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Final Report

# Field Assessment and Analytical Modeling of Ultra Thin Whitetopping

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**Florida Department of Transportation**

By

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16. **Abstract**


In this investigation the mechanical behavior of three test tracks of ultra thin White topping (UTW) overlays were studied under different conditions of loading and material properties. The emphasize was on the analytical modeling of these test tracks where various nonlinear finite element schemes were used. The full scale UTW test tracks were built at the FDOT materials office in Gainesville, and were subjected to repetitive truck loading over a period of 24 months. Along with the field testing it was decided to construct three similar FE models using geometrical and material properties similar to those in the field. In Test Track I, a 4 in. thick UTW was placed on an existing pavement composed of 1.5-inch asphalt pavement and 6-inch. concrete base. Test Track II consisted of 3 and 4 in. thick sections. Test Track III was built with a 2-inch thick concrete overlay. Joint spacings ranged 4 ft to 6 ft for Test Tracks I and II, and 3 ft x 3 ft to 12 ft for Test Track III. Fibrillated Polypropylene fibers were used in Tracks I and II. Monofilament Polyolefin fibers were used in Track III. High early strength concrete mixture was designed for the UTW. Bond strength at the concrete-asphalt interface was generally above 200 psi. The test tracks were subjected to approximately 50, 000 (18-kip) ESALs using a truck loaded with concrete blocks. During the fate of this project, Falling Weight Deflectometer test results showed significant reduction in the surface deflection which indicted an improvement in the structural capacity of the pavement after the UTW placement. A reduction in the surface deflection of about 75 percent was experienced at Track I and about 78 percent at Track II. Surface deflection at Track III was reduced by about 46 percent when UTW was used. Also, frequent condition surveys exhibited good performance of the three test tracks. Structural cracking was noticed on Track III with 12ftx12ft panel. In general, the good performance of the test tracks was attributed to the method of UTW preparation and to the good performance of the underlaying pavement layers. The same conclusion was deduced from the analytical modeling. Reducing the bonding strength, increasing the panel size, and reducing the overlay thicknesses were among the main factors for crack development, and hence, the low performance of the UTW. Other contributed factors may include the existing layering stiffnesses, fatigue characteristics of the fiber reinforced concrete, and surrounding environmental conditions.

Another practical experience on UTW was gained from the first FDOT project at the Ellaville Truck Weigh Station on I-10 in Northwest Florida. This rehabilitation project included the placement of UTW on the existing asphalt pavement which had experienced severe rutting problems. Layer thicknesses for the UTW were 3 in and 4 in. The joint spacings for the UTW panels were 4 ft m and 5.25 ft. High Early Strength concrete was used in this project. As for the Gainesville test tracks, fibrillated Polypropylene fibers were included in the concrete for the sections on the west side of the weight platform and plain concrete was used on the east sections. The joints on the East section were sealed with silicone sealant while the joints in the west section were left unsealed. Falling Weight Deflectometer (FWD) tests from 60 locations along the length of the traffic lane showed an average decrease in the UTW surface deflection of about 63 percent . It appeared that the variability in asphalt and UTW thicknesses did not play a major role in the magnitude of the FWD deflections as evident from the deflection basin of the UTW. No significant differences were observed in the FWD deflections in the various UTW sections. Likewise, The panel dimensions did not impact the magnitude of the deflections. After six months, the FWD tests showed deflections of about 56 percent lower than what they were on the original asphalt layer. This indicates that the pavement was still acting as a composite element of two bonded layers possessing sufficient support to carry the high volume of heavy trucks. In about one year period, only 5.5 percent of the 1800 panels in the traffic lane developed cracks. Majority of the cracks were corner cracks. De-bonding at the UTW/asphalt interface was the most likely cause of the corner cracks. Also, late sawing of joints was the main contributor to the curling and de-bonding of the corners.

To facilitate estimating the required dimensions of an UTW overlay, the PI has developed a software called **FDOT Ultrathin White Topping (FUTW).** Parameters needed for the input file are related to the existing pavement layers and to the anticipated UTW joint spacing and concrete properties. All the calculations for the stresses and strains in concrete and asphalt respectively are done internally by the program and when the "Find UTW thickness" button is pressed, the least possible thickness needed for the UTW is given by the program with the condition that the total concrete and asphalt fatigues are less than 100%. Since UTW means thickness with a range between 2 to 4 in., the program starts with a trial thickness of 2 in. and keeps solving the problem until the condition that total fatigue for concrete and asphalt is less than 100% is satisfied and the result is given.


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The opinions findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Florida Department of Transportation and the Federal Highway Administration.

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## EXECUTIVE SUMMERY

The purpose of this study was to gain insight into the mechanical behavior of the Ultra Thin Whitetopping (UTW) concrete overlays. Several finite element models were examined and calibrated based on full scale field testing. Three full scale test tracks of UTW overlays were used for this purpose. The test tracks were constructed with various design parameters including, overlay thickness, panel size, and base layers. The three tracks were tested using a moving truck load, where pavement stiffnesses were determined at different cyclic loadings. For Test Track I, a 4 in. thick UTW was placed on an existing pavement composed of 1.5 in. asphalt pavement and 6 in. concrete base. Test Track II consisted of 3 and 4 in. thick sections. Test Track III was built with a 2 in. thick concrete overlay. Joint spacings ranged from 4 ft. to 6 ft. for Test Tracks I and II, and 3 ft. x 3 ft. to 12 ft. for Test Track III. Fibrillated Polypropylene fibers were used in Tracks I and II. Monofilament Polyolefin fibers were used in Track III. High early strength concrete mixture was used for all the tracks. The UTW overlays were subjected to approximately 50, 000 (18-kip)Equivalent Single Axle Loads ( ESAL) using a truck loaded with concrete blocks. The Falling Weight Deflectometer test results showed significant reduction in the surface deflection which indicted an improvement in the structural capacity of the pavement after the UTW placement. A reduction in the surface deflection of about 75 percent was experienced at Track I and about 78 percent at Track II. Surface deflection at Track III was reduced by about 46 percent when UTW was used. Also, frequent condition surveys showed good performance of the three test tracks. Structural cracking was noticed on Track III with 12ft.x12ft. panel. In general, the encouraging performance of the test tracks was attributed to the method of UTW preparation and to the performance of the underlaying pavement layers. The same conclusion was deduced from the analytical modeling. It was found that the reduction of the bond strength, the increase in the panel size, and reduction of the overlay thicknesses had contributed to the development of the UTW structural cracks, and accordingly jeopardized the performance of the UTW. Bond strength between the UTW and asphalt layer was essential for the long term performance of the layer. Based on the Iowa shear test, the shear strength at the UTW asphalt interface should be at least 200 psi. The UTW should be designed using short joint spacings. Results of this study showed that the preferred joint spacing for 3 and 4 in. thick UTW should be 4 to 6 ft. For 2 in. thick UTW the joint spacing should not exceed 4 ft. The effect of fibers on performance of the UTW could not be determined. Sections using plain concrete and those using fiber concrete performed equally well. A previous study by the principal investigator, however, showed that plastic fiber did prolong stages I and II of the fatigue life expectancy of concrete beam samples. This suggested that more truck load repetitions might have been needed to establish the testing results of the concrete samples. Another practical experience on UTW was gained from the first FDOT project at the Ellaville Truck Weigh Station on I-10 in Northwest Florida. This rehabilitation project included the placement of UTW on the existing asphalt pavement which had experienced severe rutting problems. Layer thicknesses for the UTW were 3 in. and 4 in. The joint spacings for the UTW panels were 4 ft. and 5.25 ft. High Early Strength concrete was used in this project. As for the Gainesville test tracks, fibrillated Polypropylene fibers were included in the concrete for the sections on the west side of the weight platform and plain concrete was used on the east sections. 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During a one year period, only 5.5 percent of the 1800 panels in the traffic lane developed cracks. A majority of the cracks were corner cracks. De-bonding at the UTW/asphalt interface was the most likely cause of the corner cracks. Also, late sawing of joints was the main contributor to the curling and de-bonding of the corners. To facilitate the estimate of the required dimensions of an UTW overlay, the principal investigator has developed a software called Florida Department of Transportation ( FDOT) Ultra Thin Whitetopping (FUTW). Parameters needed for the input file were related to the existing pavement layers and to the anticipated UTW joint spacing and concrete properties. All the calculations for the stresses and strains in concrete and asphalt respectively could be done internally by the program and when the "Find UTW thickness" button is pressed, the least possible thickness needed for the UTW is given by the software with the condition that the total concrete and asphalt fatigues should be less than 100%. Since UTW means thickness with a range between 2 to 4 in., the program starts with a trial thickness of 2 in. and keeps solving the problem until a satisfactory condition of total fatigue for concrete and asphalt is less than 100% and the result is given.

# CHAPTER 1

## INTRODUCTION

Ultra Thin Whitetopping (UTW) is a relatively new technique for resurfacing deteriorated asphalt pavements. In this process, very thin concrete slabs (2 to 4 in. thick) are placed on old asphalt pavements to form bonded (or partially bonded) composite pavements. The reduction of thickness is justified by the use of high quality concrete with relatively high strength, shorter joint spacing and bonds between the concrete and existing asphalt pavement. This has been among the rehabilitation options for years, especially at intersections which have become deeply ruffed from the stops and starts of thousands of vehicles daily.

## PROBLEM STATEMENT

Mechanical behavior of UTW concrete overlay under various loading and environmental conditions can best be investigated by using a rigorous mechanistic approach such as the finite element (FE) method. Developing a well calibrated FE model can provide insight about the effect of many design parameters such as layer thickness, addition of concrete fiber, subgrade and base stiffnesses, panel size, and depth of joints. Additional environmental factors can also be simulated using FE modeling. These parameters can be studied without the need for a comprehensive field testing or the need of building full scale UTW prototypes. However, the need for field testing and gathering real time data on the mechanical behavior of the UTW is essential. Without such data it would be very difficult to establish a realistic finite element model.

The finite element approach was followed in this investigation and was supported by testing three tracks, which were constructed at the FDOT materials office in Gainesville, Florida. Several variables were included in this study among them were slab thickness, joint spacing, and base stiffness. In general the study was divided into five stages.

1. Conducting field tests and measurement on typical overlays.
2. Investigating current methods of analysis.
3. Using numerical techniques (FEA) to study different UTW designs and compare the field testing measurements.
4. Studying effect of slab size and bonding between UTW and asphalt layer.
5. Developing computer software to design UTW according to Florida Department of Transportation methodology.

# CHAPTER 2

# LITERATURE REVIEW

## 2.1 BACKGROUND

Historically, the first whitetopping overlay project was completed in Terre Haute, Indiana in 1918 (Website, http://www.irmca.com/utw/sld002.htm). The modern use of pavement overlay began in the 60's. Regular whitetopping overlays were of thicknesses greater than 4 in. and were bonded to the existing asphalt layers. The positive performance of the bonded concrete/asphalt section promoted the use of thinner sections of white topping. Such changes necessitate the addition of fibers in the concrete to prolong the crack initiation and possible propagation. Also, joint spacing was another parameter which was considered in the new thin sections. The development of these overlay sections which have been known as the ultra-thin whitetopping (UTW) overlays has attracted the attention of pavement engineers and state highway agencies for their possible applications in certain pavement conditions. The ultra-thin whitetopping looks and acts like regular concrete pavement overlays. The only noticeable difference being noticeable is that joint spacings (2 to 6 ft. panels) which are closer than the regular concrete overlays.

Rutted asphalt is a prime candidate for UTW, provided the asphalt does not have an underlying base failure. Severely mapped or alligatored asphalt is not a suitable choice for UTW. Pavement sections with underground conduits and drainage pipes are also not suitable for using UTW.

Good bonding between the concrete and the underlying asphalt is an important element in the performance of the UTW. When the concrete layer bonds to the underlying asphalt for a small size slab (narrow joint spacing) a composite section is formed which reduces the edge stresses and yields a continuous distribution of the structural stresses to the subsequent pavement layers.

The questions remain, however, as to what constitutes "good bonding" ? and what construction methods could we follow to achieve such bonding? Moreover, is there any testing procedure that one can follow to determine the actual concrete-asphalt bonding? Such inquires have been loosely addressed in the literature and in this study an attempt has been made to answer some of these concerns and to identify what might be required to establish a rational procedure for characterizing friction/adhesion of concrete-asphalt interface.

## 2.2 COMMON STEPS FOR UTW CONSTRUCTION

Several steps are usually followed to construct a typical UTW overlay. The most common steps include the following:

1- Core the existing surface for asphalt depth determination

2- Clean the surface to provide an adequate overlay bonding

3- Prepare the fiber reinforced mix with a selected fiber type (2.5 in monofilament plastic fibers for structural cracking, and fibrillated plastic fiber for shrinkage cracking)

4- Cast the forms and start the curing process

5- Prepare the joints with an early sawing

## 2.3 UTW EXPERIENCE

Since the early 1990s, several UTW projects have been constructed by state highway agencies. The performances of most of these UTW projects were considered encouraging. Among these UTW projects were:

1. Louisville, Kentucky
2. Georgia, I-85 Truck scales
3. New Jersey, Ramp
4. Memphis, Tennessee
5. Allentown, Pennsylvanian
6. Iowa 21
7. Spirit St.Louis Airport
8. Leawood, Kansas

## 2.4 LOUISVILLE UTW PROJECT

The importance of the Louisville project is derived from the fact that it was among the first UTW ventures with more published details than others. In that project, an experimental UTW section was constructed in Louisville, Kentucky in September 1991. The main features of the project were as follows:

Two thin sections of concrete overlay (2 in. and 3 ½ in.)

1. High-early strength concrete mixture
2. Fast track paving techniques
3. Polypropylene fibers
4. The roadway carries a high number of heavy trucks (400 – 600 trucks/day).

Also the project site offered several unique features which made it ideal for this project. These features included the fact that all trucks approaching the site were weighed upon entering the facility and all loading information was available for further analyses. The large amount of traffic could also provide an accelerated loading function simulating years of equivalent car loading in a relatively short period of heavy truck applications.

The total length of the driveway entrance was 775 ft. A 600 ft. section of this road was chosen for the test road section. The dimensions of the test section was laid out to provide a 275 ft. section of 2 in. overlay followed by a 50 ft. transition section then a 275 ft. section with 3.5 in. overlay. The existing asphalt pavement was in a serviceable condition exhibiting a minimal level of distress.

The study of this section consisted of several major components including: (1) evaluation of the foundation support; (2) pavement construction and instrumentation; (3) field stress analysis and theoretical modeling of the structure; (4) post-construction distress analysis; and (5) traffic load evaluation. Additionally, Falling Weight Deflectometer (FWD) tests were conducted to assess the pavement structure.

**Observation of Distress Progression**

Comparison of Figures 2.1, to 2.4 shows the progression of pavement cracking at different time levels. The primary observed distress was corner cracking. For the inbound lane, this corner cracking appeared to occur at about twice the rate on the 2 in. section compared to the 3.5 in. section. Corner cracking in the outbound lane occurred at a rate of about 10 percent more than the rate of cracking on the inbound lane.

### Comparison with AASHTO Test Road Cracking

The American Association of State Highway and Transportation official (AASHTO) test road was built with a 3.5 in. thickness, non-reinforced concrete pavement with paved shoulders on 6 in. untreated granular subbase subjected to 10,900 kg tandem axle loads. Longitudinal Cracking originated at the joints. The progression of crack patterns of 8 in. thick slab on untreated granular subbase subjected to 13,600 kg single axle load is presented in Figure 2.6. Transverse cracks originated near the center third of the 15 ft. slabs.

Neither of these two experiences at the AASHTO test road relate well to the observed pavement cracking pattern at the Louisville experiment. But the three notable differences were:

1. The Louisville experiment involved concrete overlays of asphalt. A bonded interface between the concrete and asphalt that led to different locations for critical stress than concrete on an untreated subbase.
2. The AASHTO test road pavements contained dowels.
3. The joint spacing at the Louisville experiment was 40% of that at the AASHTO test road.

### Performance of the Small Panels.

One of the most remarkable results of the experiment mentioned was the performance of the 2 ft x 2 ft panel in the fully loaded inbound lane. Not one crack or sign of distress was noticed in any of these panels despite the load application of almost 600 trucks per day for over a year.

**Summary of the Louisville Project**

The Louisville experiment has shown that ultra thin whitetopping is feasible and capable of carrying volumes of traffic typically associated with low-volume roads, residential streets, and parking lots. This innovative pavement rehabilitation technique warrants additional experiments with different environmental and traffic conditions, accelerated load test studies, and additional analytical work. The joint spacing most likely to hold the corner cracks to a minimum is 3 ft. Concrete sawing with soft-cut should start start as soon as the operator can walk on the material without sinking in. Brooming and tinning should be done early. Use low water-cement ratio mixes which provide sufficient strength at 18 to 24 hrs.

The study showed that the composite section had opposing effects on the corner stress. There was a reduction on the concrete stresses because the pavement section was thicker. However, because the corner behaved as a cantilever and had its maximum stresses occur at the top of the slab, the shifting down of the neutral axis increased the corner stresses. Essentially then, the corner stresses decreased because the bonding action created a thicker section, but, were increased because of the neutral axis shifted down from and away from the top surface.

If the neutral axis shifted low enough in the concrete, the critical load location may move from the edge to the corner depending on the materials and layer characteristics. This explains why many of the UTW projects have developed corner cracking. Therefore, the designer must look at both the edge and corner in order to determine the critical load location. **Figure 2.11** shows how bonding decreased on slab stresses for a 3 in. UTW over 4 in. of AC. To estimate where the critical load location may be, one can compare the stiffness of the concrete layer to the asphalt layer. Stiffness of a layer, D, is defined as:

For a fully bonded system, when the stiffness of the asphalt is approximately 20% of the stiffness of the concrete ($D_{asphalt}/D_{concrete} = 0.20$), the critical loading shifts from the edge to the corner for a fully bonded system. However, according to Wu (1998), these systems are partially bonded systems. Consequently, as bond decreases, the asphalt carries less load and the asphalt stiffness to concrete stiffness ratio ($D_{asphalt}/D_{concrete}$) needs to be even higher for the critical load location to remain at the corner. If bond is ever lost, the critical load location will be the edge. There have been no failures due to the shearing at the concrete – asphalt interface. Still, this may cause problems in the future and should be investigated further.

## 2.5 STATE OF PRACTICE OF UTW

Important performance information is emerging from UTW projects in service; but each project is only one single point of reference, usually not systematically quantifying comparisons of different design features. However, comprehensive research projects are being analyzed to shed light on UTW design questions. Up until now, six projects have been or will be instrumented with strain gauges to measure the effects of truck loads. Other tests at most of the sites are: deflections measured by a falling weight deflectometer, surface profile and temperature measurements, condition surveys, and cores to measure bond between concrete and asphalt.

The adequate performance of the UTW is attributed to two factors:

1. Concrete bond to asphalt creating a monolithic section, and
2. The use of short joint spacings.

The bond or high friction at the concrete-asphalt interface creates a composite section which lowers the neutral axis and, accordingly, the stresses in the concrete are substantially reduced. Short joint spacings also reduce stresses because the slabs are not long enough to develop as much flexural moments. Also, short slabs reduce curling and warping stresses at the edges.

The Louisville project surprised everyone by carrying many more heavy trucks than predicted (Cole, L.W, 1993). Strain measurements showed considerable bond between concrete and asphalt. Completed on Iowa 21 in July 1994, the largest project is a seven-mile test pavement that includes 65 concrete overlay sections with a wide assortment of thicknesses, joint spacings and treatments of the existing asphalt pavement strains, deflections and other data. Results were studied over a five year period by Iowa state University.

In late 1994, instrumentation was installed in the general aviation apron at the St. Louis Airport. The apron carries 500 to 600 aircraft per day with loads up to 12500lb. Construction Technology Laboratories (CTL) are monitoring the data on how the UTW behaves over time under loads and thermal stresses. Initial strain measurements indicate high bond at the interface. So CTL has developed three-dimensional (3D) finite element software; conventional computer techniques can't model variations of this condition.

The Colorado Department of Transportation completed three instrumented test projects in 1996. CTL is using the data from these projects along with the 3D-computer model to develop guidelines for UTW design. Experience gained from these research efforts and the performance of in-service projects might lead to the optimal use of UTW.

### Thickness Design

Design analysis of UTW is more complex than that of conventional design procedures and differs in the following ways:

1. a bond between concrete and asphalt creates a composite pavement, which lowers the neutral axis so that load stresses are substantially reduced
2. the short joint spacing substantially reduces load curling stresses, and reduces or eliminates slab edge uplift
3. the concrete strength is usually greater than that of conventional concrete and if fibers are used, fatigue characteristics may be improved
4. an asphalt layer provides a strong and non-erodible support for the concrete

### Mechanistic Analysis

Normal whitetopping design procedures characterize the support of the existing pavement by using an increased k-value on top of the asphalt. Also, the composite action between the concrete and asphalt is not recognized by conventional procedures. For UTW, both of these assumptions result in considerable over estimation of computed stresses and thickness requirements. If one compares pavement stresses computed by conventional two-layer model and three-layer analysis with a degree of composite action (a more realistic model), the critical stress is the tension at the bottom of the concrete and in this comparison, the value is reduced by about one-half for the three–layer analysis. Measurements of the test sections discussed previously indicate that there is a considerable bond or friction between the concrete and asphalt layers. In this case, the pavement should be analyzed as a composite system in which both the layers are characterized by their thicknesses and elastic properties, all on top of a k-value for the foundation (base course and subgrade).

The degree of composite action assigned in the analysis has a great influence on the stresses computed. At the research sites, strain gauges installed in the concrete and asphalt layers and shear strength on cores should provide guidance on what degree of composite action is sustained.

A mechanistic analysis of UTW also should include the effect of slab size (joint spacing). As shown in Fig. 2.12, there is a substantial reduction in load stresses as slab size decreases. This is also true for curling and warping stresses that occur due to temperature and moisture gradients in the concrete slab.

Although stress decreases with slab size, pavement deflections increase. With very short slabs and without substantial thickness of asphalt and base course, deflections and vertical strains are high. This could lead to excessive permanent deformation in the foundation after many load applications. Thus, there may be an optimum joint spacing for which stresses are reduced but deflections are not excessive. Performance data from the research sites and other UTW projects should help determine ideal slab size. **Figs. 2.12** through **2.13** are not meant to quantify specific values of stress reductions for use in mechanistic design but are used only to demonstrate that UTW has unique mechanistic features. The benefits of composite action and shorter-than-normal joint spacings help explain the outstanding performance of very thin pavements that would not be predicted by conventional design analysis.

The term ultra-thin whitetopping has been coined for overlays of a thickness of 4 in. or less. However, this definition is arbitrary because the same benefits should hold true for thicker whitetopping constructed with the materials and methods used earlier.

## 2.6 LOAD CARRYING CAPACITY AND SERVICE LIFE

The thickness design concept for UTW differs from the traditional design concept for other concrete pavements. UTW is essentially a maintenance strategy, which is constrained by existing pavement factors and not necessarily designed for a 20-30 year service life. The constraints that limit and often prescribe the UTW thickness include elevation of an adjacent pavement lane or curb and gutter, the depth of the existing asphalt, and the depth of milling. As a result, UTW thickness design becomes a process of evaluation, rather than design, and involves determination of two important factors:

- Load-carrying capacity
- Expected service life

## 2.7 UTW USING FIBER REINFORCED CONCRETE

The Tennessee Department of Transportation began experimenting with polypropylene fiber-reinforced ultra-thin whitetopping (UTW) overlays for urban intersections in Tennessee. The use of polypropylene fibers allows the concrete to be placed at a minimum thickness that would not have encapsulated wire mesh with sufficient concrete cover. Additionally, fibers are distributed throughout the concrete mix, providing multidirectional reinforcement that absorbs energy, increases impact and freeze-thaw resistance and further aids in reducing cracking. Fibers also reduce the permeability of concrete which is a big advantage in UTW.

Typically, 3 $lb/yd^3$ of synthetic polypropylene fibers are used in UTW applications. This amount is twice the normal quantity used in regular slab applications. Ultra-thin synthetic fiber-reinforced concrete overlays at urban intersections using the bonded concrete technique are proving to be reliable alternatives to milling and replacing with HMA every two or three years.

The work to date has shown the use of UTW overlays at high Average Daily Traffic (ADT) intersections is technically sound and reduces time spent re-paving with HMA. The safety advantages - elimination of runs and ridges, skid resistance and improved light reflection - are equally beneficial. The 1.36 kg. of synthetic fiber enhance the fatigue strength and modify the cracking mechanism. With the expected life of a UTW projects now being 8 to 12 years, use of UTW in intersections should prove to be very economical option.

The concrete used in most of the projects contained a small percentage (by volume) of steel or polypropylene fibers, which are a few mils (microns) in diameter and from 0.5 to 2.5 in. long, randomly distributed throughout the mixture. Even nylon, polyester, polyethylene, aramid, carbon and acrylic fibers in similar configurations are in use. These materials may have tensile strength and moduli of elasticity much greater than conventional concrete. Because of the greater unit cost of concrete containing reinforcing fibers, they typically are used in layers 2 to 3 in. thick for highway work. However several airfield projects have employed fiber reinforce concrete up to 7 in. thick.

Some researchers found that fiber-reinforced concrete must contain greater proportions of fine materials to be satisfactorily workable than conventional mixtures. This may be accomplished through the use of high cement factors up to10 bags per $yd^3$ is typical, or the addition of other pozzolanic materials such as fly ash. Air entraining, water-reducing super-plasticizers, and set-retarding admixtures all have been used successfully with fiber-reinforced concrete. It is said that fiber reinforcement bears a fundamental engineering inefficiency, as many fibers are not positioned to resist tensile stresses from the applied loads.

## 2.8 GUIDELINES FOR BONDED CONCRETE OVERLAYS

A thin concrete overlay provides a substantial increase in structural capacity. A bonded overlay will reduce the critical structural responses under the load at the bottom of the existing slab. Edge wheel loadings typically induce the most critical tensile stresses in a pavement system. These critical edge stresses are reduced when a monolithic slab is formed by the overlay and the existing pavement. This results in less damage per load application and consequently a higher increase in load capacity. In fact, increasing the monolithic slab thickness beyond 11 inches will practically eliminate fatigue cracking except under extremely heavy traffic conditions.

In comparison to bituminous overlays, bonded concrete overlays provide significantly more structural improvement per inch of thickness. This means that it requires 6 to 7 in. of bituminous overlay to produce the same edge load stress induced under a 3 in. bonded overlay. Bonded concrete overlays are more structurally efficient per inch of material.

Because of the structural efficiency, bonded overlays of concrete can provide a long service life at reasonable cost. Some bonded concrete overlays have sustained traffic for over 40 years. A typical design period is between 15 to 25 years. This depends on the existing pavement and expected volume and type of traffic.

### Thickness

Existing pavement condition, traffic, environmental factors and constructability influence design thickness. For most conditions, a thickness of 3 to 4 in. is sufficient. Bonded concrete overlays have been designed and built to as little as one inch thick. However, it has been determined that two inches is the practical minimum thickness for construction with slip-form paving equipment.

### Achieving Bond

Achieving bond is a key to the long-term life extension from a bonded overlay. Therefore, the steps made to develop a good bond are critical in the construction of bonded overlay. On the other hand, losing bond does not signify failure of a bonded overlay. Performance is reduced when bond is lost, but the overlay will carry a great volume of traffic.

A bond strength of 200 pounds per square inch (psi) is sufficient to withstand shearing forces and ensure bond is maintained. This number was determined in the laboratory by comparing flexural tests of beams cast monolithically and also in two layers (bonded). At bond shear strengths of around 200 psi the laminated or bonded beams were nearly as strong as the monolithic beams.

Good bond is achieved by using proper procedures for concrete consolidation, curing and mix design. These factors offset the ill-effects that drying shrinkage can have on very thin sections (3 in.). To maintain good bond, joint type, location and width from the existing pavement must be matched in the overlay. If this is not done, excessive compressive forces may develop as the underlying pavement expands during increased temperatures. This will cause the overlay to de-bond and in the worst case buckle. A cement grout can generate the necessary bond strength and has proven to be effective. Vibration is another factor, which contributes to the bond strength development without using grout.

## 2.9 TYPES OF OVERLAYS AND RESURFACING

### Jointed Plain Concrete Pavement (JPCP) Overlays

Unreinforced (plain) concrete has been and remains a concrete surfacing option. Plain concrete resurfacing may be bonded, unbonded or partially bonded. Because of the tendency of cracks in the underlying pavement to be reflected in the resurfacing if it is bonded partially, such interfaces are most often used where underlying pavement is in reasonably good condition. On distressed pavement, plain, unbonded resurfacing normally is used so that the interface material will deter reflective cracking. Thus, it is not necessary that joints coincide with those in the underlying pavement. Whitetopping accounts for about one-fourth of the Jointed Portland Concrete Pavement (JPCP) overlays reported. Portland Cement Concrete(PCC) overlays of less than 3 to 4 in. tend to be bonded to the underlying layer, though others may employ any interface.

### Jointed Reinforced Concrete Pavement (JRCP) Overlays

Like reinforced concrete pavement, reinforced resurfacing employs distributed steel to control the movement of shrinkage cracks and to accommodate curling and warping stresses in long slabs. If used as a bonded surface, a reinforced layer's joints must coincide with joints in the underlying pavement. Otherwise reflective cracking will result in an undesirable random cracking pattern. For partially bonded and unbonded resurfacing, a mismatching of joints is again recommended. Reinforced resurfacing layers are subject to one practical limitation not applicable to those without reinforcement: the need to provide a minimum cover on the reinforcing steel. Historically, few reinforced resurfacing of less than 5 in. have been used.

**Continuously Reinforced Concrete Pavement (CRCP) Overlays**

This also has similar design requirements to CRCP. Steel reinforcement of approximately 0.6 percent of the concrete cross-sectional area is used in the longitudinal direction to control random Shrinkage cracking. Transverse steel may be used. Little bonded or partially bonded CRC resurfacing has been reported, and when used, it has not performed well. Unbonded Continuous Reinforce Concrete(CRC) resurfacing has been used on all types of pavement, usually to restore ride quality in heavy traffic corridors and where it is not feasible to match the joints of underlying layers. Several CRC overlays are found as whitetopping applications.

**Fibrous Reinforced Concrete Pavement (FRCP) Overlays**

Fibrous concrete resurfacing employs fibers made of steel or other randomly distributed throughout the mix. The fibers are purported to enhance flexural, compressive, and impact strengths and to reduce Shrinkage cracking. Although most fibrous resurfacing uses steel fibers, some have also been successfully constructed using glass and polypropylene fibers. Fiber-reinforced resurfacing may be bonded, unbonded or partially bonded. Because the fibers add significantly to the cost of the resurfacing, layers tend to relatively thin. On highways, most have been applied as bonded overlays to withstand curling stresses. However because of the generally thick layers required, much of the fiber-reinforced overlay work on airfields has been unbonded, some as whitetopping for flexible pavement.

## 2.10 INTERFACES USED WITH CONCRETE RESURFACING

**Bonded Interface**

Pavement engineers and others have long recognized that bonding a resurfacing to the underlying pavement to achieve monolithic behavior of the two layers is a very efficient means of structural enhancement. Early studies showed that the bonding between the two layers is principally a mechanical process that depends primarily on the soundness and cleanliness of the underlying pavement. Late work recognized a degree of chemical bonding between the overlay and the underlying pavement. A slight degree of roughness is desirable, but an extremely rough surface is not required. In that and other work, shear bond strengths up to 600 pounds per square inch (psi) have been reported. When properly constructed, the bond strength often exceeds the strength of even the strongest layer, so that bond specimens fail in one of the layers rather than at the interface.

**Unbonded Surface**

To achieve unbonded interface, special care is taken to ensure that no significant bond develops in situations where cracking or other pavement characteristics may be reflected from the underlying pavement. Generally, a bond-breaking/separation layer that does not bond strongly to concrete is used. In many cases, the bond breaking layer is underlaid with a separation layer to prevent interlock between the overlay and the joint faults or other irregularities in the underlying pavement surface. The separation layer often is composed of Asphalt Concrete(AC) covered with membrane curing compound to impede bonding. With a few special considerations, the resurfacing may then be constructed as if the underlying pavement were a conventional subbase layer.

**Partially Bonded Interface**

If the issue of bonding between the resurfacing and the underlying pavement is of little importance, such as on thick airfield pavement, the partially bonded approach may be employed. Because no particular attention is paid to cleaning and grinding the base pavement, various degrees of bonding may occur, but will have little bearing on the performance of the resurfacing. Partially bonded overlays are sometimes referred to as direct overlays, implying simply that the little or no surface preparation is done. In most of the recent literature, partially bonded overlays are considered special cases of the unbonded type, because the evidence shows that the performance is similar.

## 2.11 EVALUATION OF EXISTING PAVEMENT

An important consideration in resurfacing design is the condition of the existing pavement on which the resurfacing is proposed. The evaluation of the existing pavement condition consists of three major elements: (1) an evaluation of the serviceability or functional condition; (2) distress surveys; and (3) structural testing. The three are not mutually exclusive; any of the three separately or in combination can contribute to a decision to resurface. Evaluating the true condition of the existing pavements is one of the most critical factors in selecting the best overlay option. This evaluation should reflect how the existing pavement will effect the behavior and performance of the overlaid pavement. Such an evaluation should be based on structural or behavioral considerations rather than serviceability considerations. Some major points in PCC overlays discussed in report published by the National Research Council (NRC) are given below:

**An evaluation of the serviceability or functional condition:** Serviceability or functional adequacy generally refers to user perception of the pavement condition usually reflected in ride quality. While panel ratings may be used for this evaluation, most agencies use objective measures of ride quality, such as responsive type road roughness measurements. Correction of a low skid-resistance problem might be a reason for a functional overlay. Generally, functional overlays are used when the pavement no longer meets one or more levels of service established as agency policy. When required for functional reasons, resurfacing will generally be of the minimum thickness required for construction expediency or to restore the level of service to an acceptable level.

**Distress Surveys**: Distress surveys are used to determine the nature and extent of deterioration of the underlying pavement. Such surveys are extremely important in deciding the extent of the reflective cracking that may develop during the service life of the overlay. While several procedures have been established to evaluate such distress there seems to be little consensus on their use, although the Federal High Way Administration (FHWA) is attempting to standardized these procedures. The popular method currently in use is the Concrete Pavement Evaluation System (COPES) for concrete pavement.

**Structural Adequacy**: The preferred approach by far to determine the structural adequacy of an existing pavement is through Non-Destructive Testing (NDT) to assess the pavement's response to applied loads. The nature of response is directly related to the structural capacity. Another approach presented by AASHTO is termed the "remaining life." In this approach, the pavement is tested to determine the proportion of its original design life that has been consumed. Consumption may be based on time or on accumulated equivalent single axle loadings. A third approach, also presented by AASHTO, provides for an estimation of structural capacity through the analysis of distress surveys and existing pavement material properties.

## 2.12 RESURFACING/THICKNESS REQUIREMENTS

### AASHTO Design – Portland Cement Concrete (PCC) Overlays of Asphalt Concrete (AC) Pavement (Whitetopping)

The first step is to evaluate the existing pavement design to determine types and thicknesses of materials. Then, the projected 80kN (18-kip) ESAL in the design period are determined. Next, a general condition survey identifies distortions and stripping and quantifies the types and severities of the distresses present, while deflection tests are used to determine the effective dynamic k-value. Part II of the 1993 AASHTO Design Guide is used to determine the slab thickness ($D_f$) required for future traffic. The overlay is effectively a new concrete pavement built on an old AC pavement, so the effective slab thickness ($D_{eff}$) is zero and

$$D_{ol} = D_f - D_{eff}$$

While the later equation is used in the AASHTO design procedure, recent research in several locations suggests that assigning an effective thickness D = 0 for the existing pavement is an oversimplification. Iowa has constructed test sections where an effort was made to enhance the bond between the overlay and the underlying AC pavement. The strong bonding extended the contribution of the AC layer to the composite action of the structure. Similar results were found for a thin whitetopping of an AC pavement in Kentucky. These results suggested that some types of composite design approach may be justified for whitetopping. The whole area of whitetopping appears deserving of further research, some of which is underway.

## 2.13 JOINTS IN UTW

Placing joints in whitetopping layers is similar to placement in conventional on-grade concrete pavement construction, with the exception of possible adjustment to saw-cuts on overlays of distorted existing pavement. If distortions are excessive, it may be necessary to increase the saw-cut depth to achieve the desired crack control. Some engineers caution that too much distortion may lead to random cracking, and that PCC overlays on rutted pavement should be restricted to low-volume roads.

## 2.14 UTW EXPERIENCE AND PERFORMANCE

As noted earlier, whitetopping is an increasingly popular use of PCC resurfacing as a rehabilitation or structural strengthening alternative on AC pavement. Plain concrete, reinforced concrete and continuously reinforced concrete all have been used successfully as whitetopping. The performance reports of 1981 provided some performance data on PCC resurfacing of AC pavement. In that work, eight plain un-doweled overlays from 4 to 24 years old were evaluated and found to be in very good condition. The American Concrete pavement Association(ACPA) followed up the previous report with 1989-1990 reviews of 18 projects. One project reviewed in the above reports is located on US-101 in Orange County, California. The resurfacing consists of an inch of plain concrete placed in 1966. When reviewed in 1989, the project had carried more than 10 million ESAL and was considered to be in excellent condition. A 6 in. overlay in Iowa has carried truck traffic to a grain elevator for some 20 years and was rated in fair condition with some mid-panel cracking of 12 m. (40 ft.) long slabs.

The Florida DOT reported on recent (1988) construction of a concrete overlay of an existing flexible pavement which consists of 19 sections with slab thickness of 6,7 and 8 in. For each thickness, slab lengths were 12,14,16,18 and 20 ft., respectively. The resurfacing was designed for a 10-year life. The 7 in. thickness then increased by 1 in. and decreased by 1 in. to form the experiment. The designers anticipated relatively early distress on the 6 in. thick sections. Some sections were doweled and others relied on aggregate interlock for load transfer. Because of the poor condition of the existing pavement, a 1 in. leveling course of asphalt surface mix was placed immediately under the PCC overlay. After three years of service, all sections were reported to be in excellent condition with no signs of the anticipated early distress in the thin sections.

Recent reports from the Wyoming DOT show that whitetopping projects built without doweled joints can be subject to significant faulting in a few years of interstate traffic. Similarly, un-doweled whitetopping projects in Utah were found to fault significantly in fewer than 15 years, even with very low annual rainfall. They also reported some benefits in using a drain able layer between the PCC overlay and the underlying AC pavement.

PCC overlays of both AC and PCC pavements have been used in situations where it is desirable to rehabilitate only a portion of the width or vertical clearances of a pavement or where other geometric factors prohibit raising the grade with a conventional overlay. Instances of the first kind were most frequently observed on wide airfield pavement. As reported earlier, it is not uncommon to rehabilitate only the center third of 300 ft. wide runways where the loading intensity is the greatest. An inlay is one way such rehabilitation has been accomplished. Federal Aviation Administration (FAA) engineers have reported that this method of strengthening (resurfacing) has proven more economical than overlaying the entire width of the pavement and after two to three years of service, the pavements are entirely satisfactory.

It is not unusual to see one lane (usually the most heavily traveled) of highway projects rehabilitated with an inlay, especially where asphalt layers have been subject to recurring rutting. Inlays also are used where vertical clearances are limited and where an inlay is more economical because raising guardrails and filling slopes would be required with an overlay. Still others have been used to restore pavement damaged by freezing or by chemical spills.

A whitetopping inlay demonstration was conducted in Colorado in June 1990. Two 300 ft. overlay sections were placed at thicknesses of 3.5 and 5 in., respectively. These were jointed by a 100 ft. transition section. One short segment had proprietary polypropylene fibers added to the mix at a rate of 1.5 lb/yd$^3$. Another unique feature was a jointing arrangement, which resulted in very small slabs on the 3.5 in. section. Green concrete sawing began about 2 hrs. after concrete placement began and was configured to result in a 6.5 ft. x 6.5 ft. joint pattern. It was reported that small slabs were successful in controlling shrinkage cracking on the thin section. A 12.5 ft. x 13 ft. joint pattern was used on the 5 in. thick section. All saw cuts were 0.025 in. wide and 0.75 in. deep. Some were sealed with a cold-poured emulsified asphalt sealer, though most were left unsealed. The pavement was reopened to traffic in 24 hrs. after the start of construction.

Early evaluations showed some spalling at nearly every joint, three weeks after the joints were sawed. Performance after two years in service is considered satisfactory. Inlays constructed on I-70 in Kansas in the mid 1980's have not been very successful. These projects were partial inlays. The top 4 in. of an existing 10 in. AC pavement were removed and the inlay placed in a "bathtub". In addition, the design depended on aggregate interlock rather than dowels for load transfer. After about 3 years, the two projects exhibited joint faulting of more than 0.025 in. Some showed uncontrolled longitudinal cracking, and a minor amount of pumping and corner breaking. Kansas DOT concluded that the absence of dowels was major contributor to poor performance under the interstate traffic sustained by the projects.

### 2.15 THE GROWTH OF UTW

Perhaps the most dramatic change in PCC resurfacing is the rapid increase over the past decade in the use of whitetopping on highway pavement. Whitetopping is used either as an additional surface or as an inlay to an AC pavement. The literature suggests that much of this growth has taken place in response to increased rutting and other distresses on AC pavement in heavy truck corridors. Given the results of theoretical, laboratory and field studies indicating that rigid layers would be more resistant to the effects of higher axle loads and tire pressures, some agencies have chosen to combat these distresses through the provision of a more rigid surface course.

The number of documented whitetopping projects has grown from approximately 70 in 1982 to more than 150 in 1993. Again, much of that growth has been in the states where rutting of asphalt concrete has been a recurring problem. Colorado, Nevada and Nebraska combined have added more than 30 whitetopping projects in the past few years.

## 2.16 FAST TRACK PAVING

The efforts initiated in the mid 1980's demonstrating the feasibility of constructing PCC resurfacing in a "fast track" mode have proven to be worthwhile and have resulted in another advancement to the technology. This advancement coupled with the introduction of the zero-clearance paver, permits PCC overlay rehabilitation projects to be reopened to traffic within a day or two of the beginning of paving operations. This speed clearly enables fast-track PCC overlays to compete more readily with other resurfacing alternatives.

