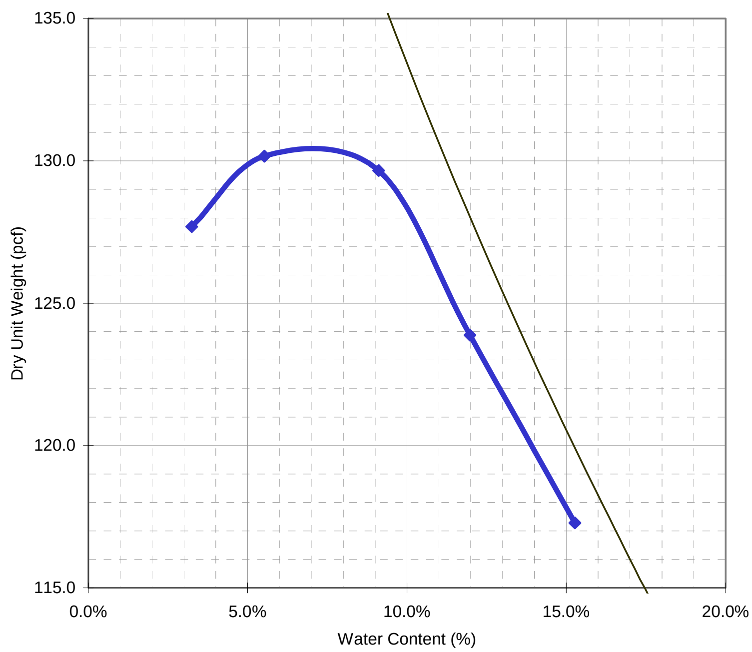

CBR VTP Coarse Aggregate
CE-12 Soaked
Compacted@3.2%wc CBR@11.8%wc
Unit Load (psi)
180
160
140
120
100
80
60
40
20
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 4


CBR VTP Coarse Aggregate
CE-12 Soaked
Compacted@6.2%wc CBR@11.2%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 12


CBR VTP Coarse Aggregate
CE-12 Soaked
Compacted@8.8%wc CBR@10.2%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked=6


CBR VTP Coarse Aggregate
CE-12 Soaked
Compacted@11.7%wc CBR@11.9%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked=2

## COMPACTION TEST REPORT




**MATERIAL**
VTP Base Course

**Test Specification:**
MIL-STD-621A, Method 100, CE-26

**Hammer Wt.:** 10 lbs
**Hammer Drop:** 18 in
**Number of Layers:** 5
**Blows per Layer:** 26
**Mold Size:** .075 cu.ft.

**Test Performed on Material**
**Passing** 3/4 in. **Sieve**

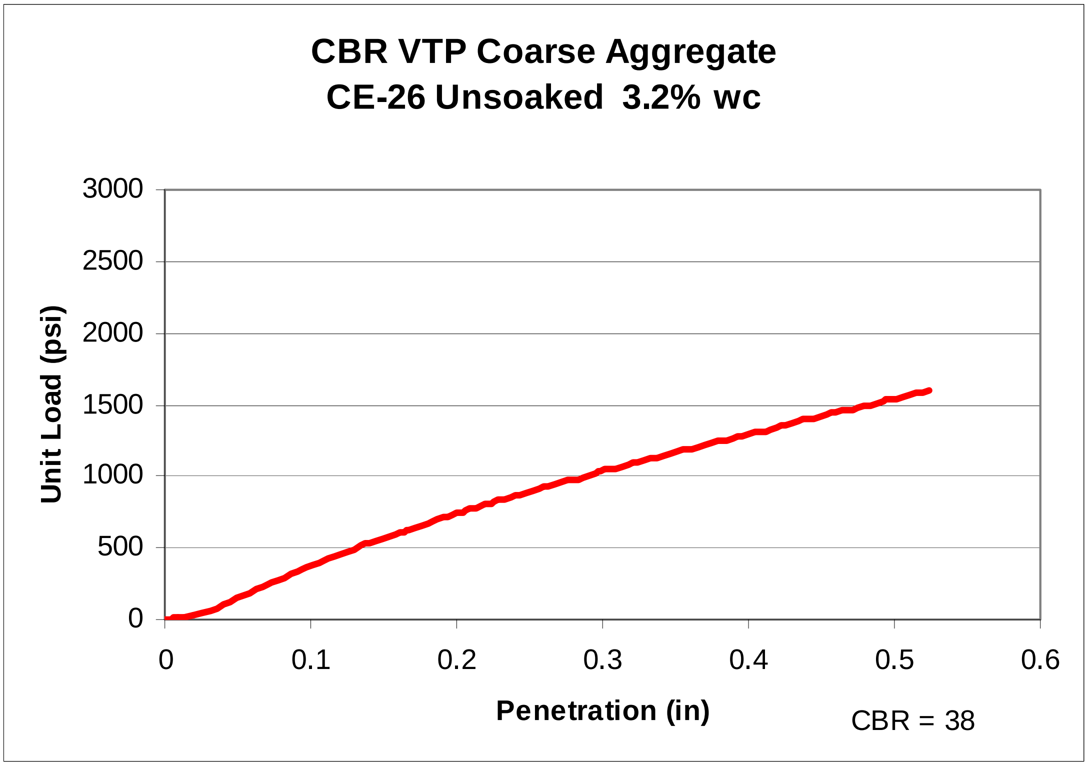
**SOIL DATA**
**SpG:** 2.72
**LL:** 0
**PI:** 0
**% >3/4 in.:** 11
**% <#200:** 20

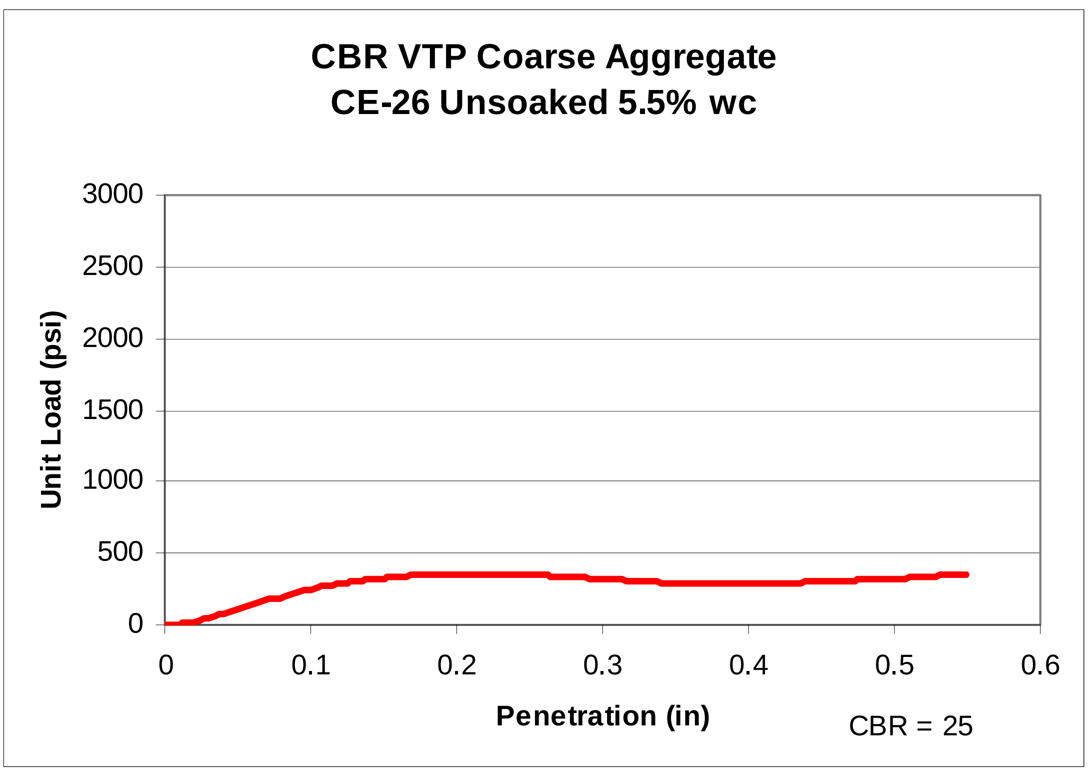
**CLASSIFICATION**
**USCS:** SM
**AASHTO:** A-1-b

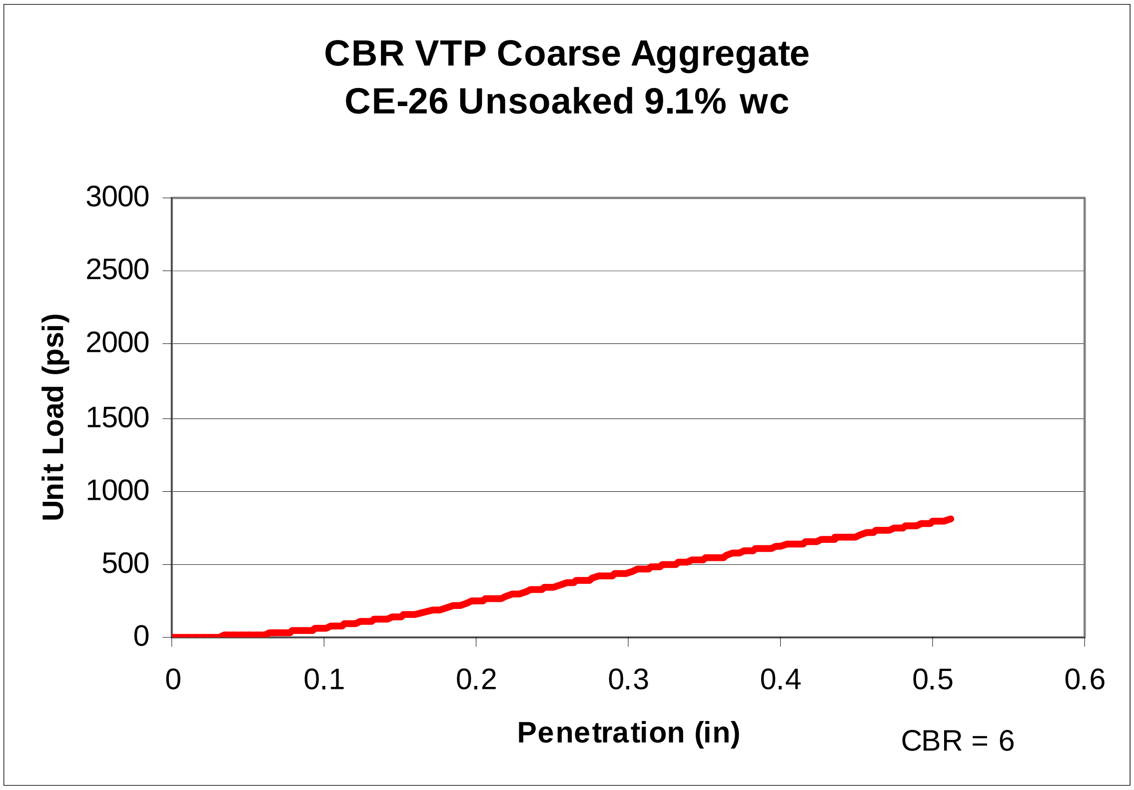
| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| **Wet Soil + Mold** | 20325.00 | 20515.00 | 20580.00 | 20562.00 | 20448.00 |
| **Mold** | 15840.00 | 15842.00 | 15767.00 | 15843.00 | 15849.00 |
| **Wet Density** | 131.83 | 137.36 | 141.48 | 138.71 | 135.19 |
| **Wet Soil + Tare 1** | 260.11 | 222.63 | 352.35 | 356.50 | 495.28 |
| **Dry Soil + Tare 1** | 252.17 | 211.52 | 324.72 | 324.11 | 426.94 |
| **Tare 1** | 7.18 | 7.11 | 7.23 | 7.20 | 7.24 |
| **Moisture 1** | 3.24% | 5.44% | 8.70% | 10.22% | 16.28% |
| **Wet Soil + Tare 2** | 276.49 | 301.60 | 367.79 | 421.26 | 566.59 |
| **Dry Soil + Tare 2** | 268.02 | 285.94 | 336.44 | 371.26 | 496.80 |
| **Tare 2** | 7.12 | 7.12 | 7.17 | 7.16 | 7.22 |
| **Moisture 2** | 3.25% | 5.62% | 9.52% | 13.73% | 14.26% |
| **Moisture** | 3.2% | 5.5% | 9.1% | 12.0% | 15.3% |
| **Dry Density** | 127.7 | 130.2 | 129.7 | 123.9 | 117.3 |

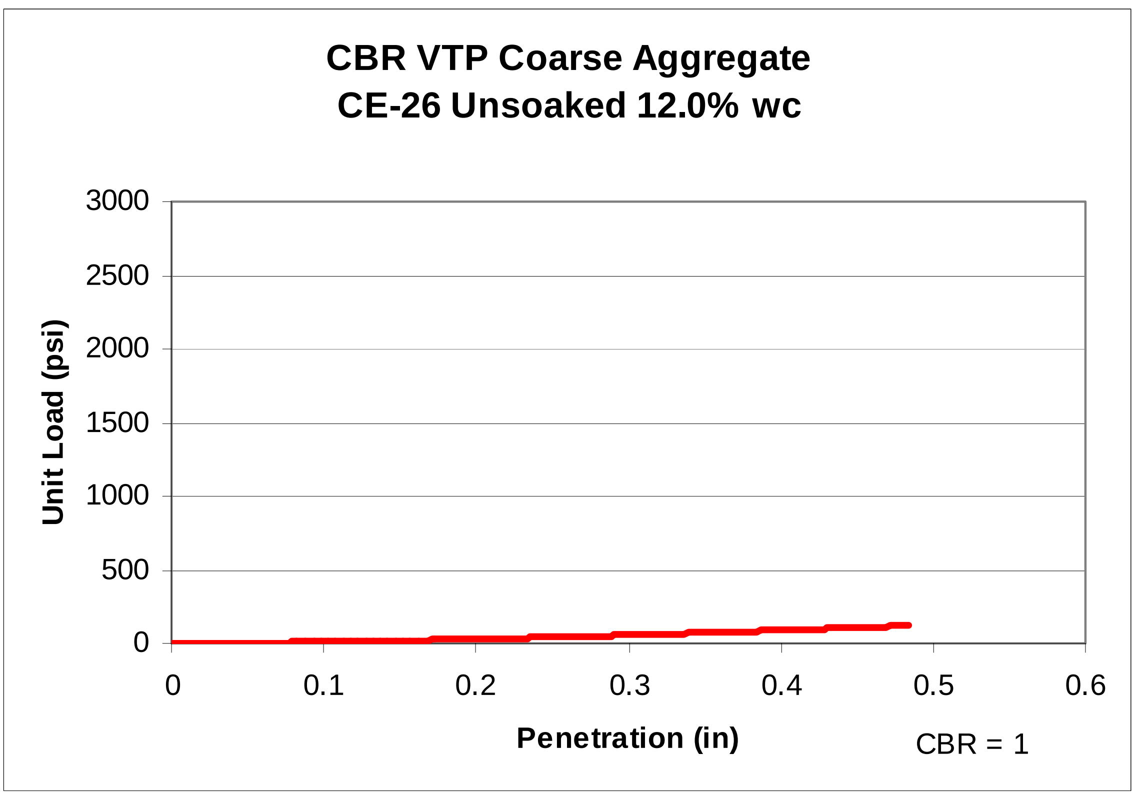
**MAXIMUM DRY DENSITY =** **131 pcf**

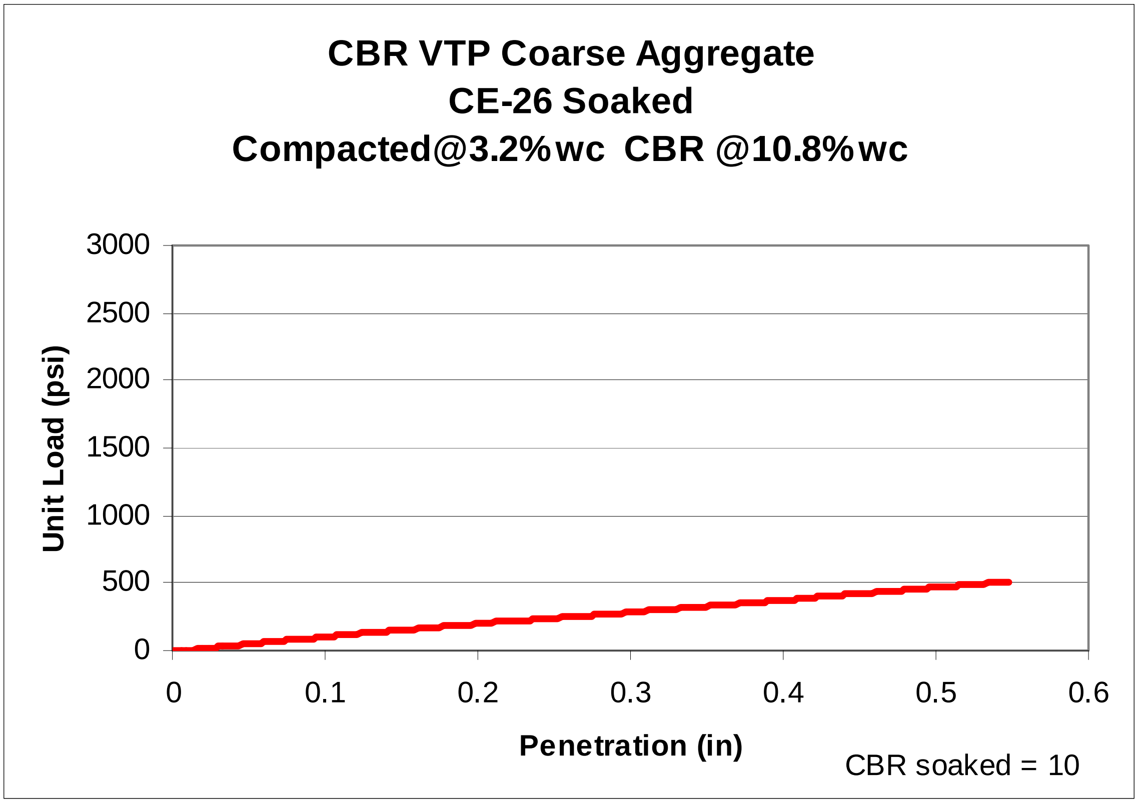
**OPTIMUM MOISTURE =** **7.00%**

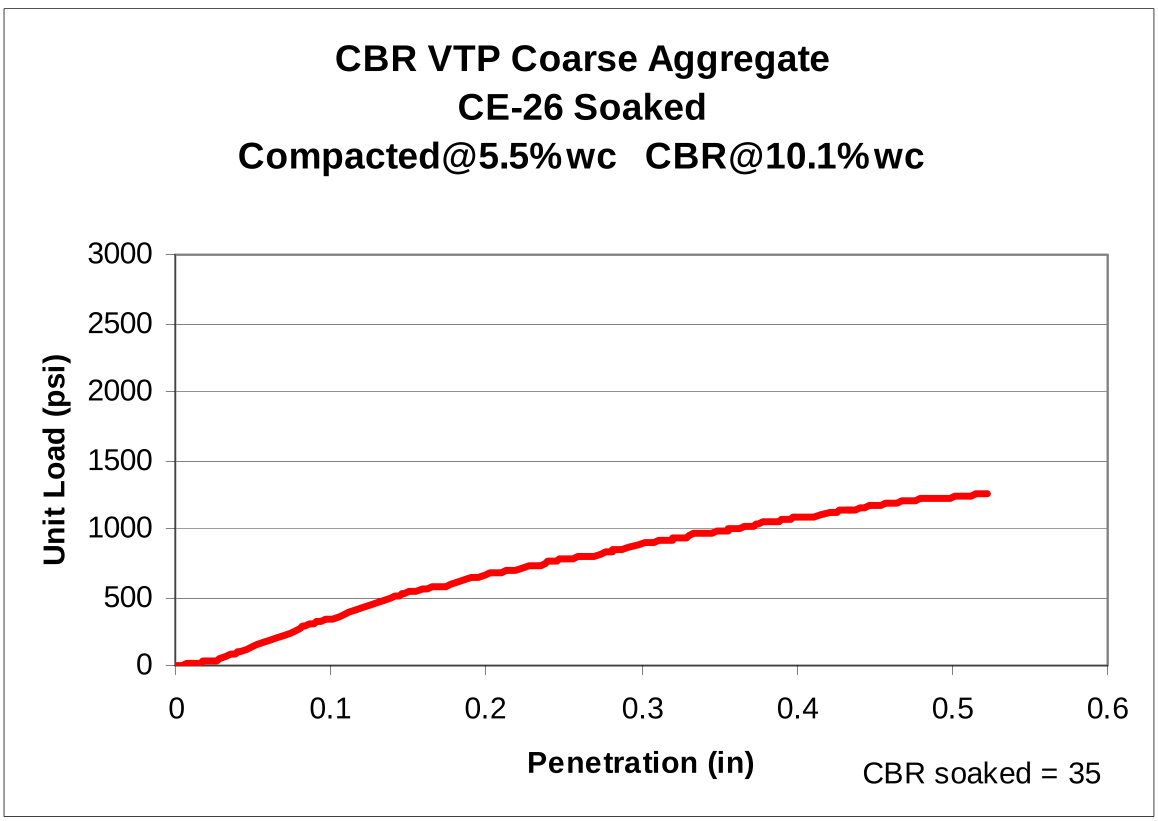

CBR VTP Coarse Aggregate
CE-26 Unsoaked 3.2% wc
3000
2500
2000
1500
1000
500
0
Unit Load (psi)
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 38

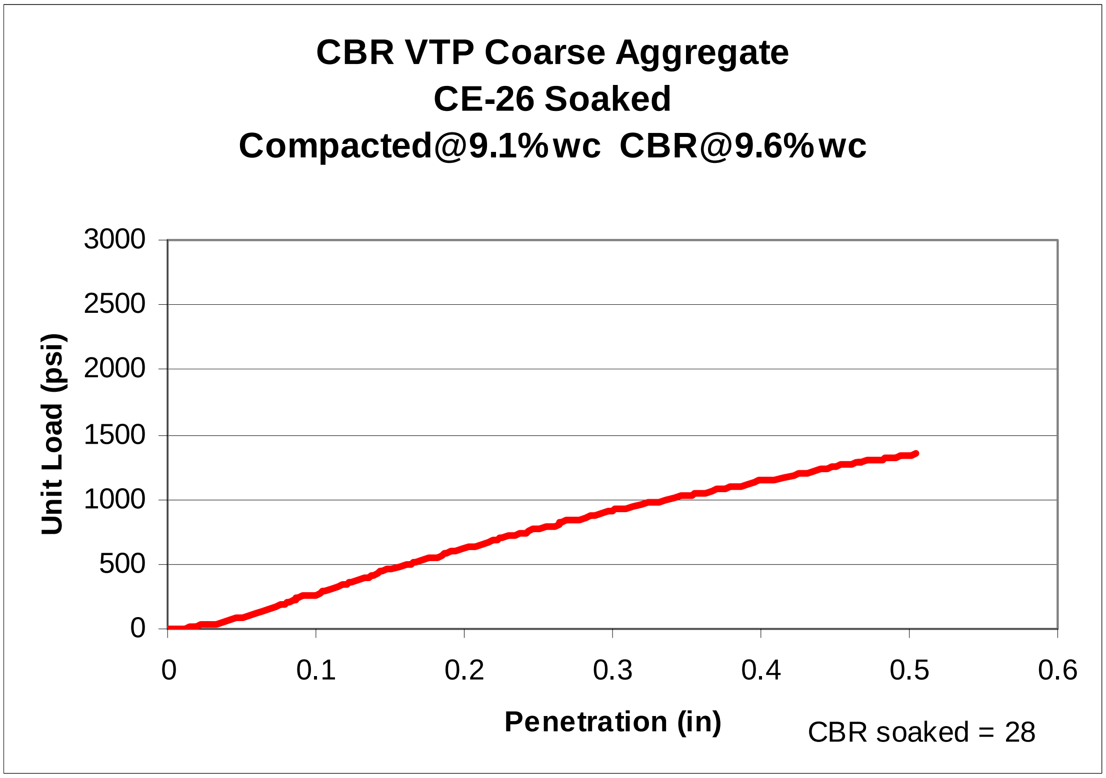

CBR VTP Coarse Aggregate
CE-26 Unsoaked 5.5% wc
3000
2500
2000
1500
1000
500
0
Unit Load (psi)
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 25

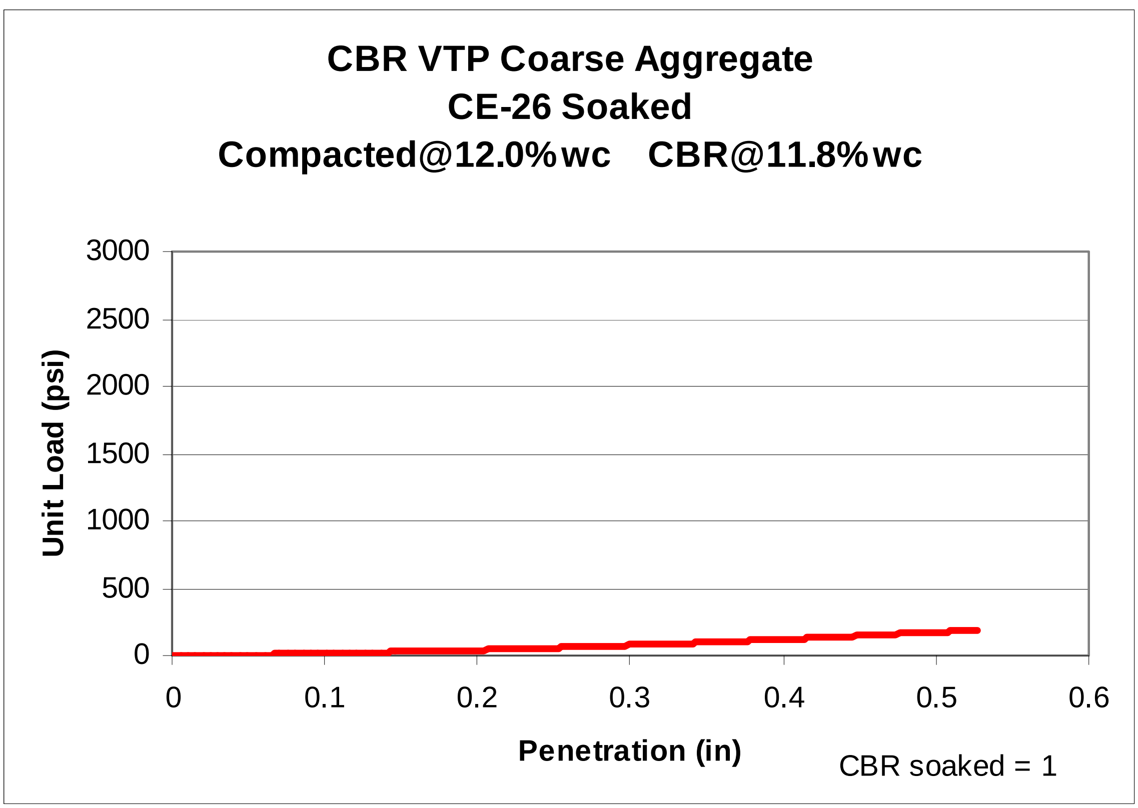

CBR VTP Coarse Aggregate
CE-26 Unsoaked 9.1% wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 6


CBR VTP Coarse Aggregate
CE-26 Unsoaked 12.0% wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 1

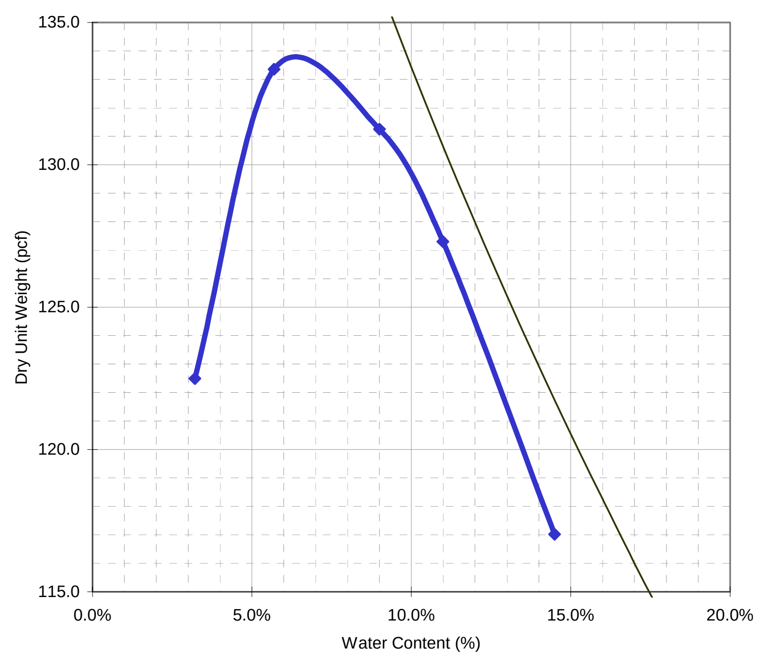

CBR VTP Coarse Aggregate
CE-26 Soaked
Compacted@3.2%wc CBR @10.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 10


CBR VTP Coarse Aggregate
CE-26 Soaked
Compacted@5.5%wc CBR@10.1%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 35


CBR VTP Coarse Aggregate
CE-26 Soaked
Compacted@9.1%wc CBR@9.6%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 28


CBR VTP Coarse Aggregate
CE-26 Soaked
Compacted@12.0%wc CBR@11.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 1

## COMPACTION TEST REPORT




**MATERIAL**
VTP Base Course

**Test Specification:**
MIL-STD-621A, Method 100A, CE-55

| | |
|---|---|
| **Hammer Wt.:** | 10 lbs |
| **Hammer Drop:** | 18 in |
| **Number of Layers:** | 5 |
| **Blows per Layer:** | 55 |
| **Mold Size:** | .075 cu.ft. |

**Test Performed on Material**
**Passing** 3/4 in. **Sieve**

**SOIL DATA**

| | |
|---|---|
| **SpG:** | 2.72 |
| **LL:** | 0 |
| **PI:** | 0 |
| **% >3/4 in.:** | 11 |
| **% <#200:** | 20 |

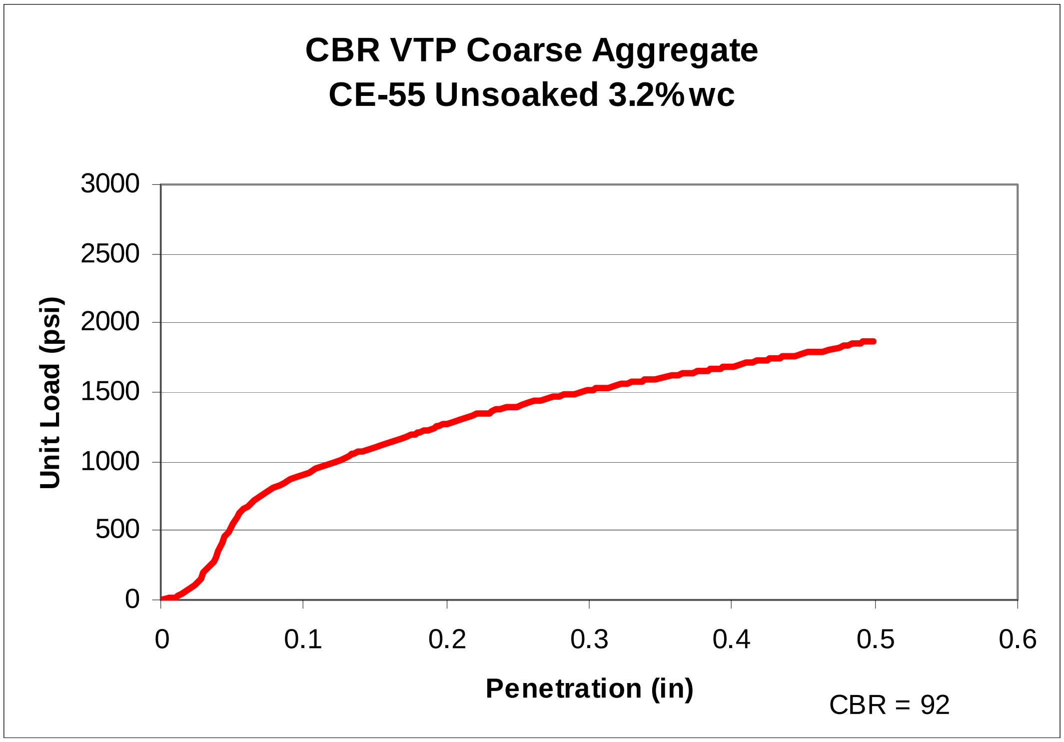
**CLASSIFICATION**

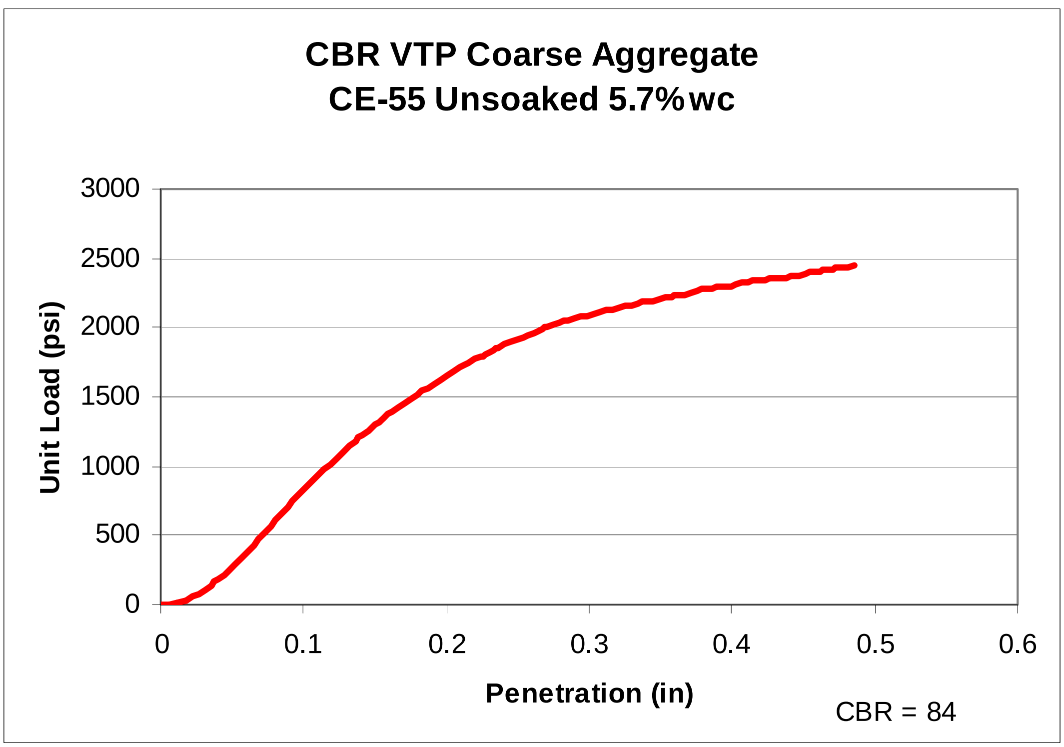
| | |
|---|---|
| **USCS:** | SM |
| **AASHTO:** | A-1-b |

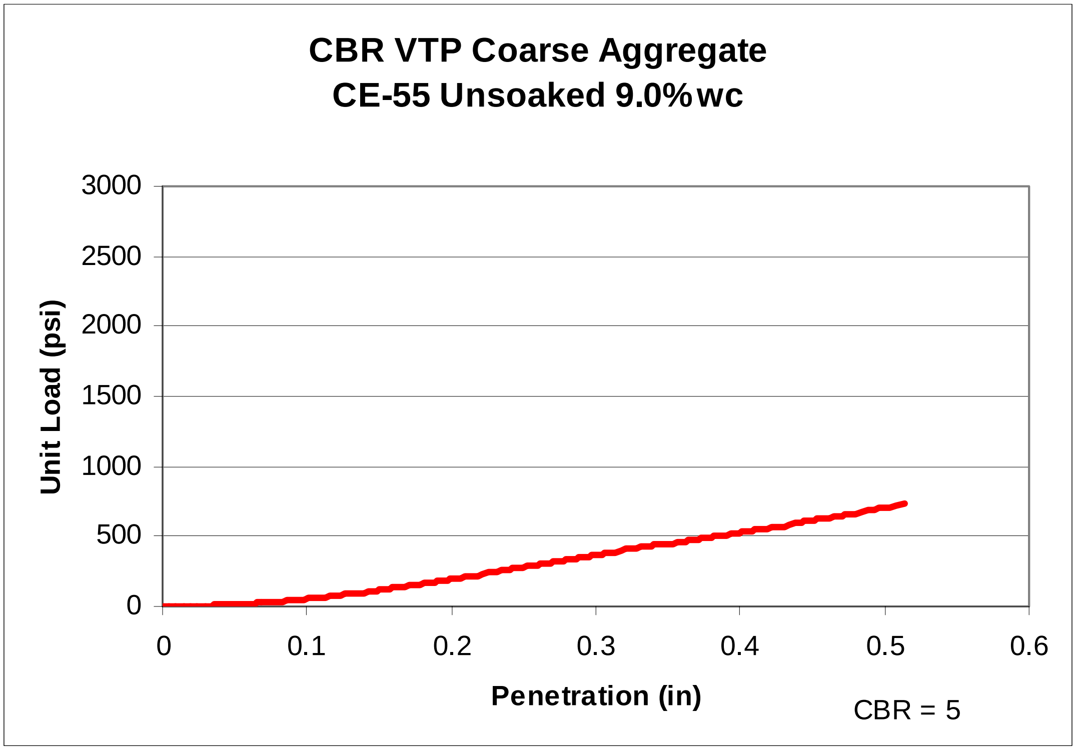
| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| **Wet Soil + Mold** | 20269.00 | 20570.00 | 20631.00 | 20507.00 | 20336.00 |
| **Mold** | 15968.00 | 15775.00 | 15764.00 | 15700.00 | 15778.00 |
| **Wet Density** | 126.43 | 140.95 | 143.06 | 141.30 | 133.98 |
| **Wet Soil + Tare 1** | 329.64 | 384.81 | 363.93 | 471.63 | 597.90 |
| **Dry Soil + Tare 1** | 319.66 | 364.37 | 334.40 | 419.94 | 527.01 |
| **Tare 1** | 9.03 | 9.07 | 9.04 | 9.07 | 9.03 |
| **Moisture 1** | 3.21% | 5.75% | 9.08% | 12.58% | 13.69% |
| **Wet Soil + Tare 2** | 331.47 | 298.02 | 435.21 | 422.46 | 635.55 |
| **Dry Soil + Tare 2** | 321.44 | 282.60 | 400.30 | 386.92 | 552.40 |
| **Tare 2** | 9.06 | 9.07 | 9.06 | 9.10 | 8.98 |
| **Moisture 2** | 3.21% | 5.64% | 8.92% | 9.41% | 15.30% |
| **Moisture** | 3.2% | 5.7% | 9.0% | 11.0% | 14.5% |
| **Dry Density** | 122.5 | 133.4 | 131.3 | 127.3 | 117.0 |

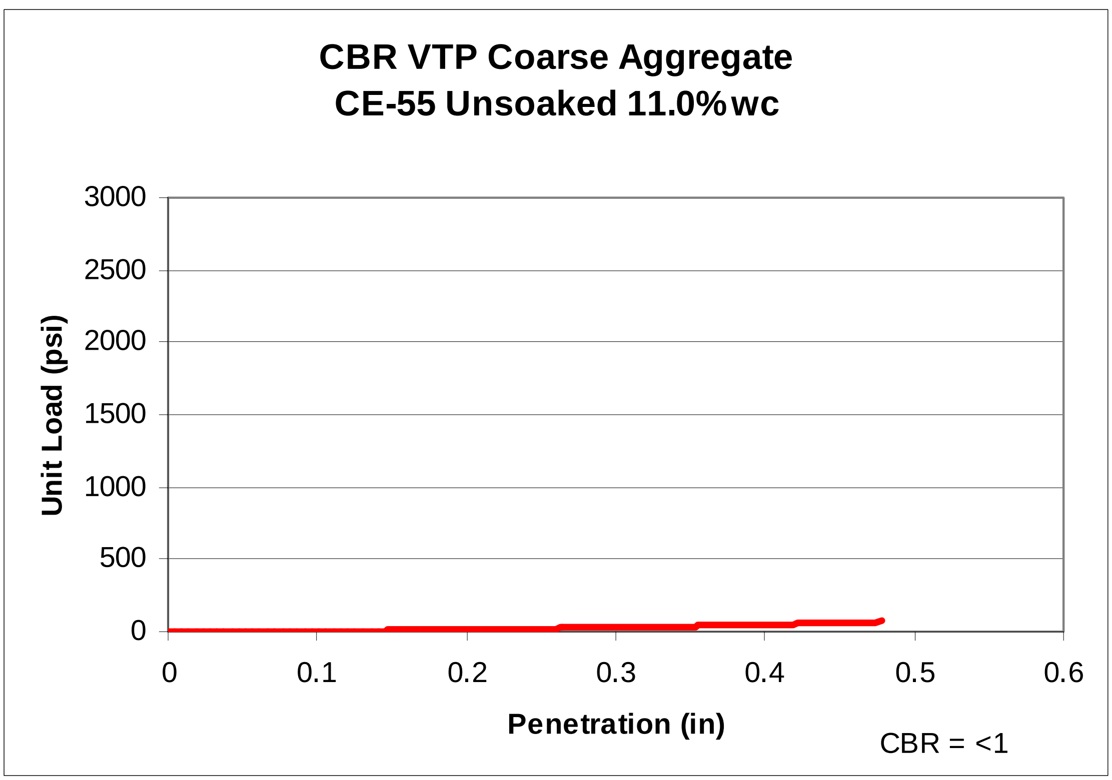
**MAXIMUM DRY DENSITY =** **134.5 pcf**

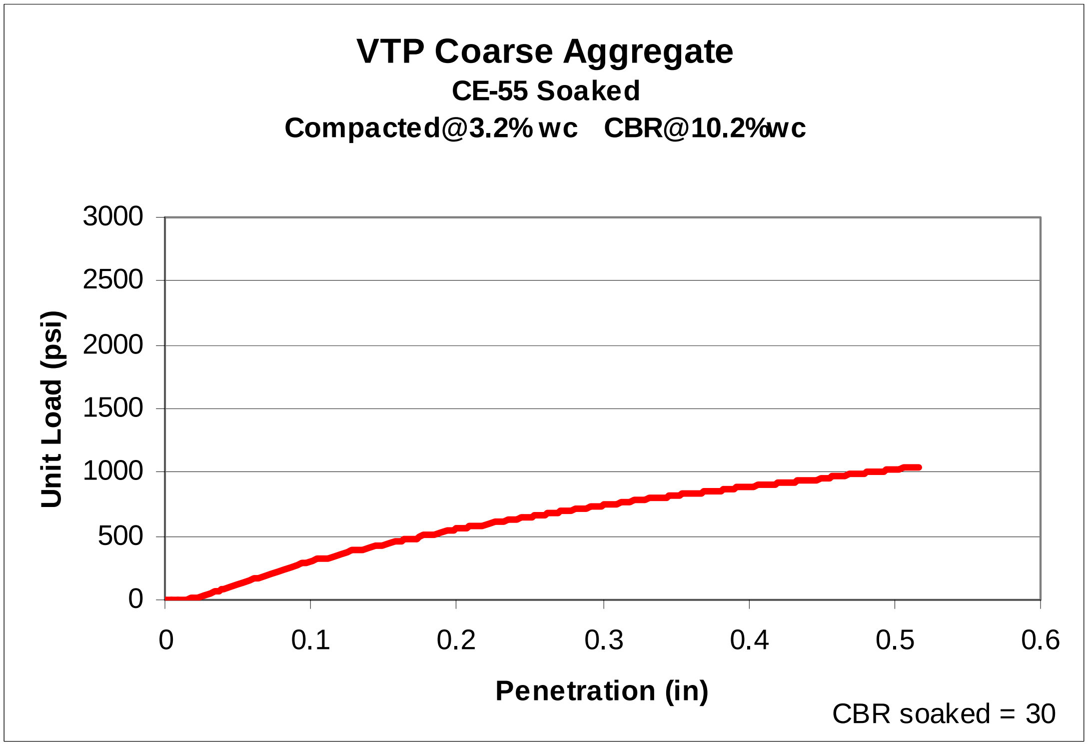
**OPTIMUM MOISTURE =** **6.50%**

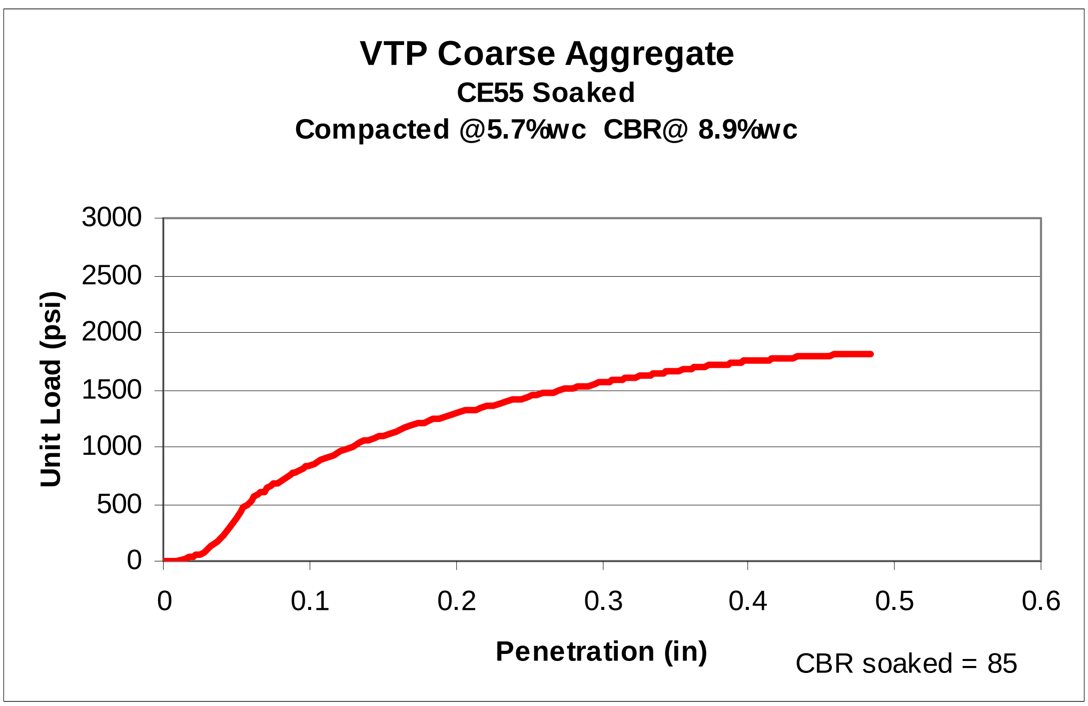

CBR VTP Coarse Aggregate
CE-55 Unsoaked 3.2%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 92

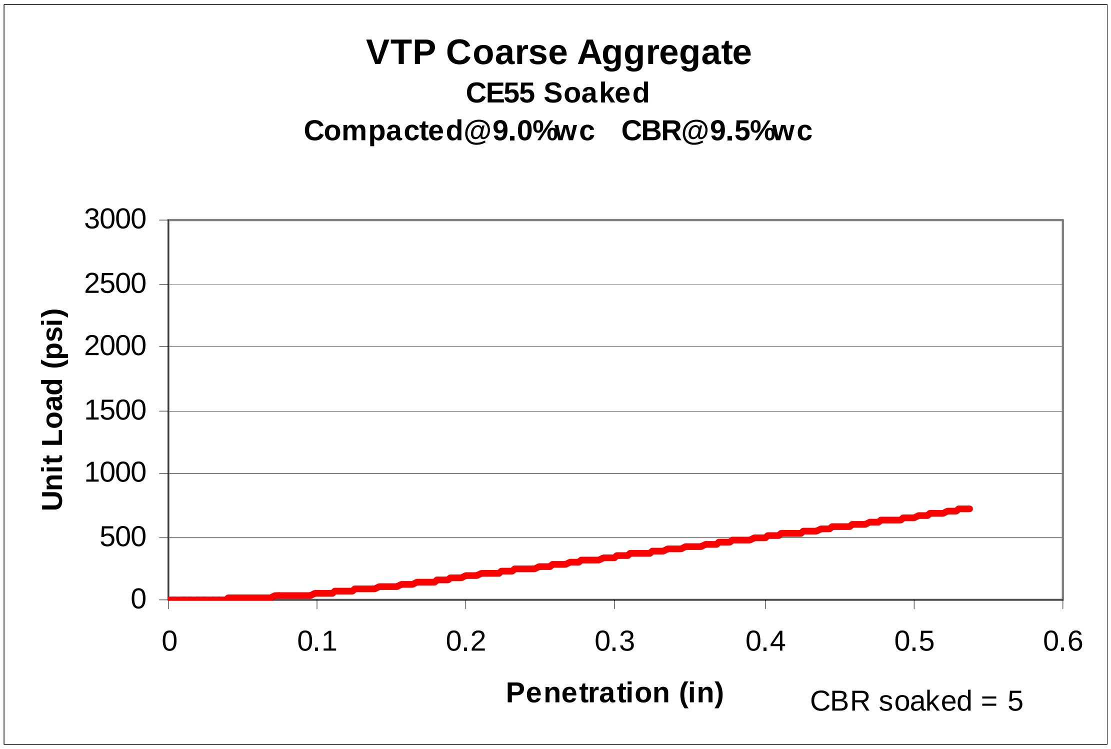

CBR VTP Coarse Aggregate
CE-55 Unsoaked 5.7%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 84

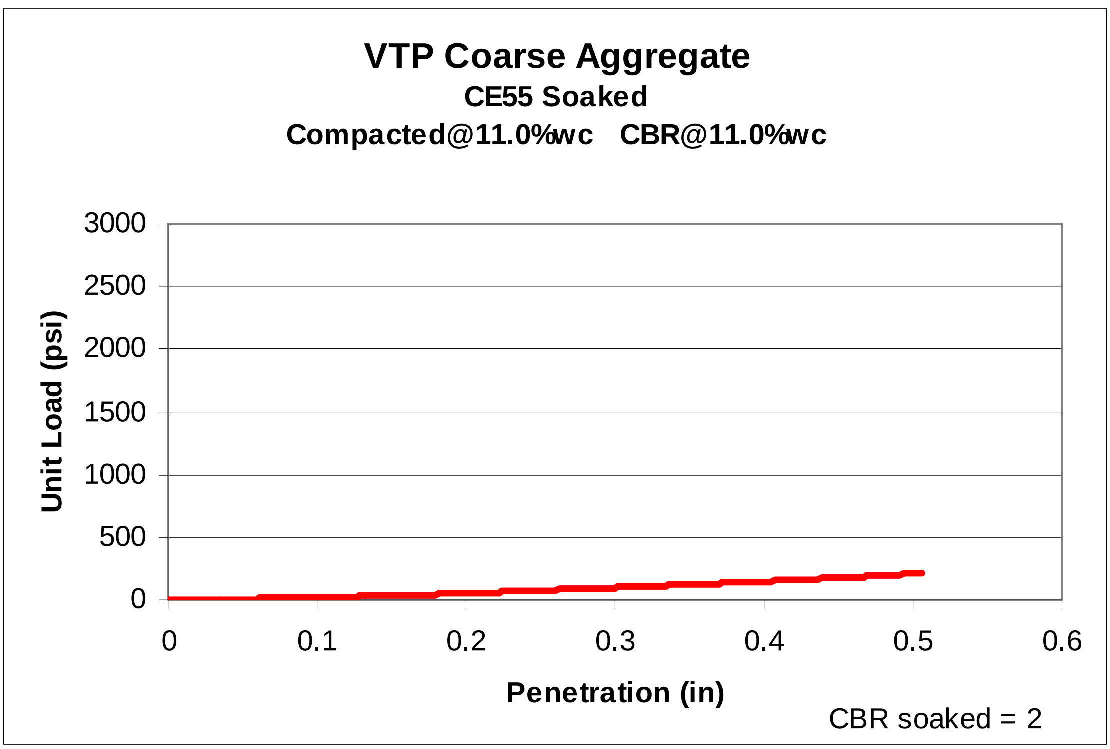

CBR VTP Coarse Aggregate
CE-55 Unsoaked 9.0%wc
3000
2500
2000
1500
1000
500
0
Unit Load (psi)
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 5


CBR VTP Coarse Aggregate
CE-55 Unsoaked 11.0%wc
3000
2500
2000
1500
1000
500
0
Unit Load (psi)
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = <1

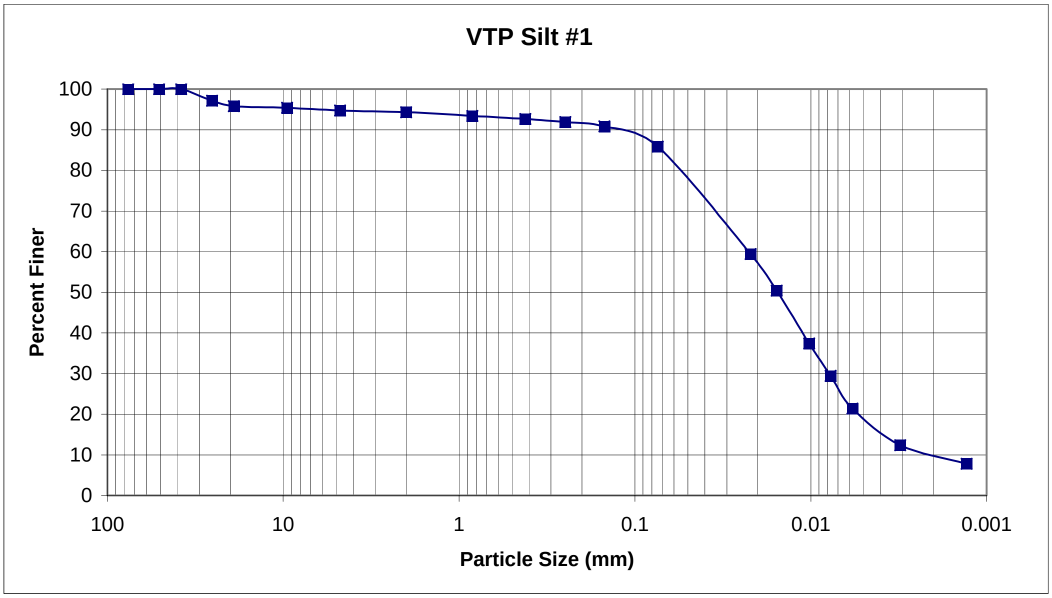

VTP Coarse Aggregate
CE-55 Soaked
Compacted@3.2% wc CBR@10.2%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 30


VTP Coarse Aggregate
CE55 Soaked
Compacted @5.7%wc CBR@ 8.9%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 85


VTP Coarse Aggregate
CE55 Soaked
Compacted@9.0%wc CBR@9.5%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 5


VTP Coarse Aggregate
CE55 Soaked
Compacted@11.0%wc CBR@11.0%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR soaked = 2

## PARTICLE SIZE ANALYSIS




**Mechanical Analysis Data**

| Seive opening in | Seive opening mm | Percent Passing |
|---|---|---|
| 3 | 76.2 | 100 |
| 2 | 50.8 | 100 |
| 1.5 | 38.1 | 100 |
| 1 | 25.4 | 97.1 |
| 0.75 | 19.1 | 95.8 |
| 0.375 | 9.52 | 95.4 |
| # 4 | 4.76 | 94.7 |
| # 10 | 2 | 94.3 |
| # 20 | 0.84 | 93.4 |
| # 40 | 0.42 | 92.6 |
| # 60 | 0.249 | 91.9 |
| # 100 | 0.149 | 90.8 |
| # 200 | 0.074 | 85.8 |
| Hydrometer | 0.022 | 59.4 |
| | 0.0156 | 50.4 |
| | 0.0102 | 37.4 |
| | 0.0077 | 29.4 |
| | 0.0058 | 21.4 |
| | 0.0031 | 12.4 |
| | 0.0013 | 7.8 |

**Sample Data**

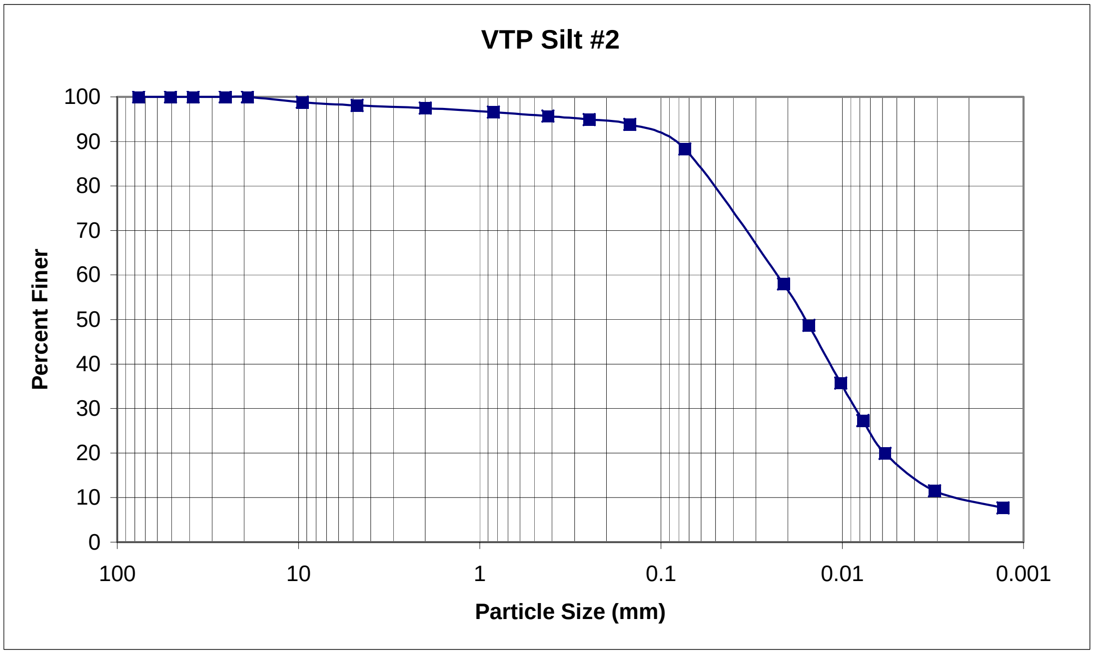
Location: VTP Silt #1
Elev or Depth:
Description: Silt
USCS Classification: ML
AASHTO Classification:A-4 (0)
Plastic limit: 25
Liquid limit:0
Plastic index: NP
Remarks:

**Fractional Components**

Gravel/sand based on: # 4
Sand/fines based on:# 200
% Cobbels = 0.0
% Gravel = 5.3
% Sand = 8.9
% fines = 85.8 (silt =67.5 clay =18.3)

| | |
|---|---|
| D85 = 0.07 | D60 = 0.02 |
| D50 = 0.02 | D30 = 0.01 |
| D15 = 0.00 | D10 = 0.00 |
| Cc = 1.3139 | Cu = 10.7416 |

## PARTICLE SIZE ANALYSIS




**Mechanical Analysis Data**

| Seive opening in | Seive opening mm | Percent Passing |
|---|---|---|
| 3 | 76.2 | 100 |
| 2 | 50.8 | 100 |
| 1.5 | 38.1 | 100 |
| 1 | 25.4 | 100 |
| 0.75 | 19.1 | 100 |
| 0.375 | 9.52 | 98.8 |
| # 4 | 4.76 | 98.1 |
| # 10 | 2 | 97.5 |
| # 20 | 0.84 | 96.6 |
| # 40 | 0.42 | 95.7 |
| # 60 | 0.249 | 94.9 |
| # 100 | 0.149 | 93.8 |
| # 200 | 0.074 | 88.3 |
| Hydrometer | 0.0211 | 58 |
| | 0.0153 | 48.7 |
| | 0.0102 | 35.7 |
| | 0.0077 | 27.3 |
| | 0.0058 | 19.9 |
| | 0.0031 | 11.5 |
| | 0.0013 | 7.7 |

**Sample Data**

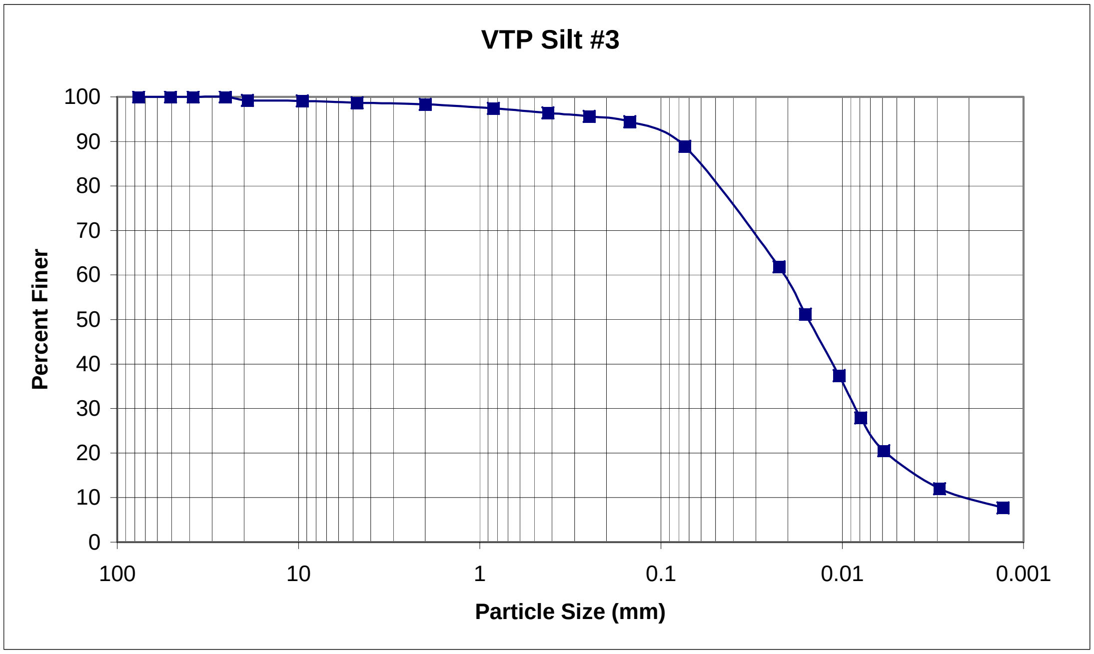
Location: VTP Silt #2
Elev or Depth:
Description: Silt
USCS Classification: ML
AASHTO Classification:A-4 (0)
Plastic limit: 25
Liquid limit:0
Plastic index: NP
Remarks:

**Fractional Components**

Gravel/sand based on: # 4
Sand/fines based on:# 200
% Cobbels = 0.0
% Gravel = 1.9
% Sand = 9.8
% fines = 88.3 (silt =71.2 clay =17.1)

| | |
|---|---|
| D85 = 0.06 | D60 = 0.02 |
| D50 = 0.02 | D30 = 0.01 |
| D15 = 0.00 | D10 = 0.00 |
| Cc = 1.2887 | Cu = 9.2465 |

## PARTICLE SIZE ANALYSIS




**Mechanical Analysis Data**

| Seive opening in | Seive opening mm | Percent Passing |
|---|---|---|
| 3 | 76.2 | 100 |
| 2 | 50.8 | 100 |
| 1.5 | 38.1 | 100 |
| 1 | 25.4 | 100 |
| 0.75 | 19.1 | 99.2 |
| 0.375 | 9.52 | 99.1 |
| # 4 | 4.76 | 98.7 |
| # 10 | 2 | 98.3 |
| # 20 | 0.84 | 97.4 |
| # 40 | 0.42 | 96.4 |
| # 60 | 0.249 | 95.6 |
| # 100 | 0.149 | 94.4 |
| # 200 | 0.074 | 88.9 |
| Hydrometer | 0.0224 | 61.8 |
| | 0.016 | 51.2 |
| | 0.0104 | 37.4 |
| | 0.0079 | 27.9 |
| | 0.0059 | 20.5 |
| | 0.0029 | 12 |
| | 0.0013 | 7.7 |

**Sample Data**

Location: VTP Silt #3
Elev or Depth:
Description: Silt
USCS Classification: ML
AASHTO Classification:A-4 (0)
Plastic limit: 25
Liquid limit:0
Plastic index: NP
Remarks:

**Fractional Components**

Gravel/sand based on: # 4
Sand/fines based on:# 200
% Cobbels = 0.0
% Gravel = 1.3
% Sand = 9.8
% fines = 88.9 (silt =71.2 clay =17.7)

| | |
|---|---|
| D85 = 0.06 | D60 = 0.02 |
| D50 = 0.02 | D30 = 0.01 |
| D15 = 0.00 | D10 = 0.00 |
| Cc = 1.5846 | Cu = 9.9335 |

## Liquid & Plastic Limits

| | | | |
|---|---|---|---|
| **Project:** | VTP | **Date:** | 22-Jun-01 |
| **Description:** | Silt | **Tech:** | S. Orchino |

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|
| **Tare No.** | 1 | 6 | | | | | | |
| **No. of Blows** | 10 | 17 | | | | | | |
| **Tare + Wet Soil** | 35.00 | 38.76 | | | | | | |
| **Tare + Dry Soil** | 31.45 | 34.76 | | | | | | |
| **Water** | 3.55 | 4.00 | 0 | 0 | 0 | 0 | 0 | 0 |
| **Tare** | 19.01 | 18.86 | | | | | | |
| **Dry Soil** | 12.44 | 15.90 | 0 | 0 | 0 | 0 | 0 | 0 |
| **Water Content** | 28.5% | 25.2% | #DIV/0! | #DIV/0! | #DIV/0! | | | |


| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|
| **Tare No.** | 21 | 22 | 23 | | | | | |
| **Tare + Wet Soil** | 23.83 | 23.23 | 23.13 | | | | | |
| **Tare + Dry Soil** | 22.85 | 22.27 | 22.17 | | | | | |
| **Water** | 0.98 | 0.96 | 0.96 | 0 | 0 | 0 | 0 | 0 |
| **Tare** | 19.04 | 18.26 | 18.39 | | | | | |
| **Dry Soil** | 3.81 | 4.01 | 3.78 | 0 | 0 | 0 | 0 | 0 |
| **Water Content** | 25.7% | 23.9% | 25.4% | | | | | |