## 2.17 CHANGES IN TECHNOLOGY

Several other recent changes in the technology of PCC re-surfacing deserve special mention. The preparation of the old pavement surface to receive a bonded overlay has undergone some major changes in the last decade. The application of chemicals as the preferred cleaning method has given way to the use of mechanical devices. This change is due in part to improvements in the equipment needed to do the job mechanically. Fast shot blasting machines are now available that clean the surface without causing any damage to the underlying concrete.

Another change in bonded overlays is a change in attitude toward not using grout at the interface. Several agencies have successfully constructed bonded resurfacing with excellent bond strength without using grout.

On unbonded overlays, the practice of matching the joints in the overlay to those in the underlying pavement has completely given way to mismatching. Guide specifications now call for an offset of at least 0.9 m. (3 ft.) to provide for a sleeper slab effect, which provides improved load transfer and performance.

AC is no longer considered to be a bond breaker for unbonded overlays and whitetopping projects, although it is still useful as an interface layer. The AC tends to adhere to the PCC overlays, especially in hot weather. A "whitewash" film of membrane curing compound or loam-water is now used on the surface of the AC to inhibit overheating. The surface cooling both reduces the adhesion between layers and assists in avoiding some early curing problems associated with too much heat in the newly placed concrete overlay.

Another new development in interfaces for unbonded overlays is the use of drainage layers. The drainage layer serves the dual purposes of de-bonding and enhancing drainage. The National Council of Highway Research Program (NCHRP) project is evaluating the performance of un

overlays with special emphasis on the interlayer.

## 2.18 PERFORMANCE OVERVIEW OF UTW

While many UTW are too new to provide performance data, most of those that have been in service for some years are providing good to excellent performance. The argument can be made that the underlying AC pavement provides an ideal subbase course for the PCC resurfacing layer. Generally, the asphalt concrete layer contains the desired properties of strength and uniformity of support. Among experts, it is generally agreed that whitetopping will continue to be a popular alternative. Where performance problems have been identified, they generally have been related to the failure to provide adequate drainage and to the absence of dowels in joints. If either condition exists, PCC overlays in service for 10 to 15 years can be expected to demonstrate significant joint faulting. Overlays without adequate drainage features may also be subject to pumping. Inlays, a special class of whitetopping, also can provide good service, but must be designed with both positive drainage and positive load transfer features.

## 2.19 METHODS OF OVERLAY DESIGN

Four methods of overlay design are available depending on the type of the existing pavement. These methods are; (1)Hot Mix Asphalt (HMA) overlays on asphalt pavement; (2) HMA overlays on PCC pavement ; (3) PCC overlays on asphalt pavement; and (4) PCC overlays on PCC pavement. Our concern for this project is the design of white topping concrete overlay on asphalt pavement. Methods available for designing such an overlay are rather limited and may not necessarily address the subject matter itself. In other words the existing procedure for designing Ultra-Thin

Whitetopping concrete overlay are mostly either empirical or have been adopted from the analysis and design of rigid pavements.

### PCC Overlays On Asphalt Pavements

PCC overlay on asphalt method is being currently practiced all over the United States. It may be cost effective if the asphalt pavement is severely distressed and can be used only as a base layer for the PCC overlay. The design procedure is similar to that of new pavements and uses the existing pavement as the foundation. The finite element plate programs can be used for the mechanistic method of design. To prevent reflection cracking, all cracks of high severity in the existing asphalt pavement should be repaired and sealed. Because the existing asphalt pavement can be considered a non erodible subbase, only fatigue cracking needs to be considered for determining the thickness of overlay required. The design methodologies for the existing procedures are as follows:

### Effective Thickness Approach

The basic concept of this method is that the required thickness of the overlay($h_{OL}$) is the difference between the thickness required for a new pavement( $h_n$ ) and the effective thickness of the existing pavement($h_e$).

$$h_{OL} = h_n - h_e$$

### Deflection Approach

The basic concept of this method is that larger pavement surface deflections imply weaker pavement and subgrade and thus require thicker overlays. The overlay must be thick enough to reduce the deflection to a tolerable amount. Usually only the maximum deflection directly under the load is measured. The deflection method is based on the empirical relationship between pavement deflection and overlay thickness.

**Rigid Overlays On Flexible Pavements**

The design of rigid overlays on flexible pavements is a straight forward problem because the existing flexible pavement can be considered as the composite foundation support for a new rigid pavement. The purpose of the design is simply to determine a composite modulus of subgrade reaction k, so that the design procedure for a new PCC pavements can be used as defined by AASHTO. To evaluate the value of the subgrade modulus (k), a nondestructive testing procedure can be used or it can be estimated from the following Table 2.2 (PCA, 1984):

Table 2.2 k Values for Different Types of Soils

| **Type of Soil** | **Support** | **K value (pci)** |
|---|---|---|
| *Fine-grained soils in which silt* | *Low* | *75-120* |
| *Sand and sand-gravel* | *Medium* | *13-170* |
| *Sands and Sand-gravel* | *High* | *180-220* |
| *Cement treated subbases* | *Very High* | *250-400* |

*Note: 1 pci = 271.3 kN/m*$^3$

## 2.20 AASHTO METHOD FOR RIGID PAVEMENTS

The design procedure is based on the following empirical equations:

$$\log W_{18} = Z_R S_o + 7.35 \log (D+1) - 0.06 + [\log \{\Delta PSI / 4.5\text{-}1.5\} /\{1+ 1.624 \times 10^7/(D + 1)^{8.46} \}] +$$

$$(4.22-0.32\, p_t) \log \{S_c C_d (D^{0.75} - 1.132) / (215.63\, J\, [D^{0.75} - 18.42/(E_c/k)^{0.25}]\,)\}$$

*where:*

$W_{18}$ = allowable 18-kip single-axle load applications for a given reliability

$Z_R$ = normal deviate for a given reliability R

$S_o$ = overall standard deviation in AASHTO design guide

D = the required thickness

$\Delta$PSI = serviceability loss

$p_t$ = terminal serviceability (4.5 - DPSI)

$S_c$ = modulus of rupture of concrete

$C_d$ =drainage factor for rigid pavements

J = load transfer factor for rigid pavements

$E_c$ = elastic modulus of concrete

k = modulus of subgrade reaction

**Illustrative Example 1**

**Given**: k = 72 pci(19.5MN/$m^3$), $E_c = 5x10^6$ psi(34.5GPa), $S_c$ = 650 psi(4.5 Gpa), J = 3.2, $C_d$ = 1.0 , $\Delta$PSI = 4.2-2.5 = 1.7 , R = 95% , $S_o$ = 0.29 and $W_t = 5.1x10^6$.

**Find:** the thickness D of the overlay.

**Solution**: the required thickness D can be determined by the following steps:

1. Starting from figure 2.20 with k = 72 psi (19.5MN/$m^3$) , a series of lines as indicated by arrows are drawn through $E_c = 5x10^6$ psi(34.5GPa) ,$S_c$ = 650 psi(4.5 Gpa),J = 3.2, and $C_d$ = 1.0 until a scale of 74 is obtained at the match line.
2. Starting at 74 on the match line in Fig. 2.21, a line is drawn throgh $\Delta$PSI = 1.7 until it intersects the vertical axis.
3. From the scale with R=95% , a line is drawn through $S_o$ = 0.29and then through $W_{18} = 5.1x10^6$. Until it intersects the horizontal axis.

4. A horizontal line is drawn from the last point in steps 2 and a vertical line from that in step 3. The intersection of these two lines gives a D of 9.75 in. (246 mm)which is rounded to 10 in.(250mm).

**Illustrative Example 2**

Example 2 is the same as Example 1, except that D is given as 9.75 in.(246 mm). Determine $W_{18}$ by using the given design equation.

**Solution**: For R = 95% , $Z_R = -1.645$.

$$\log W_{18}=-1.645\times0.29+7.35\log(9.75+1)-0.06+\log(1.7/2.7)/[1+1.624\times10^{7}/(9.75+1)^{8.46}]$$

$$+(4.22-0.32\times2.5)\log\{[650\times1.0)/215.63\times3.2)][(9.75)^{0.75}-1.132]/[(9.75)^{0.75}$$

$$-18.42/(5\times10^{6}/72)^{0.25}$$

$$= -0.477 + 7.581 - 0.06 - 0.195 - 0.088$$

$$= 6.761$$

$W_{18} = 5.8 \times 10^6$ , which checks as well with the $5.2 \times 10^6$ obtained from the chart.

The load transfer coefficient, **J**, is defined as the factor used in a rigid pavement design to account for the ability of a concrete pavement structure to transfer a load across joints and cracks. The use of load transfer devices and tied concrete shoulders increases the amount of load transfer and decreases the load transfer coefficient. The parameter **J** seems to influencing the design equation significantly. The standard value taken by the FDOT and AASHTO is ***3.2***. In case of UTW the slabs don't have any dowel bars, tie bar reinforcement, or any load transfer components between the slabs except the plastic fibers in the concrete which do not provide any major reinforcement to the concrete matrix except improving the tensile strength and improve the crack initiation and propagation in concrete. Therefore, the current practice of considering the load transfer coefficient should be revised for the UTW applications. The current recommended values for J is presented in the following **Table 2.5**.

**TABLE 2.5** Recommended (J) Values for Various Pavement Types and Design Conditions

| Type Of Shoulder | Asphalt | | Tied PCC | |
|---|---|---|---|---|
| Load Transfer | YES | NO | YES | NO |
| JPCP & JRCP | 3.2 | 3.8 – 4.4 | 2.5 - 3.1 | 3.6 - 4.2 |
| CRCP | 2.9 - 3.2 | N/A | 2.3 - 2.9 | N/A |

JPCP - JOINTED PLAIN CONCRETE PAVEMENT
JRCP- JOINTED REINFORCED CONCRETE PAVEMENT
CRCP - CONTINUOUS REINFORCED CONCRETE PAVEMENT

## 2.21 THE MODIFIED ACPA-AASHTO METHOD

Treating the white-topping as a new PCC pavement, the adopted approach is to use the current FDOT method of rigid pavement design (FDOT Pavement Design Manual 1996). The FDOT Rigid Pavement Manual is an adaptation of the same principles as those in the AASHTO 1993 Design Guide. As mentioned earlier, one of the requirements is to determine the support the existing layers provide to the overlay. Non-destructive testing such as Falling Weight Deflectometer (FWD) can be used to determine the strength of each pavement layer. In lieu of testing, the "foundation support modulus" ($k_{fs}$), as described under the ACPA method, provides a reasonable estimate of support from existing layers (Concrete 1991). The "foundation support modulus" is similar to the effective modulus of the subgrade reaction for several base layers. This estimation is particularly useful where FWD measurements are too costly or simply unavailable.

The foundation support modulus ($k_{fs}$) is estimated using a combination of charts (Concrete 1991).

First, information such as type of soil and its support value ($k$) for the subgrade can be obtained from the original soils report or estimated from the soil borings. Based on the thickness of the exiting subbase (or base) and subgrade $k$ values, an estimate of the effective $k$ value, atop the subbase (or base) is then obtained from the charts. The next step is to use the estimated effective k (atop subbase or base), coupled with the known thickness of the existing asphalt layer, to determine a value for the "foundation support modulus" for the proposed white-topping overlay. The procedure described above is shown in **Figure 2.22**, as part of a representation of the existing approach in the design of white-topping overlays. The estimate of $k_{fs}$ was then substituted as the value of the Modulus of Subgrade Reaction, an input in the AASHTO equation for rigid pavement design. Other parameters of the AASHTO equation include the anticipated traffic loading, properties of the concrete, statistical values, drainage coefficients, and serviceability indices. The output of the equation is the required depth of the concrete overlay.

To enhance an automation of the design procedure, the design charts (Concrete 1991) referenced earlier were reproduced through a selection of various pertinent points on the actual charts. High-order polynomial equations were then tried and formulated to fit the curves on the charts. From the given equations or charts, the current PCA method cannot predict a thickness of a concrete overlay of less than 4in. Therefore, the PCA method may fall short of predicting the suitable size of an UTW overlay.

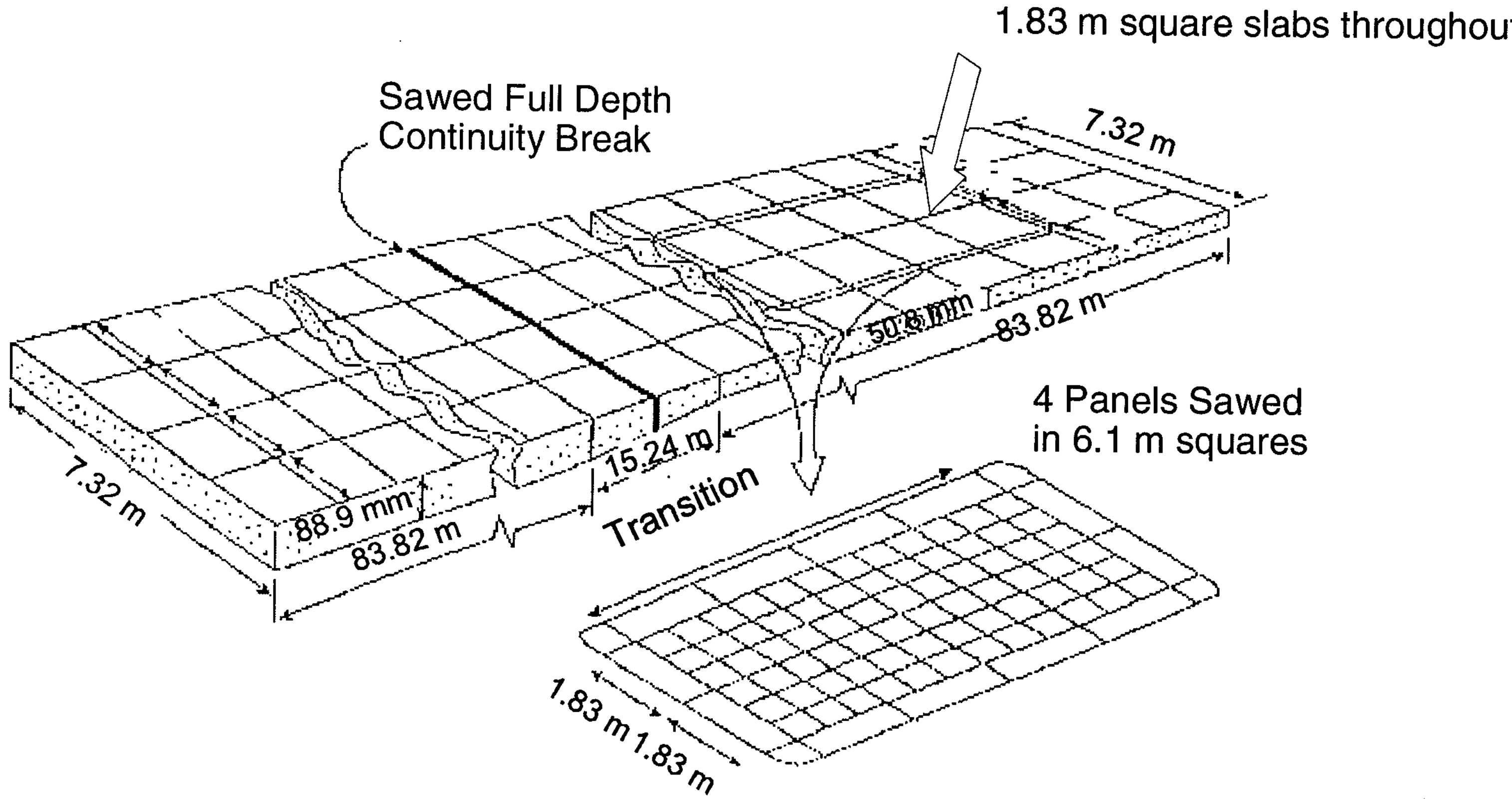



Figure 2.1 Experimental Section Layout

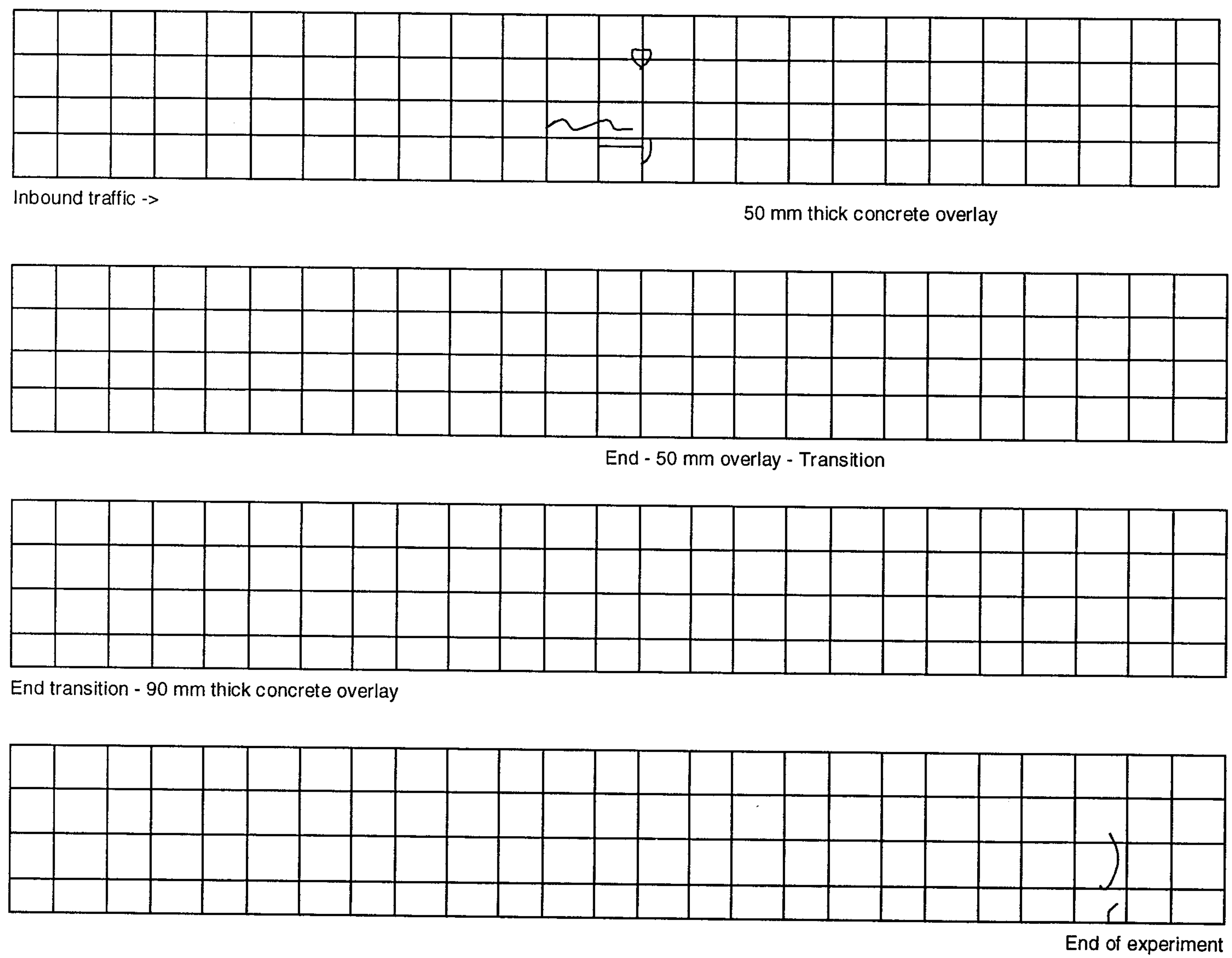



Figure 2.2 Cracking at week 2

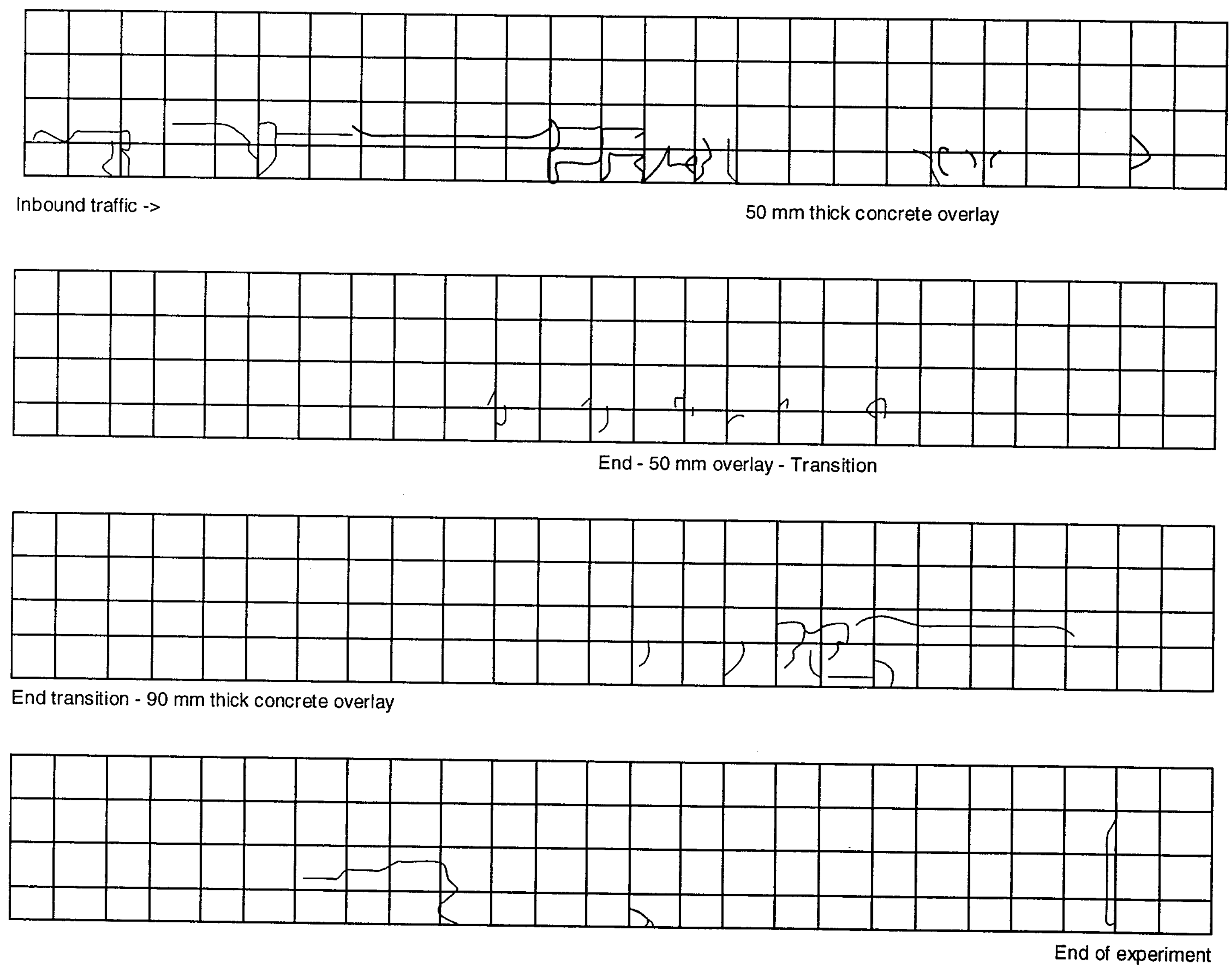



Figure 2.3 Cracking at week 8

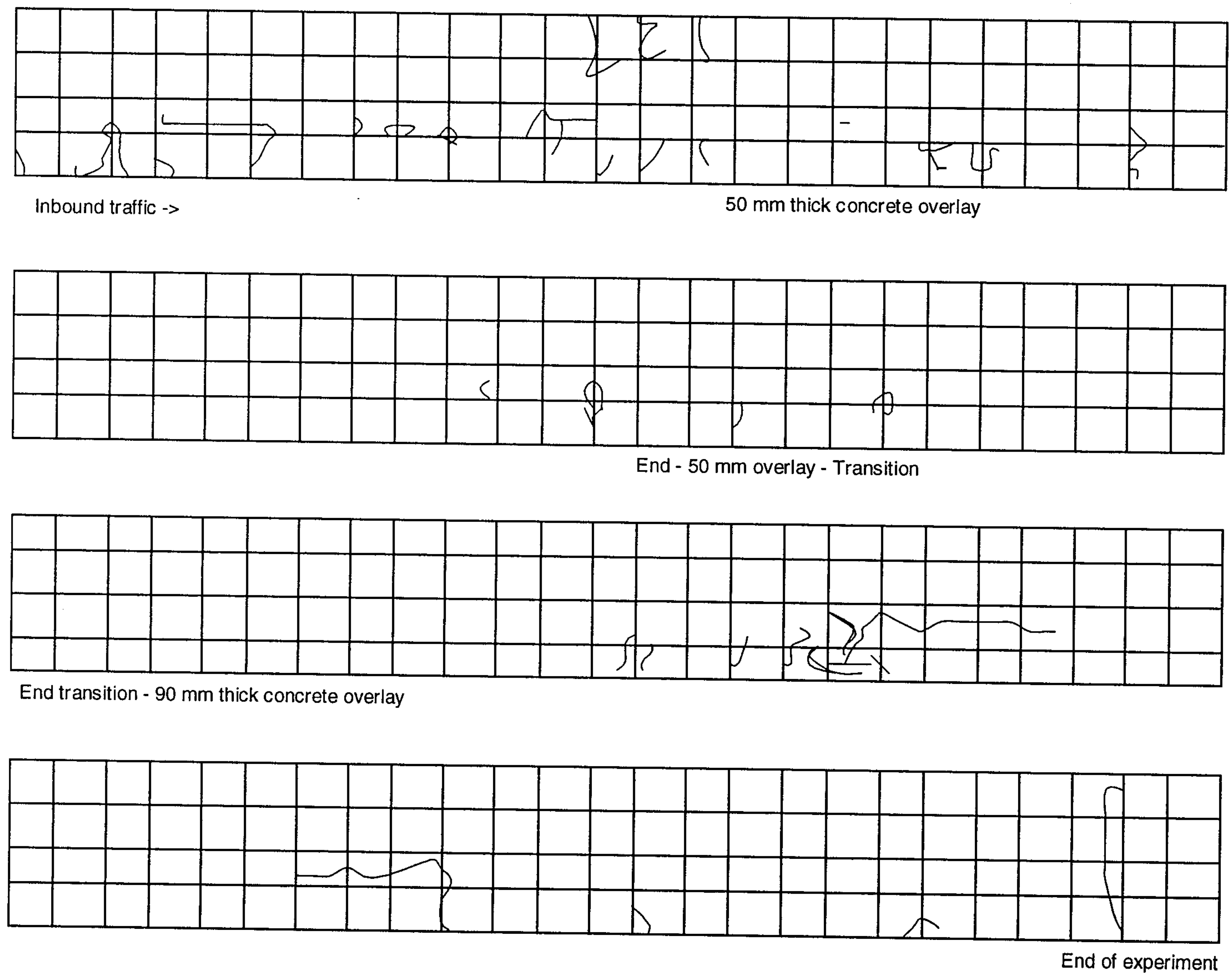



Figure 2.4 Cracking at week 13

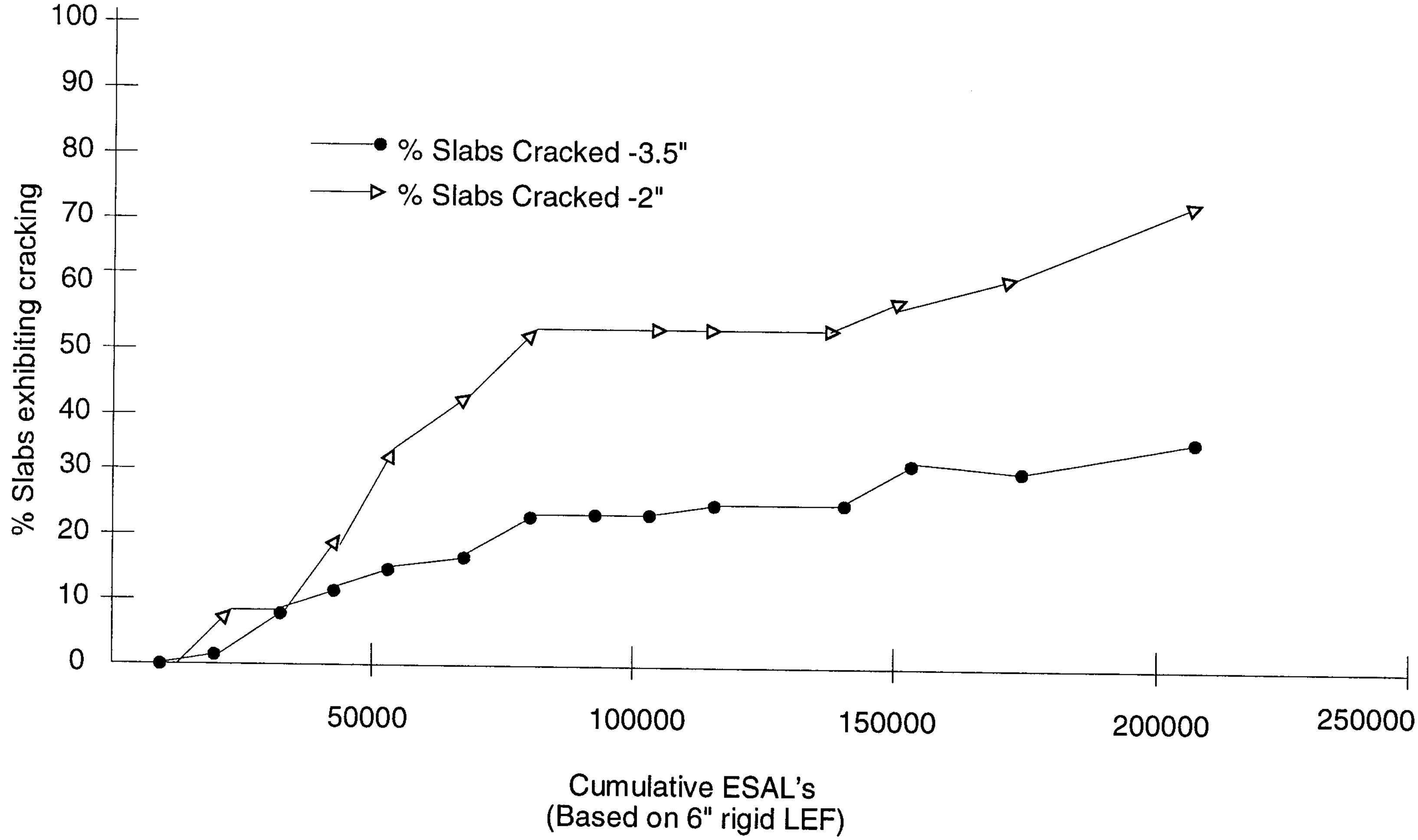
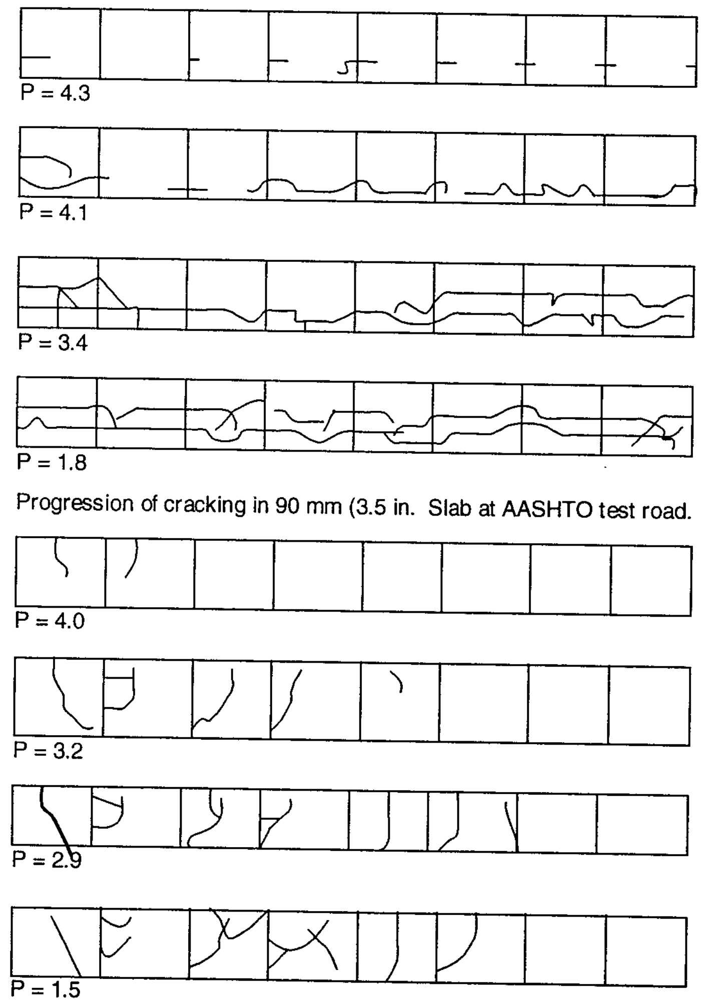



Figure 2.5: ESAL's vs % Slabs Cracked




Progression of cracking in 90 mm (3.5 in. Slab at AASHTO test road.

Progression of cracking in 200 mm (8 in. Slab at AASHTO test road.

Figure 2.6: Progression of Cracking in 90 and 200 mm Slabs at AASHTO Test Road

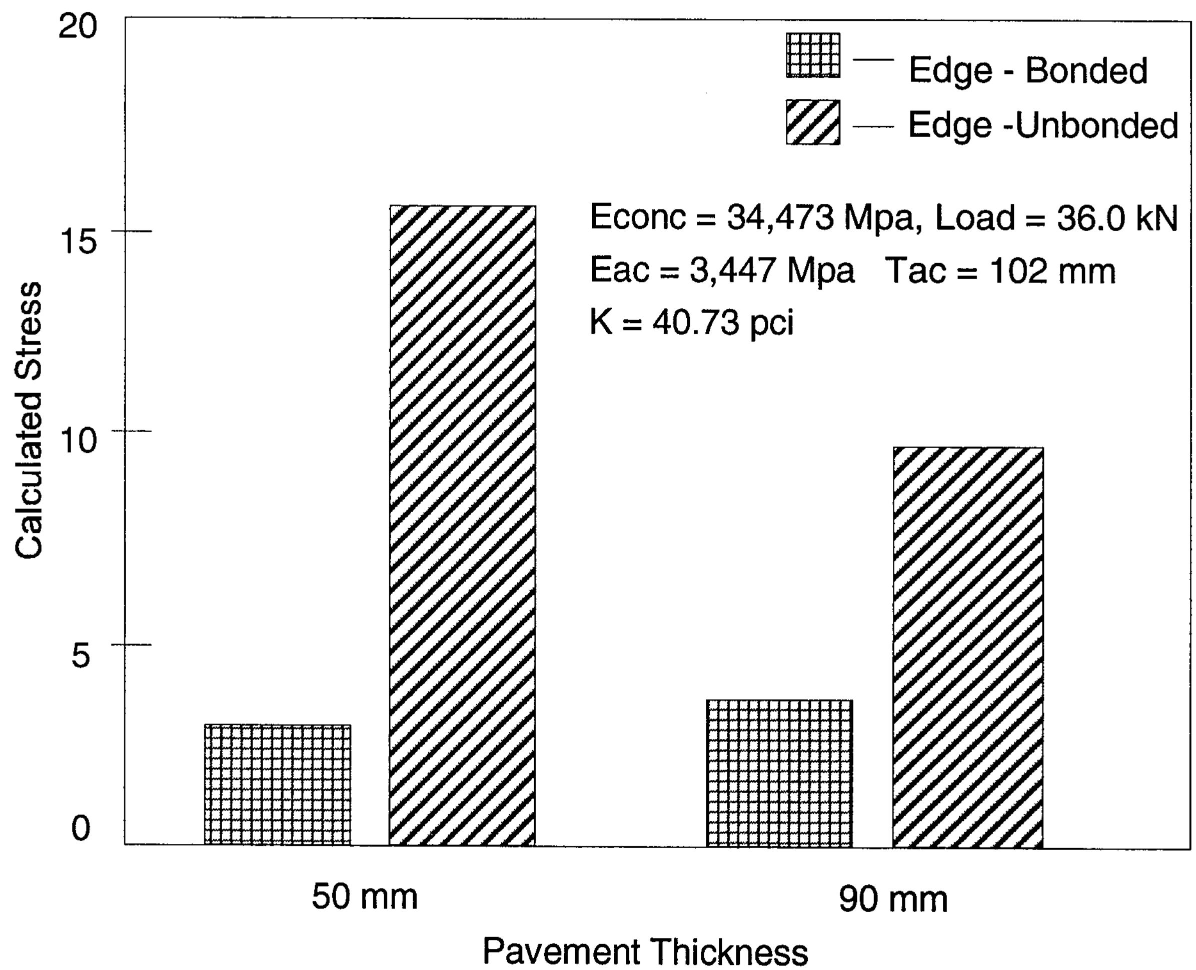



Figure 2.7: Effect of bond on calculated edge stress

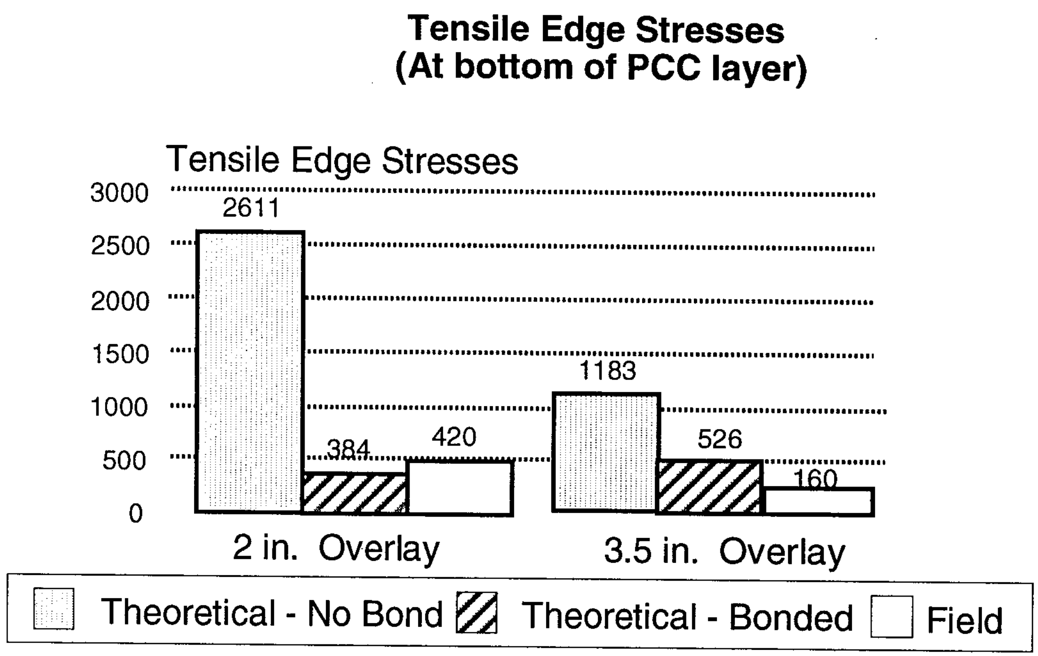



Figure 2.8

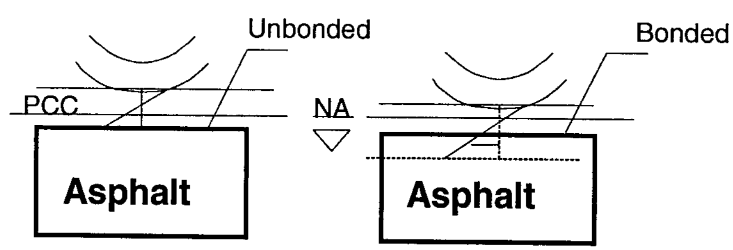



Figure 2.9: Lowering of neutral axis in composite section

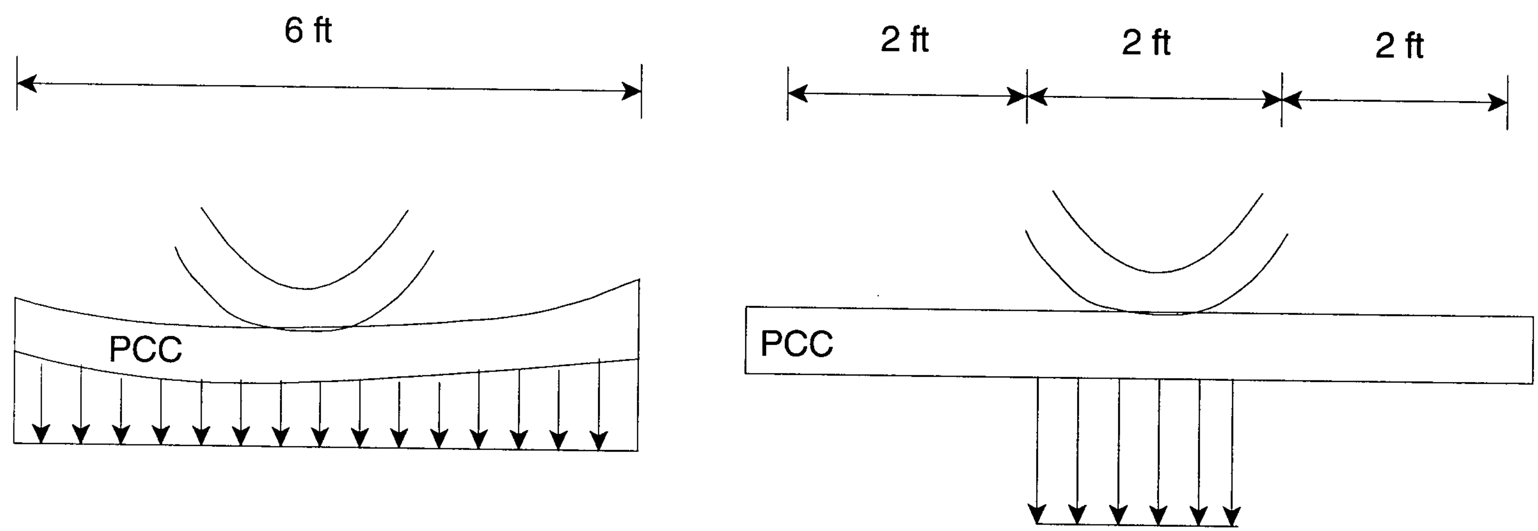



Figure 2.10: Importance of Short Joint Spacing

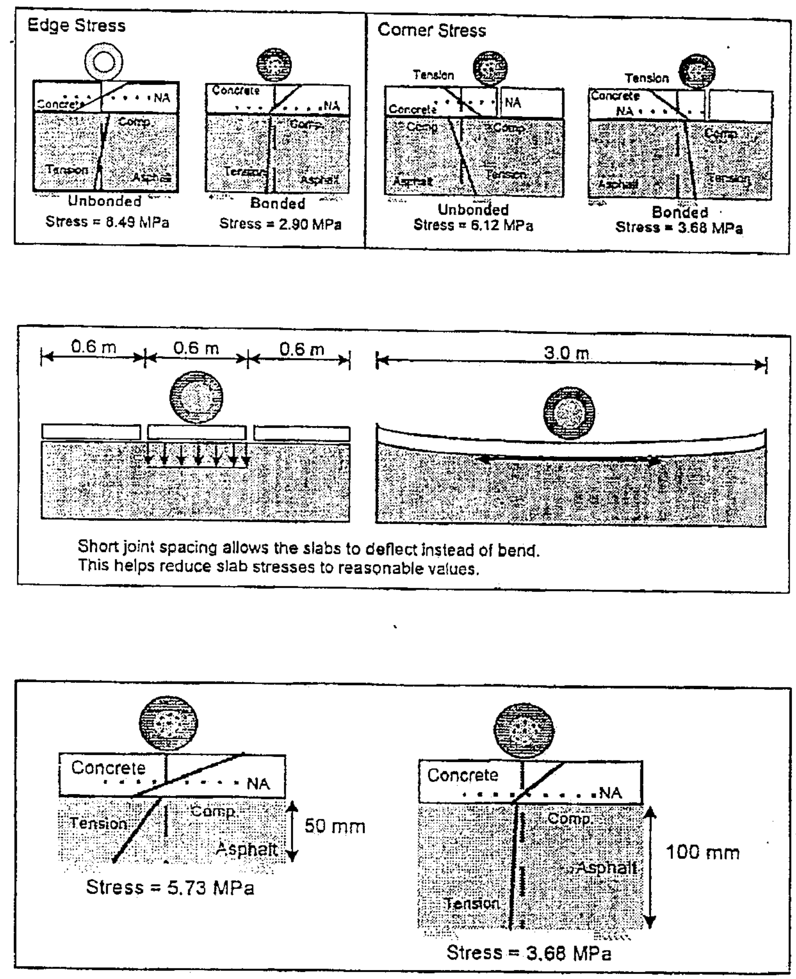
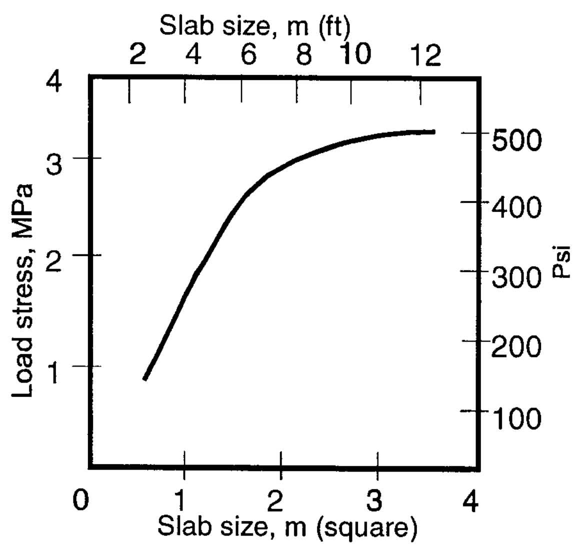
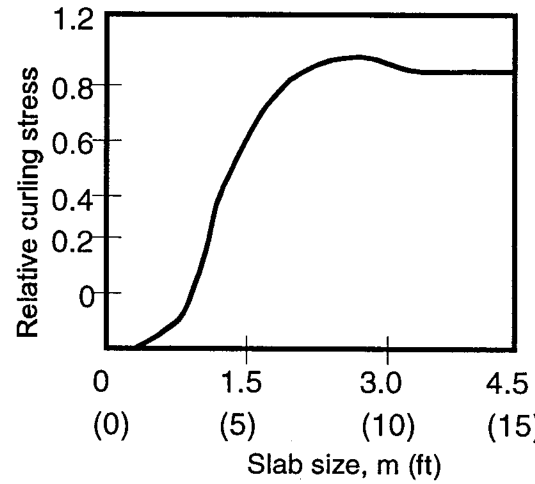



Figure 2.11: Effect of Bonding, Joint Spacing and Slab Thickness




40 kN (9 kip) edge load, 100 mm (4 in.) thick asphalt on k = 27 Mpa(100 pci).