**Remarks:**

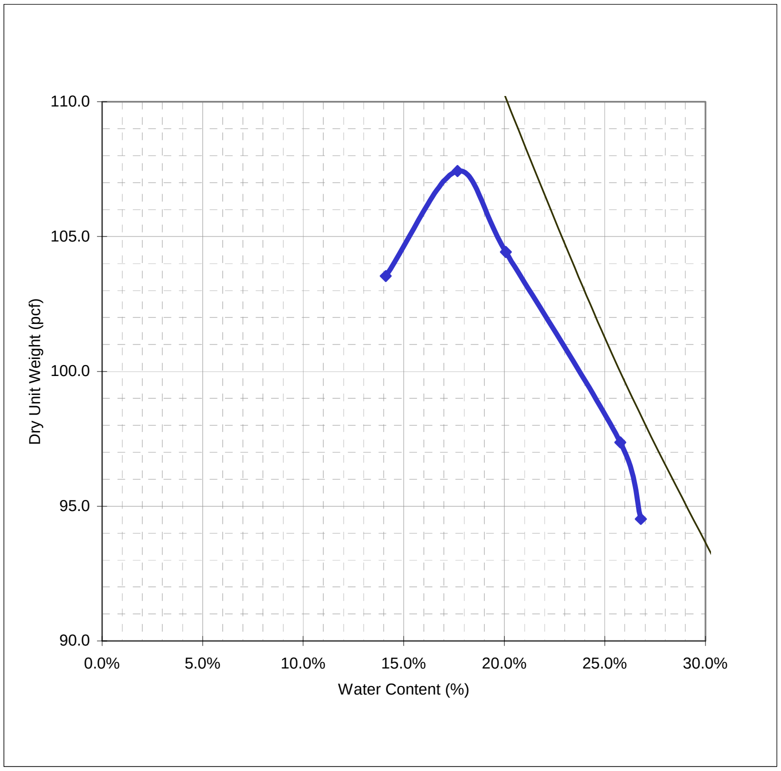
LL=
PL= 25
PI=
Class= ML

**SPECIFIC GRAVITY**

| | | | |
|---|---|---|---|
| **Project:** | **VTP** | **Date:** | **18-Jun-01** |
| **Remarks:** | **Silt** | **Technician:** | **S. Orchino** |

| **Sample Number** | 1 | 2 | 3 | | | |
|---|---|---|---|---|---|---|
| **Flask** | S-5 | SPG-5 | P-5 | | | |
| **Temp. of water and soil T** | 21.6 | 21.6 | 21.7 | | | |
| **Dish No.** | C1 | C6 | Blank | | | |
| **Dish + Dry Soil (g)** | 291.09 | 293.09 | 299.22 | | | |
| **Dish (g)** | 143.86 | 147.85 | 141.5 | | | |
| **Dry Soil (g) Ws** | 147.23 | 145.24 | 157.72 | 0 | 0 | 0 |
| **Flask + water at T Wbw** | 695.89 | 679.95 | 681.05 | | | |
| **Ws + Wbw** | 843.12 | 825.19 | 838.77 | 0 | 0 | 0 |
| **Flask + water + soil Wbws** | 789.16 | 771.98 | 780.53 | | | |
| **Displaced Water Ws+Wbw-Wbws** | 53.96 | 53.21 | 58.24 | 0 | 0 | 0 |
| **Correction Factor K** | 0.9996 | 0.9996 | 0.9996 | | | |
| **(WsK)/(Ws+Wbw-Wbws) Gs** | 2.73 | 2.73 | 2.71 | #DIV/0! | #DIV/0! | #DIV/0! |

## COMPACTION TEST REPORT




**MATERIAL**
VTP Silt

**Test Specification:**
MIL-STD-621A, Method A, CE-12

| | |
|---|---|
| **Hammer Wt.:** | 10 lbs |
| **Hammer Drop:** | 18 in |
| **Number of Layers:** | 5 |
| **Blows per Layer:** | 12 |
| **Mold Size:** | .075 cu.ft. |

**Test Performed on Material**
**Passing** 3/4 in. **Sieve**

**SOIL DATA**

| | |
|---|---|
| **SpG:** | 2.73 |
| **LL:** | 0 |
| **PI:** | 0 |
| **% >3/4 in.:** | 1.7 |
| **% <#200:** | 87.7 |

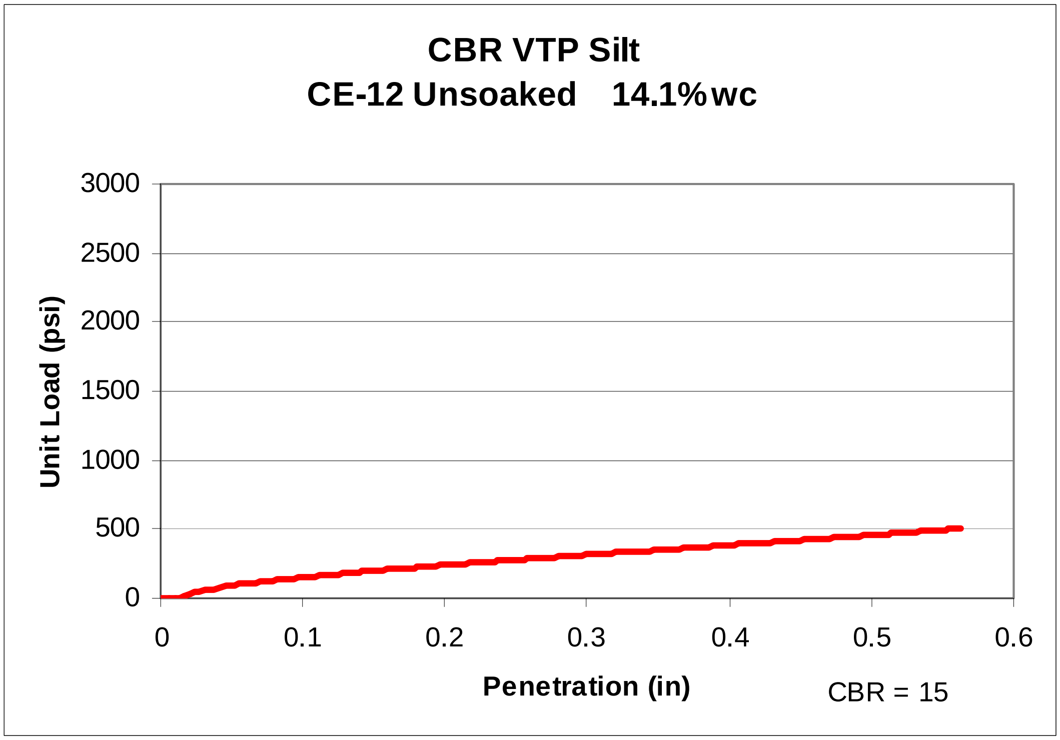
**CLASSIFICATION**

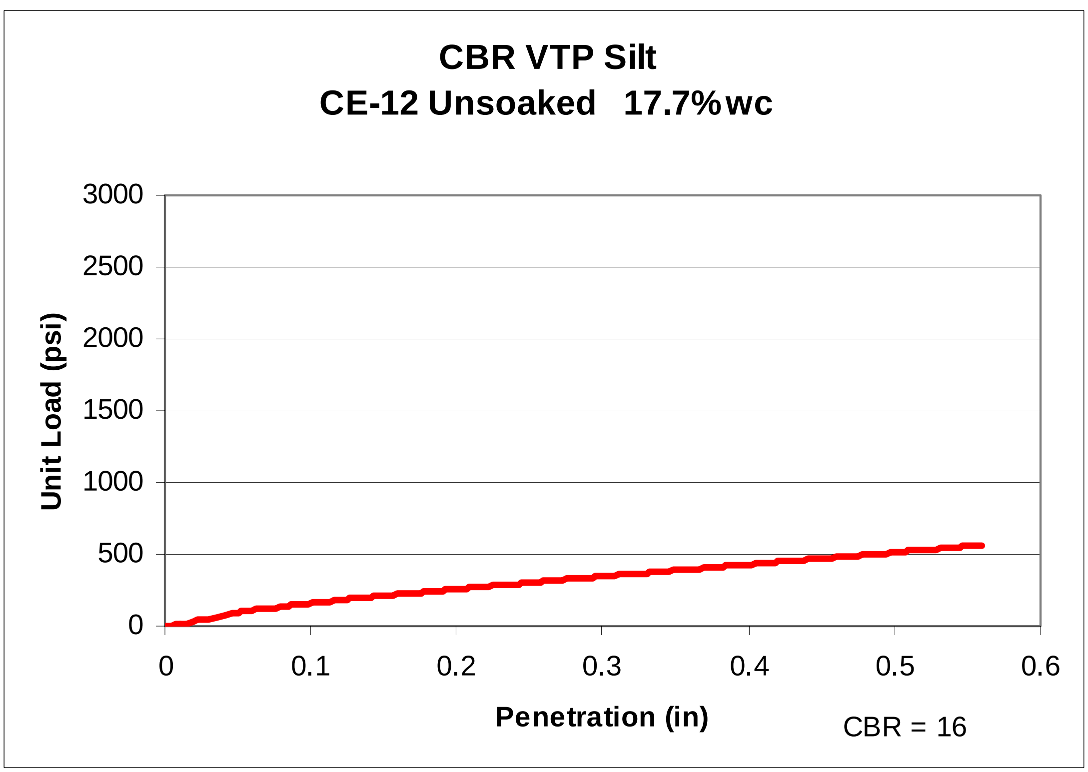
| | |
|---|---|
| **USCS:** | ML |
| **AASHTO:** | A-4 |

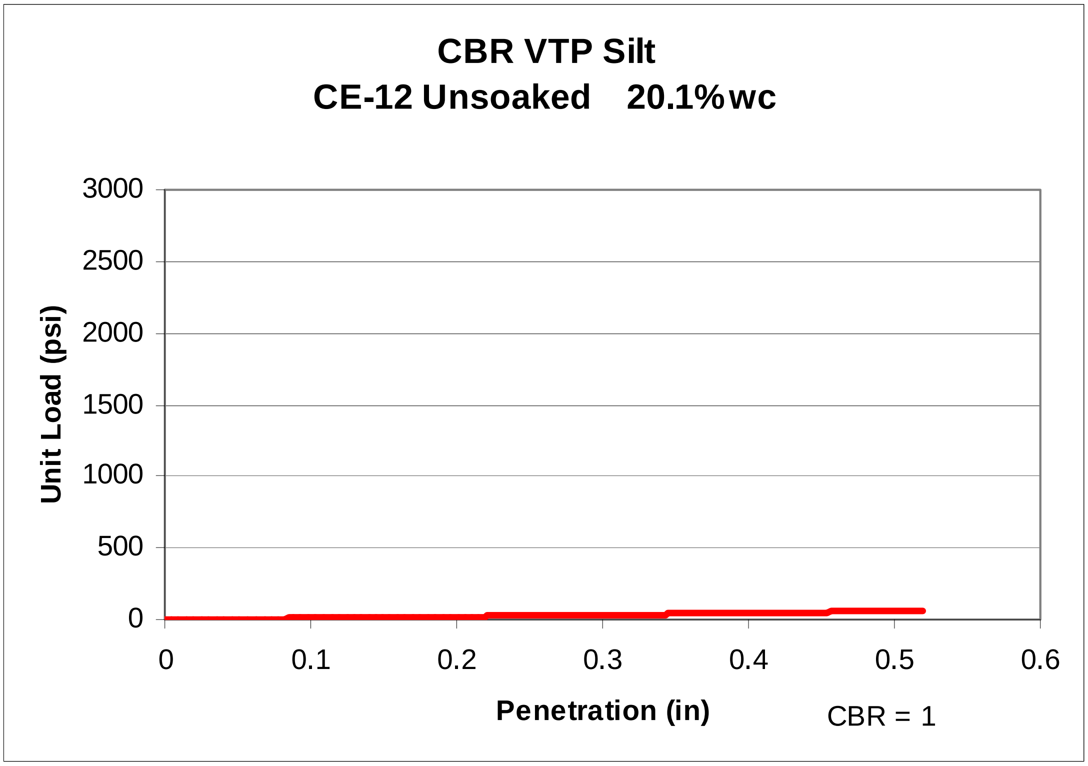
| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| **Wet Soil + Mold** | 19699.00 | 19894.00 | 19860.00 | 19761.00 | 19830.00 |
| **Mold** | 15680.00 | 15593.00 | 15594.00 | 15595.00 | 15753.00 |
| **Wet Density** | 118.14 | 126.43 | 125.40 | 122.46 | 119.84 |
| **Wet Soil + Tare 1** | 294.15 | 285.02 | 435.64 | 341.56 | 468.29 |
| **Dry Soil + Tare 1** | 258.30 | 243.71 | 365.47 | 271.87 | 371.50 |
| **Tare 1** | 7.02 | 7.17 | 7.37 | 7.20 | 7.40 |
| **Moisture 1** | 14.27% | 17.46% | 19.60% | 26.33% | 26.58% |
| **Wet Soil + Tare 2** | 249.04 | 295.59 | 463.83 | 400.82 | 412.84 |
| **Dry Soil + Tare 2** | 219.45 | 251.83 | 386.00 | 321.58 | 326.65 |
| **Tare 2** | 7.29 | 7.24 | 7.42 | 7.18 | 7.27 |
| **Moisture 2** | 13.95% | 17.89% | 20.56% | 25.20% | 26.99% |
| **Moisture** | 14.1% | 17.7% | 20.1% | 25.8% | 26.8% |
| **Dry Density** | 103.5 | 107.4 | 104.4 | 97.4 | 94.5 |

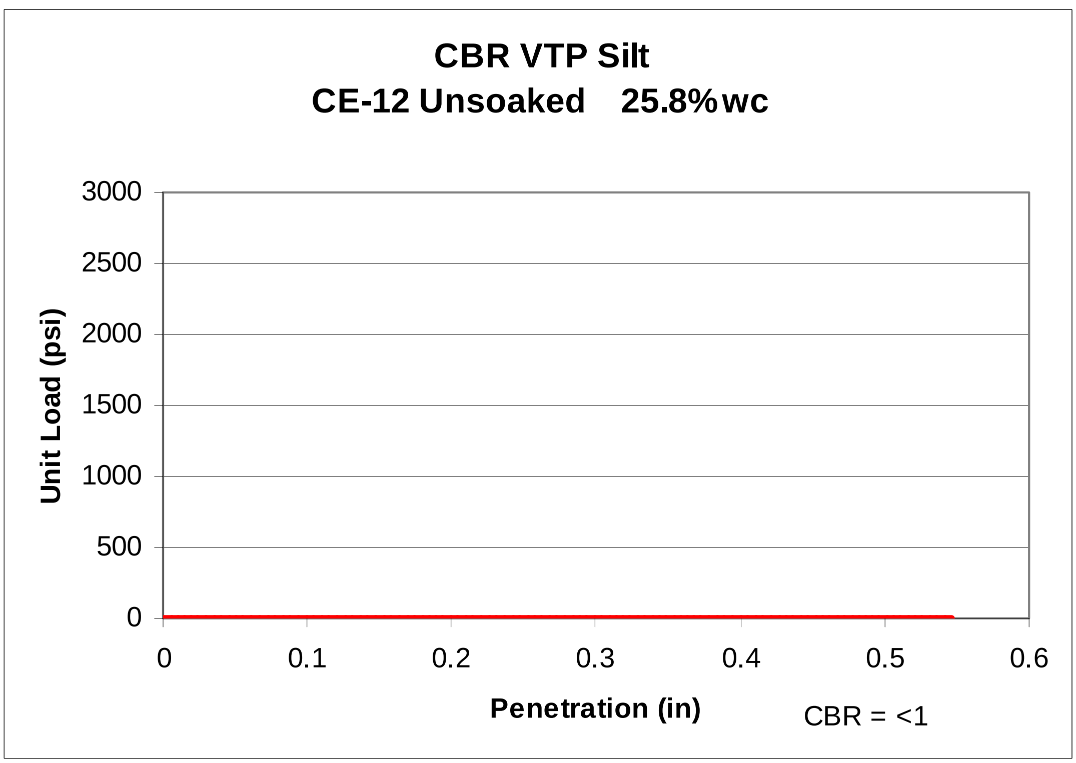
**MAXIMUM DRY DENSITY =** **107.5 pcf**

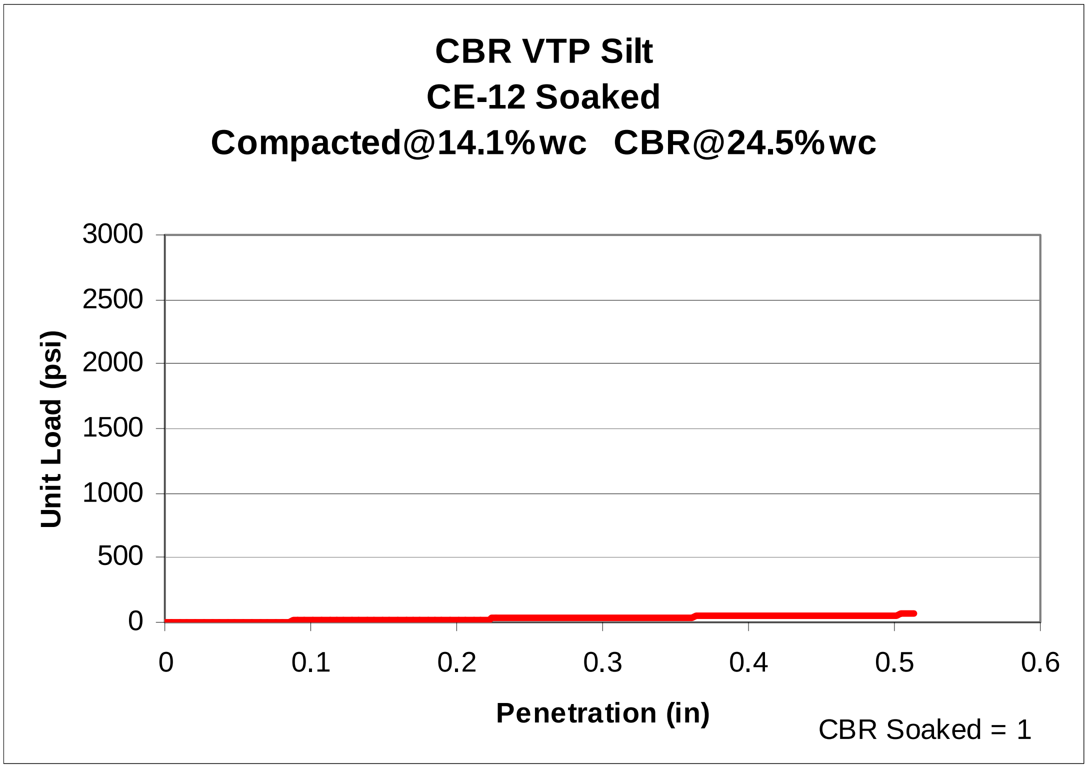
**OPTIMUM MOISTURE =** **17.50%**

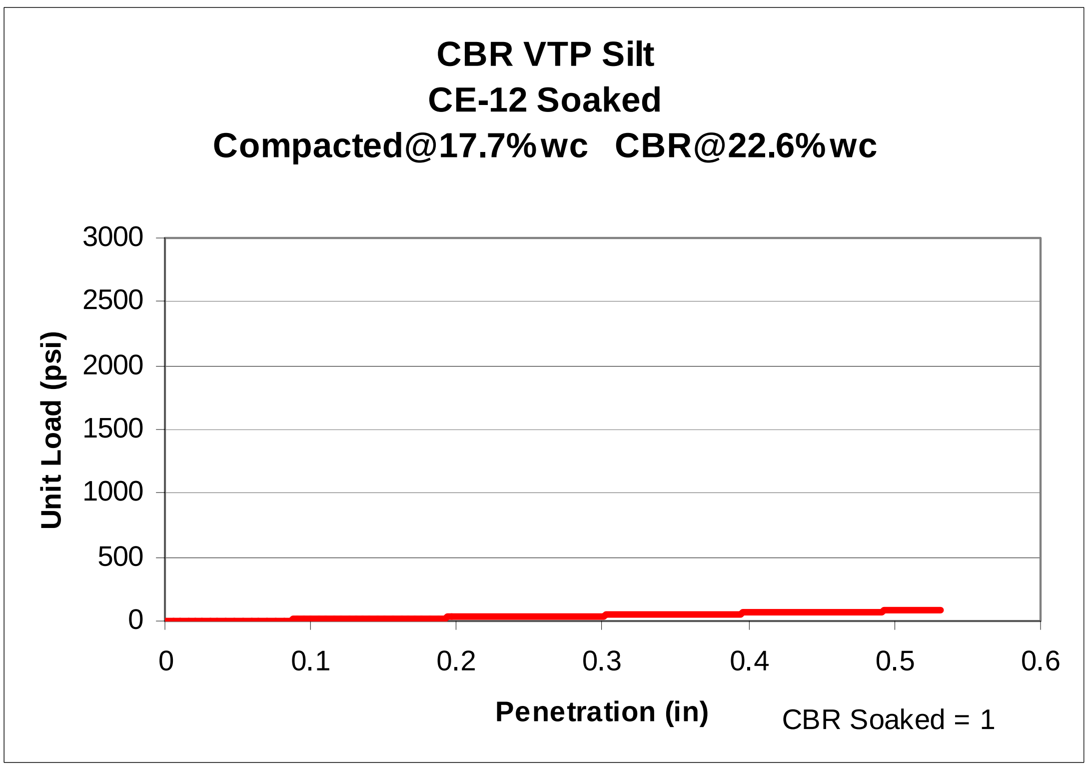

CBR VTP Silt
CE-12 Unsoaked 14.1%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 15

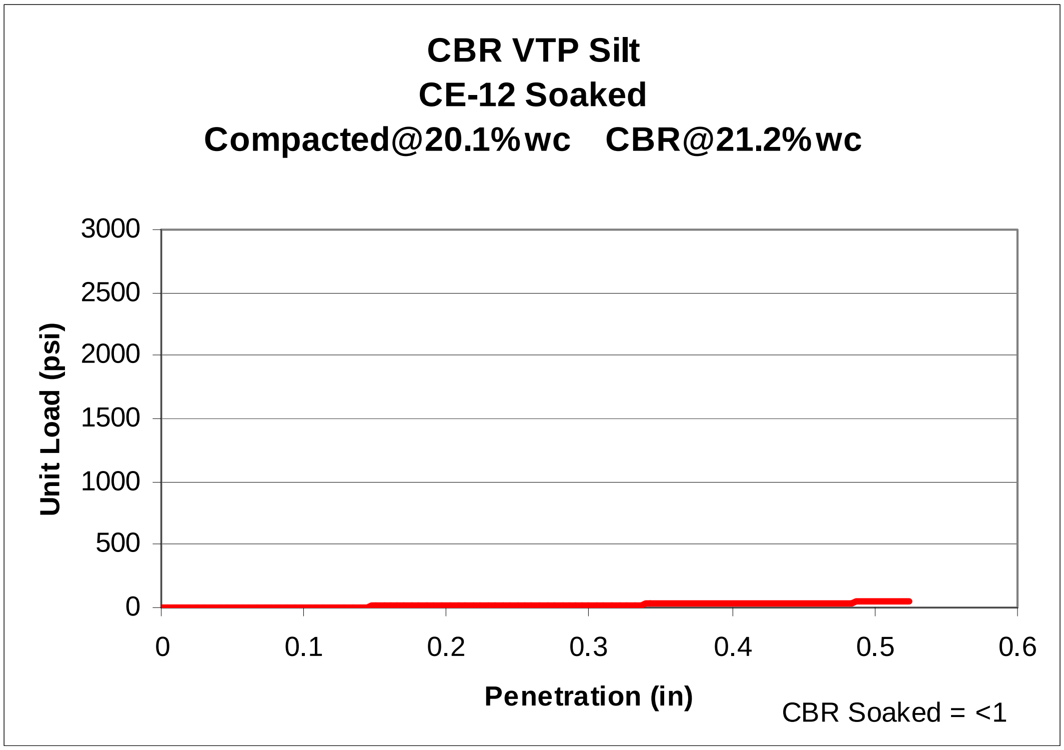

CBR VTP Silt
CE-12 Unsoaked 17.7%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 16

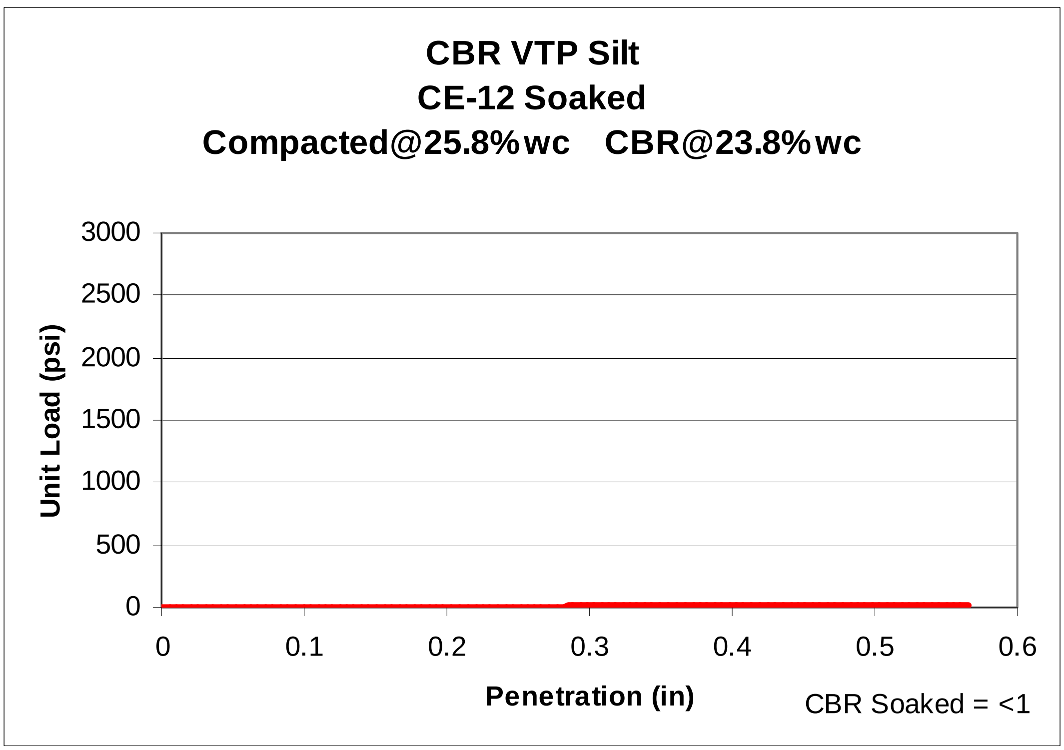

CBR VTP Silt
CE-12 Unsoaked 20.1%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 1


CBR VTP Silt
CE-12 Unsoaked 25.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = <1

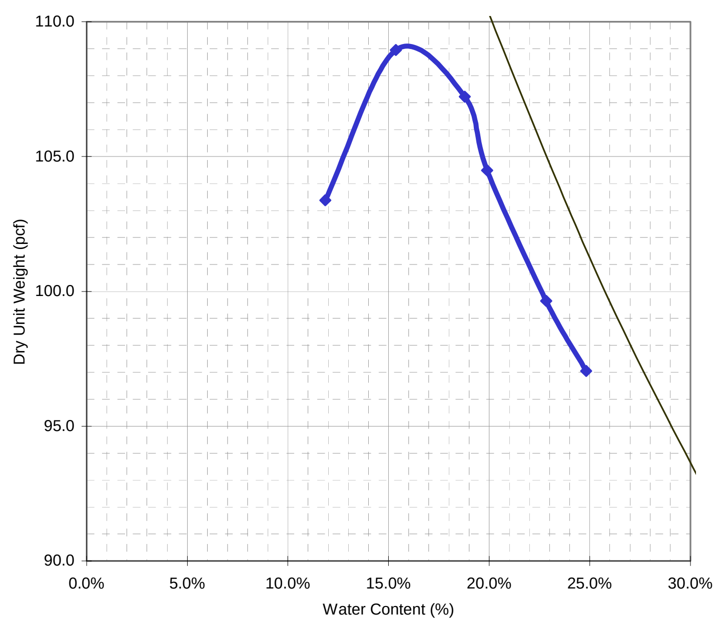

CBR VTP Silt
CE-12 Soaked
Compacted@14.1%wc CBR@24.5%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 1


CBR VTP Silt
CE-12 Soaked
Compacted@17.7%wc CBR@22.6%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 1


CBR VTP Silt
CE-12 Soaked
Compacted@20.1%wc CBR@21.2%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = <1


CBR VTP Silt
CE-12 Soaked
Compacted@25.8%wc CBR@23.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = <1

## COMPACTION TEST REPORT




**MATERIAL**
VTP Silt

**Test Specification:**
MIL-STD-621A, Method 100, CE-26

| | |
|---|---|
| **Hammer Wt.:** | 10 lbs |
| **Hammer Drop:** | 18 in |
| **Number of Layers:** | 5 |
| **Blows per Layer:** | 26 |
| **Mold Size:** | .075 cu.ft. |

**Test Performed on Material**
**Passing** 3/4 in. **Sieve**

**SOIL DATA**

| | |
|---|---|
| **SpG:** | 2.73 |
| **LL:** | 0 |
| **PI:** | 0 |
| **% >3/4 in.:** | 1.7 |
| **% <#200:** | 87.7 |

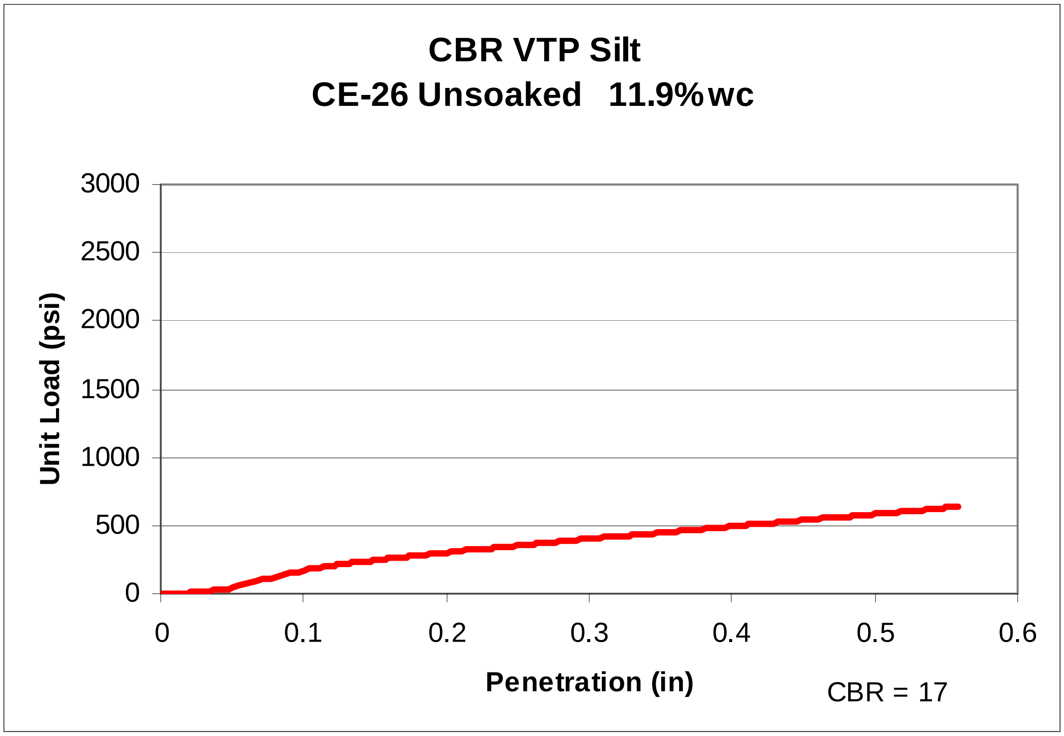
**CLASSIFICATION**

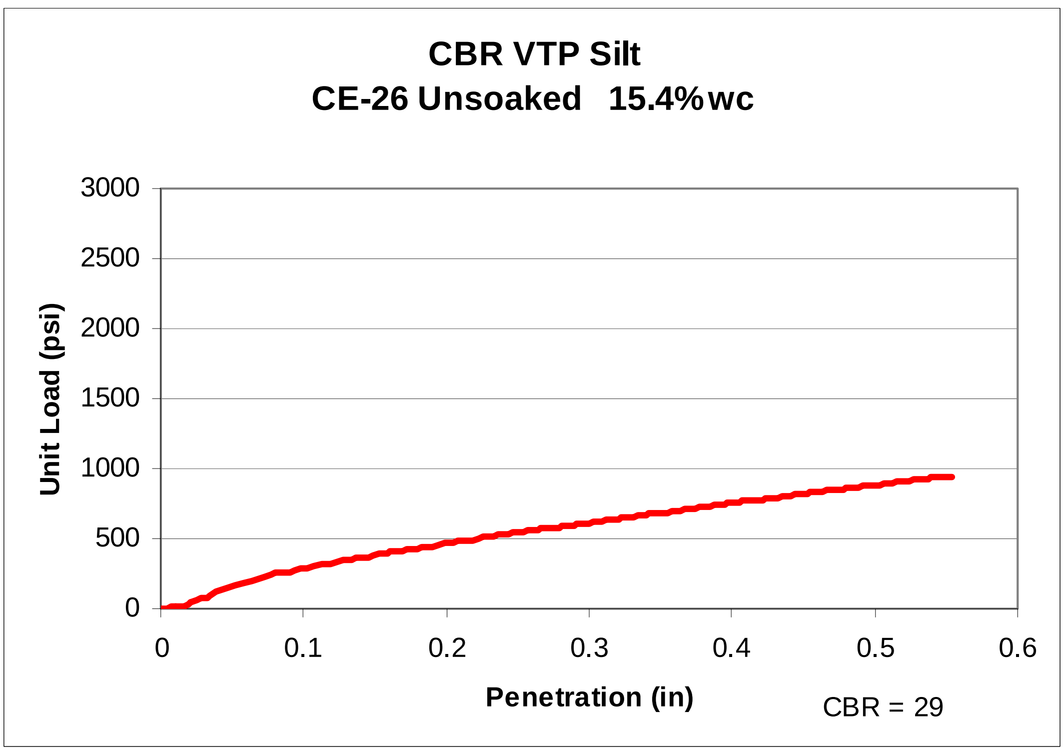
| | |
|---|---|
| **USCS:** | ML |
| **AASHTO:** | A-4 |

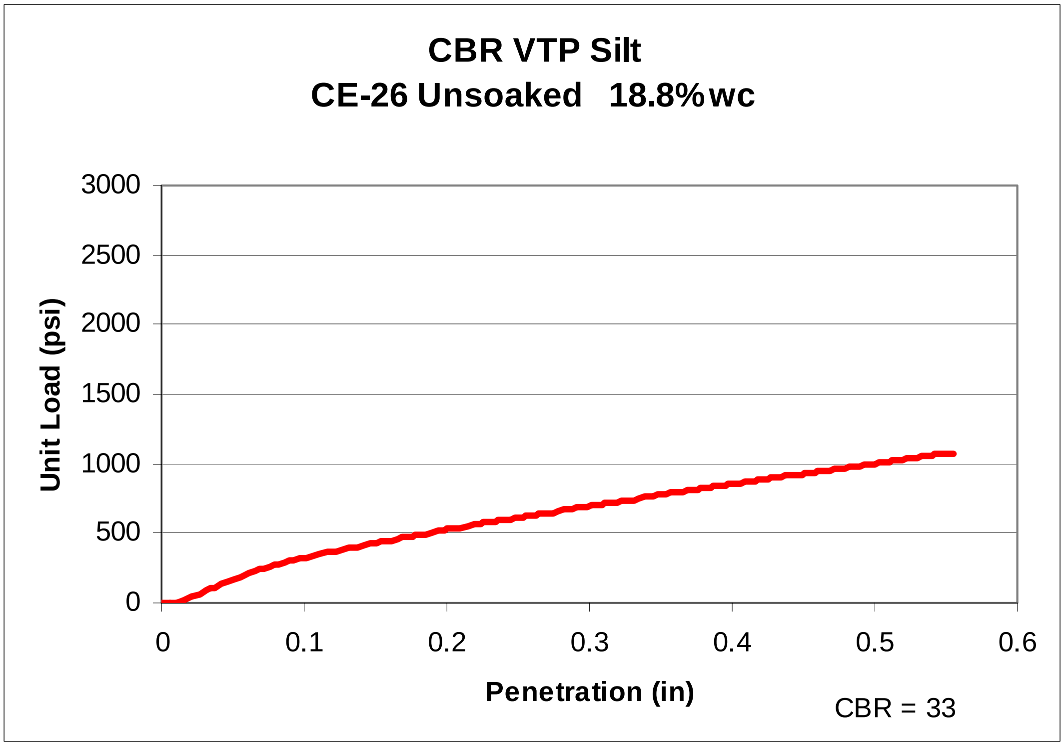
| | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| **Wet Soil + Mold** | 19614.00 | 20022.00 | 19998.00 | 19947.00 | 19911.00 | 19880.00 |
| **Mold** | 15680.00 | 15746.00 | 15665.00 | 15685.00 | 15747.00 | 15759.00 |
| **Wet Density** | 115.64 | 125.69 | 127.37 | 125.28 | 122.40 | 121.13 |
| **Wet Soil + Tare 1** | 100.00 | 301.42 | 357.58 | 458.70 | 509.17 | 358.17 |
| **Dry Soil + Tare 1** | 89.40 | 261.72 | 302.04 | 384.52 | 417.25 | 286.01 |
| **Tare 1** | 0.00 | 7.01 | 7.17 | 7.23 | 7.40 | 7.15 |
| **Moisture 1** | 11.86% | 15.59% | 18.84% | 19.66% | 22.43% | 25.88% |
| **Wet Soil + Tare 2** | 100.00 | 177.34 | 276.60 | 398.89 | 380.33 | 417.43 |
| **Dry Soil + Tare 2** | 89.40 | 154.97 | 234.07 | 333.23 | 309.97 | 338.67 |
| **Tare 2** | 0.00 | 7.20 | 7.15 | 7.14 | 7.20 | 7.19 |
| **Moisture 2** | 11.86% | 15.14% | 18.74% | 20.14% | 23.24% | 23.76% |
| **Moisture** | 11.9% | 15.4% | 18.8% | 19.9% | 22.8% | 24.8% |
| **Dry Density** | 103.4 | 109.0 | 107.2 | 104.5 | 99.6 | 97.0 |

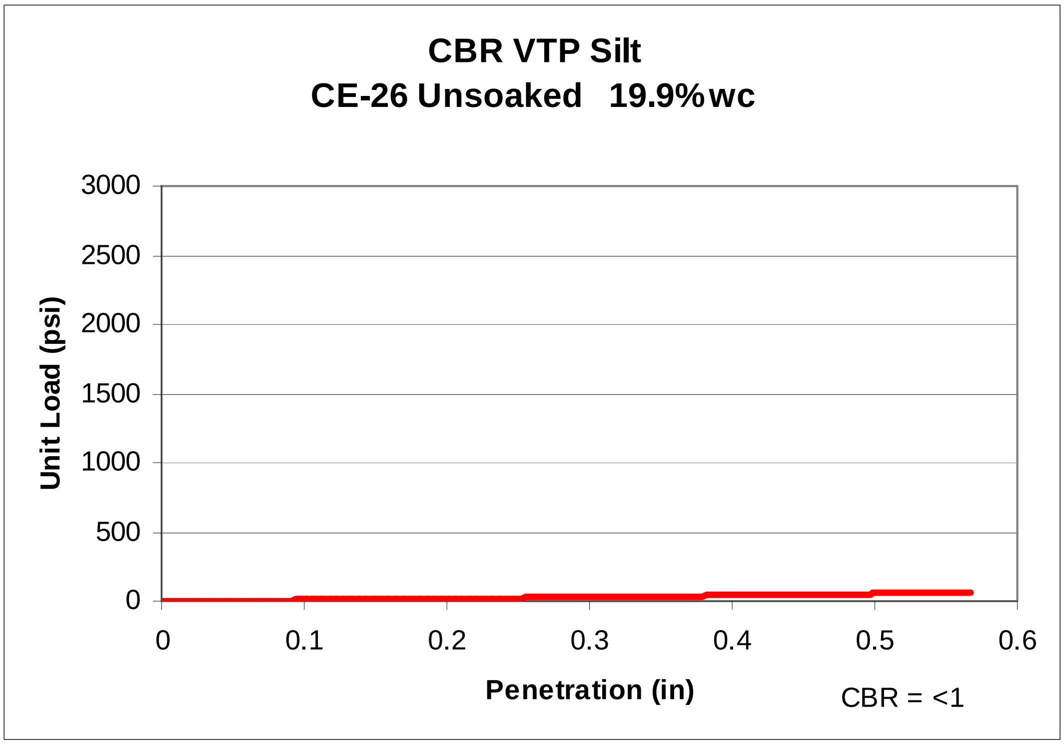
**MAXIMUM DRY DENSITY =** **109.5 pcf**

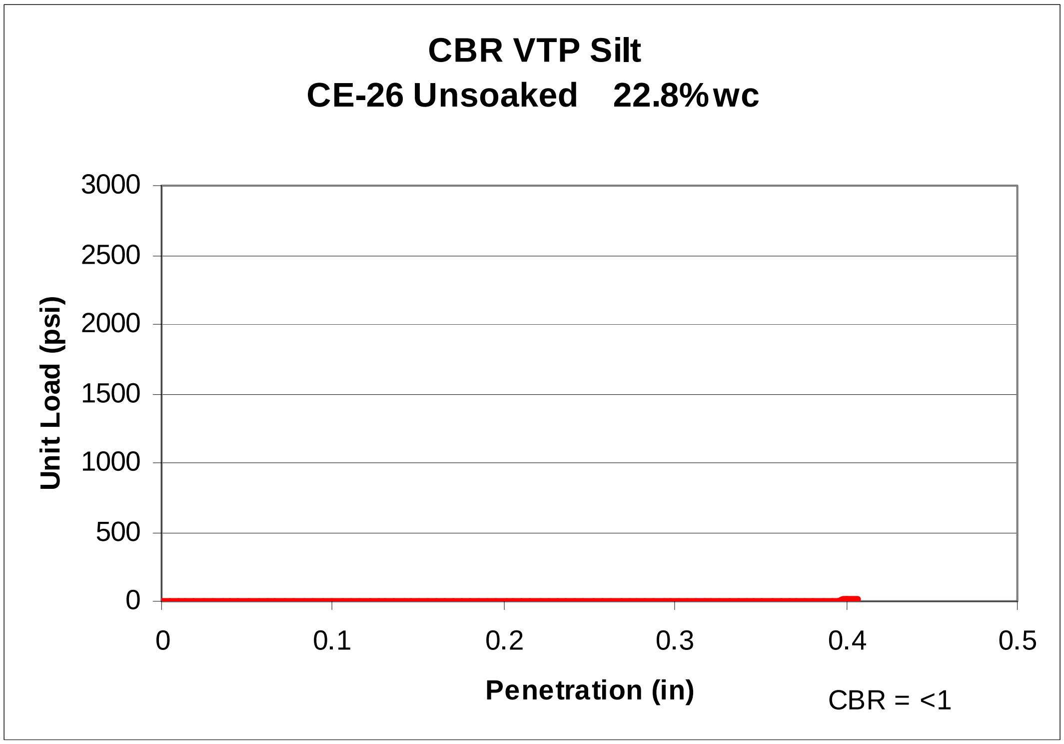
**OPTIMUM MOISTURE =** **16.50%**

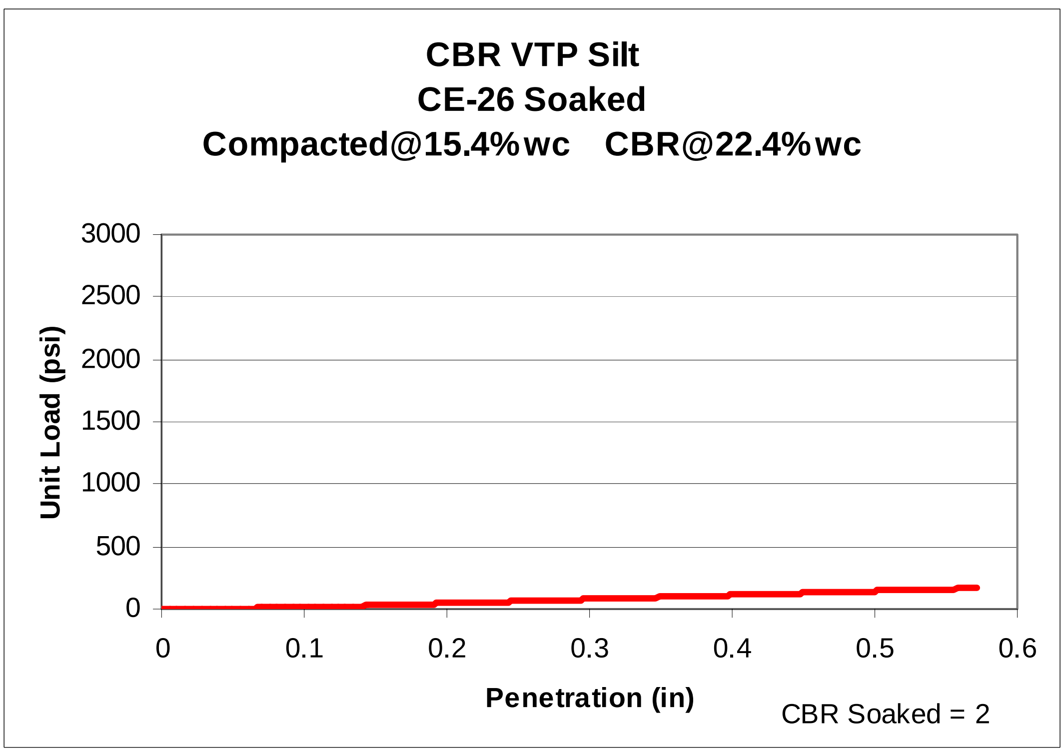

CBR VTP Silt
CE-26 Unsoaked 11.9%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 17

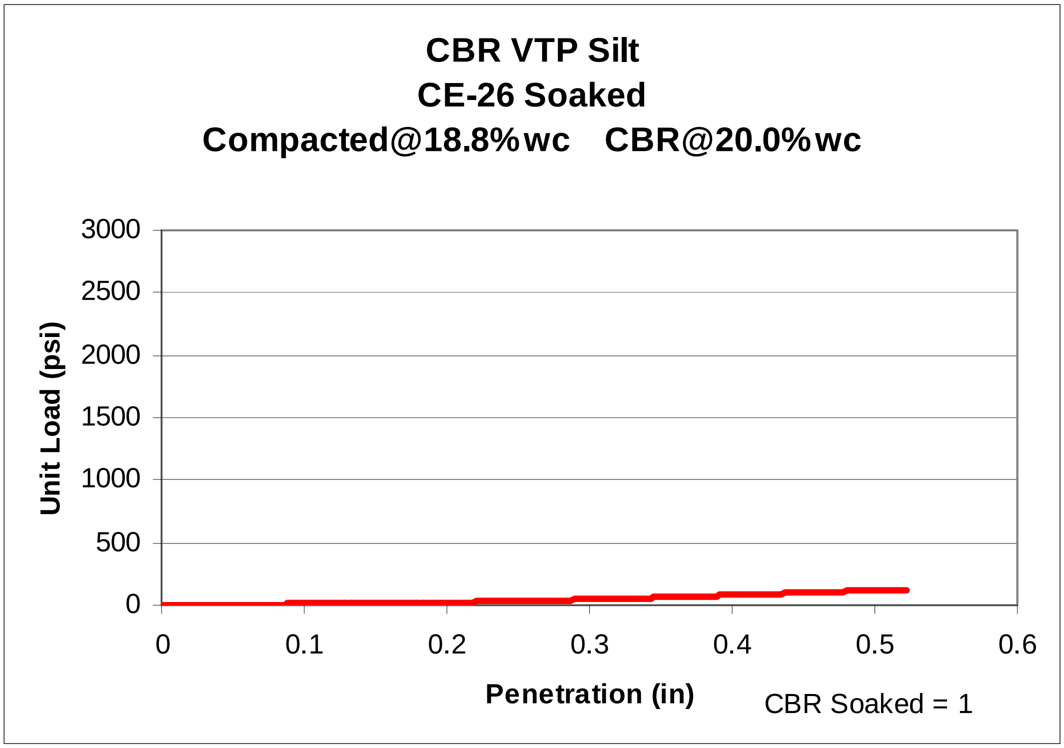

CBR VTP Silt
CE-26 Unsoaked 15.4%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 29

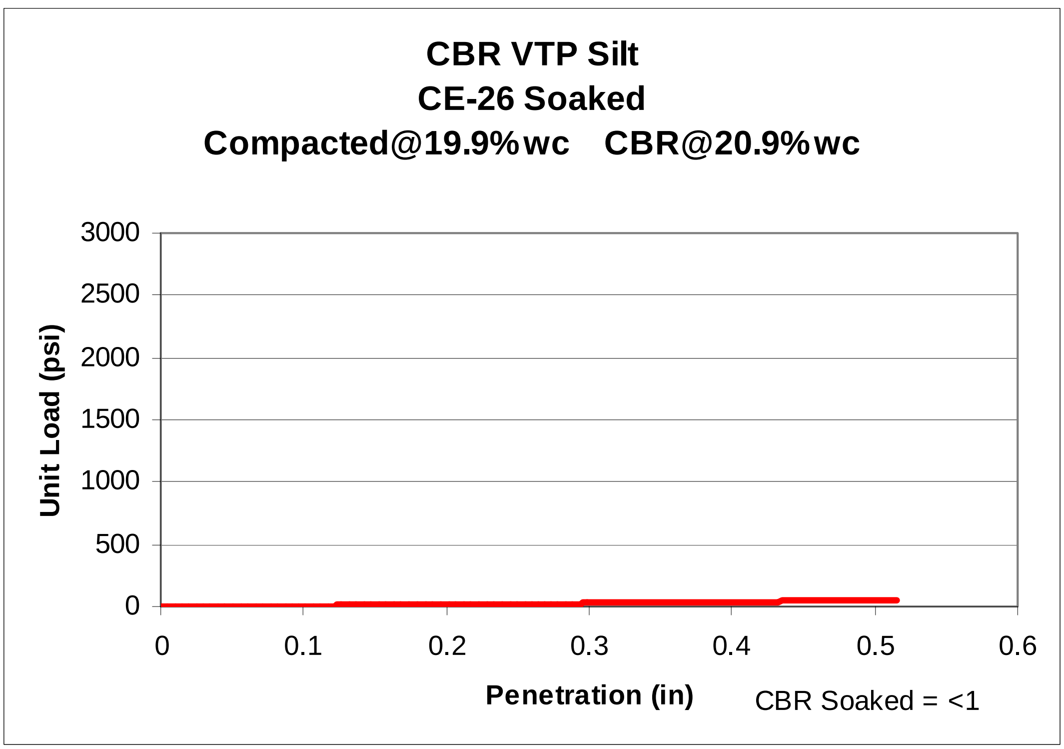

CBR VTP Silt
CE-26 Unsoaked 18.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = 33

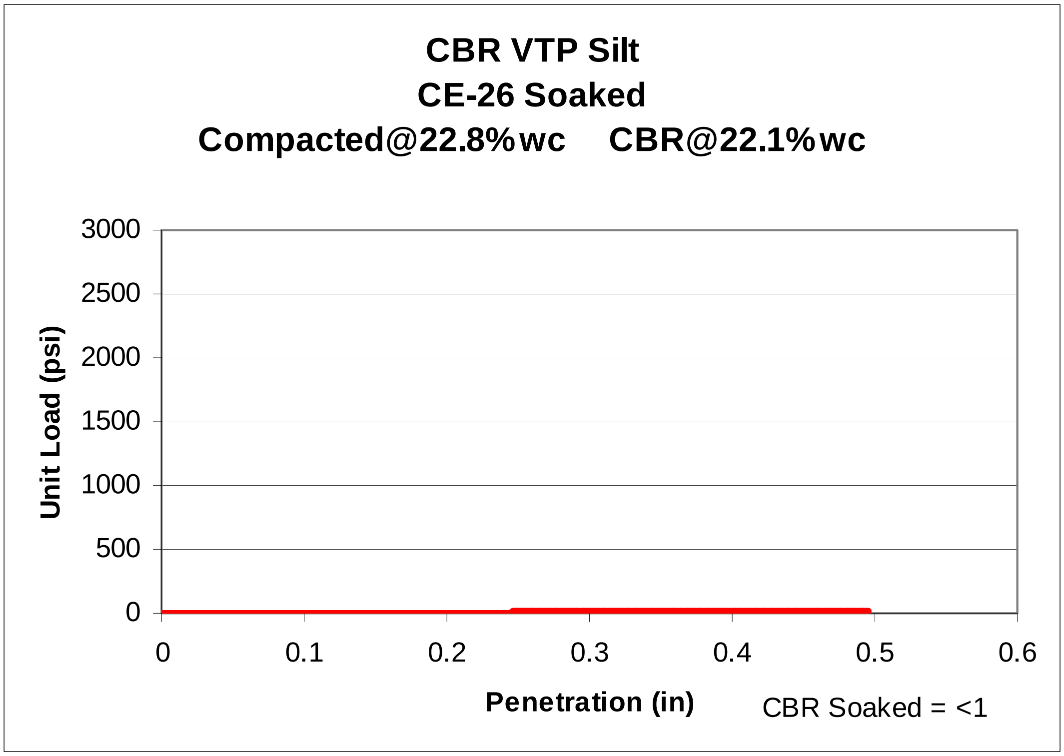

CBR VTP Silt
CE-26 Unsoaked 19.9%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR = <1


CBR VTP Silt
CE-26 Unsoaked 22.8%wc
3000
2500
2000
1500
1000
500
0
Unit Load (psi)
0
0.1
0.2
0.3
0.4
0.5
Penetration (in)
CBR = <1

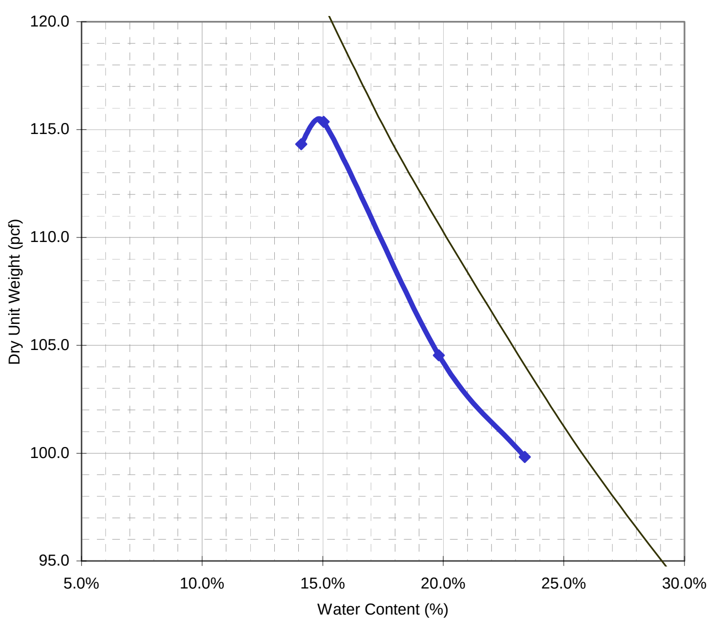

CBR VTP Silt
CE-26 Soaked
Compacted@15.4%wc CBR@22.4%wc
3000
2500
2000
1500
1000
500
0
Unit Load (psi)
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 2


CBR VTP Silt
CE-26 Soaked
Compacted@18.8%wc CBR@20.0%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 1


CBR VTP Silt
CE-26 Soaked
Compacted@19.9%wc CBR@20.9%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = <1


CBR VTP Silt
CE-26 Soaked
Compacted@22.8%wc CBR@22.1%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = <1

## COMPACTION TEST REPORT




**MATERIAL**
VTP Silt

**Test Specification:**
MIL-STD-621A, Method 100A, CE-55

**Hammer Wt.:** 10 lbs
**Hammer Drop:** 18 in
**Number of Layers:** 5
**Blows per Layer:** 55
**Mold Size:** .075 cu.ft.

**Test Performed on Material**
**Passing** 3/4 in. **Sieve**

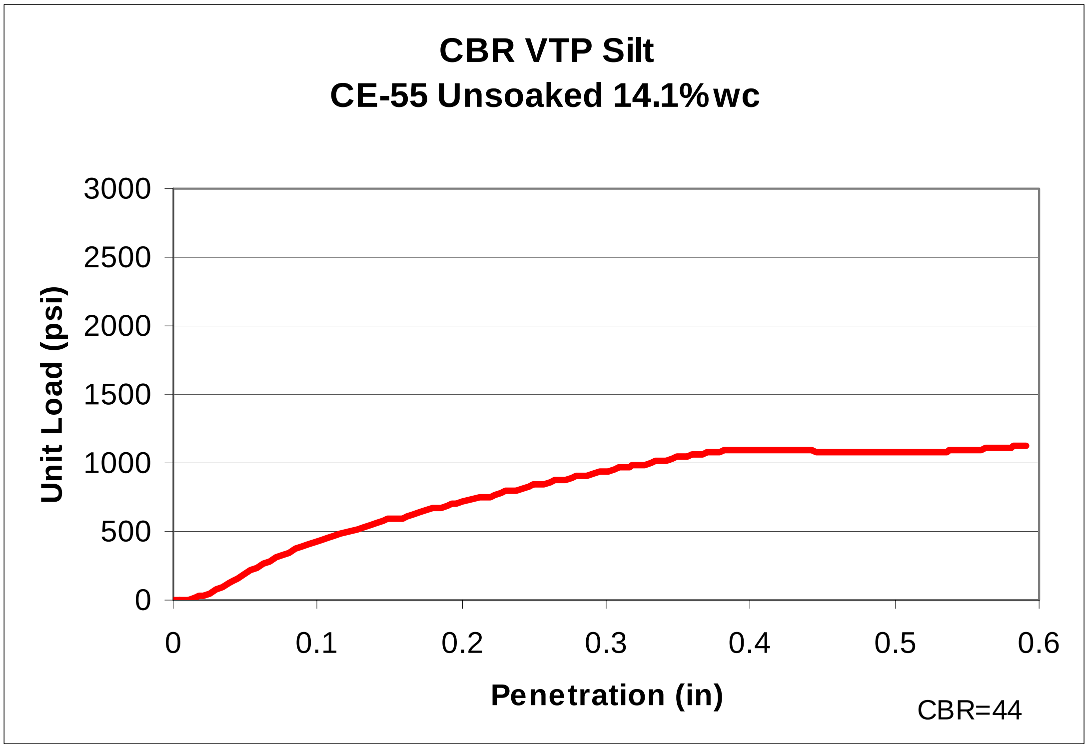
**SOIL DATA**
**SpG:** 2.73
**LL:** 0
**PI:** 0
**% >3/4 in.:** 1.7
**% <#200:** 87.7

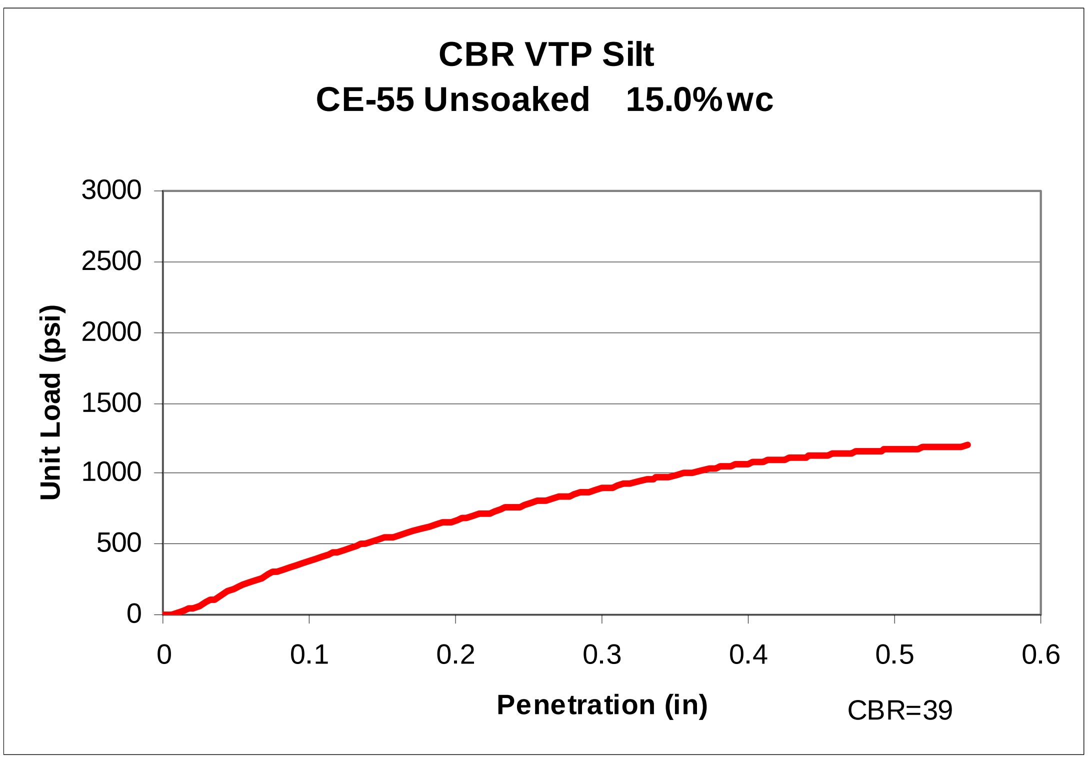
**CLASSIFICATION**
**USCS:** ML
**AASHTO:** A-4

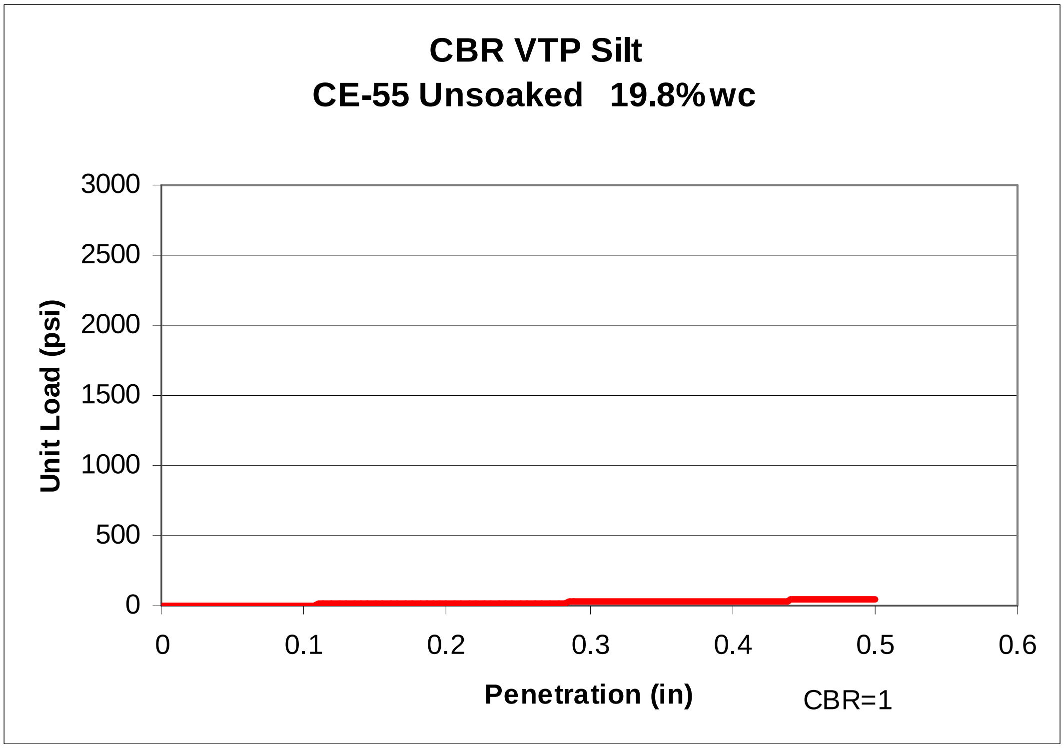
| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| **Wet Soil + Mold** | 20124.00 | 20110.00 | 19855.00 | 19876.00 | |
| **Mold** | 15686.00 | 15595.00 | 15594.00 | 15686.00 | |
| **Wet Density** | 130.45 | 132.72 | 125.25 | 123.16 | |
| **Wet Soil + Tare 1** | 296.78 | 225.70 | 389.03 | 409.09 | |
| **Dry Soil + Tare 1** | 260.70 | 196.80 | 326.72 | 334.23 | |
| **Tare 1** | 7.21 | 7.21 | 7.24 | 7.24 | |
| **Moisture 1** | 14.23% | 15.24% | 19.50% | 22.89% | |
| **Wet Soil + Tare 2** | 271.80 | 285.06 | 315.74 | 380.07 | |
| **Dry Soil + Tare 2** | 239.32 | 249.15 | 264.06 | 308.20 | |
| **Tare 2** | 7.14 | 7.19 | 7.24 | 7.10 | |
| **Moisture 2** | 13.99% | 14.84% | 20.12% | 23.87% | |
| **Moisture** | 14.1% | 15.0% | 19.8% | 23.4% | |
| **Dry Density** | 114.3 | 115.4 | 104.5 | 99.8 | |

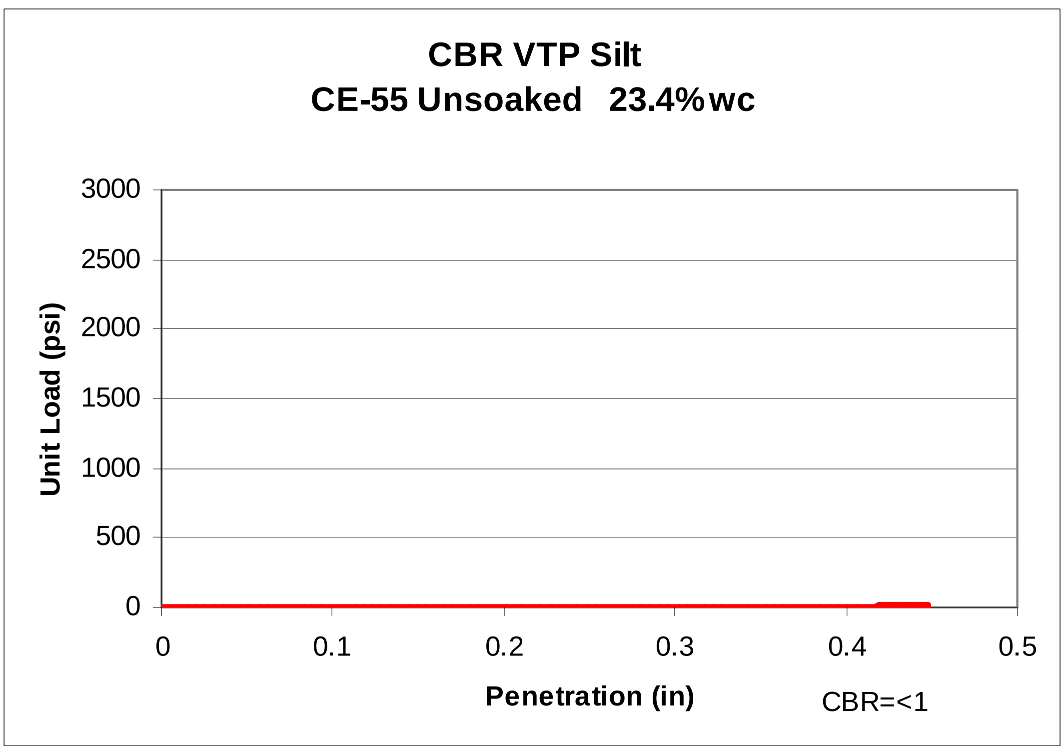
**MAXIMUM DRY DENSITY =** **115.5 pcf**

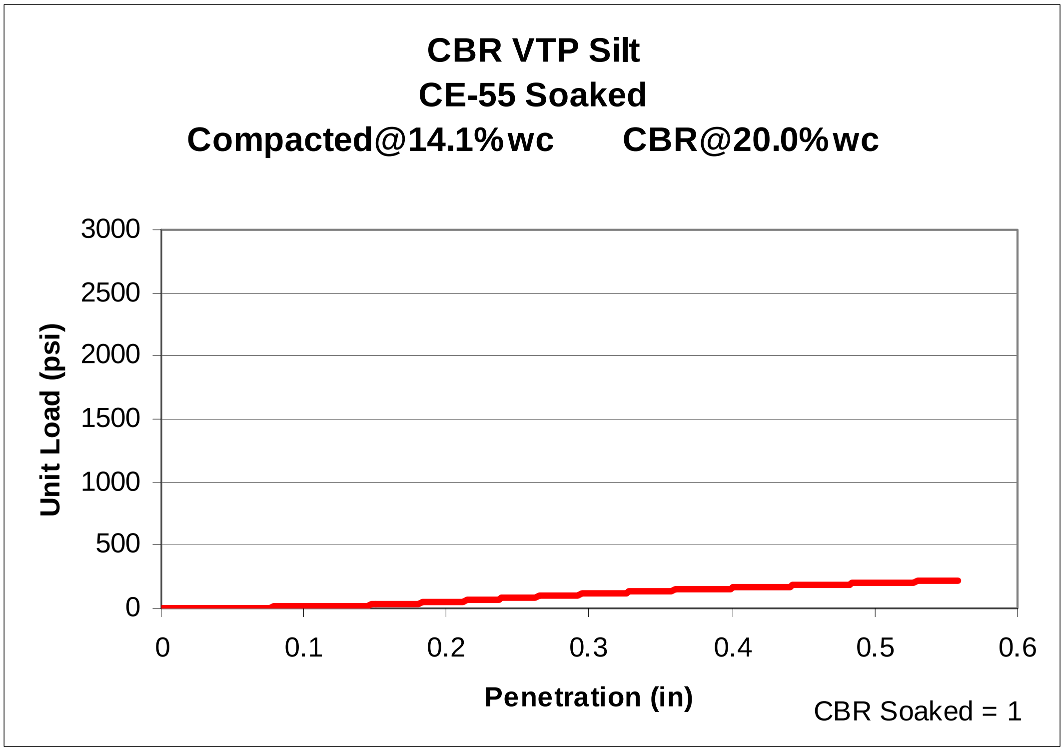
**OPTIMUM MOISTURE =** **15.50%**

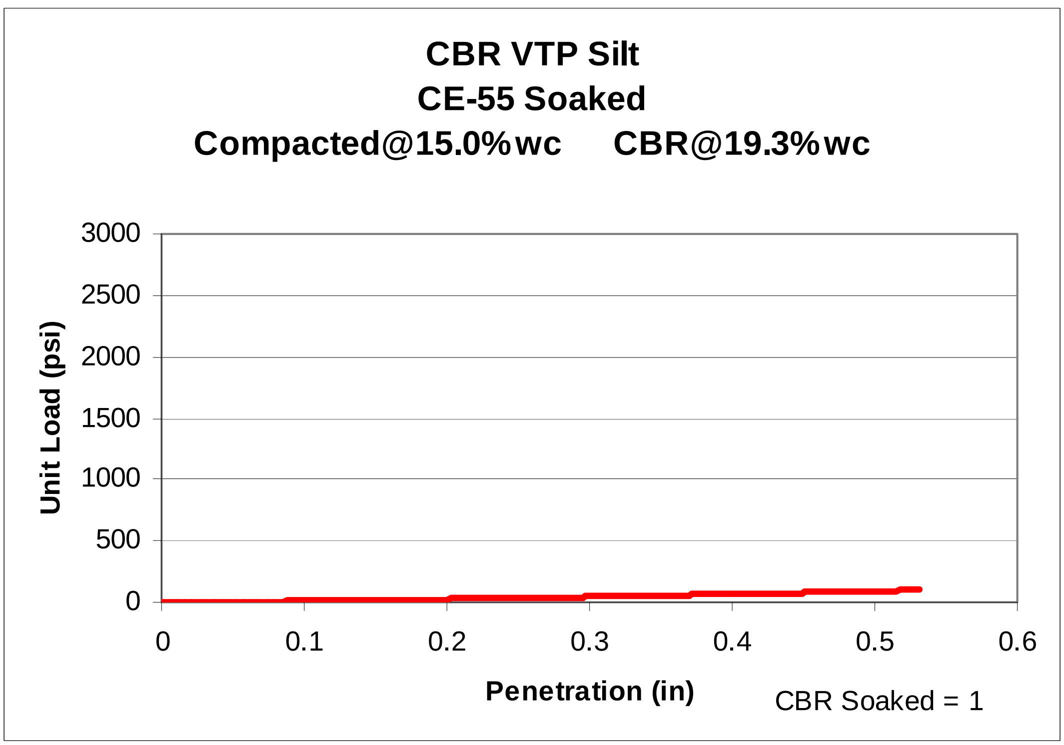

CBR VTP Silt
CE-55 Unsoaked 14.1%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR=44

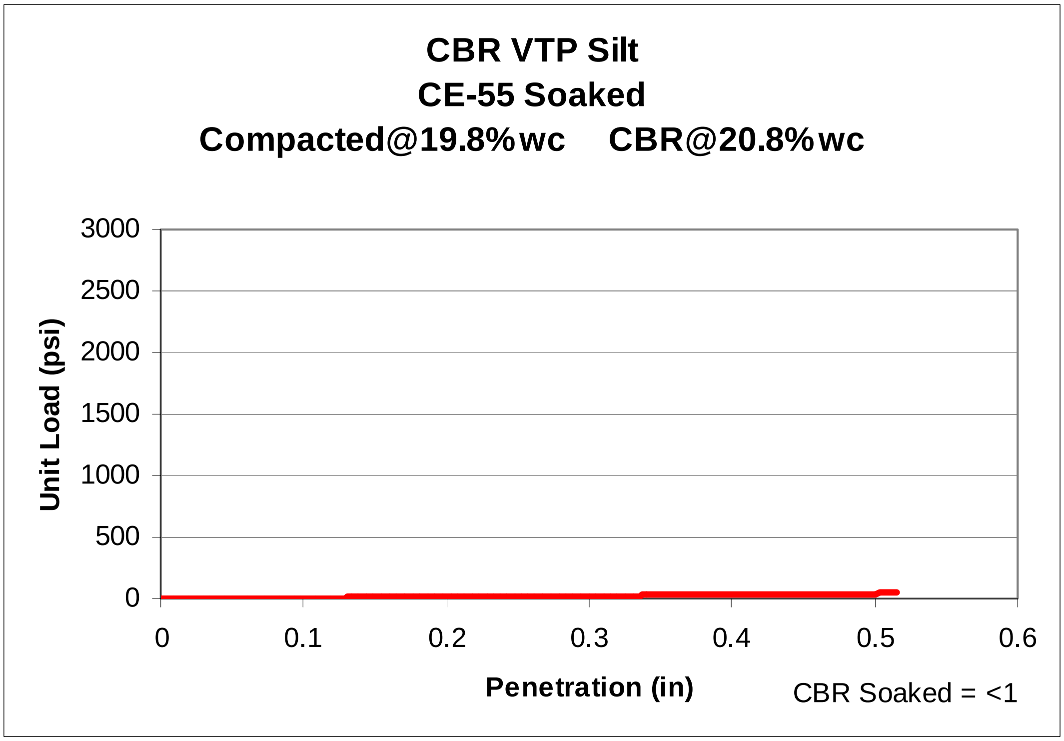

CBR VTP Silt
CE-55 Unsoaked 15.0%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR=39

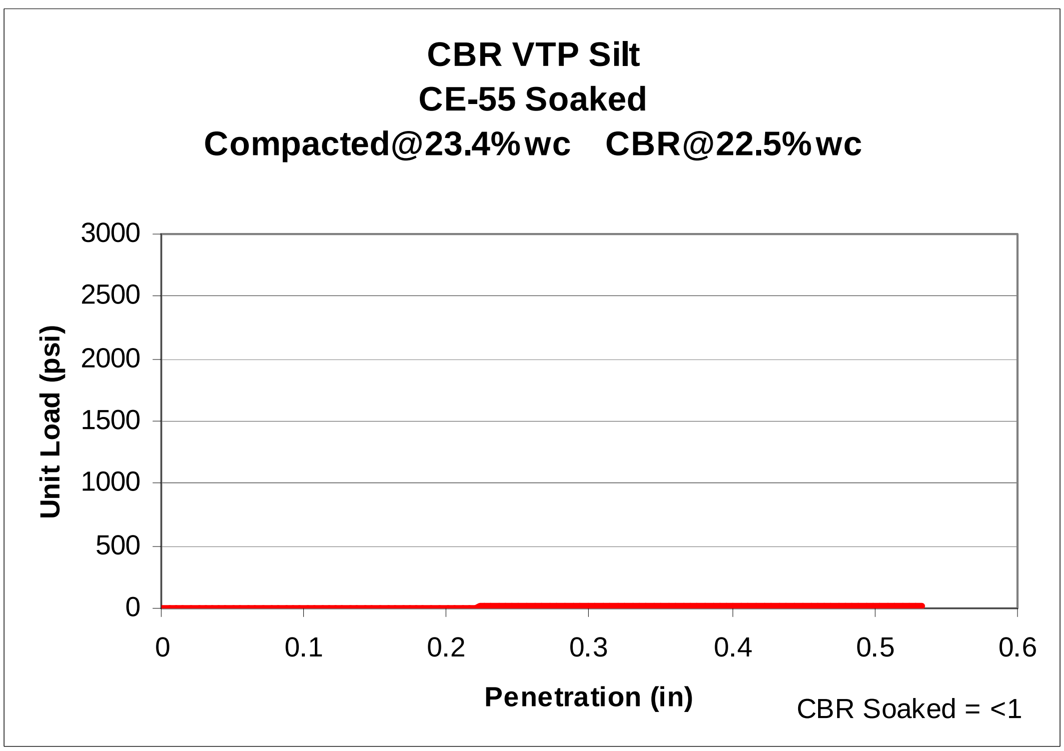

CBR VTP Silt
CE-55 Unsoaked 19.8% wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR=1


CBR VTP Silt
CE-55 Unsoaked 23.4% wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
Penetration (in)
CBR=<1


CBR VTP Silt
CE-55 Soaked
Compacted@14.1%wc CBR@20.0%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 1


CBR VTP Silt
CE-55 Soaked
Compacted@15.0%wc CBR@19.3%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = 1

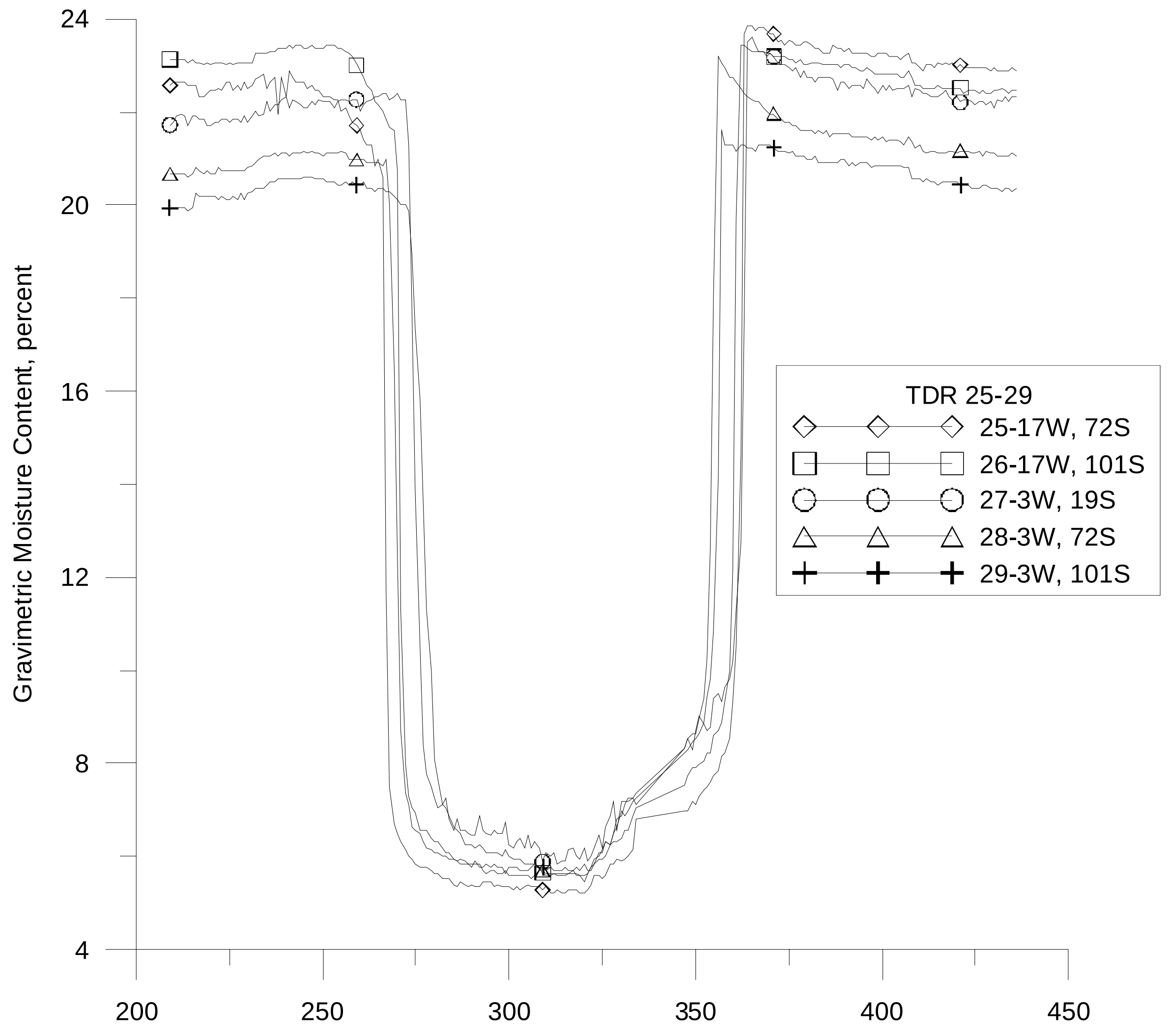

CBR VTP Silt
CE-55 Soaked
Compacted@19.8%wc CBR@20.8%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = <1


CBR VTP Silt
CE-55 Soaked
Compacted@23.4%wc CBR@22.5%wc
Unit Load (psi)
3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
Penetration (in)
CBR Soaked = <1

# Appendix B:
# Moisture Data

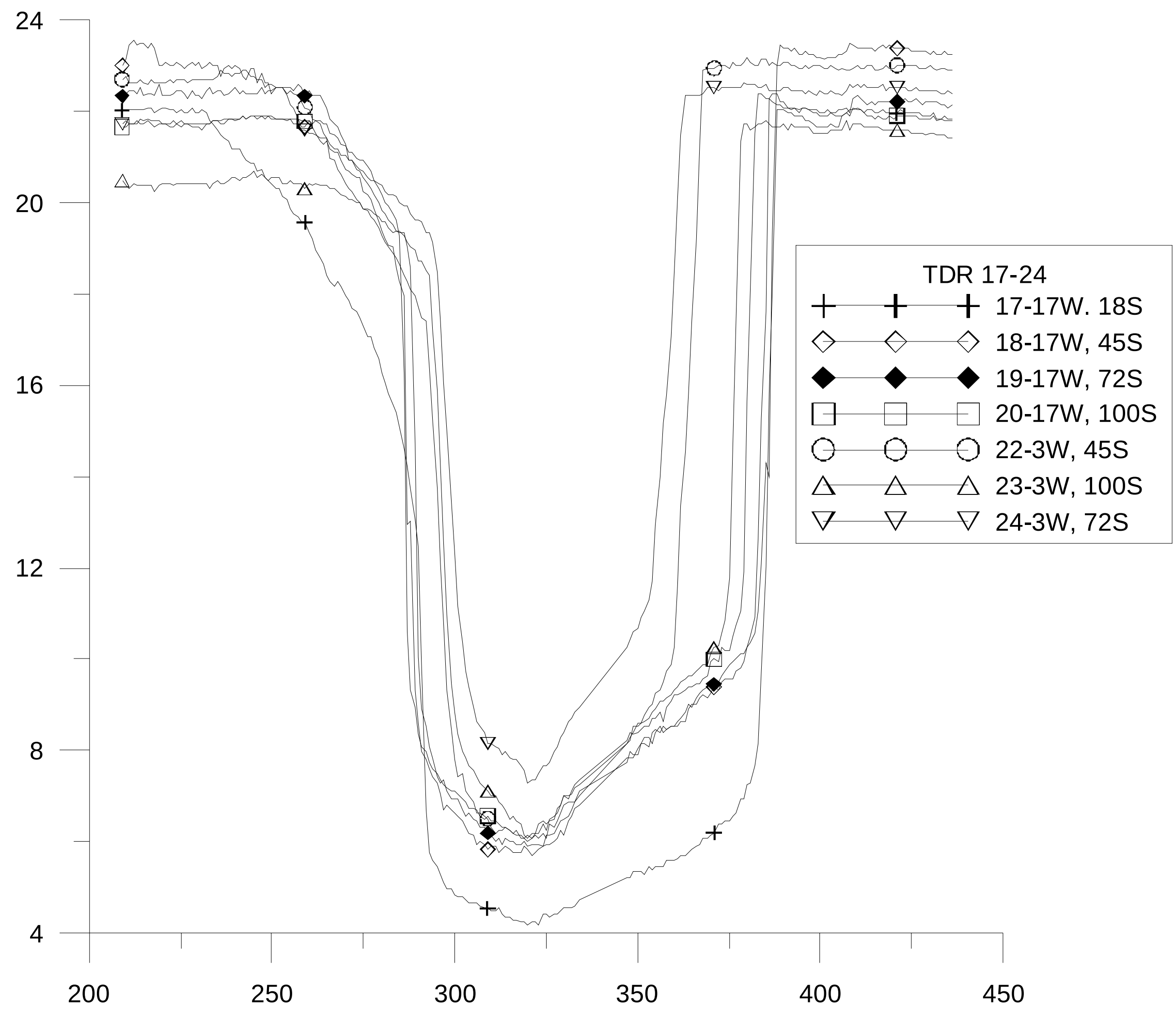
# Variable Tire Pressure

FERF 2000-01
TDR, 2.5 Ft Depth



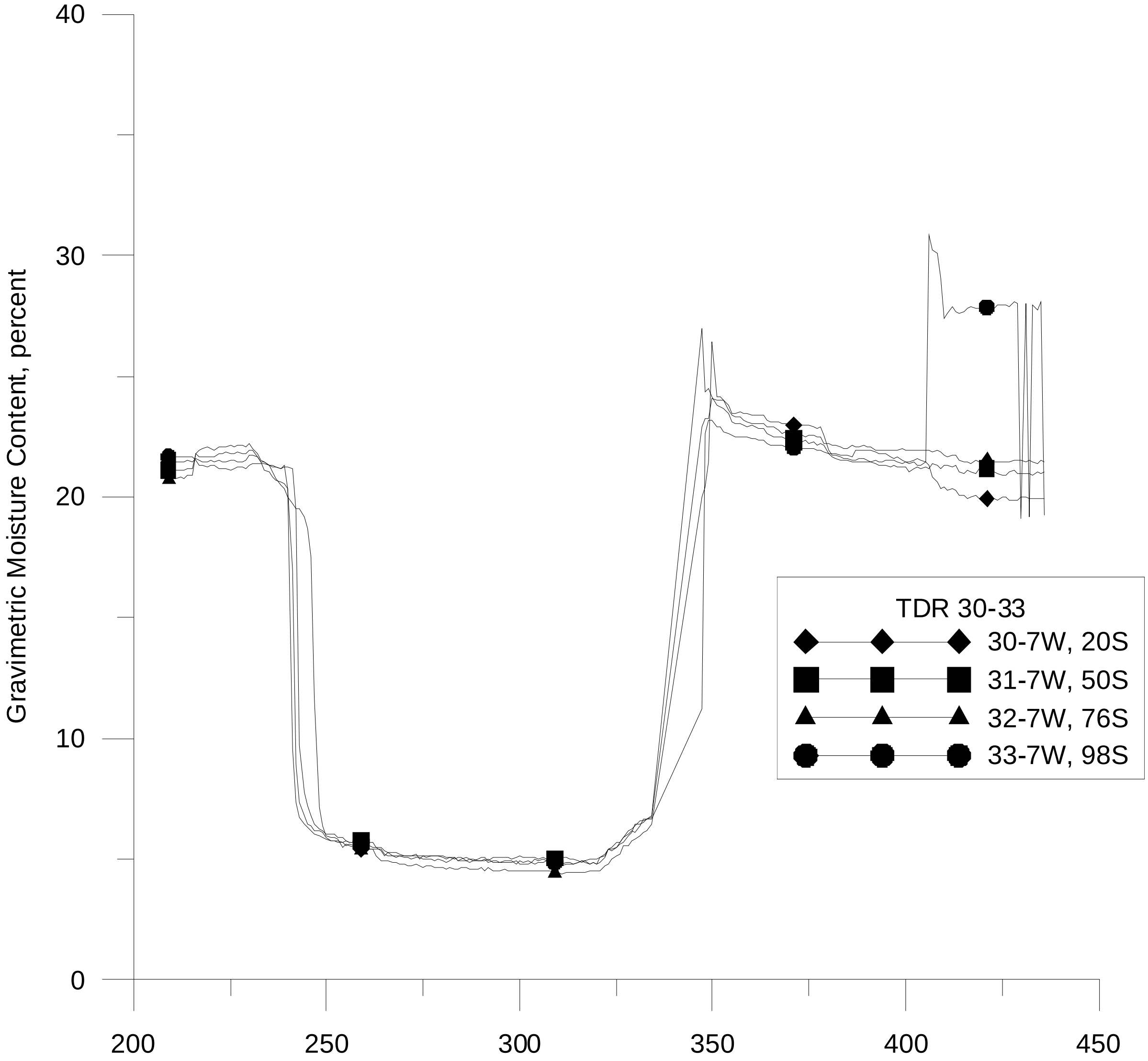
# Variable Tire Pressure

**FERF 2000-01**
**TDR, 4 Ft Depth**



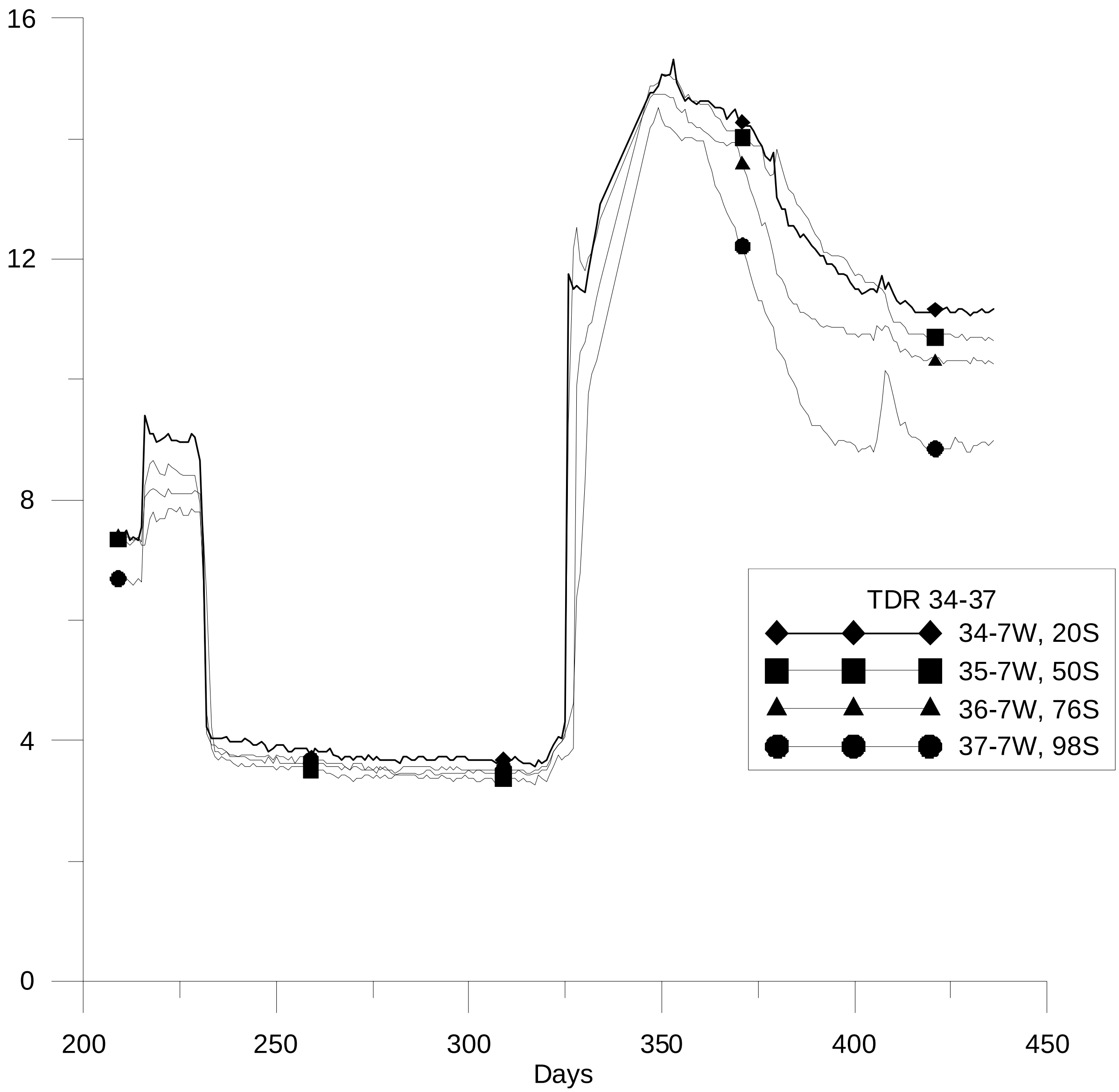
# Variable Tire Pressure

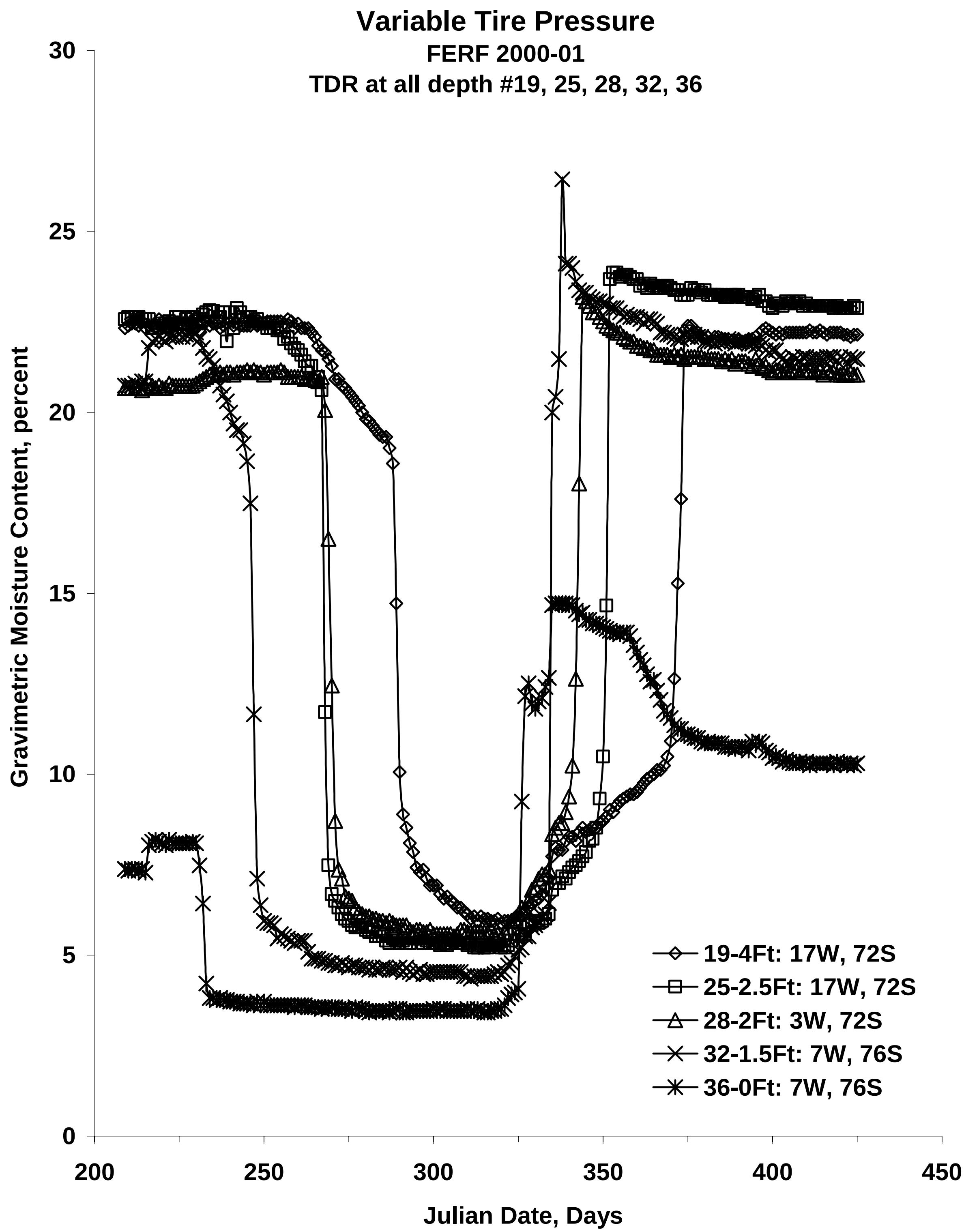
FERF 2000-01
TDR, 1.5 Ft Depth



# Variable Tire Pressure

FERF 2000-01
TDR, 0.6 Ft Depth



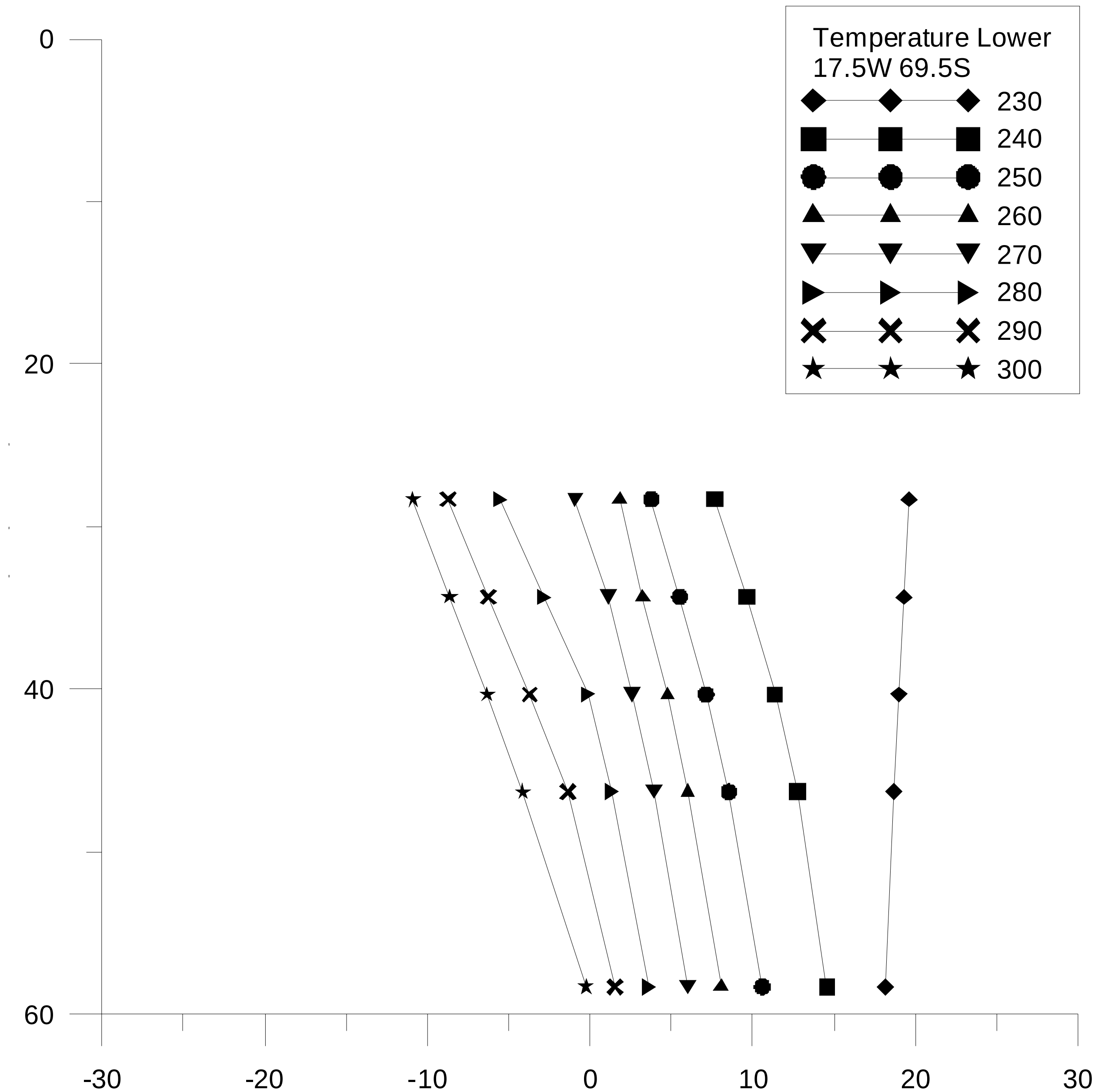

Variable Tire Pressure
FERF 2000-01
TDR at all depth #19, 25, 28, 32, 36
Gravimetric Moisture Content, percent
Julian Date, Days
0
5
10
15
20
25
30
200
250
300
350
400
450
19-4Ft: 17W, 72S
25-2.5Ft: 17W, 72S
28-2Ft: 3W, 72S
32-1.5Ft: 7W, 76S
36-0Ft: 7W, 76S

# Appendix C:
# Temperature Data

# Variable Tire Pressure

FERF 2000-01

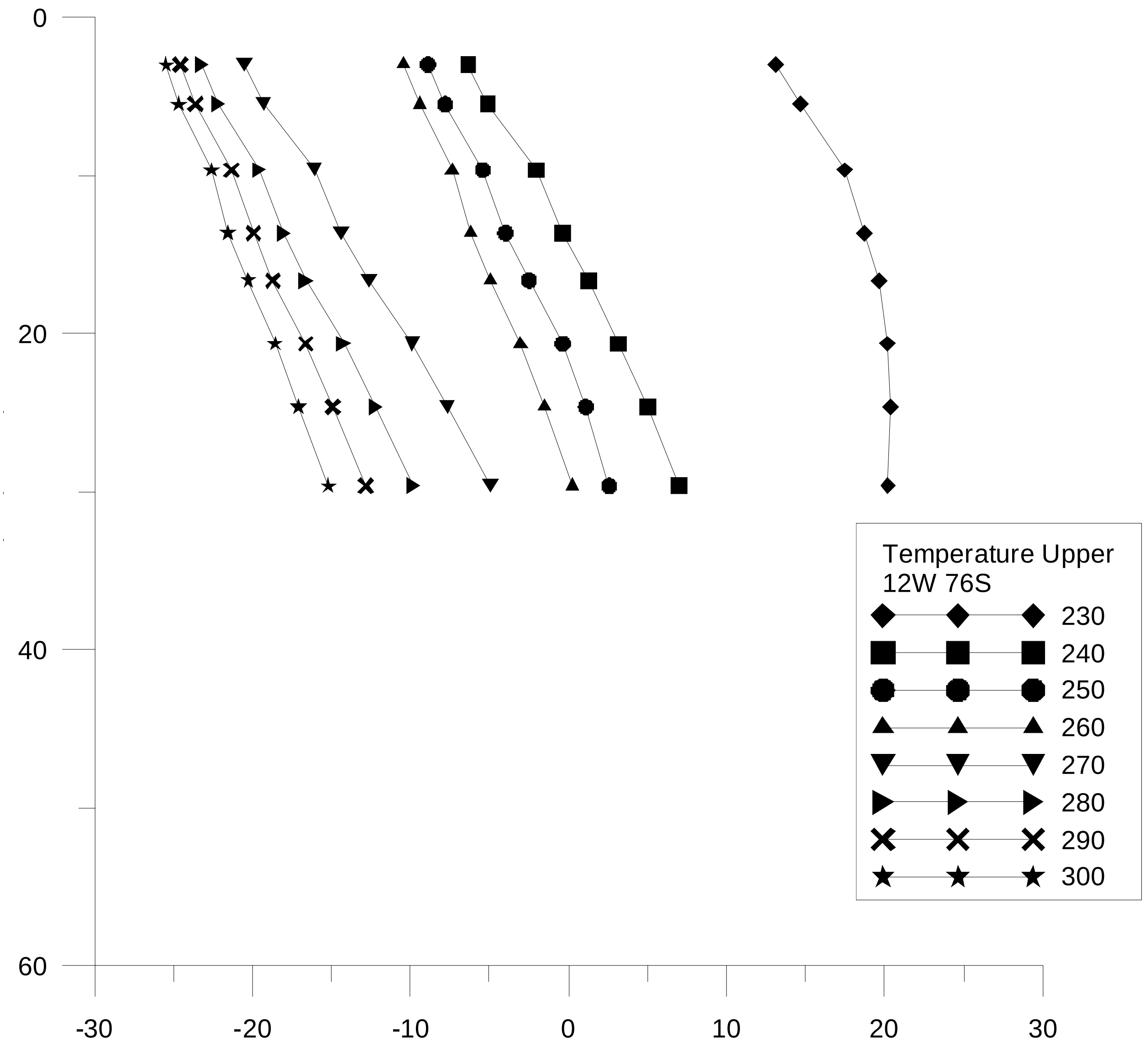
Depth Vs. Temperature Lower 17.5Ft-W, 69.5-S



# Variable Tire Pressure

FERF 2000-01

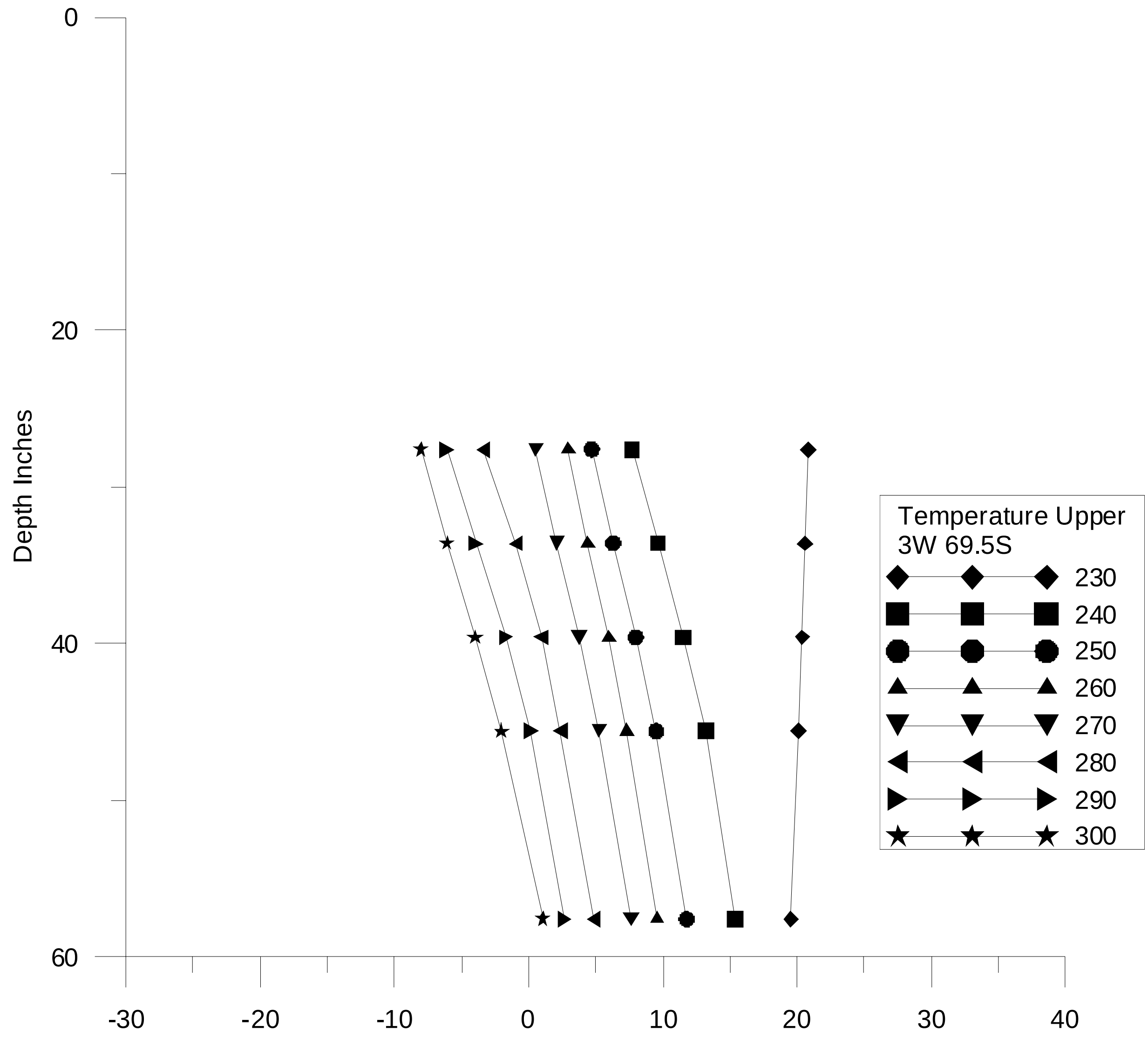
Depth Vs Temperature Upper 12Ft-W, 76Ft-S



# Variable Tire Pressure

FERF 2000-01

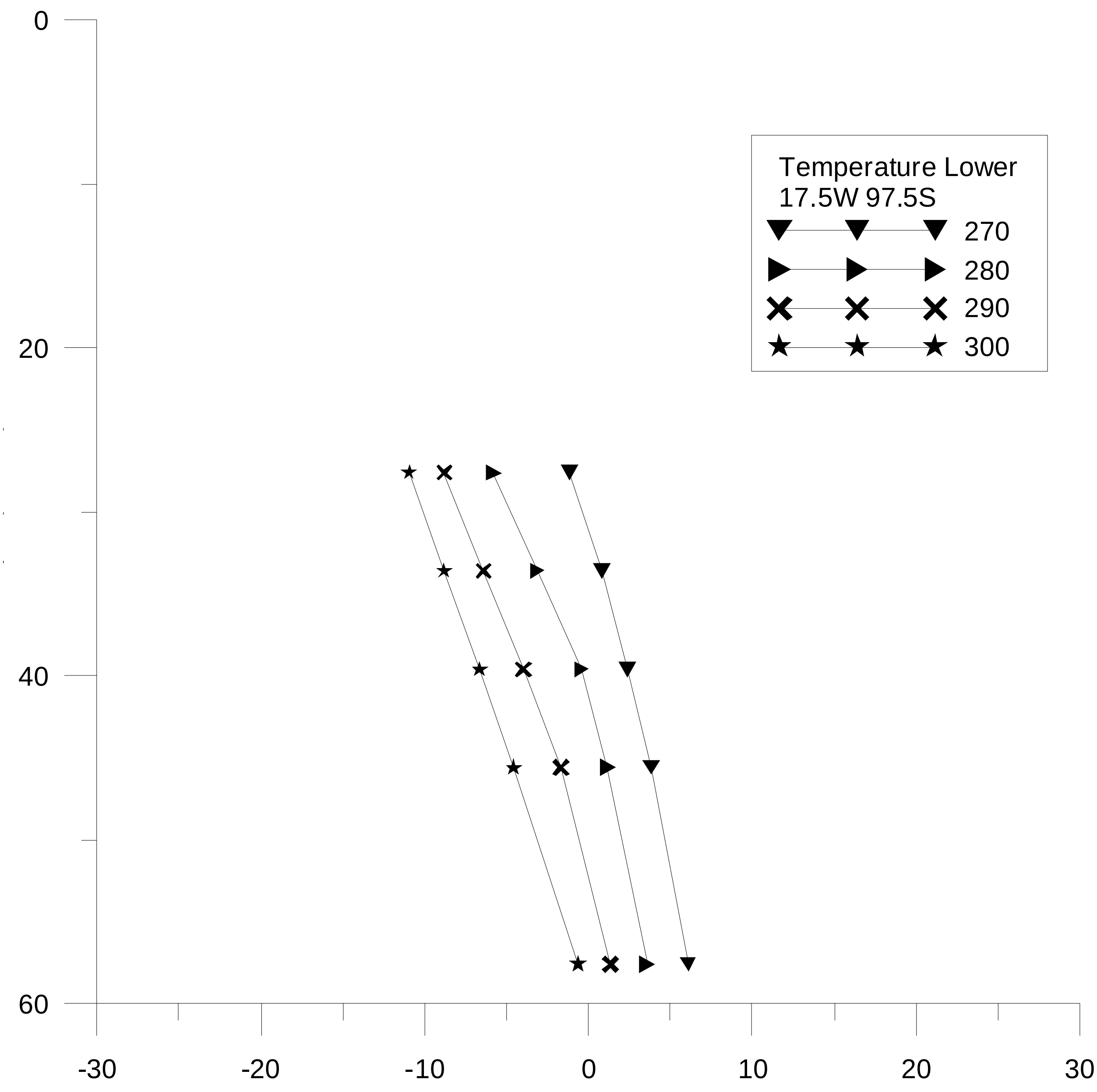
Depth Vs Temperature Lower 3Ft-W, 69.5Ft-S



# Variable Tire Pressure

FERF 2000-01

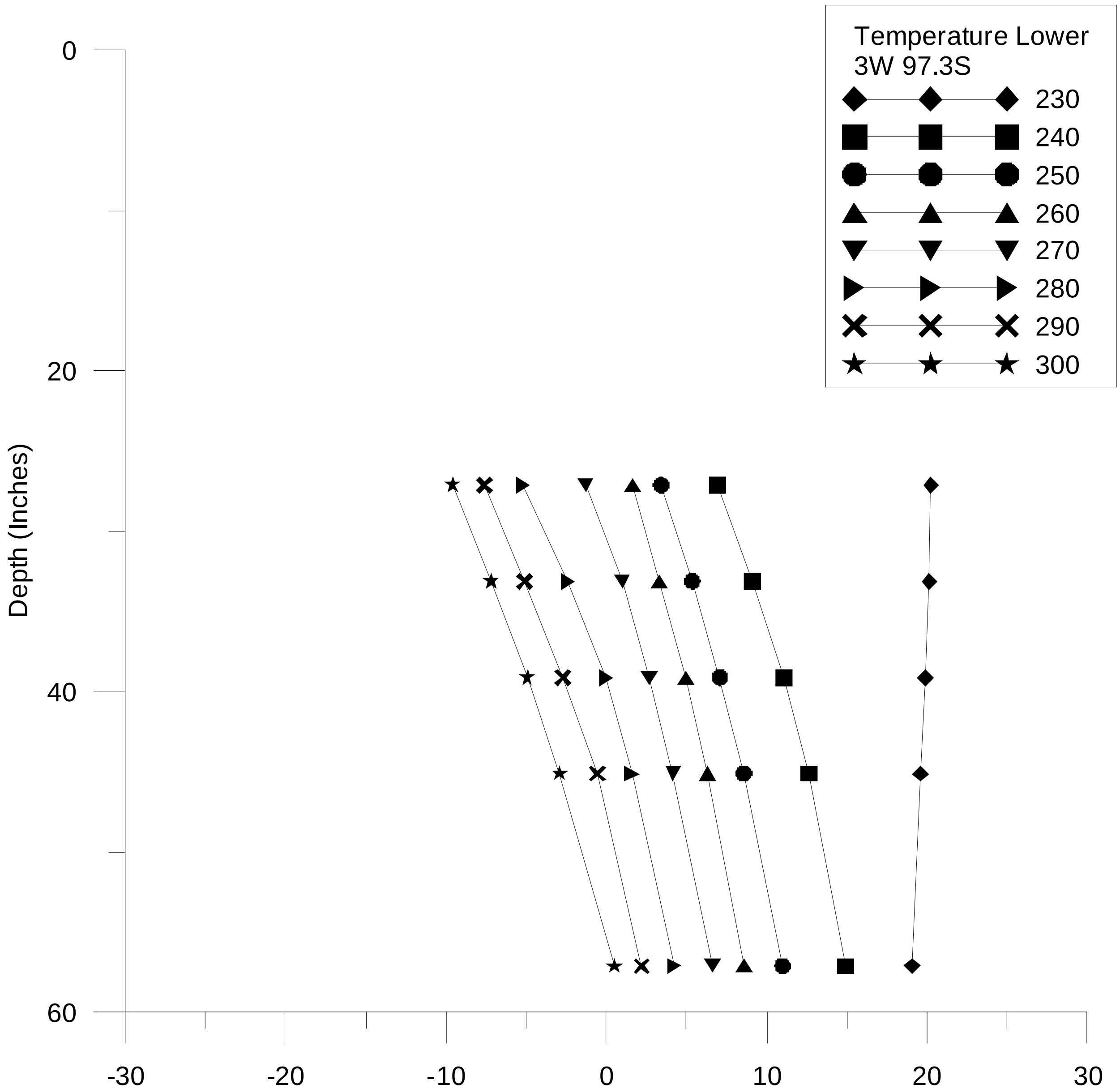
Depth Vs. Temperature Lower 17.5Ft-W, 97.3Ft-S



# Variable Tire Pressure

FERF 2000-01

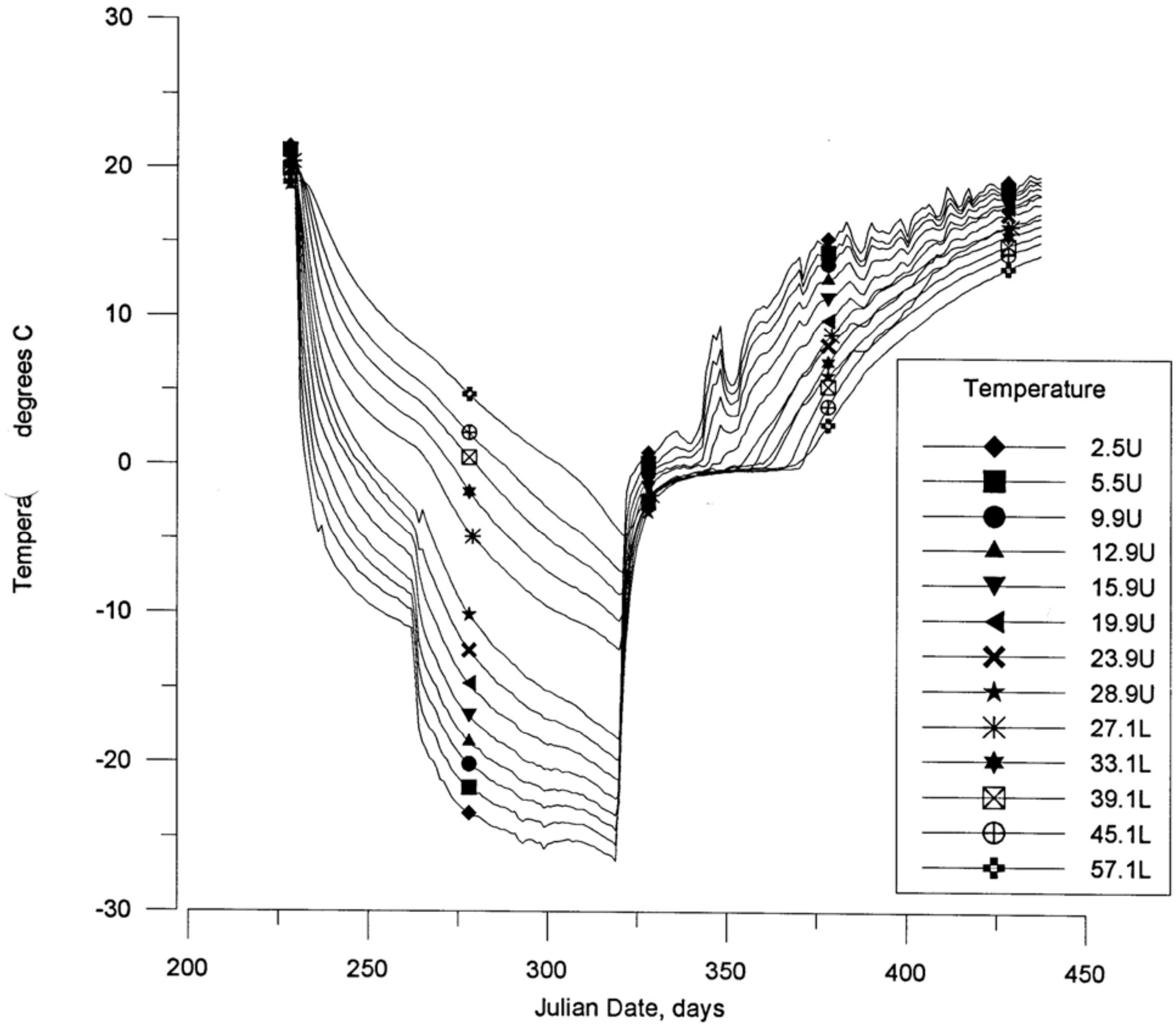
Depth Vs. Temperature Lower 3Ft-W, 97.3Ft-S

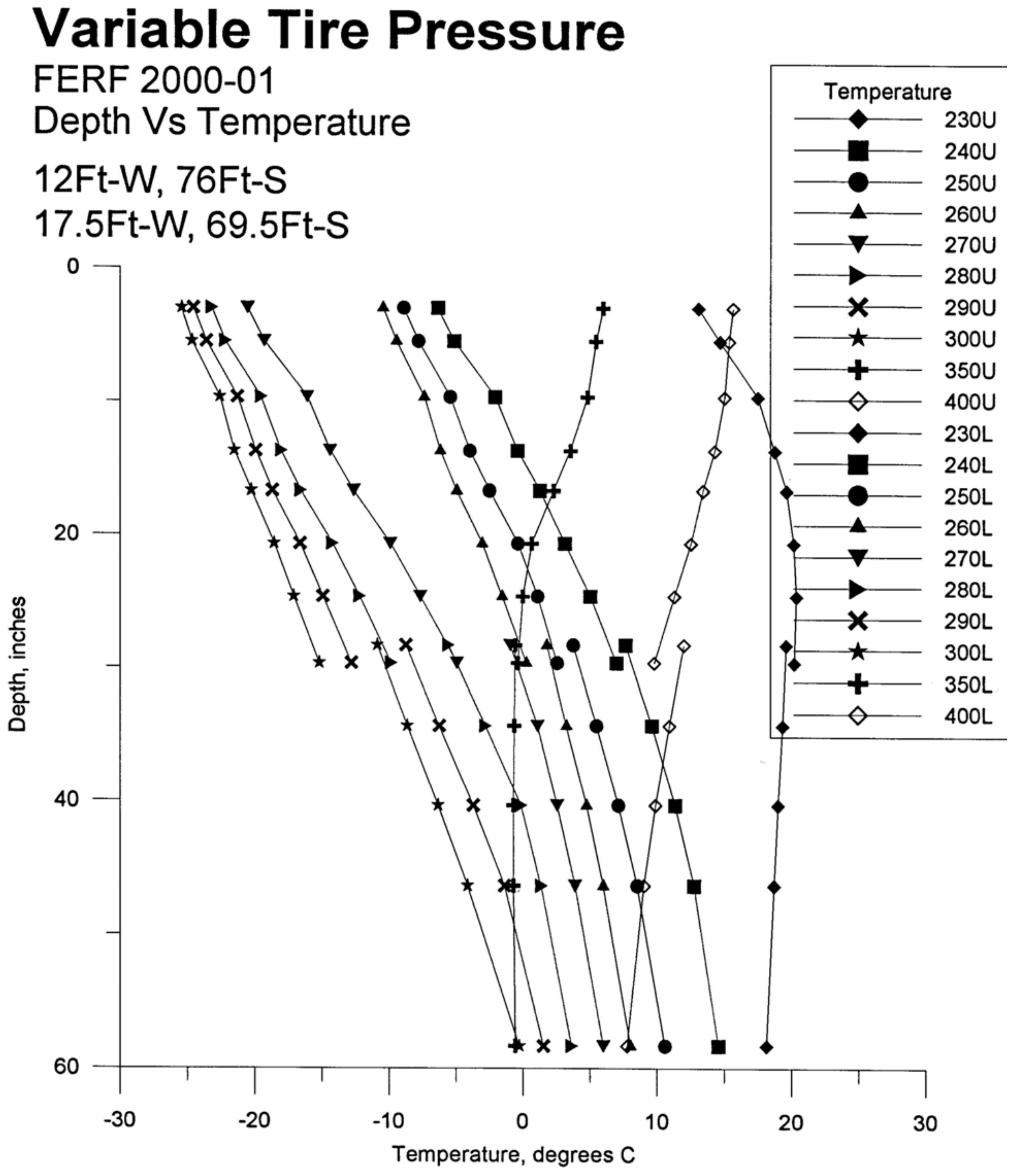


# Variable Tire Pressure

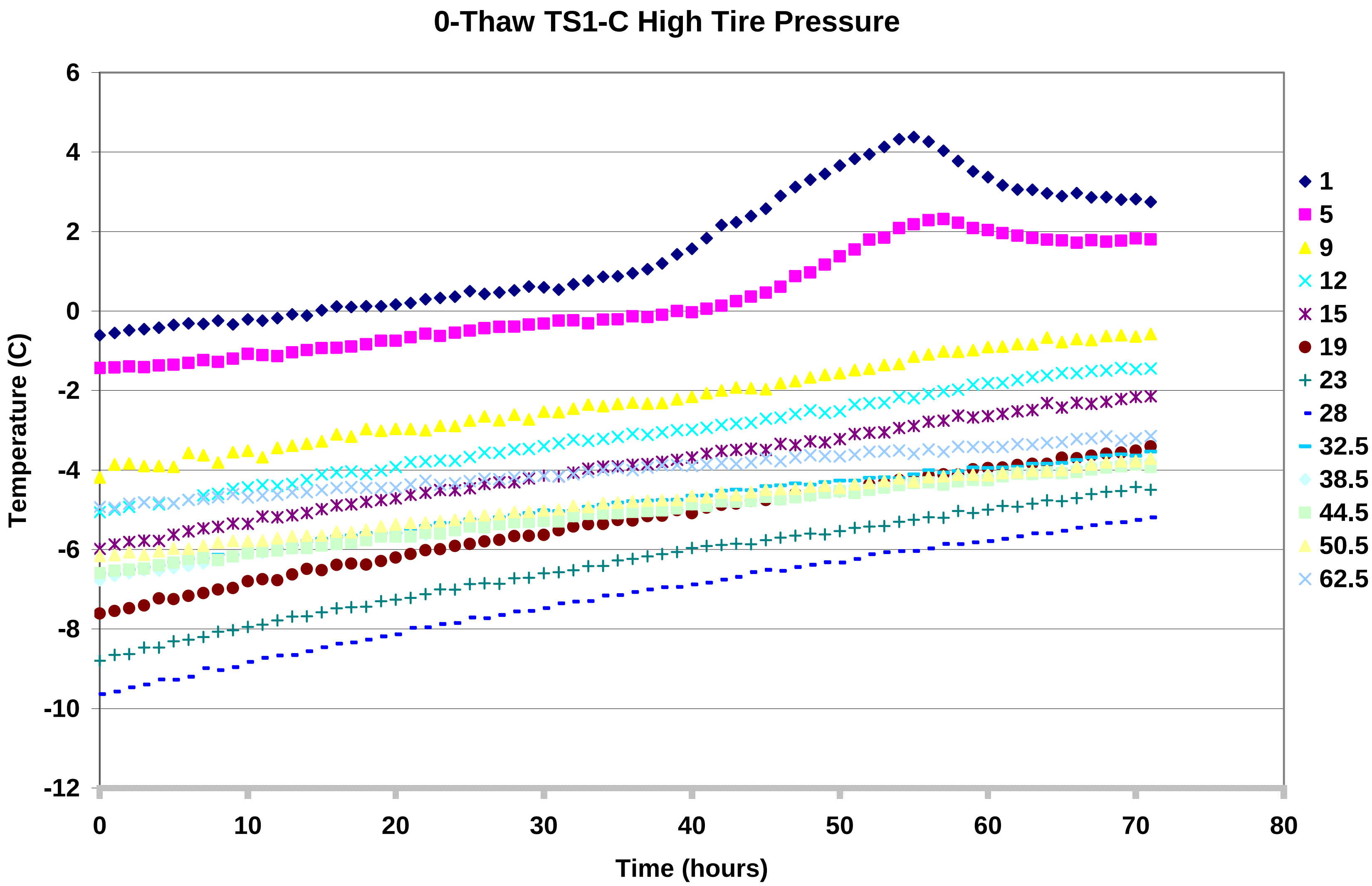
FERF 2000-01

Temperature- 12Ft-W, 98Ft-S and 3Ft-W, 97.3Ft-S

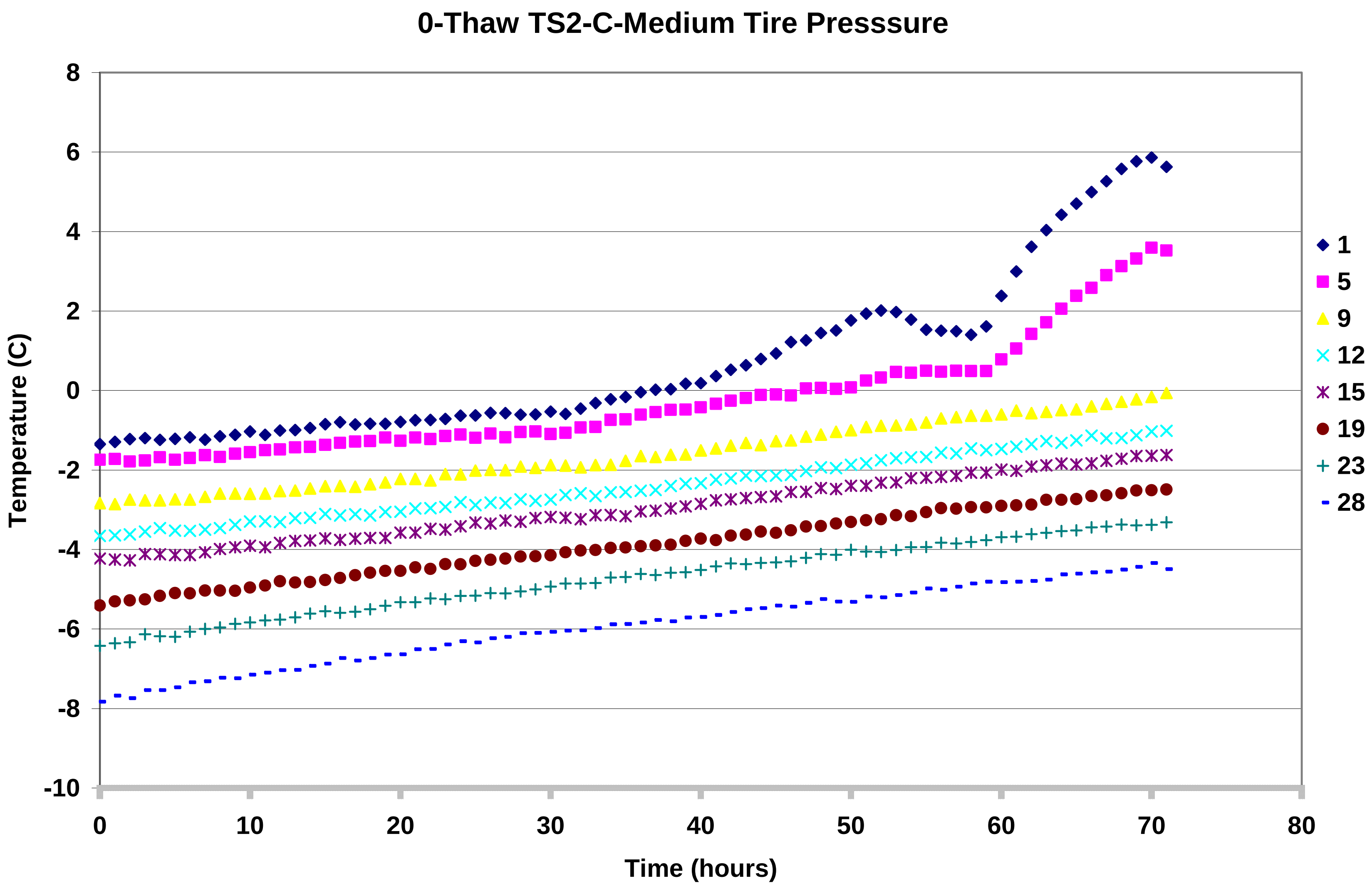


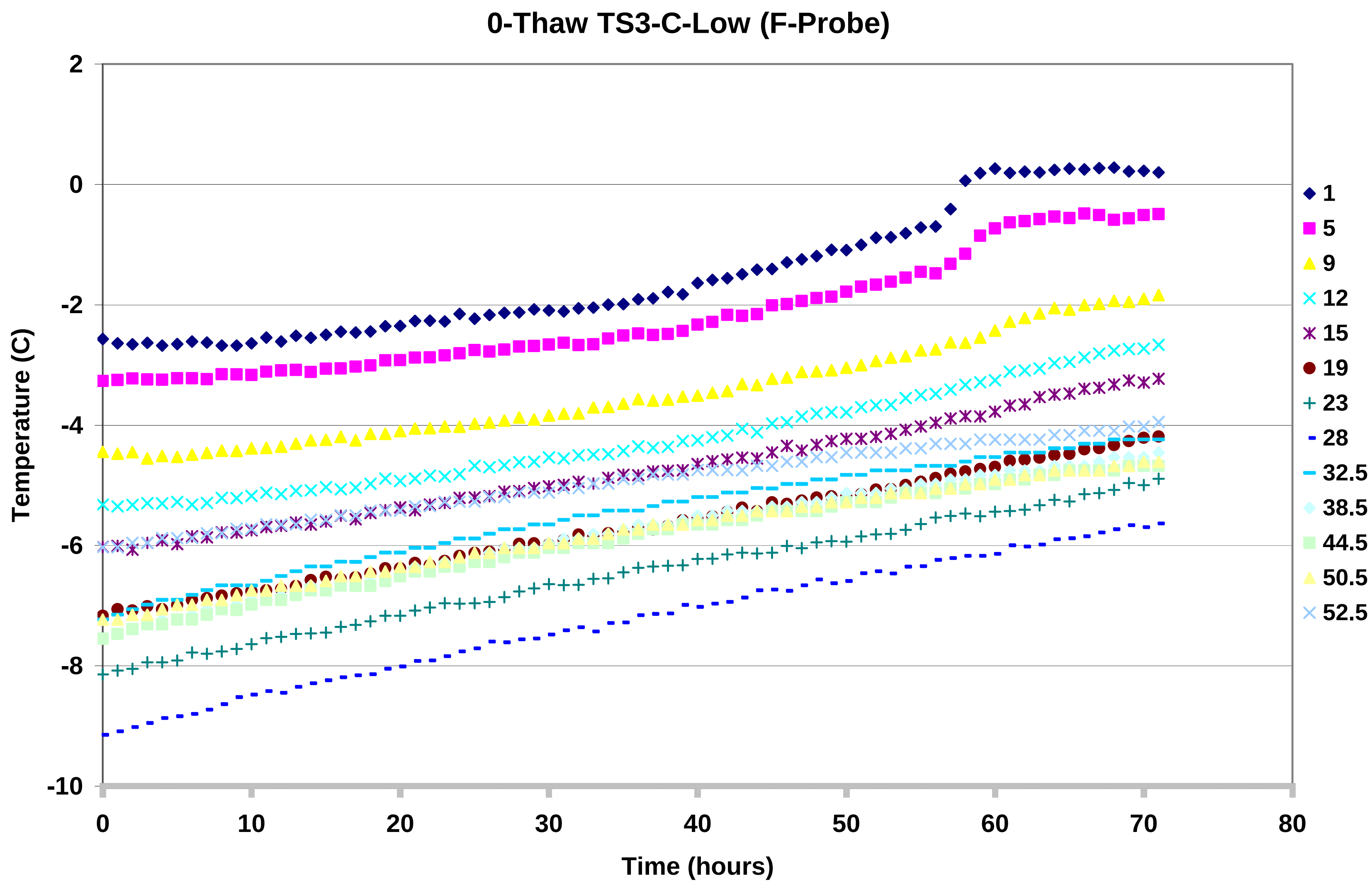


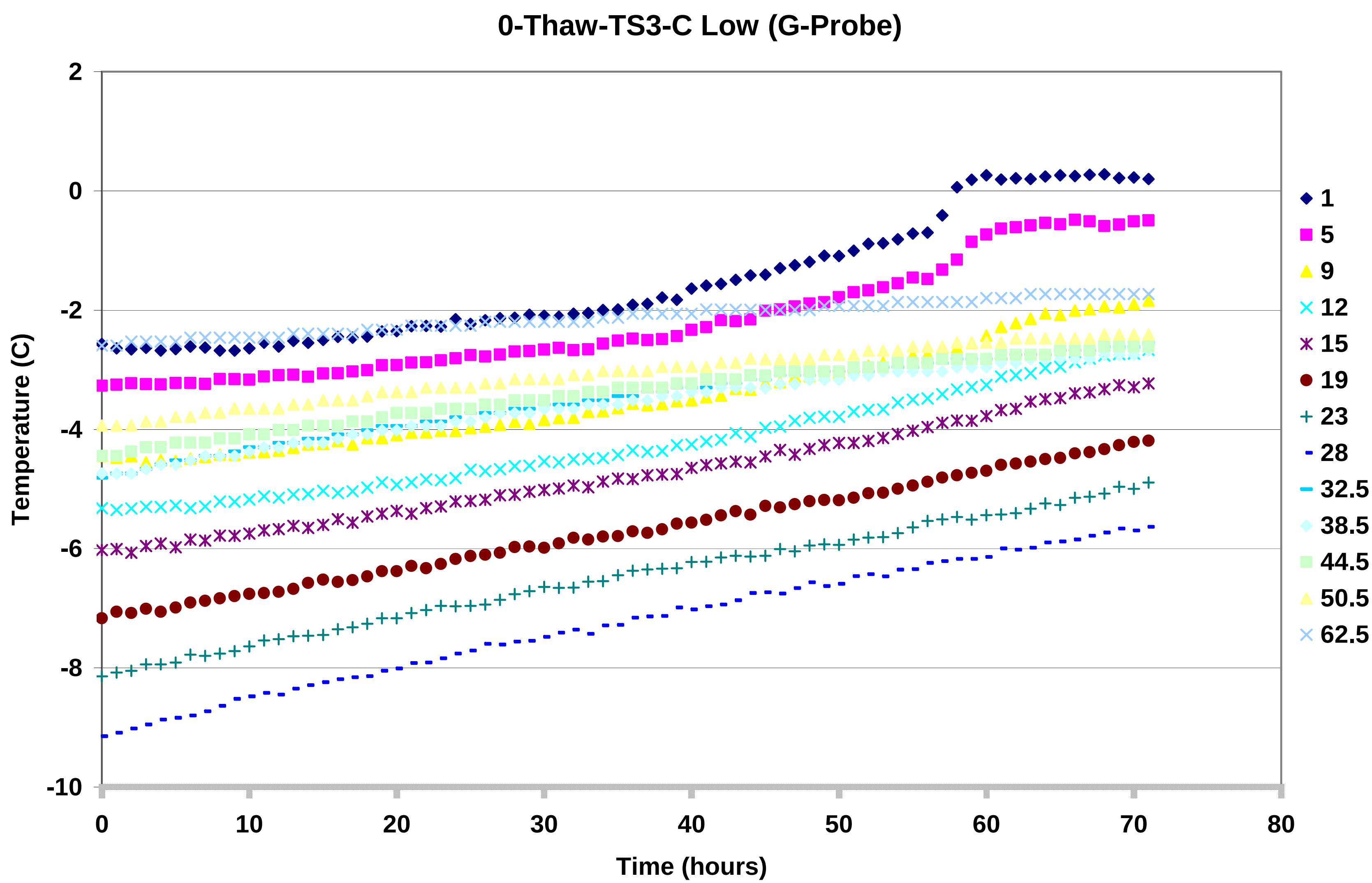
Variable Tire Pressure
FERF 2000-01
Depth Vs Temperature
12Ft-W, 76Ft-S
17.5Ft-W, 69.5Ft-S
Temperature
230U
240U
250U
260U
270U
280U
290U
300U
350U
400U
230L
240L
250L
260L
270L
280L
290L
300L
350L
400L
0
20
40
60
Depth, inches
-30
-20
-10
0
10
20
30
Temperature, degrees C