100 mm (4 in.) concrete on 100 mm (4 in.)asphalt on k = 27 Mpa (100 pci)

Figure 2.12: Effect of slab size on load stress.

Figure 2.13: Effect of slab size on curling stress.

TABLE 2.2
NUMBER OF CONCRETE RESURFACINGS BY TYPE AND UNDERLYING PAVEMENT

| TYPE(a) | Underlying Pavement (a) JPCP | JRCP | CRCP | AC/F | OTHER | TOTALS |
|---|---|---|---|---|---|---|
| JPCP | 220 | 44 | 25 | 175 | 12 | 476 |
| JRCP | 88 | 18 | 2 | 14 | 7 | 129 |
| CRCP | 22 | 26 | 2 | 17 | - | 67 |
| FRC | 10 | 2 | 4 | 14 | 2 | 32 |
| PRC | 2 | - | 1 | 1 | - | 4 |
| Totals | 342 | 90 | 34 | 221 | 21 | 703 |

(a) JPCP = jointed plain concrete
JRCP = jointed reinforced concrete
CRCP = continuously reinforced concrete
FRC = fiber reinforced concrete
PRC = prestressed concrete
AC/F = asphalt concrete or flexible

TABLE 2.3
NUMBER OF CONCRETE RESURFACINGS BY TYPE AND INTERFACE

| TYPE(a) | Interface(b) Bonded | U/P Bonded | Whitetopping | Total |
|---|---|---|---|---|
| JPCP | 105 | 218 | 151 | 474 |
| JRCP | 10 | 116 | 6 | 132 |
| CRCP | 3 | 50 | 13 | 66 |
| FRC | 8 | 16 | 8 | 32 |
| PRC | - | 4 | - | 4 |
| Totals | 126 | 404 | 178 | 708 |

(a) JPCP = jointed plain concrete pavement
JRCP = jointed reinforced concrete pavement
CRCP = continuously reinforced concrete pavement
FRC = fiber reinforced concrete
PRC = prestressed concrete pavements

(b) As classified by the reporting agency.

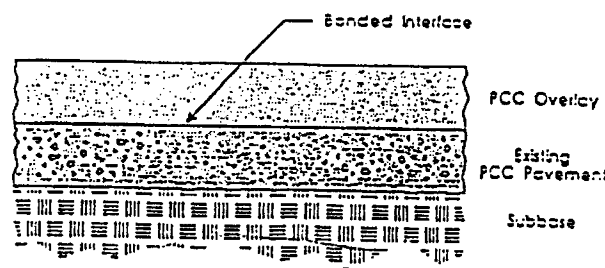



Figure 2.14 Bonded PCC Overlay

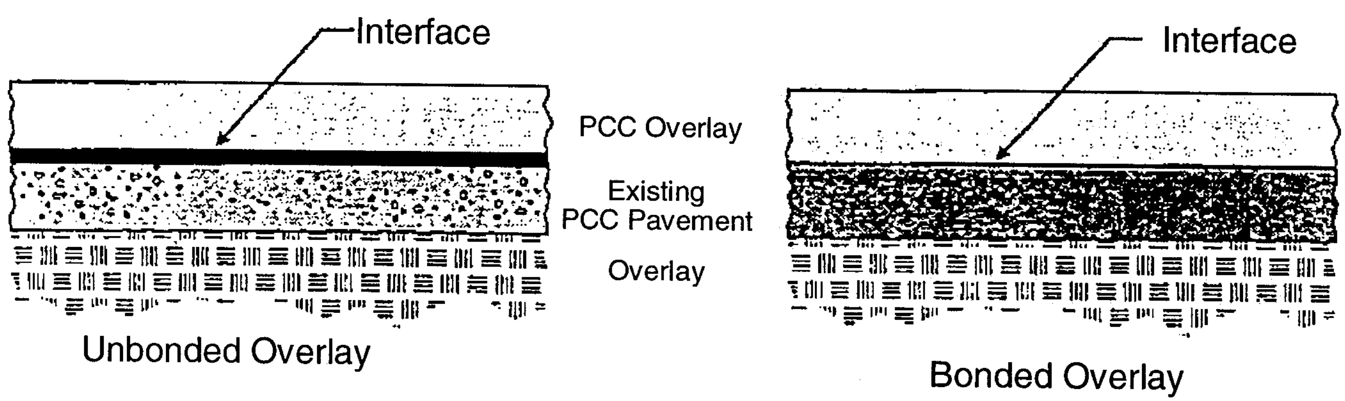



Figure 2.15: Bonded and Unbonded Overlays

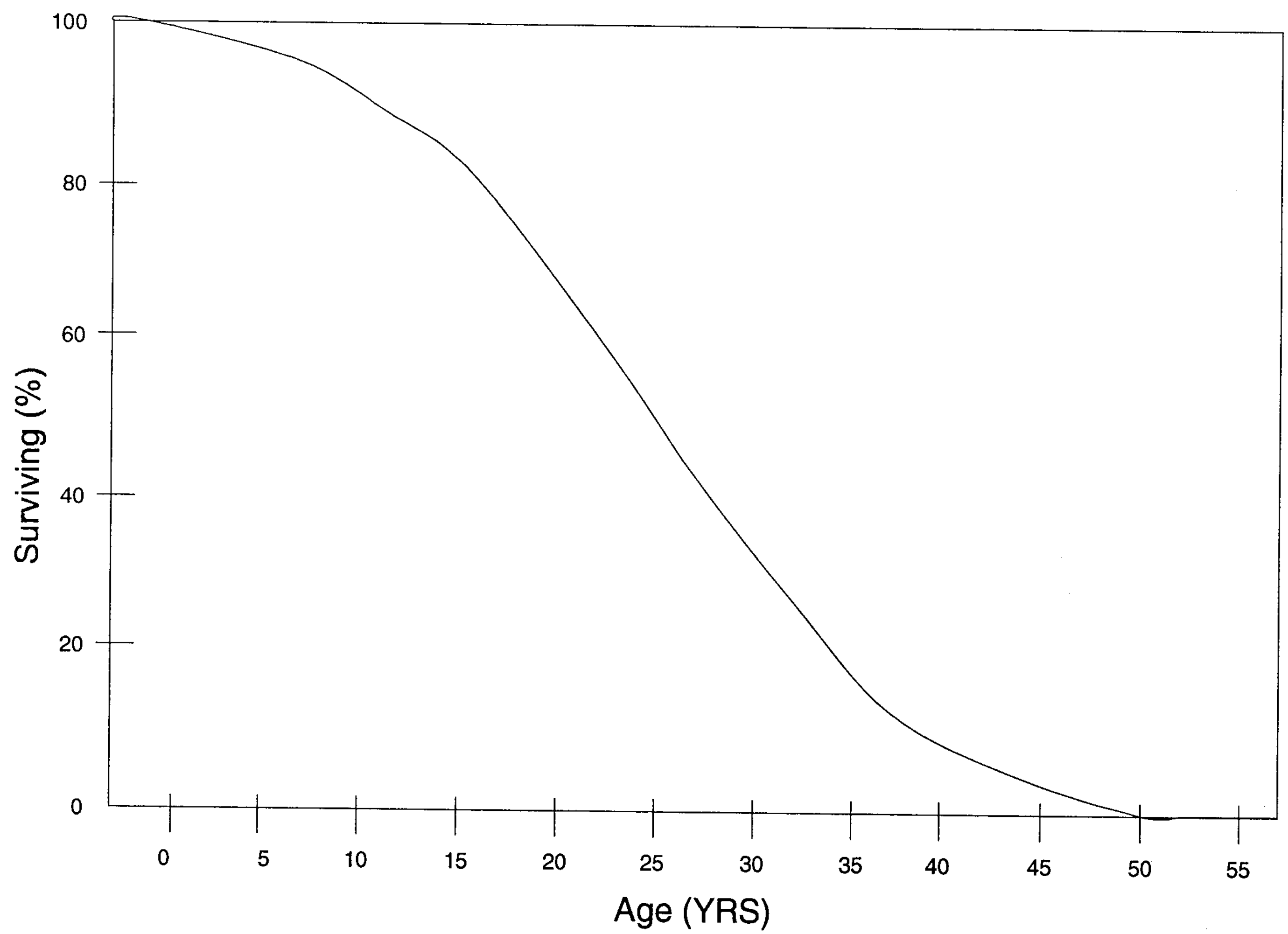



Figure 2.16: Survivor curve for bonded and unbonded overlays

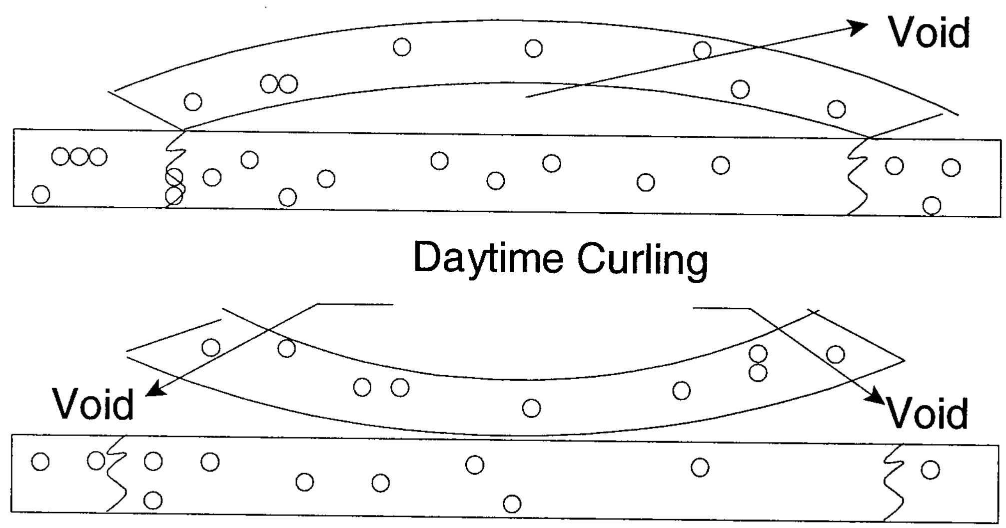



Figure 2.17: Curling of Slabs

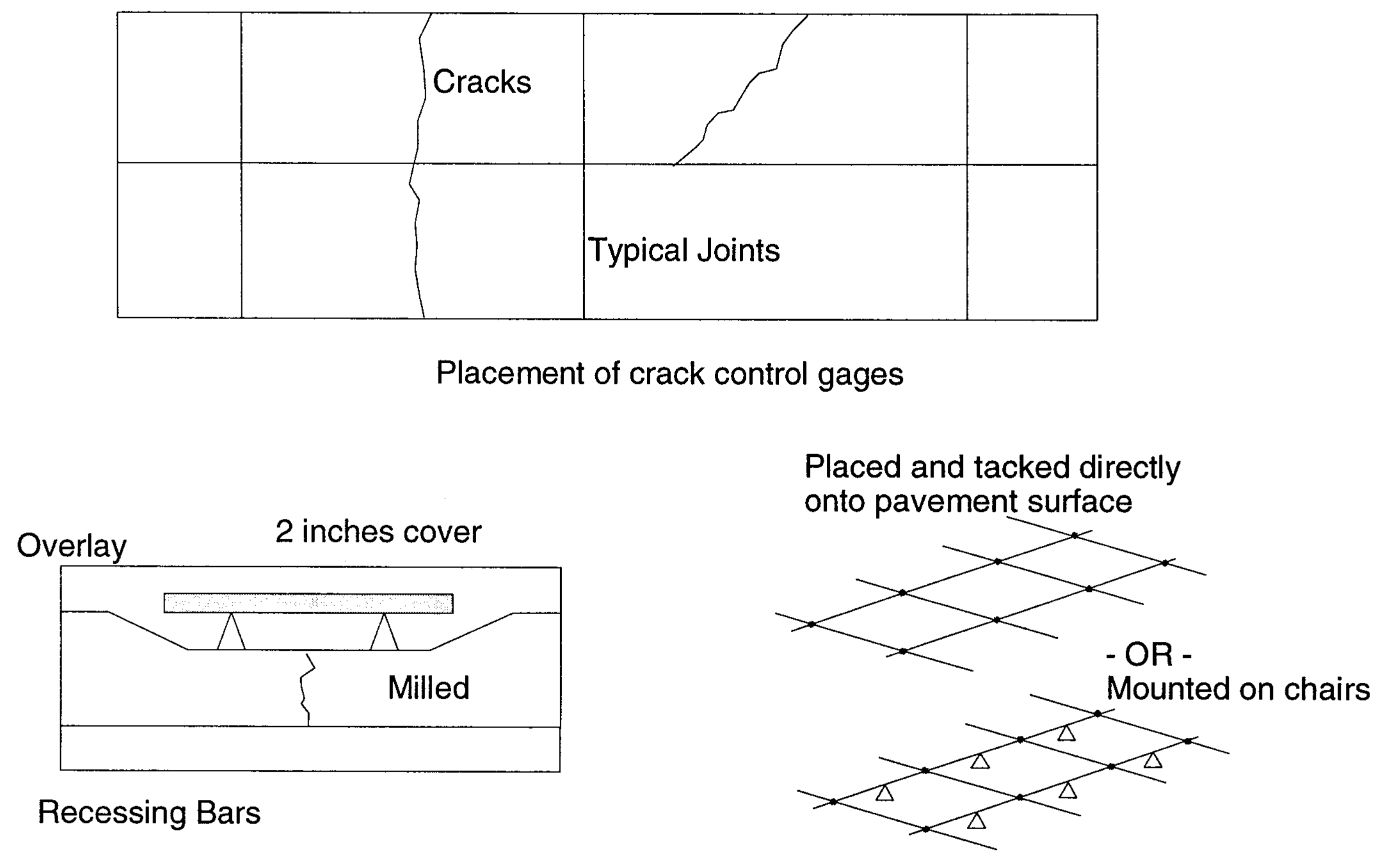
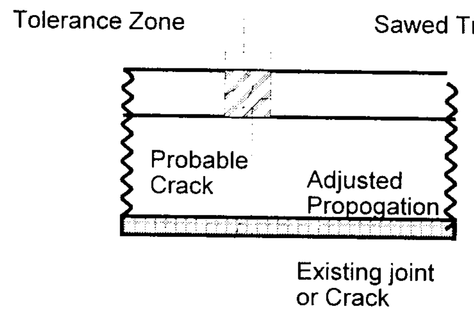
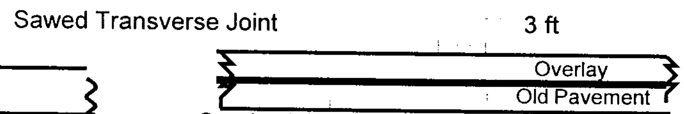


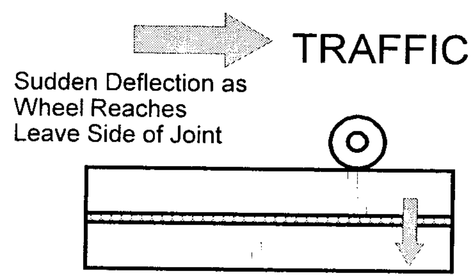

Figure 2.18: Details of Random Crack Control




Crack adjustment in thin overlays (9)




Overlay joint mismatched from existing joint to provide a sleeper-slab arrangement and improved load transfer (25)

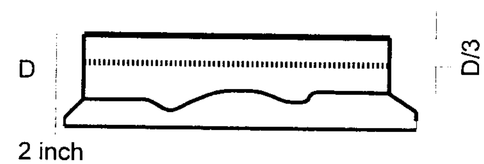



Puming Action

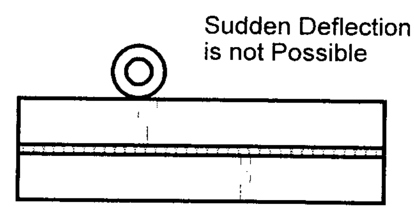
**Joint sawing recommendations, Bonded Ovelays (9, 10)**

| Joint Type | Depth <= 4" | Depth > 4" |
|---|---|---|
| Transverse | D + 1/2" | 1/3 D |
| Longitudinal | ½ D | ½ D |
| Expansion | D + 1/2" | D + 1/2" |

D = Nominal Overlay Depth

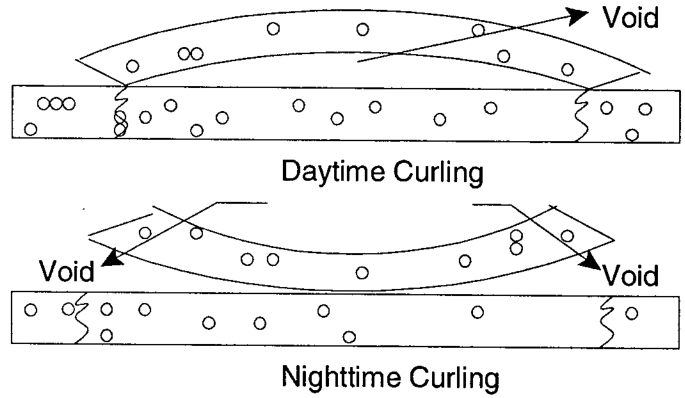



Consideration should be given to increasing the depth of sawing where distortions exceed 2 in. (54).




Placing the overlay joint on the approach side of the existing joint reduces sudden deflections through both the overlay and underlying slabs. This inhibits the typical pumping action as loads reach the leave side of the overlay joint (25).

Figure 2.19



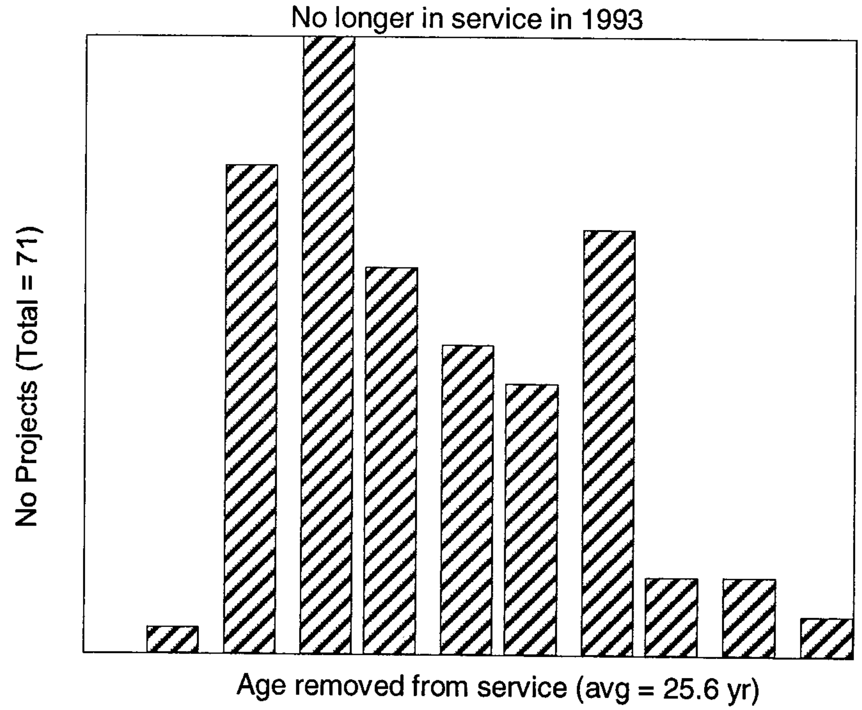
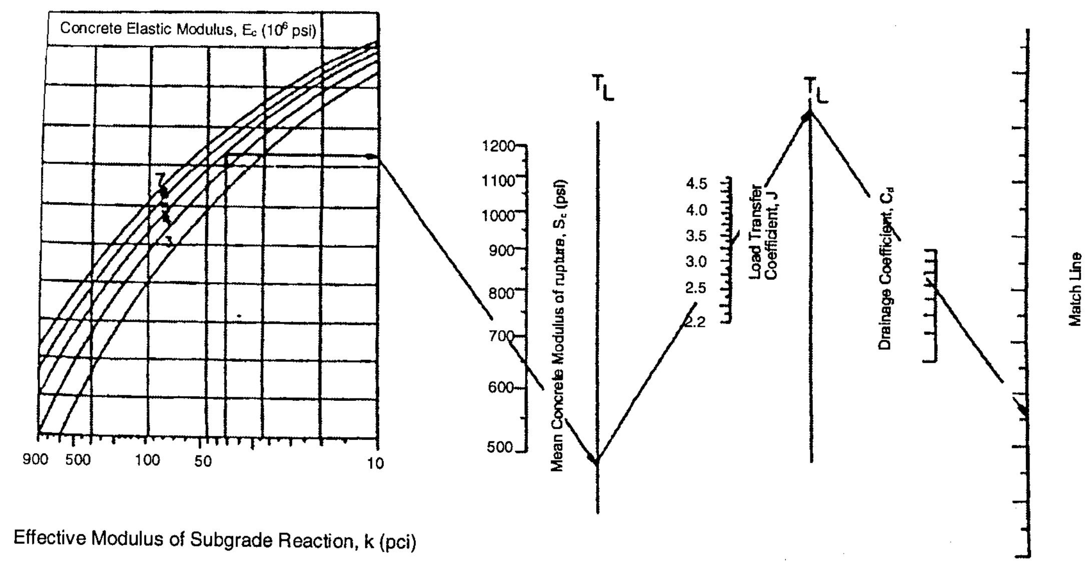
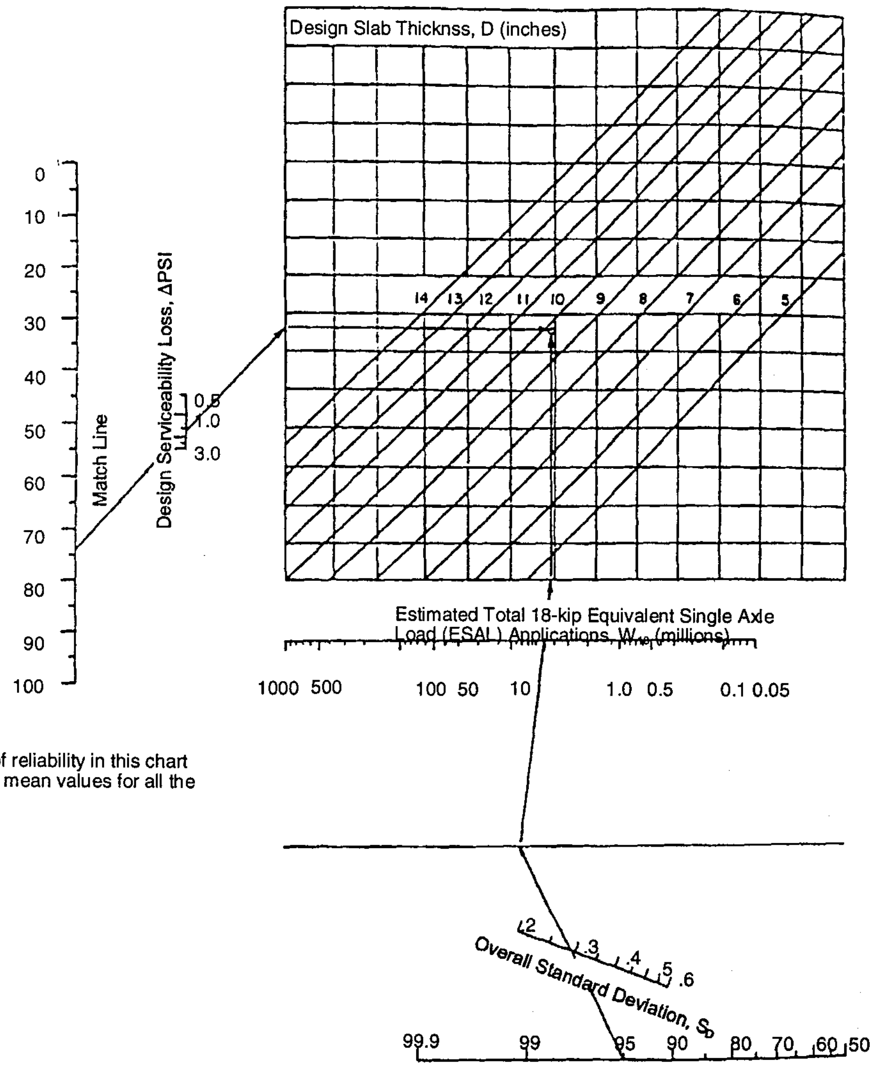




Figure 20
Ages at which unbonded and partially bonded overlays have been removed from srevice




Based on mean values (1 in. = 25.4 mm, 1 psi = 6.9 kpa, 1 pci = 271.3 kN/m$^3$).
(From the AASHTO Guide for design of pavement structures)

Figure 2.20: AASHTO Design Chart for Rigid Pavements




Note: Application of reliability in this chart requires the use of mean values for all the input variables

Figure 2.21: AASHTO Chart for Pavement Thickness Design

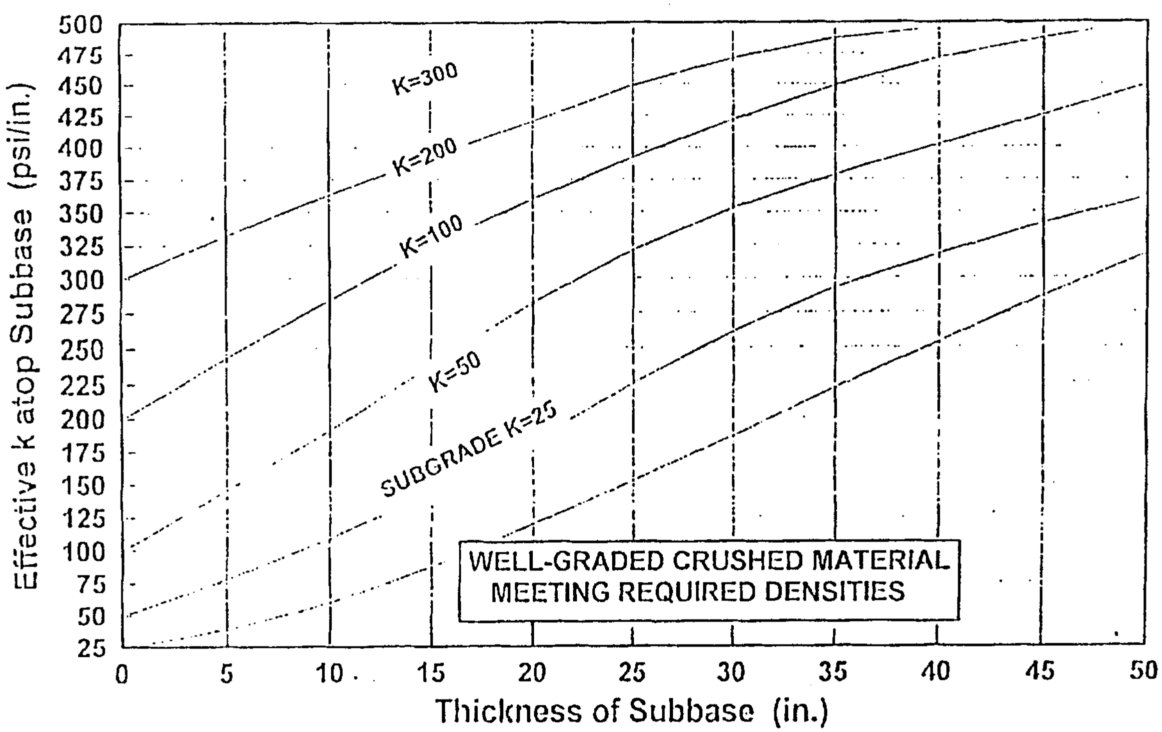
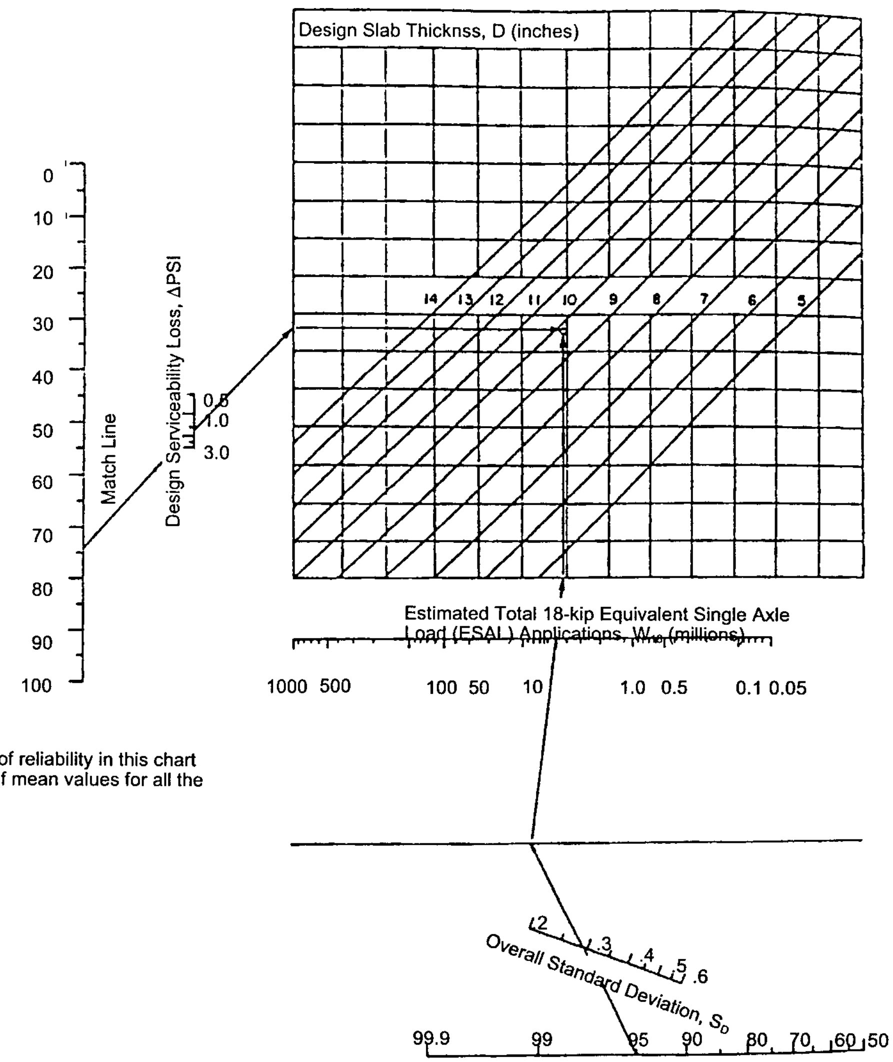



Figure 2.22 Chart for determining the effective k atop well graded crushed stone base materials




Note: Application of reliability in this chart requires the use of mean values for all the input variables

Figure 2.21: AASHTO Chart for Pavement Thickness Design

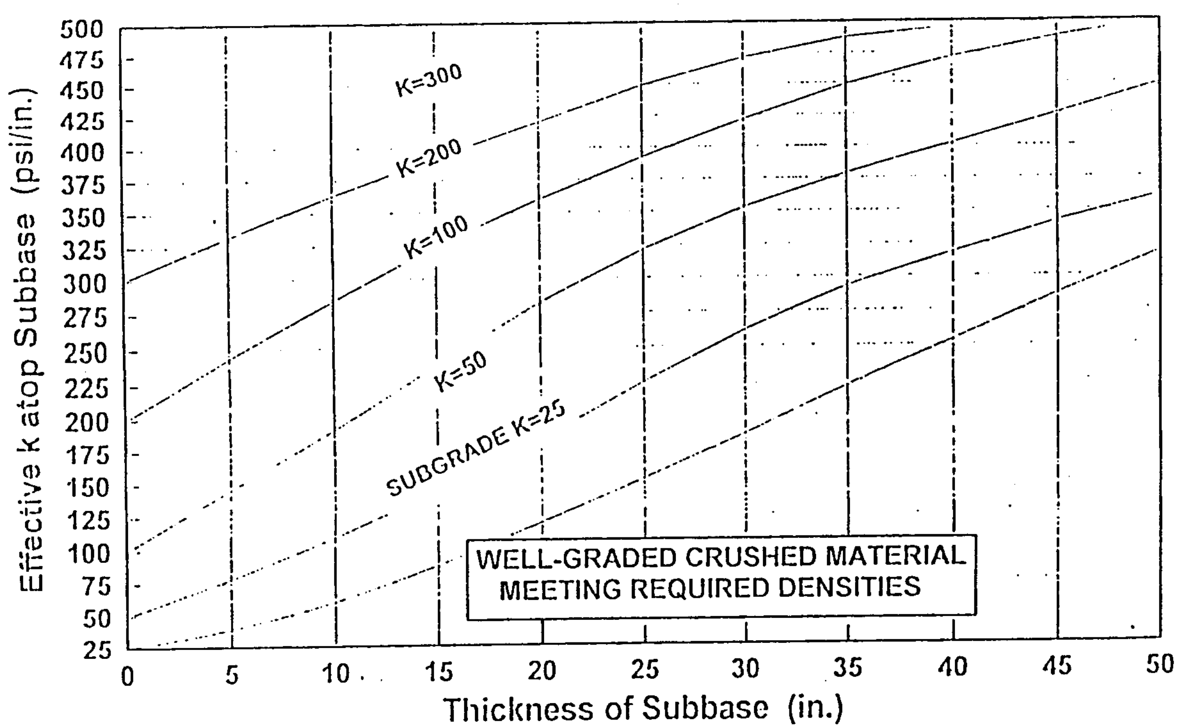



Figure 2.22 Chart for determining the effective k atop well graded crushed stone base materials

Table 2.1 Number of Concrete Resurfacings By Type and Use Done Till 1995

| Type | Use | | | Total |
|---|---|---|---|---|
| | Highways | Streets | Airfields | |
| JPCP | 319 | 38 | 119 | 476 |
| JRCP | 99 | 24 | 6 | 129 |
| CRCP | 57 | 1 | 9 | 67 |
| FRC | 6 | 8 | 18 | 32 |
| PRC | 2 | - | 2 | 4 |
| Totals | 483 | 71 | 154 | 708 |

# CHAPTER 3

# FIELD TESTING

## 3.1 INTRODUCTION

Three UTW test tracks were constructed in Gainesville, Florida for the purpose of testing the distribution of longitudinal/transverse stresses, and vertical deflections. Fatigue cracking was monitored through cycles of load application that were applied by using an FDOT truck. Additional nondestructive testing was performed on these tracks using FWD, and bonding characteristics were determined using the Iowa shear test. Among the three test tracks, one track was constructed on a 4 in. existing concrete pavement overlaid by an asphalt layer, thus the stress measurements were very low as compared with the UTW tracks on an asphalt layer only. Four types of gauges were used in testing. These gauges were for strain as well as deflection measurements. The most efficient gauges used in this phase were the displacement gauges. The reason for designating them as the most efficient was because they can be mounted on the surface immediately before applying the loads and can be removed for other uses afterward. This could keep their stability and could avoid their damage due environmental conditions changes. Measurements from the field-testing were used to compare the FE models of the same test tracks.

## 3.2 GAINESVILLE TEST TRACKS

### Design Concept

Two factors, which have a significant effect on the long-term performance of an UTW, are bonding at the concrete-asphalt interface and the condition of the structural support of the subbase layer beneath the asphalt. For the purpose of consistency, the asphalt layer beneath the UTW will be referred to as the base and the original base beneath the asphalt layer will be referred to as the subbase. The UTW can be successful only if the subbase is not the source of the distress in the original asphalt pavement. Thicker concrete overlays should be considered if the subbase has exhibited poor structural support by contributing to the deformation and or cracking of the original asphalt pavement.

Bonding between the concrete overlay and the asphalt base is essential for long term performance of the UTW. Without adequate bonding, the UTW layer will respond independently to the applied loads and will likely develop excessive stresses to the point of failure. Also, temperature curling will tend to lift the corners of the concrete panels (small slabs), giving rise to tensile stresses and expediting the cracking of the UTW. Proper bonding of concrete-asphalt layers can resist curling forces responsible for lifting of panel edges and corners. The key to a successful design is to ensure bonding by proper surface treatment of the asphalt base. Effective surface preparation for asphalt includes milling and/or thorough cleaning. Milling will remove ruts, oil residue and other surface distortions, and will provide a rough surface texture, which ensures adequate bonding. Thorough cleaning will be effective in cases of pavements that already have rough surface textures (e.g. friction courses) and have not developed ruts deeper than 1/8 in. Another surface treatment includes application of a Crack Relief Layer (CRL) which consists of a thin coat of asphalt and aggregates to prevent reflective cracking, a common practice in Florida when resurfacing asphalt pavement. The CRL helps in providing a rough surface texture to allow a good bonding with the concrete overlay. Joint spacing plays an important role in the performance of the UTW. Design of short joint spacing will minimize curling and significantly reduce curling stresses. A combination of short joint spacing and adequate bond will create a composite-monolithic layer of concrete and asphalt which will posses greater load carrying capacity. It has been suggested that under such conditions the concrete layer will likely be permanently in compression. This hypothesis is based on the fact that a lower neutral axis would be developed due to this monolithic effect from which the concrete portion is mostly under compression.