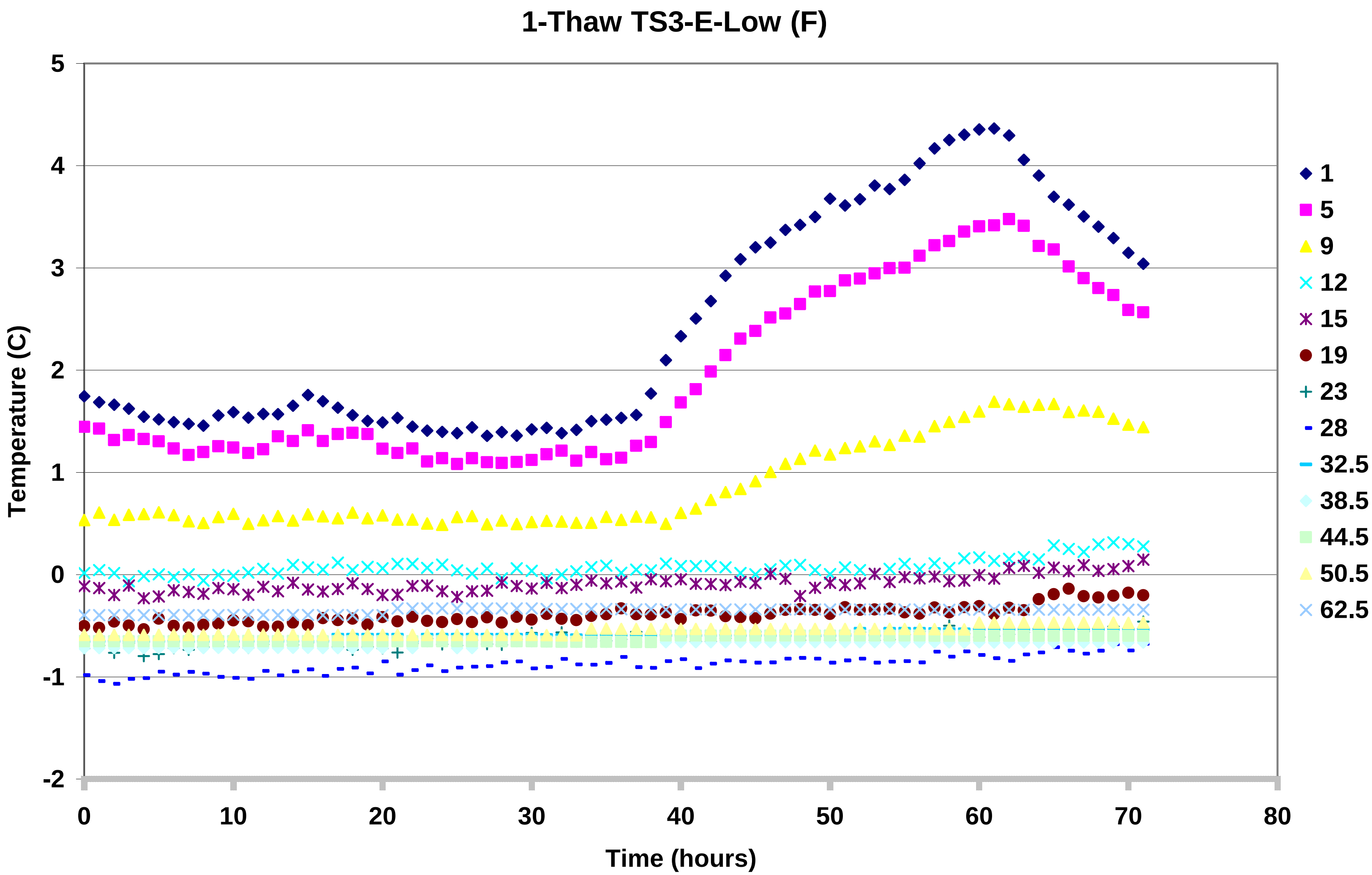
0-Thaw TS1-C High Tire Pressure
Temperature (C)
6
4
2
0
-2
-4
-6
-8
-10
-12
0
10
20
30
40
50
60
70
80
Time (hours)
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

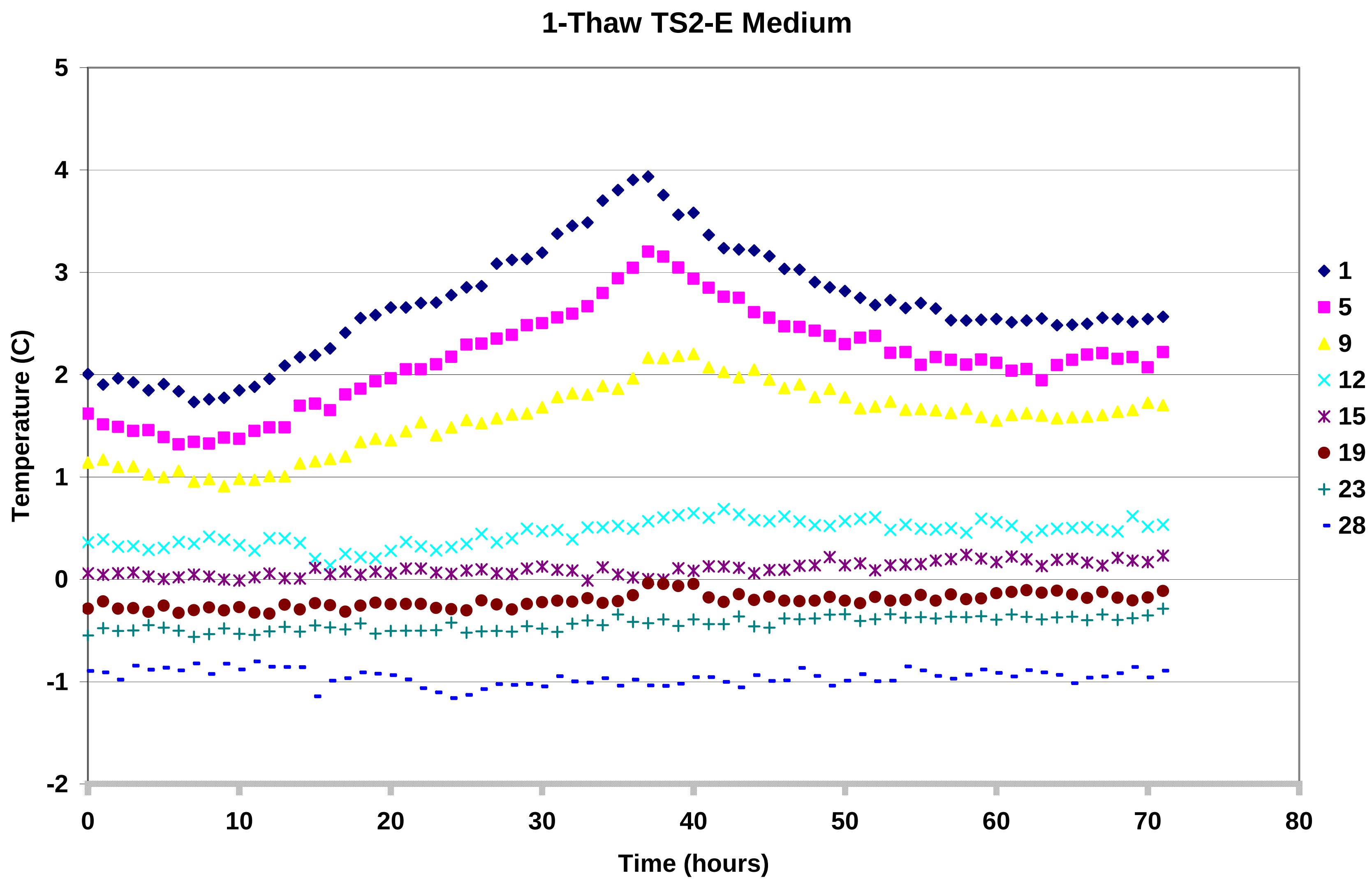

0-Thaw TS2-C-Medium Tire Presssure
Temperature (C)
8
6
4
2
0
-2
-4
-6
-8
-10
0
10
20
30
40
50
60
70
80
Time (hours)
1
5
9
12
15
19
23
28

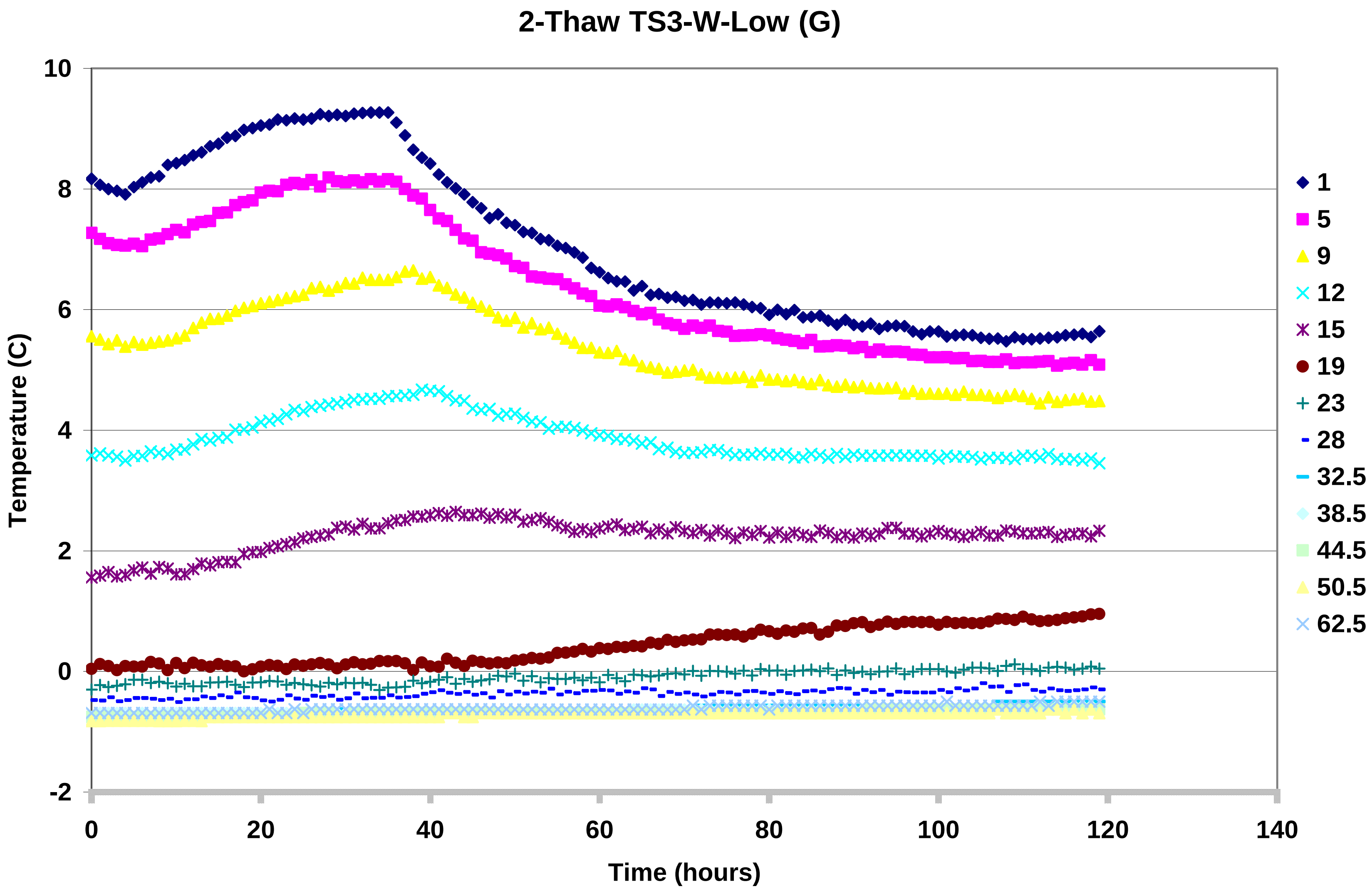

0-Thaw TS3-C-Low (F-Probe)
Temperature (C)
Time (hours)
2
0
-2
-4
-6
-8
-10
0
10
20
30
40
50
60
70
80
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
52.5



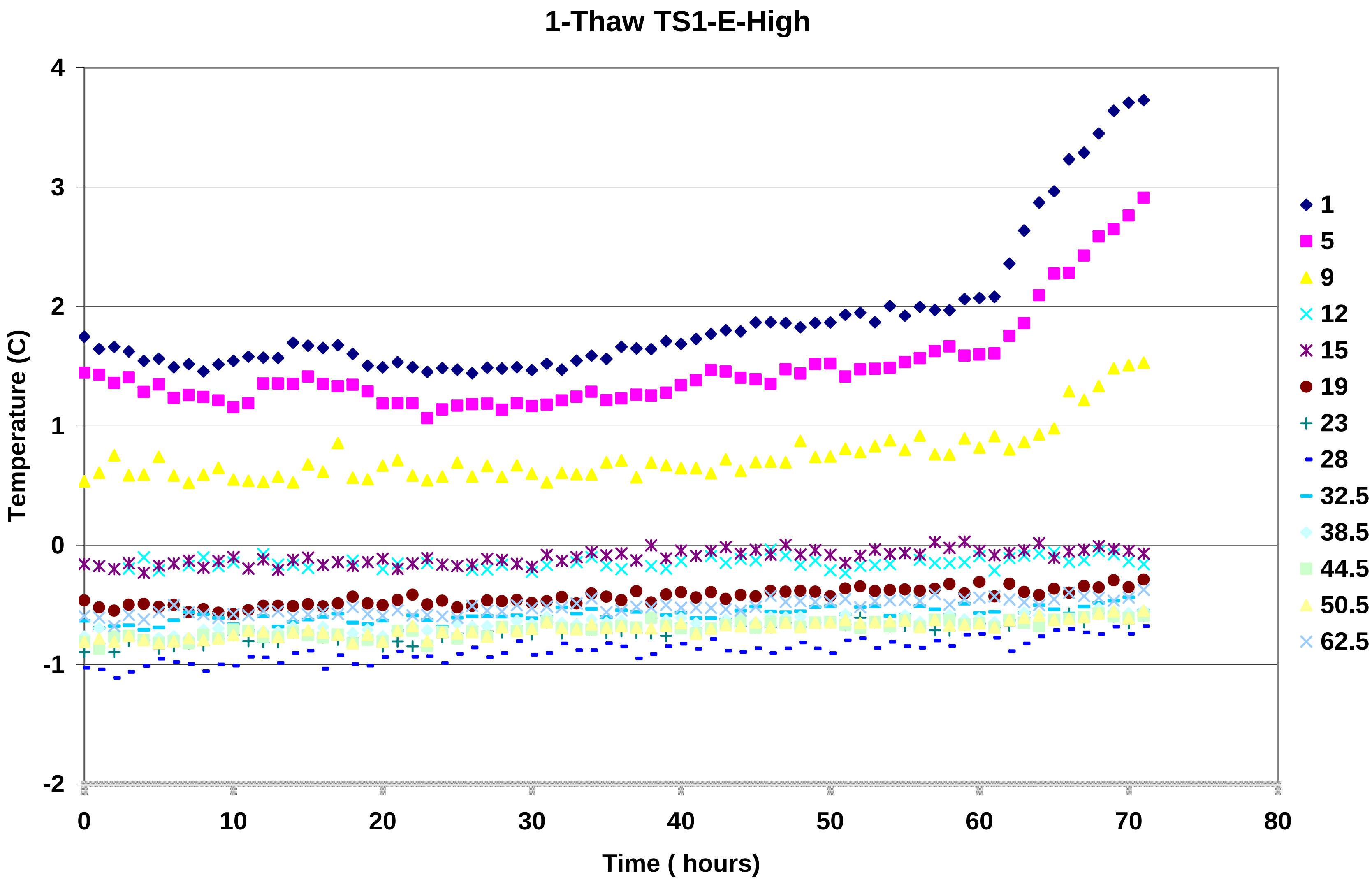
0-Thaw-TS3-C Low (G-Probe)
Temperature (C)
Time (hours)
2
0
-2
-4
-6
-8
-10
0
10
20
30
40
50
60
70
80
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

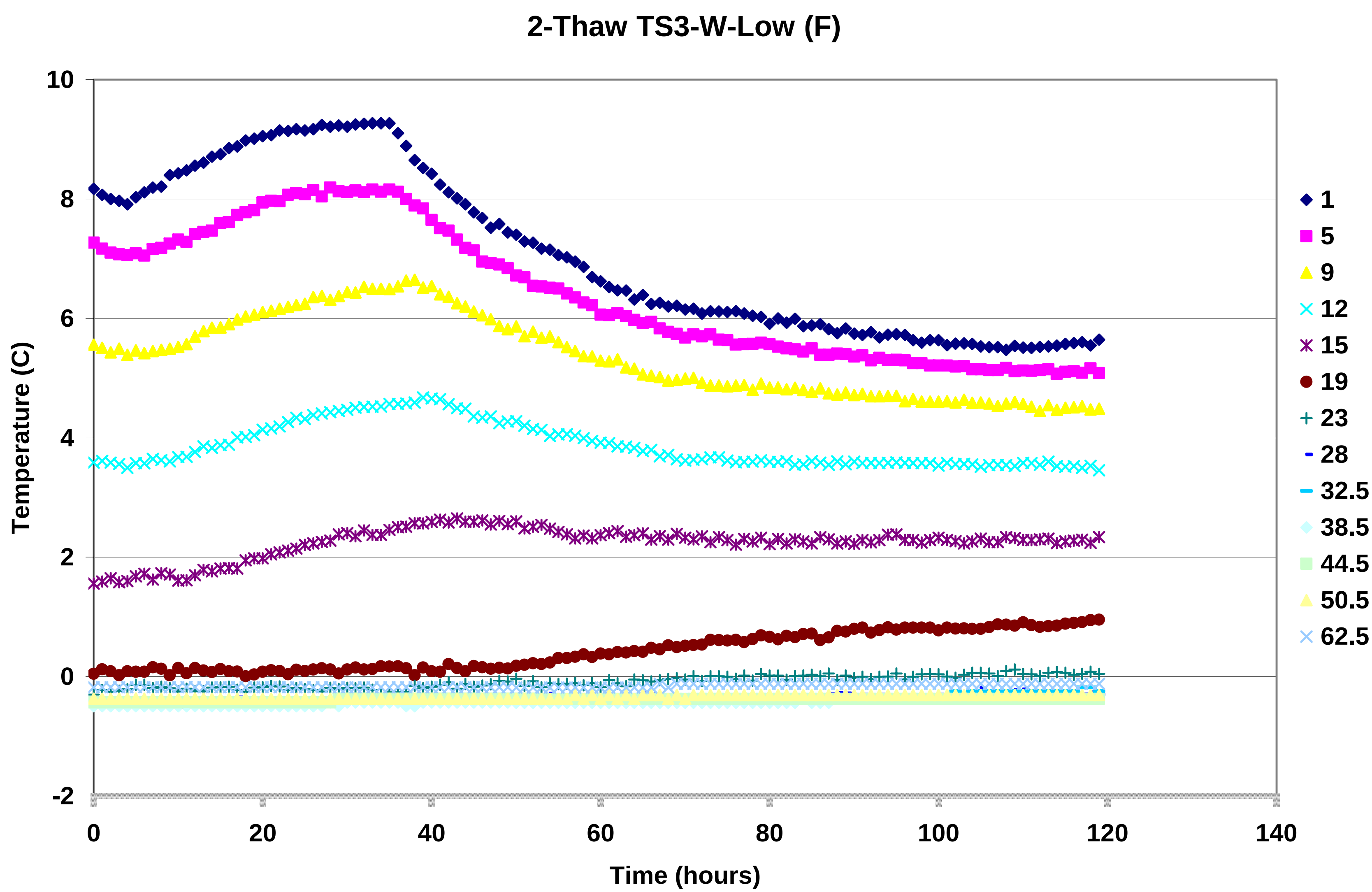

1-Thaw TS3-E-Low (F)
Temperature (C)
Time (hours)
5
4
3
2
1
0
-1
-2
0
10
20
30
40
50
60
70
80
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

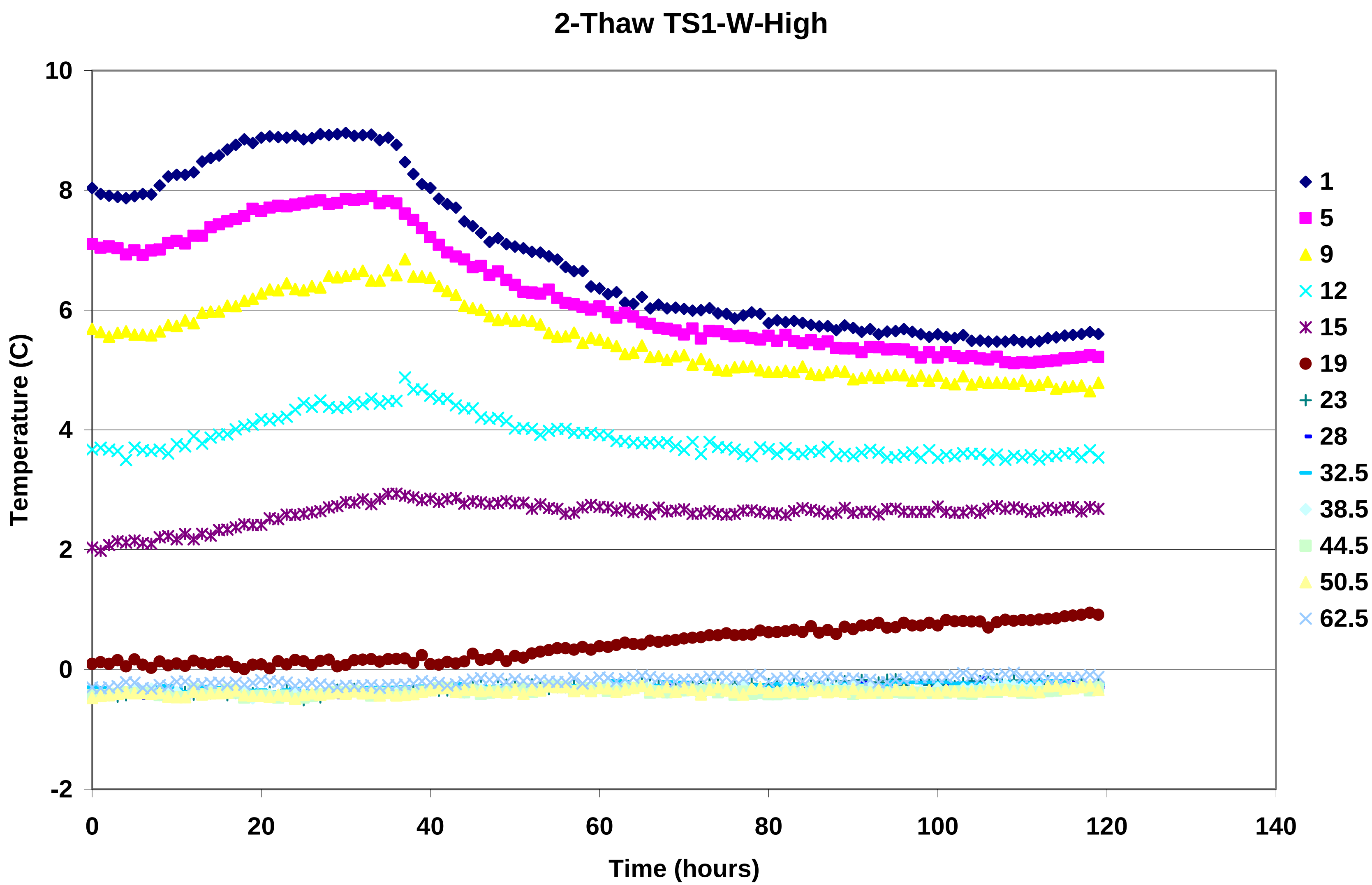

1-Thaw TS2-E Medium
Temperature (C)
Time (hours)
5
4
3
2
1
0
-1
-2
0
10
20
30
40
50
60
70
80
1
5
9
12
15
19
23
28



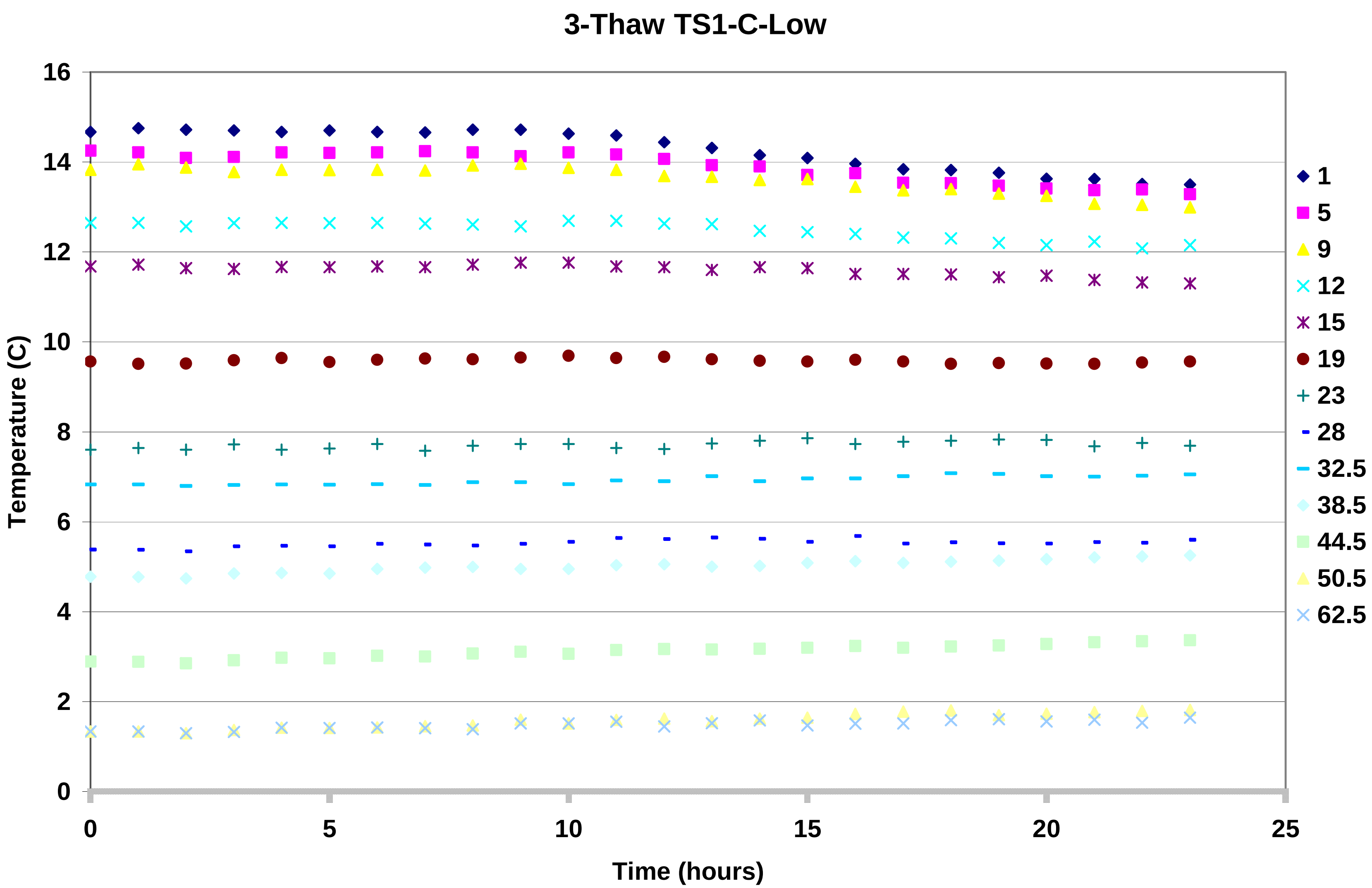
2-Thaw TS3-W-Low (G)
Temperature (C)
Time (hours)
10
8
6
4
2
0
-2
0
20
40
60
80
100
120
140
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5


1-Thaw TS1-E-High
Temperature (C)
4
3
2
1
0
-1
-2
0
10
20
30
40
50
60
70
80
Time ( hours)
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

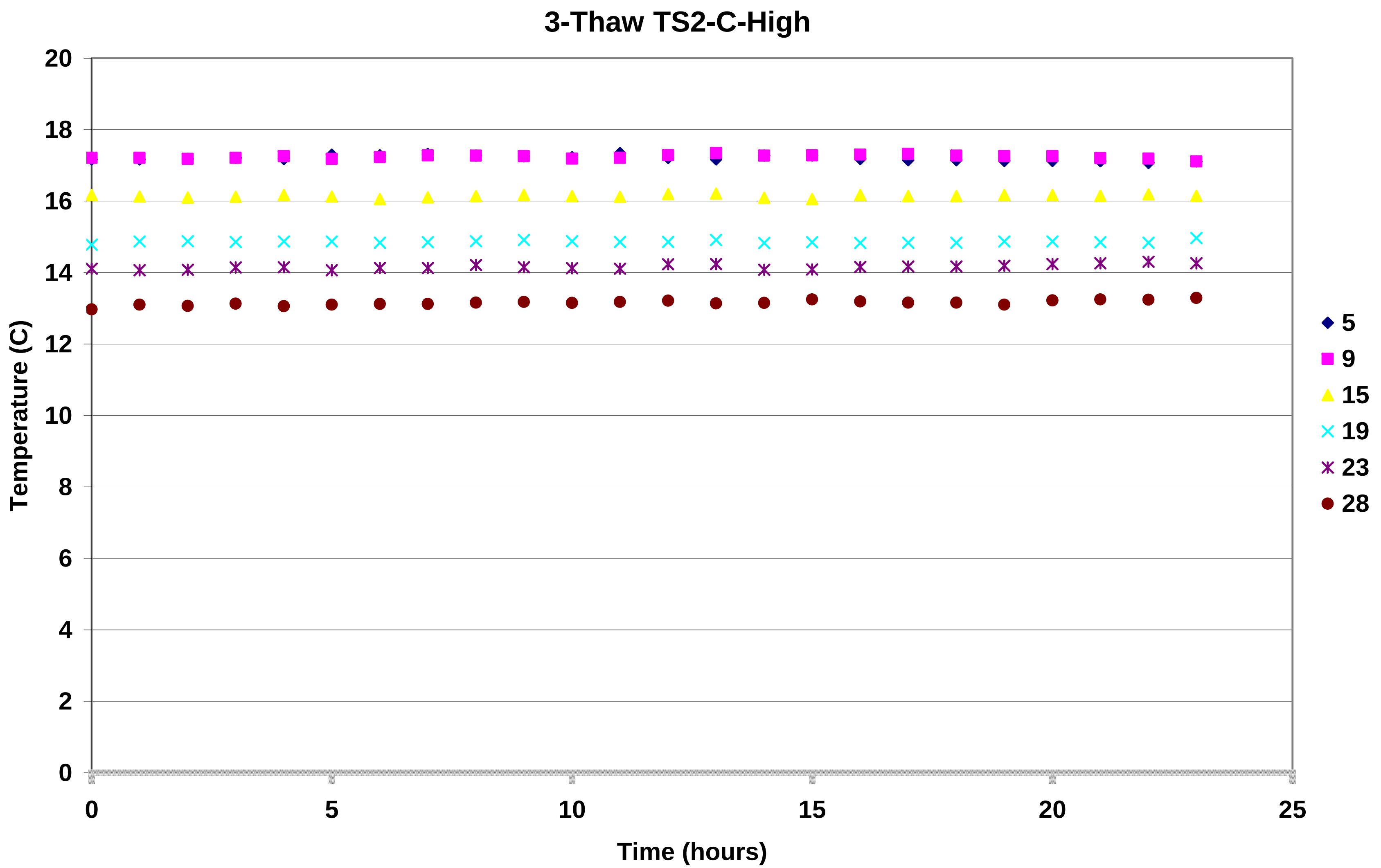

2-Thaw TS3-W-Low (F)
Temperature (C)
Time (hours)
10
8
6
4
2
0
-2
0
20
40
60
80
100
120
140
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5



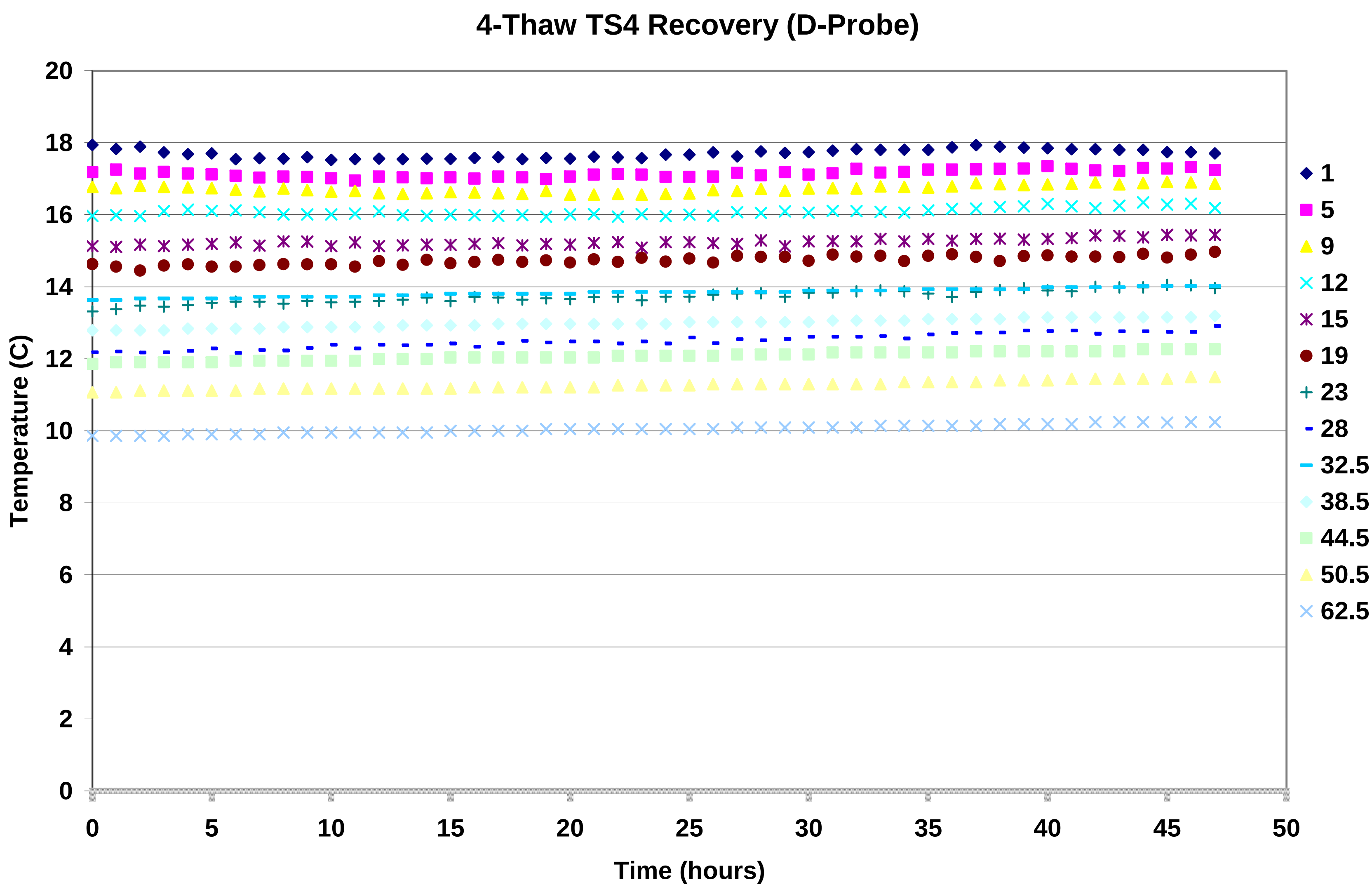
2-Thaw TS1-W-High
Temperature (C)
10
8
6
4
2
0
-2
0
20
40
60
80
100
120
140
Time (hours)
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

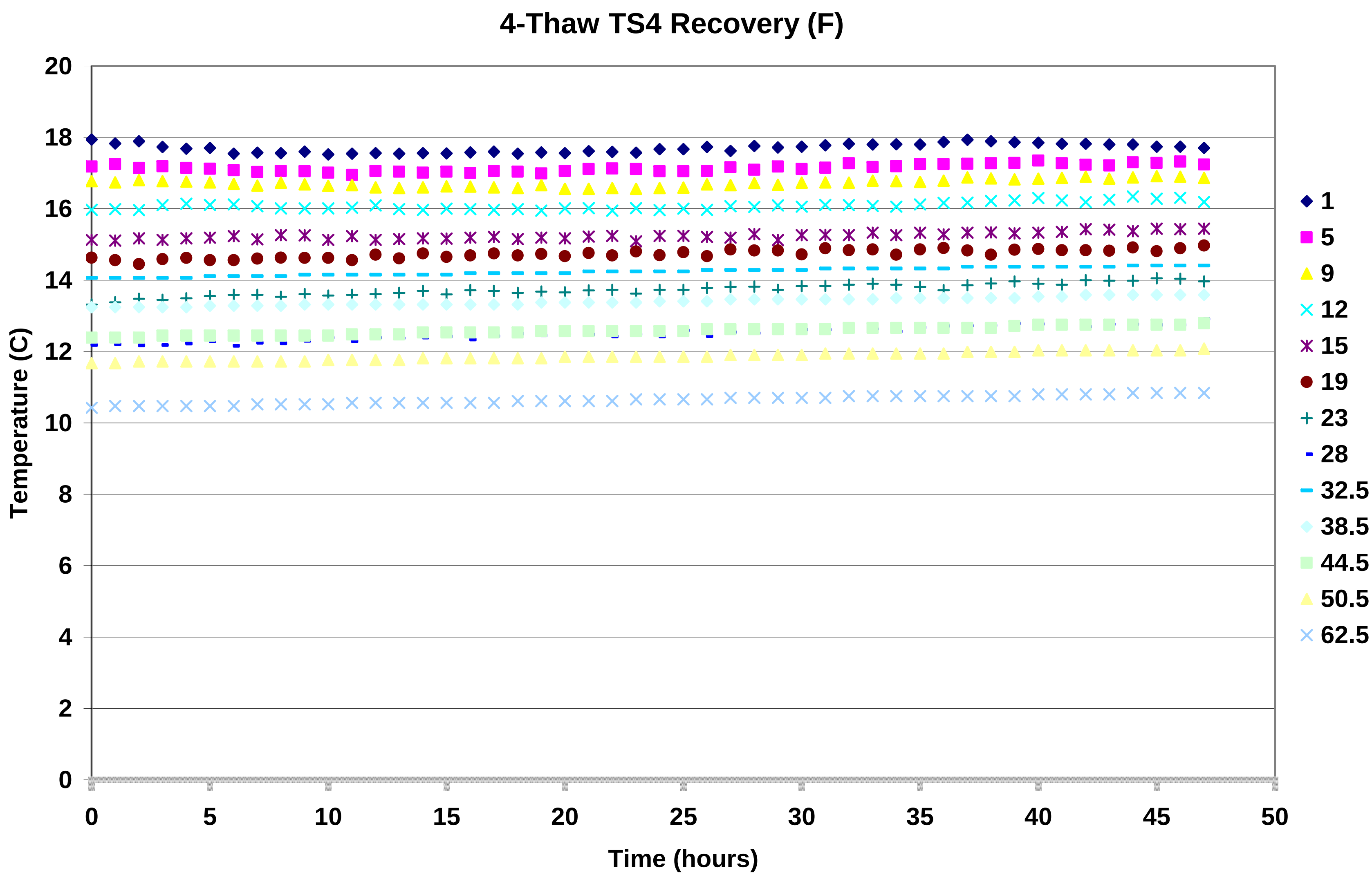


3-Thaw TS1-C-Low
Temperature (C)
Time (hours)
0
2
4
6
8
10
12
14
16
0
5
10
15
20
25
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5



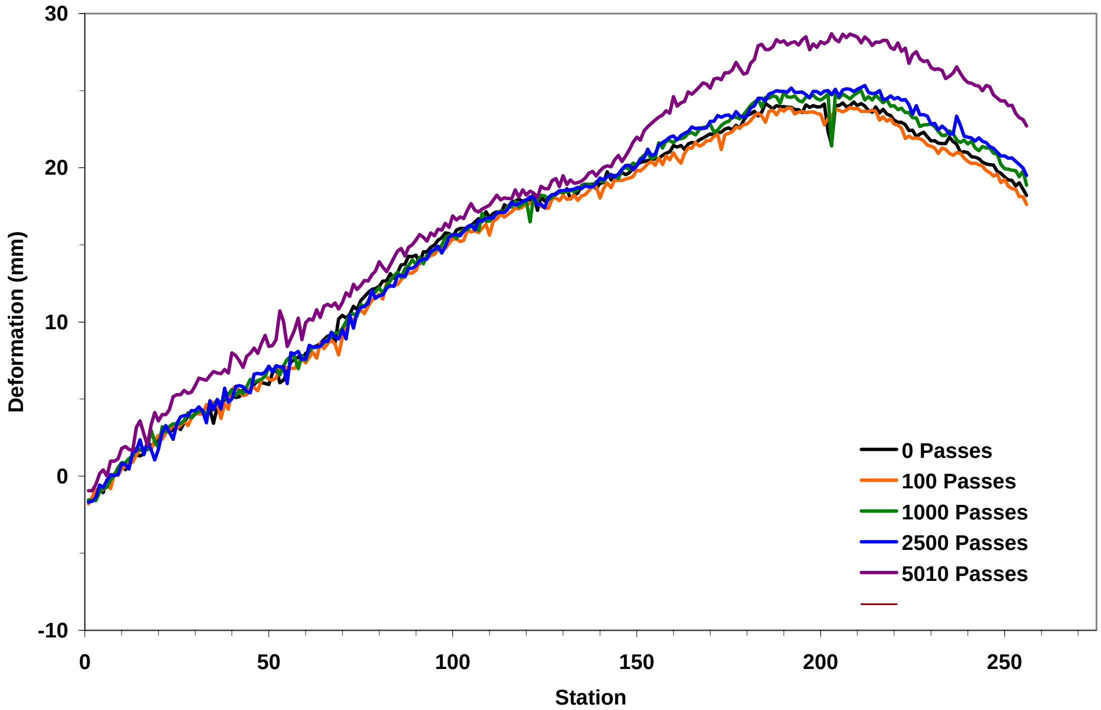
3-Thaw TS2-C-High
Temperature (C)
Time (hours)
0
2
4
6
8
10
12
14
16
18
20
0
5
10
15
20
25
5
9
15
19
23
28


4-Thaw TS4 Recovery (D-Probe)
Temperature (C)
Time (hours)
0
2
4
6
8
10
12
14
16
18
20
0
5
10
15
20
25
30
35
40
45
50
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

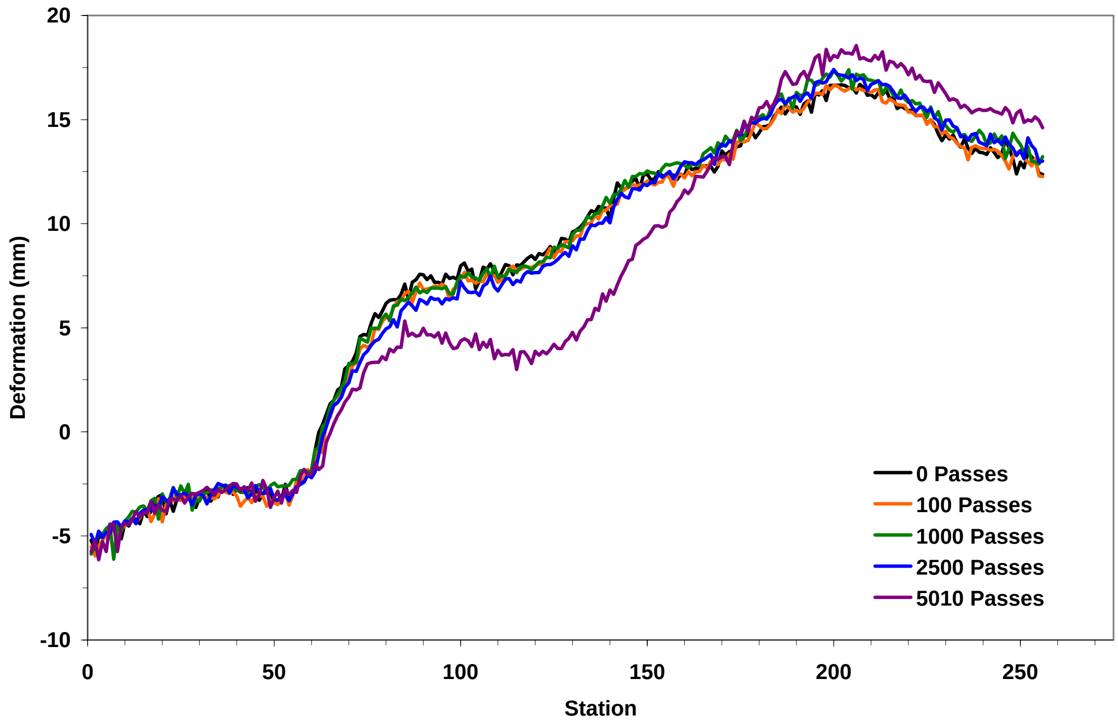

4-Thaw TS4 Recovery (F)
Temperature (C)
Time (hours)
0
2
4
6
8
10
12
14
16
18
20
0
5
10
15
20
25
30
35
40
45
50
1
5
9
12
15
19
23
28
32.5
38.5
44.5
50.5
62.5

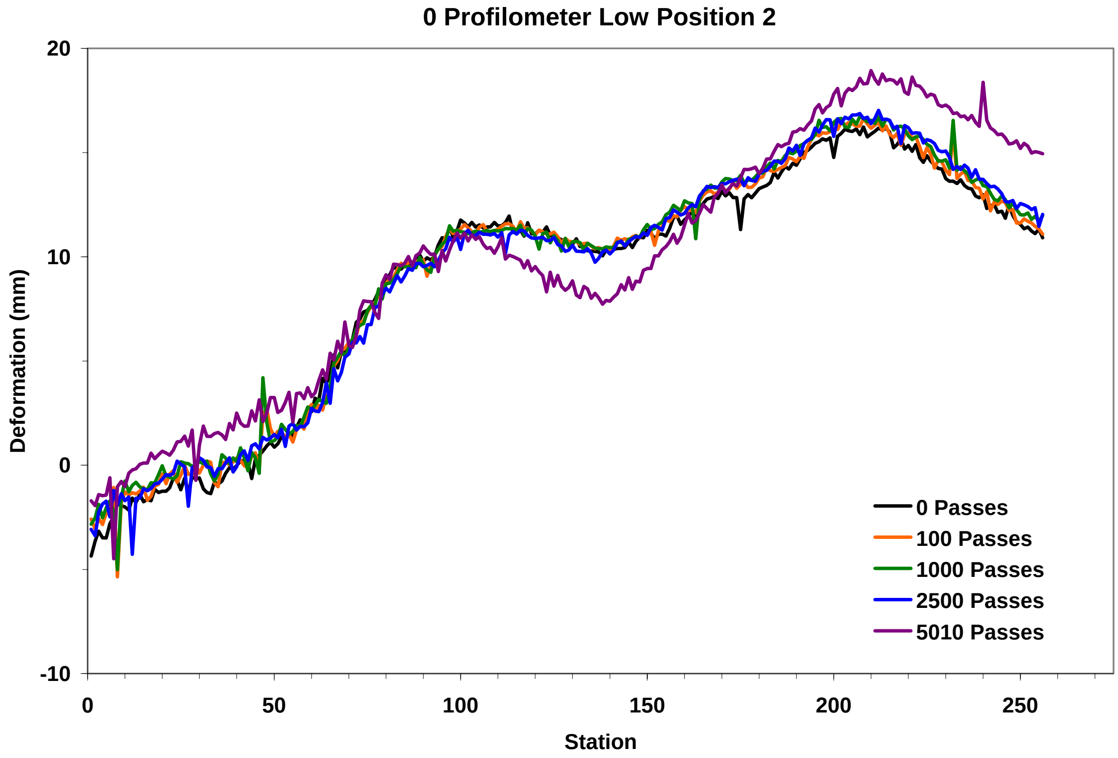
# Appendix D:
# Profilometer Data

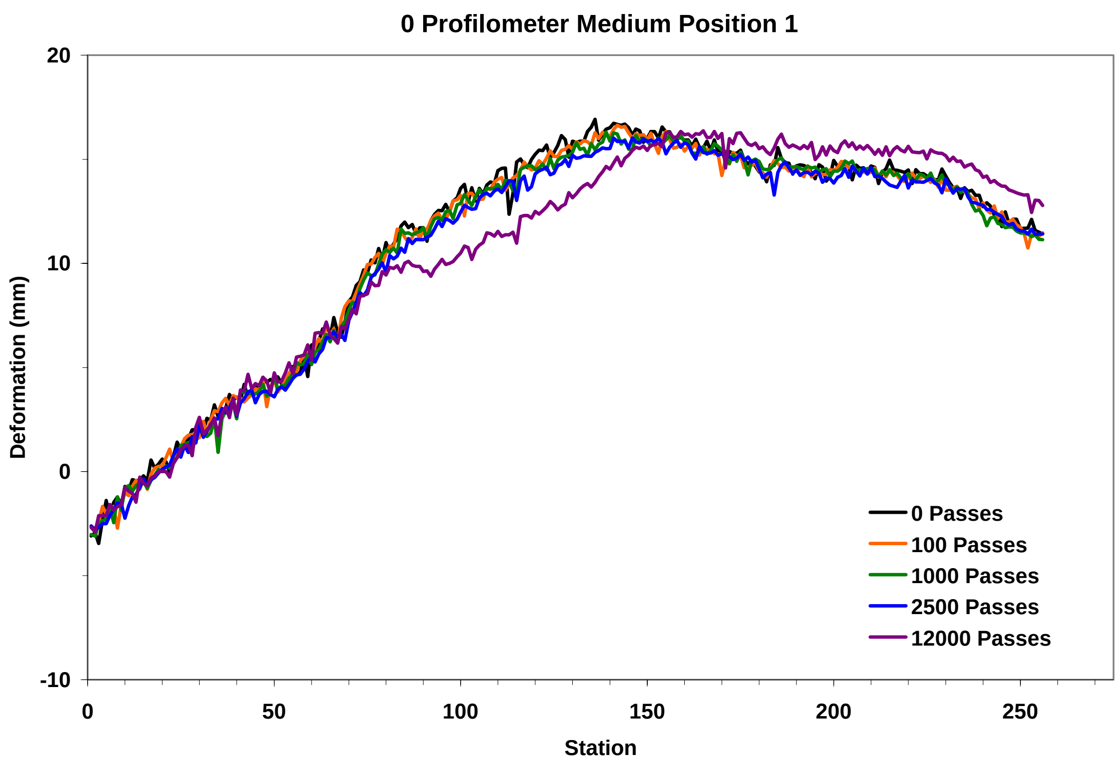
**0 Profilometer Low Position 1**

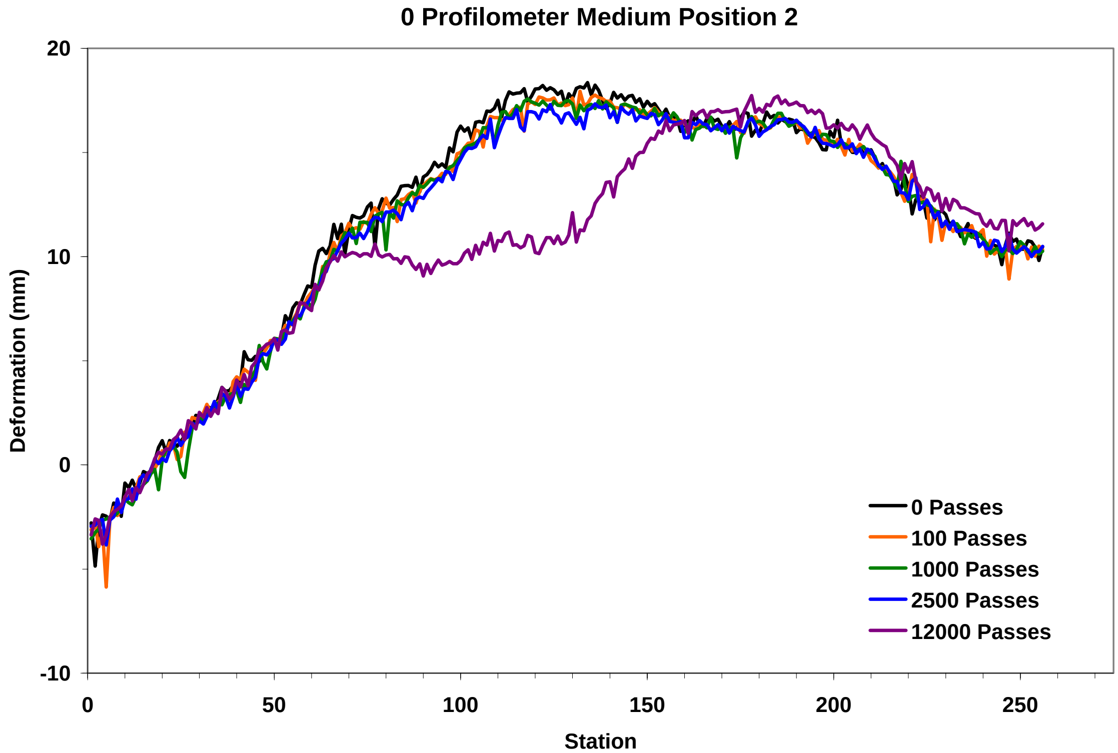


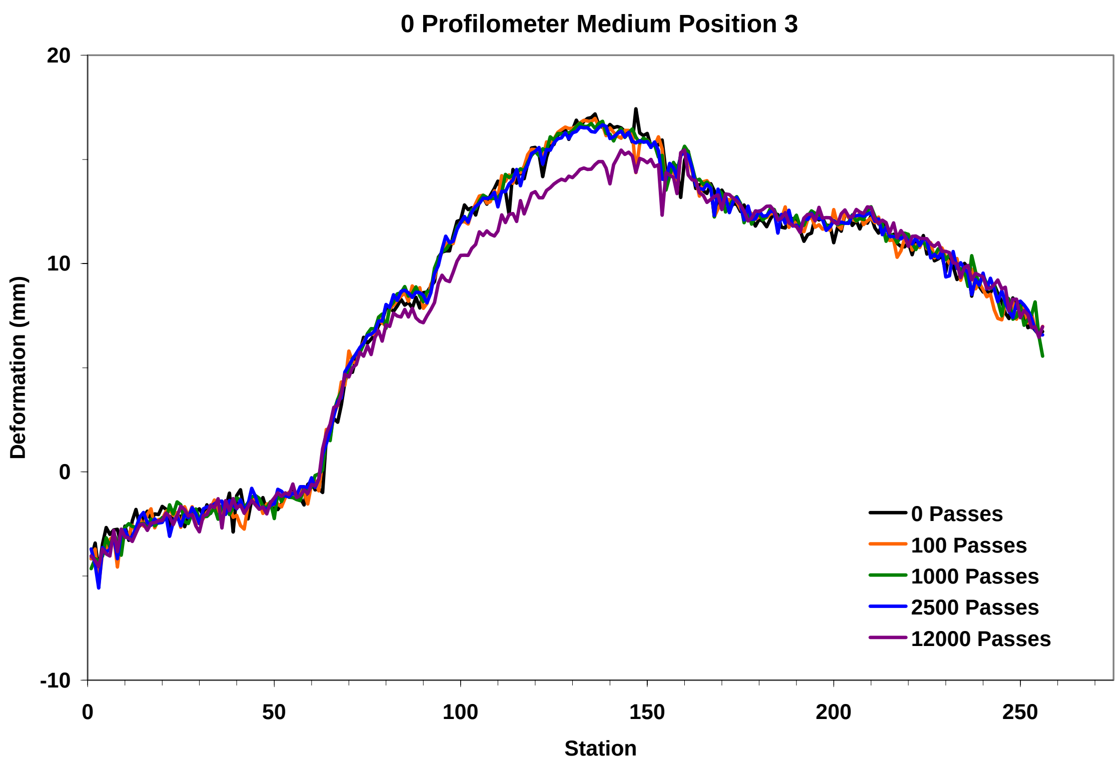

**0 Profilometer Low Position 3**

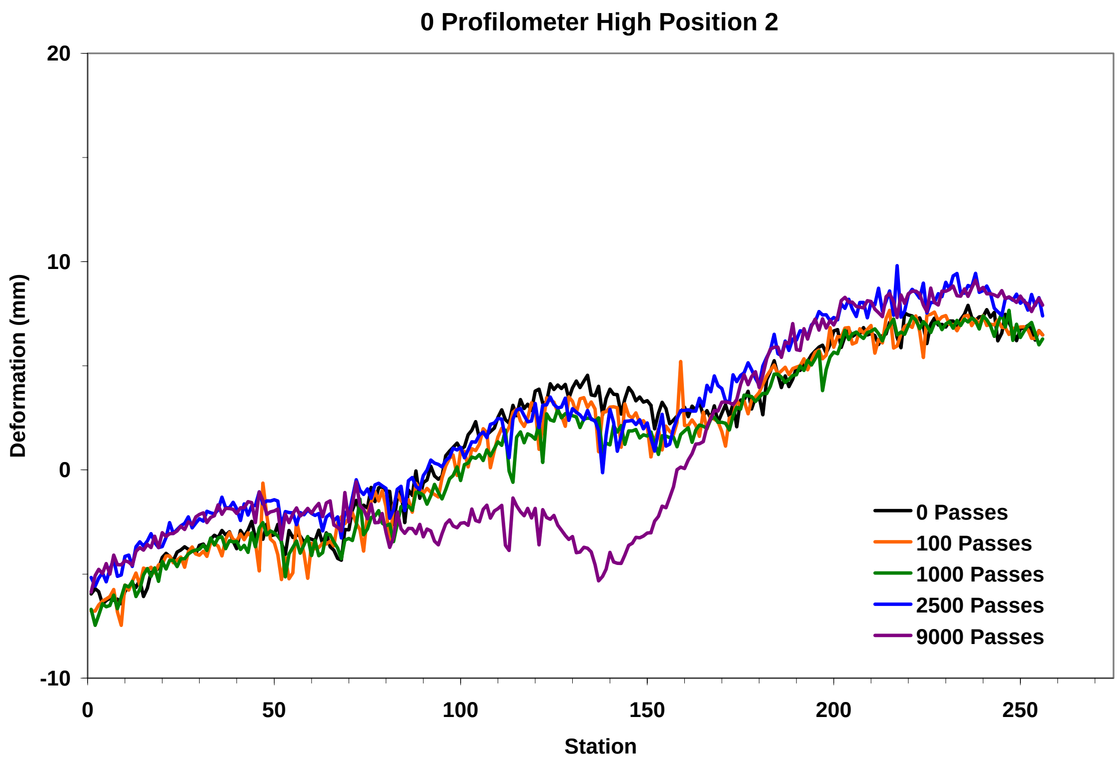


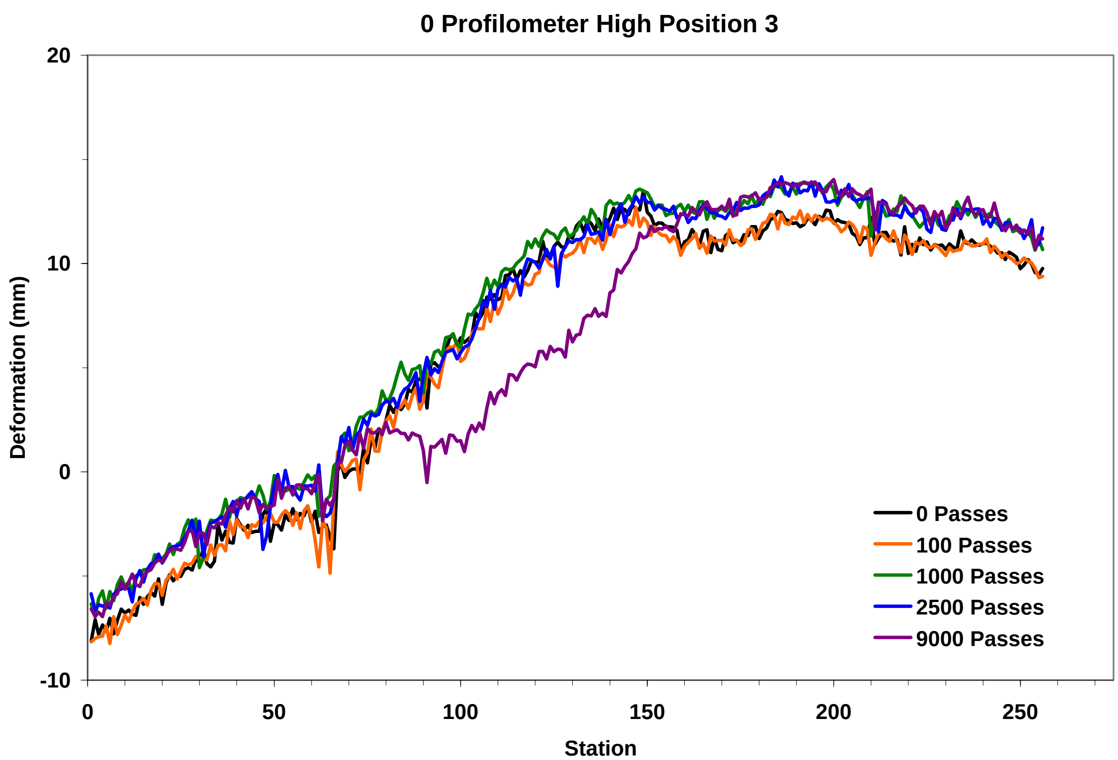

0 Profilometer Low Position 2
Deformation (mm)
20
10
0
-10
0
50
100
150
200
250
Station
0 Passes
100 Passes
1000 Passes
2500 Passes
5010 Passes

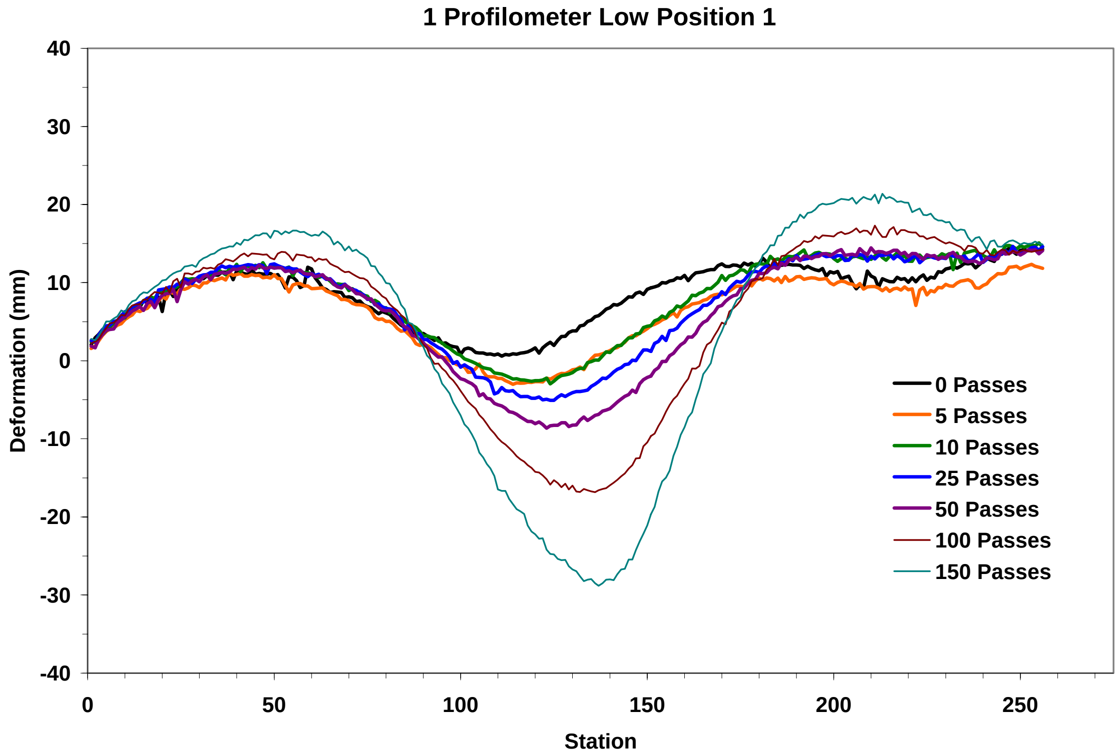

0 Profilometer Medium Position 1
Deformation (mm)
20
10
0
-10
0
50
100
150
200
250
Station
0 Passes
100 Passes
1000 Passes
2500 Passes
12000 Passes

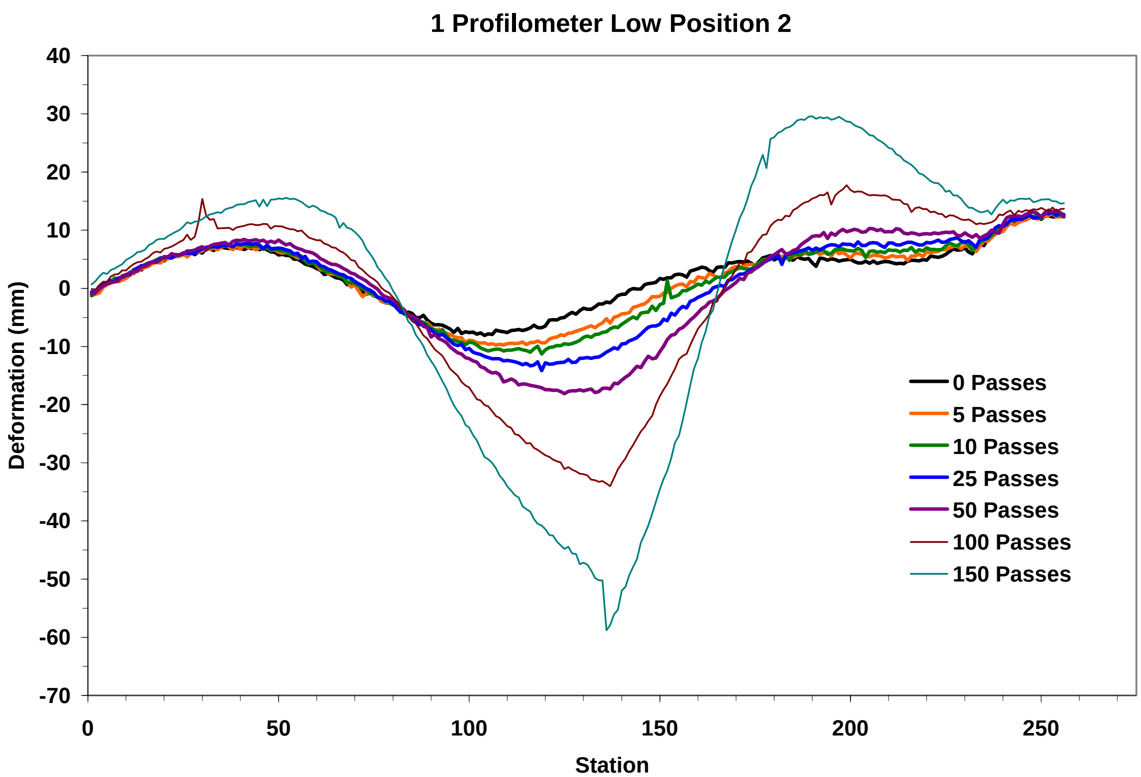

0 Profilometer Medium Position 2
Deformation (mm)
Station
20
10
0
-10
0
50
100
150
200
250
0 Passes
100 Passes
1000 Passes
2500 Passes
12000 Passes

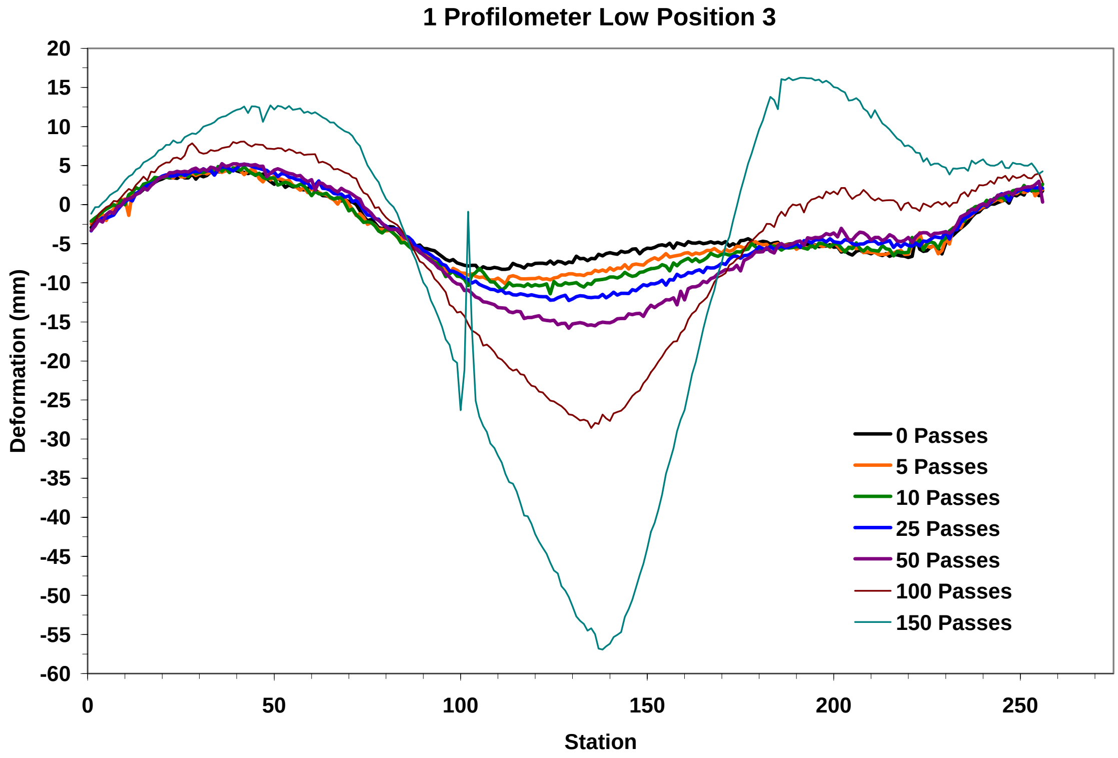

0 Profilometer Medium Position 3
Deformation (mm)
Station
20
10
0
-10
0
50
100
150
200
250
0 Passes
100 Passes
1000 Passes
2500 Passes
12000 Passes

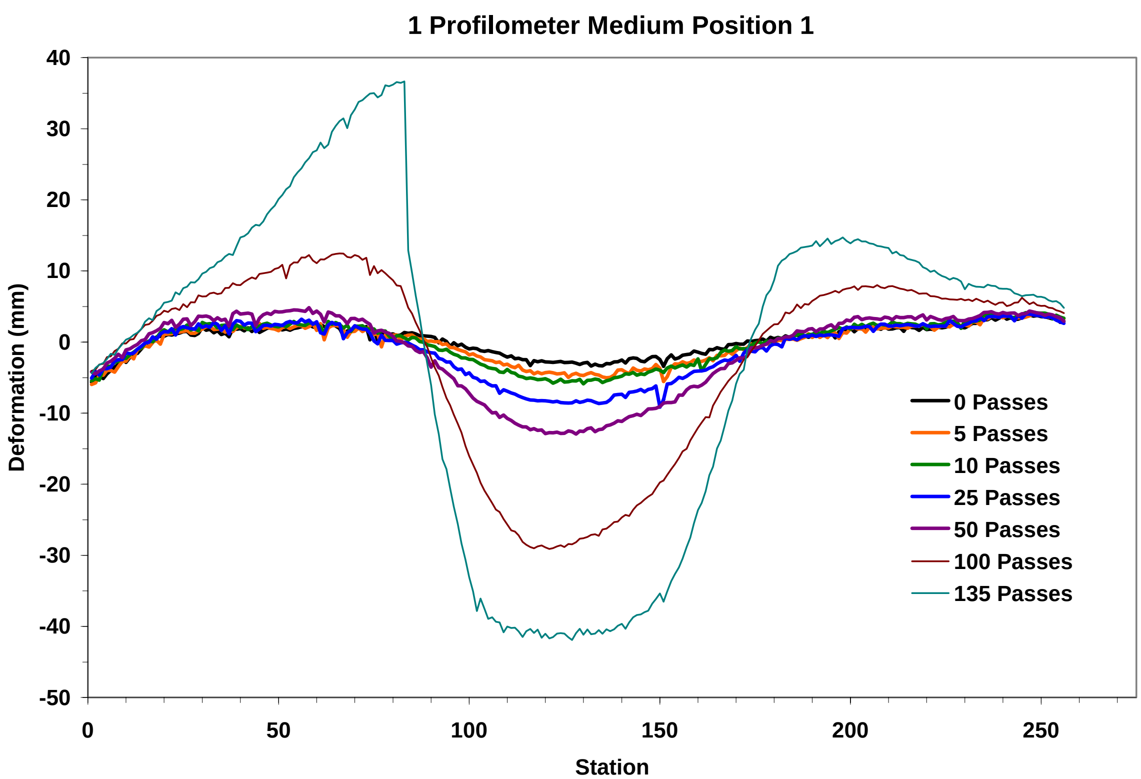

0 Profilometer High Position 2
Deformation (mm)
Station
20
10
0
-10
0
50
100
150
200
250
0 Passes
100 Passes
1000 Passes
2500 Passes
9000 Passes

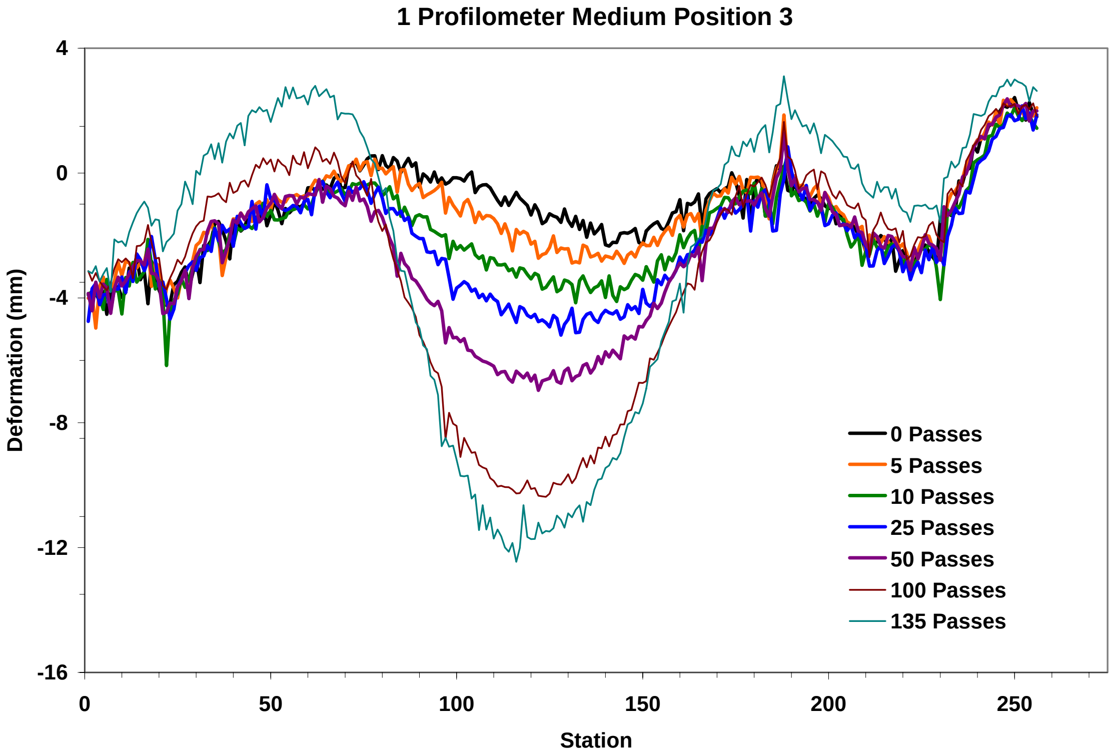

0 Profilometer High Position 3
Deformation (mm)
Station
20
10
0
-10
0
50
100
150
200
250
0 Passes
100 Passes
1000 Passes
2500 Passes
9000 Passes

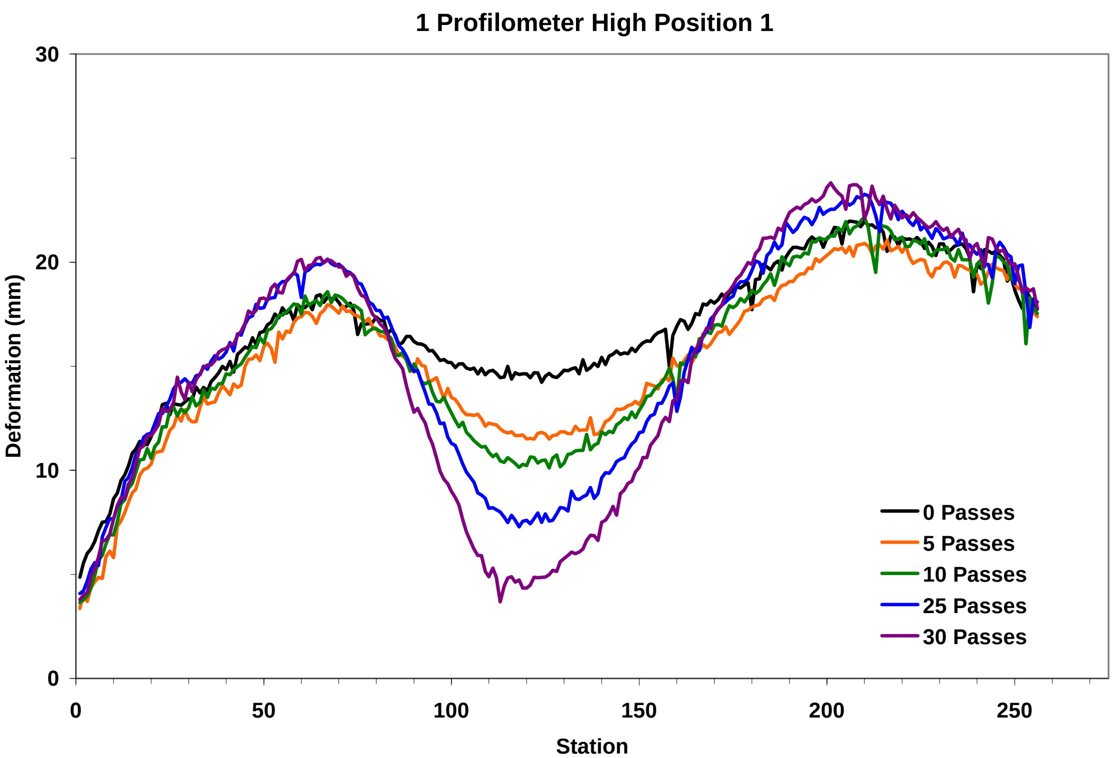

1 Profilometer Low Position 1
Deformation (mm)
40
30
20
10
0
-10
-20
-30
-40
0
50
100
150
200
250
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes

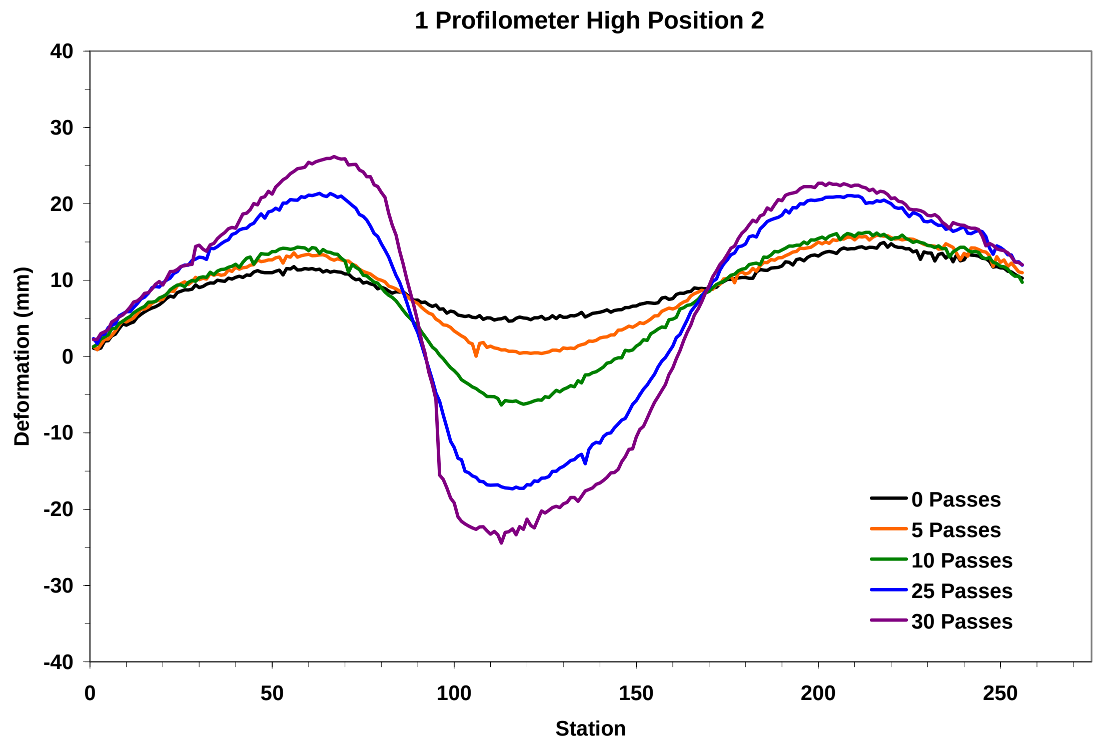

1 Profilometer Low Position 2
Deformation (mm)
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
0
50
100
150
200
250
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes


1 Profilometer Low Position 3
Deformation (mm)
Station
20
15
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
-55
-60
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes

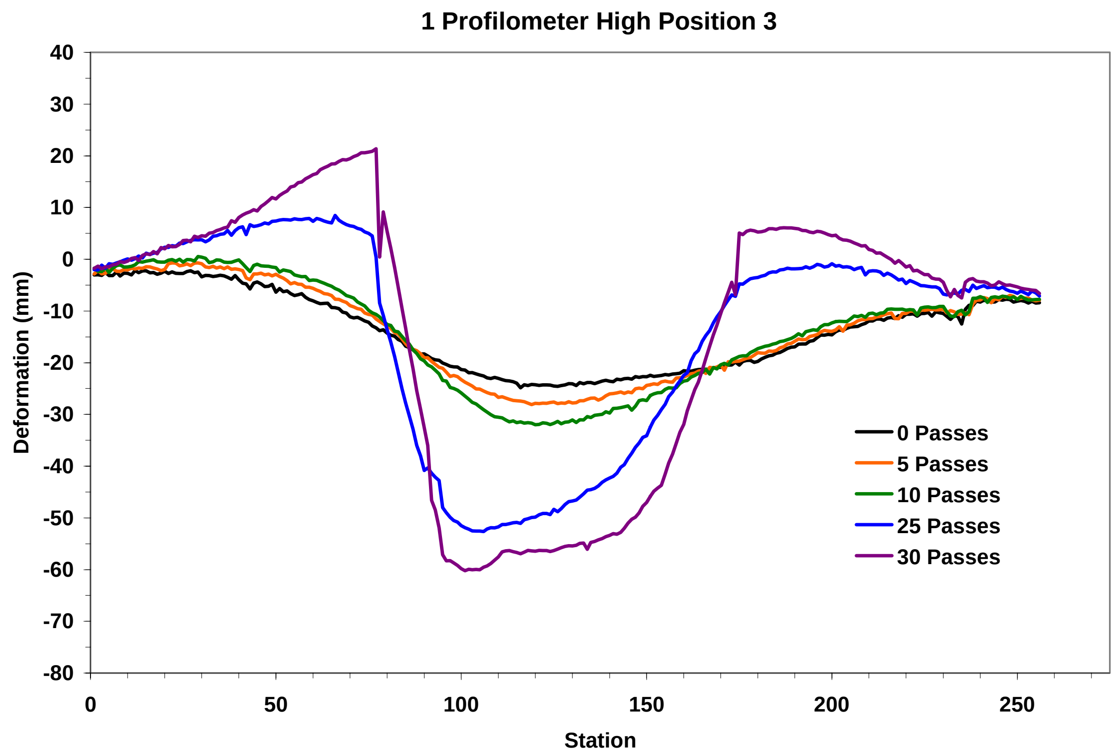

1 Profilometer Medium Position 1
Deformation (mm)
Station
40
30
20
10
0
-10
-20
-30
-40
-50
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
135 Passes


1 Profilometer Medium Position 3
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
135 Passes

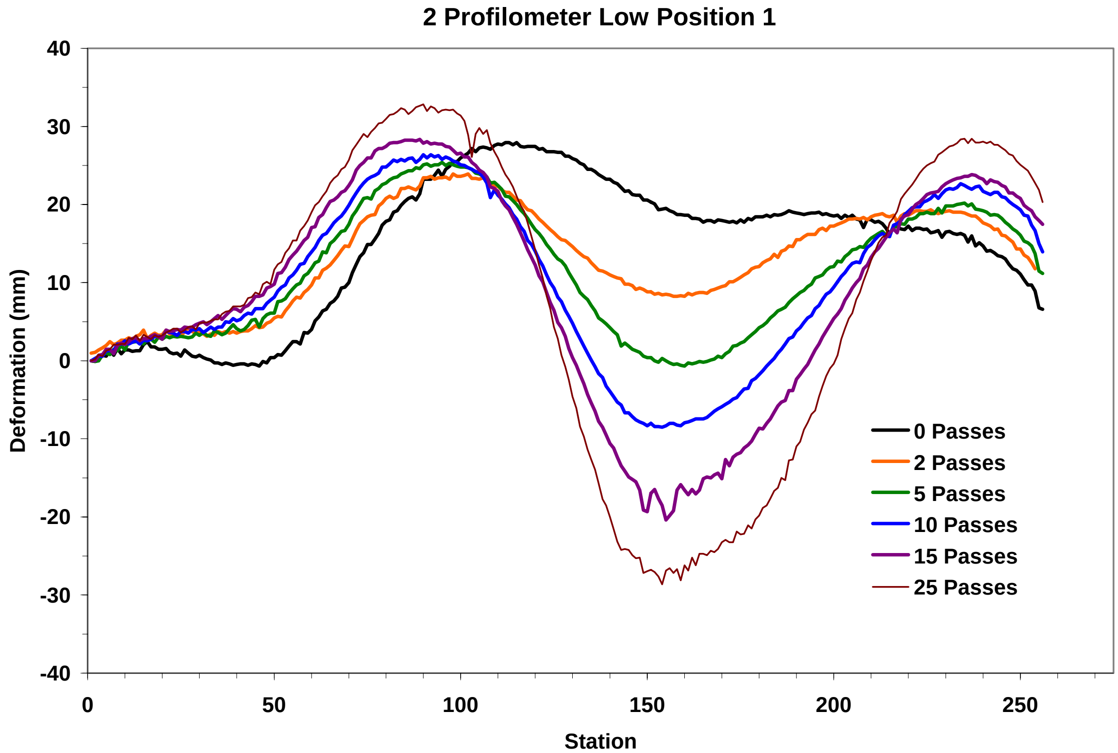

1 Profilometer High Position 1
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
30 Passes

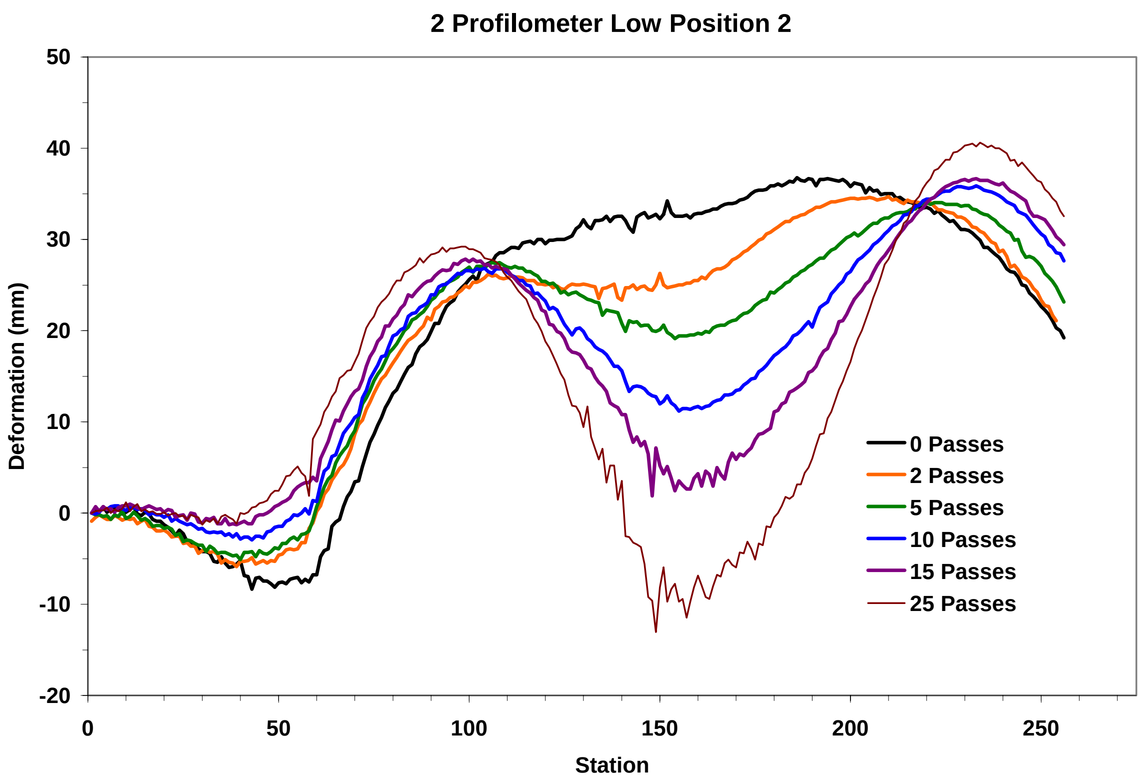


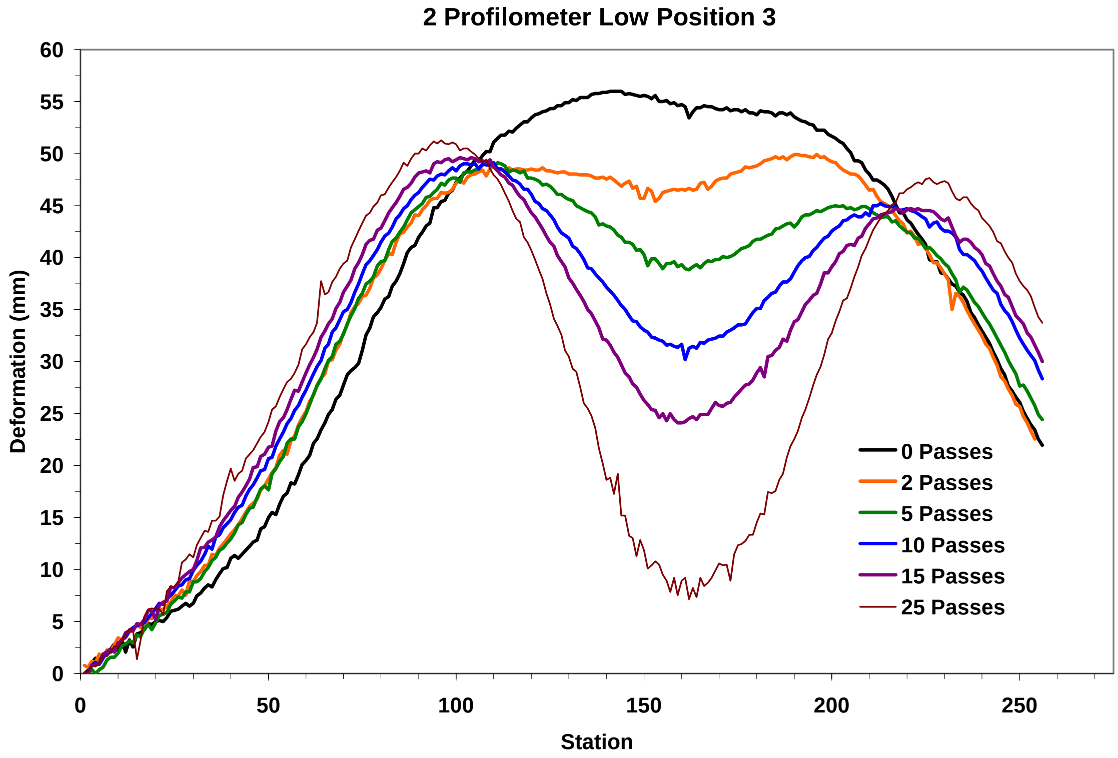
1 Profilometer High Position 2
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
30 Passes

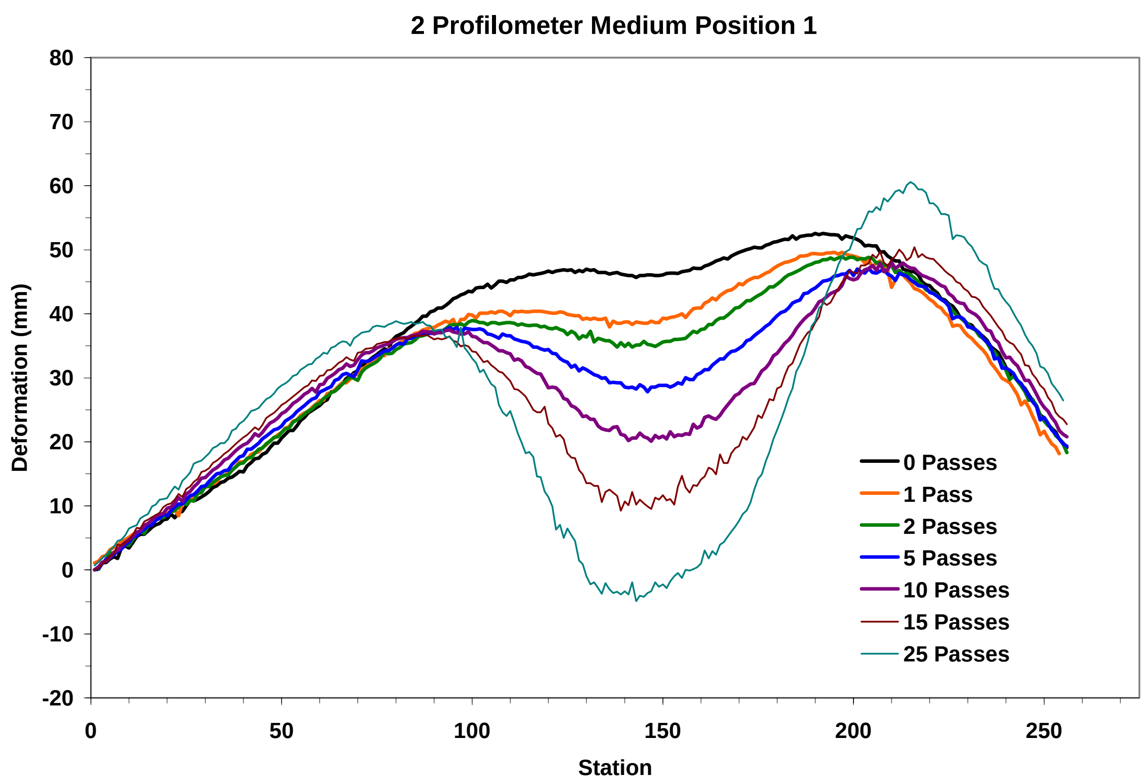



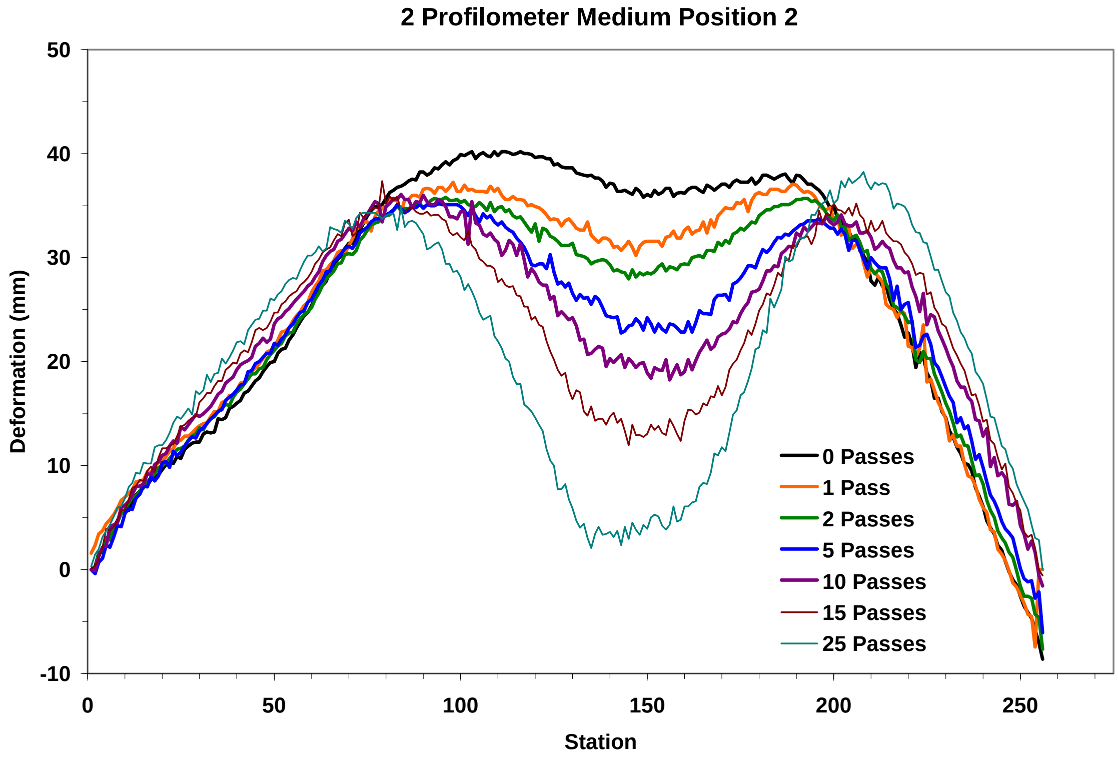
1 Profilometer High Position 3
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
30 Passes

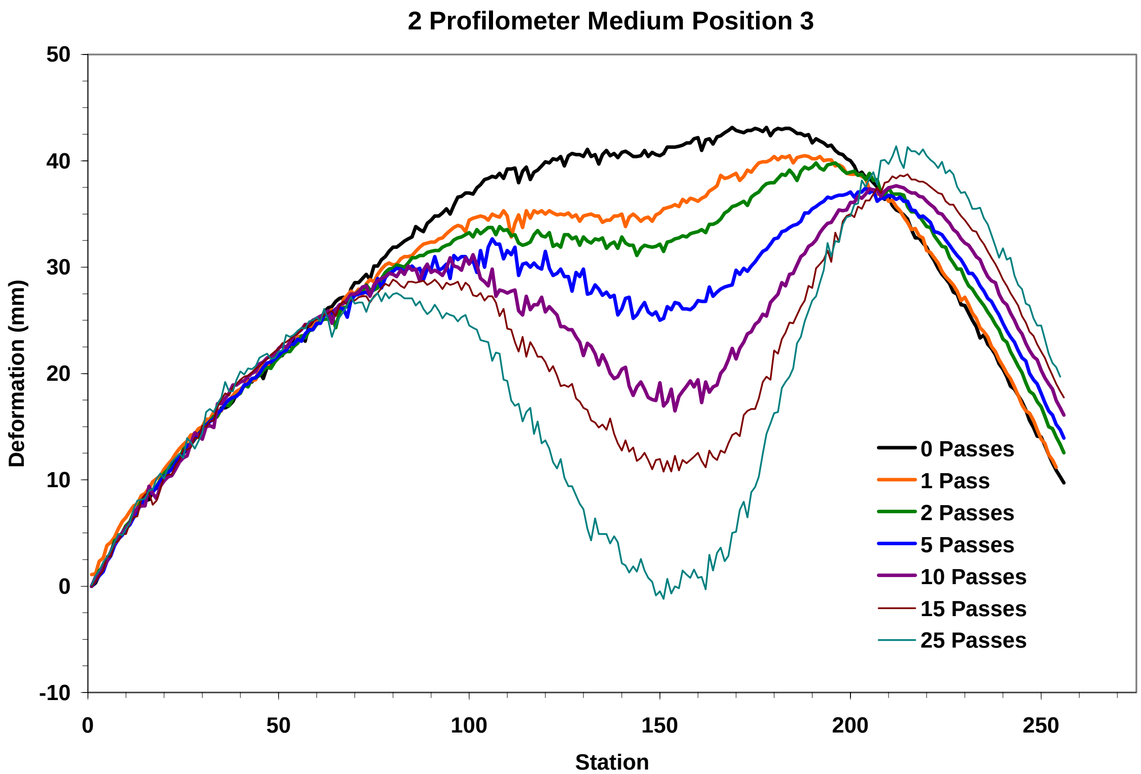

2 Profilometer Low Position 1
Deformation (mm)
40
30
20
10
0
-10
-20
-30
-40
0
50
100
150
200
250
Station
0 Passes
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes

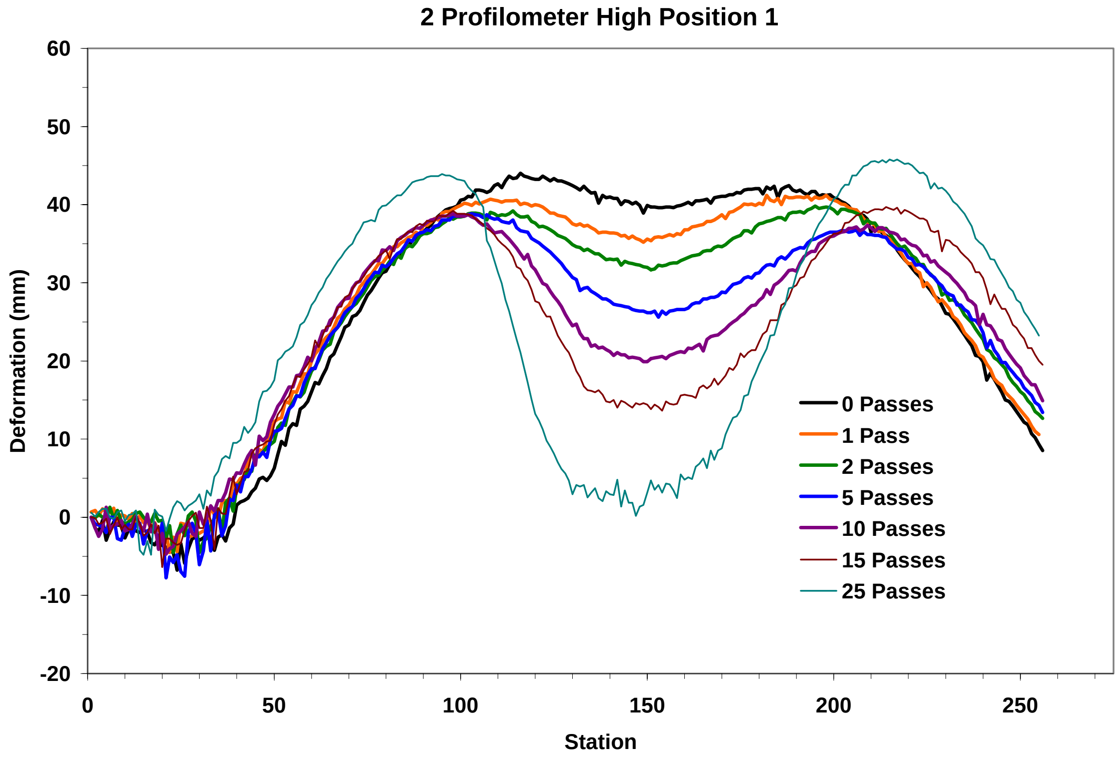

2 Profilometer Low Position 2
Deformation (mm)
50
40
30
20
10
0
-10
-20
0
50
100
150
200
250
Station
0 Passes
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes

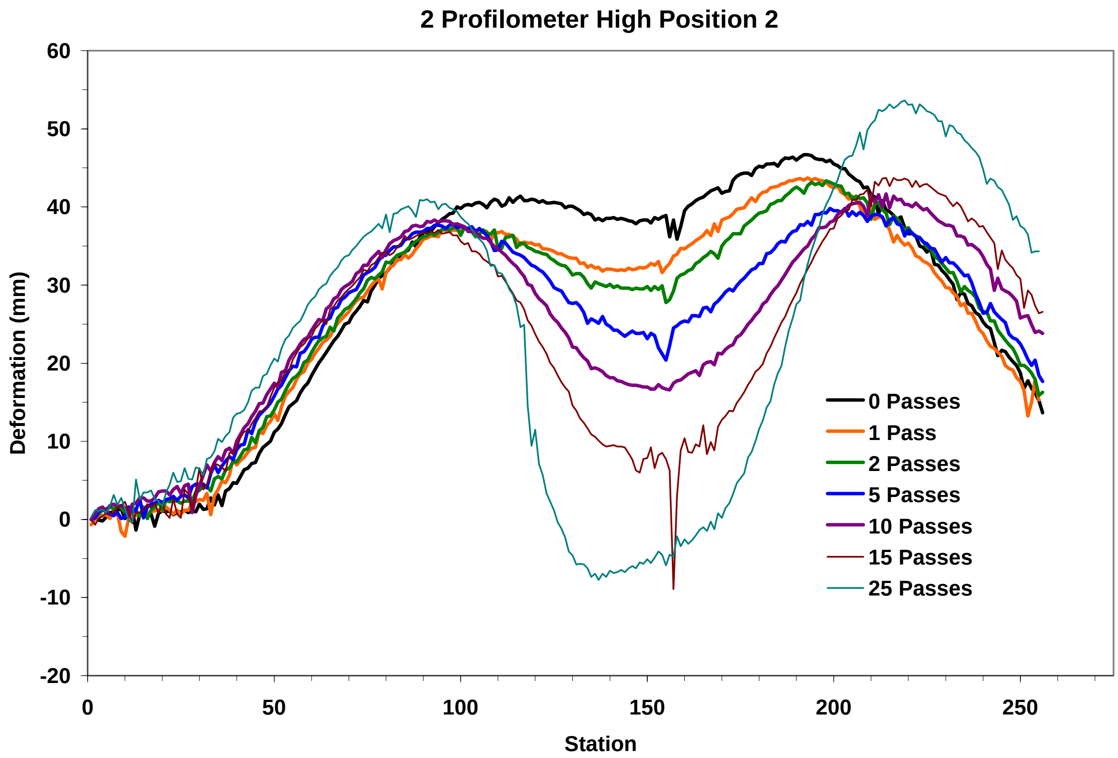

2 Profilometer Low Position 3
Deformation (mm)
Station
0 Passes
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes

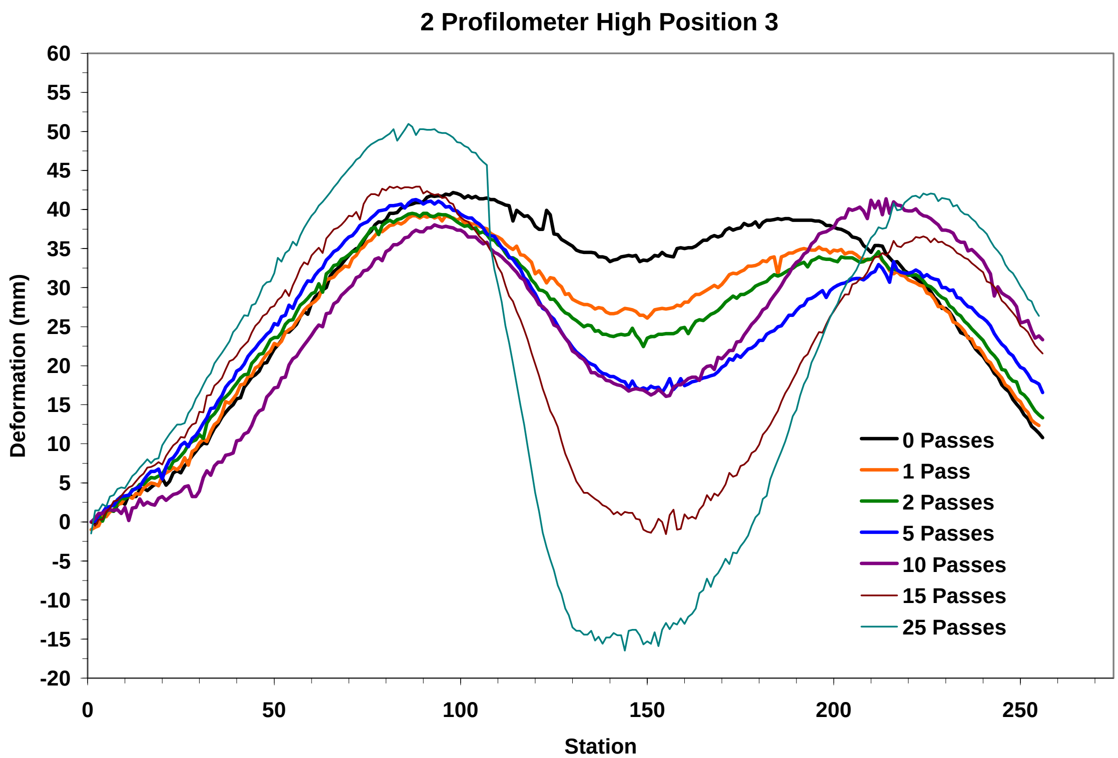

2 Profilometer Medium Position 1
Deformation (mm)
Station
0 Passes
1 Pass
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes

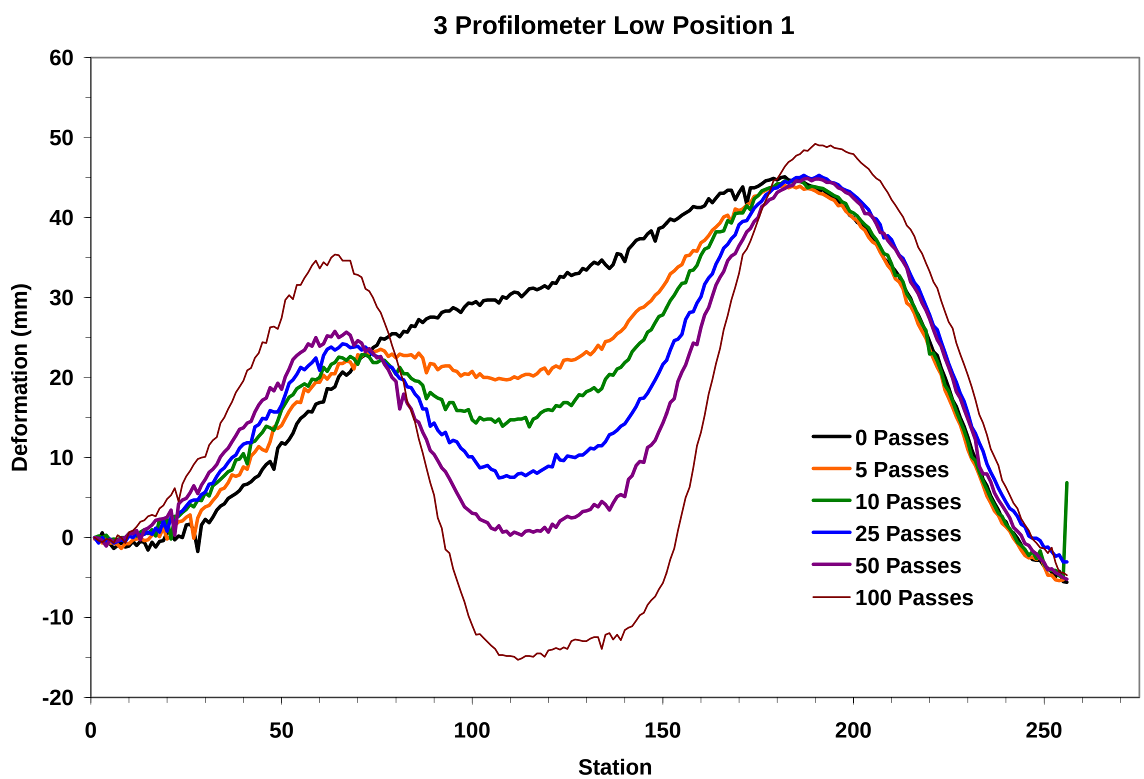

2 Profilometer Medium Position 2
Deformation (mm)
Station
0 Passes
1 Pass
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes
50
40
30
20
10
0
-10
0
50
100
150
200
250

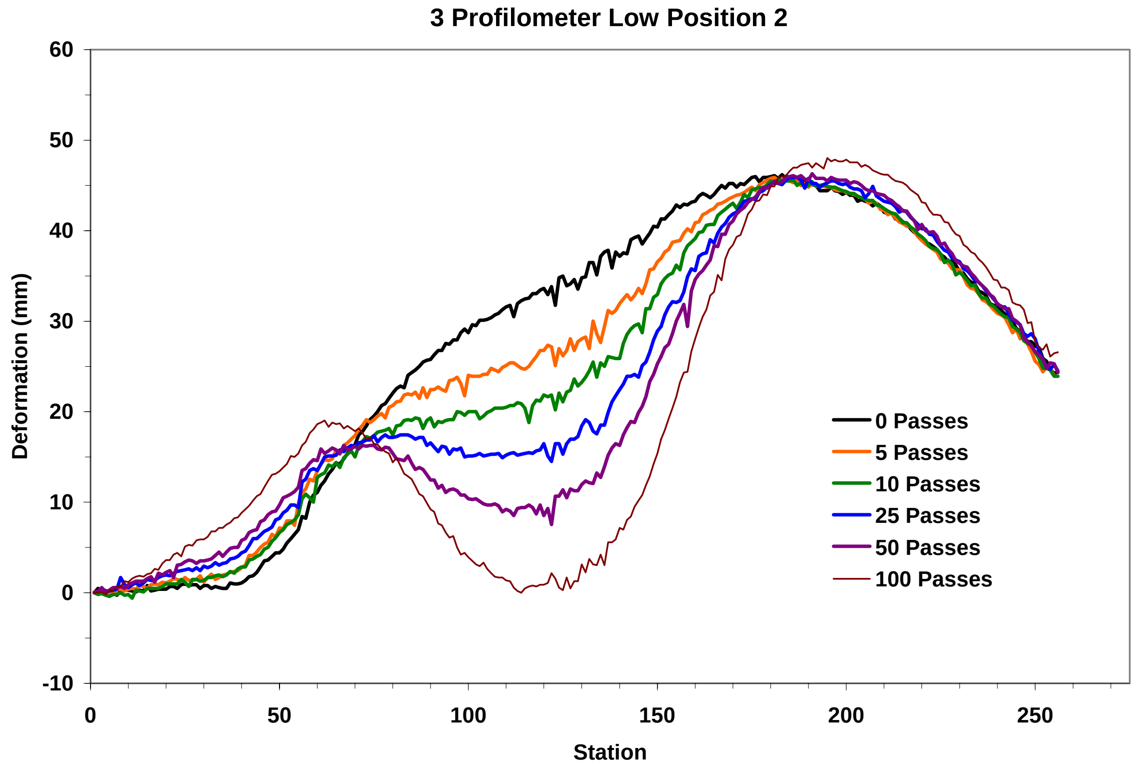

2 Profilometer Medium Position 3
Deformation (mm)
Station
0 Passes
1 Pass
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes
50
40
30
20
10
0
-10
0
50
100
150
200
250

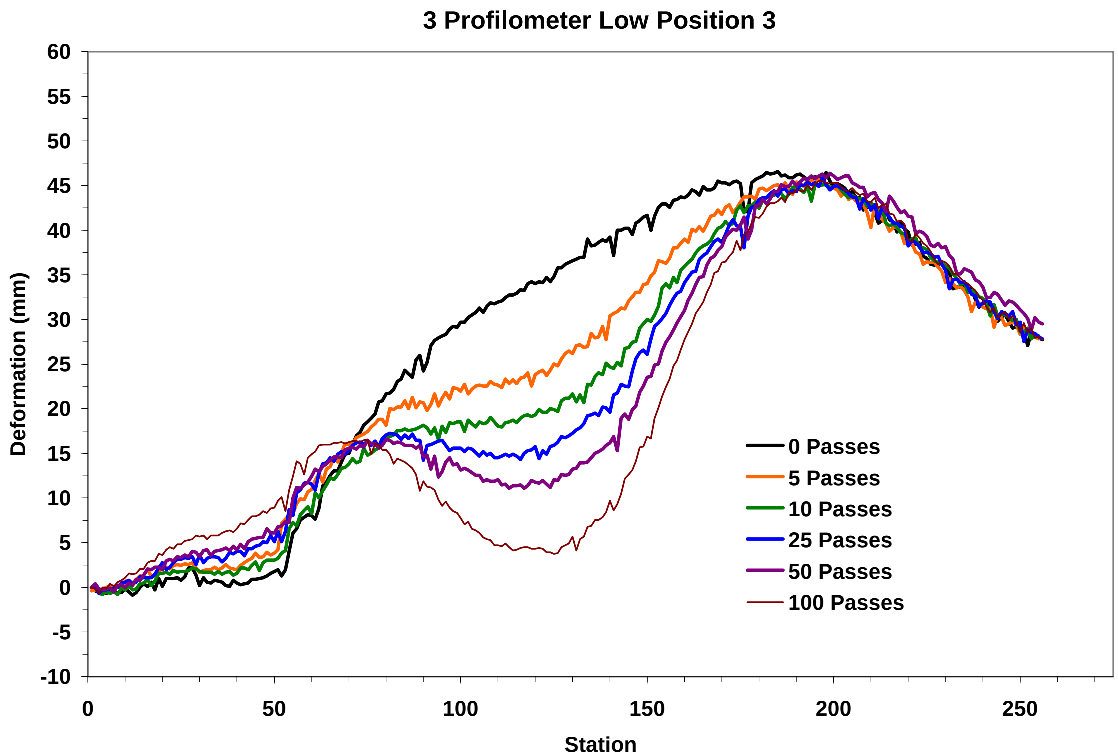

2 Profilometer High Position 1
Deformation (mm)
Station
0 Passes
1 Pass
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes
60
50
40
30
20
10
0
-10
-20
0
50
100
150
200
250

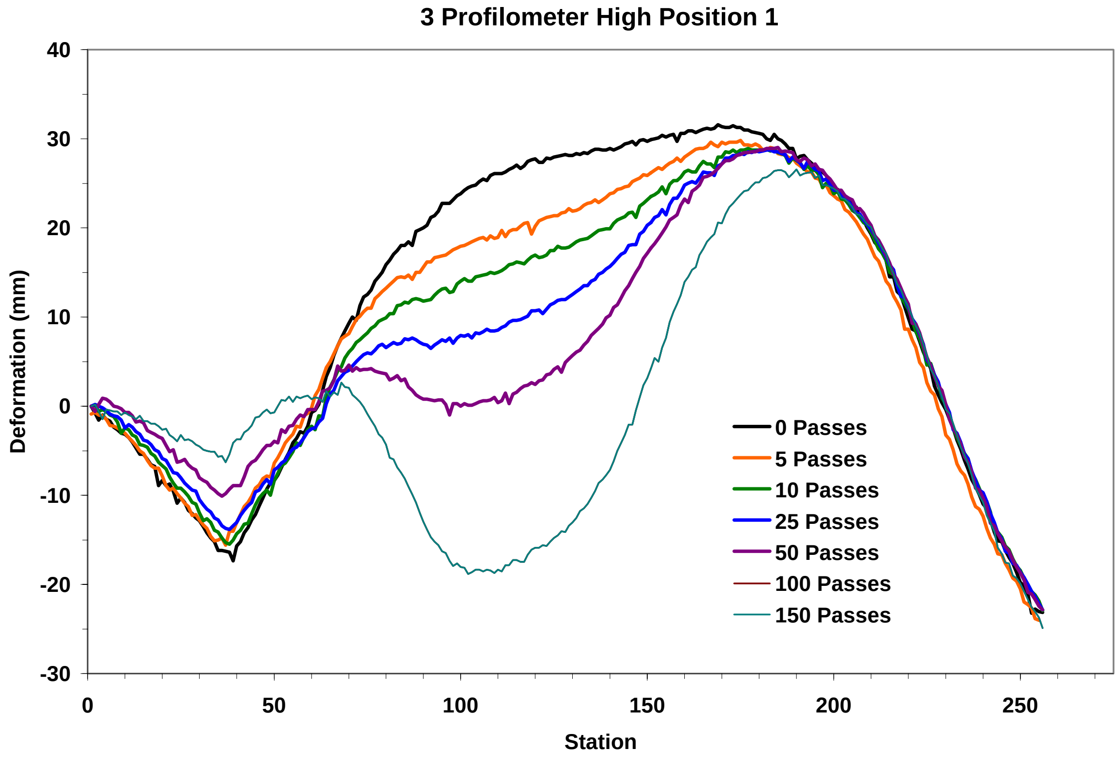

2 Profilometer High Position 2
Deformation (mm)
Station
0 Passes
1 Pass
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes
60
50
40
30
20
10
0
-10
-20
0
50
100
150
200
250

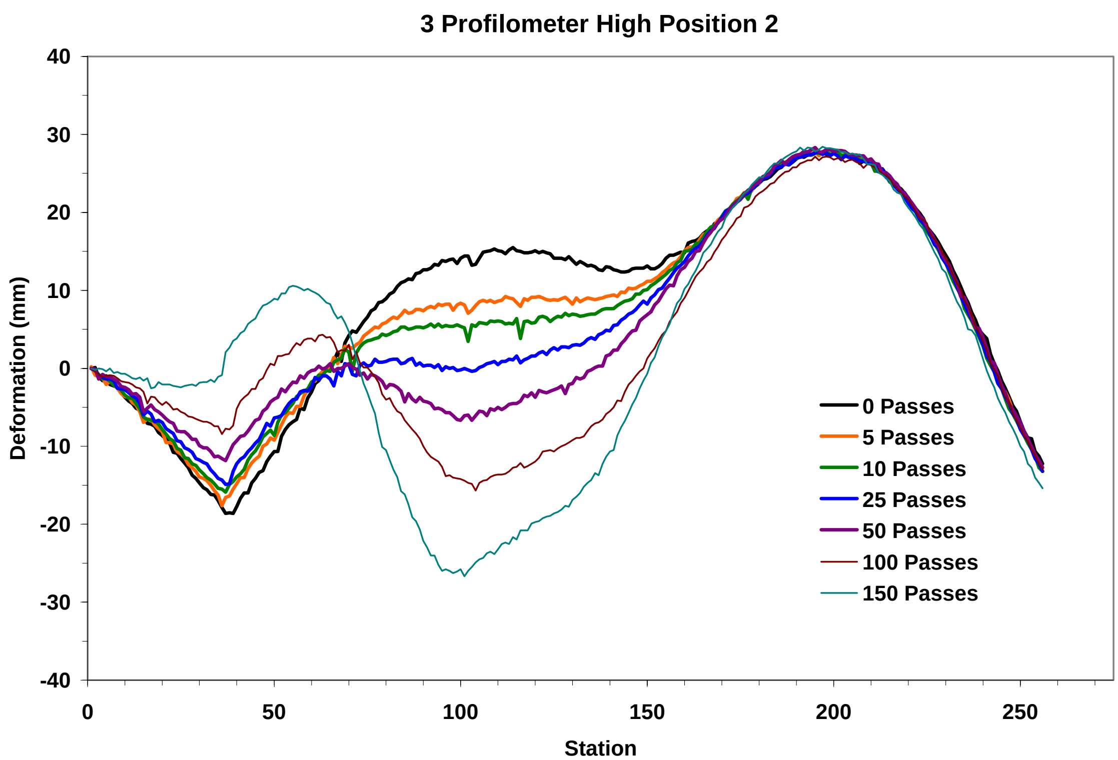

2 Profilometer High Position 3
Deformation (mm)
Station
0 Passes
1 Pass
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes

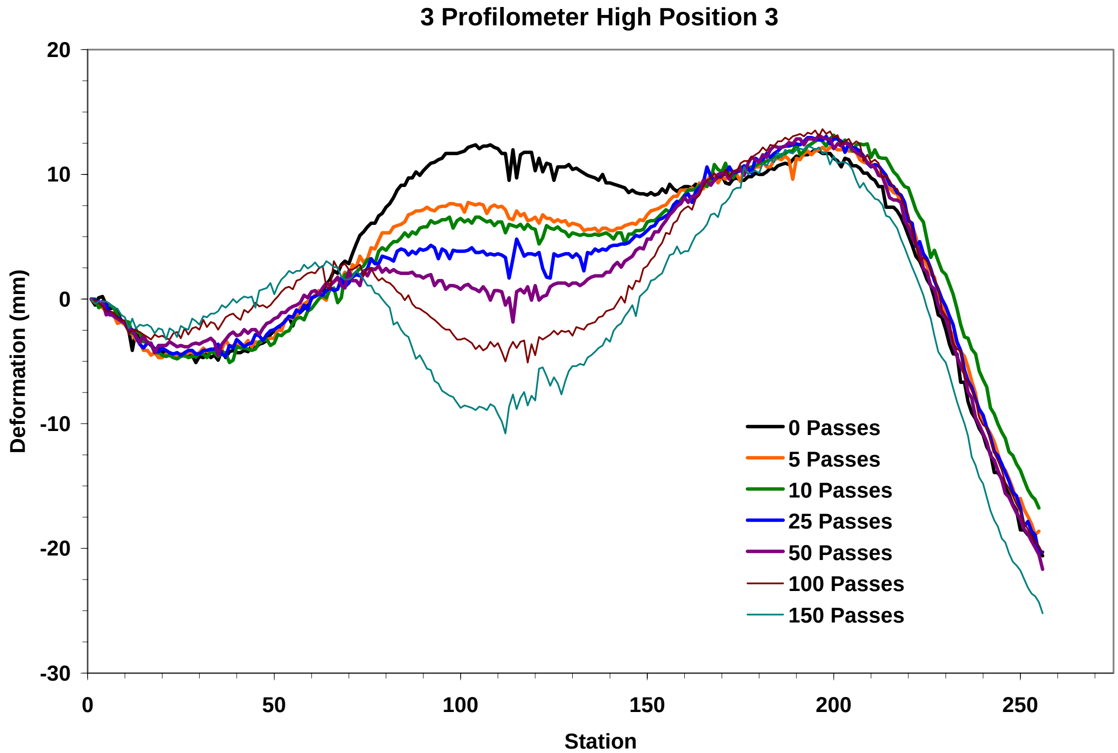

3 Profilometer Low Position 1
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes


3 Profilometer Low Position 2
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes

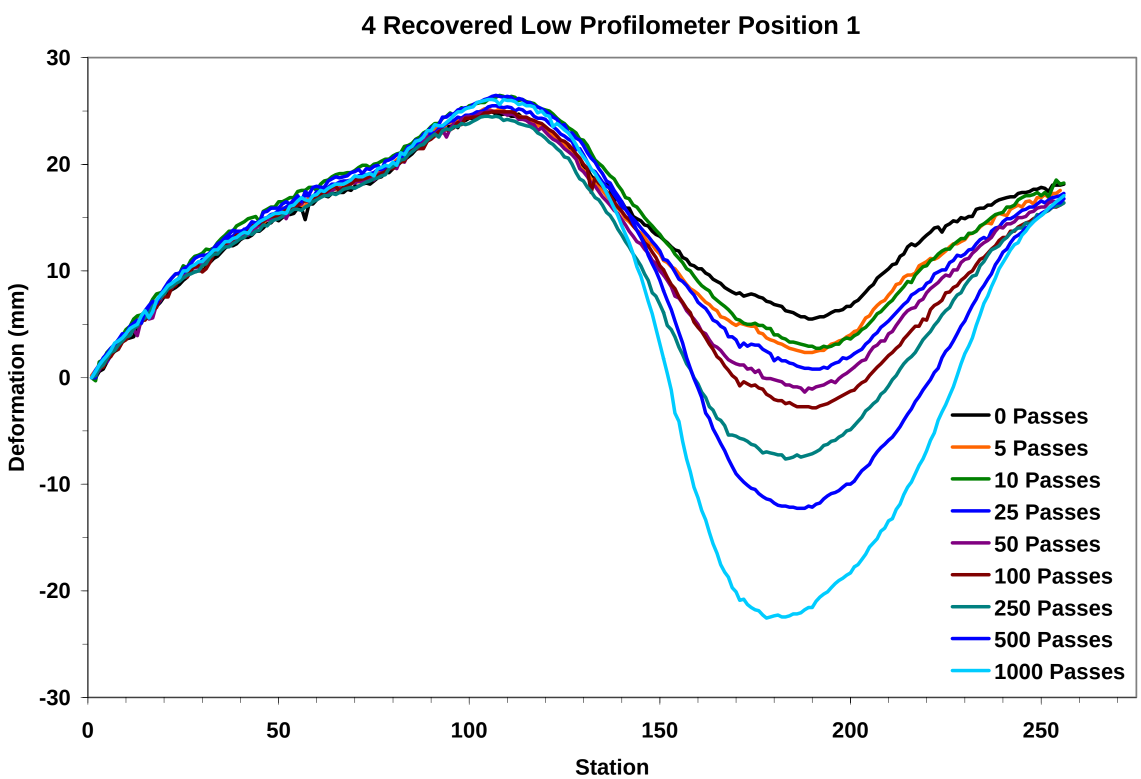

3 Profilometer Low Position 3
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes


3 Profilometer High Position 1
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes
40
30
20
10
0
-10
-20
-30
0
50
100
150
200
250