**Design Details**

Plan and cross-section views of Test Tracks 1,2 and 3 are shown in Figures 3.1 to 3.6 and respectively. Test Track 1 is 18.29m. (60ft.) long, including two 1.83m. (6ft.) ramps and consists of 100mm. (4in.) thick concrete overlay. It includes a combination of 1.22m. x 1.22m. (4ft. x 4ft.) and 1.83m. x 1.83m. (6ft. X 6ft.) joint spacing (panels). Track 3 is 43.89m. (144-ft.) long including two 1.83m. (6ft.) ramps. This track consists of two sections with overlay thicknesses of 75 and 100 mm. (3 and 4 m.). The joint spacing patterns in each subsection include 1.22 m. x 1.22 m. (4 ft. x 4 ft.) and 1.83 m. x 1.83 m. (6ft. x 6 ft.). A 3.66m. x 3.66m. (12ft. x 12ft.) transition slab was constructed between the 75mm. and 100mm. (3in. and 4 in.) sections. Test Track 2 is 14.63-m. (48ft.) long, including two 1.83-m. (6ft.) ramps. The concrete overlay is 50mm. (2in.) thick and includes 0.92 m. x 0.92 m., 1.22 m. x 1.22 m. and 3.66m. x 3.66m. (3 ft. x 3 ft., 4ft. x 4ft. and 12 ft. x 12 ft.) panels.

The test tracks were constructed in a maintenance yard located behind the FDOT State Materials Office. In the area where Track I was constructed, the asphalt covers an old 150-mm. (6-in.) concrete pavement which used to be the floor for aggregate bins. A nominal 25mm. (1in.) asphalt base and a 150mm. (6in.) concrete subbase support the UTW in Track 1. In the area of Tracks 2 and 3 the pavement has a nominal 38mm. (1.5in.) asphalt thickness (the thickness may actually vary from 25 to 50 mm.). The subbase beneath the asphalt pavement in areas where Tracks 2 and 3 were constructed consists of nominal 163-mm. (6.5-in.) stabilized layer of Florida limerock and sand. The limerock is soft limestone commonly used as base material beneath asphalt pavement. Tracks 2 and 3 are also supported by 38 mm. (1.5-in.) asphalt base but with a stabilized subbase of sand and limerock. The stabilized subbase in Tracks 2 and 3 is substantially weaker than the concrete subbase in Track 1.

The asphalt and concrete layers beneath Track 1 represent a strong support system comparable, if not exceeding, the stiffness of most support systems beneath pavements in south Florida. Test Tracks 2 and 3 are being supported by a weaker subbase. This subbase is not considered standard for Florida roadways.

Prior to overlay, the surface of the asphalt pavement was prepared using several treatments. In Track 1, three treatments were implemented as shown in Figure 3.1. The surface treatments included a 10.97 m. (36 ft.) long section of asphalt CRL, a 3.66m. (12 ft.) section of broom - cleaned surface without further treatment and a 3.66 m. (12 ft.) section of milled and broom-cleaned surface. The asphalt surface beneath Track 3 was divided into three sections. Two 10.97m. (36 ft.) sections at both ends of the track were milled and the remaining 18.29 m. (60 ft.) section in the middle was broom-cleaned and left without further treatment. The entire asphalt surface in **Track 2** was milled and broom-cleaned to ensure adequate bond, considering the extremely thin 50. mm (2 in.) concrete overlay as shown in Figure 3.2.

### Concrete Mixture Design

During the planning phase, several trial mixtures were batched including plain concrete and concrete with fiber. It was decided to design High Early Strength (HES) concrete mixtures to allow Fast Track paving. Criteria selected for the concrete mixture included the use of fibers, a low water-cement ratio (W/C) and good work ability for fiber concrete by using the High Range Water Reducers (HRWR), if necessary. The HES concrete allows early opening of pavements to traffic. Maintaining a workable mixture assures successful placement and finishing of the UTW. The mixture designs for the three test tracks are shown in Table 3.1.

Cement type I and II were used with and without fly ash. Three aggregate sizes were included. Track 1 used 25 mm. (1 in.) nominal size aggregate. Track 2 used 19 mm. (3/4 in.) nominal size aggregate. For Track 3, a 9 mm. (3/8 in.) size aggregate was used because of 50-mm. (2-in.) thick overlay. The WIC for Tracks 1, 2, and 3 were 0.38, 0.38 and 0.39, respectively. High dosage rate of HRWR was used to overcome the loss of work ability from the addition of fibers. Fibrillated polypropylene fibers (50 mm. long) were used in the mixtures for Tracks 1 and 2 at a rate of 1.8 kg/m$^3$ (3 lb/yd$^3$) of concrete. Polyolefin fibers were used at a dosage rate of 11.86 kg/m$^3$ (20 lb/yd$^3$) for Track 3. The plastic properties of three concrete mixtures were tested at the construction site and are shown in Table 3.2. The ambient temperatures were similar for the three tracks. Slump values represent the concrete prior to the addition of the HRWR and fibers. It should be noted that the slump values represent the concrete work ability prior to the addition of the HRWR.

### Test Track Construction

The concrete was delivered in truck mixers. Prior to concrete placement, all asphalt surfaces were sprayed with water to keep the surface in a damp condition during concrete placement. In each track, a control section was placed using plain concrete without the addition of fibers. The remaining section was placed using fiber concrete. The concrete was distributed and compacted in the form using immersion vibrators and vibratory screed.

After placement, the concrete surface was broom-finished in Track 1 and turf-finished in Tracks 2 and 3. White water pigment compound was used for initial curing. Two to three hours after concrete placement, the concrete was sawed to form the joint grooves. An early saw cutting machine was used. This machine uses a special saw blade, which cuts through relatively fresh concrete without the need for water. The depth of the joint groves was 1/3 of the overlay thickness. The width of the saw cut was 3 mm. (1/8 in.). The joints could have been cut sooner; however, there was a delay in the setting time of the concrete as a result of the use of large dosages of HRWR to achieve proper work ability for construction. The compressive strength was tested at the time of saw cutting the joint grooves. The strength of concrete was 5.5 MPa (800 psi). At this strength level, the concrete can carry the load of the saw machine and will allow cutting the joint groves without raveling or distortion of the concrete surface. The joint patterns for the three tracks are shown in **Figures 3.1 to 3.6**. After all the joints were cut, the pavement surface for each track was covered with plastic sheets, kept wet using soaker hoses and allowed to cure for three days.

**Hardened Concrete Test Results**

Concrete samples obtained during placement of the three tracks were tested for compressive, tensile and flexural strengths. The modulus of elasticity (E) for each concrete sample was also measured. Table 3.3 shows average values of at least three test results obtained at different stages. It should be noted that all test samples were fabricated from concrete containing polypropylene or polyolefin fibers.

The compressive strength was tested eight hours after placement in order to evaluate strength development of the HES concrete. The compressive strength was also tested at 24 hours as well as 3.7, 28, and 91 days. It should be noted that samples were tested at the time of saw cutting the concrete to determine strength of concrete at this event.

The 24-hr strength for Tracks 1 and 2 exceeded 21 MPa (3000 psi). For Track 2, the 24-hr strength reached only 14 Mpa (2000 psi). This can be attributed to a relatively large dosage of the HRWR, which delayed concrete setting and strength development. However, at 28 days the strength of concrete in Tracks 1 and 2 were very close, as shown in Table 3.3.

The split tensile and flexural strengths, and modulus of elasticity samples were tested at 28 days. The flexural strength at 28 days is very high. The fibers may have contributed to the high flexural strength, and may also have contributed to the variability of these results as shown in **Table 3.3.**

Shear tests were performed according to the Iowa shear test method. During the 18- month testing period, more than 30 core samples were obtained from different sections, thicknesses and surface treatments. Table 3.4 shows an average shear strength representing the three asphalt surface treatments. There were failures (layer separation) in some samples during the coring operation. However 25 samples were extracted intact and were subsequently tested. The test results indicated that the milled and broom-cleaned surfaces develop a strong bond with the UTW. Based on these test results and excellent performance after 60,000 (1 8-kip) ESAL, a shearing strength of 1.4 MPa (200 psi) is recommended for quality assurance purposes on Florida UTW construction projects. Also, it would not be recommended to use the CRL as a surface treatment, due to poor shearing strength results.

**Pavement Field Testing**

Loading the tracks began seven days after construction. The most frequently used loading truck weighed 55,700 pounds. Over a loading period of 18 months, the test tracks were subjected to more than 60,000 (80 kN.) ESAL . The number of ESAL is relatively low when compared to in-service roadways. Subjecting the test tracks to 60,000 (18-kip) ESAL in the confined area of the experimental project is a major accomplishment. Truck loading simulated loading conditions on actual highways. This allowed true evaluation of the structural capacity of the test tracks and provided the opportunity to observe any changes in the condition of the pavement throughout the loading period.

**Falling Weight Deflectometer Tests**

The existing asphalt pavement was tested using the FWD and Dynaflect prior to placement of UTW (Figure 3.7). A standard spacing of 300 mm was used between the sensors for the FWD testing. Immediately after construction and prior to loading, FWD tests were performed on the loading surface of the UTW at approximately the same positions as those tested on the existing asphalt surface in order to determine the level of improvement in the load carrying capacity of the pavement after placement of UTW.

Figure 3.7 shows the FWD deflection basins for the three test tracks prior to and after the placement of UTW. The deflections in Figure 3.7 correspond to the FWD dynamic load of 550 kPa. (80 psi).

Significant improvement can be observed in the structural capacity of the pavement as a result of the concrete overlay. The values of the deflections at the six FWD sensors are also presented in **Table 3.5**. The table also shows the percent reduction in deflection after placement of the UTW. Observations should that the 100-mm. (4-in.) sections in Track 1 and 3 produced the greatest reduction in maximum deflection (78 percent), indicating significant improvement in the structural capacity of the pavement system. The 50-mm. (2-in.) section in Track 2 produced the least percent reduction (44 percent) in the maximum deflection. However, when the maximum deflection and the deflection basin were compared with deflections on conventional 225-mm. (9-in.) slab 3.66 m. X 6.10 m. (12 ft. X 20 ft.), the structural capacity of the two pavement systems is very comparable.

During the loading period, the three tracks were tested four times using the FWD to evaluate their structural capacity. Deflections were measured at three FWD load levels: 550 kPa., 750 kPa. and 950 kPa. The tests were performed during the early morning hours (AM) and in the mid-afternoon (PM) to determine the extent of curling and evaluate the condition of the bonding between the UTW and the asphalt base. Significant change in the deflections between early morning and afternoon would suggest loss of bond, which would allow large curling in the concrete panels. Also, a significant increase in deflections from one FWD test date to another would indicate degradation in the structural capacity of the pavement system as a result of increased loading cycles. Deflection values of FWD testing that corresponds to a 550 kPa. load are given in Tables 3.6 **and** 3.7. The deflections in each FWD test cycle are generally low. The change in deflections from one test series to another was not significant enough to suggest deterioration in the structural capacity despite the continuation of the truck loading. With few exceptions, deflections obtained in either AM or PM were only slightly different, indicating good bond and minimum, if any, curling of the concrete panels. A mechanistic model may provide a better tool for analysis of stresses induced by truck and FWD loading.

**Condition Monitoring**

After 60,000 (80 kN) ESAL, only one crack was detected in a 1.22 m. x 1.22 m. (4 ft. x 4 ft.) panel in Track 1. This panel was subsequently replaced. In track 2, the 3.66 m. x 3.66 m. (12 ft. x 12 ft.) slab cracked into four 1.83 m. x 1.83 m. (6 ft. x 6 ft.) panels. This was expected for a 50 mm. (2 in.) overlay. This cracking pattern may also suggest that 1.83 m. (6 ft.) joint spacing should be used as the maximum joint spacing for the UTW. A few corners of the 1.22 m. x 1.22 m. (4 ft. x 4 ft.) panels were slightly chipped. The corner chipping may have been caused by the dislodgment of an aggregate particle during early saw cutting of the joint grooves. Also, these corners happened to be in the wheel path of the loading trucks. A possible solution is to increase the joint spacing to 1.52 m. x 1.52 m. (5 ft. x 5 ft.) or 1.83 m. x 1.83 m. (6 ft. x 6 ft.) to move the corners away from the wheel path.

Sections using fiber concrete performed equally well as those using plain concrete. It was not possible to establish the impact of fibers in concrete on the performance of the UTW. More load repetitions may be needed to determine the effect, if any, of fibers on performance.

## 3.3 ELLAVILLE WEIGH STATION

The second testing site was the Ellaville Weigh Station which was located on I-10 in north Florida (Figure 3.8). It included a 620 m. long, 4.8 m. wide traffic lane, and a 1.2 m. shoulder along both sides of the traffic lane. The shoulders were widened to 3 m. forming a bypass lane around the weighing platform area and the agricultural inspection post. A 7 m. wide temporary parking area for trucks was also available on the east side of the weighing platform. The existing pavement composed of 80 mm. (3 in.) to 175 mm. (7 in.) asphalt layer on a 275 mm. (11 in.) of limerock base which had been stabilized according to FDOT Roadway Design standards.

More than 1400 trucks pass through the weigh station each day. As they enter the station from I-10, they begin to slow down until coming to a complete stop prior to proceeding on to the weighing platform. After completing the weighing process, the trucks continue at a slow speed toward the agricultural check point. From the check point to the exit, the trucks increase speed gradually reaching a maximum speed at the merging lane with I-10.

This travel pattern had caused a gradual increase in rutting from the entrance reaching maximum levels prior to the weighing platform and then gradually decreasing toward the exit. The rutting ranged from 9 mm. to 45 mm. Such severe rutting caused significant roughness of the pavement surface and suggested inadequate load carrying capacity of the asphalt layer. By applying thin asphalt overlays, this maintenance effort did restore the smooth ride initially. However, with the daily heavy truck traffic, rutting resumed within a short period of time (Armaghani and Diep, 1999).

The UTW alternative was among other options that were available to the FDOT to resurface the existing pavement. In fact, the UTW was the choice as an interim solution for the chronic rutting problem of the asphalt pavement knowing that there were plans to relocate the entire weigh station further west of its current location to handle the increasing truck traffic on I-10.

The two main considerations that were taken into account were (a) the UTW overlay would restore the ride smoothness, and (b) the composite concrete/asphalt layer would enhance the load carrying capacity of the pavement structure. Also, the high volume of truck traffic would provide an excellent opportunity for a real time accelerated evaluation of the UTW performance and the best prediction of its service life. This information would be valuable for the life cycle cost analysis of the UTW as a viable option for the rehabilitation of intersections and other medium-volume roads. These and other considerations made the Ellaville Weigh Station an ideal site to implement the first field application of UTW on Florida highways.

**Testing and Measurements of the Existing Pavement Conditions**

Various field and laboratory tests including condition surveys were performed at the weigh station to obtain data for the development of design and construction specifications for the UTW. The Falling Weight Deflectometer (FWD) was used to test about 60 test points along the length of the traffic lane. Results from the FWD showed that the average deflection of the asphalt pavement under 550 kPa load was 300 microns. This deflection constituted about 60% of the average deflection measured on the asphalt surface at the site of the Gainesville test track 3, and was comparable to the deflections obtained from the good performing interstate asphalt pavements. Accordingly, it was concluded that the support layers had adequate stiffness to carry the heavy truck traffic and that the support layers did not contribute to the rutting problem of the asphalt layer.

Twenty-nine cores were obtained at the locations where the FWD tests were performed. The thickness of each asphalt core was measured and a thickness profile of the existing asphalt layer along the traffic lane and shoulders was established. The thickness profile showed a variable asphalt measurement ranging from 80 mm. to 175 mm. in the traffic lane. A condition survey of the asphalt surface revealed a significant rutting problem and a moderate level of alligator cracking. Rut measurements were taken in the general vicinity of the core holes and the FWD test points. The rut depth and asphalt layer thickness data were used to determine the required depth of milling in the asphalt layer. Static Plate Load tests were also performed according to Florida Method 5-527 to measure the moduli of the pavement support layers. The layer moduli values from tests on the limerock base, stabilized subgrade and embankment were 362 MPa., 267 MPa., and 194 MPa. respectively. These values are typical of strong support layers beneath asphalt pavements. Results of the Static Plate Load tests also confirmed the conclusions from the FWD tests that the support layers had adequate stiffness.

**UTW Design**

Figure 3.8 shows the design layout of the Ellaville project (Armaghain and Diep, 1999). The main objective of the design was to achieve the maximum bond at the interface between the UTW and the asphalt surface. To achieve that, two different UTW thickness and panel sizes were used. The 620 m. traffic lane included six sections. Three sections were on the west side of the weighing platform and the remaining three were on the east side. Three main designs were provided in the West section and were repeated in the east section. The three designs included one 80 mm section (S1W) and two 100 mm sections (S2W and S3W). The S1W section included 1.2 m. x 1.2 m. panels. The S2W and S3W sections included 1.6 m. x 1.6 m. panels, and 1.2 m. x 1.2 m. panels respectively. Section S4E, S5E and S6E mirrored the designs of S3W, S2W and S1W respectively. The thickness of the shoulders, bypass lanes and parking sections was 80 mm. The panel dimensions in these areas matched those in the traffic lane. These thicknesses and panel sizes were chosen based on the results obtained earlier from the Gainesville test tracks.

The design also called for milling the entire asphalt surface. The milling was specified to remove the rutting, oil spots from the asphalt surface, and produce a level surface with a rough texture. The maximum depth of the milling was set at 40 mm. for the 100 mm. UTW sections and 20. mm for the 80 mm. sections. Based on the thickness profile of the original asphalt layer, the milling depth on the traffic lane was selected to ensure that at least 50 mm. layer of asphalt remained after milling.

The width of the joint opening was 3 mm. to 5 mm., requiring only one pass of the joint saw. The depth of the joint groove was one third of the thickness of the UTW panels. The initial design did not call for the sealing of the panel joints. However after construction, all joints in the east sections were sealed, using a self-leveling silicone sealant. The joints in the west sections were left unsealed with the exception of the last 20 m. of S3W closest to the weighing platform. The joints within the segments were sealed due the fact that the UTW panels were constructed on a new asphalt layer as will be explained in the section on construction.

**Project Specifications**

The Technical Special Provisions (TSP) for the project called for use of Fast Track concrete mixtures. Fibers were specified in the concrete mixtures for the west sections. In the east sections plain concrete was specified. Other provisions covered construction aspects such as asphalt milling, brooming and power washing of the milled surface, as well as placement, curing and joint sawing of the UTW. The joint sawing requirement emphasized the early saw cutting of joints using green-concrete sawing machines to prevent uncontrolled shrinkage cracks. The Technical Special Provisions also required the use of a slip form paver in the traffic lane. However, the use of the slip form paver on the shoulder, bypass and parking areas was optional. In these areas, the contractor opted to use a vibratory roller screed.

Pavement acceptance was based on three main parameters; compressive strength, bond strength, and surface roughness. These parameters were considered to ensure the quality of the overall structure of the pavement.

The bond strength was evaluated based on the Iowa Shearing Test. The specified requirement for the average shearing strength was 1.4 MPa. The TSP required that six cores be obtained from each concrete lot. If the average shearing strength of the six cores was less than 1.4 MPa, the contractor had the option to obtain three additional cores. The lowest and the highest test results of the nine samples would be discarded and an average shearing strength of the remaining seven samples would be considered for lot acceptance. A concrete lot constituted 4000 $m^2$ or one half of the paved area in one day.

Compressive strength requirements at 24 hours and 28 days were specified at 17.5 MPa. and 40 MPa. respectively. The 24-hour strength was used as a criteria for opening the pavement to traffic. The 28-day test results were used for strength acceptance.

The ride acceptance was based on maximum profile index of 110 mm/km as measured by the California Profilograph. The TSP called for penalties if test results failed to meet any of the three acceptance criteria.

### Construction

The construction began by milling the existing asphalt surface. Table 3.8 shows thicknesses of the asphalt layer before and after milling. In the west sections, the milling machine removed more asphalt than what was specified in the design. In some areas the thickness of the asphalt layer after milling was only 29 mm. At a 20 m. segment in S3W just prior to the weighing platform and the bypass lane around the platform, the depth of the milling left the lime rock base exposed. The contractor was notified of the control problem with the milling operation and was asked to remove whatever was left of the asphalt layer in these areas, and place a new layer of asphalt prior to overlay with UTW. He was also asked to adjust the depth of milling to comply with the design plans. The milling resumed on the east sections and was completed in a satisfactory manner. The thickness of the asphalt pavement throughout the project after milling ranged between 92 mm. and 29 mm. with an overall average of 63 mm. as shown in Table 3.8. Areas of very thin asphalt layer were identified for post construction monitoring.

A condition survey after milling revealed random and alligator cracking in some areas of the traffic lane and parking. These observations confirmed the results of visual examination of the core samples of the original asphalt layer. However, since no cracks reflected through the UTW in the Gainesville Tracks 3, it was not anticipated that the cracks would reflect through or affect the performance of the UTW at the weigh station. The milled asphalt surface was broomed and pressure washed to remove debris and dust to ensure a clean and rough surface. This was important to achieve the best bond with the UTW.

Both fiber and plain concrete were used in the UTW. Table 3.9 shows the concrete mixture designs at the production stage. The fiber concrete included 1.8 kg/m$^3$ fibrillated polypropylene (50 mm long) fibers. The fiber concrete was used in the west sections. Plain concrete was used in the east sections. All ingredients including the fibers were placed and mixed at a concrete batching plant and transported to the project site by truck mixers.

The UTW was placed on the traffic lane using a slip form paver. The asphalt surface was kept in damp condition during the paving operation to prevent a loss of water from the concrete mix, and to possibly improve the bond between the UTW and the asphalt. Paving the entire length of the traffic lane was completed in two days. During the first day, the west sections were paved using fiber concrete. Some difficulties were experienced in controlling the consistency of the fiber concrete mixture. On the second day the east sections were paved with plain concrete. A more desirable consistency was achieved with the plain concrete.

During the paving operation, concrete samples were obtained and tested for slump, air content and unit weight. Table 3.9 shows the results of slump, air content and unit weight for fiber and plain concrete mixtures. The slip form paver produced best surface finish when the concrete had a slump of 50 to 75 mm.

The concrete surface was also finished manually to achieve the smoothest possible surface to meet or exceed the TSP requirements for ride acceptance. Surface texturing was followed using a mechanical rake that produced transverse tines on the pavement surface. After texturing, the pavement surface was covered with a white water-based curing compound at a rate of one liter per five square meters of surface. The TSP also called for wet curing or the Contractor's modified curing procedure. The Contractor opted to replace wet curing with a second coating of the curing compound.

The saw cutting of the joints was not completed early enough. Only conventional hard-concrete saw machines were available at the project site. This type of machine required that the concrete achieve a fully hardened state. As a result, more than six hours passed before the first joint was cut. With the large number of joints in the UTW, and the need to expedite the saw cutting process, every second joint was cut initially using only two saw machines. Following this sequence, the joints close to the exit were cut last. The delay in cutting the joints, especially those close to the exit, may have caused weakness in the bond or complete de-bonding between the UTW and the asphalt base. Diamond-shaped cracking has been observed in a series of adjacent panels close to the exit. This cracking pattern will be discussed in the next section. Green concrete saw machines should have been used, as specified in the TSP to cut all the joints while the concrete was in early stages of the hardening process, which would have prevented the concrete curling and de-bonding of the UTW panels.

The UTW was placed in fixed forms on the shoulders, bypass and in parking areas or lots. A vibratory roller screed was used to compact and finish the surface. This paving operation continued for almost three weeks and was not completed with the same quality as on the traffic lane. A similar problem of delayed saw cutting of the joints occurred in these areas. The shoulders and parking areas were in relatively warm temperatures. Several uncontrolled cracks on the shoulder and parking panels did develop due to a combination of warm temperatures and delayed sawing of the joints. This experience demonstrates the importance of using green concrete saw machines to cut joints at a much earlier age compared to the use of the conventional hard-concrete saw machines. As a general rule, joint sawing should begin as soon as the partially hardened concrete can support the weight of the green saw without causing damage to the concrete surface or raveling of the joint groove. It should be noted that after construction, the thicknesses of the UTW in the different sections varied from the original design plans. Table 3.8 shows the high, low and average thickness for the UTW in the six sections. These values are different from the original plans where 80 mm. and 100 mm. thicknesses were designed for the UTW sections, as shown in Figure 3.8. The reasons for this variability were problems in controlling the milling depth in the original asphalt layer, and the need to maintain the standard cross slope for the pavement surface.

**Falling Weight Deflectometer (FWD)Tests**

The FWD tests were performed on the surface of the UTW at approximately the same testing points as for the original and the milled asphalt surfaces. Table 3.10 summarizes the FWD test results. From these results it can seen that milling the pavement caused an average increase of about 12 percent in the deflection measurements. However, after placement of the UTW, the deflection decreased by an average of 63 percent. This reduction implied an improvement in the load carrying capacity of the UTW-asphalt pavement.

It appeared that the variability in asphalt and UTW thicknesses did not play a major role in the magnitude of the FWD deflections as evident from the deflection basin of the UTW. No significant differences were observed in the FWD deflections in the various UTW sections.

Also, the panel dimensions did not influence the magnitude of the deflections. The average deflections in the different sections were very close to each other, as shown in **Table 3.10**. The lowest average deflection was 102 microns in S1W, and the highest average deflection was 120 microns in S3W. This difference is very insignificant.

After six months, the FWD tests were repeated on the original test locations. The deflections were still 56 percent lower than what they were on the original asphalt layer. This indicated that the pavement was still acting as a composite element of two bonded layers possessing sufficient support to carry the high volume of heavy trucks.

### Condition Evaluation

After construction, additional factors emerged which impacted the condition of the pavement and the rate of cracking in the different UTW sections. These factors included excessive milling and further thinning of the asphalt layer, and re-paving areas with a new asphalt where the limerock base had been exposed from excessive milling. These factors also included weak support at the shoulder edge-panels adjacent to the grass embankment and delayed sawing of the joint grooves.

Several crack surveys were performed between the opening of the areas to traffic on December 1997 and January 1999. Table 3.11 presents the results of three crack surveys performed in December 1997, August 1998, and January 1999. The survey results are shown in terms of the number and percentage of cracked panels in the six UTW sections including a traffic lane, shoulder, bypass and parking areas.

The December 1997 survey was taken immediately after construction of the project. Many cracks were observed in the shoulder and parking areas where sawing of the joints had been delayed. These cracks extended along the panel in the vicinity of the joint grooves. Panels with hairline cracks were left in place and recorded in this survey. However, those panels that had wide cracks, with the potential of spalling, were replaced.

The panel replacement operation was simple and relatively quick. A cracked panel was first cut with a saw machine along its four joints to isolate it from the surrounding sound panels. Then the panel was broken up into small pieces with a jackhammer and removed. The condition of the asphalt surface was examined to ensure that the asphalt layer suffered no damage. The asphalt surface was cleaned thoroughly, and was subsequently resurfaced with fresh concrete. Had the asphalt been severely damaged or completely removed upon extraction of the broken concrete, then steps would have been taken to cut into the base to a depth of at least 200 mm. This would have been followed by a thorough compaction of the base and placement of a 275 mm. to 300 mm. full depth concrete panel.

The August 1998 and January 1999 surveys showed a drastic increase in the number of cracked panels. Careful examination of the crack patterns, location and type, revealed some likely culprits. The shoulders in S1W and S2W had no cracks since they were not under traffic. In S3W, S4E and S5E, the shoulders, bypass and parking areas were under traffic from diverting some trucks around the weighing platform toward the widened shoulder edge. This condition, combined with possible de-bonding of the panels from late sawing, is most likely the cause of the cracks on the shoulders.

Along the shoulder adjacent to the grass embankment, a series of panels were cracked as a result of trucks getting off and back on the pavement. Reflector rods were placed along the edges to prevent the trucks from coming off and on the shoulders. This prevented further cracks along the edge panels.

In future projects, where there are no curb and gutter, an asphalt shoulder will be added in the design to provide support to the edge panels of the UTW. In projects where curb and gutter are present, they tend to provide adequate support to the edge panels provided that the surface of the UTW is even with that of the gutter.

The panels in the 20-meter segment of the S3W (closest to the weighing platform) and the bypass area were placed on a new asphalt layer, as explained earlier. In these areas, bonding was poor as indicated by the results of the Iowa Shear Tests. The panels in these areas began to rock under traffic and eventually failed as a result of complete de-bonding from the asphalt base.

Almost all cracks observed on the traffic lane in the August 1998 and January 1999 surveys were corner cracks. These corner cracks were clustered in confined areas in S1W and near the exit (S6E). They developed as a result of delaminating at the interface between the asphalt surface and the UTW panels. These delaminating were caused by delayed joint sawing which allowed the concrete layer to curl at a very early age when the bond with the asphalt had not been sufficiently developed. Figure 3.10 shows a diamond-shaped crack formed by corner cracks of four panels. This is a typical cracking pattern on eight consecutive clusters of panels in S6E close to the exit. A review of the construction record indicated that this segment of S6E was the last to be saw cut. The thickness of the asphalt base did not play a significant role in the cracking rate. This observation is argued by the fact that the highest rate of cracking in the traffic lane occurred in S6E, where the thickness of the milled asphalt was the highest among the six sections.

Panel dimensions had no impact on the rate of cracking and other deteriorations. Sections with 1.2

m. X 1.2 m. panels performed similar to the sections with 1.6 m. X 1.6 m. Likewise, the fiber sections and non fiber sections were performing in a similar manner. Fibers in the concrete may reduce the shrinkage and keep the joint width tight. This effect can be considered as an advantage to prevent water from infiltrating the joints. However, The fibers did not prevent the formation of cracks or the widening of the crack widths.

Sealing of the joints in the east sections did not make a noticeable difference in the rate of cracking compared to the non-sealed west sections. Water infiltration was not a concern. The surveys did not show water problems except in areas where de-bonding occurred, such as the areas where the panels were placed on new asphalt layers.

The thickness of the UTW did have some impact on the rate of cracking. Less cracking occurred in the thick panels of the traffic lane compared to the thin panels in the shoulders. There were no cracks in S4E where the average thickness of the panels was 114 mm. compared to S6E. However, thickness only contributed to prolonging the fatigue resistance. De-bonding of the panels is still considered the main factor in the ultimate cracking of the UTW panels.

After one year of service, the percentage of cracked panels on the traffic lane was 5.5 percent. On the shoulders, bypass and parking areas the number of cracked panels was also 4.4 percent.

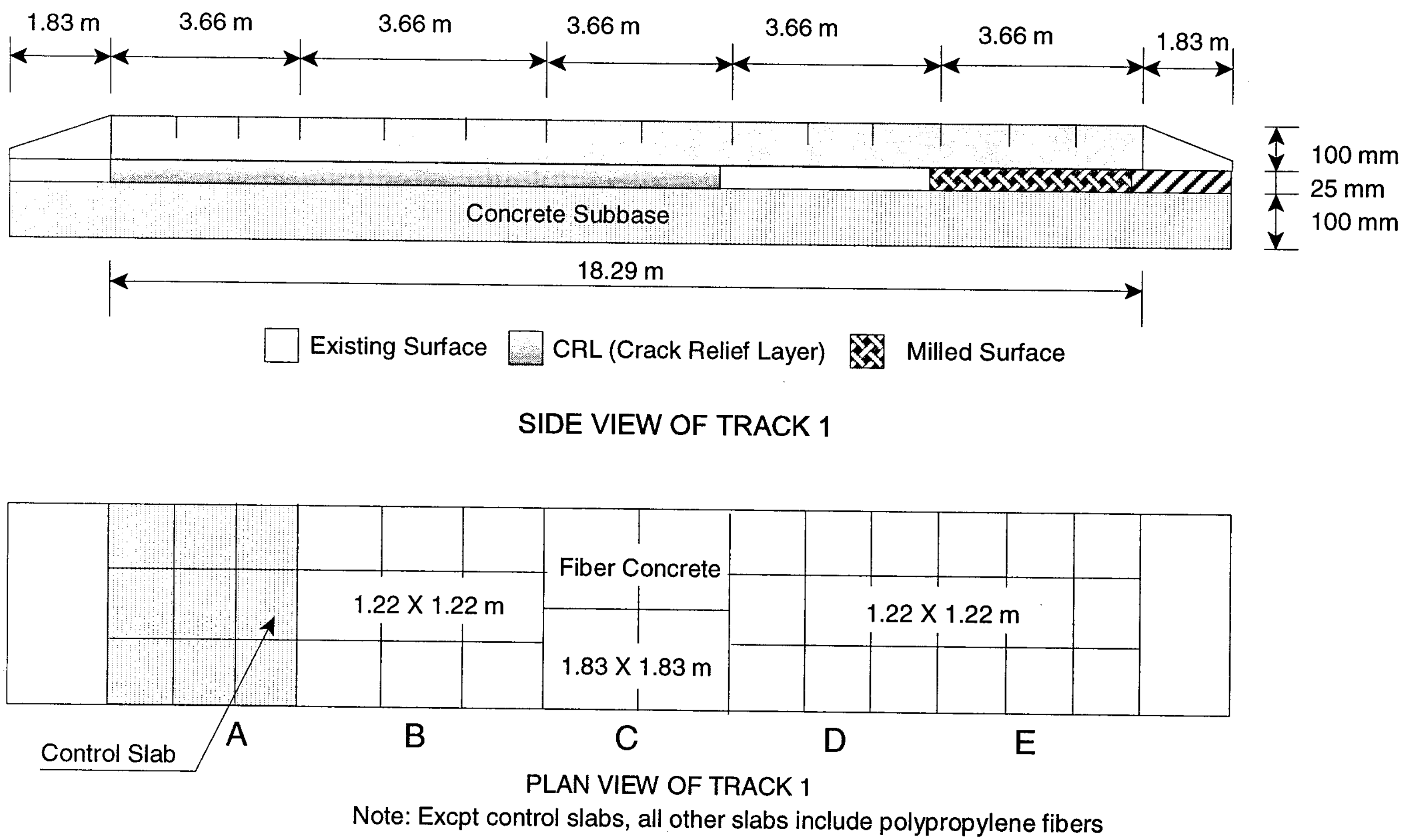
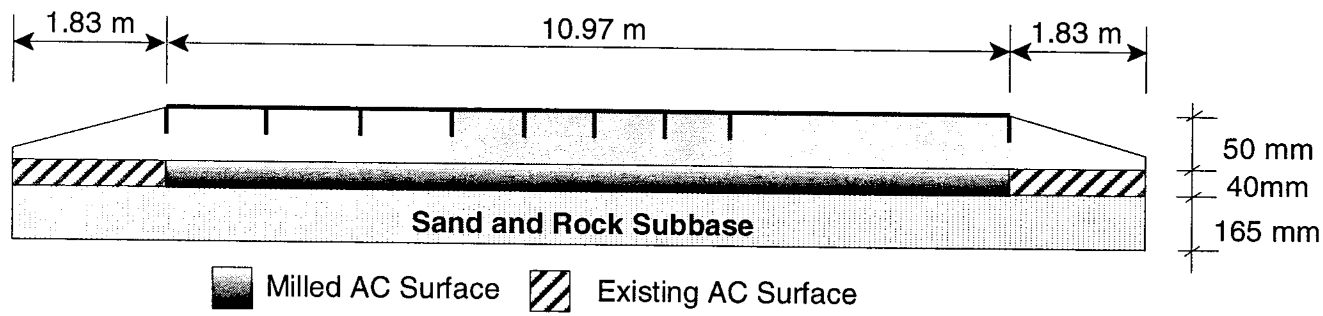
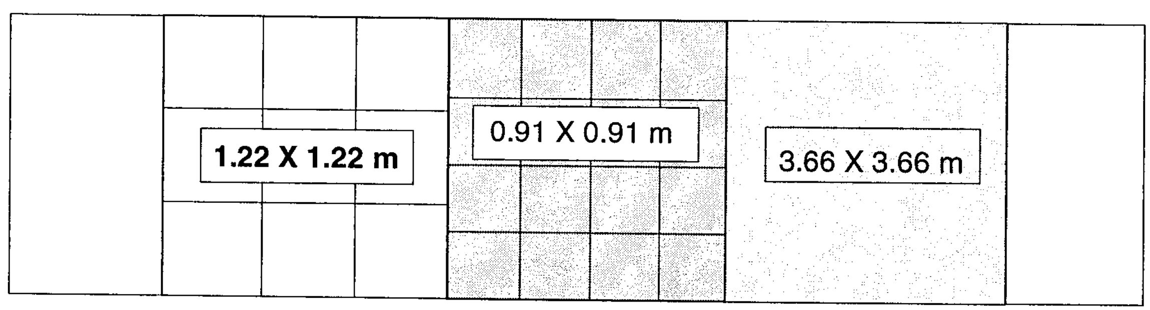