3 Profilometer High Position 2
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes
40
30
20
10
0
-10
-20
-30
-40
0
50
100
150
200
250

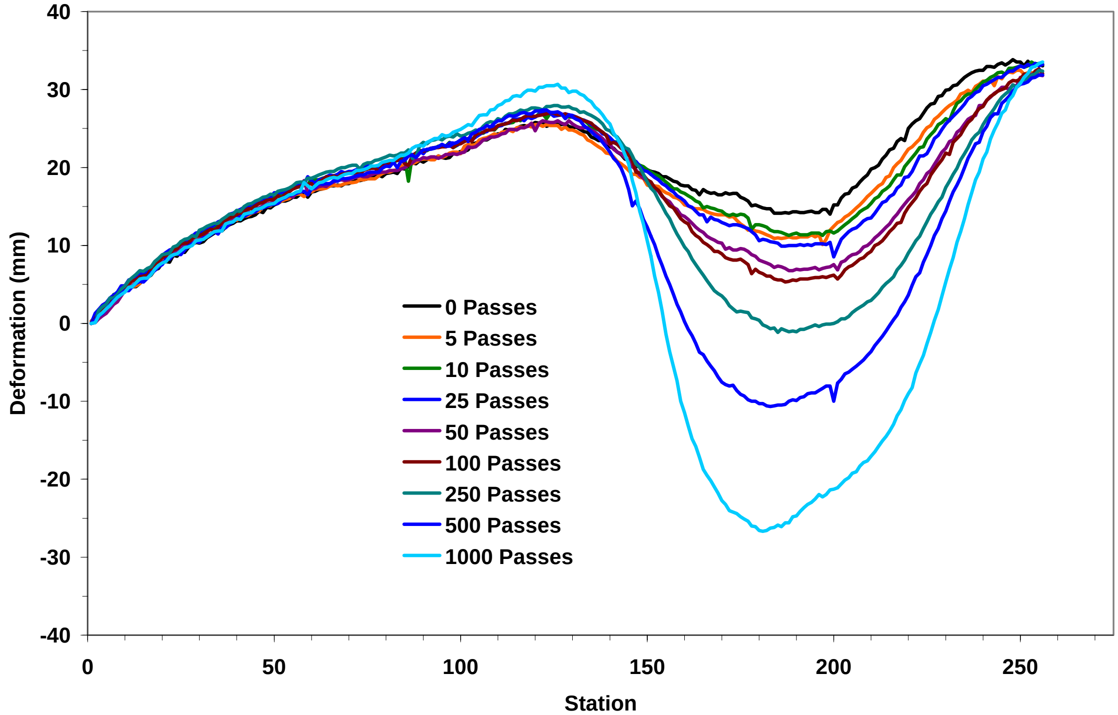


3 Profilometer High Position 3
Deformation (mm)
Station
20
10
0
-10
-20
-30
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes

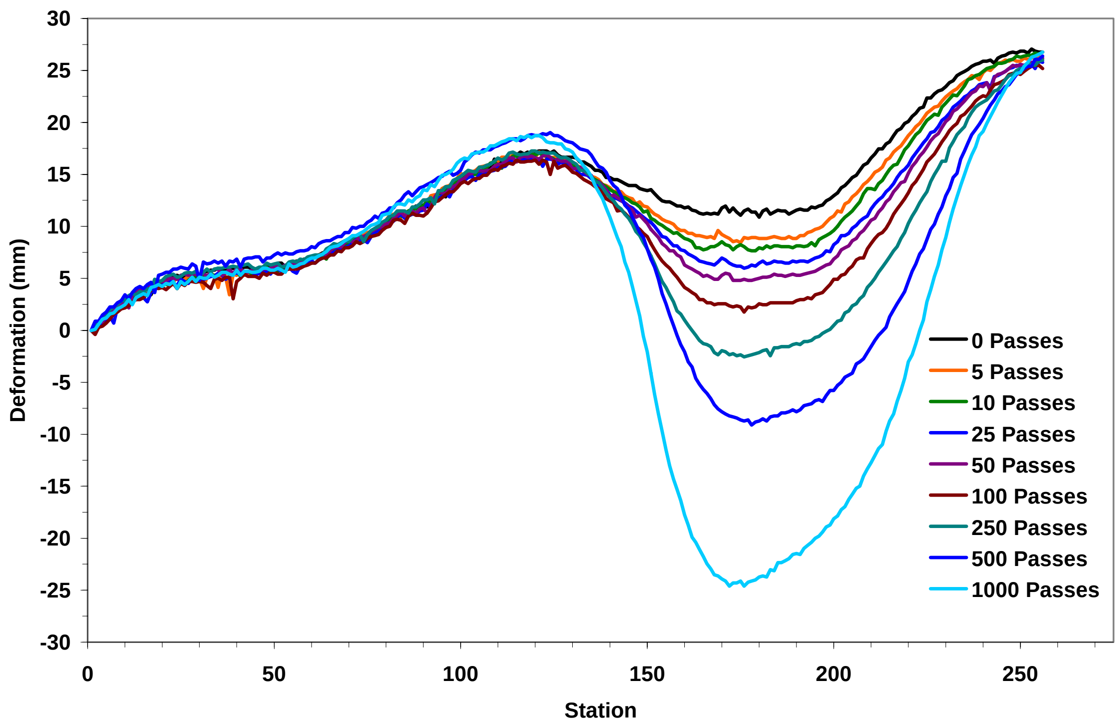



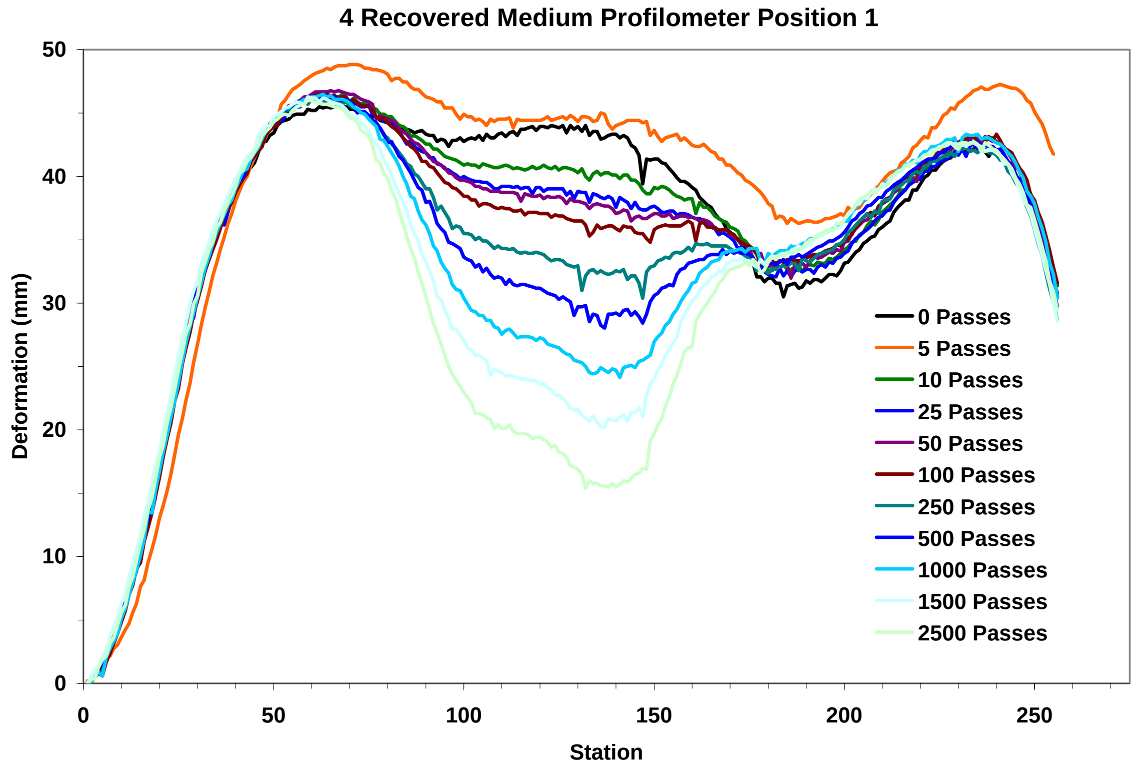
4 Recovered Low Profilometer Position 1
Deformation (mm)
Station
30
20
10
0
-10
-20
-30
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
500 Passes
1000 Passes

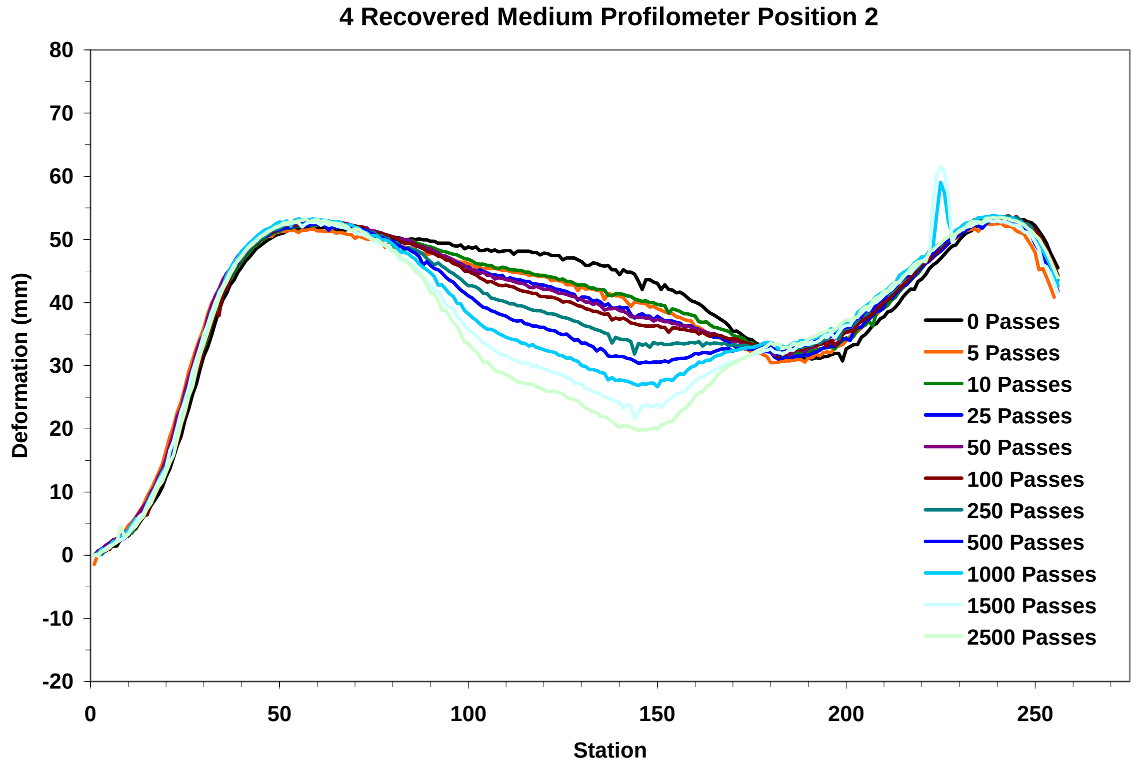
4 Recovered Low Profilometer Position 2

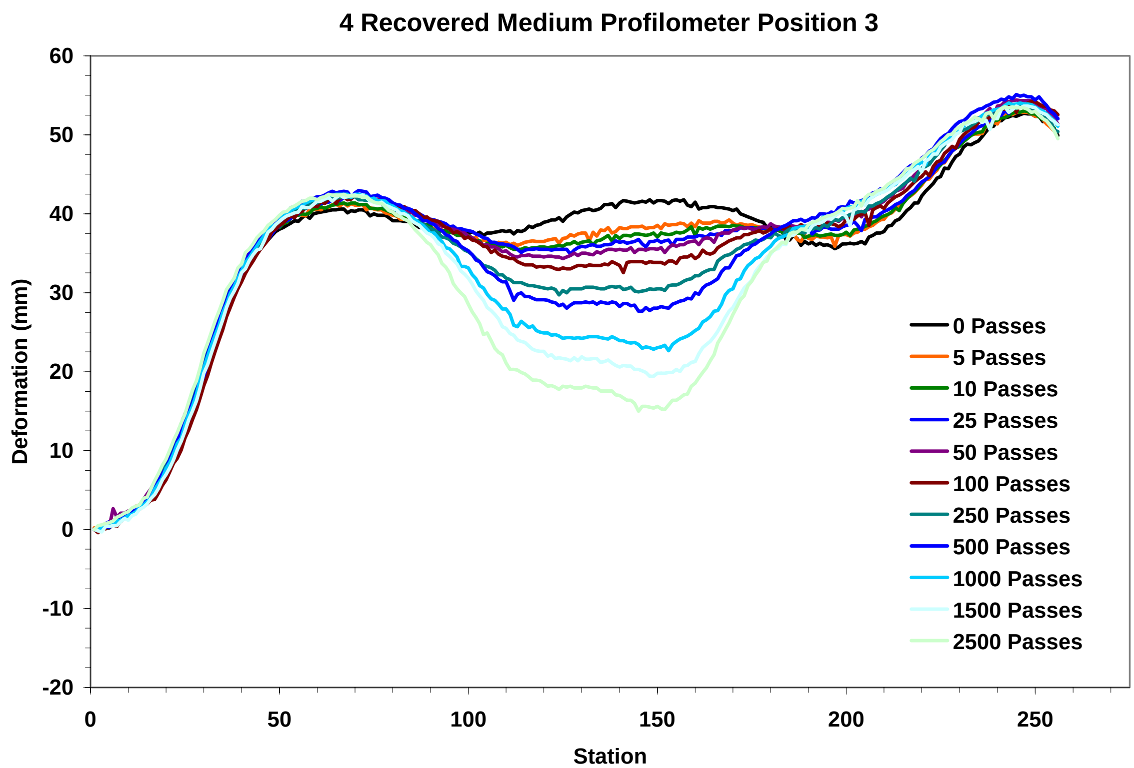


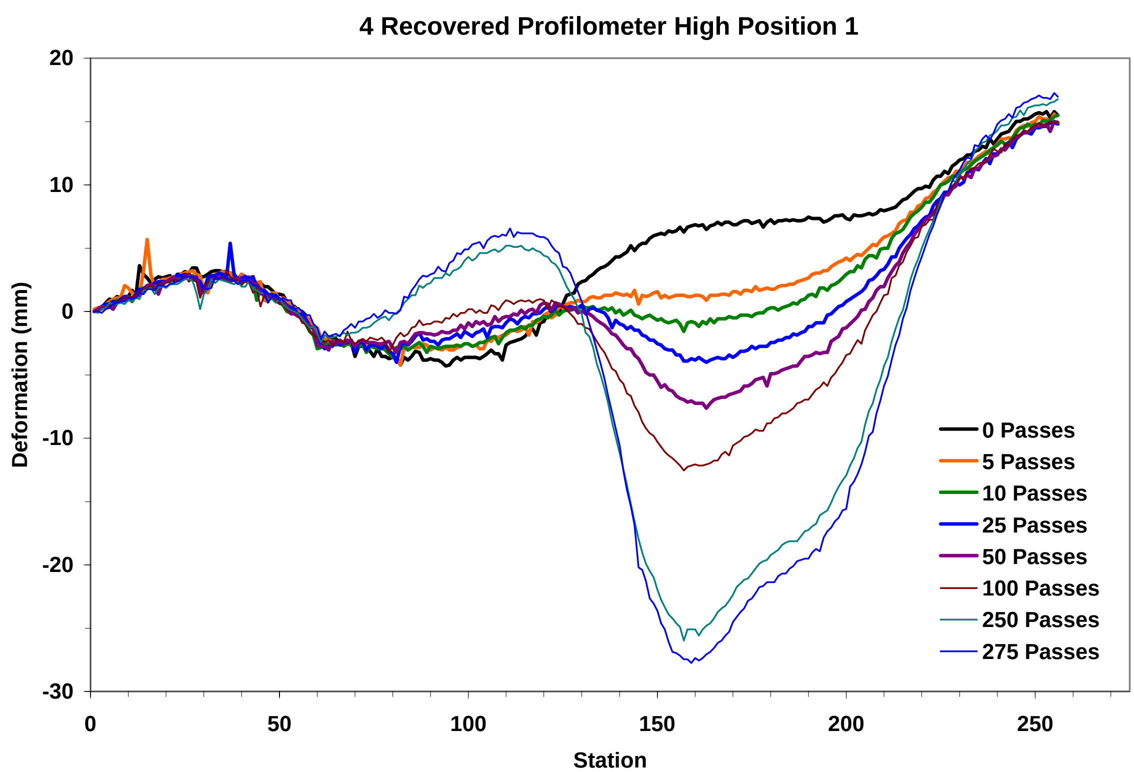

4 Recovered Low Profilometer Position 3

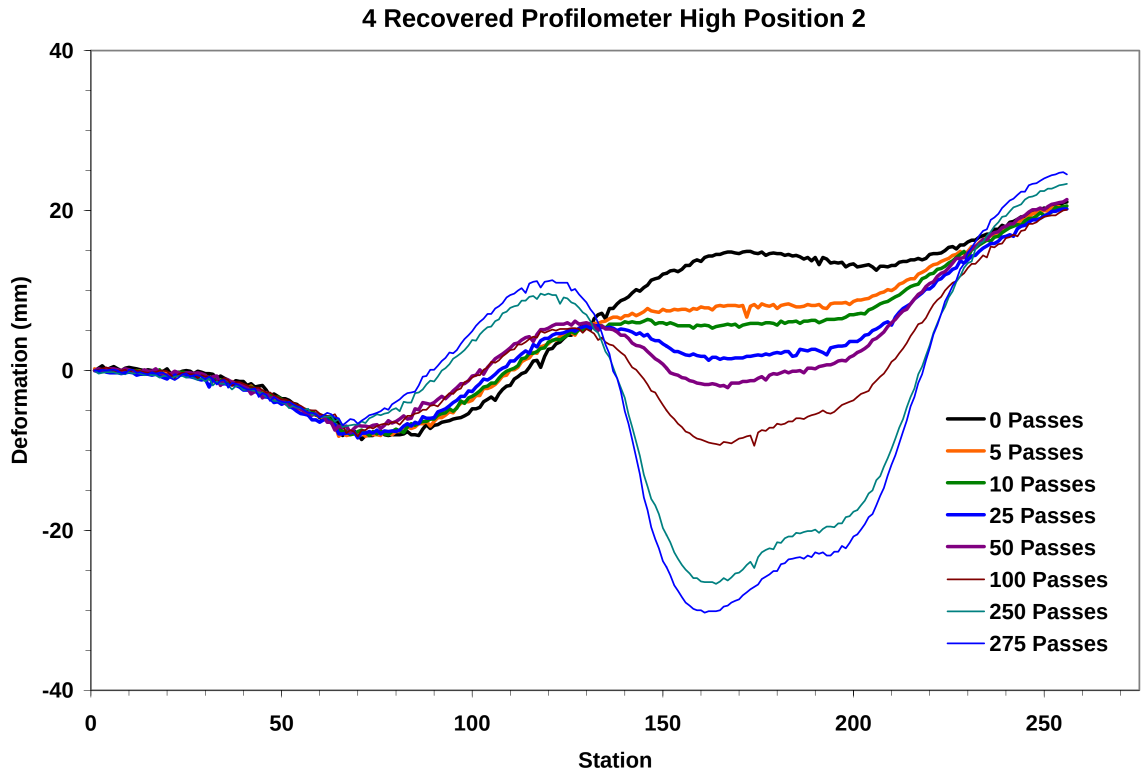



4 Recovered Medium Profilometer Position 1
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
500 Passes
1000 Passes
1500 Passes
2500 Passes

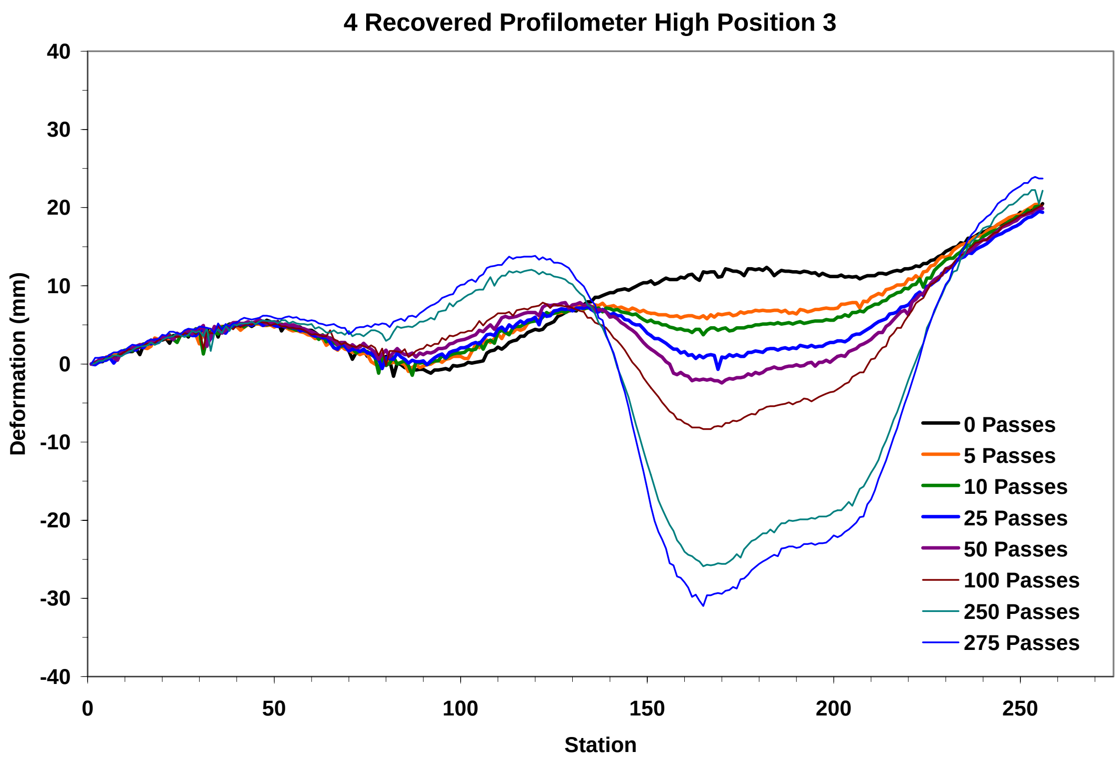

4 Recovered Medium Profilometer Position 2
Deformation (mm)
Station
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
500 Passes
1000 Passes
1500 Passes
2500 Passes


4 Recovered Medium Profilometer Position 3
Deformation (mm)
Station
60
50
40
30
20
10
0
-10
-20
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
500 Passes
1000 Passes
1500 Passes
2500 Passes


4 Recovered Profilometer High Position 1
Deformation (mm)
Station
20
10
0
-10
-20
-30
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
275 Passes

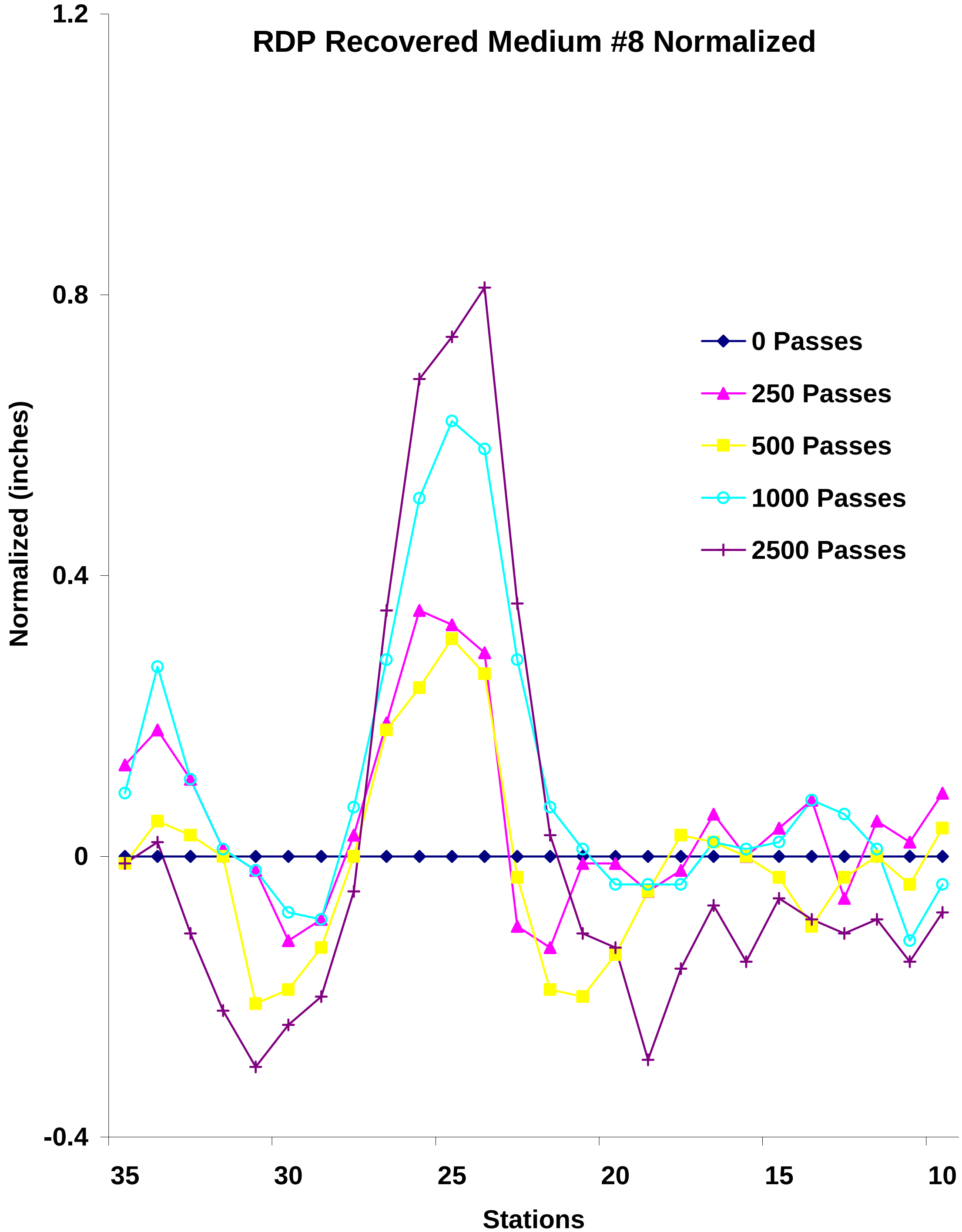


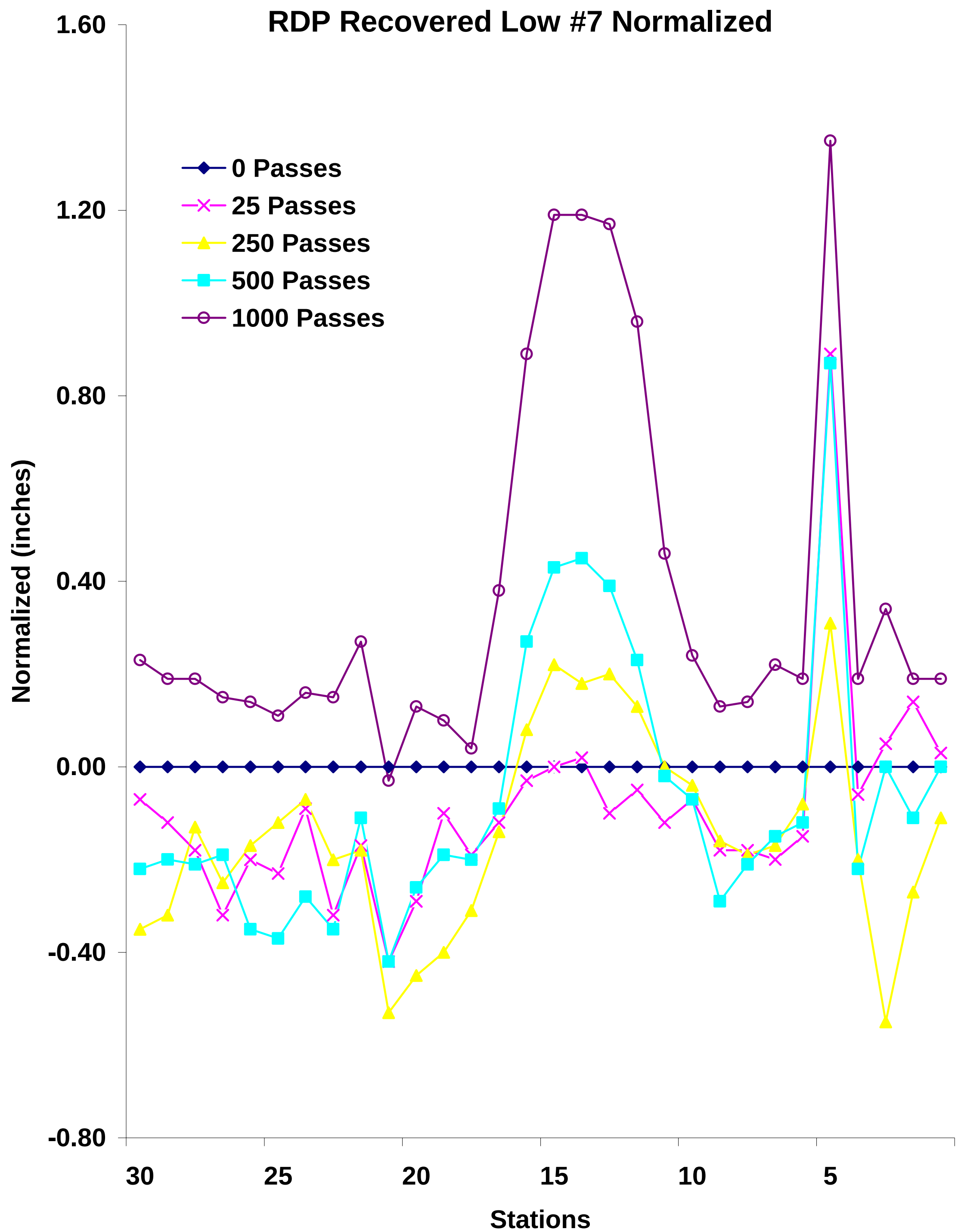
4 Recovered Profilometer High Position 2
Deformation (mm)
Station
40
20
0
-20
-40
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
275 Passes

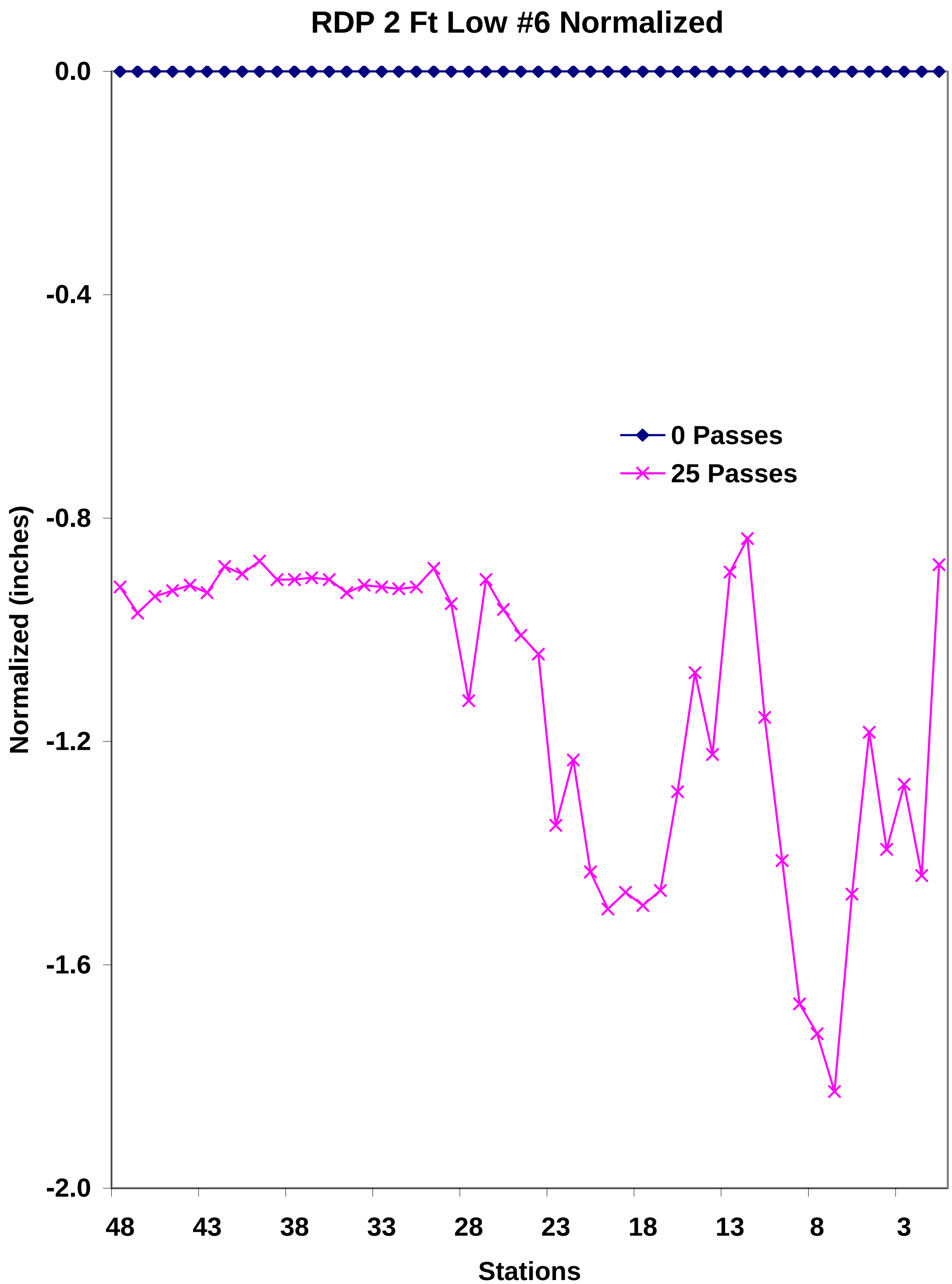



4 Recovered Profilometer High Position 3
Deformation (mm)
Station
40
30
20
10
0
-10
-20
-30
-40
0
50
100
150
200
250
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
250 Passes
275 Passes

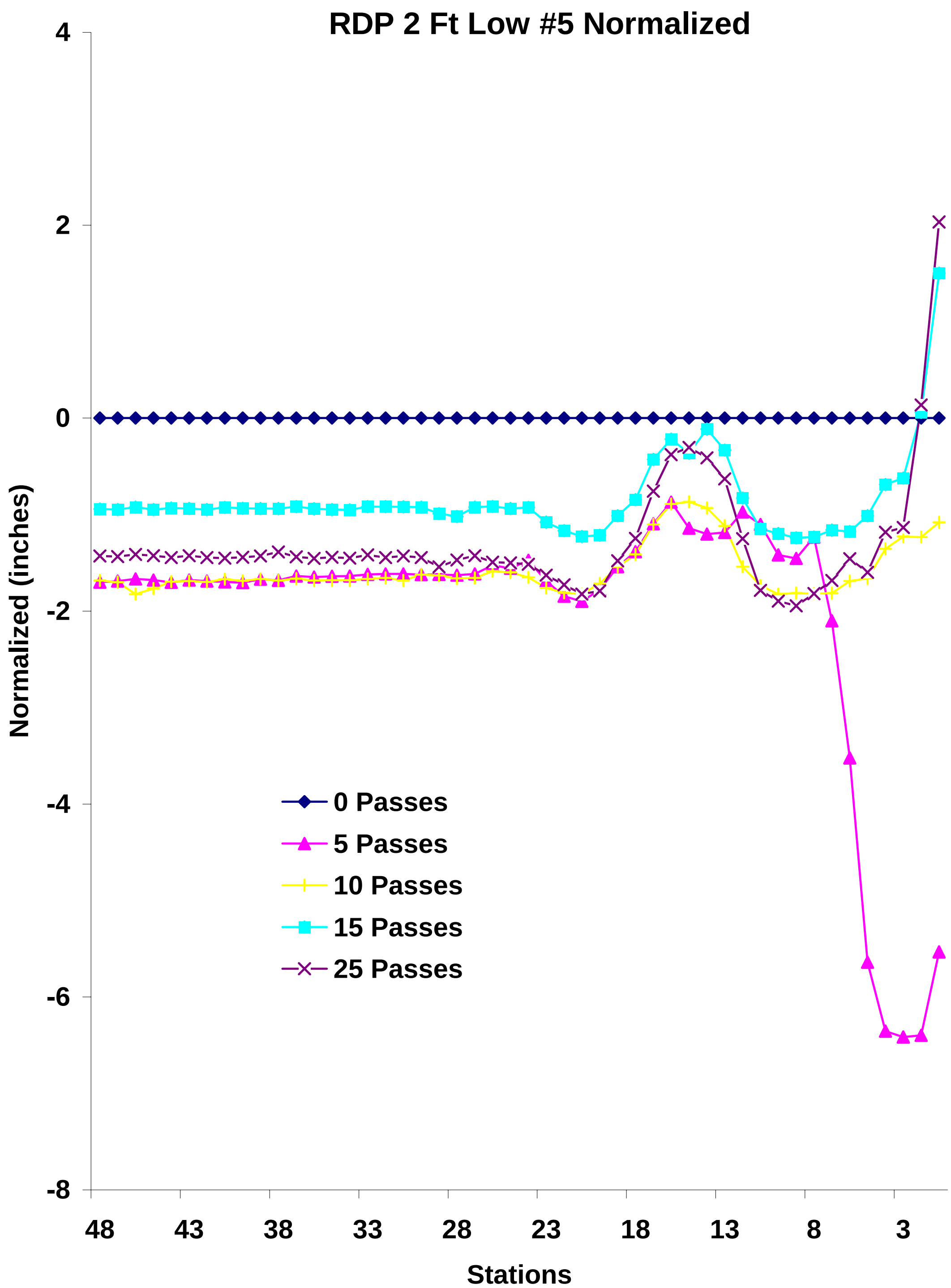
# Appendix E:
# Rut Depth Probe Data

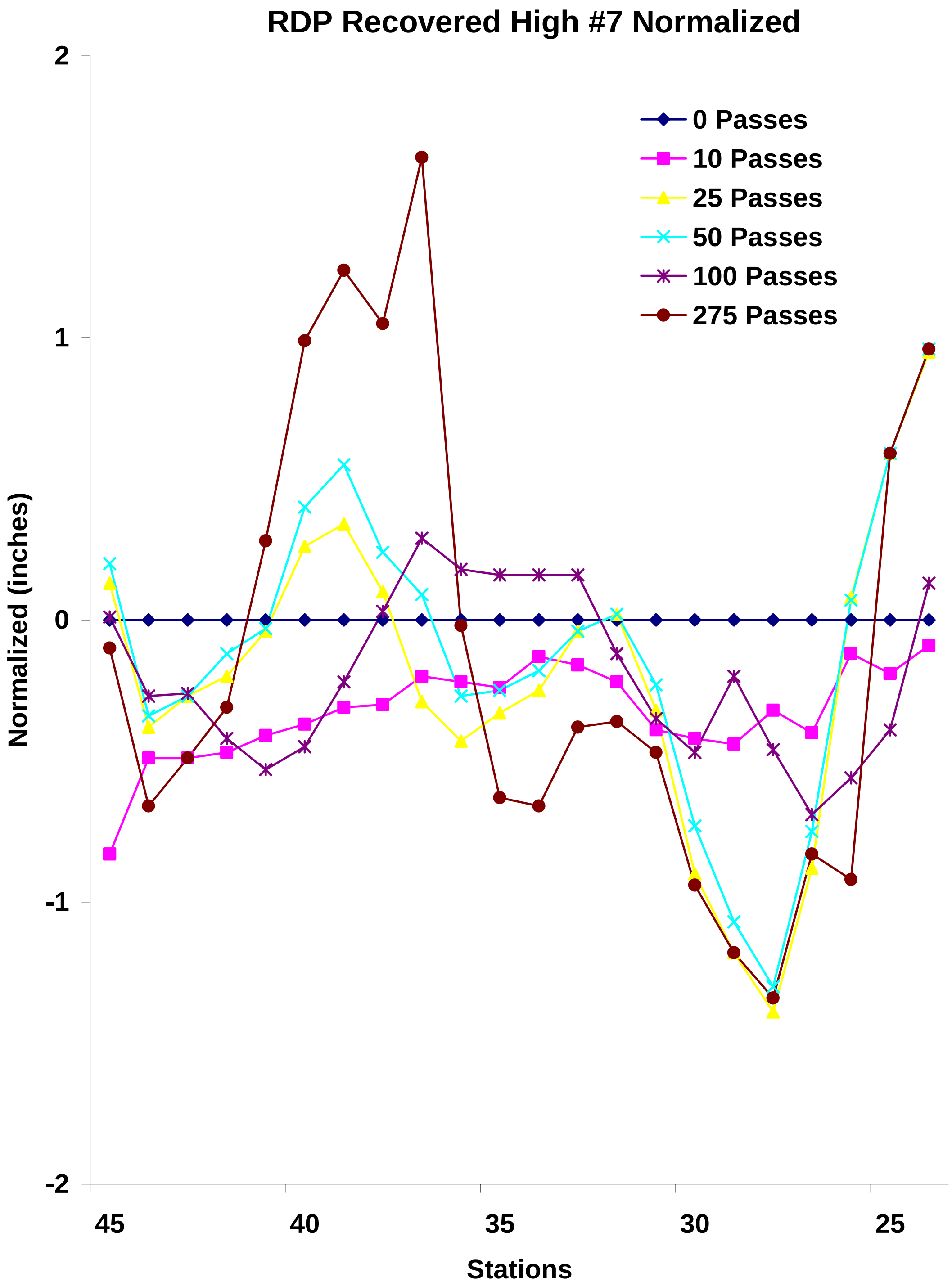

RDP Recovered Medium #8 Normalized
Normalized (inches)
1.2
0.8
0.4
0
-0.4
35
30
25
20
15
10
Stations
0 Passes
250 Passes
500 Passes
1000 Passes
2500 Passes

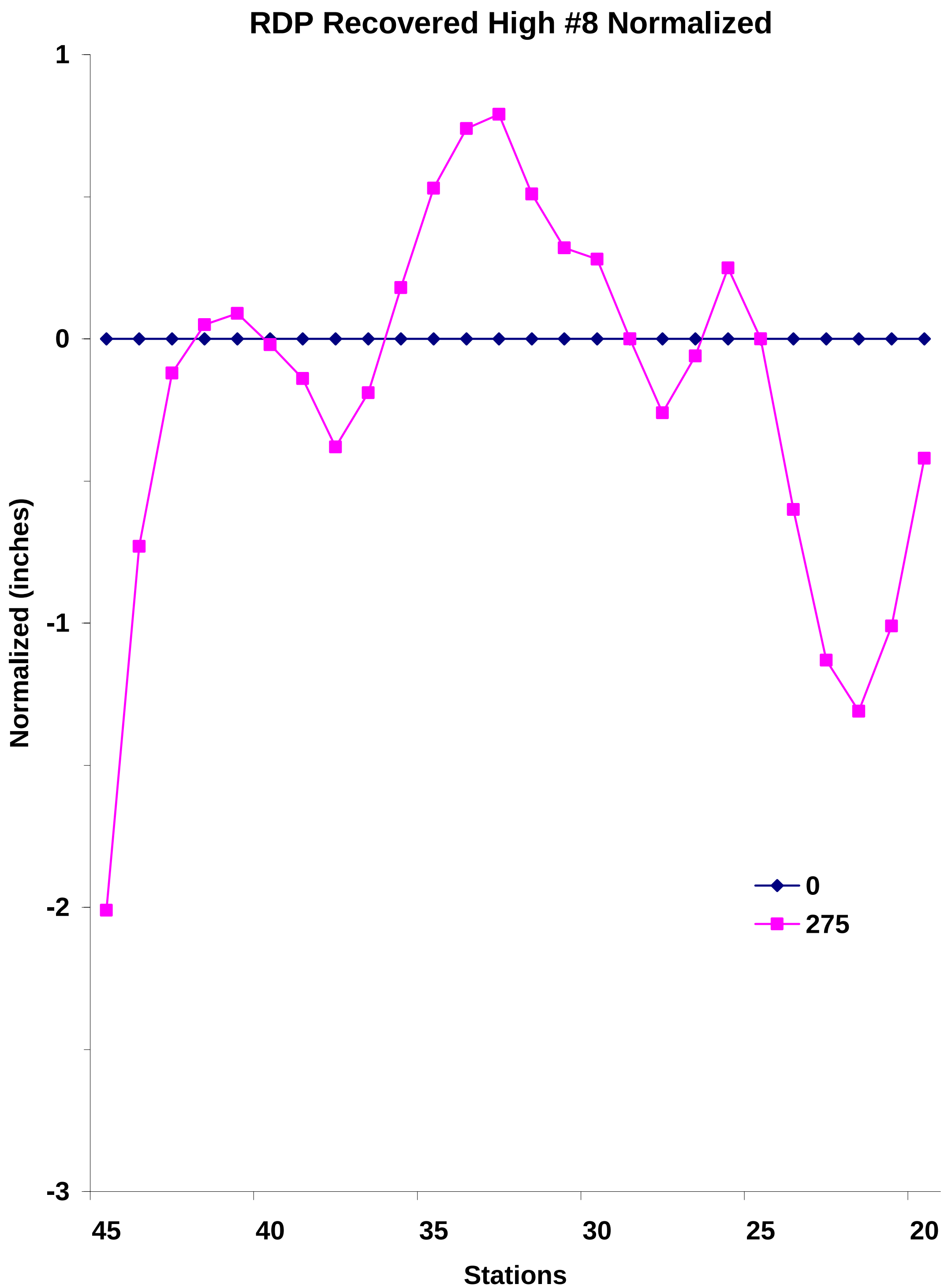

RDP Recovered Low #7 Normalized
1.60
1.20
0.80
0.40
0.00
-0.40
-0.80
Normalized (inches)
0 Passes
25 Passes
250 Passes
500 Passes
1000 Passes
30
25
20
15
10
5
Stations

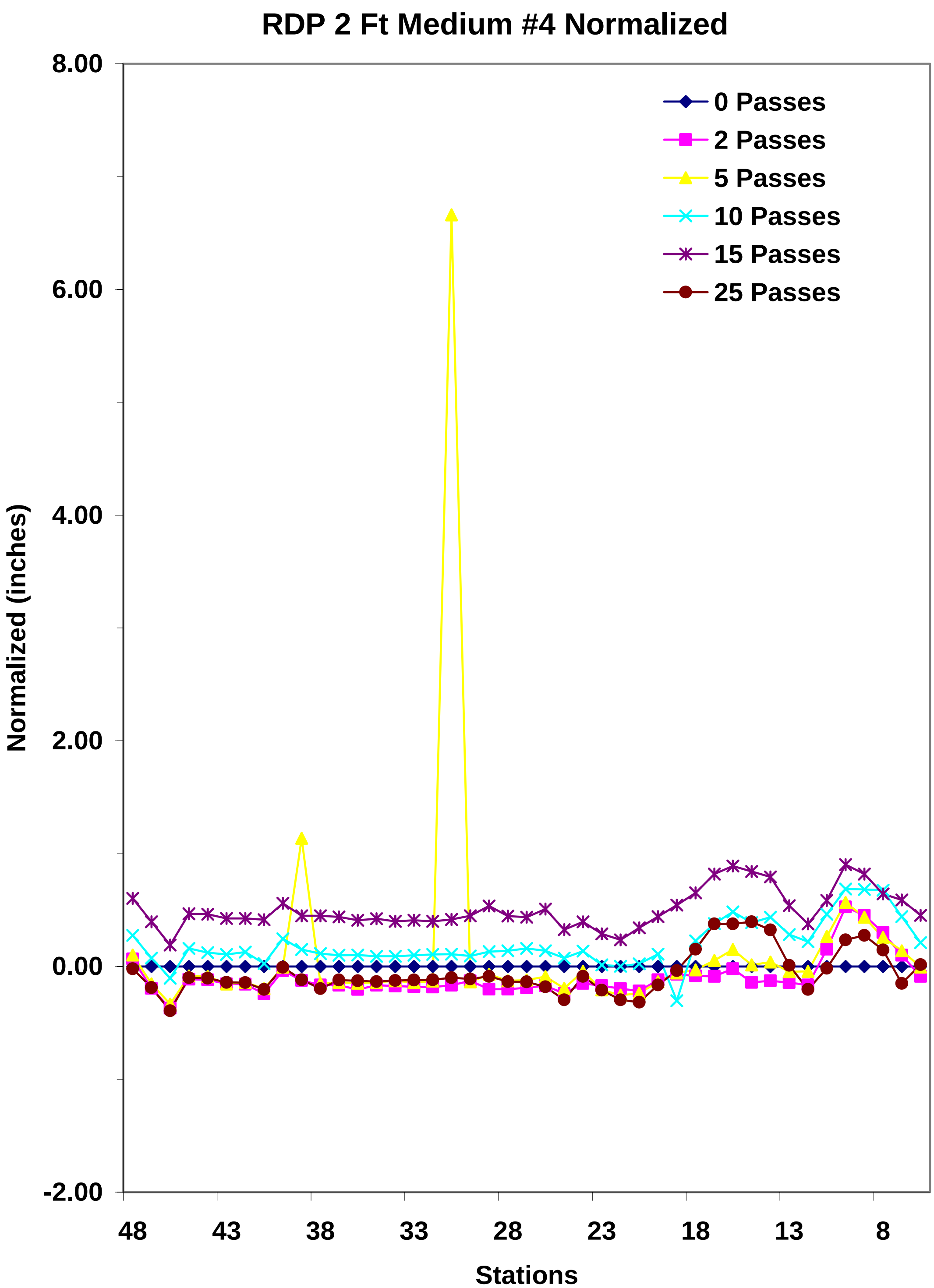


RDP 2 Ft Low #6 Normalized
0.0
-0.4
-0.8
-1.2
-1.6
-2.0
Normalized (inches)
0 Passes
25 Passes
48
43
38
33
28
23
18
13
8
3
Stations

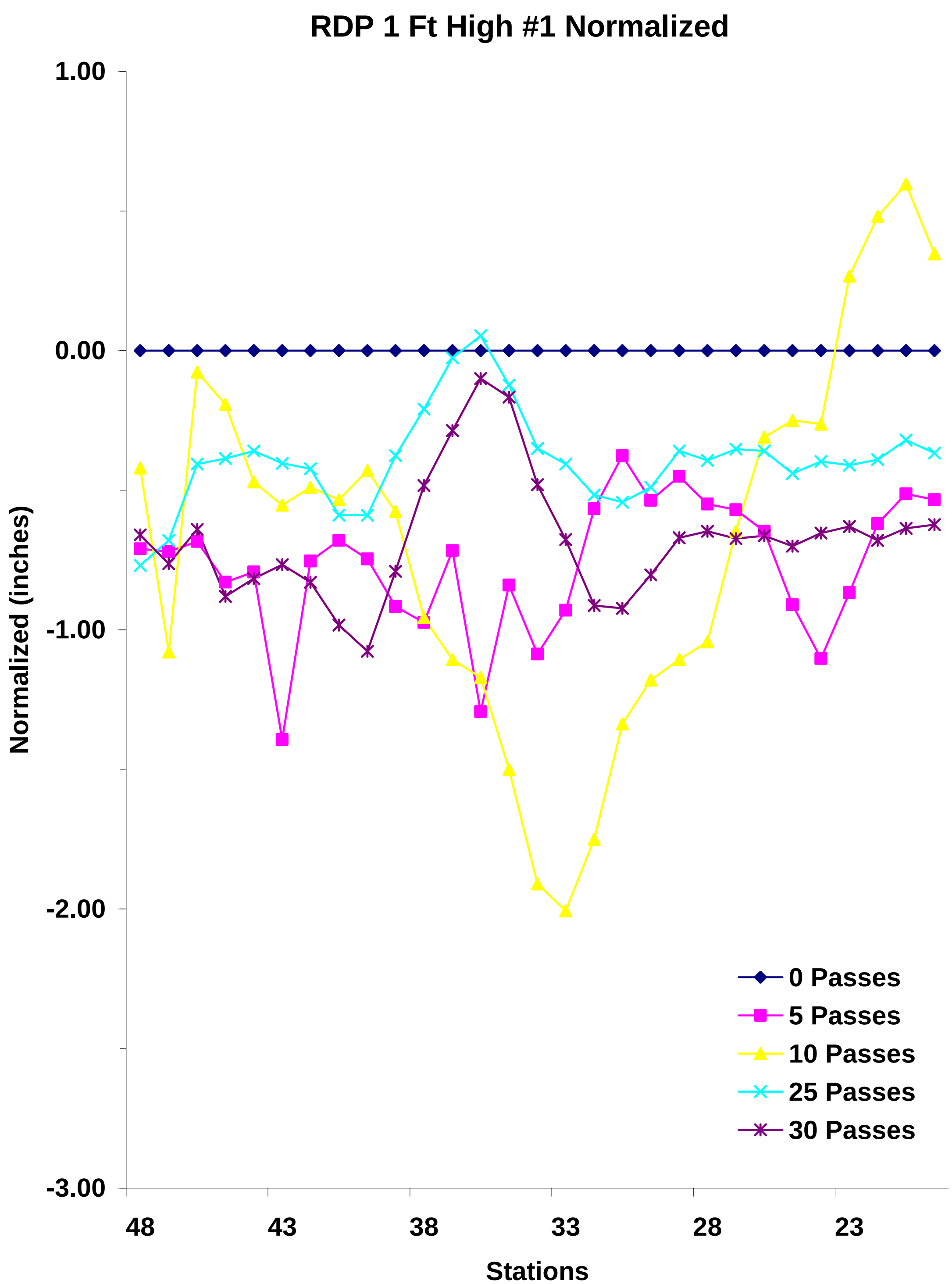

RDP 2 Ft Low #5 Normalized
Normalized (inches)
4
2
0
-2
-4
-6
-8
48
43
38
33
28
23
18
13
8
3
Stations
0 Passes
5 Passes
10 Passes
15 Passes
25 Passes

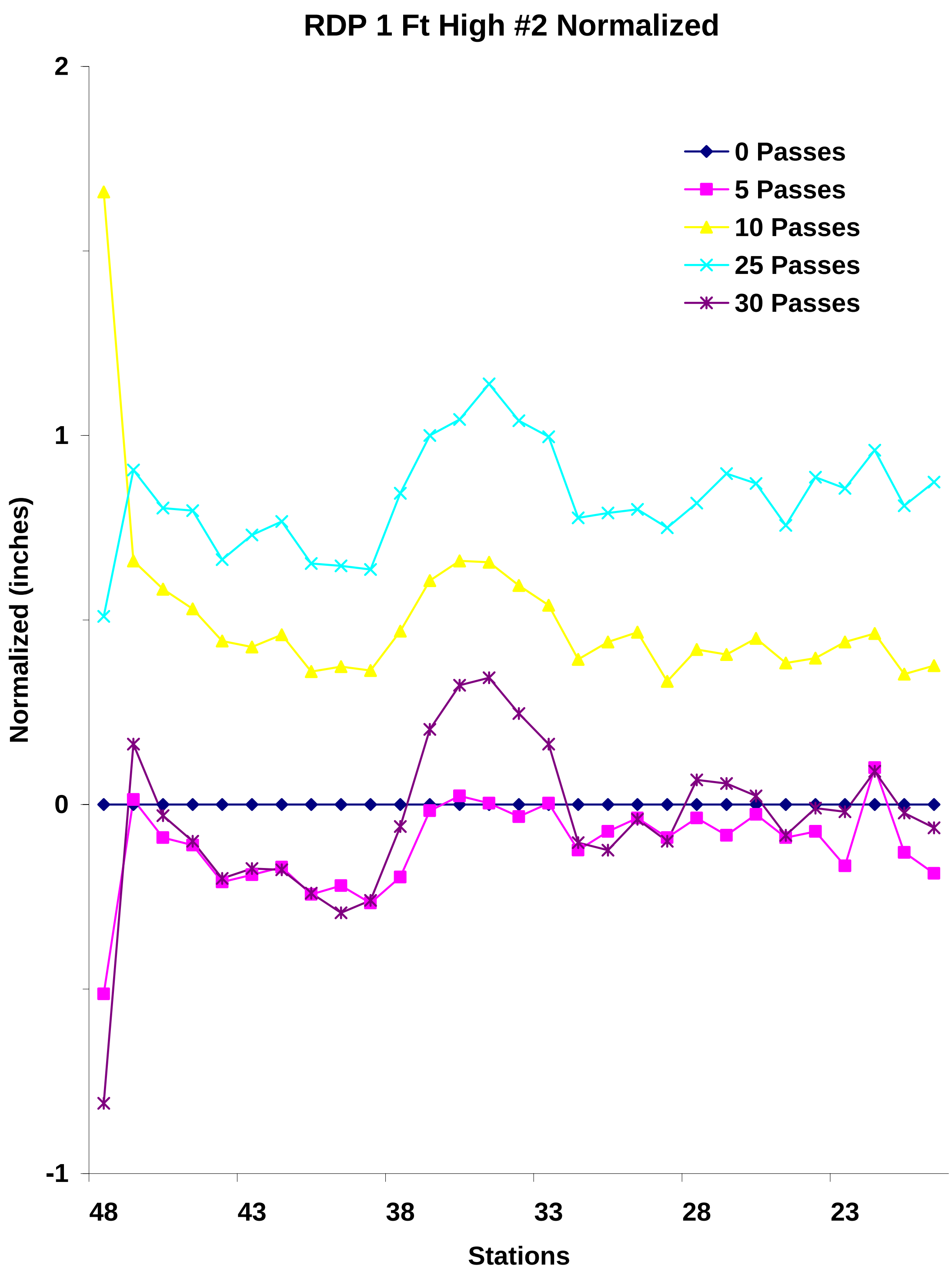

RDP Recovered High #7 Normalized
0 Passes
10 Passes
25 Passes
50 Passes
100 Passes
275 Passes
2
1
0
-1
-2
Normalized (inches)
45
40
35
30
25
Stations

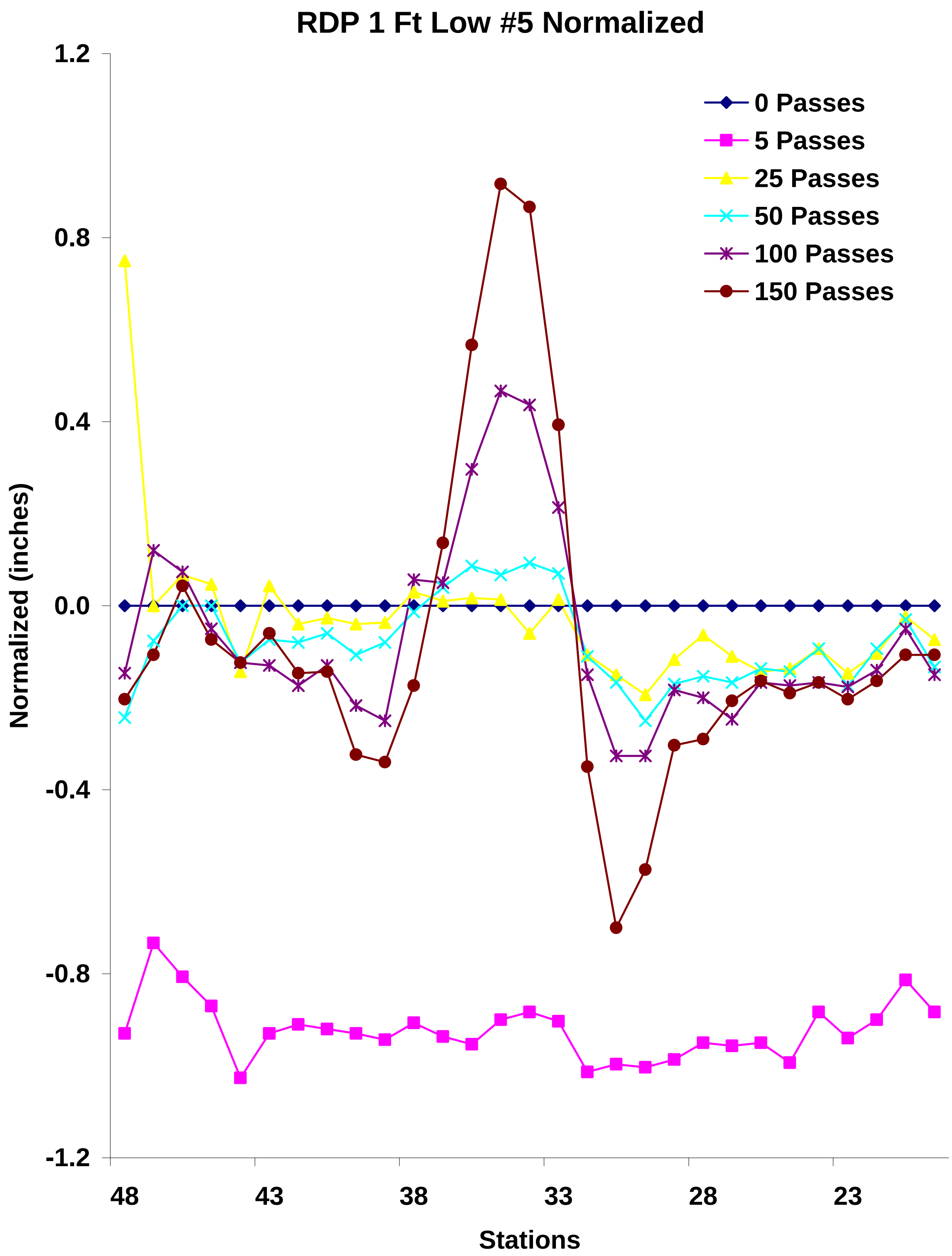

RDP Recovered High #8 Normalized
1
0
-1
-2
-3
Normalized (inches)
45
40
35
30
25
20
Stations
0
275

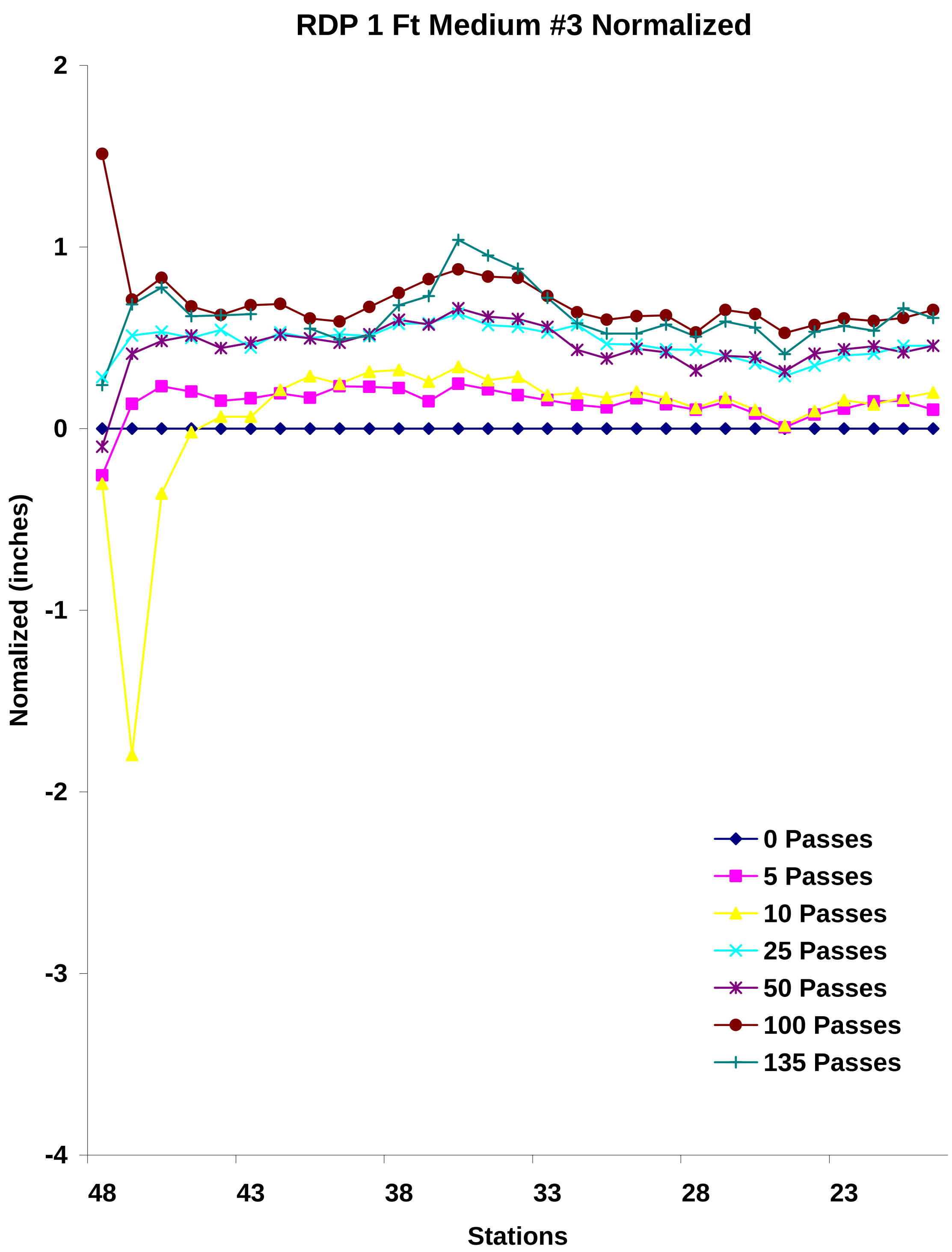


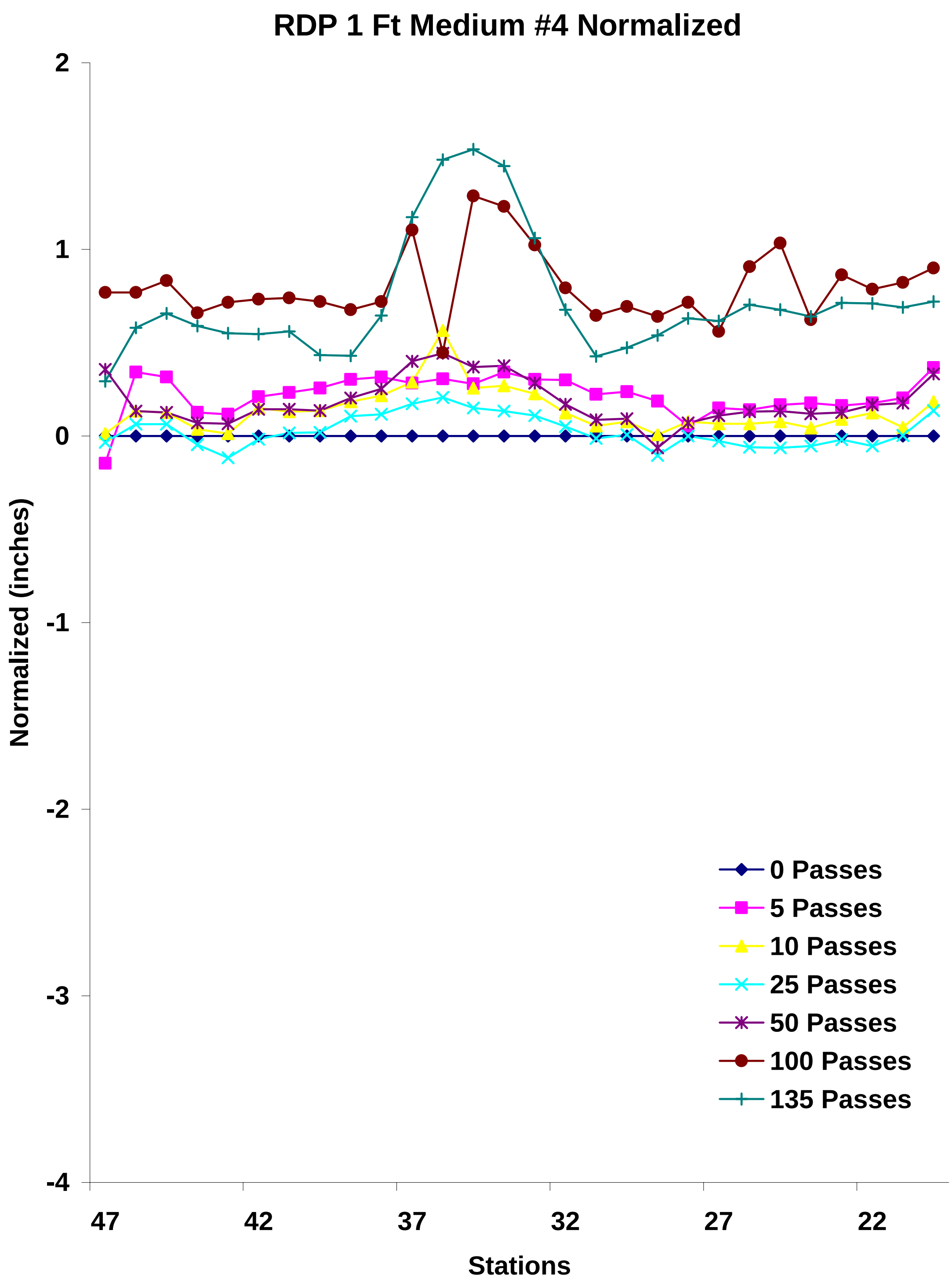
RDP 2 Ft Medium #4 Normalized
8.00
6.00
4.00
2.00
0.00
-2.00
Normalized (inches)
0 Passes
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes
48
43
38
33
28
23
18
13
8
Stations