Figure 3.1 Side and Plan View of Track # 1




SIDE VIEW OF TRACK 2




PLAN VIEW OF TRACK II
Note: All slabs include polyolefin fibers

Figure 3.2 Side and Plan View of Track # 2

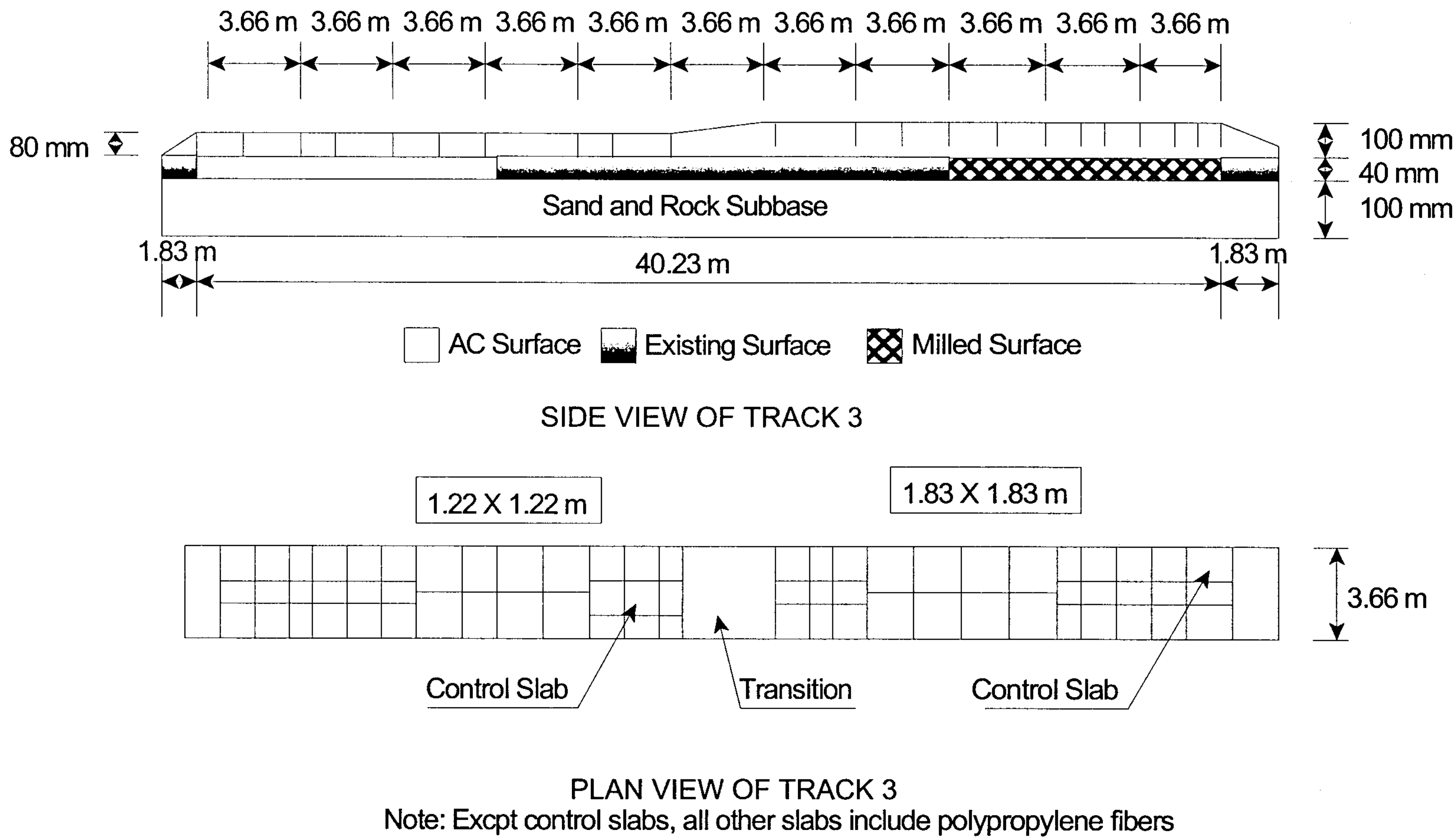



Figure 3.3 Side and Plan View of Track # 3

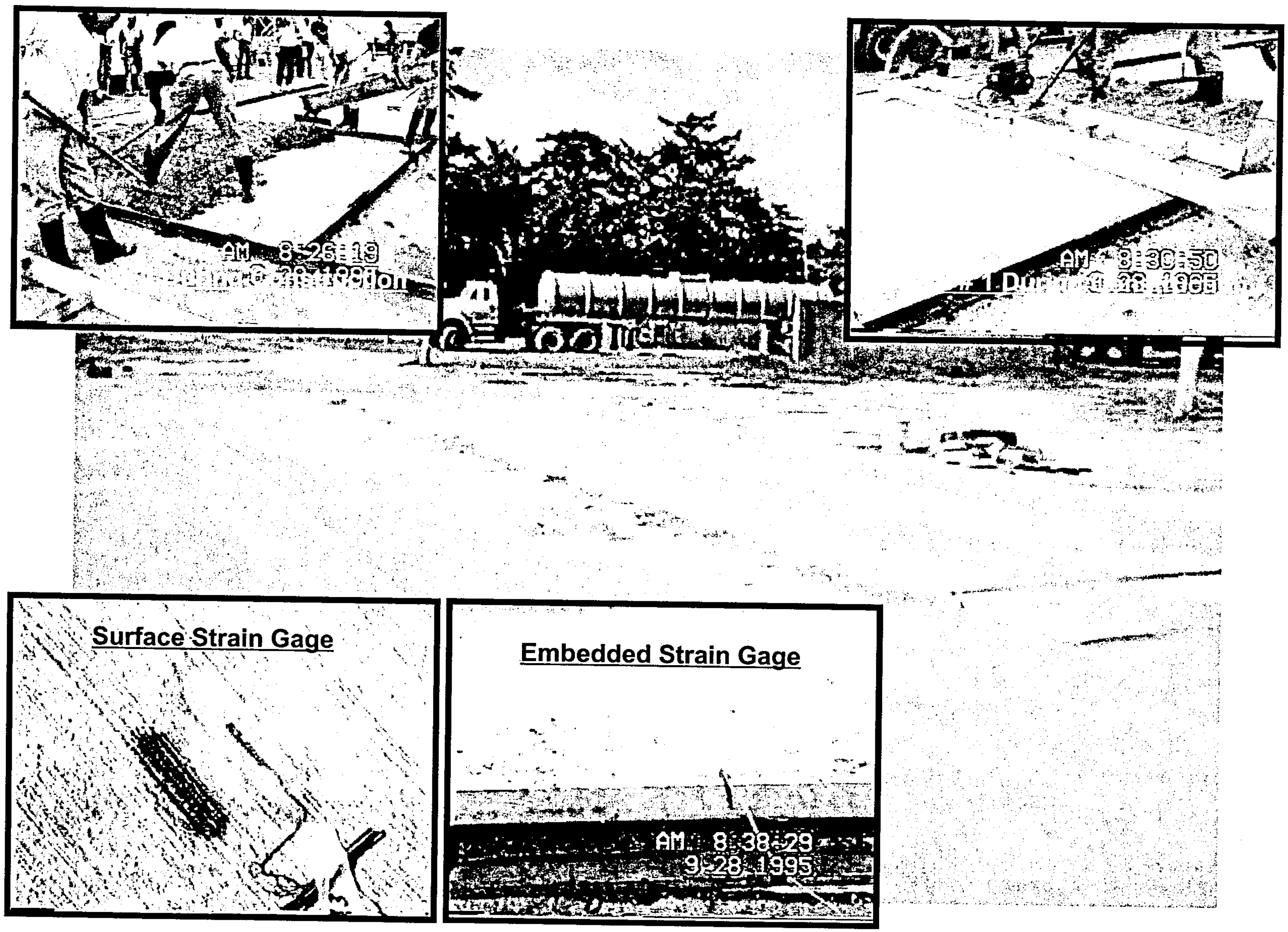



Figure 3.4 Side and Plan View of Track # 1

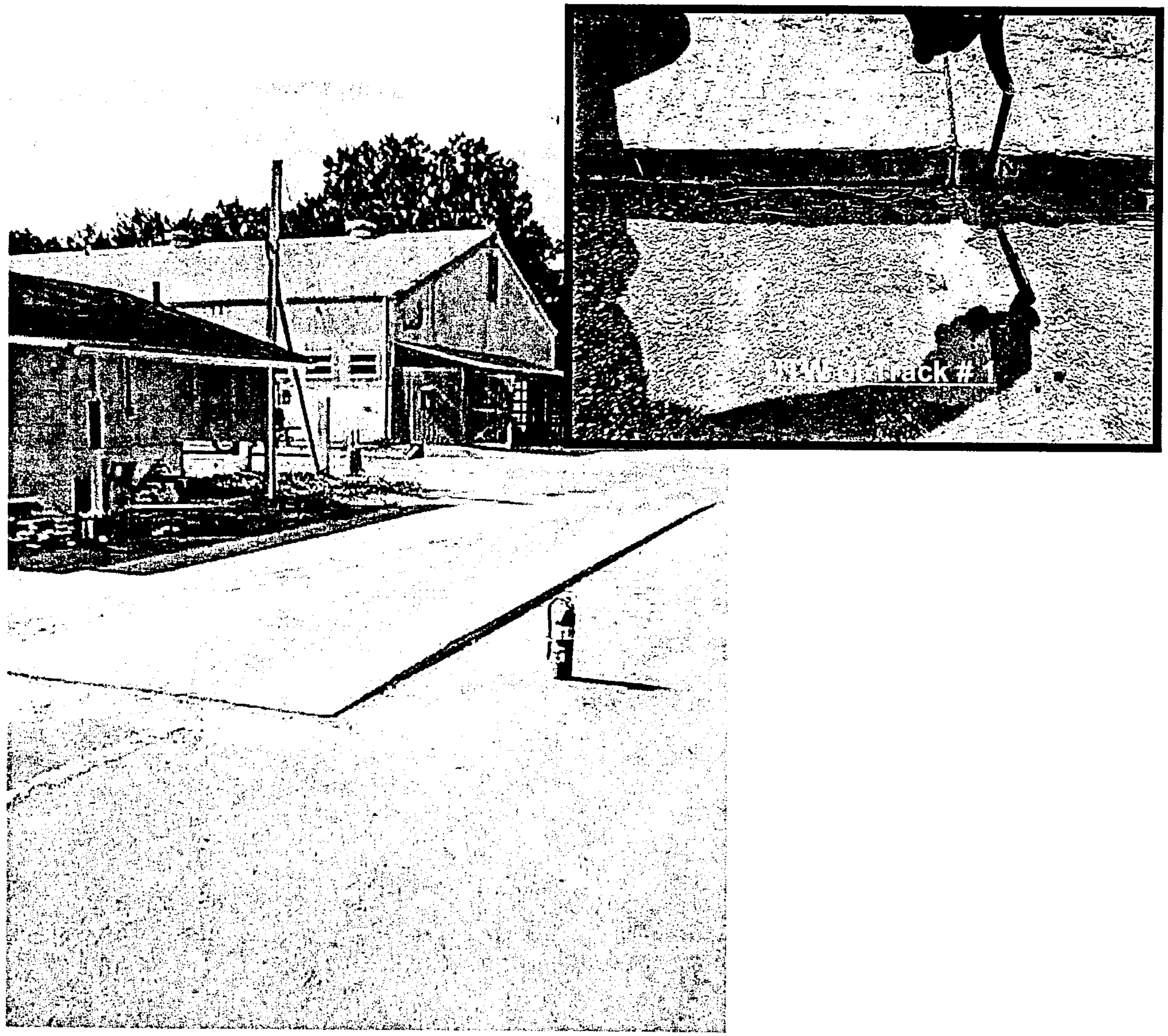



Figure 3.5 Side and Plan View of Track # 2

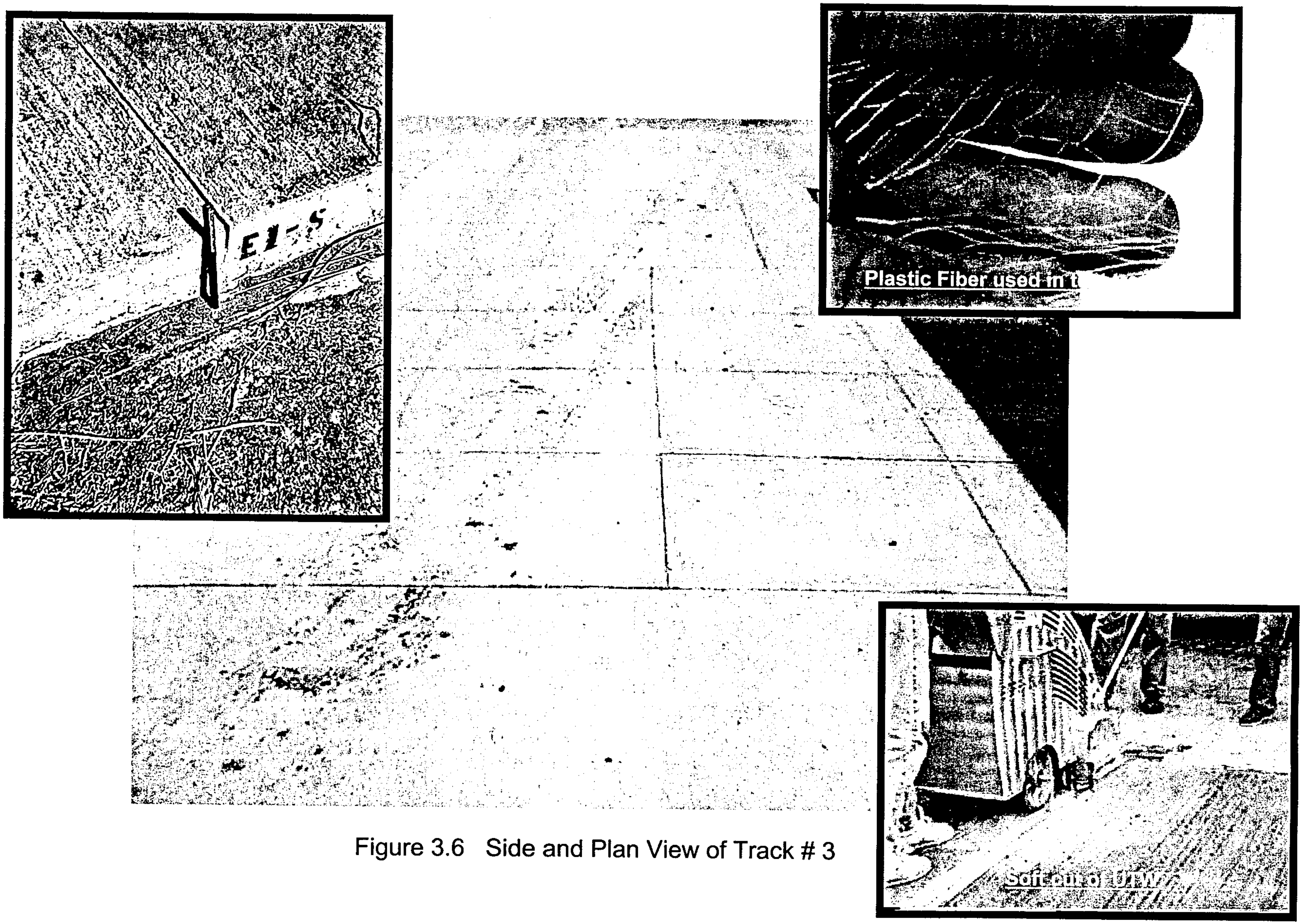



Figure 3.6 Side and Plan View of Track # 3

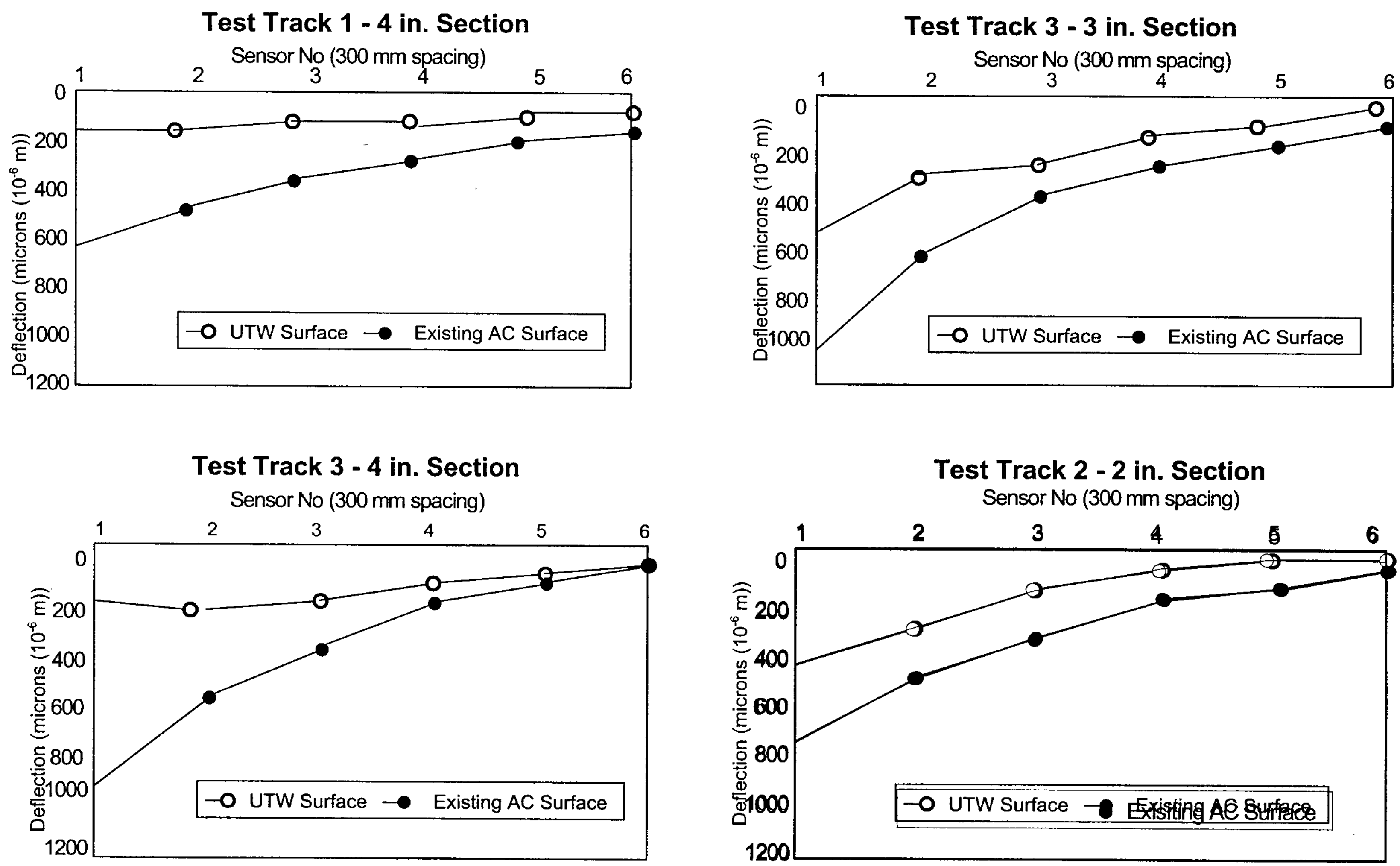



Figure 3.7 FWD Deflection Curves for Existing Asphalt and UTW Surfaces

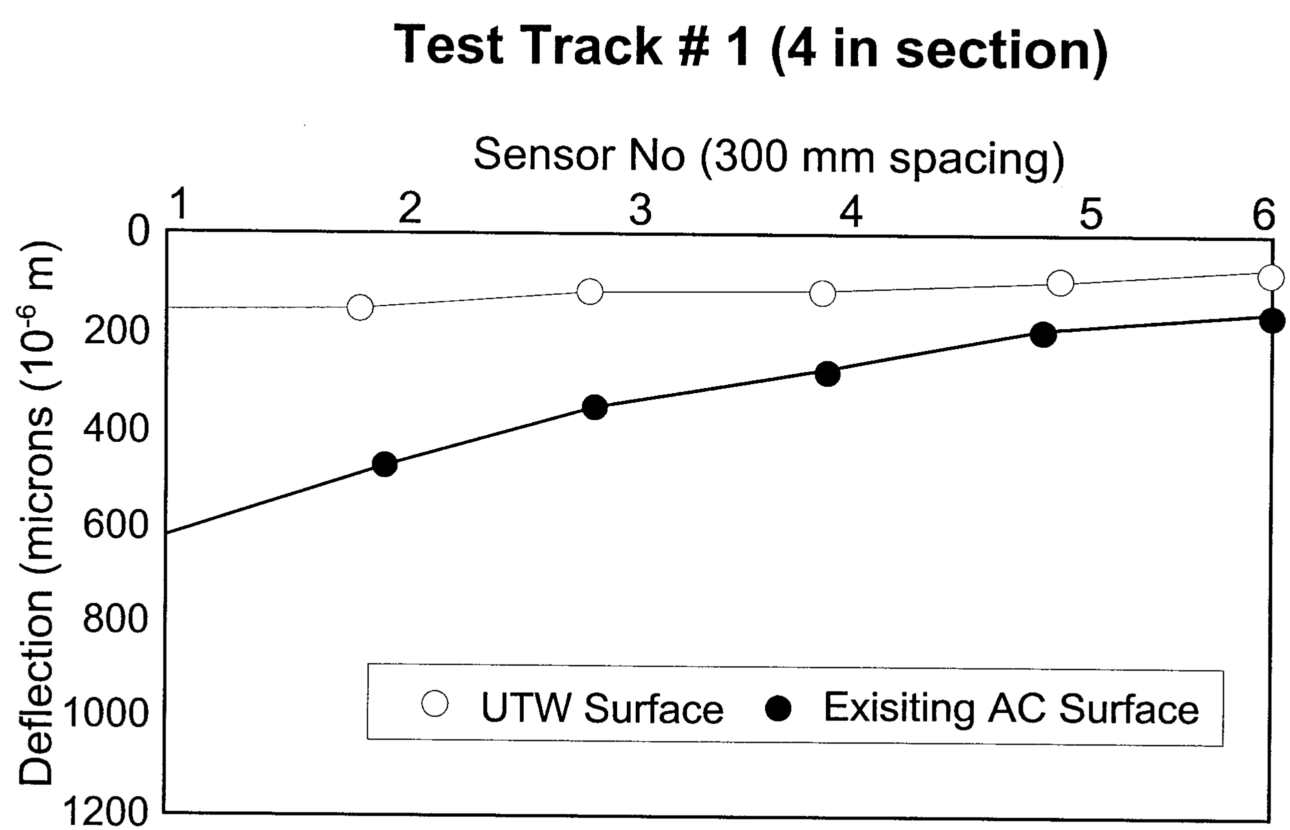



Figure 3.7a FWD Deflection Curves for Existing Asphalt and UTW Surfaces

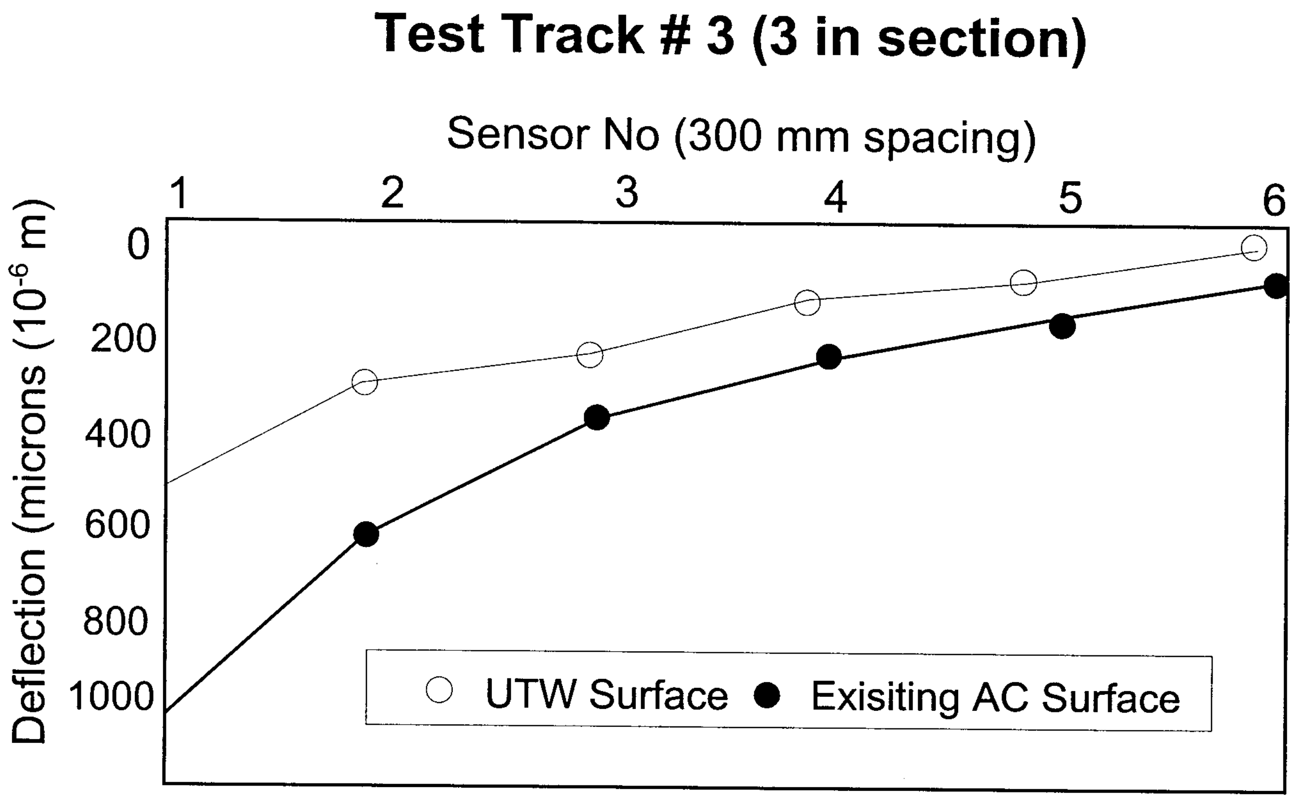



Figure 3.7b FWD Deflection Curves for Existing Asphalt and UTW Surfaces

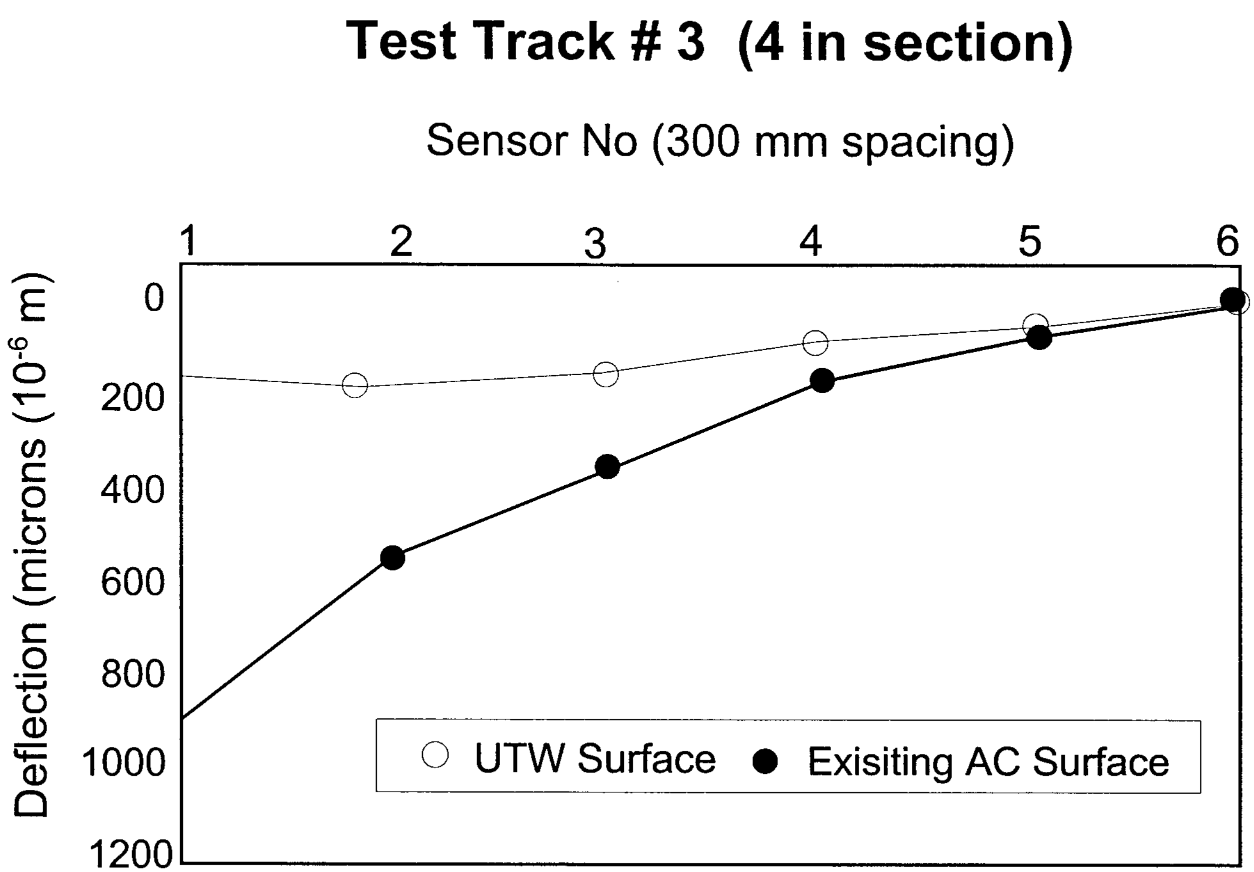



Figure 3.7c FWD Deflection Curves for Existing Asphalt and UTW Surfaces

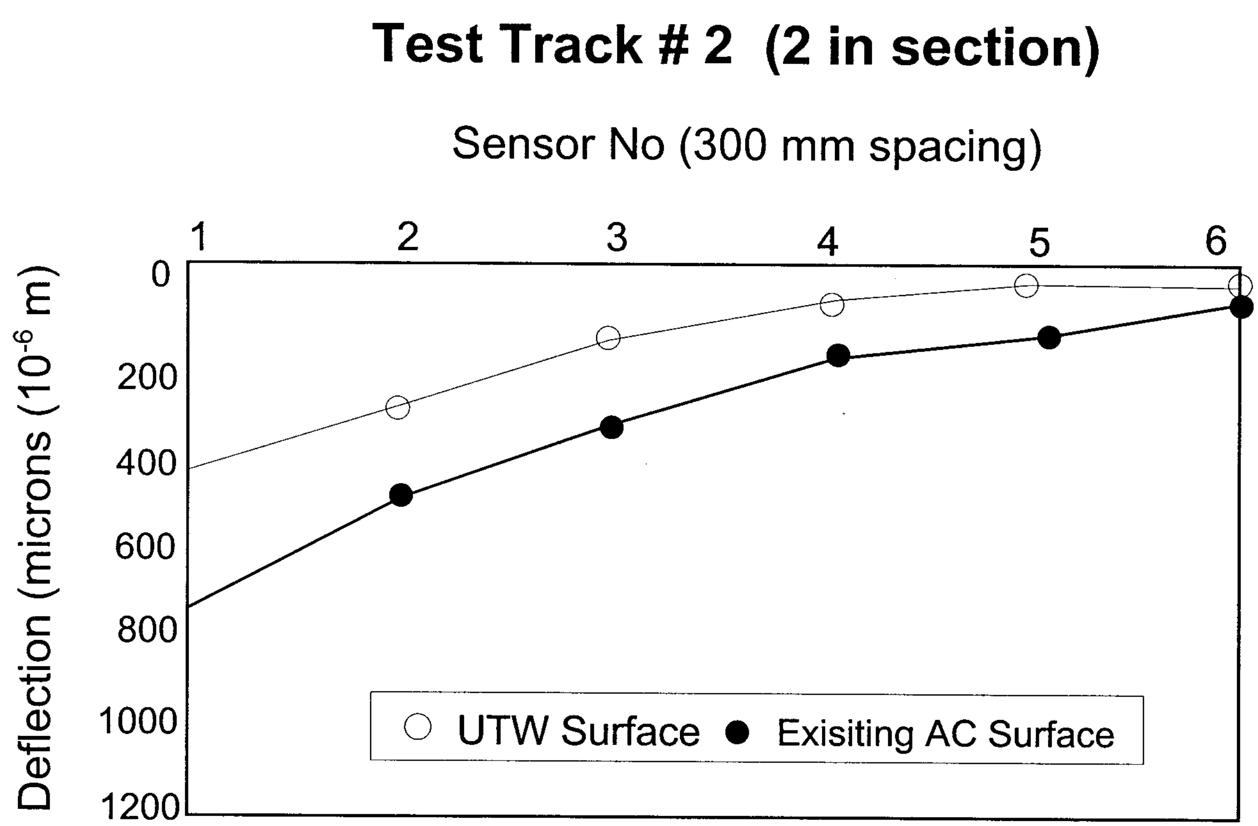



Figure 3.7d FWD Deflection Curves for Existing Asphalt and UTW Surfaces

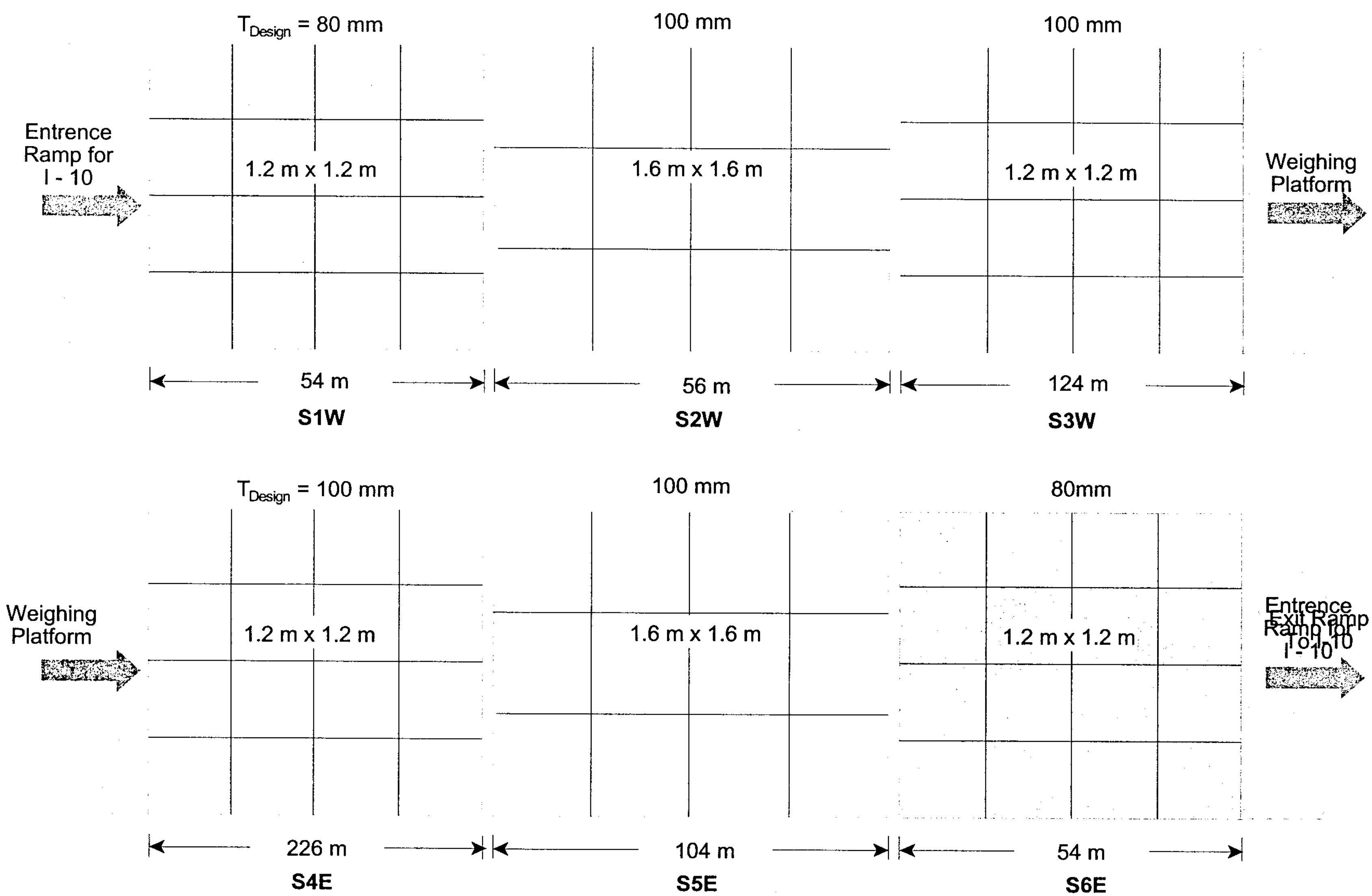



Figure 3.8  Design Layout of UTW at Ellaville Weigh Station

Figure 3.9 Ellaville Wigh Station on I-10.

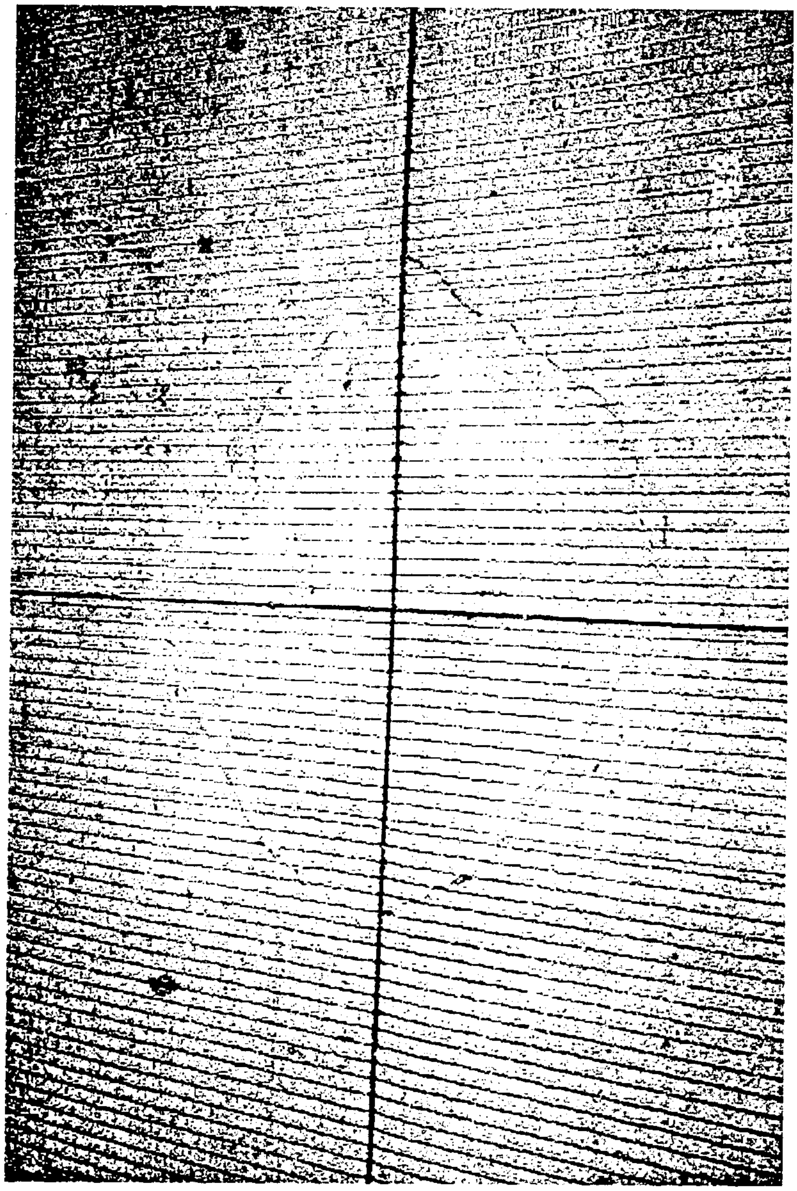


Figure 3.10 Diamond Cracking

Table 3.1: Concrete Mixture Proportions

| Material (kg/m^3) | Track 1 | Track 2 | Track 3 |
|---|---|---|---|
| | | | |
| Type II Portland Cement | | | 445 |
| Type I Portland Cement | 350 | 451 | |
| Type F Fly Ash | 89 | | |
| Coarse Aggregate (Nominal Size) | | | |
| # 57 Crushed Limestone (25mm) | | | 1067 |
| # 67 Crushed Limestone (19mm) | 1038 | | |
| # 89 Crushed Limestone (9.5mm) | | 913 | |
| Silica Sand | 599 | 830 | 485 |
| Total Water | 166 | 174 | 169 |
| Water- Cement Ratio (W/C) | 0.38 | 0.39 | 0.38 |
| Ordinary Air Entrainment (ml) | 622 | 15 | 118 |
| Ordinary Water Reducer (ml) | 681 | 444 | 148 |
| High Range Water Reducer (HRWR) ( | 3552 | 4055 | 3552 |
| Polypropylene Fibers (kg/m^3) | 1.8 | | 1.8 |
| Polyolefin Fibers (kg/m^3) | | 12 | |

Table 3.2: Plastic Properties

| Tests | Track 1 | Track 2 | Track 3 |
|---|---|---|---|
| | | | |
| Slump (Prior to HRWR) (mm) | 57 | 88 | 95 |
| Entrained Air (%) | 2.5 | 1.6 | NA |
| Unit Weight (kg/m^3) | NA | 2266 | 2039 |
| Ambient Temp (°C) | 28 | 28 | 22 |
| Concrete Temp (°C) | 28 | 27 | 26 |

Table 3.3: Average Test Results of Hardened Concrete

| Property (Mpa) | 8-hr | 24-hr | 3 | 7 | 28 | 91 |
|---|---|---|---|---|---|---|
| | | | Age (day) | | | |
| | | | Track 1 | | | |
| Compressive (fc') | 11 | 26 | 37 | 41 | 51 | NA |
| Tensile (ft') | - | - | - | - | 4 | - |
| Flexural (Mr) | - | - | - | - | 7 | - |
| Modulus of Elasticity (E) | - | - | - | - | 33810 | - |
| | | | Track 2 | | | |
| Compressive (fc') | 3 | 14 | 32 | 44 | 51 | 55 |
| Tensile (ft') | - | - | - | - | 4 | - |
| Flexural (Mr) | - | - | - | - | 6 | - |
| Modulus of Elasticity (E) | - | - | - | - | 28980 | - |
| | | | Track 3 | | | |
| Compressive (fc') | 17 | 24 | 41 | 49 | 56 | 59 |
| Tensile (ft') | - | - | - | - | 5 | - |
| Flexural (Mr) | - | - | - | - | 6 | - |
| Modulus of Elasticity (E) | - | - | - | - | 32430 | - |

Table 3.4: Average Shear Test Results (Mpa)

| Track | UTW Thickness (mm) | Asphalt Surfact Treatment: Cleaned | Milled | CRL |
|---|---|---|---|---|
| 1 | 100 | 1 | 2.4 | 0.8 |
| 2 | 50 | - | 2.3 | - |
| 3 | 80 | 2.5 | 2.6 | - |
| 3 | 100 | 1.7 | 3 | - |

Table 3.5: Comparison of FWD Deflections on Asphalt and UTW Surfaces

Track 1
4 in Section

| Sensor No. | deflection (microns) Existing AC Surface | UTW Surface | Percent Reduction in Deflection |
|---|---|---|---|
| 1 | 615 | 131 | 78% |
| 2 | 464 | 109 | 76% |
| 3 | 345 | 89 | 74% |
| 4 | 236 | 69 | 71% |
| 5 | 175 | 51 | 71% |
| 6 | 98 | 38 | 61% |

Track 2
2 in Section

| Sensor No. | deflection (microns) Existing AC Surface | UTW Surface | Percent Reduction in Deflection |
|---|---|---|---|
| 1 | 749 | 420 | 44% |
| 2 | 487 | 297 | 39% |
| 3 | 331 | 167 | 50% |
| 4 | 212 | 100 | 53% |
| 5 | 153 | 60 | 61% |
| 6 | 94 | 42 | 55% |

Track 3
3 in Section

| Sensor No. | deflection (microns) Existing AC Surface | UTW Surface | Percent Reduction in Deflection |
|---|---|---|---|
| 1 | 909 | 457 | 50% |
| 2 | 536 | 277 | 48% |
| 3 | 351 | 222 | 37% |
| 4 | 219 | 137 | 37% |
| 5 | 154 | 70 | 55% |
| 6 | 93 | 51 | 45% |

Track 3
4 in Section

| Sensor No. | deflection (microns) Existing AC Surface | UTW Surface | Percent Reduction in Deflection |
|---|---|---|---|
| 1 | 936 | 202 | 78% |
| 2 | 601 | 238 | 60% |
| 3 | 386 | 204 | 47% |
| 4 | 220 | 150 | 32% |
| 5 | 136 | 100 | 26% |
| 6 | 87 | 71 | 20% |

Table 3.6: Falling Weight Deflectometer Test Results on April 18, 96- AM vs PM (microns)

| Tests | Track 1 | | | Track 2 | | | Track 3 | | |
|---|---|---|---|---|---|---|---|---|---|
| | AM | PM | % Change | AM | PM | % Change | AM | PM | % Change |
| Positions | Entire Track | | | | | | | | |
| Center Panels | 120 | 133 | 11 | 415 | 399 | -4 | 246 | 261 | 6 |
| Edge Panels | 163 | 166 | 2 | 579 | 501 | -13 | 439 | 322 | -27 |
| | 1.22m X 1.22m (4ft X 4ft) Panels | | | | | | | | |
| Center Panels | 119 | 123 | 4 | 385 | 395 | 3 | 263 | 267 | 1 |
| Edge Panels | 163 | 166 | 2 | 429 | 433 | 1 | 330 | 289 | -12 |
| | 1.83m X 1.83m (6ft X6ft) Panels | | | | | | | | |
| Center Panels | 135 | | 19 | | | | 226 | 255 | 13 |
| Edge Panels | 219 | 212 | -3 | | | | 583 | 386 | -34 |
| | 0.91m X 0.91m (3ft X 3ft) Panels | | | | | | | | |
| Center Panels | | | | 405 | 416 | 3 | | | |
| Edge Panels | | | | 570 | 541 | -5 | | | |

Table 3.7: Falling Weight Deflectometer Test Results on August 1, 96- AM vs PM (microns)

| Tests | Track 1 | | | Track 2 | | | Track 3 | | |
|---|---|---|---|---|---|---|---|---|---|
| | AM | PM | % Change | AM | PM | % Change | AM | PM | % Change |
| Positions | | | | | Entire Track | | | | |
| Center Panels | 151 | 185 | 22 | 405 | 366 | -10 | 226 | 290 | 28 |
| Edge Panels | 190 | 192 | 1 | 413 | 401 | -3 | 325 | 265 | -19 |
| | | | | | 1.22m X 1.22m (4ft X 4ft) Panels | | | | |
| Center Panels | 150 | 178 | 18 | 452 | 392 | -13 | 232 | 270 | 16 |
| Edge Panels | 190 | 192 | 1 | 373 | 333 | -11 | 309 | 260 | -16 |
| | | | | | 1.83m X 1.83m (6ft X6ft) Panels | | | | |
| Center Panels | 155 | 214 | 38 | | | | 214 | 335 | 57 |
| Edge Panels | 215 | 235 | 9 | | | | 358 | 275 | -23 |
| | | | | | 0.91m X 0.91m (3ft X 3ft) Panels | | | | |
| Center Panels | | | | 389 | 334 | -14 | | | |
| Edge Panels | | | | 407 | 426 | 5 | | | |

Table 3.8 Pavement Layer Thicknesses (mm)

| **LAYER** | **S1W** | | | **S2W** | | | **S3W** | | |
|---|---|---|---|---|---|---|---|---|---|
| | High | Low | Avg. | High | Low | Avg. | High | Low | Avg. |
| **Original Asphalt** | 115 | 107 | **111** | 107 | 86 | **97** | 134 | 86 | **110** |
| **Milled Asphalt** | 90 | 37 | **64** | 83 | 38 | **55** | 67 | 29 | **43** |
| **UTW** | 130 | 86 | **108** | 110 | 86 | **98** | 112 | 103 | **108** |

(Layer thicknesses were measured from cores)

| **LAYER** | **S4E** | | | **S5E** | | | **S6E** | | |
|---|---|---|---|---|---|---|---|---|---|
| | High | Low | Avg. | High | Low | Avg. | High | Low | Avg. |
| **Original Asphalt** | 147 | 92 | **120** | 100 | N/A | **100** | 120 | 88 | **104** |
| **Milled Asphalt** | 76 | 51 | **62** | 80 | 45 | **67** | 92 | 86 | **89** |
| **UTW** | 129 | 98 | **114** | 115 | 96 | **106** | 91 | 83 | **87** |

**Table 3.9 Concrete Mixtures and Properties of Plastic Concrete**

| **Material** | **Quantity ($kg/m^3$)** | |
|---|---|---|
| Mixture Component | Fiber | Plain |
| Type I Portland Cement | 362 | 364 |
| Coarse Aggregate (25 mm size) | 984 | 991 |
| Silica Sand | 792 | 802 |
| Total Water | 142 | 142 |
| Water - Cement Ratio (W/C) | 0.39 | 0.39 |
| Air Entrainment (ml) | 73 | 116 |
| Water Reducer (ml) | 1393 | 1393 |
| Polypropylene Fibers ($kg/m^3$) | 1.8 | N/A |

| **Tests** | **Fiber** | **Plain** |
|---|---|---|
| Slump (mm) | 58 | 52 |
| Entrained-Air (%) | 4 | 4 |
| Unit Weight ( ($kg/m^3$) | 2282 | 2299 |
| Ambient Temp (°C) | 22 | 24 |
| Concrete Temp (°C) | 24 | 25 |

Table 3.10 Average FWD Deflections

| Pavement Surface Tested | Deflection (microns) | Percent Change based on Existing AC |
|---|---|---|
| **Original Asphalt** | 307 | N/A |
| **Milled Asphalt** | 343 | **12%** |
| **UTW Overlay:** | | |
| **S1W** | 102 | **-67%** |
| **S2W** | 113 | **-63%** |
| **S3W** | 120 | **-61%** |
| **S4E** | 113 | **-63%** |
| **S5E** | 108 | **-65%** |
| **S6E** | 107 | **-65%** |
| **UTW Average:** | | |
| **(6-Month UTW)** | 134 | **-56%** |

Table 3.11 Crack Surveys Results

| SECTION | LOCATION | SURVEY DATE | | | Percent Crack |
|---|---|---|---|---|---|
| | | Dec. '97 | Aug. '98 | Jan. '99 | |
| **S1W** | Traffic Lane | | 18 | | **1.0%** |
| | Shoulder | | 1 | | **0.0%** |
| | Parking | | None | | |
| **S2W** | Traffic Lane | | 9 | | **0.5%** |
| | Shoulder | | | | **0.0%** |
| | Parking | | None | | |
| **S3W** | Traffic Lane | | | 3 | **0.2%** |
| | Shoulder | | 8 | | **1.2%** |
| | Parking | | None | | |
| | Bypass | | 16 | 9 | |
| **S4E** | Traffic Lane | | | | **0.0%** |
| | Shoulder | 3 | 17 | 15 | **1.7%** |
| | Parking | 4 | 7 | | |
| **S5E** | Traffic Lane | | | 1 | **0.1%** |
| | Shoulder | | 10 | | **1.2%** |
| | Parking | 5 | 18 | | |
| **S6E** | Traffic Lane | | 66 | 2 | **3.8%** |
| | Shoulder | | 5 | 2 | **0.26%** |
| | Parking | | | | |

**Total:**

**Traffic Lane** = **99 (5.5%)**

**Shoulder/Parking/Bypass** = **119 (4.4%)**

Traffic Lane = 1800 panels
Shoulders/Parking/Bypass = 2700 panels

# CHAPTER 4

## COMPUTER MODELING AND ANALYTICAL RESULTS

### 4.1 INTRODUCTION

Pavement overlay models were created using an ANSYS 5.3 finite element program. This program was chosen because it is a less expensive approach to model simulation, unlike testing the physical model for various conditions which is a costly approach. This program enables us to study the performance of a pavement under different environmental and loading conditions and also to study the effects of various parameters without the need to conduct actual tests on the physical model.