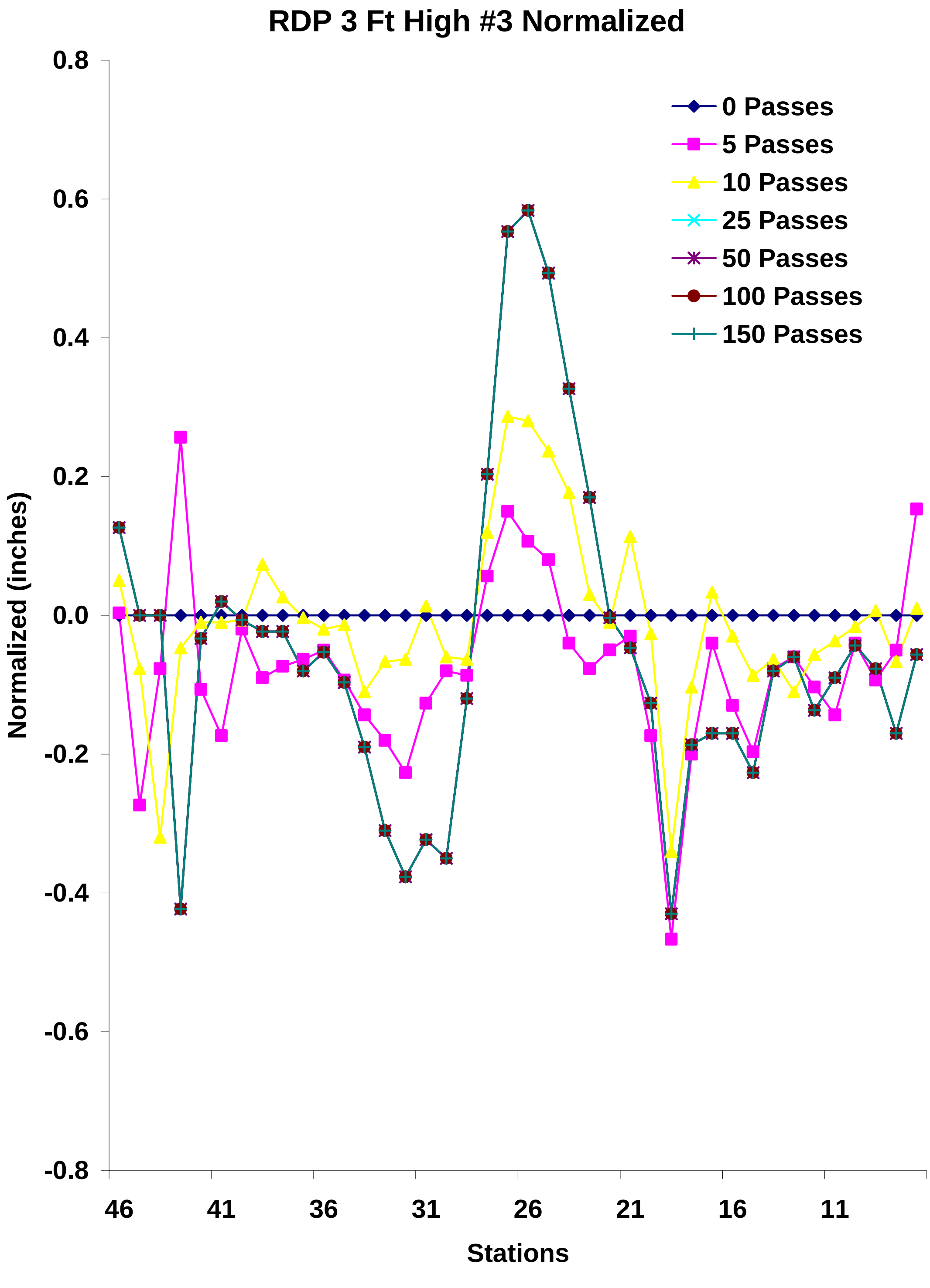

RDP 1 Ft High #1 Normalized
1.00
0.00
-1.00
-2.00
-3.00
Normalized (inches)
48
43
38
33
28
23
Stations
0 Passes
5 Passes
10 Passes
25 Passes
30 Passes


RDP 1 Ft High #2 Normalized
2
1
0
-1
Normalized (inches)
0 Passes
5 Passes
10 Passes
25 Passes
30 Passes
48
43
38
33
28
23
Stations

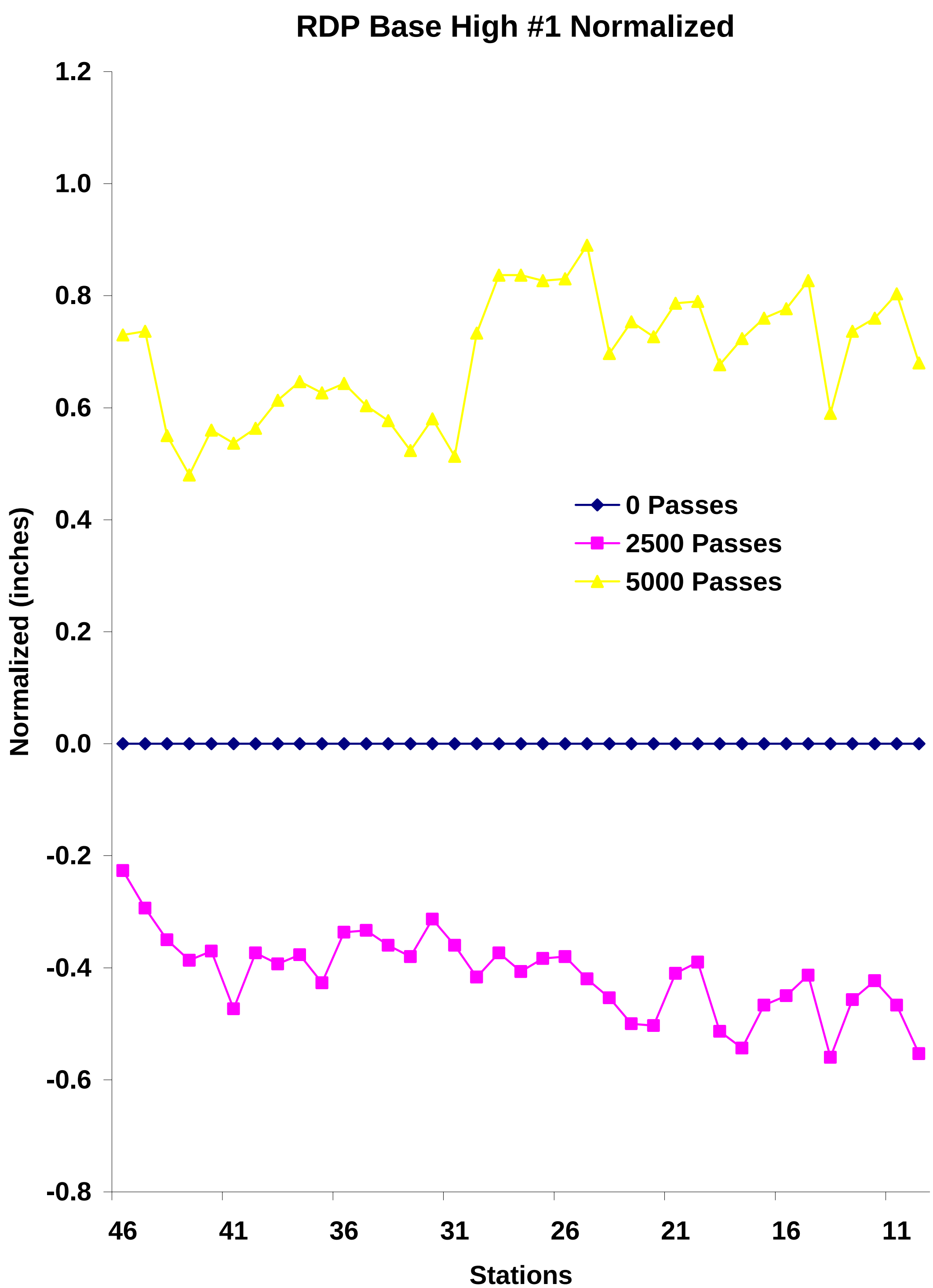

RDP 1 Ft Low #5 Normalized
1.2
0.8
0.4
0.0
-0.4
-0.8
-1.2
Normalized (inches)
0 Passes
5 Passes
25 Passes
50 Passes
100 Passes
150 Passes
48
43
38
33
28
23
Stations

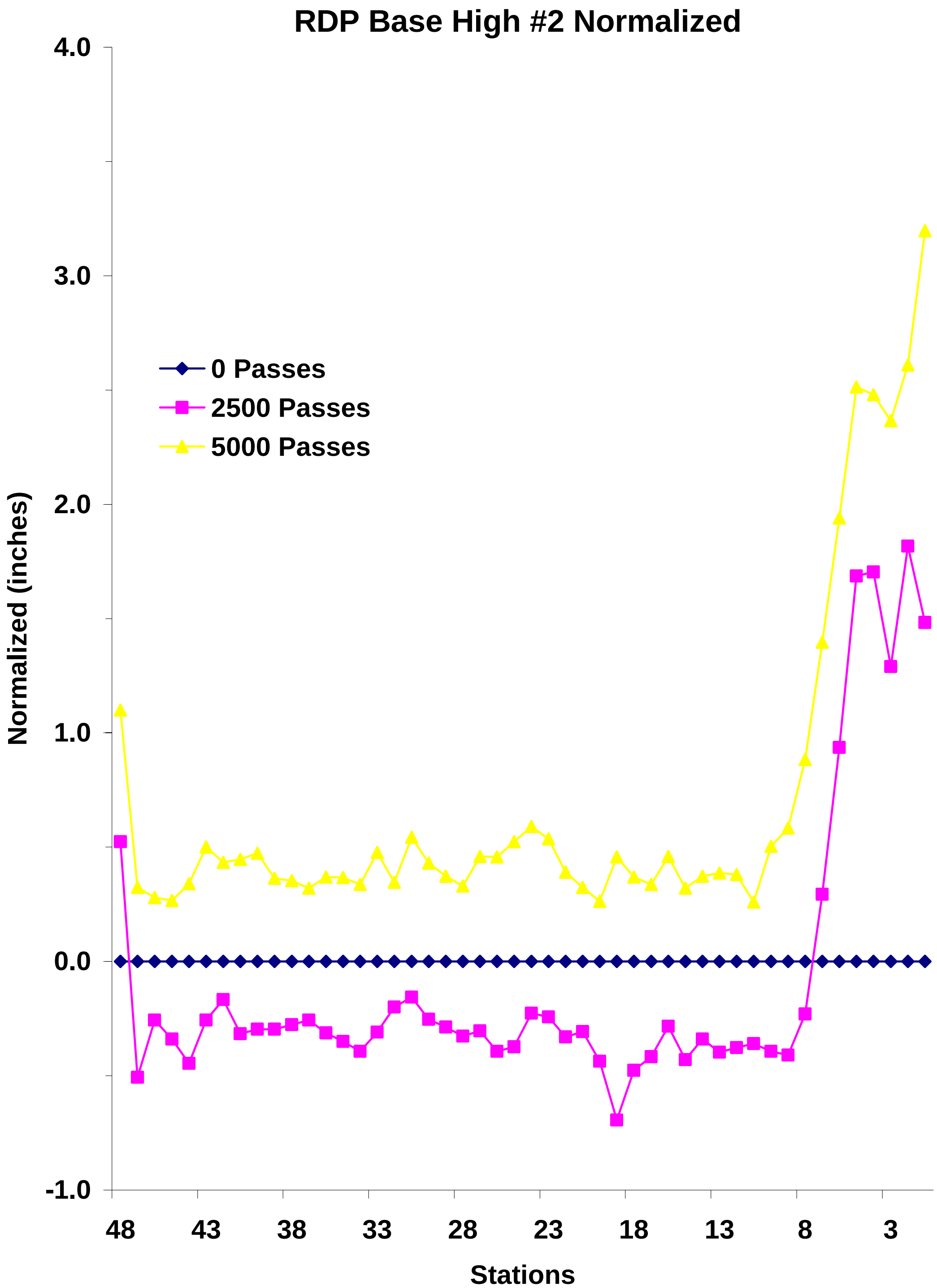

RDP 1 Ft Medium #3 Normalized
Nomalized (inches)
Stations
2
1
0
-1
-2
-3
-4
48
43
38
33
28
23
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
135 Passes

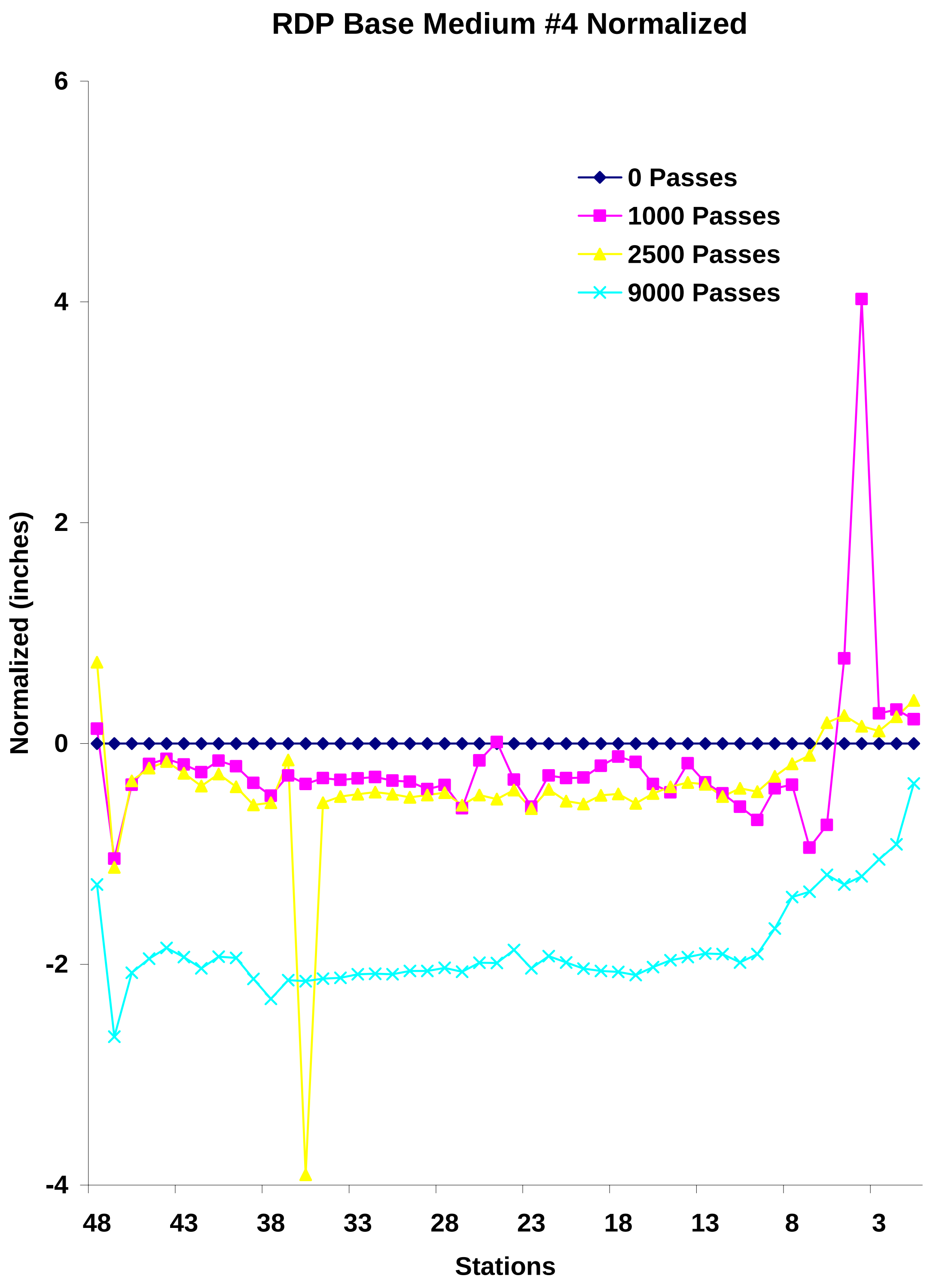

RDP 1 Ft Medium #4 Normalized
Normalized (inches)
Stations
2
1
0
-1
-2
-3
-4
47
42
37
32
27
22
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
135 Passes

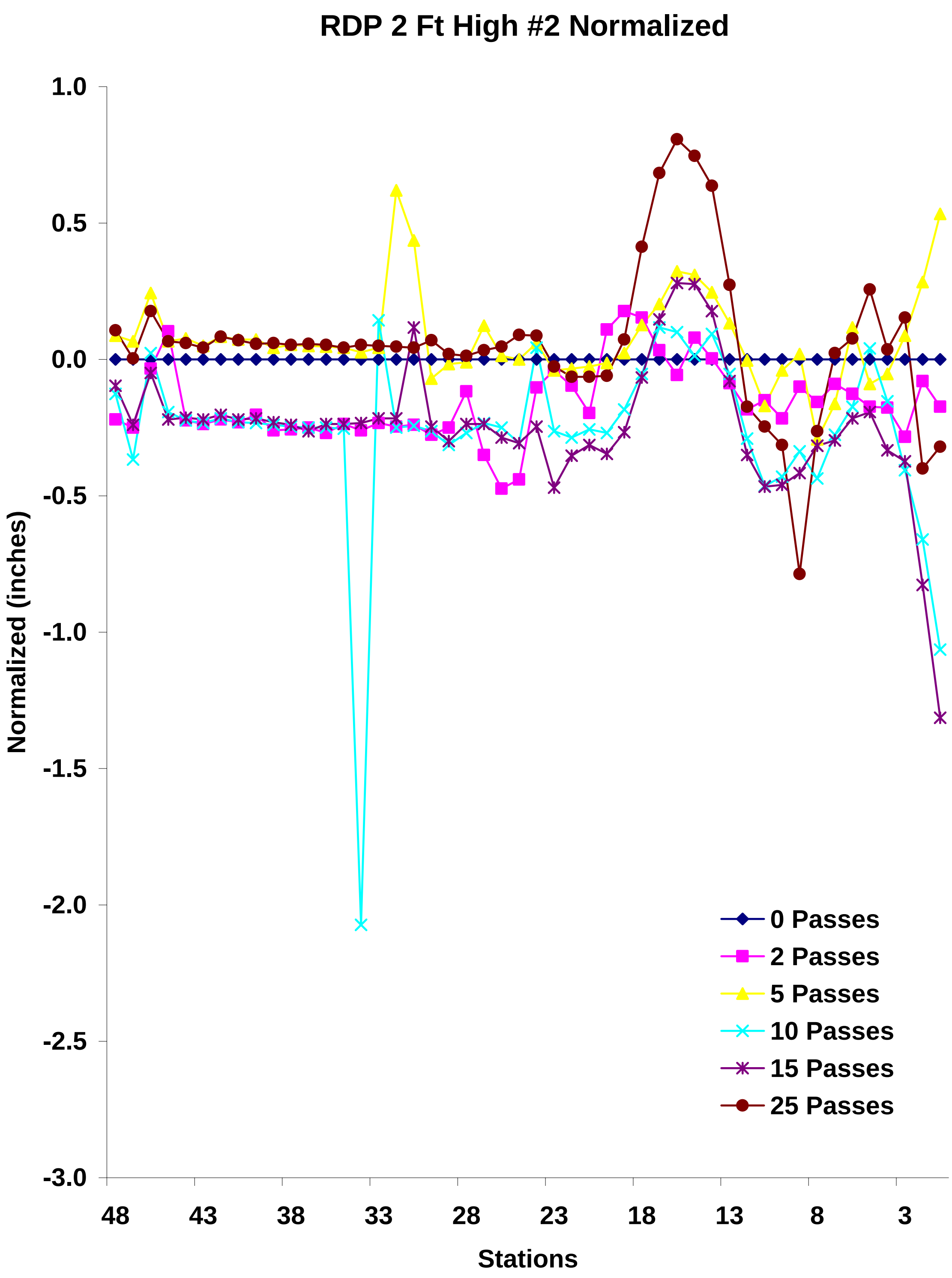


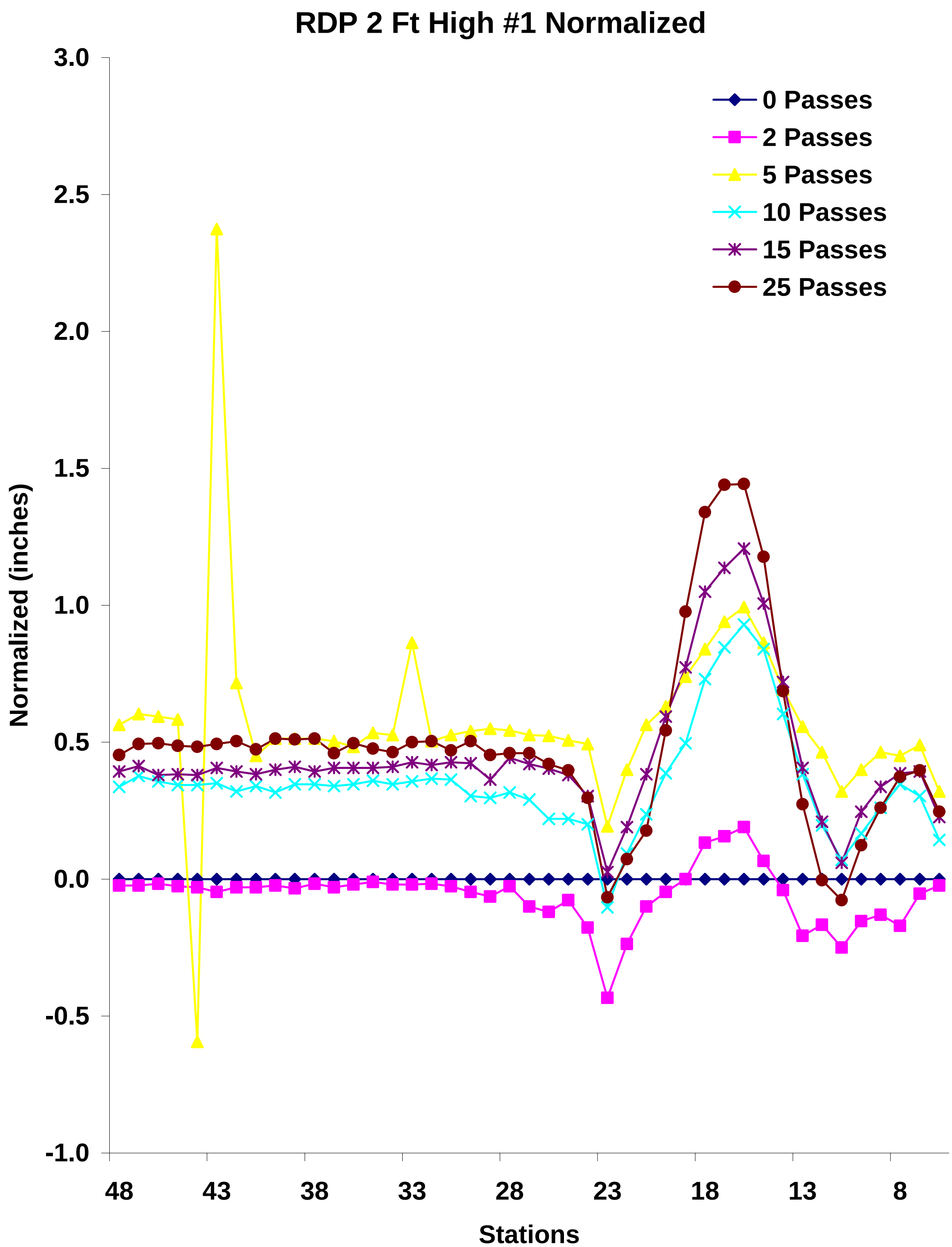
RDP 3 Ft High #3 Normalized
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
Normalized (inches)
46
41
36
31
26
21
16
11
Stations
0 Passes
5 Passes
10 Passes
25 Passes
50 Passes
100 Passes
150 Passes


RDP Base High #1 Normalized
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
Normalized (inches)
0 Passes
2500 Passes
5000 Passes
46
41
36
31
26
21
16
11
Stations

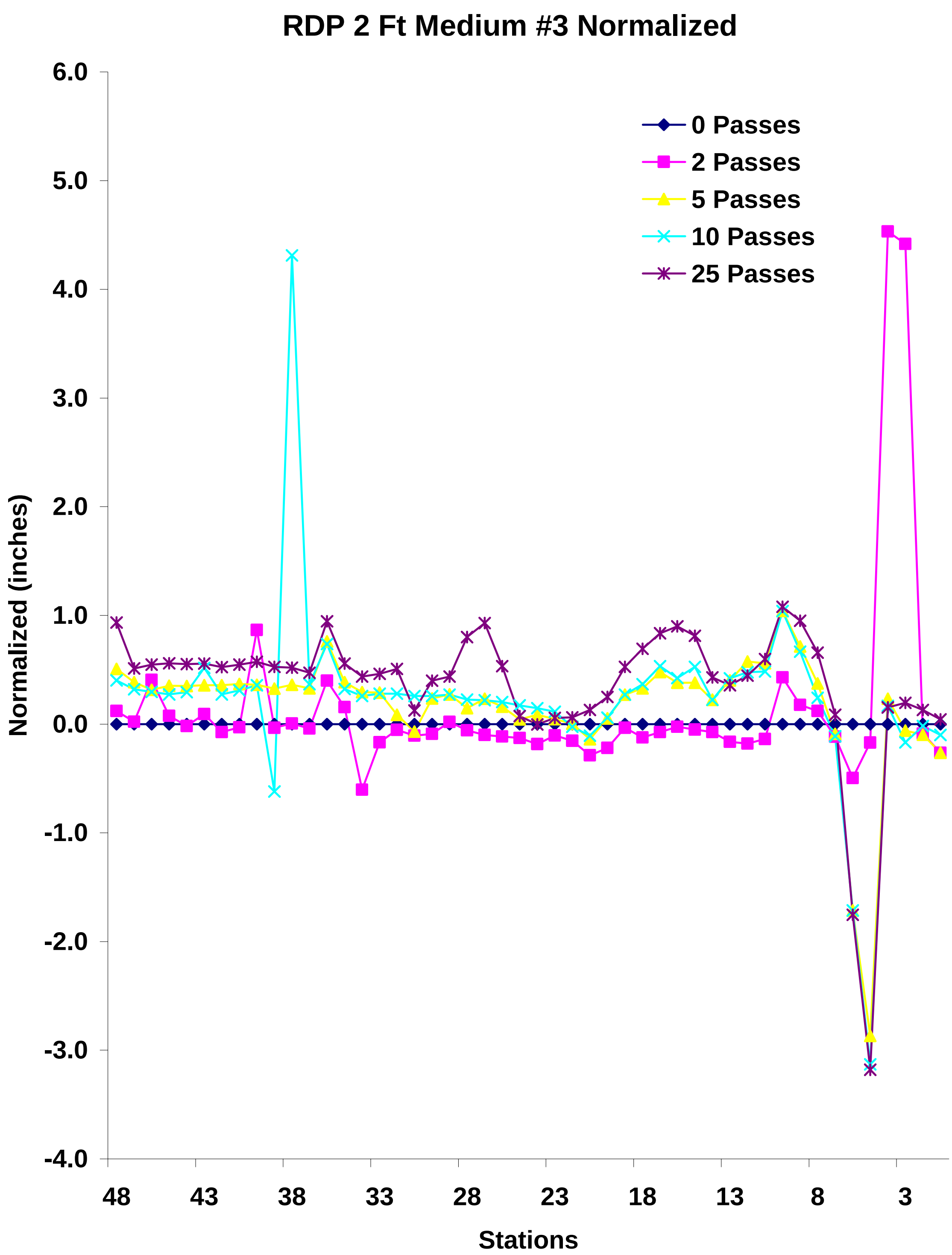

RDP Base High #2 Normalized
4.0
3.0
2.0
1.0
0.0
-1.0
Normalized (inches)
0 Passes
2500 Passes
5000 Passes
48
43
38
33
28
23
18
13
8
3
Stations

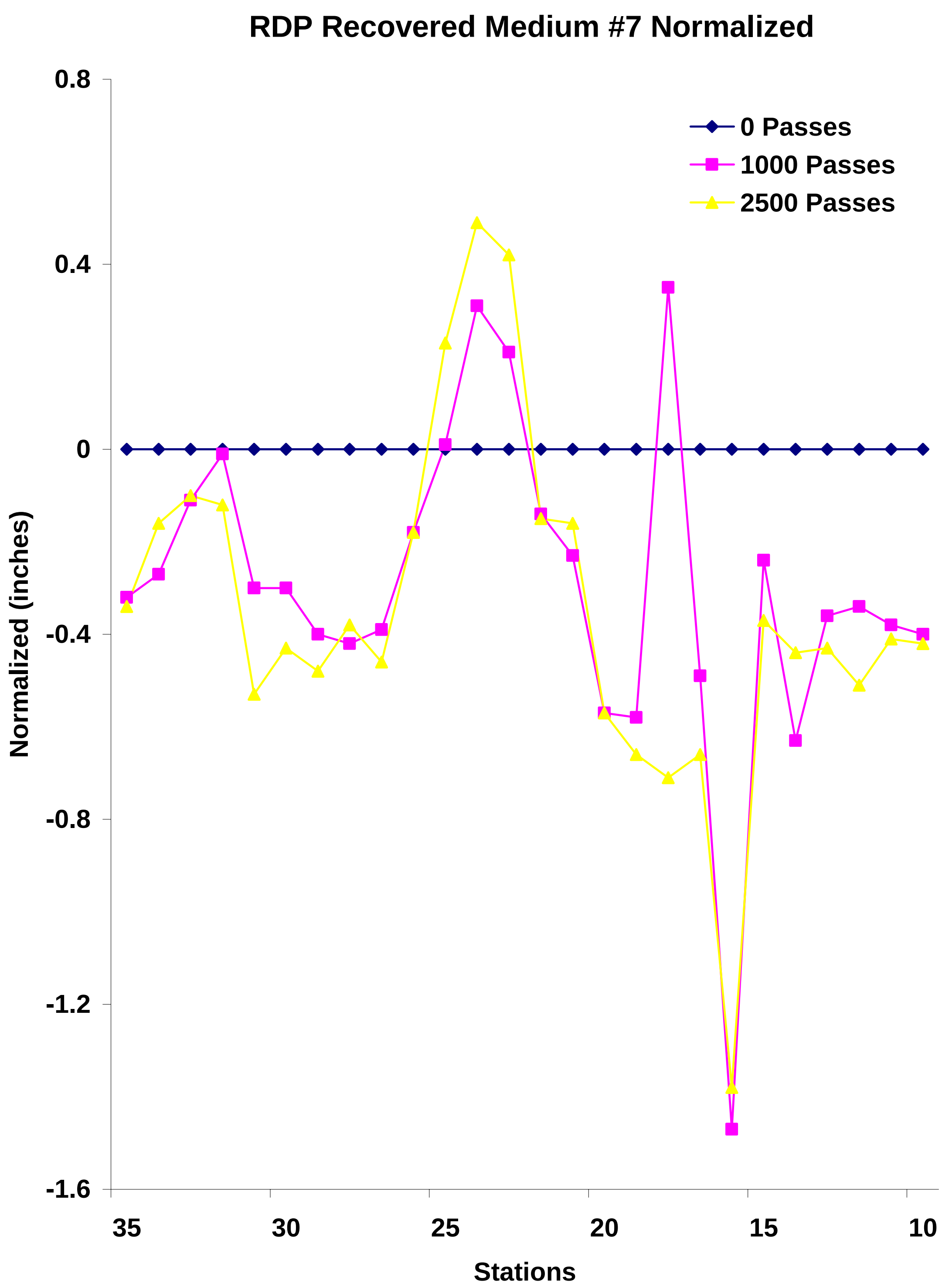

RDP Base Medium #4 Normalized
0 Passes
1000 Passes
2500 Passes
9000 Passes
Normalized (inches)
6
4
2
0
-2
-4
48
43
38
33
28
23
18
13
8
3
Stations

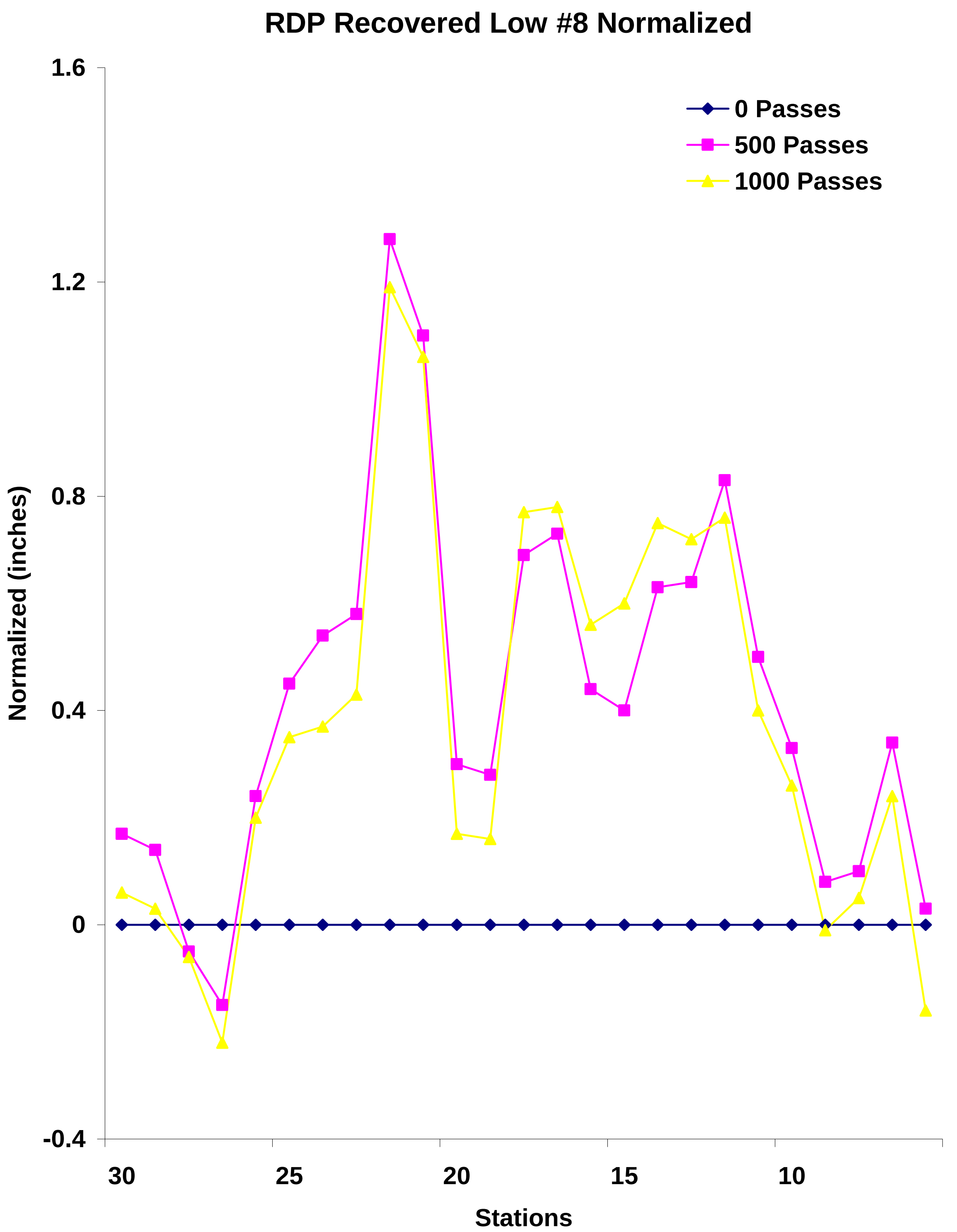

RDP 2 Ft High #2 Normalized
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
Normalized (inches)
48
43
38
33
28
23
18
13
8
3
Stations
0 Passes
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes



RDP 2 Ft High #1 Normalized
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
Normalized (inches)
0 Passes
2 Passes
5 Passes
10 Passes
15 Passes
25 Passes
48
43
38
33
28
23
18
13
8
Stations


RDP 2 Ft Medium #3 Normalized
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
-4.0
Normalized (inches)
0 Passes
2 Passes
5 Passes
10 Passes
25 Passes
48
43
38
33
28
23
18
13
8
3
Stations


RDP Recovered Medium #7 Normalized
0.8
0.4
0
-0.4
-0.8
-1.2
-1.6
Normalized (inches)
0 Passes
1000 Passes
2500 Passes
35
30
25
20
15
10
Stations


RDP Recovered Low #8 Normalized
1.6
1.2
0.8
0.4
0
-0.4
Normalized (inches)
0 Passes
500 Passes
1000 Passes
30
25
20
15
10
Stations

# Appendix F:
# Falling Weight Deflectometer Data

**FWD File from November 30, 2000**

```
R80    20941201VTP30NOV36F25
70   08002-008    000 120 .
 228  0 305 610 914121915241829 9.00 0.00 12.01 24.02 35.98 47.99 60.00 72.01
C:\DYNATEST\DATA\      .FWD
FERF-VTP
S8      40.0  40 45I51402 40 40 45
S8       4.4  4 7I51404 40 40 45
SUR-MAN AIR-MAN N0   N0
  815.0 3.5 5.0 2.015.0 2.0 8.0
LdF0332  1.002  88.0    .
D13171  0.995  0.990    .
D23172  0.998  1.047    .
D33173  0.996  1.027    .
D43174  0.995  1.028    .
D53175  0.996  1.022    .
D63176  0.997  1.019    .
D73177  0.995  0.990    .
D0NA   0.000  0.000    .
D0NA   0.000  0.000    .
D0NA   0.000  0.000    .
Charlie Smith
0010151000002 1 1       .
  0 0.0  0 0.0        .
Southwest
RoadNumrRoadway Name
RoadNumr
000+0.0 000+0.0 St
 457  0  0  0  0  0  0  0 17.99 0.00 0.00 0.00 0.00 0.00 0.00 0.00
    8   128  1860  29272
                .
CCC1111222233334444............................................................
                .
...****************..............................................................
                .
...................................................................................
VTP12  FERF
*Southwest
S1       4.4  4 7I51322 40 40 45
 141 496 252 35  4  7  5  5 5199 19.52 9.93 1.40 0.17 0.28 0.21 0.19
 141 466 235 31  4  7  5  5 5192 18.34 9.27 1.24 0.17 0.29 0.21 0.20
 142 455 229 29  3  7  6  5 5229 17.91 9.02 1.13 0.10 0.28 0.23 0.18
 142 445 224 28  2  7  6  4 5236 17.53 8.82 1.08 0.09 0.28 0.24 0.17
 251 666 335 35  5 10 10  8 9266 26.23 13.19 1.36 0.21 0.40 0.39 0.32
 252 661 333 35  5 10 10  8 9285 26.04 13.12 1.38 0.20 0.39 0.39 0.32
 253 655 331 35  5 10 10  8 9329 25.79 13.02 1.36 0.20 0.39 0.39 0.32
 253 646 326 34  6 10 10  8 9322 25.43 12.83 1.33 0.22 0.40 0.40 0.33
 352 806 403 35  5 11 13 11 12980 31.74 15.85 1.36 0.18 0.43 0.51 0.43
 353 803 402 34  6 11 13 11 12998 31.62 15.81 1.33 0.22 0.44 0.52 0.43
 353 796 398 33  6 11 13 11 13028 31.33 15.68 1.30 0.22 0.45 0.51 0.43
 353 787 394 32  6 11 13 11 13028 30.96 15.49 1.27 0.24 0.45 0.52 0.43
 460 944 475 30  1 13 16 14 16955 37.16 18.68 1.17 0.05 0.50 0.63 0.55
 460 940 472 28  2 14 16 14 16973 37.01 18.58 1.12 0.09 0.56 0.65 0.57
```

```
 461 931 467 27  3 15 16 14 16992 36.65 18.38 1.08 0.11 0.57 0.65 0.56
 460 918 461 27  3 15 16 14 16944 36.16 18.13 1.05 0.13 0.58 0.65 0.55
S2       4.4  4 7I51333 40 40 45
 132 884 440 69  2  3  5  4 4860 34.80 17.33 2.70 0.08 0.13 0.19 0.17
 133 874 437 70  3  4  6  3 4923 34.40 17.20 2.74 0.13 0.17 0.23 0.13
 133 855 428 68  3  4  5  4 4923 33.68 16.86 2.69 0.11 0.16 0.21 0.14
 134 843 421 67  3  4  5  4 4941 33.20 16.58 2.62 0.13 0.17 0.19 0.17
 2371269 620 77  1  6 10  6 8739 49.95 24.40 3.04 0.06 0.24 0.37 0.25
 2371271 623 80  2  7 10  6 8721 50.02 24.54 3.13 0.08 0.28 0.38 0.25
 2371265 621 79  2  6  9  7 8721 49.82 24.45 3.12 0.09 0.25 0.36 0.27
 2371265 622 80  1  7 10  7 8750 49.79 24.48 3.14 0.04 0.28 0.39 0.26
 3421584 770 80  0  4 11 10 12611 62.37 30.31 3.16 0.00 0.15 0.44 0.41
 3421598 779 81  0  5 11 10 12592 62.90 30.67 3.19 0.02 0.18 0.44 0.41
 3411596 781 81  0  4 11 11 12585 62.85 30.75 3.18 0.01 0.16 0.44 0.42
 3421597 782 80  0  5 12 10 12629 62.87 30.78 3.16 0.00 0.20 0.48 0.39
 4631898 914 73  0  3 12 14 17054 74.72 35.97 2.89 0.00 0.10 0.46 0.56
 4631909 924 72  0  6 14 14 17054 75.17 36.39 2.85 0.01 0.23 0.53 0.55
 4631907 925 72  0  6 13 14 17054 75.07 36.42 2.82 0.01 0.24 0.52 0.55
 4641903 925 71  0  7 13 14 17091 74.93 36.42 2.80 0.00 0.26 0.51 0.54
S3       4.4  4 7I51339 40 40 45
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**FWD File from December 4, 2000**

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**FWD File from December 18, 2000**

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D43174  0.995  1.028    .
D53175  0.996  1.022    .
D63176  0.997  1.019    .
D73177  0.995  0.990    .
D0NA   0.000  0.000    .
D0NA   0.000  0.000    .
D0NA   0.000  0.000    .
Charlie Smith
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Southwest
RoadNumrRoadway Name
RoadNumr
000+0.0 000+0.0 St
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**FWD File from December 22, 2000**

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D23172  0.998  1.047    .
D33173  0.996  1.027    .
D43174  0.995  1.028    .
D53175  0.996  1.022    .
D63176  0.997  1.019    .
D73177  0.995  0.990    .
D0NA   0.000  0.000    .
D0NA   0.000  0.000    .
D0NA   0.000  0.000    .
Charlie Smith
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Southwest
RoadNumrRoadway Name
RoadNumr
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731006 691 271 47  3 29 27 2684 39.62 27.21 10.67 1.84 0.11 1.13 1.06
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Charlie Smith
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| 254 | 1396 | 804 | 288 | 124 | 85 | 67 | 53 | 9366 | 54.97 | 31.63 | 11.34 | 4.88 | 3.33 | 2.63 | 2.07 | |
| 256 | 1404 | 810 | 293 | 128 | 88 | 69 | 54 | 9440 | 55.28 | 31.91 | 11.53 | 5.05 | 3.46 | 2.71 | 2.13 | |
| 254 | 1400 | 812 | 296 | 131 | 90 | 69 | 54 | 9377 | 55.13 | 31.98 | 11.64 | 5.16 | 3.55 | 2.73 | 2.14 | |
| 256 | 1400 | 815 | 298 | 133 | 91 | 70 | 54 | 9421 | 55.12 | 32.08 | 11.74 | 5.26 | 3.60 | 2.75 | 2.15 | |
| S4 | 16.3 | 16 | 18I51121 | 61 | 61 | 65 | | | | | | | | | | |
| 80 | 390 | 224 | 95 | 52 | 34 | 25 | 20 | 2950 | 15.36 | 8.83 | 3.75 | 2.05 | 1.35 | 1.00 | 0.80 | |
| 81 | 390 | 225 | 96 | 52 | 36 | 27 | 22 | 2998 | 15.37 | 8.87 | 3.79 | 2.06 | 1.40 | 1.08 | 0.85 | |
| 82 | 386 | 224 | 96 | 52 | 37 | 28 | 22 | 3024 | 15.21 | 8.81 | 3.78 | 2.05 | 1.45 | 1.12 | 0.88 | |
| 83 | 384 | 223 | 96 | 53 | 36 | 28 | 22 | 3053 | 15.12 | 8.78 | 3.77 | 2.09 | 1.43 | 1.10 | 0.87 | |
| 145 | 735 | 429 | 178 | 96 | 63 | 48 | 38 | 5365 | 28.93 | 16.87 | 6.99 | 3.77 | 2.49 | 1.87 | 1.52 | |
| 146 | 738 | 432 | 180 | 96 | 63 | 47 | 38 | 5384 | 29.07 | 16.99 | 7.08 | 3.80 | 2.48 | 1.85 | 1.50 | |
| 145 | 731 | 428 | 180 | 97 | 64 | 48 | 39 | 5358 | 28.80 | 16.87 | 7.07 | 3.80 | 2.51 | 1.90 | 1.54 | |
| 146 | 732 | 430 | 180 | 97 | 64 | 48 | 39 | 5384 | 28.83 | 16.92 | 7.08 | 3.82 | 2.51 | 1.91 | 1.53 | |
| 197 | 1014 | 596 | 242 | 129 | 83 | 63 | 51 | 7275 | 39.91 | 23.47 | 9.53 | 5.07 | 3.28 | 2.47 | 2.00 | |
| 198 | 1021 | 602 | 246 | 130 | 85 | 65 | 53 | 7294 | 40.18 | 23.71 | 9.67 | 5.10 | 3.33 | 2.56 | 2.08 | |
| 197 | 1019 | 603 | 246 | 130 | 84 | 63 | 51 | 7257 | 40.12 | 23.74 | 9.70 | 5.13 | 3.29 | 2.48 | 2.02 | |
| 197 | 1014 | 599 | 246 | 129 | 85 | 65 | 53 | 7246 | 39.92 | 23.59 | 9.68 | 5.06 | 3.34 | 2.54 | 2.07 | |
| 260 | 1337 | 798 | 311 | 153 | 100 | 77 | 63 | 9587 | 52.66 | 31.42 | 12.24 | 6.04 | 3.92 | 3.03 | 2.46 | |
| 259 | 1339 | 803 | 316 | 157 | 101 | 77 | 63 | 9550 | 52.73 | 31.60 | 12.43 | 6.20 | 3.97 | 3.03 | 2.48 | |
| 259 | 1340 | 806 | 319 | 159 | 103 | 79 | 64 | 9550 | 52.75 | 31.72 | 12.55 | 6.25 | 4.06 | 3.11 | 2.53 | |
| 260 | 1340 | 807 | 321 | 159 | 104 | 81 | 67 | 9569 | 52.75 | 31.79 | 12.63 | 6.28 | 4.10 | 3.20 | 2.63 | |
| S5 | 16.3 | 16 | 18I51124 | 61 | 61 | 65 | | | | | | | | | | |
| 73 | 540 | 287 | 91 | 43 | 28 | 22 | 19 | 2684 | 21.26 | 11.28 | 3.60 | 1.71 | 1.10 | 0.86 | 0.74 | |
| 74 | 541 | 291 | 94 | 44 | 29 | 23 | 19 | 2721 | 21.31 | 11.46 | 3.68 | 1.73 | 1.13 | 0.89 | 0.74 | |
| 74 | 530 | 287 | 93 | 44 | 29 | 23 | 19 | 2721 | 20.87 | 11.31 | 3.67 | 1.74 | 1.15 | 0.89 | 0.73 | |
| 74 | 524 | 285 | 93 | 44 | 29 | 22 | 18 | 2747 | 20.65 | 11.22 | 3.65 | 1.73 | 1.15 | 0.88 | 0.72 | |
| 132 | 992 | 552 | 176 | 78 | 45 | 34 | 27 | 4860 | 39.04 | 21.74 | 6.93 | 3.08 | 1.76 | 1.33 | 1.06 | |
| 133 | 988 | 555 | 179 | 78 | 44 | 34 | 23 | 4897 | 38.89 | 21.86 | 7.06 | 3.09 | 1.73 | 1.33 | 0.91 | |
| 133 | 981 | 556 | 181 | 80 | 45 | 34 | 23 | 4904 | 38.62 | 21.87 | 7.13 | 3.13 | 1.76 | 1.35 | 0.93 | |
| 133 | 977 | 556 | 182 | 81 | 46 | 35 | 25 | 4897 | 38.47 | 21.87 | 7.17 | 3.18 | 1.79 | 1.37 | 0.98 | |
| 182 | 1371 | 787 | 250 | 97 | 53 | 41 | 29 | 6722 | 53.99 | 30.99 | 9.83 | 3.83 | 2.11 | 1.60 | 1.16 | |
| 183 | 1372 | 794 | 252 | 102 | 55 | 43 | 32 | 6729 | 54.00 | 31.27 | 9.91 | 4.01 | 2.18 | 1.68 | 1.25 | |
| 182 | 1363 | 797 | 257 | 102 | 55 | 42 | 30 | 6722 | 53.66 | 31.39 | 10.11 | 4.01 | 2.18 | 1.65 | 1.20 | |
| 182 | 1350 | 797 | 259 | 105 | 57 | 42 | 34 | 6722 | 53.14 | 31.36 | 10.20 | 4.13 | 2.23 | 1.66 | 1.34 | |
| 232 | 1789 | 1073 | 340 | 126 | 63 | 46 | 37 | 8555 | 70.44 | 42.24 | 13.39 | 4.96 | 2.46 | 1.79 | 1.46 | |
| 231 | 1785 | 1081 | 347 | 131 | 65 | 48 | 40 | 8536 | 70.26 | 42.57 | 13.65 | 5.14 | 2.57 | 1.88 | 1.57 | |
| 233 | 1778 | 1086 | 349 | 131 | 66 | 48 | 39 | 8573 | 69.98 | 42.78 | 13.74 | 5.15 | 2.59 | 1.89 | 1.55 | |
| 233 | 1768 | 1088 | 351 | 131 | 66 | 48 | 39 | 8584 | 69.61 | 42.83 | 13.81 | 5.15 | 2.61 | 1.90 | 1.54 | |
| S6 | 16.3 | 16 | 18I51127 | 61 | 61 | 65 | | | | | | | | | | |
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| 84 | 429 | 276 | 104 | 47 | 30 | 23 | 21 | 3097 | 16.87 | 10.87 | 4.09 | 1.85 | 1.17 | 0.92 | 0.81 | |
| 84 | 427 | 275 | 104 | 48 | 30 | 24 | 22 | 3079 | 16.83 | 10.84 | 4.11 | 1.88 | 1.17 | 0.94 | 0.86 | |
| 84 | 427 | 276 | 104 | 48 | 30 | 24 | 22 | 3097 | 16.80 | 10.85 | 4.10 | 1.87 | 1.17 | 0.94 | 0.85 | |
| 153 | 789 | 514 | 196 | 82 | 47 | 36 | 32 | 5653 | 31.07 | 20.23 | 7.70 | 3.23 | 1.83 | 1.41 | 1.26 | |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 154 | 792 | 516 | 198 | 83 | 47 | 36 | 33 | 5671 | 31.16 | 20.31 | 7.78 | 3.28 | 1.85 | 1.43 | 1.28 |
| 154 | 789 | 516 | 198 | 84 | 48 | 37 | 32 | 5690 | 31.08 | 20.30 | 7.80 | 3.30 | 1.87 | 1.45 | 1.27 |
| 155 | 787 | 516 | 199 | 84 | 47 | 36 | 32 | 5727 | 31.00 | 20.32 | 7.83 | 3.30 | 1.85 | 1.43 | 1.27 |
| 206 | 1101 | 719 | 277 | 113 | 61 | 47 | 42 | 7614 | 43.36 | 28.30 | 10.89 | 4.46 | 2.41 | 1.86 | 1.66 |
| 206 | 1099 | 719 | 278 | 114 | 62 | 48 | 43 | 7589 | 43.27 | 28.32 | 10.95 | 4.50 | 2.43 | 1.88 | 1.68 |
| 205 | 1094 | 718 | 279 | 115 | 62 | 48 | 43 | 7570 | 43.06 | 28.26 | 10.98 | 4.53 | 2.45 | 1.90 | 1.69 |
| 204 | 1085 | 713 | 277 | 114 | 62 | 48 | 43 | 7515 | 42.74 | 28.07 | 10.91 | 4.50 | 2.45 | 1.90 | 1.70 |
| 253 | 1435 | 948 | 368 | 145 | 74 | 56 | 50 | 9348 | 56.51 | 37.32 | 14.50 | 5.72 | 2.91 | 2.20 | 1.98 |
| 253 | 1434 | 950 | 371 | 148 | 76 | 57 | 51 | 9322 | 56.44 | 37.40 | 14.59 | 5.81 | 2.98 | 2.25 | 2.01 |
| 254 | 1428 | 949 | 372 | 148 | 76 | 58 | 52 | 9377 | 56.21 | 37.35 | 14.63 | 5.83 | 3.00 | 2.29 | 2.03 |
| 253 | 1419 | 946 | 372 | 149 | 77 | 59 | 52 | 9348 | 55.87 | 37.23 | 14.65 | 5.87 | 3.04 | 2.31 | 2.06 |
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| 73 | 415 | 247 | 96 | 49 | 33 | 25 | 20 | 2684 | 16.32 | 9.72 | 3.76 | 1.92 | 1.30 | 0.98 | 0.78 |
| 75 | 414 | 248 | 96 | 49 | 33 | 26 | 22 | 2777 | 16.29 | 9.77 | 3.79 | 1.92 | 1.32 | 1.04 | 0.85 |
| 76 | 417 | 249 | 96 | 49 | 33 | 26 | 22 | 2795 | 16.41 | 9.81 | 3.78 | 1.91 | 1.31 | 1.04 | 0.86 |
| 76 | 412 | 246 | 96 | 48 | 33 | 26 | 22 | 2802 | 16.21 | 9.67 | 3.77 | 1.91 | 1.31 | 1.04 | 0.85 |
| 130 | 759 | 464 | 180 | 87 | 57 | 44 | 35 | 4775 | 29.88 | 18.26 | 7.08 | 3.44 | 2.23 | 1.72 | 1.39 |
| 129 | 760 | 466 | 182 | 88 | 57 | 44 | 35 | 4768 | 29.91 | 18.34 | 7.15 | 3.47 | 2.24 | 1.73 | 1.40 |
| 130 | 756 | 465 | 181 | 88 | 57 | 43 | 35 | 4775 | 29.77 | 18.29 | 7.13 | 3.44 | 2.22 | 1.71 | 1.39 |
| 129 | 754 | 463 | 181 | 88 | 57 | 44 | 35 | 4757 | 29.67 | 18.24 | 7.11 | 3.45 | 2.24 | 1.72 | 1.40 |
| 170 | 1053 | 647 | 248 | 117 | 74 | 57 | 45 | 6261 | 41.46 | 25.47 | 9.76 | 4.59 | 2.93 | 2.24 | 1.78 |
| 170 | 1055 | 651 | 251 | 118 | 75 | 58 | 46 | 6280 | 41.52 | 25.65 | 9.88 | 4.65 | 2.95 | 2.27 | 1.80 |
| 171 | 1055 | 654 | 253 | 119 | 75 | 58 | 46 | 6298 | 41.54 | 25.74 | 9.94 | 4.68 | 2.96 | 2.27 | 1.82 |
| 169 | 1041 | 646 | 250 | 118 | 75 | 57 | 46 | 6224 | 41.00 | 25.42 | 9.85 | 4.66 | 2.96 | 2.26 | 1.79 |
| 218 | 1394 | 864 | 326 | 145 | 89 | 68 | 54 | 8038 | 54.89 | 34.02 | 12.82 | 5.72 | 3.50 | 2.66 | 2.11 |
| 217 | 1393 | 867 | 328 | 148 | 91 | 69 | 55 | 7994 | 54.84 | 34.12 | 12.93 | 5.83 | 3.59 | 2.73 | 2.16 |
| 215 | 1378 | 860 | 328 | 149 | 92 | 70 | 55 | 7909 | 54.26 | 33.87 | 12.90 | 5.85 | 3.62 | 2.74 | 2.16 |
| 216 | 1378 | 862 | 330 | 151 | 93 | 71 | 56 | 7957 | 54.26 | 33.95 | 12.99 | 5.93 | 3.67 | 2.79 | 2.19 |
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| 86 | 374 | 243 | 92 | 49 | 33 | 25 | 21 | 3171 | 14.72 | 9.57 | 3.62 | 1.94 | 1.31 | 0.98 | 0.83 |
| 87 | 375 | 243 | 93 | 50 | 33 | 25 | 21 | 3201 | 14.76 | 9.58 | 3.64 | 1.95 | 1.31 | 0.99 | 0.83 |
| 84 | 367 | 238 | 92 | 50 | 34 | 25 | 21 | 3090 | 14.46 | 9.36 | 3.61 | 1.97 | 1.33 | 0.98 | 0.83 |
| 141 | 697 | 452 | 174 | 90 | 58 | 43 | 36 | 5181 | 27.44 | 17.79 | 6.87 | 3.55 | 2.28 | 1.70 | 1.43 |
| 140 | 695 | 451 | 175 | 91 | 58 | 43 | 35 | 5162 | 27.37 | 17.76 | 6.89 | 3.56 | 2.27 | 1.68 | 1.40 |
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| 184 | 978 | 634 | 246 | 125 | 79 | 58 | 48 | 6785 | 38.51 | 24.96 | 9.69 | 4.92 | 3.11 | 2.30 | 1.91 |
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| 183 | 972 | 632 | 246 | 125 | 79 | 58 | 49 | 6741 | 38.28 | 24.87 | 9.69 | 4.93 | 3.13 | 2.30 | 1.91 |
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| 231 | 1295 | 845 | 326 | 161 | 99 | 73 | 60 | 8529 | 50.98 | 33.25 | 12.83 | 6.33 | 3.91 | 2.85 | 2.35 |
| 230 | 1286 | 841 | 326 | 161 | 100 | 73 | 60 | 8492 | 50.61 | 33.10 | 12.84 | 6.35 | 3.94 | 2.87 | 2.37 |
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| 80 | 392 | 291 | 102 | 48 | 32 | 26 | 22 | 2943 | 15.43 | 11.45 | 4.01 | 1.89 | 1.25 | 1.02 | 0.85 |
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| 142 | 766 | 555 | 195 | 81 | 53 | 41 | 35 | 5236 | 30.16 | 21.83 | 7.69 | 3.20 | 2.07 | 1.61 | 1.36 |
| 143 | 767 | 555 | 196 | 81 | 53 | 41 | 35 | 5265 | 30.21 | 21.86 | 7.73 | 3.20 | 2.09 | 1.63 | 1.37 |
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**FWD File from January 24, 2001**

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Charlie Smith
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Southwest
RoadNumrRoadway Name
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000+0.0 000+0.0 St
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| | | | | | | | | | | | | | | | |
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| 148 | 604 | 421 | 166 | 88 | 60 | 44 | 37 | 5439 | 23.78 | 16.58 | 6.54 | 3.45 | 2.35 | 1.74 | 1.45 |
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| 192 | 844 | 590 | 231 | 119 | 80 | 59 | 48 | 7098 | 33.22 | 23.23 | 9.10 | 4.68 | 3.15 | 2.32 | 1.90 |
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| 144 | 626 | 424 | 159 | 84 | 57 | 42 | 35 | 5321 | 24.65 | 16.69 | 6.26 | 3.33 | 2.22 | 1.67 | 1.37 |
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| 203 | 986 | 670 | 225 | 104 | 57 | 48 | 40 | 7467 | 38.82 | 26.38 | 8.86 | 4.09 | 2.26 | 1.88 | 1.59 |
| 202 | 979 | 745 | 225 | 104 | 59 | 48 | 40 | 7441 | 38.56 | 29.33 | 8.87 | 4.09 | 2.31 | 1.89 | 1.59 |
| 2601335 | | 854 | 292 | 129 | 70 | 57 | 48 | 9587 | 52.56 | 33.61 | 11.51 | 5.08 | 2.76 | 2.26 | 1.91 |
| 2601340 | | 842 | 294 | 132 | 69 | 57 | 49 | 9569 | 52.75 | 33.16 | 11.56 | 5.18 | 2.72 | 2.24 | 1.92 |
| 2601339 | | 852 | 296 | 133 | 71 | 58 | 49 | 9587 | 52.72 | 33.55 | 11.64 | 5.22 | 2.78 | 2.27 | 1.93 |
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 138 670 459 185  88  56  41  34  5070 26.37 18.08  7.28  3.44  2.19  1.61  1.34
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 137 669 460 186  89  57  42  35  5063 26.36 18.12  7.34  3.51  2.24  1.66  1.39
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**FWD File from January 31, 2001**

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S11          17.2   17 19I51329  63  63  67
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D23172    0.998   1.047        .
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D73177    0.995   0.990        .
D0NA      0.000   0.000        .
D0NA      0.000   0.000        .
D0NA      0.000   0.000        .
Charlie Smith
0010151000002 1 1              .
   0 0.0   0 0.0              .
Southwest
RoadNumrRoadway Name
RoadNumr
000+0.0 000+0.0 St
 457   0   0   0   0   0   0   0 17.99  0.00  0.00  0.00  0.00  0.00  0.00  0.00
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 1321066 515 140  74  53  41  33  4860 41.97 20.29  5.51  2.92  2.09  1.61  1.31
 1321058 517 141  74  53  41  33  4867 41.67 20.35  5.54  2.92  2.09  1.61  1.32
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 1751472 752 188  95  67  52  42  6445 57.94 29.60  7.41  3.76  2.65  2.05  1.67
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 1741453 763 194  97  68  52  43  6427 57.21 30.05  7.64  3.81  2.69  2.06  1.67
 1741436 763 195  97  68  53  43  6409 56.56 30.02  7.70  3.83  2.69  2.08  1.68
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 186 947 556 198 100  67  51  43  6877 37.27 21.89  7.78  3.96  2.65  2.00  1.70
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 2401113 521 139  19 110   0   0  8861 43.83 20.51  5.46  0.74  4.32  0.00  0.01
 2411118 563 145   0   3   0   0  8887 44.01 22.17  5.70  0.00  0.12  0.00  0.00
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85 348 203 85 48 36 29 25 3145 13.71 8.00 3.34 1.87 1.43 1.13 0.96
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1991110 645 226 106 72 56 46 7319 43.72 25.38 8.88 4.17 2.81 2.19 1.79
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85 257 162 63 36 26 21 17 3134 10.12 6.38 2.48 1.42 1.02 0.81 0.69
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141 494 312 124 68 46 35 30 5199 19.44 12.26 4.89 2.66 1.80 1.36 1.16
141 496 313 125 68 46 35 30 5210 19.52 12.33 4.93 2.68 1.81 1.36 1.17
141 494 311 124 68 46 35 30 5192 19.44 12.25 4.90 2.67 1.81 1.37 1.17
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182 713 449 180 96 64 48 41 6722 28.05 17.69 7.07 3.76 2.52 1.90 1.61
183 713 451 181 97 65 48 41 6741 28.09 17.77 7.13 3.80 2.56 1.91 1.61
182 707 446 179 95 64 48 40 6711 27.81 17.57 7.04 3.74 2.52 1.88 1.59
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231 964 617 247 127 83 62 52 8499 37.94 24.28 9.71 5.00 3.28 2.44 2.06
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76 345 194 76 43 31 23 19 2802 13.60 7.66 2.99 1.70 1.21 0.90 0.75

| | | | | | | | | | | | | | | | |
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| 76 | 348 | 196 | 76 | 43 | 31 | 23 | 19 | 2813 | 13.69 | 7.70 | 2.98 | 1.68 | 1.20 | 0.90 | 0.75 |
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| 134 | 675 | 387 | 149 | 80 | 55 | 42 | 35 | 4960 | 26.57 | 15.22 | 5.88 | 3.15 | 2.18 | 1.65 | 1.37 |
| 135 | 675 | 387 | 150 | 80 | 56 | 42 | 35 | 4989 | 26.56 | 15.22 | 5.89 | 3.13 | 2.19 | 1.65 | 1.36 |
| 135 | 672 | 387 | 149 | 80 | 56 | 42 | 35 | 4989 | 26.48 | 15.22 | 5.87 | 3.13 | 2.20 | 1.67 | 1.38 |
| 179 | 949 | 544 | 207 | 109 | 75 | 56 | 44 | 6593 | 37.35 | 21.42 | 8.15 | 4.28 | 2.94 | 2.19 | 1.75 |
| 179 | 947 | 545 | 208 | 109 | 76 | 56 | 45 | 6600 | 37.30 | 21.46 | 8.19 | 4.30 | 2.98 | 2.22 | 1.78 |
| 179 | 946 | 546 | 209 | 109 | 75 | 56 | 45 | 6600 | 37.23 | 21.50 | 8.22 | 4.30 | 2.96 | 2.19 | 1.76 |
| 179 | 938 | 542 | 207 | 110 | 75 | 57 | 46 | 6593 | 36.93 | 21.35 | 8.17 | 4.31 | 2.96 | 2.22 | 1.81 |
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| 224 | 1245 | 725 | 272 | 139 | 96 | 71 | 57 | 8260 | 49.00 | 28.54 | 10.71 | 5.49 | 3.77 | 2.79 | 2.22 |
| 224 | 1240 | 725 | 273 | 141 | 96 | 72 | 57 | 8260 | 48.81 | 28.52 | 10.75 | 5.54 | 3.79 | 2.82 | 2.24 |
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| 85 | 338 | 220 | 82 | 46 | 32 | 25 | 20 | 3127 | 13.33 | 8.66 | 3.23 | 1.82 | 1.28 | 0.97 | 0.80 |
| 85 | 339 | 220 | 83 | 46 | 33 | 25 | 21 | 3134 | 13.33 | 8.65 | 3.25 | 1.81 | 1.28 | 0.98 | 0.81 |
| 86 | 338 | 219 | 82 | 46 | 33 | 25 | 21 | 3182 | 13.30 | 8.63 | 3.22 | 1.82 | 1.29 | 0.98 | 0.81 |
| 142 | 634 | 407 | 155 | 84 | 58 | 43 | 36 | 5254 | 24.96 | 16.02 | 6.09 | 3.33 | 2.28 | 1.71 | 1.40 |
| 142 | 635 | 408 | 156 | 85 | 57 | 43 | 35 | 5254 | 24.99 | 16.08 | 6.13 | 3.33 | 2.26 | 1.70 | 1.38 |
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| 181 | 880 | 567 | 219 | 118 | 81 | 60 | 48 | 6674 | 34.65 | 22.34 | 8.62 | 4.66 | 3.17 | 2.35 | 1.89 |
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| 228 | 1168 | 757 | 290 | 154 | 103 | 76 | 61 | 8407 | 46.00 | 29.78 | 11.43 | 6.05 | 4.07 | 2.98 | 2.38 |
| 228 | 1164 | 756 | 291 | 154 | 104 | 76 | 61 | 8418 | 45.83 | 29.77 | 11.47 | 6.08 | 4.09 | 3.01 | 2.40 |
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| 79 | 307 | 185 | 72 | 42 | 32 | 25 | 19 | 2913 | 12.08 | 7.28 | 2.83 | 1.65 | 1.24 | 0.99 | 0.76 |
| 80 | 312 | 188 | 74 | 43 | 34 | 27 | 23 | 2943 | 12.28 | 7.39 | 2.93 | 1.68 | 1.35 | 1.07 | 0.89 |
| 81 | 305 | 184 | 73 | 43 | 33 | 26 | 22 | 2987 | 12.00 | 7.24 | 2.88 | 1.68 | 1.32 | 1.04 | 0.85 |
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| 254 | 1178 | 749 | 274 | 124 | 89 | 66 | 50 | 9359 | 46.37 | 29.48 | 10.79 | 4.88 | 3.49 | 2.59 | 1.96 |
| 254 | 1173 | 748 | 275 | 125 | 86 | 65 | 50 | 9377 | 46.19 | 29.46 | 10.82 | 4.91 | 3.39 | 2.54 | 1.98 |
| S10 | 17.2 | 17 | 19I51326 | 63 | 63 | 67 | | | | | | | | | |
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| 78 | 380 | 235 | 89 | 44 | 28 | 20 | 17 | 2895 | 14.97 | 9.26 | 3.52 | 1.73 | 1.09 | 0.81 | 0.68 |
| 78 | 376 | 233 | 89 | 44 | 28 | 21 | 17 | 2876 | 14.80 | 9.16 | 3.52 | 1.74 | 1.11 | 0.81 | 0.67 |
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 2141305 851 330 149  85  58  48  7891 51.36 33.52 13.00  5.86  3.36  2.29  1.89
 2141306 856 333 151  87  59  48  7891 51.43 33.69 13.13  5.93  3.42  2.32  1.90
 2141307 859 336 152  88  60  49  7884 51.47 33.81 13.22  5.98  3.46  2.35  1.92
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  86 353 234  96  50  33  26  22  3153 13.88  9.21  3.76  1.96  1.31  1.02  0.86
  86 350 231  95  49  33  25  22  3153 13.77  9.10  3.74  1.95  1.31  1.00  0.85
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 2381239 833 353 176 107  76  59  8769 48.79 32.79 13.91  6.93  4.21  2.97  2.32
EOF
```

# Appendix G:
# Dynamic Cone Penetrometer Data

# DCP TEST DATA

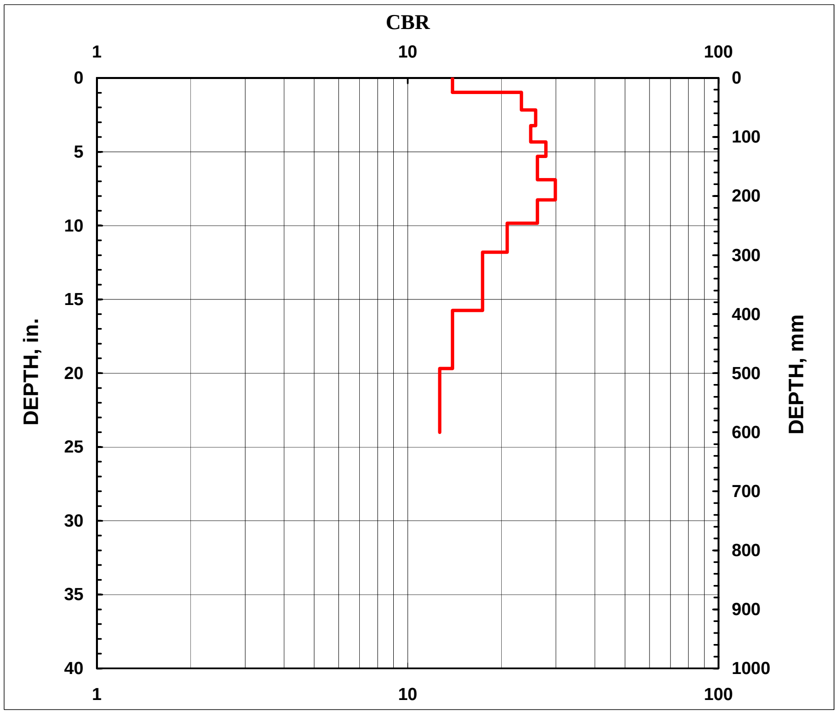
**File Name:** DCPVTP1

**Project:** *VTP Autopsy*
**Date:** 9-May-01
**Location:** 5'6" x 20'
**Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 25 | 1 |
| 2 | 55 | 1 |
| 2 | 82 | 1 |
| 2 | 110 | 1 |
| 2 | 135 | 1 |
| 3 | 175 | 1 |
| 3 | 210 | 1 |
| 3 | 250 | 1 |
| 3 | 300 | 1 |
| 3 | 360 | 1 |
| 2 | 400 | 1 |
| 2 | 450 | 1 |
| 2 | 500 | 1 |
| 2 | 555 | 1 |
| 2 | 610 | 1 |



# DCP TEST DATA

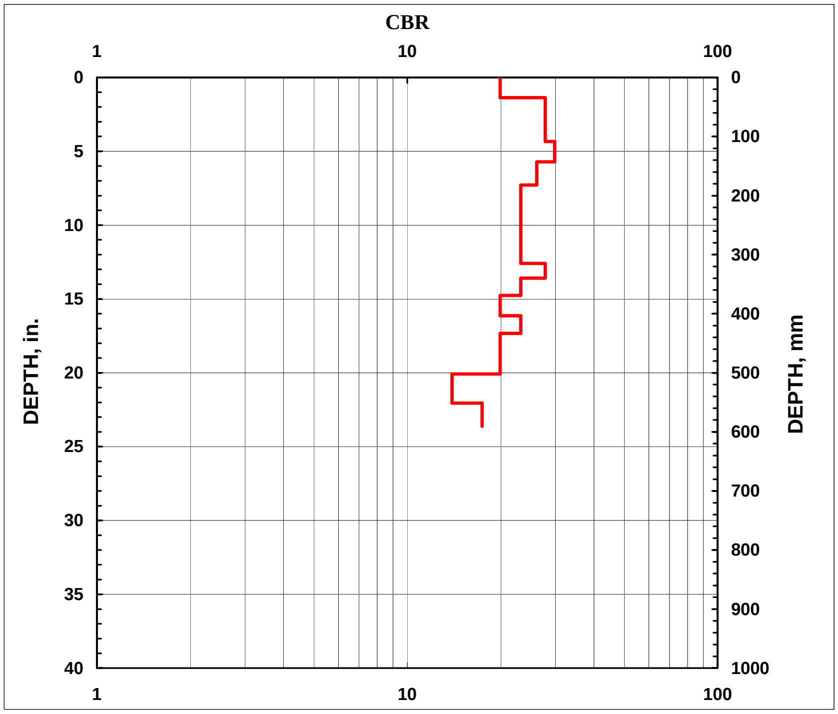
**File Name:** DCPVTP2

**Project:** *VTP Autopsy* **Date:** 9-May-01
**Location:** 10' x 20' **Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 35 | 1 |
| 2 | 60 | 1 |
| 2 | 85 | 1 |
| 2 | 110 | 1 |
| 3 | 145 | 1 |
| 3 | 185 | 1 |
| 3 | 230 | 1 |
| 2 | 260 | 1 |
| 2 | 290 | 1 |
| 2 | 320 | 1 |
| 2 | 345 | 1 |
| 2 | 375 | 1 |
| 2 | 410 | 1 |
| 2 | 440 | 1 |
| 2 | 475 | 1 |
| 2 | 510 | 1 |
| 2 | 560 | 1 |
| 2 | 600 | 1 |



# DCP TEST DATA

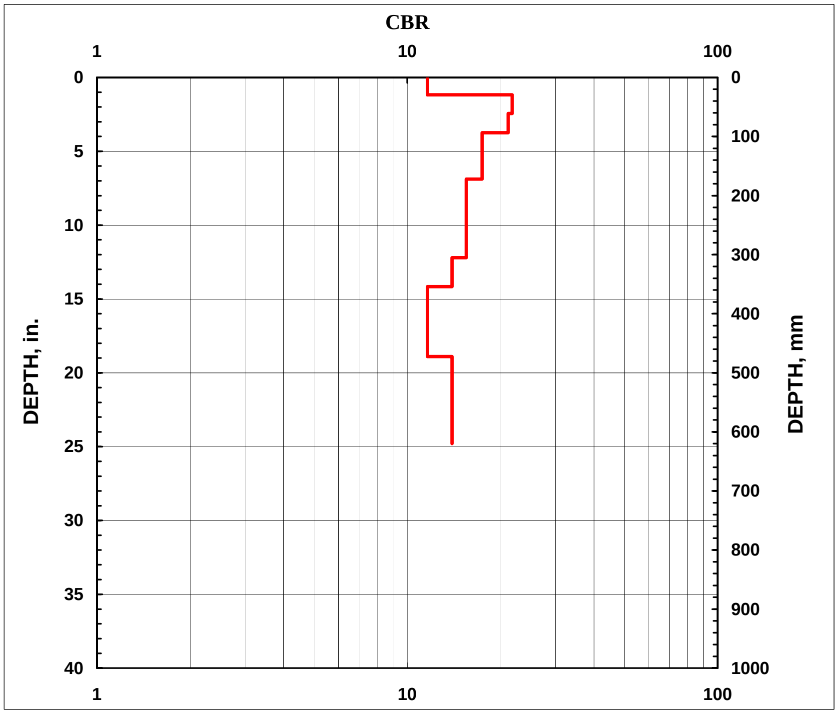
**File Name:** DCPVTP3

**Project:** *VTP Autopsy* **Date:** 9-May-01
**Location:** 14' x 20' **Soil Type(s):** Hanover Silt

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 30 | 1 |
| 2 | 62 | 1 |
| 2 | 95 | 1 |
| 2 | 135 | 1 |
| 2 | 175 | 1 |
| 2 | 220 | 1 |
| 2 | 265 | 1 |
| 2 | 310 | 1 |
| 2 | 360 | 1 |
| 2 | 420 | 1 |
| 2 | 480 | 1 |
| 2 | 530 | 1 |
| 2 | 580 | 1 |
| 2 | 630 | 1 |



# DCP TEST DATA

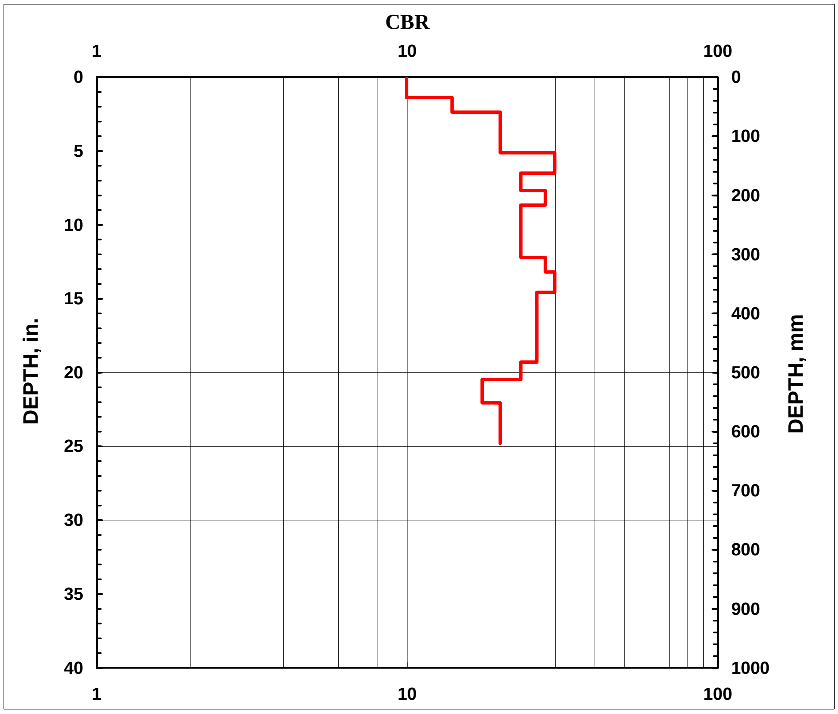
**File Name:** DCPVTP4

**Project:** *VTP Autopsy*
**Location:** 5'6" x 45'
**Date:** 9-May-01
**Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 35 | 1 |
| 1 | 60 | 1 |
| 2 | 95 | 1 |
| 2 | 130 | 1 |
| 3 | 165 | 1 |
| 2 | 195 | 1 |
| 2 | 220 | 1 |
| 2 | 250 | 1 |
| 2 | 280 | 1 |
| 2 | 310 | 1 |
| 2 | 335 | 1 |
| 3 | 370 | 1 |
| 3 | 410 | 1 |
| 3 | 450 | 1 |
| 3 | 490 | 1 |
| 2 | 520 | 1 |
| 2 | 560 | 1 |
| 2 | 595 | 1 |
| 2 | 630 | 1 |



# DCP TEST DATA

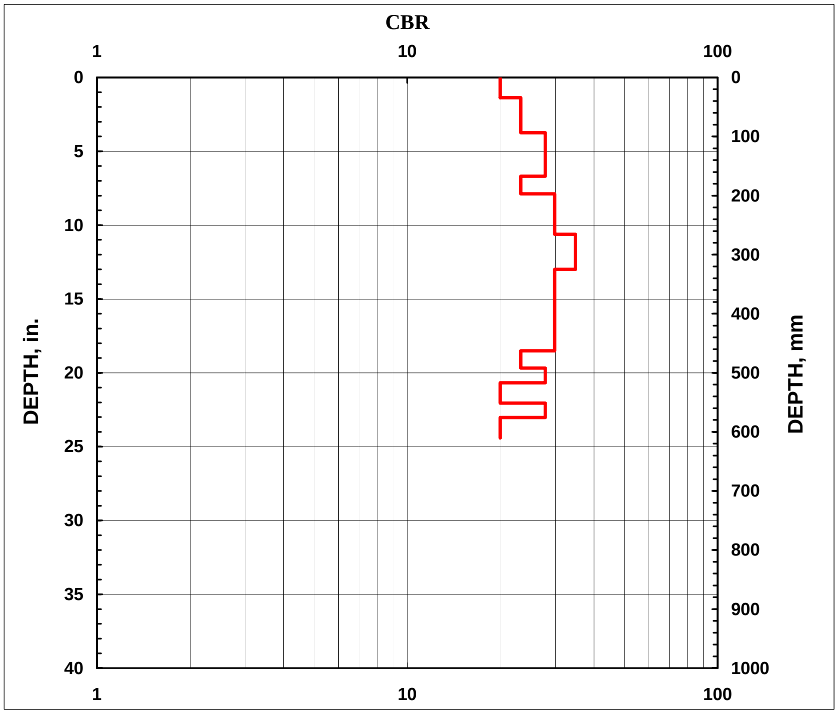
**File Name:** DCPVTP5

**Project:** *VTP Autopsy* **Date:** 9-May-01
**Location:** 10' x 45' **Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 35 | 1 |
| 2 | 65 | 1 |
| 2 | 95 | 1 |
| 2 | 120 | 1 |
| 2 | 145 | 1 |
| 2 | 170 | 1 |
| 2 | 200 | 1 |
| 3 | 235 | 1 |
| 3 | 270 | 1 |
| 3 | 300 | 1 |
| 3 | 330 | 1 |
| 3 | 365 | 1 |
| 3 | 400 | 1 |
| 3 | 435 | 1 |
| 3 | 470 | 1 |
| 2 | 500 | 1 |
| 2 | 525 | 1 |
| 2 | 560 | 1 |
| 2 | 585 | 1 |
| 2 | 620 | |



# DCP TEST DATA

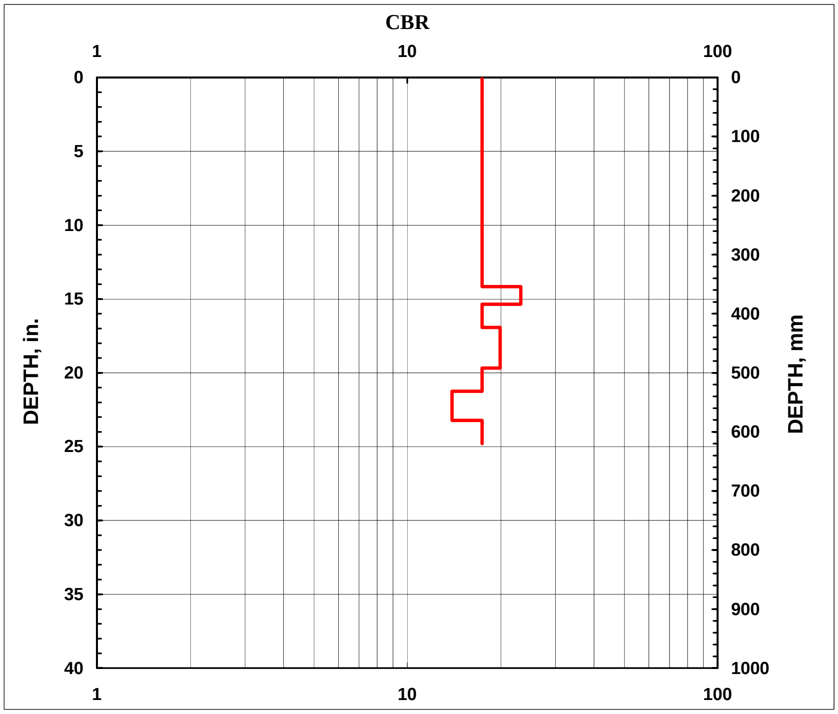
**File Name:** DCPVTP6

**Project:** *VTP Autopsy* **Date:** 9-May-01
**Location:** 14' x 45' **Soil Type(s):** Hanover Silt

Hammer
◯ 10.1 lbs.
◉ 17.6 lbs.
◯ Both hammers used

Soil Type
◯ CH
◯ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 40 | 1 |
| 2 | 80 | 1 |
| 2 | 120 | 1 |
| 2 | 160 | 1 |
| 2 | 200 | 1 |
| 2 | 240 | 1 |
| 2 | 280 | 1 |
| 2 | 320 | 1 |
| 2 | 360 | 1 |
| 2 | 390 | 1 |
| 2 | 430 | 1 |
| 2 | 465 | 1 |
| 2 | 500 | 1 |
| 2 | 540 | 1 |
| 2 | 590 | 1 |
| 2 | 630 | 1 |



**File Name:** DCPVTP7

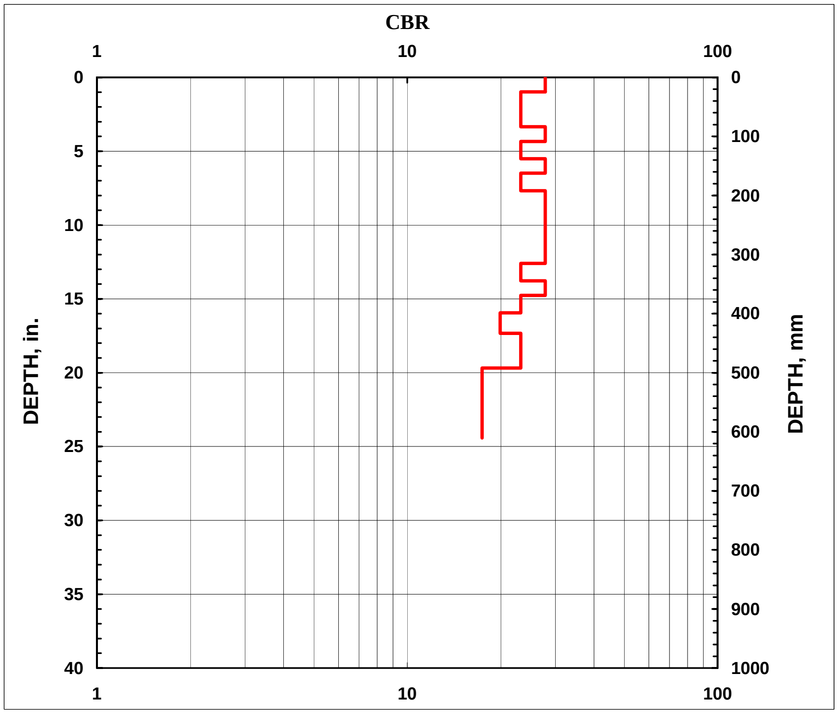
**Project:** *VTP Autopsy* **Date:** 9-May-01

**Location:** 5'6" x 70' **Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 25 | 1 |
| 2 | 55 | 1 |
| 2 | 85 | 1 |
| 2 | 110 | 1 |
| 2 | 140 | 1 |
| 2 | 165 | 1 |
| 2 | 195 | 1 |
| 2 | 220 | 1 |
| 2 | 245 | 1 |
| 2 | 270 | 1 |
| 2 | 295 | 1 |
| 2 | 320 | 1 |
| 2 | 350 | 1 |
| 2 | 375 | 1 |
| 2 | 405 | 1 |
| 2 | 440 | 1 |
| 2 | 470 | 1 |
| 2 | 500 | 1 |
| 2 | 540 | 1 |
| 2 | 580 | 1 |
| 2 | 620 | 1 |



# DCP TEST DATA

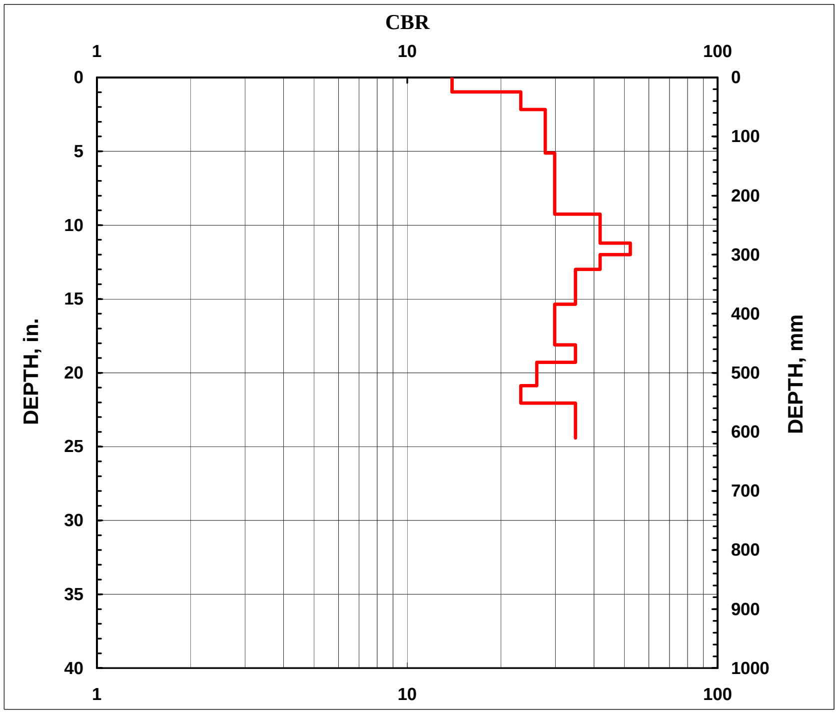
**File Name:** DCPVTP8

**Project:** *VTP Autopsy*
**Date:** 9-May-01
**Location:** 10' x 70'
**Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 25 | 1 |
| 2 | 55 | 1 |
| 2 | 80 | 1 |
| 2 | 105 | 1 |
| 2 | 130 | 1 |
| 3 | 165 | 1 |
| 3 | 200 | 1 |
| 3 | 235 | 1 |
| 3 | 260 | 1 |
| 3 | 285 | 1 |
| 3 | 305 | 1 |
| 3 | 330 | 1 |
| 3 | 360 | 1 |
| 3 | 390 | 1 |
| 3 | 425 | 1 |
| 3 | 460 | 1 |
| 3 | 490 | 1 |
| 3 | 530 | 1 |
| 2 | 560 | 1 |
| 3 | 590 | 1 |
| 3 | 620 | 1 |



# DCP TEST DATA

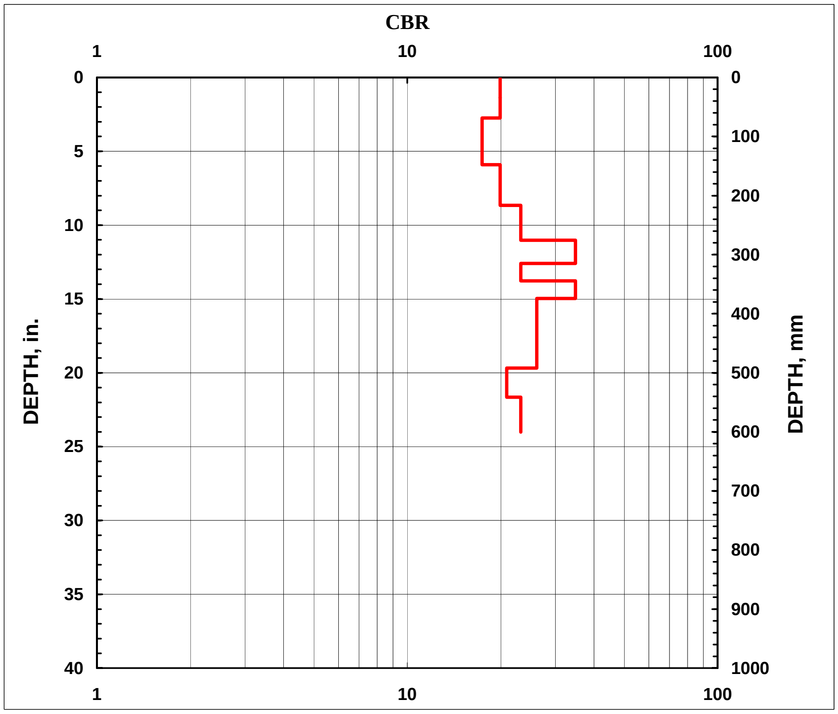
**File Name:** DCPVTP9

**Project:** *VTP Autopsy*
**Location:** 14' x 70'
**Date:** 9-May-01
**Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 35 | 1 |
| 2 | 70 | 1 |
| 2 | 110 | 1 |
| 2 | 150 | 1 |
| 2 | 185 | 1 |
| 2 | 220 | 1 |
| 2 | 250 | 1 |
| 2 | 280 | 1 |
| 4 | 320 | 1 |
| 2 | 350 | 1 |
| 3 | 380 | 1 |
| 3 | 420 | 1 |
| 3 | 460 | 1 |
| 3 | 500 | 1 |
| 3 | 550 | 1 |
| 2 | 580 | 1 |
| 2 | 610 | 1 |



# DCP TEST DATA

**File Name:** DCPVTP10

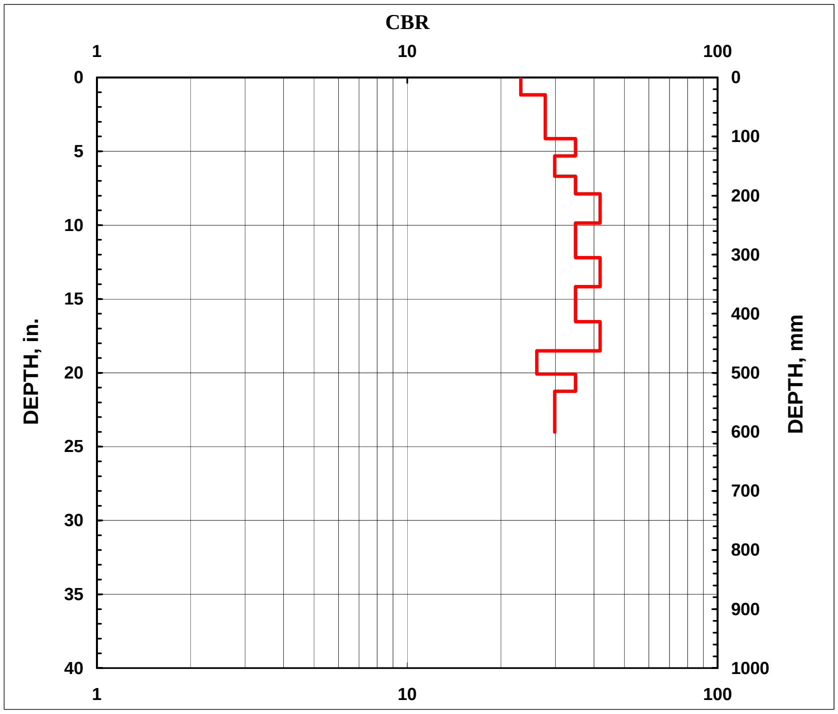
**Project:** *VTP Autopsy** **Date:** 9-May-01

**Location:** 5'6" x 95 **Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 30 | 1 |
| 2 | 55 | 1 |
| 2 | 80 | 1 |
| 2 | 105 | 1 |
| 3 | 135 | 1 |
| 3 | 170 | 1 |
| 3 | 200 | 1 |
| 3 | 225 | 1 |
| 3 | 250 | 1 |
| 3 | 280 | 1 |
| 3 | 310 | 1 |
| 3 | 335 | 1 |
| 3 | 360 | 1 |
| 3 | 390 | 1 |
| 3 | 420 | 1 |
| 3 | 445 | 1 |
| 3 | 470 | 1 |
| 3 | 510 | 1 |
| 3 | 540 | 1 |
| 3 | 575 | 1 |
| 3 | 610 | 1 |



# DCP TEST DATA

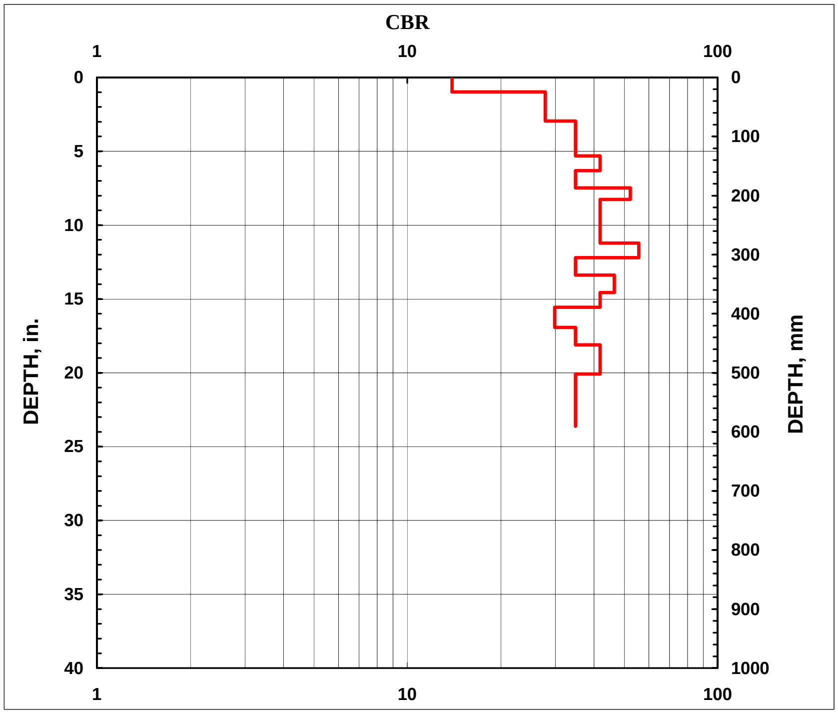
**File Name:** DCPVTP11

**Project:** *VTP Autopsy* **Date:** 9-May-01
**Location:** 10' x 95 **Soil Type(s):** Hanover Silt

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 25 | 1 |
| 2 | 50 | 1 |
| 2 | 75 | 1 |
| 3 | 105 | 1 |
| 3 | 135 | 1 |
| 3 | 160 | 1 |
| 3 | 190 | 1 |
| 3 | 210 | 1 |
| 3 | 235 | 1 |
| 3 | 260 | 1 |
| 3 | 285 | 1 |
| 4 | 310 | 1 |
| 3 | 340 | 1 |
| 4 | 370 | 1 |
| 3 | 395 | 1 |
| 3 | 430 | 1 |
| 3 | 460 | 1 |
| 3 | 485 | 1 |
| 3 | 510 | 1 |
| 3 | 540 | 1 |
| 3 | 570 | 1 |
| 3 | 600 | 1 |



# DCP TEST DATA

**File Name:** DCPVTP12

**Project:** *VTP Autopsy* **Date:** 9-May-01

**Location:** 14' x 95 **Soil Type(s):** Hanover Silt

Hammer
◯ 10.1 lbs.
◉ 17.6 lbs.
◯ Both hammers used

Soil Type
◯ CH
◯ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 2 | 35 | 1 |
| 2 | 60 | 1 |
| 2 | 85 | 1 |
| 2 | 110 | 1 |
| 2 | 135 | 1 |
| 2 | 160 | 1 |
| 3 | 190 | 1 |
| 3 | 225 | 1 |
| 3 | 255 | 1 |
| 3 | 280 | 1 |
| 3 | 310 | 1 |
| 3 | 340 | 1 |
| 3 | 370 | 1 |
| 3 | 400 | 1 |
| 3 | 440 | 1 |
| 3 | 470 | 1 |
| 3 | 500 | 1 |
| 3 | 530 | 1 |
| 3 | 560 | 1 |
| 3 | 600 | 1 |
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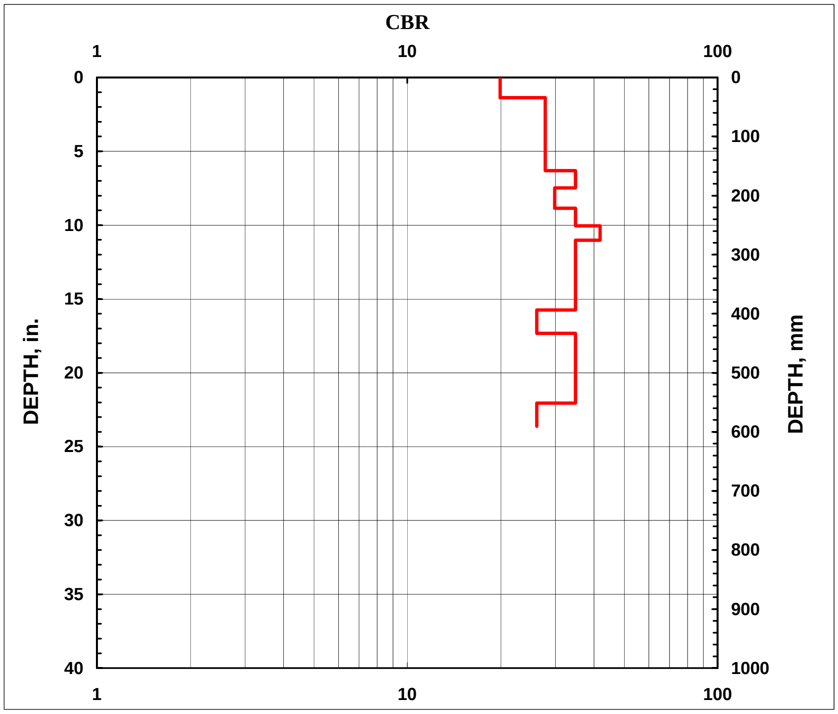


# DCP TEST DATA

**File Name:** DCP

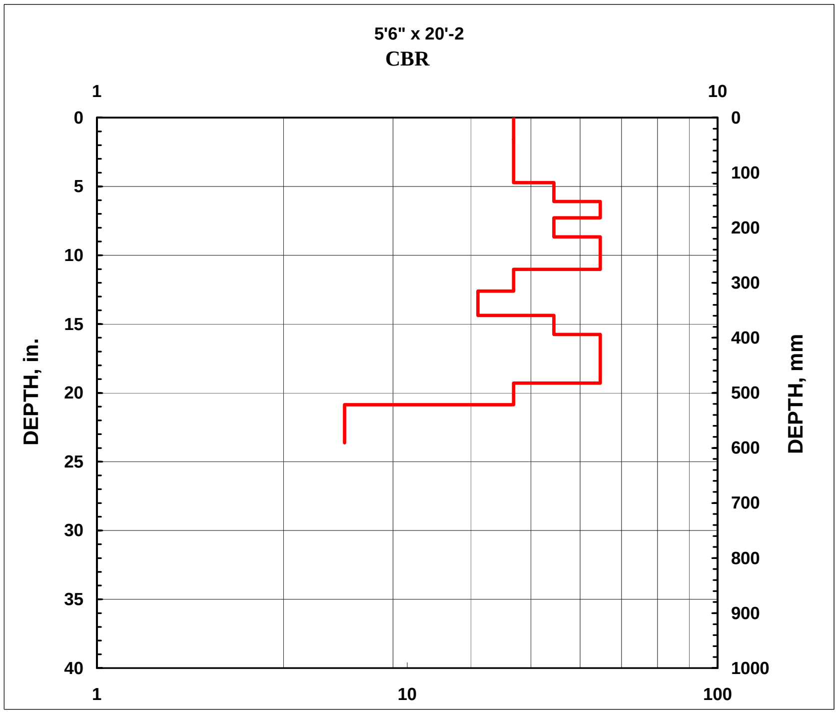
**Project:** *VTP-FERF*
**Location:** 5'6" x 20'-2

**Date:** 30-May-01
**Soil Type(s):** A4-Hanover Silt

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 40 | 1 |
| 1 | 80 | 1 |
| 1 | 120 | 1 |
| 1 | 155 | 1 |
| 1 | 185 | 1 |
| 1 | 220 | 1 |
| 1 | 250 | 1 |
| 1 | 280 | 1 |
| 1 | 320 | 1 |
| 1 | 365 | 1 |
| 1 | 400 | 1 |
| 1 | 430 | 1 |
| 1 | 460 | 1 |
| 1 | 490 | 1 |
| 1 | 530 | 1 |
| 1 | 600 | 1 |



# DCP TEST DATA

**File Name:** DCP

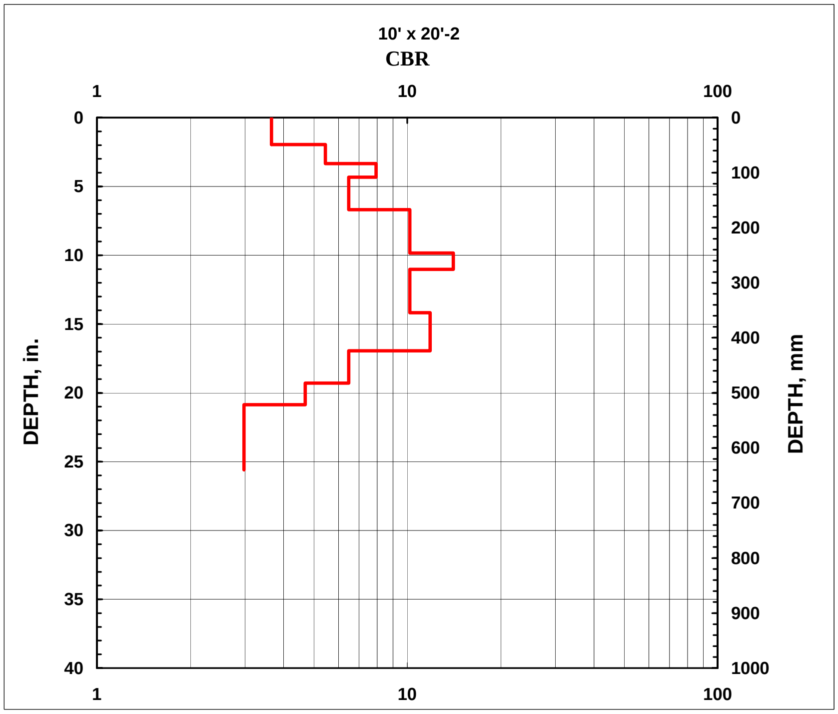
**Project:** *VTP-FERF*
**Location:** 10' x 20'-2

**Date:** 30-May-01
**Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 50 | 1 |
| 1 | 85 | 1 |
| 1 | 110 | 1 |
| 1 | 140 | 1 |
| 1 | 170 | 1 |
| 2 | 210 | 1 |
| 2 | 250 | 1 |
| 2 | 280 | 1 |
| 2 | 320 | 1 |
| 2 | 360 | 1 |
| 2 | 395 | 1 |
| 2 | 430 | 1 |
| 2 | 490 | 1 |
| 1 | 530 | 1 |
| 1 | 590 | 1 |
| 1 | 650 | 1 |



# DCP TEST DATA

**File Name:** DCP

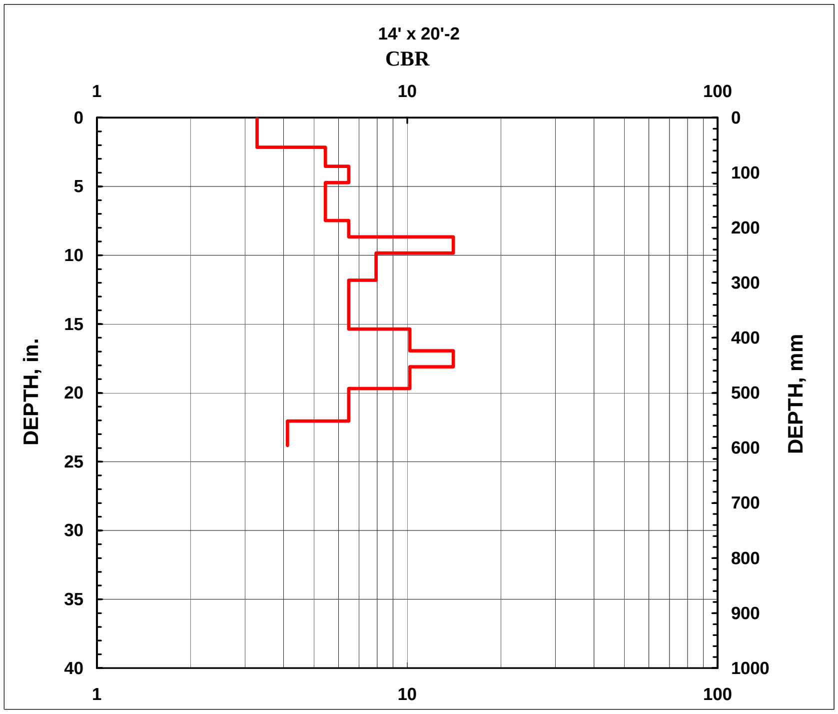
**Project:** *VTP-FERF*
**Location:** 14' x 20'-2

**Date:** 30-May-01
**Soil Type(s):** A4-Hanover Silt

Hammer
◯ 10.1 lbs.
◉ 17.6 lbs.
◯ Both hammers used

Soil Type
◯ CH
◯ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 55 | 1 |
| 1 | 90 | 1 |
| 1 | 120 | 1 |
| 1 | 155 | 1 |
| 1 | 190 | 1 |
| 1 | 220 | 1 |
| 2 | 250 | 1 |
| 2 | 300 | 1 |
| 2 | 360 | 1 |
| 1 | 390 | 1 |
| 2 | 430 | 1 |
| 2 | 460 | 1 |
| 2 | 500 | 1 |
| 2 | 560 | 1 |
| 1 | 605 | 1 |



# DCP TEST DATA

**File Name:** DCP

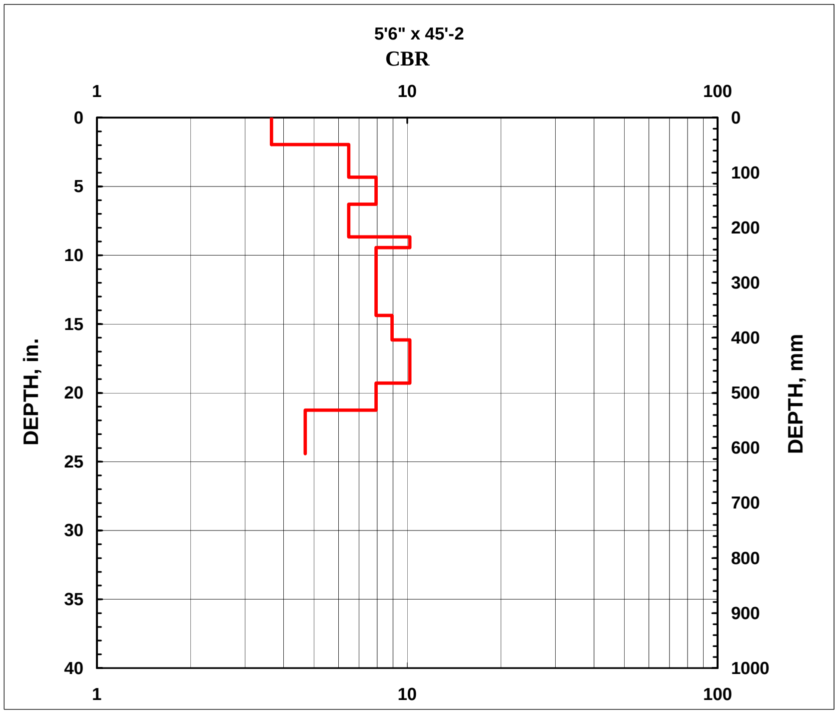
**Project:** *VTP-FERF*
**Location:** 5'6" x 45'-2

**Date:** 30-May-01
**Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 50 | 1 |
| 1 | 80 | 1 |
| 1 | 110 | 1 |
| 1 | 135 | 1 |
| 1 | 160 | 1 |
| 1 | 190 | 1 |
| 1 | 220 | 1 |
| 1 | 240 | 1 |
| 1 | 265 | 1 |
| 1 | 290 | 1 |
| 1 | 315 | 1 |
| 2 | 365 | 1 |
| 2 | 410 | 1 |
| 2 | 450 | 1 |
| 2 | 490 | 1 |
| 2 | 540 | 1 |
| 1 | 580 | 1 |
| 1 | 620 | 1 |



# DCP TEST DATA

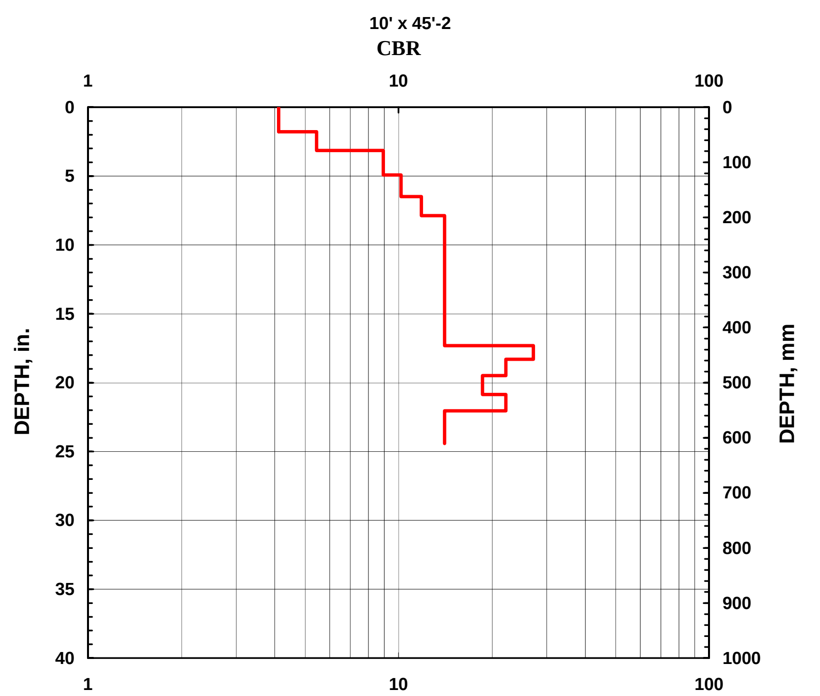
**File Name:** DCP

**Project:** *VTP-FERF* **Date:** 30-May-01
**Location:** 10' x 45'-2 **Soil Type(s):** A4-Hanover Silt

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 45 | 1 |
| 1 | 80 | 1 |
| 2 | 125 | 1 |
| 2 | 165 | 1 |
| 2 | 200 | 1 |
| 2 | 230 | 1 |
| 2 | 260 | 1 |
| 2 | 290 | 1 |
| 2 | 320 | 1 |
| 2 | 350 | 1 |
| 2 | 380 | 1 |
| 2 | 410 | 1 |
| 2 | 440 | 1 |
| 3 | 465 | 1 |
| 3 | 495 | 1 |
| 3 | 530 | 1 |
| 3 | 560 | 1 |
| 2 | 590 | 1 |
| 2 | 620 | 1 |



# DCP TEST DATA

**File Name:** DCP

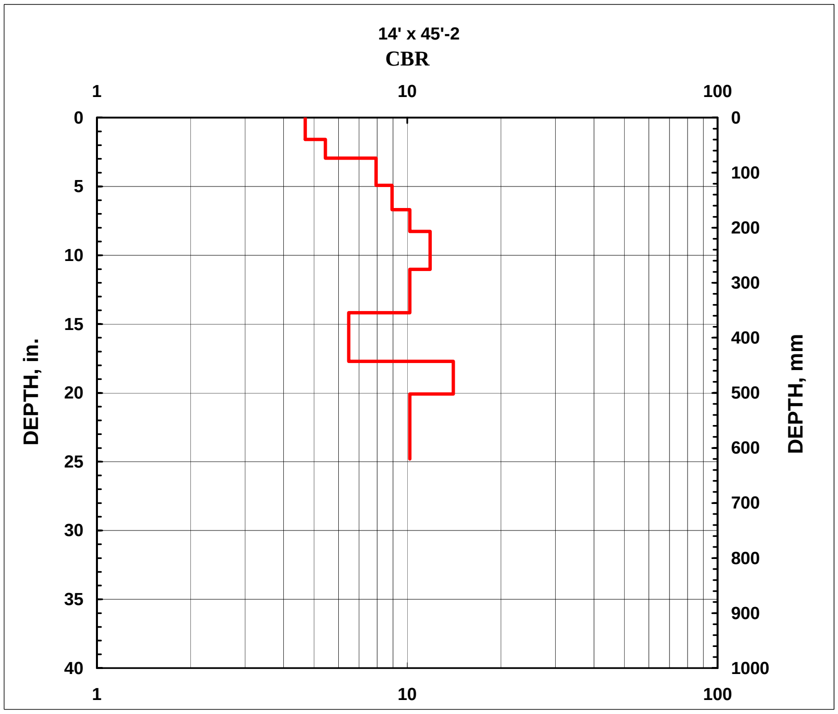
**Project:** *VTP-FERF* **Date:** 30-May-01

**Location:** 14' x 45'-2 **Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 40 | 1 |
| 1 | 75 | 1 |
| 1 | 100 | 1 |
| 1 | 125 | 1 |
| 2 | 170 | 1 |
| 2 | 210 | 1 |
| 2 | 245 | 1 |
| 2 | 280 | 1 |
| 2 | 320 | 1 |
| 2 | 360 | 1 |
| 2 | 420 | 1 |
| 1 | 450 | 1 |
| 2 | 480 | 1 |
| 2 | 510 | 1 |
| 2 | 550 | 1 |
| 2 | 590 | 1 |
| 2 | 630 | 1 |



# DCP TEST DATA

**File Name:** DCP

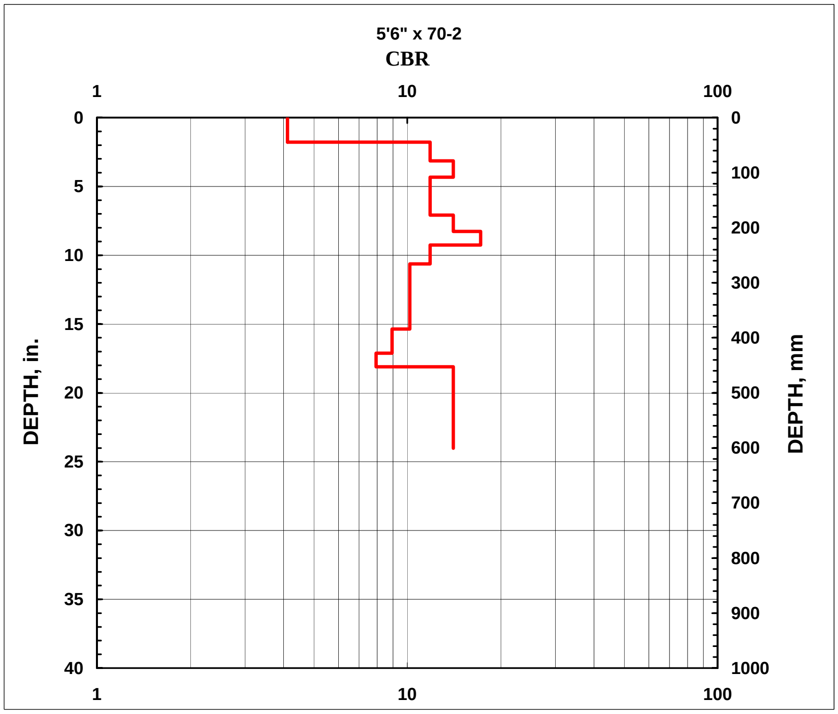
**Project:** *VTP-FERF**
**Location:** 5'6" x 70'-2

**Date:** 30-May-01
**Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 45 | 1 |
| 2 | 80 | 1 |
| 2 | 110 | 1 |
| 2 | 145 | 1 |
| 2 | 180 | 1 |
| 2 | 210 | 1 |
| 2 | 235 | 1 |
| 2 | 270 | 1 |
| 2 | 310 | 1 |
| 2 | 350 | 1 |
| 2 | 390 | 1 |
| 2 | 435 | 1 |
| 1 | 460 | 1 |
| 2 | 490 | 1 |
| 2 | 520 | 1 |
| 2 | 550 | 1 |
| 2 | 580 | 1 |
| 2 | 610 | 1 |



# DCP TEST DATA

**File Name:** DCP

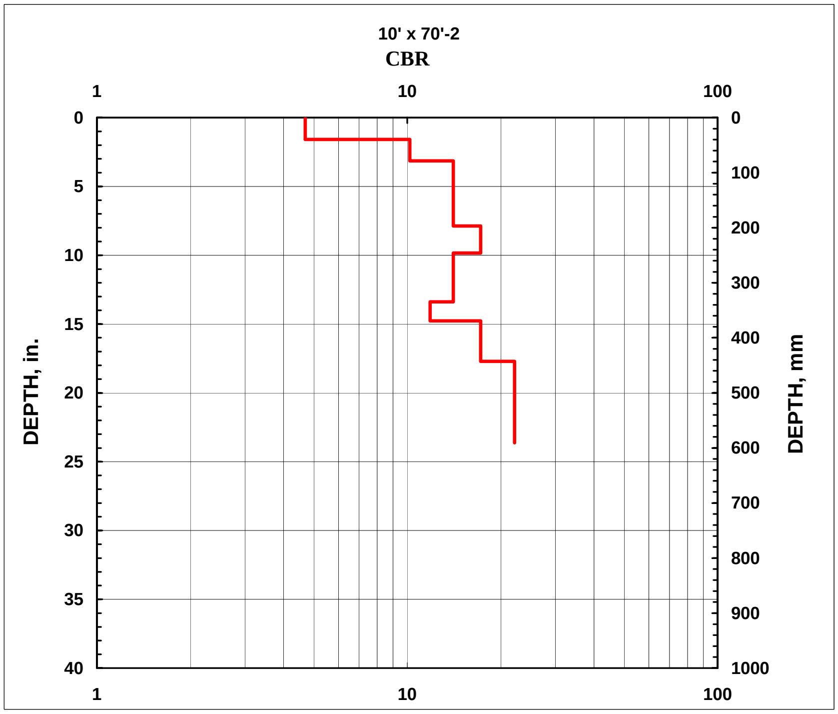
**Project:** *VTP-FERF*
**Location:** 10' x 70'-2

**Date:** 30-May-01
**Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 40 | 1 |
| 2 | 80 | 1 |
| 2 | 110 | 1 |
| 2 | 140 | 1 |
| 2 | 170 | 1 |
| 2 | 200 | 1 |
| 2 | 225 | 1 |
| 2 | 250 | 1 |
| 2 | 280 | 1 |
| 2 | 310 | 1 |
| 2 | 340 | 1 |
| 2 | 375 | 1 |
| 2 | 400 | 1 |
| 2 | 425 | 1 |
| 2 | 450 | 1 |
| 3 | 480 | 1 |
| 3 | 510 | 1 |
| 3 | 540 | 1 |
| 3 | 570 | 1 |
| 3 | 600 | 1 |



# DCP TEST DATA

**File Name:** DCP

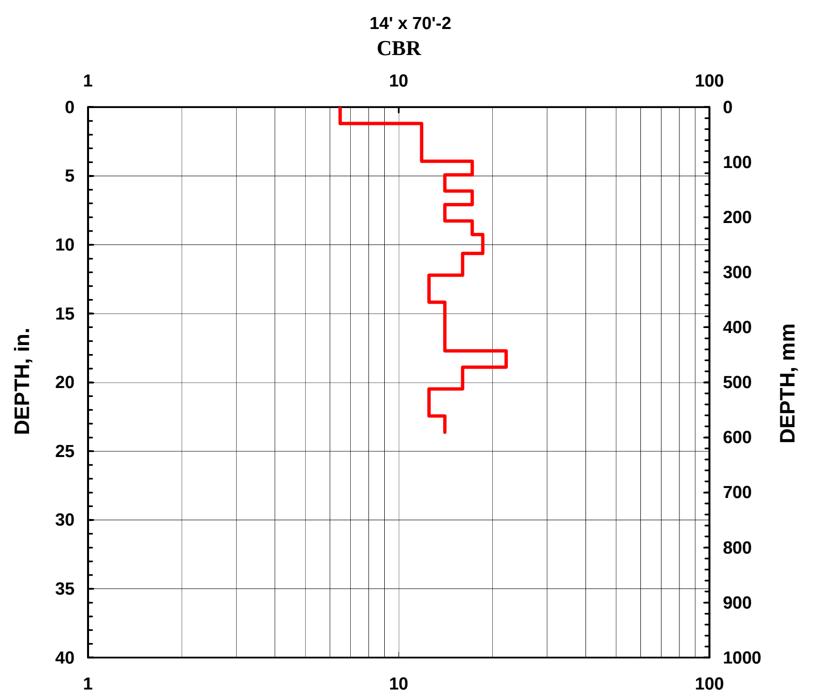
**Project:** *VTP-FERF* **Date:** 30-May-01

**Location:** 14' x 70'-2 **Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 30 | 1 |
| 2 | 65 | 1 |
| 2 | 100 | 1 |
| 2 | 125 | 1 |
| 2 | 155 | 1 |
| 2 | 180 | 1 |
| 2 | 210 | 1 |
| 2 | 235 | 1 |
| 3 | 270 | 1 |
| 3 | 310 | 1 |
| 3 | 360 | 1 |
| 2 | 390 | 1 |
| 2 | 420 | 1 |
| 2 | 450 | 1 |
| 3 | 480 | 1 |
| 3 | 520 | 1 |
| 3 | 570 | 1 |
| 2 | 600 | 1 |



# DCP TEST DATA

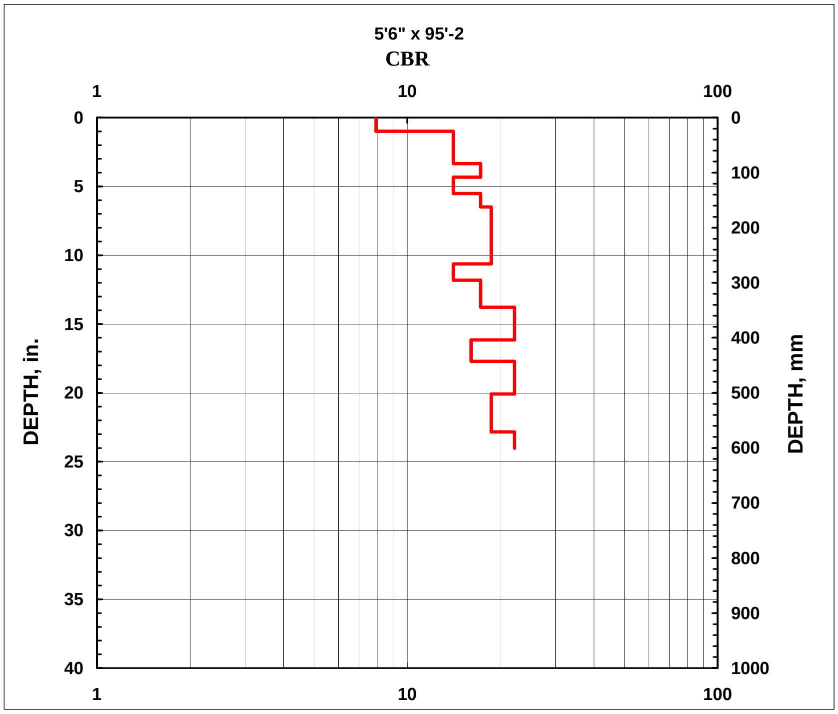
**File Name:** DCP

**Project:** *VTP-FERF*
**Date:** 30-May-01
**Location:** 5'6" x 95'-2
**Soil Type(s):** A4-Hanover Silt

Hammer
- ◯ 10.1 lbs.
- ◉ 17.6 lbs.
- ◯ Both hammers used

Soil Type
- ◯ CH
- ◯ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 25 | 1 |
| 2 | 55 | 1 |
| 2 | 85 | 1 |
| 2 | 110 | 1 |
| 2 | 140 | 1 |
| 2 | 165 | 1 |
| 3 | 200 | 1 |
| 3 | 235 | 1 |
| 3 | 270 | 1 |
| 2 | 300 | 1 |
| 2 | 325 | 1 |
| 2 | 350 | 1 |
| 3 | 380 | 1 |
| 3 | 410 | 1 |
| 3 | 450 | 1 |
| 3 | 480 | 1 |
| 3 | 510 | 1 |
| 3 | 545 | 1 |
| 3 | 580 | 1 |
| 3 | 610 | 1 |



# DCP TEST DATA

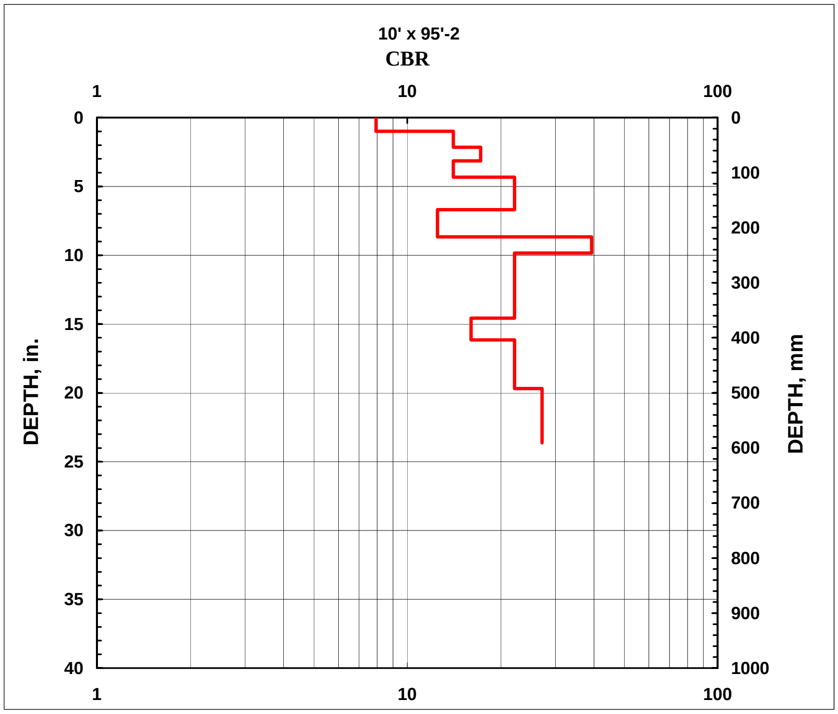
**File Name:** DCP

**Project:** *VTP-FERF**Date:** 30-May-01
**Location:** 10' x 95'-2 **Soil Type(s):** A4-Hanover Silt

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 25 | 1 |
| 2 | 55 | 1 |
| 2 | 80 | 1 |
| 2 | 110 | 1 |
| 3 | 140 | 1 |
| 3 | 170 | 1 |
| 3 | 220 | 1 |
| 5 | 250 | 1 |
| 3 | 280 | 1 |
| 3 | 310 | 1 |
| 3 | 340 | 1 |
| 3 | 370 | 1 |
| 3 | 410 | 1 |
| 3 | 440 | 1 |
| 3 | 470 | 1 |
| 3 | 500 | 1 |
| 3 | 525 | 1 |
| 3 | 550 | 1 |
| 3 | 575 | 1 |
| 3 | 600 | 1 |



# DCP TEST DATA

**File Name:** DCP

**Project:** *VTP-FERF* **Date:** 30-May-01

**Location:** 14' x 95'-2 **Soil Type(s):** A4-Hanover Silt

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 1 | 30 | 1 |
| 2 | 65 | 1 |
| 2 | 90 | 1 |
| 2 | 120 | 1 |
| 2 | 145 | 1 |
| 3 | 175 | 1 |
| 3 | 210 | 1 |
| 3 | 250 | 1 |
| 3 | 290 | 1 |
| 3 | 330 | 1 |
| 3 | 370 | 1 |
| 3 | 400 | 1 |
| 3 | 430 | 1 |
| 3 | 470 | 1 |
| 3 | 500 | 1 |
| 3 | 540 | 1 |
| 3 | 585 | 1 |
| 3 | 625 | 1 |