The pavement models consist of four layers: soil, plain cement concrete, asphalt, fiber reinforced/plain concrete. The dimensions of each layer are as follows.

### 4.2 DIMENSIONS AND LAYERS OF GAINESVILLE TEST TRACKS

**Track 1**

| | |
|---|---|
| Soil layer | 144 in. X 60 in. X 720 in. |
| Plain reinforced concrete layer | 144 in. X 4 in. X 720 in. |
| Asphalt layer | 144 in. X 0.75 in. X 720in. |
| Fiber reinforced concrete layer | 144 in. X 4 in. X 720 in. |

**Track 2**

| | |
|---|---|
| Soil layer | 144 in. X 60 in. X 576 in. |
| Plain reinforced concrete layer | 144 in. X 6 in. X 576 in. |

| | |
|---|---|
| Asphalt layer | 144 in. X 1.5 in. X 576 in. |
| Fiber reinforced concrete layer | 144 in. X 2 in. X 576 in. |
| Track 3Soil layer | 144 in. X 60 in. X 720 in. |
| Plain reinforced concrete layer | 144 in. X 4 in. X 720 in. |
| Asphalt layer | 144 in. X 1.5 in. X 720 in. |
| Fiber reinforced concrete layer | 144 in. X 3 in. X 720 in. |

## 4.3 MODEL DEVELOPMENT

### Model Creation

Each layer was considered a volume and was created separately. All the layers were then joined by gluing the volumes. The top layer, a fiber reinforced concrete overlay layer, was again divided into two separate portions. The top part was equivalent to the depth of the joint and the bottom one represented the remaining thickness of the UTW layer. The top part was divided into panels with equal spacings between the panels. The thickness of all the panels was equal to the depth of the joint. Thus, the top layer resembled the overlay with joints.

### Meshing

The next step consisted of meshing each layer. Volume attributes were then assigned to each layer to assign the designated material properties for each layer of the pavement. The mapped meshing was the method used in this study. The advantage of such method was the elimination of the development of any tetrahedral elements

## Elements

Following are the kind of elements used in the Finite Element Method (FEM) models:

| | |
|---|---|
| Solid 65 | 3D Plastic Reinforced Concrete Solid Element (for UTW) |
| Solid 45 | 3D Structural Solid (for remaining layers) |
| Contact 49 | 3D General Contact Element |

## Friction Layer

The next step was to create a friction layer between asphalt layer and fiber reinforced concrete layer. Attributes were then assigned to this layer. The model was thus completed.

## Boundary Conditions

The next step consisted of assigning boundary conditions to the model. The boundary conditions are assigned to the nodes. At the bottom the model were restrained in all the three directions i.e., X, Y, Z. The nodes on all the four sides of the model are restrained in both X and Z directions. The top layer i.e., fiber reinforced concrete layer was not restrained in any direction.

## Loading

The next step was to study the performance of the pavement under different loading conditions. Loads in the form of a sixteen-wheeler semi-truck were applied on the model. These loads were moved on the model from one end to the other to study different stress conditions.

Because of the size of the models and the CPU time needed to run each load step, it was decided to combine the truck loading in a series of load steps. Each load step represented a certain load positioning.

In addition, smaller 3D models were built to facilitate running as many models as possible in a timely fashion. Each model had blocks of same slab size so as to study the effect of slab size and compare them with the other sizes. These sectioned models were also subjected to the same calibration process performed on the 3D full-scaled models. Truck-loads were distributed on the pertaining nodes considering the tire pressure and spacing between tires. A total of eight Finite Element (FE) models were built to complete the study of the UTW parameters. Each model s run under moving loads consisting of four to seven load steps.

One model for each of the above models was prepared again but making the model act as one, which means having a full bonding. All the above steps were carried out on these models too and results were obtained for studying the effect of bonding. Some of the computer model figures are shown in the following pages.

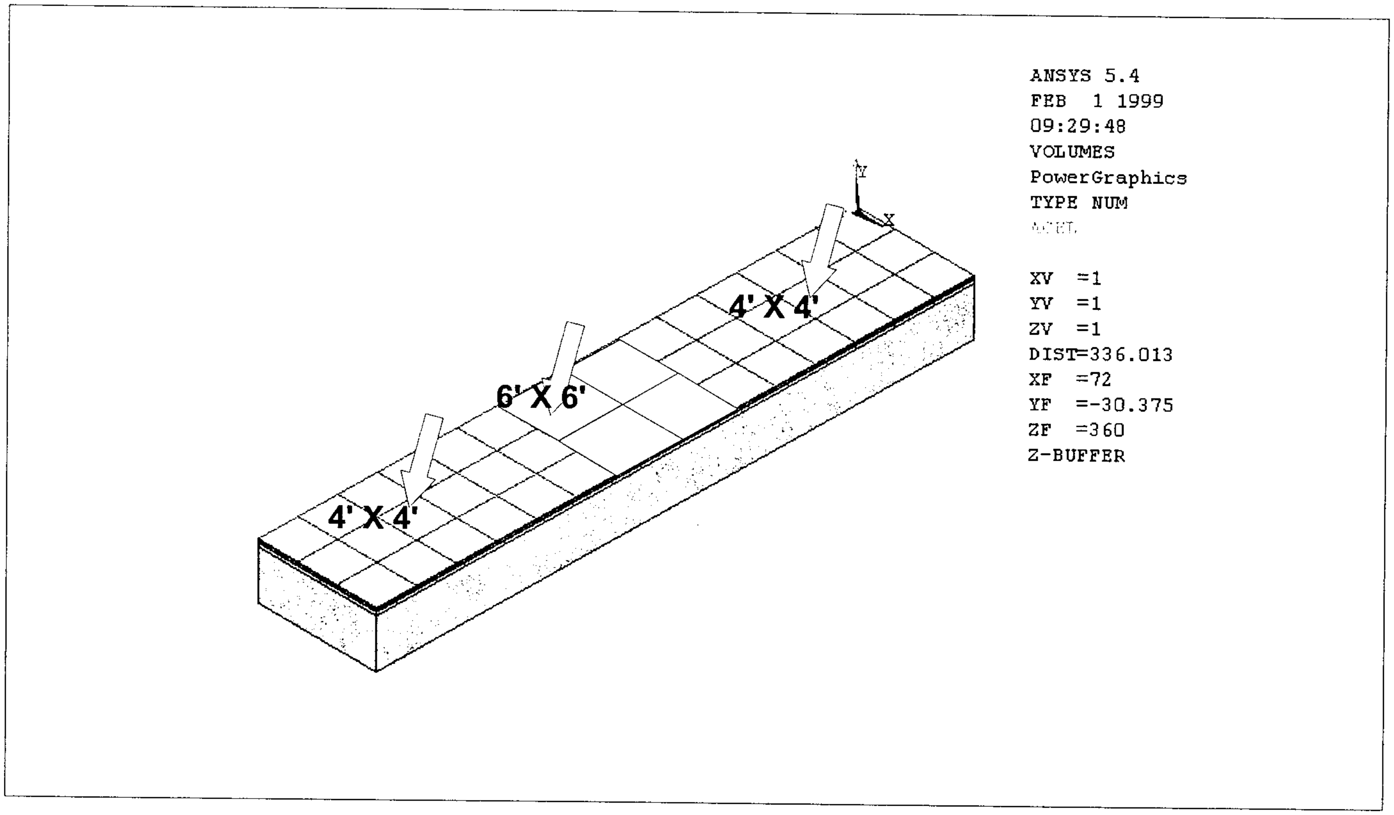



Figure 4.1 Track # 1 of 4" UTW

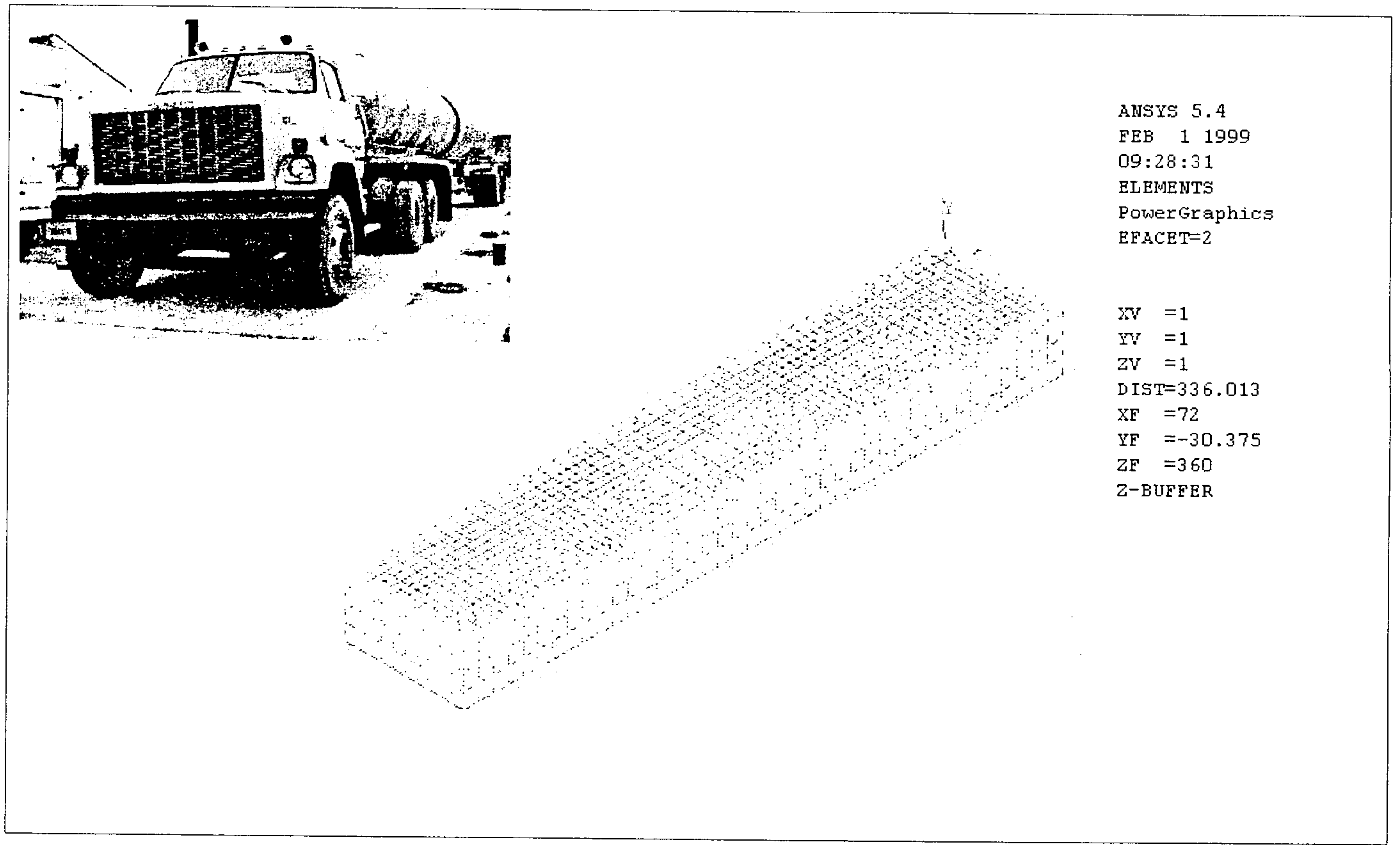



Figure 4.2 The 4" UTW Showing Boundary Conditions

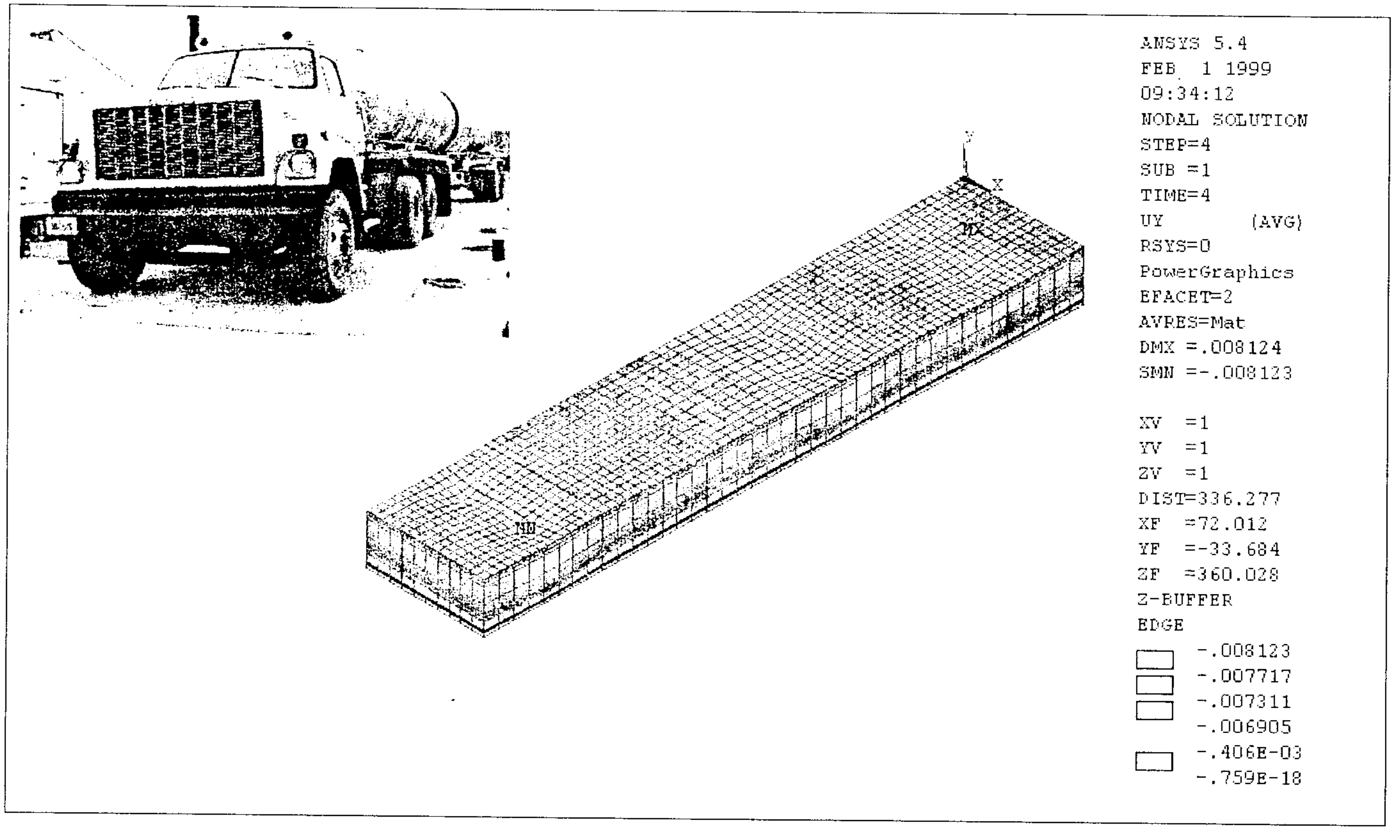



Figure 4.3 Pattern of Vertical Deflection on Track # 1 Due to Truck Loading.

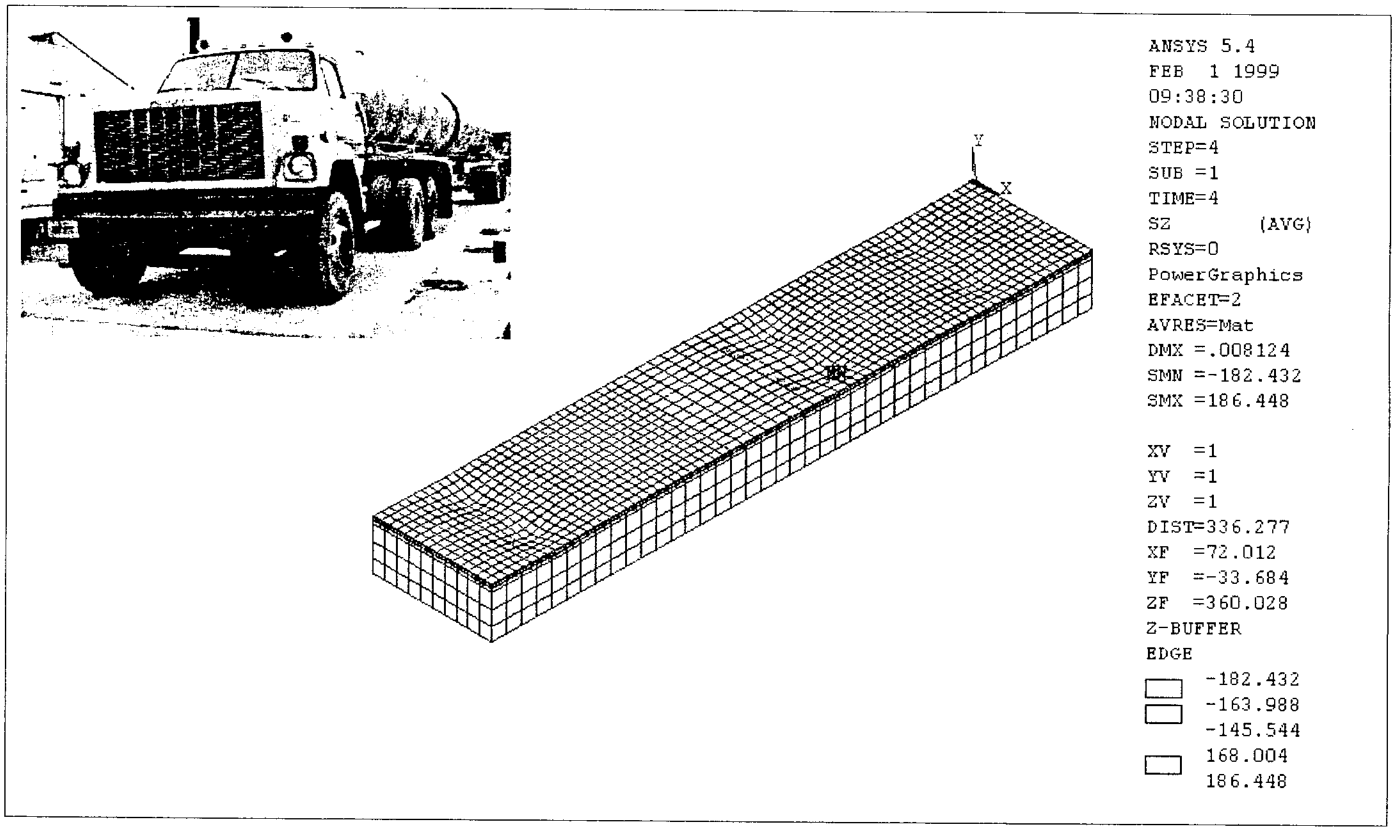



Figure 4.4 Stress on Track # 1, With The Load

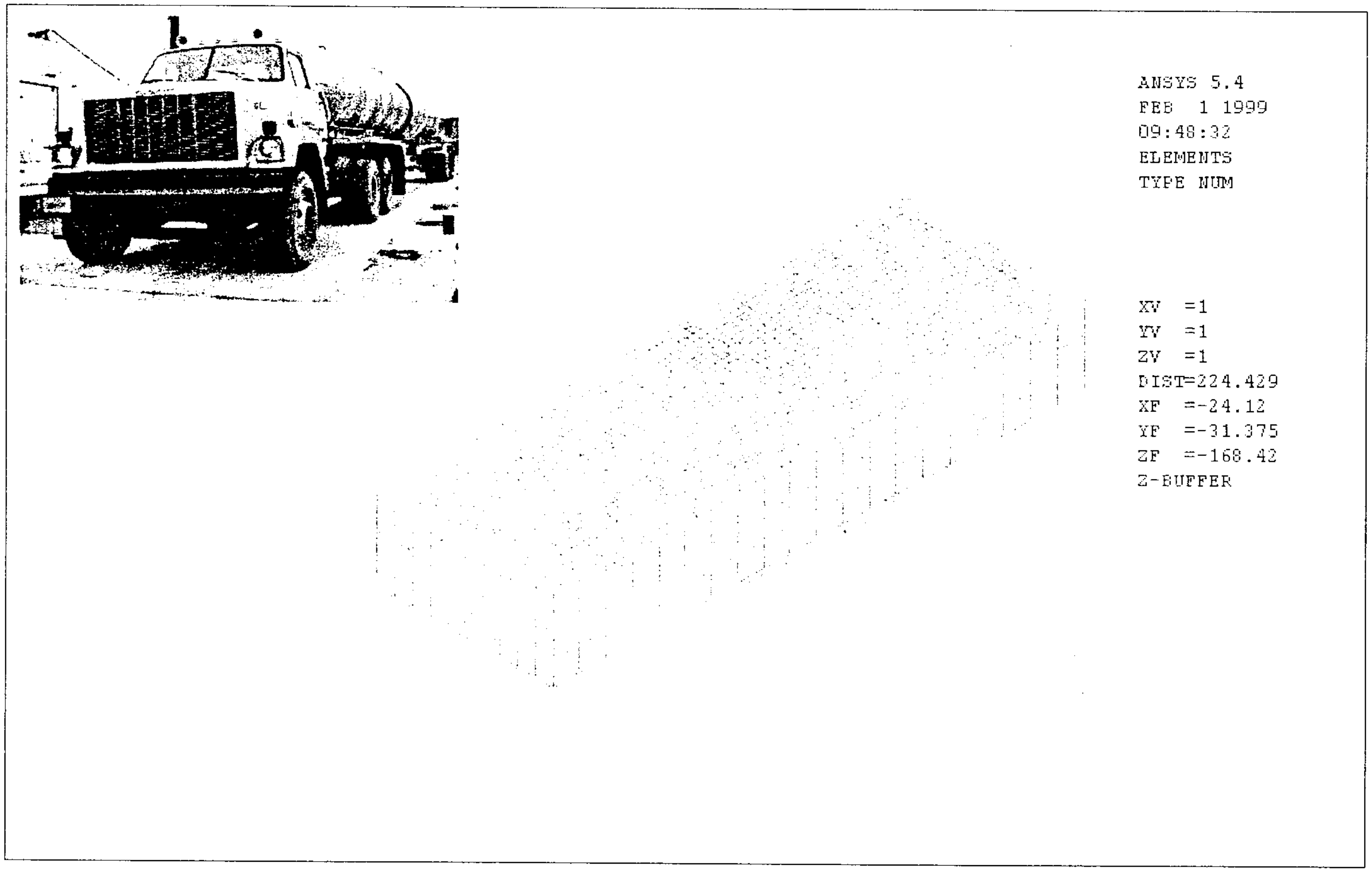



Figure 4.5 Track # 2

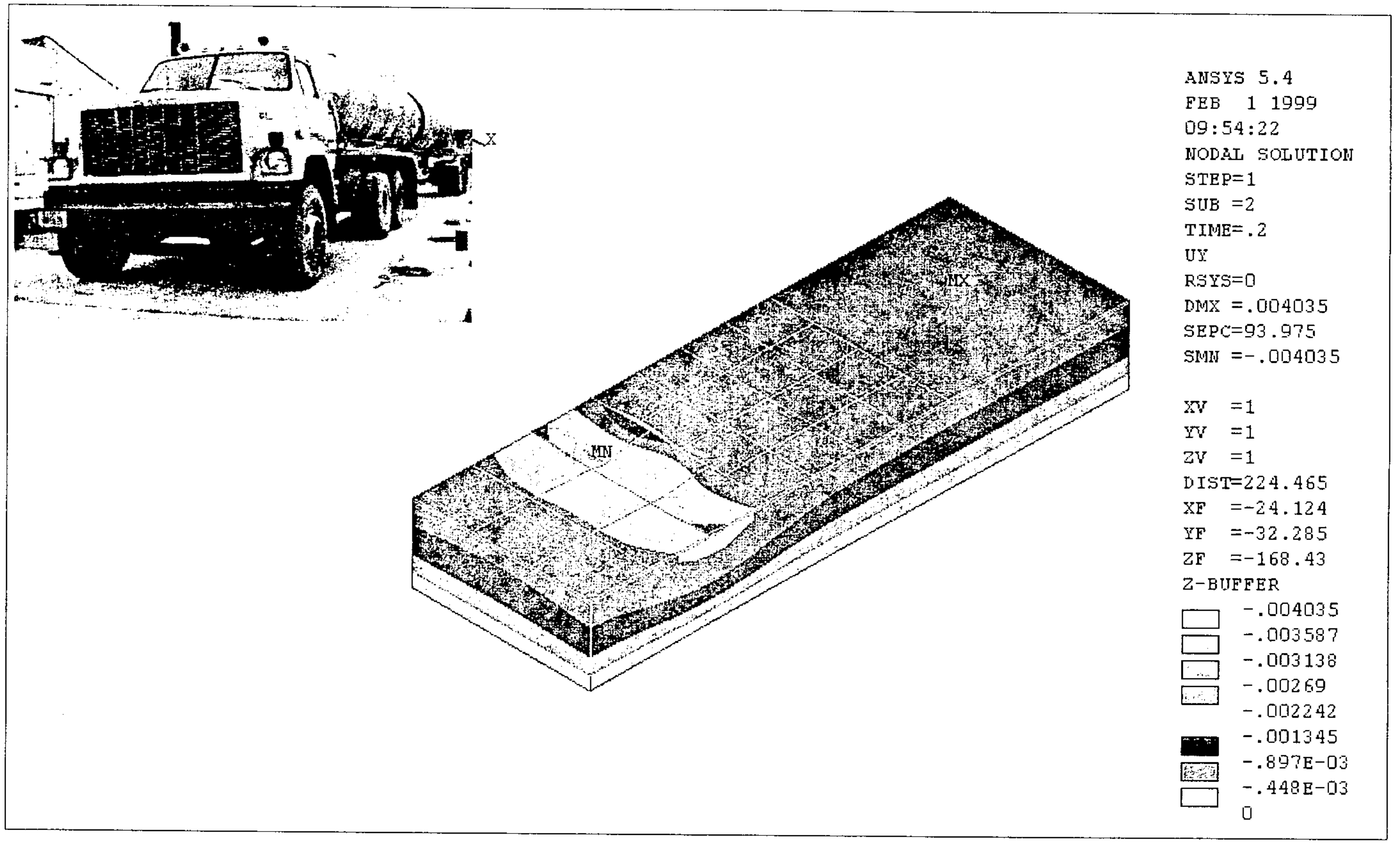



Figure 4.6 Vertical Deflection on Track # 2 Due to Rear Axle

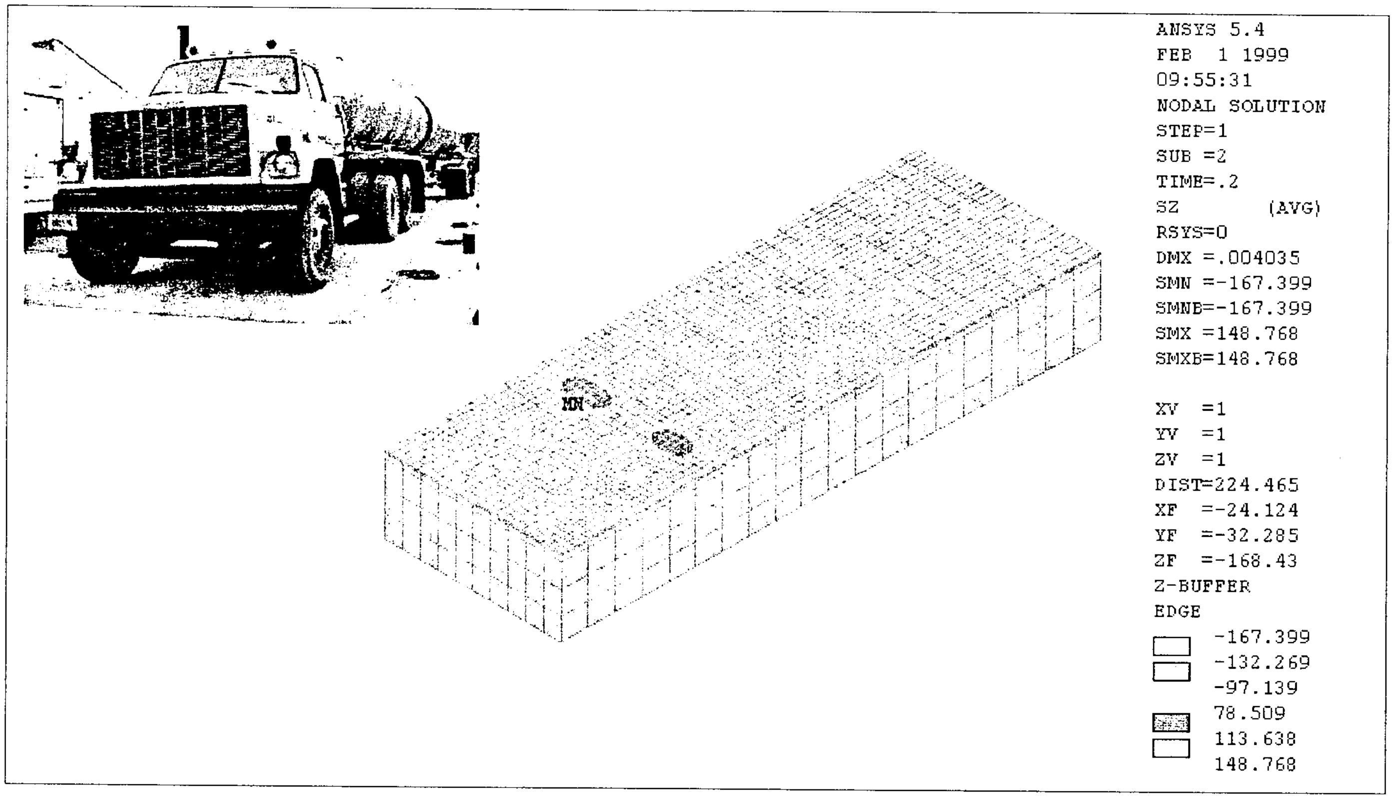



Figure 4.7 Stress on Track # 2, Due to Rear Axle

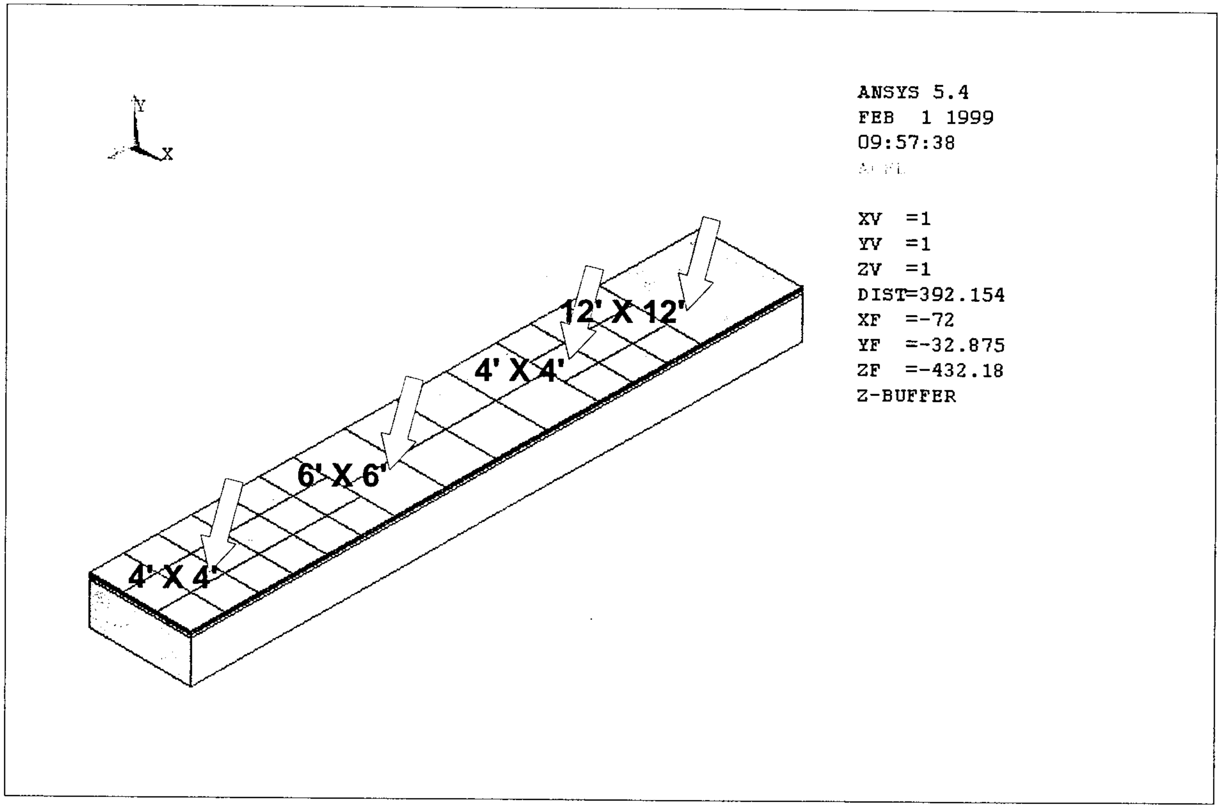



Figure 4.8 UTW Track # 3 with 3" Slab Thickness

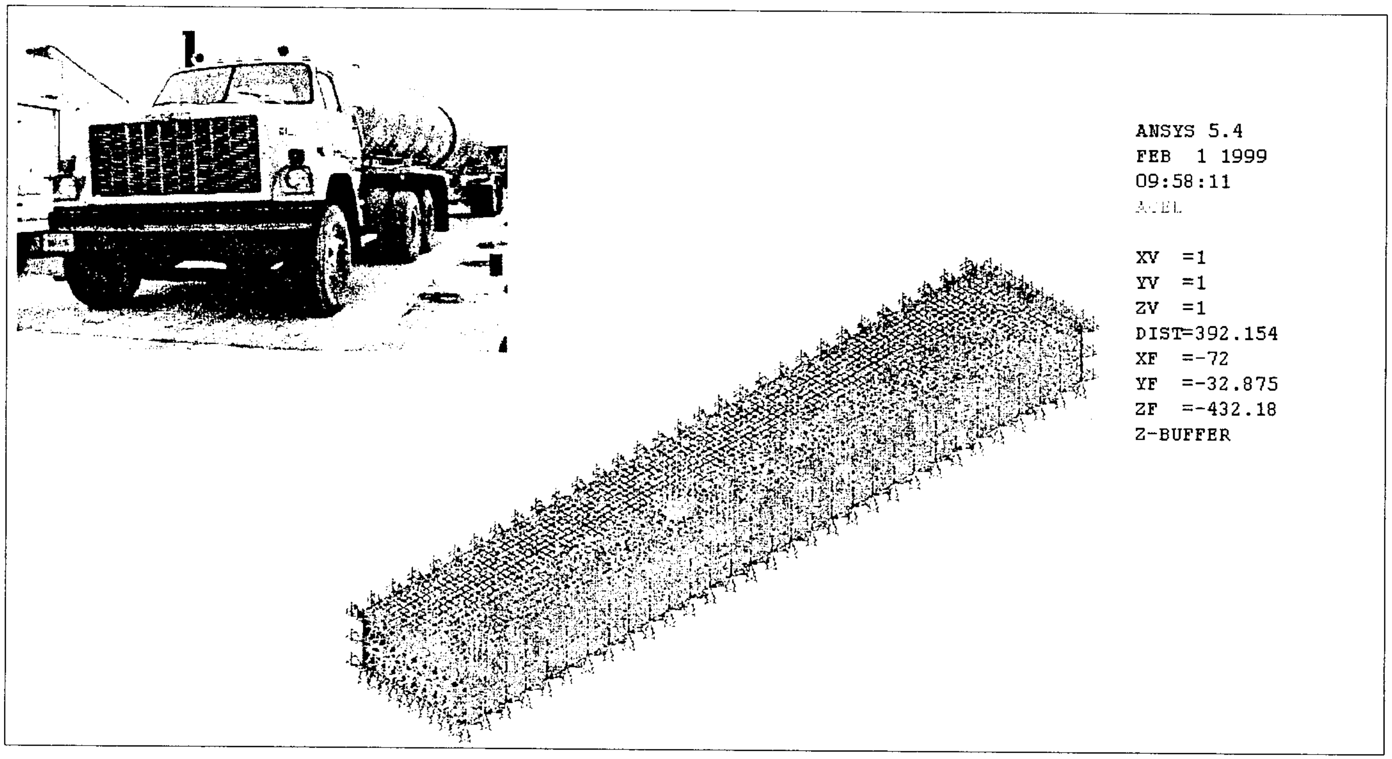



Figure 4.9 Track # 3, Showing Boundary Conditions

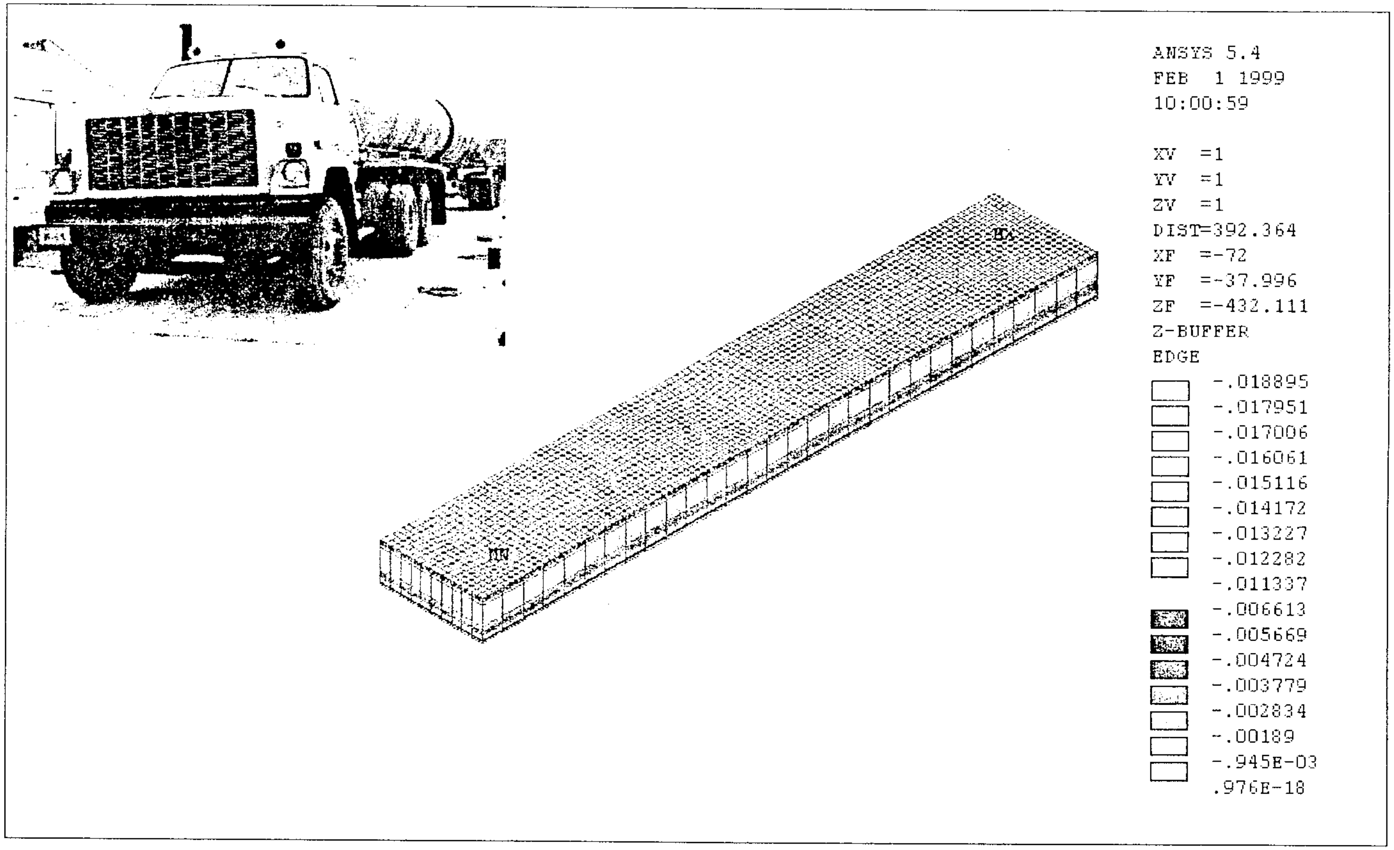



Figure 4.10 Vertical Deflection on Track # 3, Due to Truck Loading

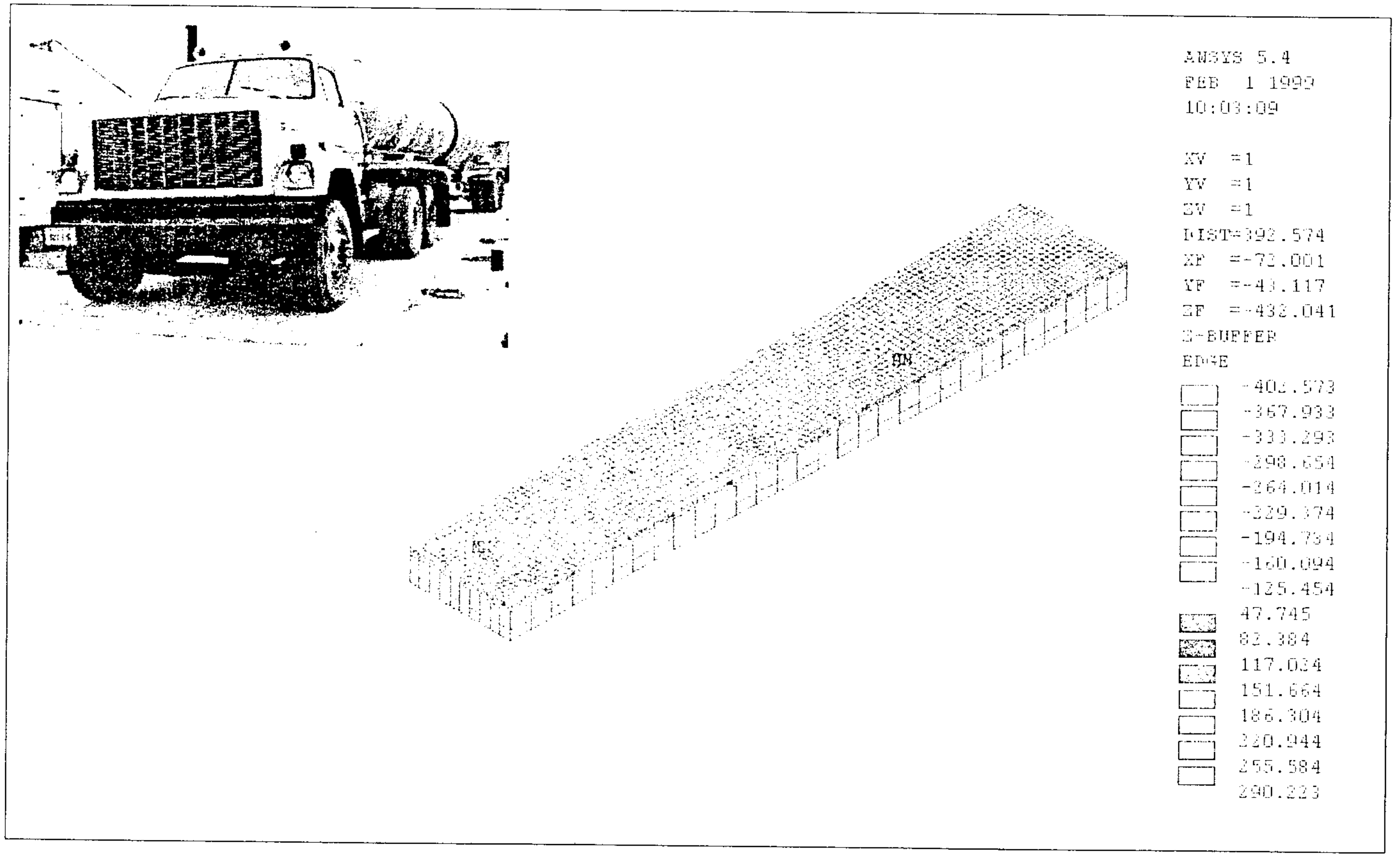



Figure 4.11 Stress on Track # 3, Due to Truck Loading

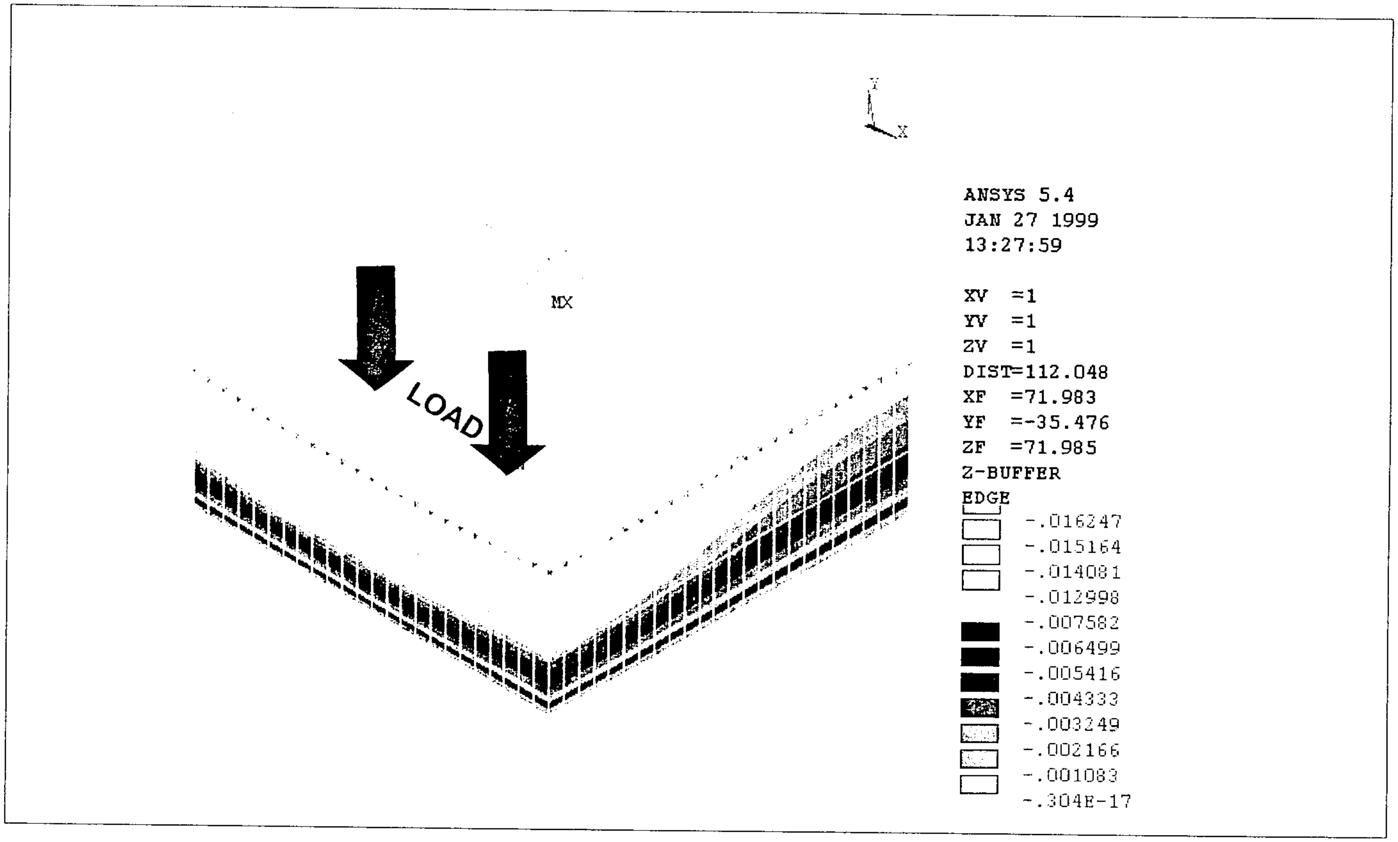



Figure 4.12 Vertical Deflection (in.) onTrack 1, Slab Size - 4' X 4' (Weak Bonding)

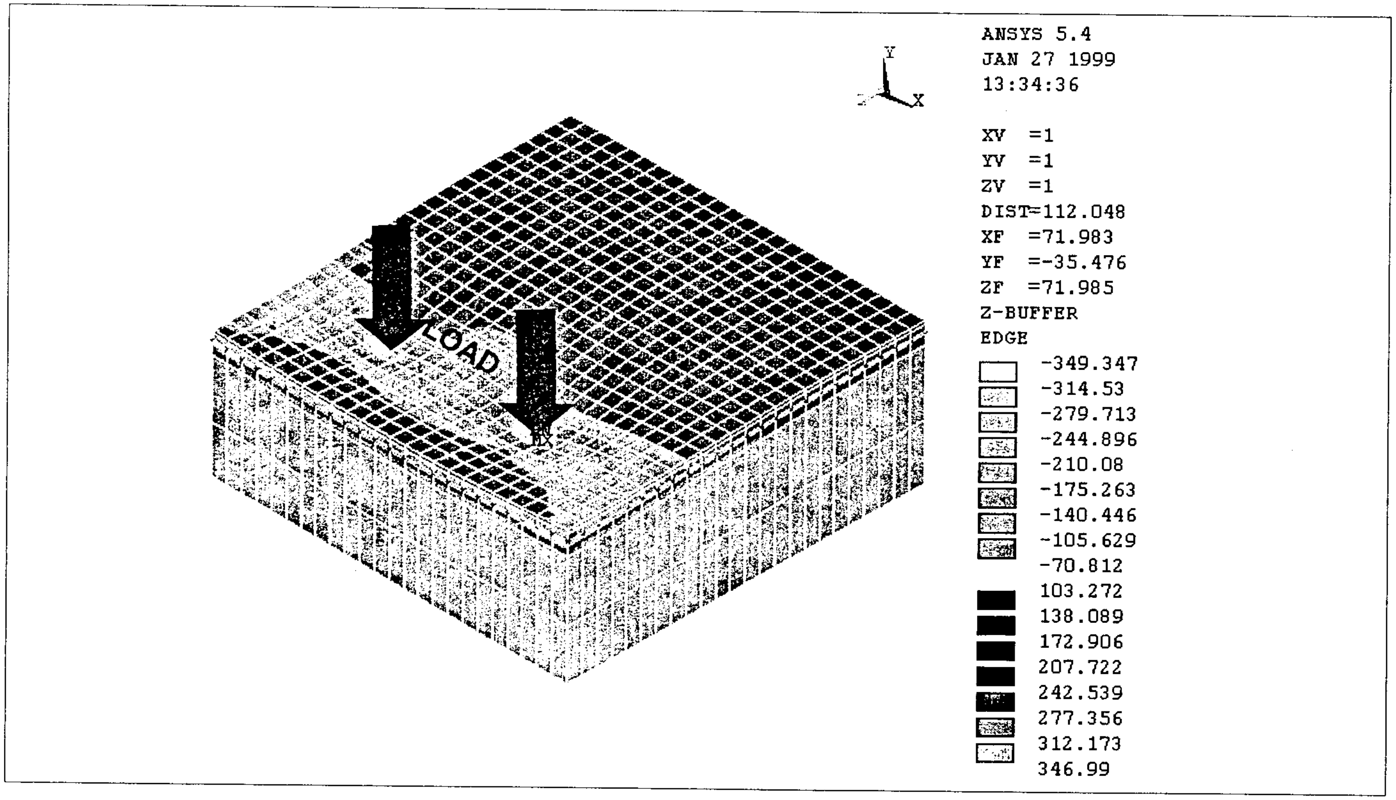



Figure 4.13 Stress (psi.) onTrack 1, Slab Size - 4' X 4' (Weak Bonding)

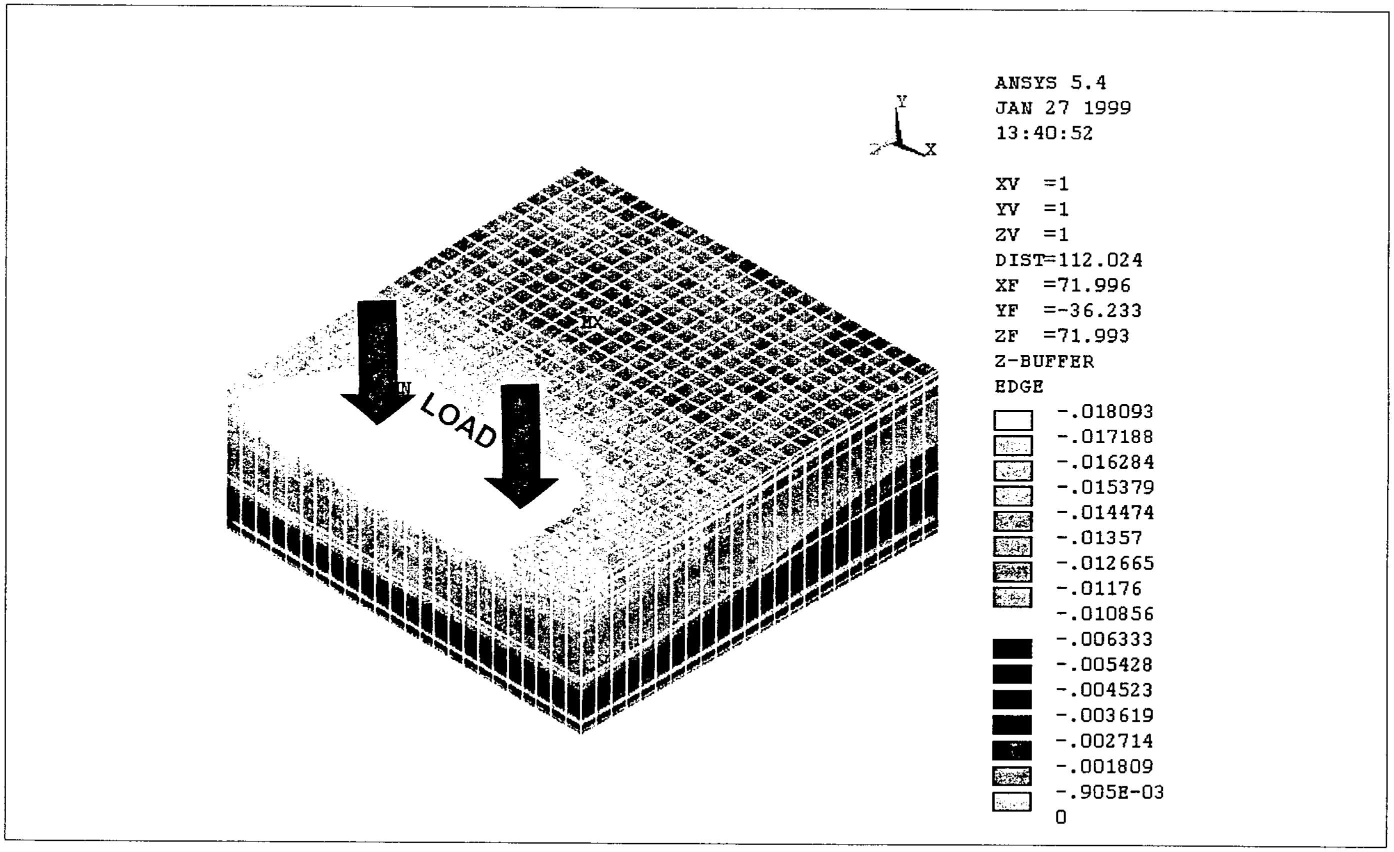



Figure 4.14 Vertical Deflection (in.) onTrack 1, Slab Size - 4' X 4' (Strong Bonding)

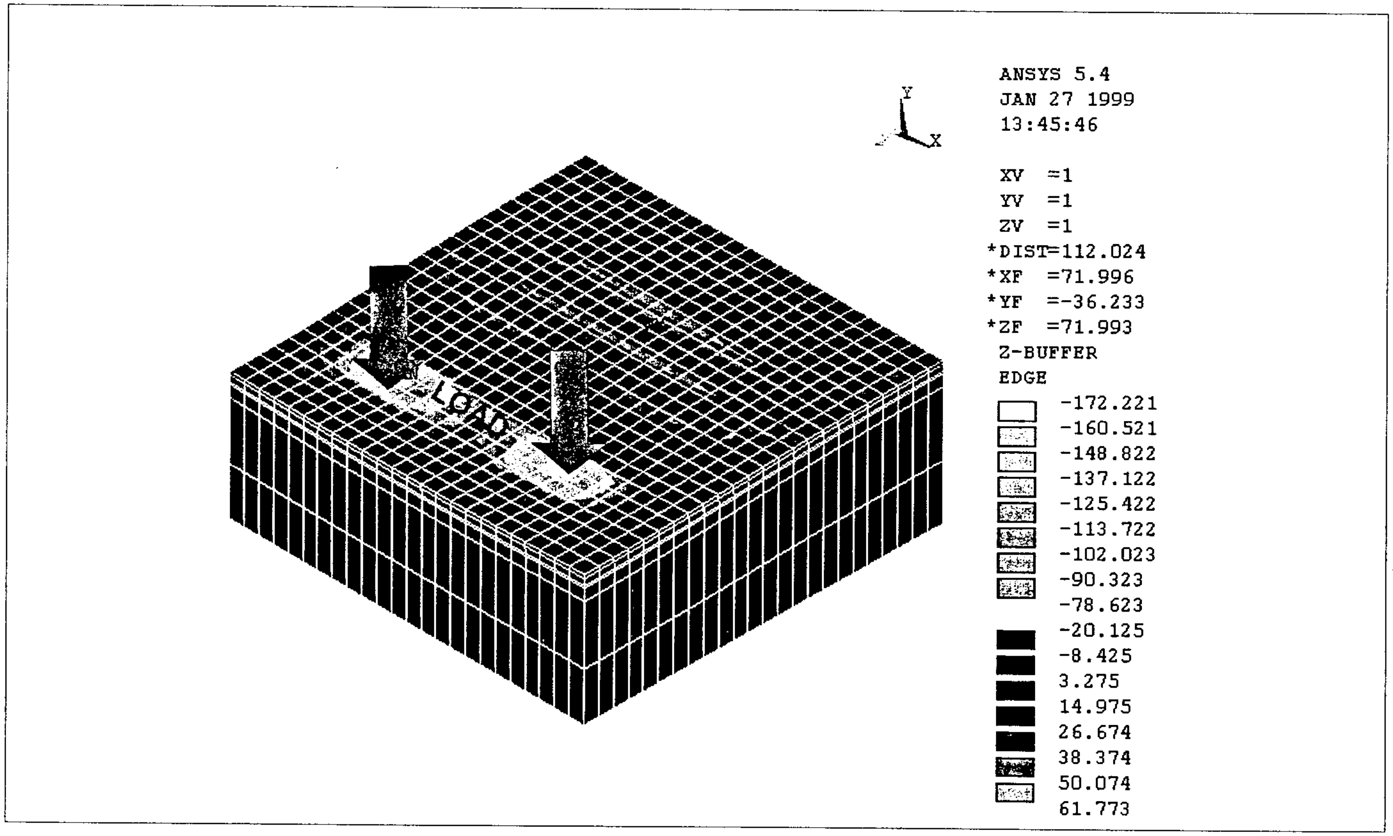



Figure 4.15 Stress (psi.) onTrack 1, Slab Size - 4' X 4' (Strong Bonding)

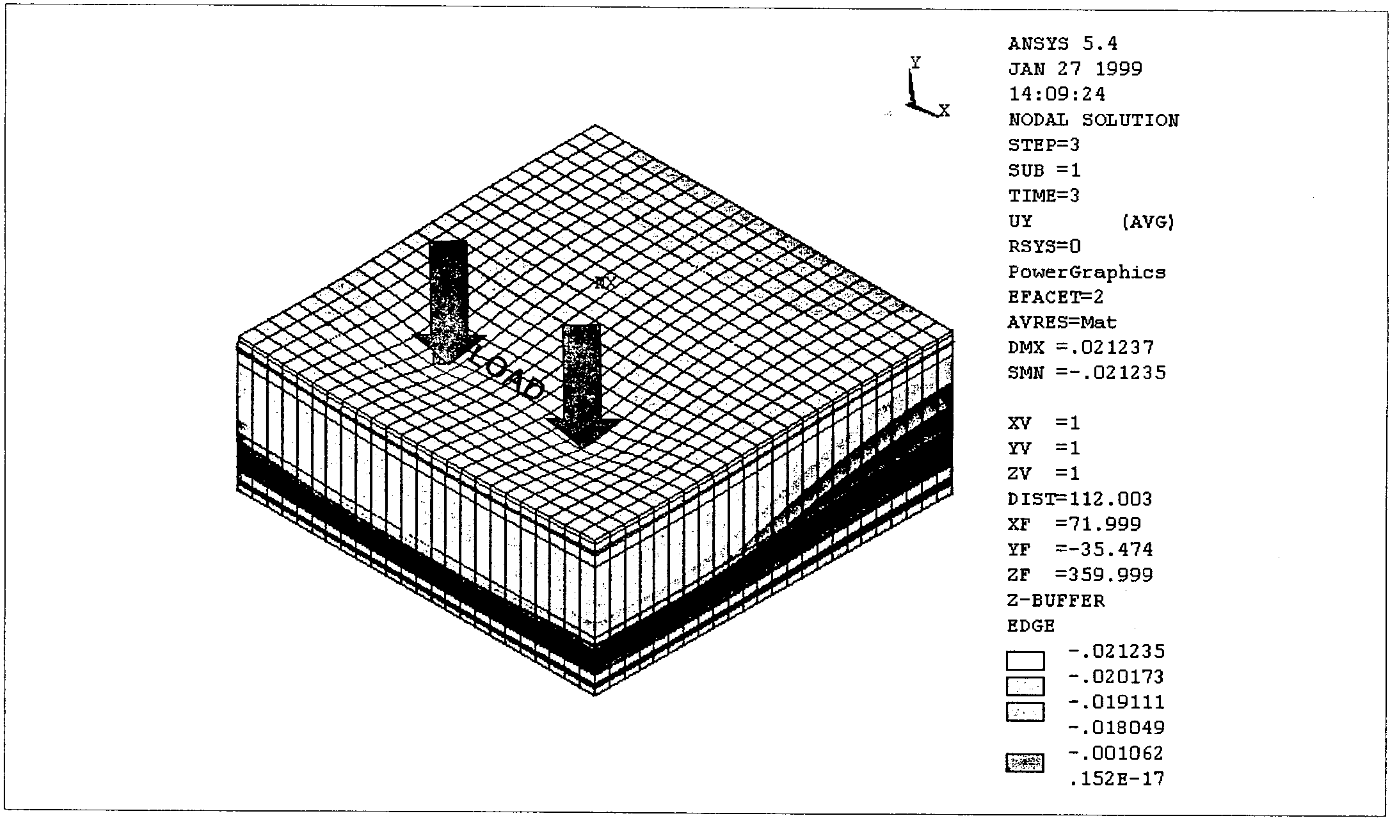



Figure 4.16 Vertical Deflection (in.) onTrack 1, Slab Size - 6' X 6' (Weak Bonding)

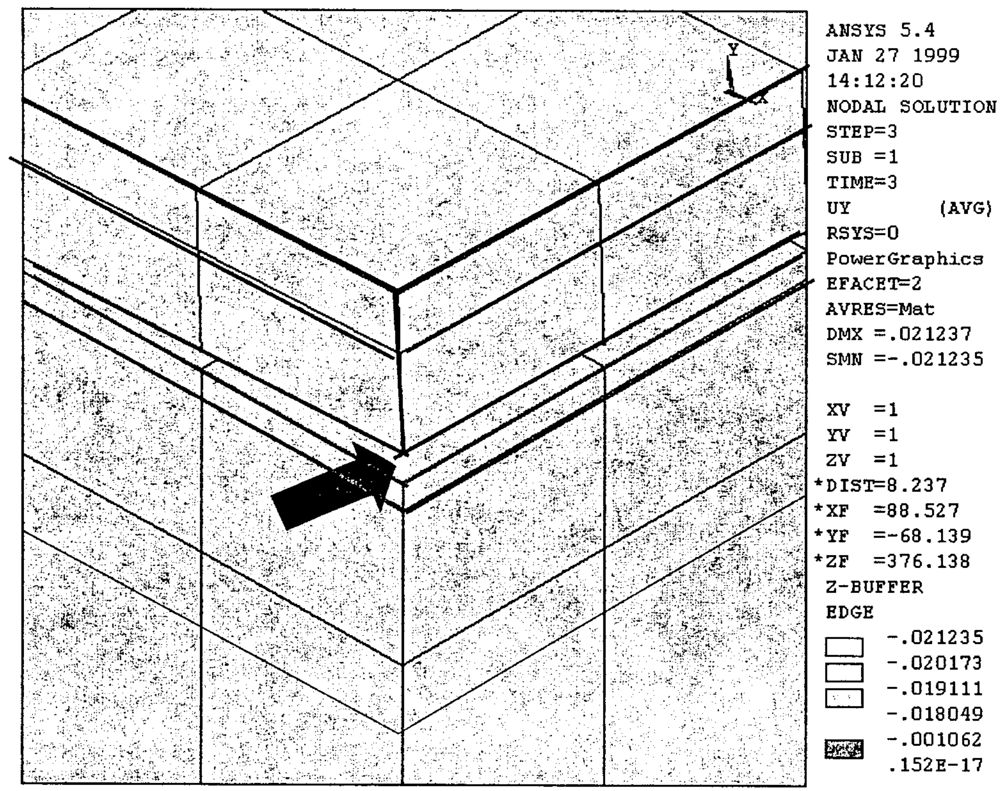



Figure 4.17 Prooping up of slab because of edge loading, Track # 1, 6' x 6' (Weak Bonding)

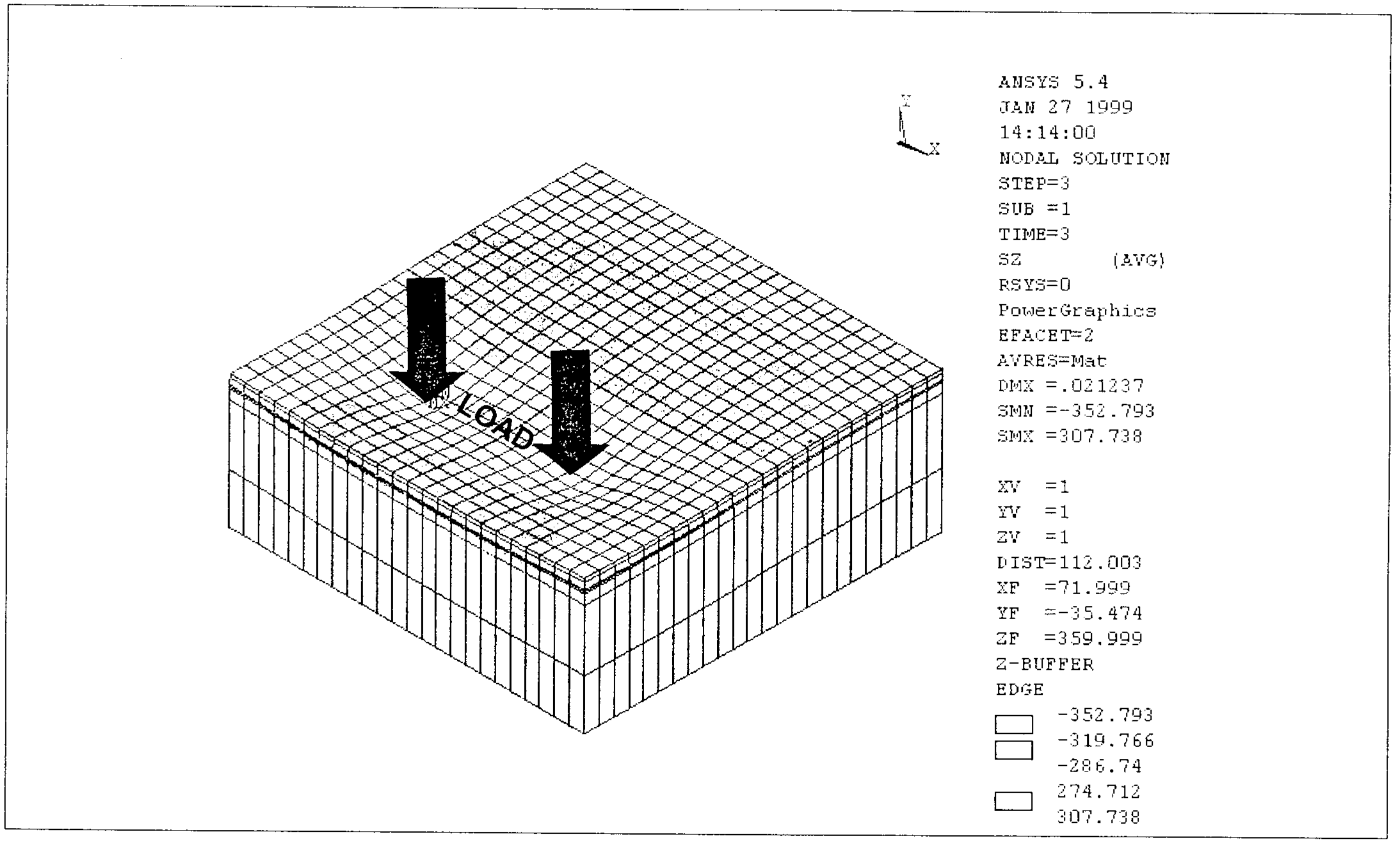



Figure 4.18 Stress (psi) onTrack 1, Slab Size - 6' X 6 (Weak Bonding)

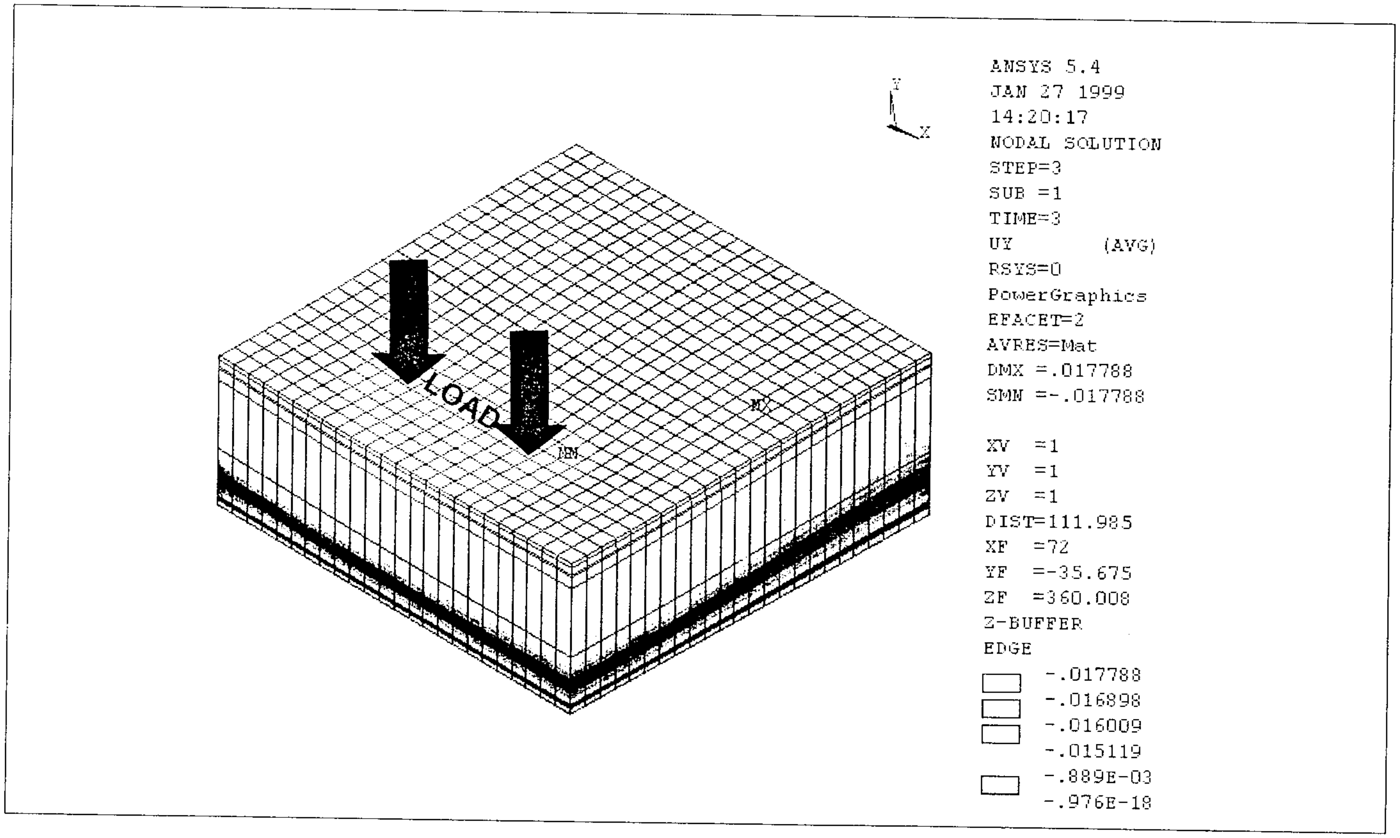



Figure 4.19 Vertical Deflection (in.) onTrack 1, Slab Size - 6' X 6' (Strong Bonding)

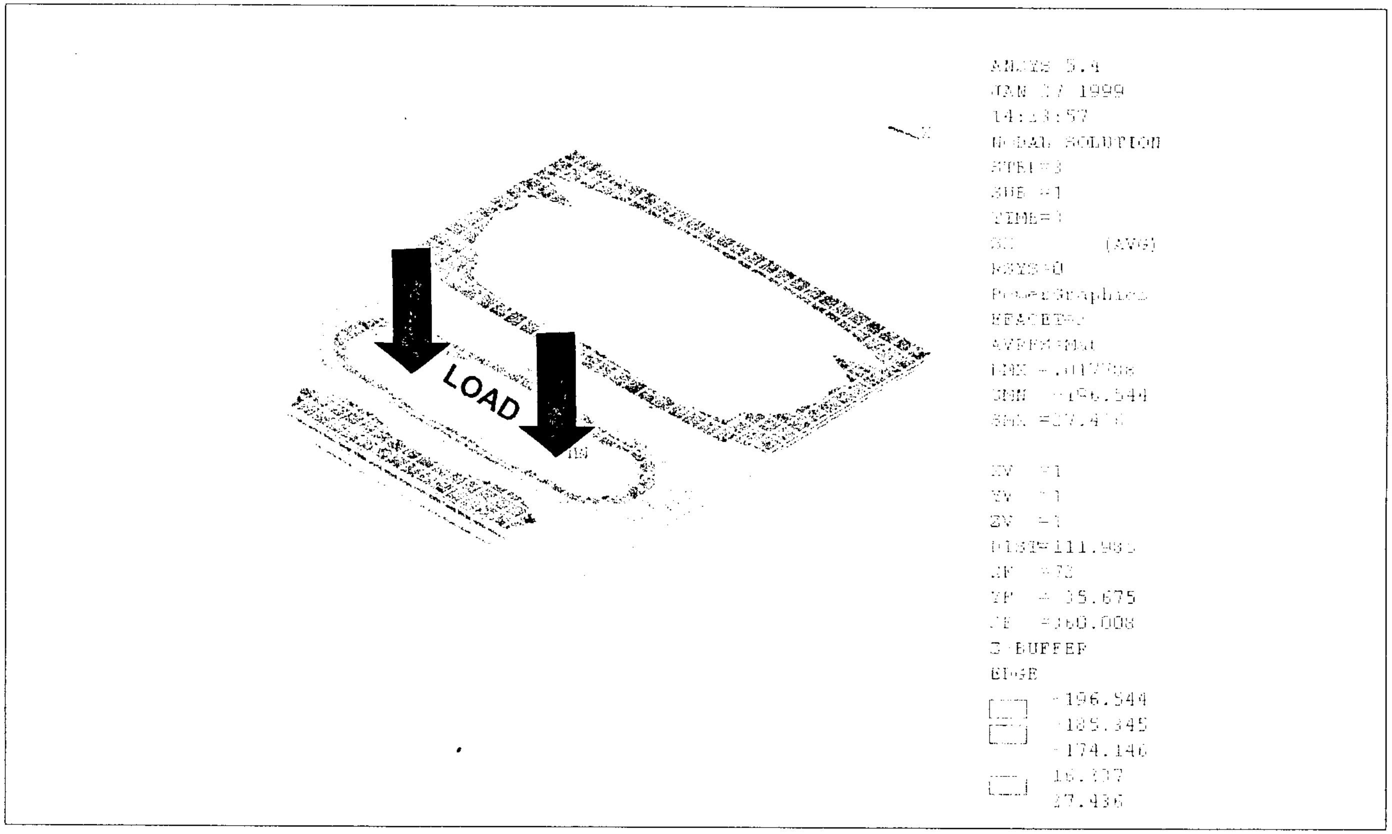



Figure 4.20 Stress (psi) onTrack 1, Slab Size - 6' X 6' (Strong Bonding)

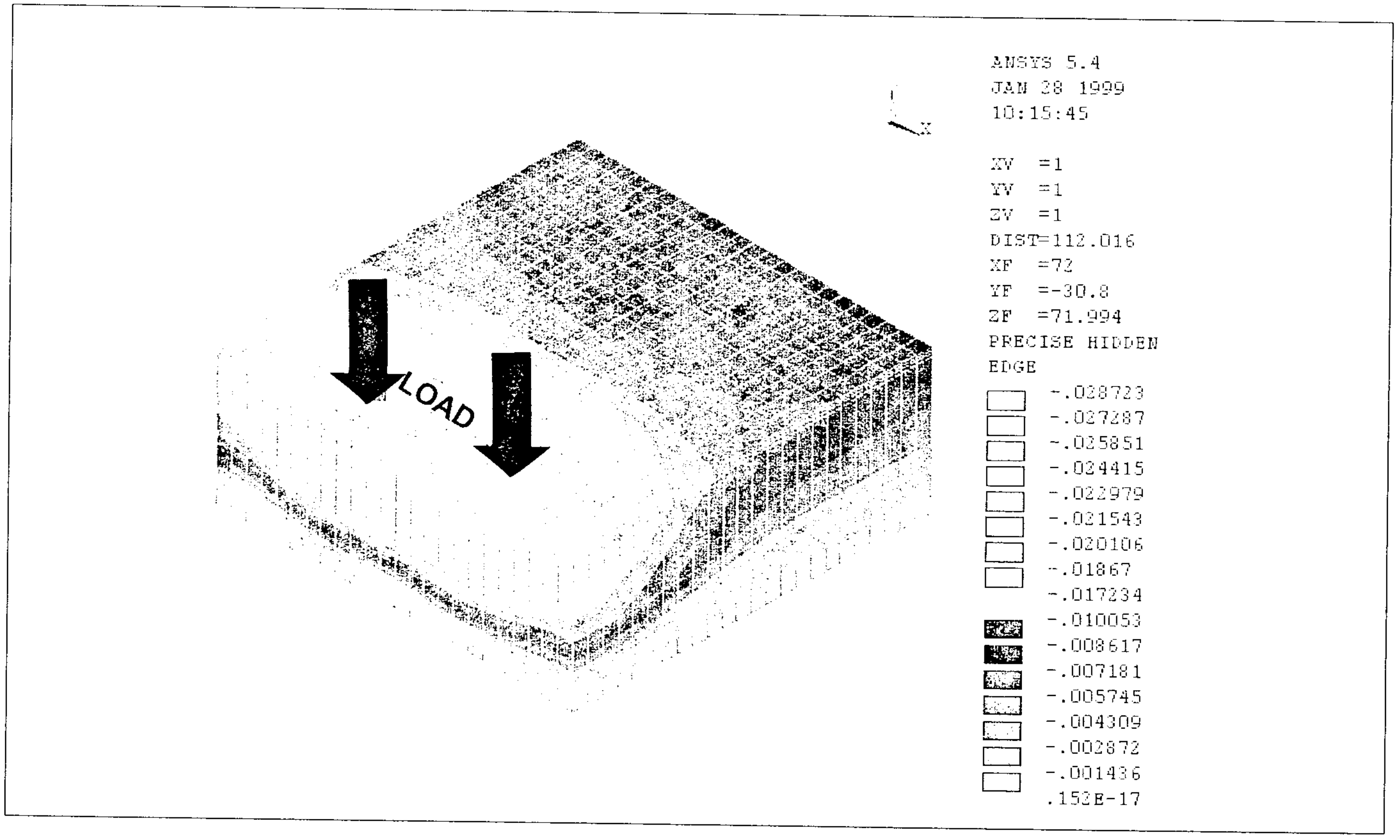



Figure 4.21 Vertical Deflection (in.) onTrack 2, Slab Size - 3' X 3' (Weak Bonding)

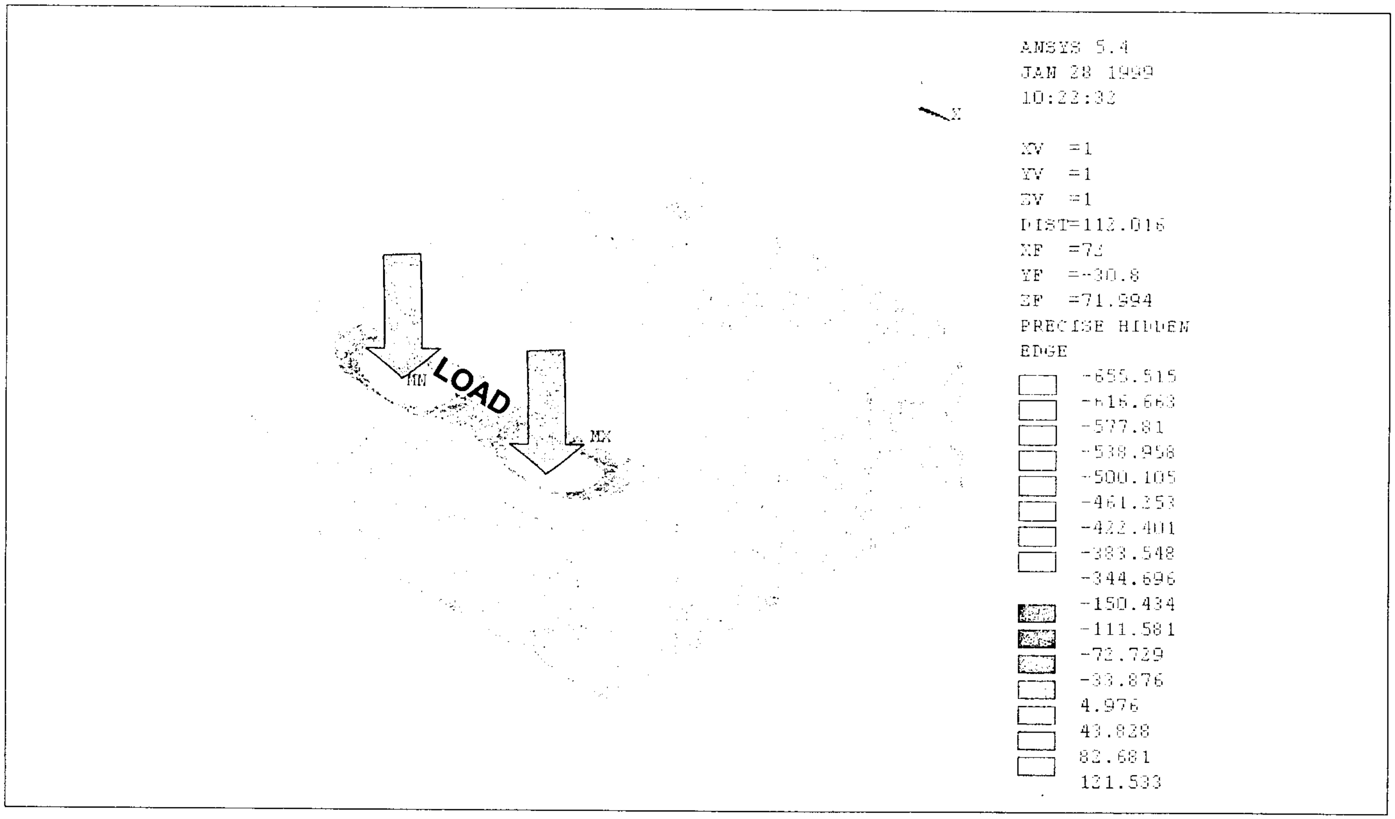



Figure 4.22 Stress (psi) onTrack 2, Slab Size - 3' X 3' (Weak Bonding)

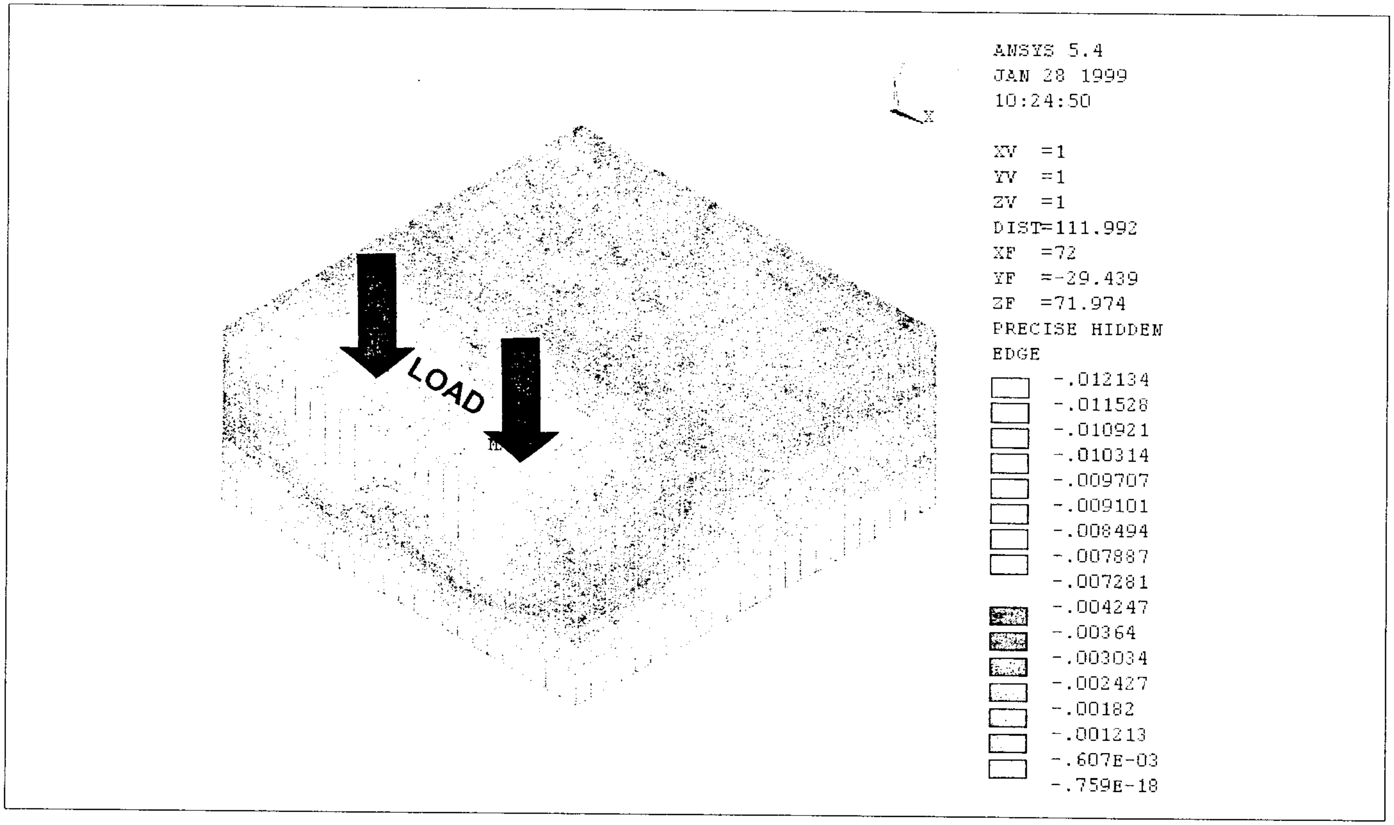



Figure 4.23 Vertical Deflection (in.) onTrack 2, Slab Size - 3' X 3' (Strong Bonding)

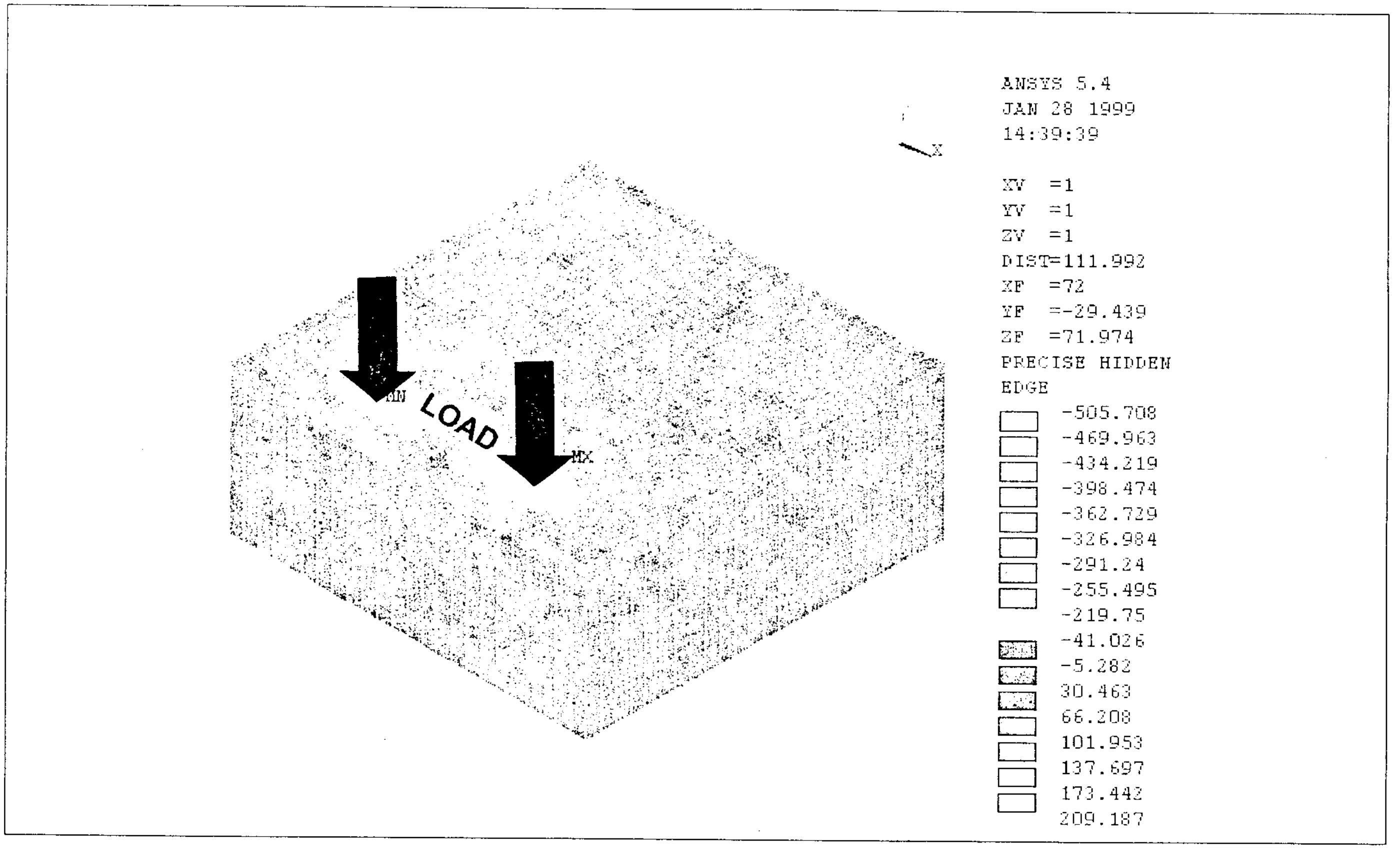



Figure 4.24 Stress (psi) onTrack 2, Slab Size - 3' X 3' (Strong Bonding)

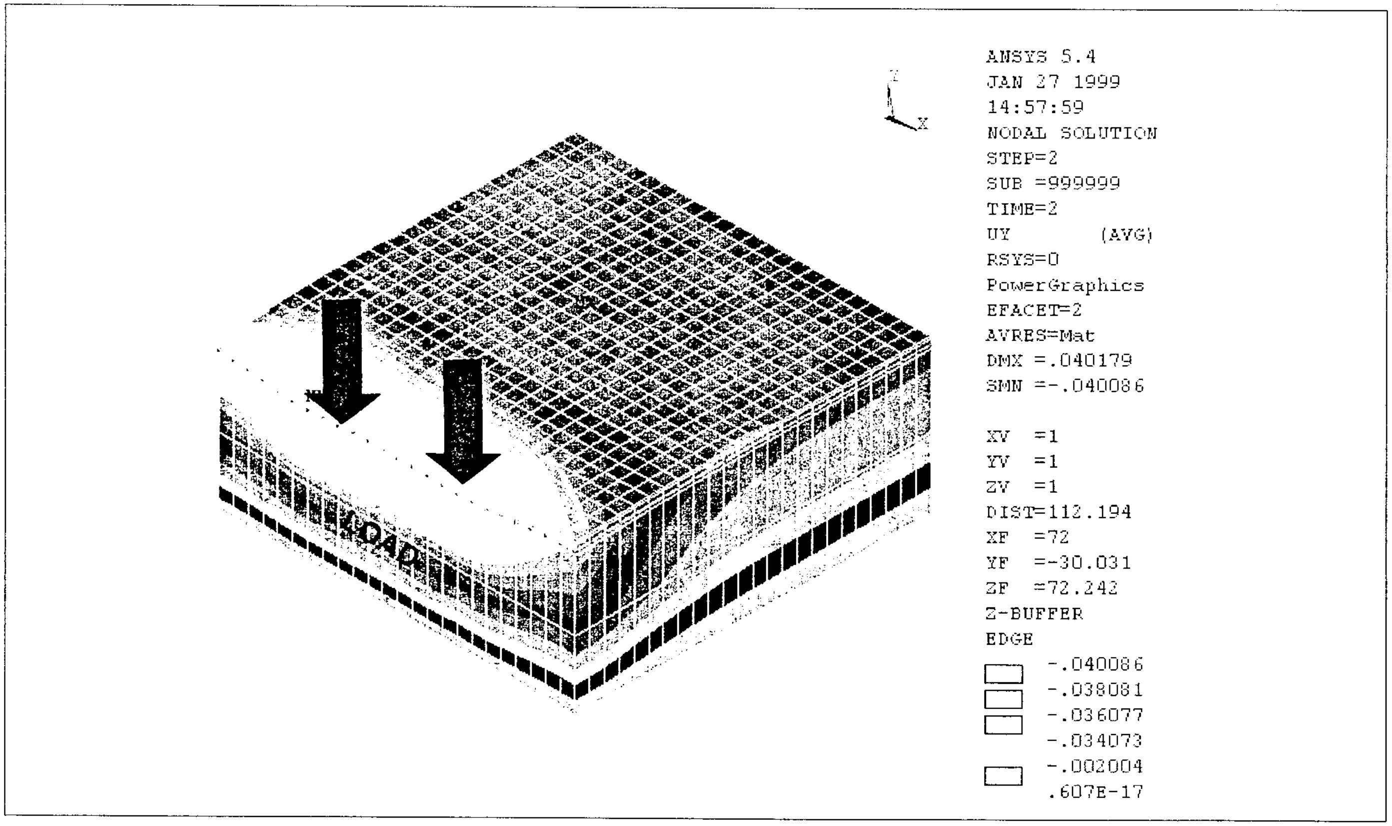



Figure 4.25 Vertical Deflection (in.) onTrack 3, Slab Size - 4' X 4' (Weak Bonding)

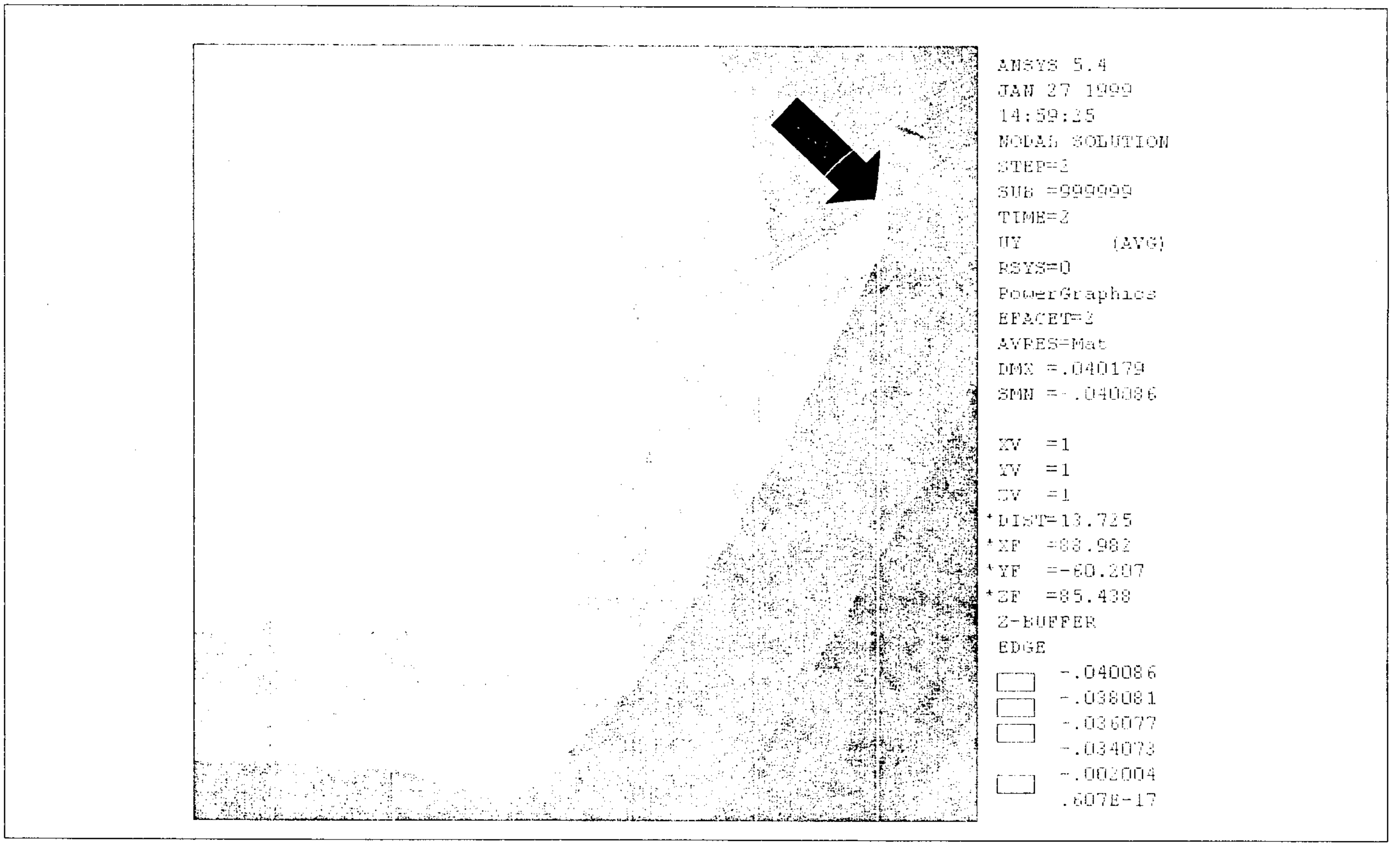



Figure 4.26 Lifting up of Slab Because of Edge Loading, Track 3, Slab Size - 4' X 4' (Weak Bonding)

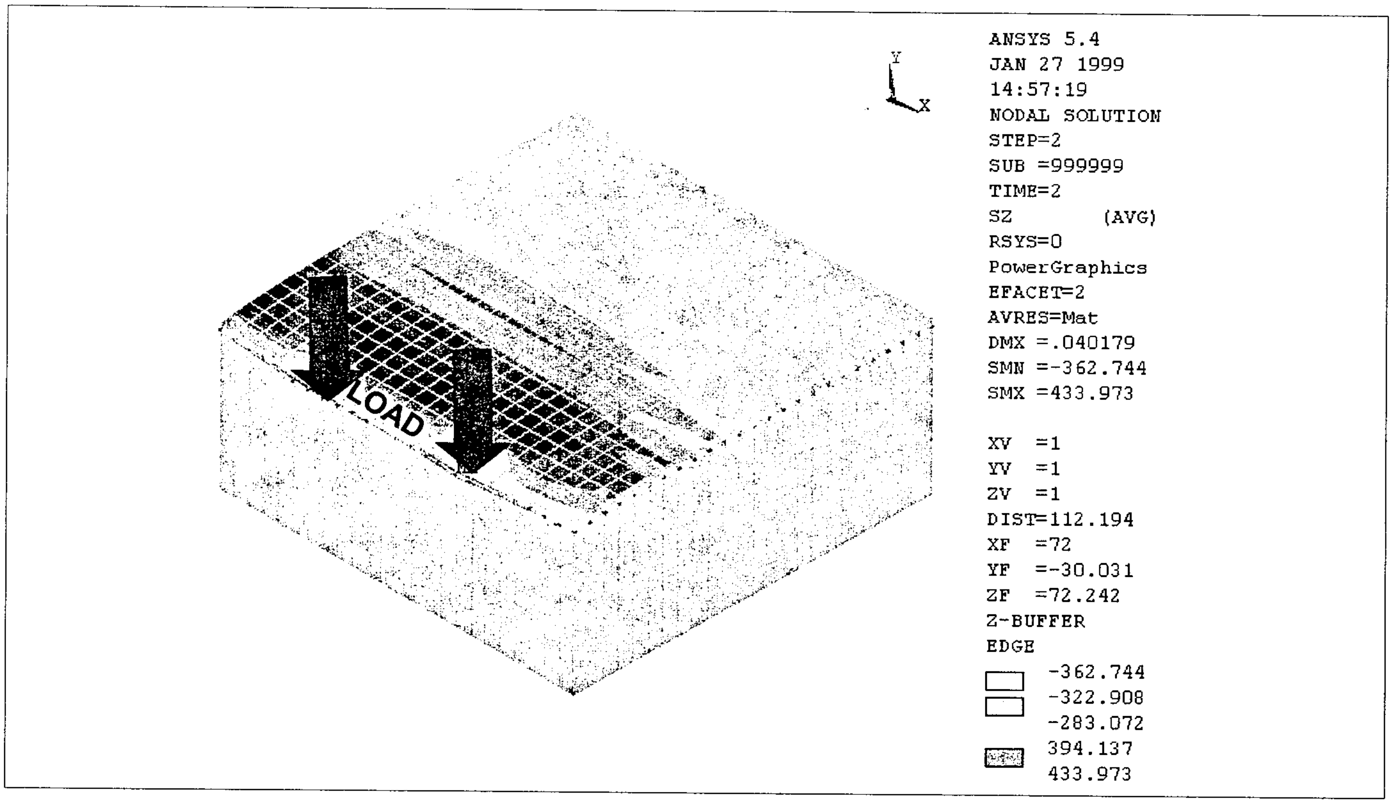



Figure 4.27 Stress (psi) onTrack 3, Slab Size - 4' X 4' (Weak Bonding)

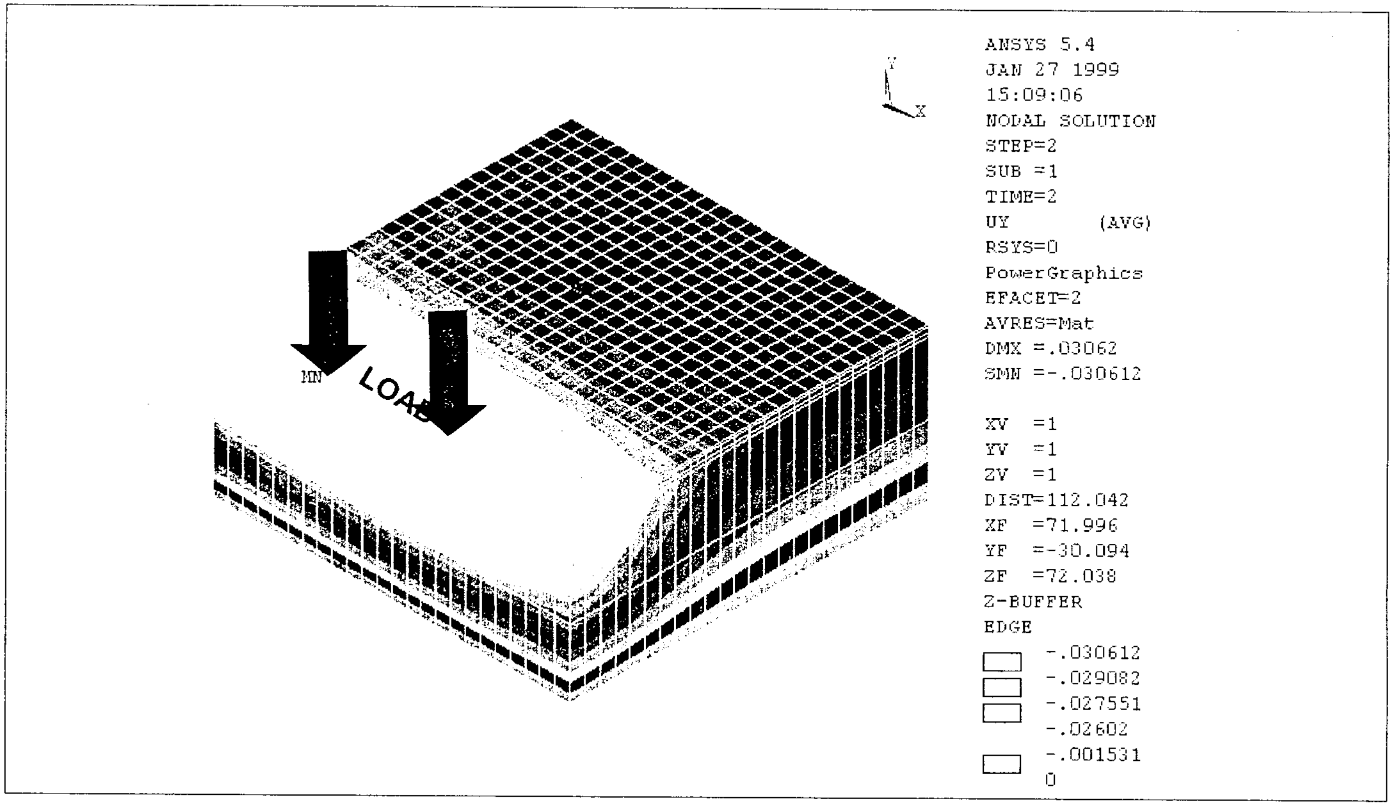



Figure 4.28 Vertical Deflection (in.) onTrack 3, Slab Size - 4' X 4' (Strong Bonding)

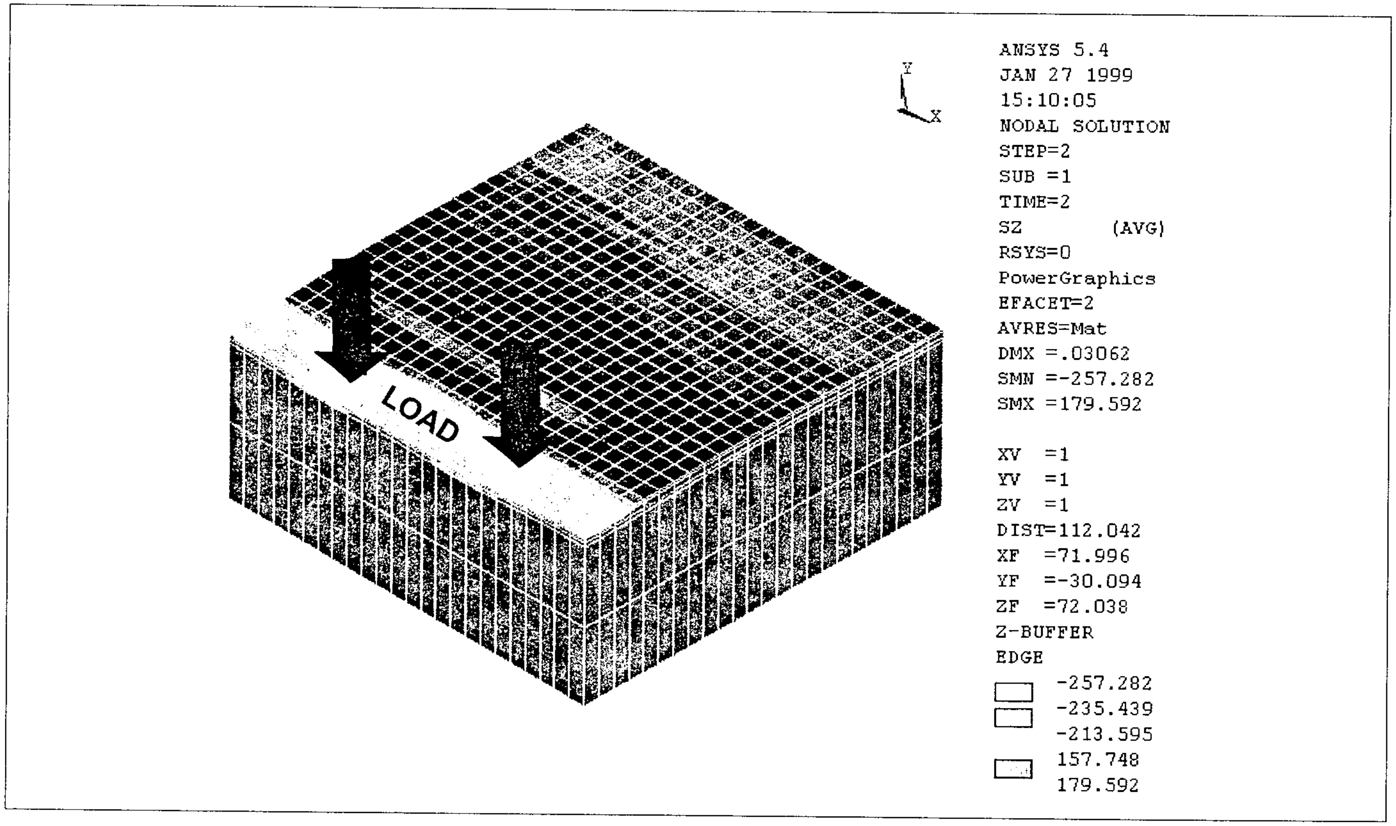



Figure 4.29 Stress (psi) onTrack 3, Slab Size - 4' X 4' (Strong Bonding)

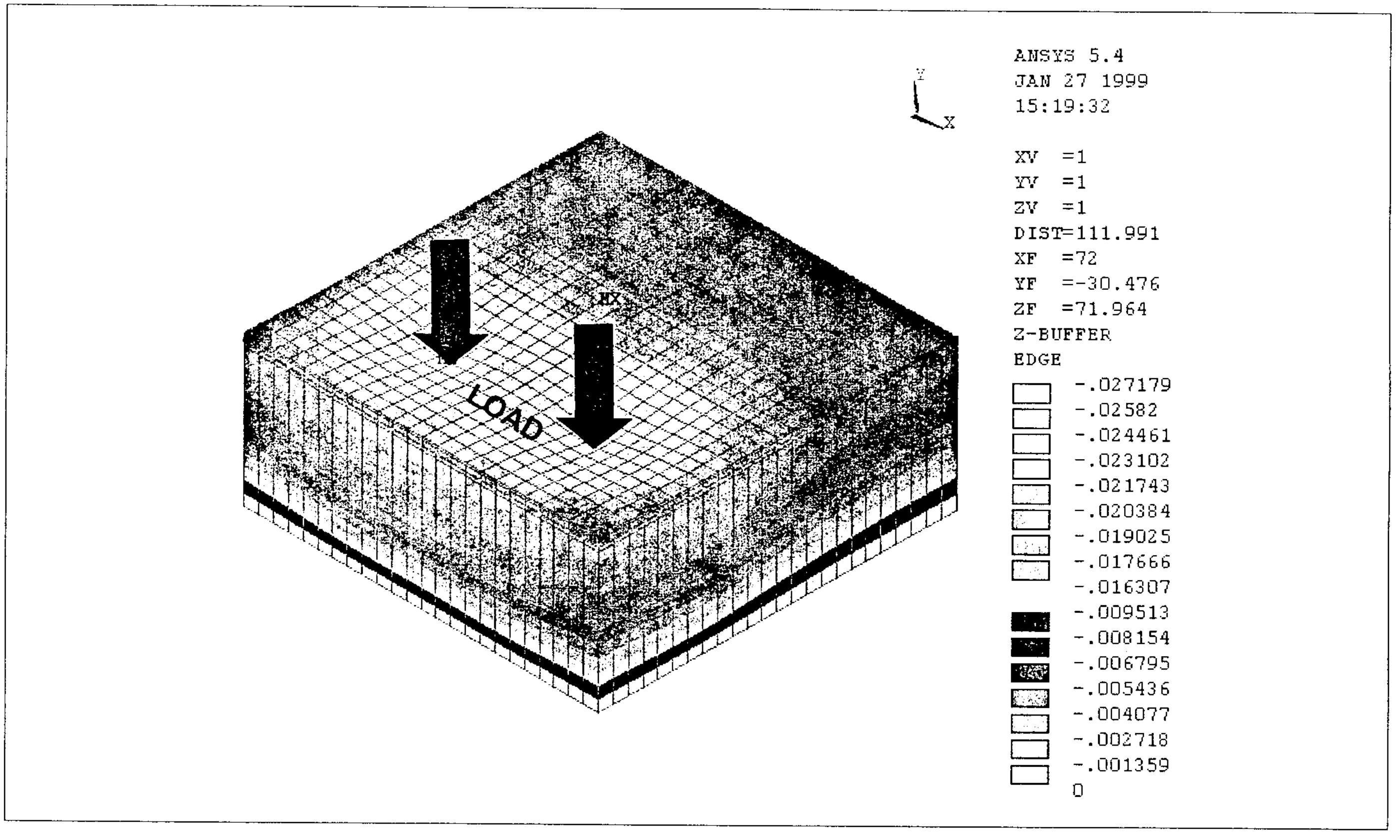



Figure 4.30 Vertical Deflection (in.) onTrack 3, Slab Size - 6' X 6' (Weak Bonding)

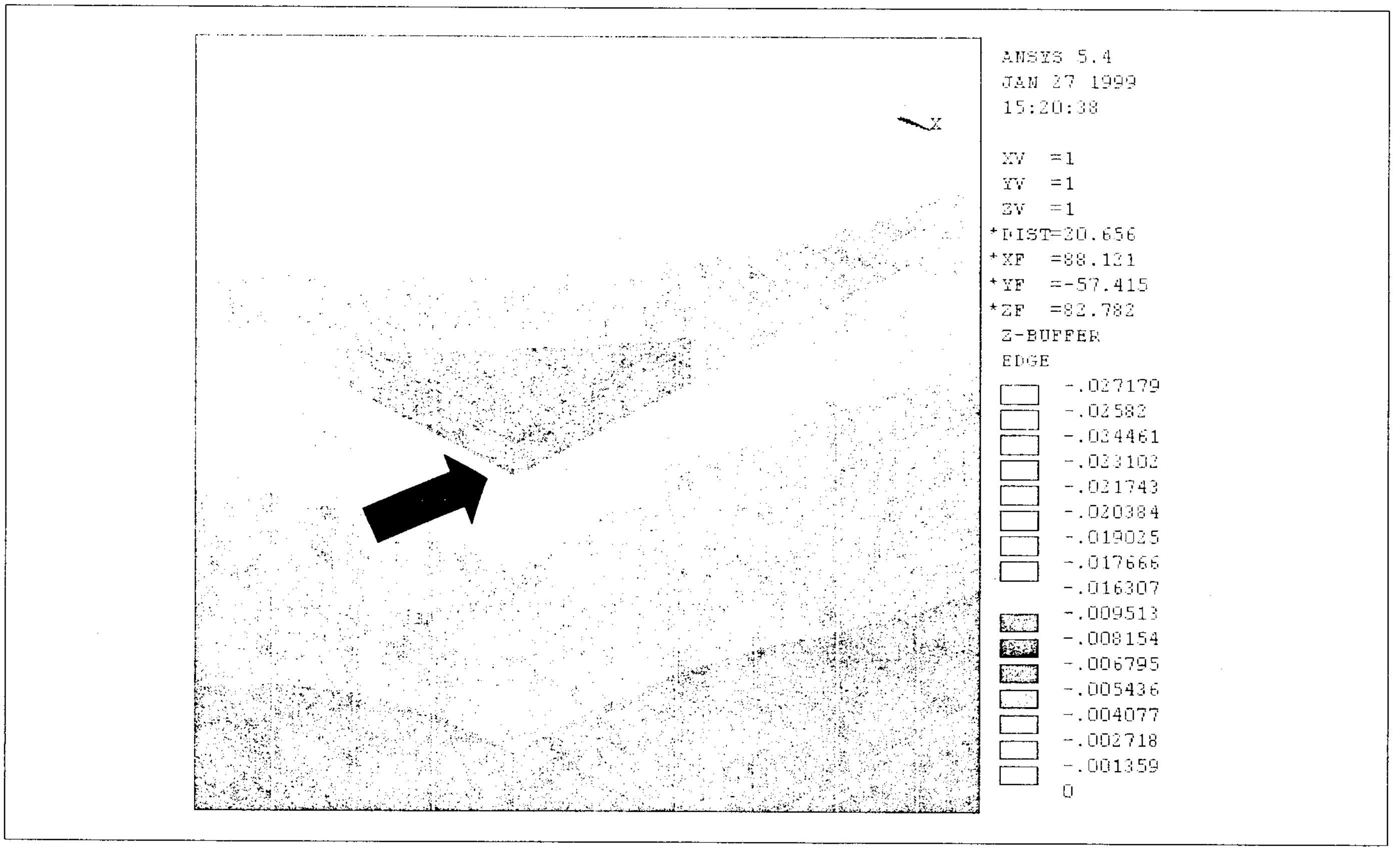



Figure 4.31 Propping up of Slab Becase of Edge Loading,Track 3, Slab Size - 6' X 6' (Weak Bonding)

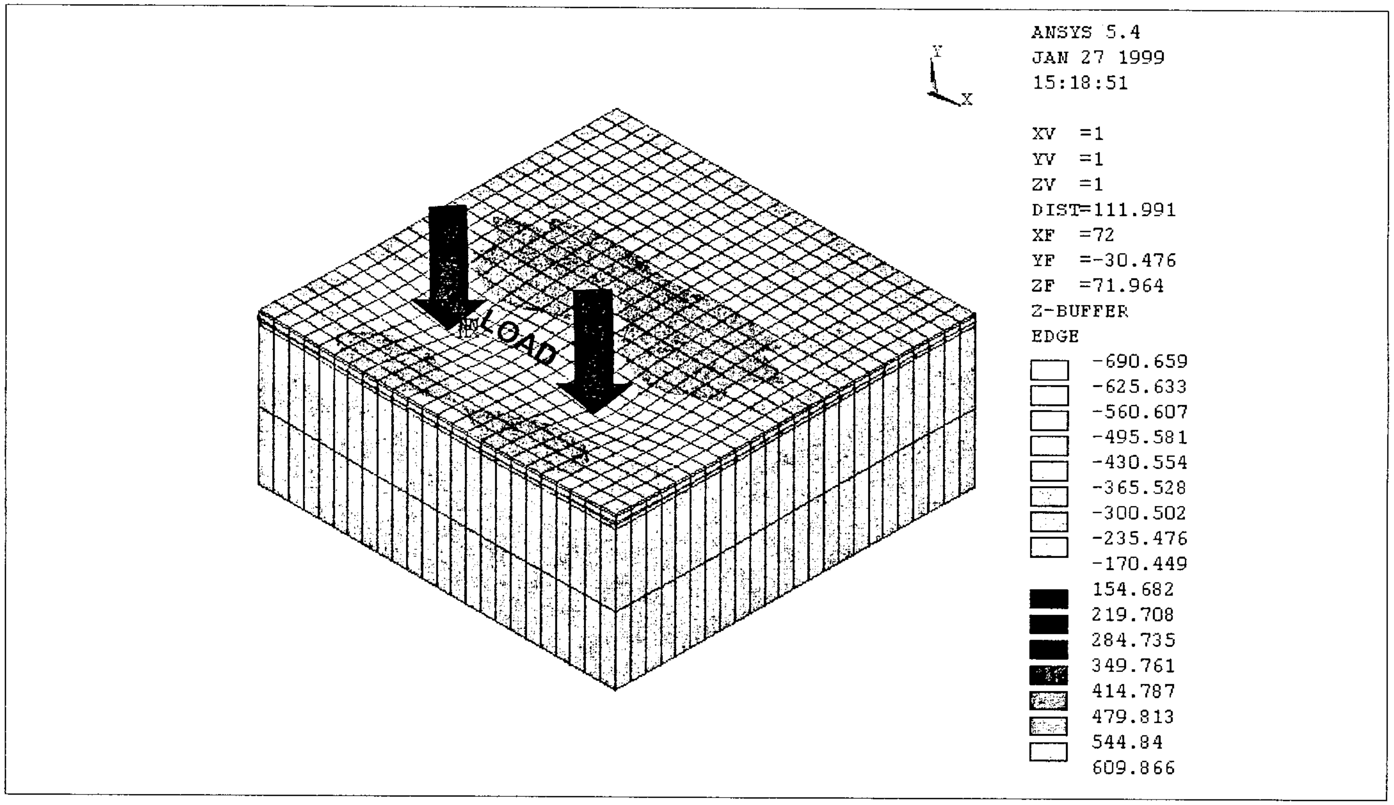



Figure 4.32 Stress (psi) onTrack 3, Slab Size - 6' X 6' (Weak Bonding)

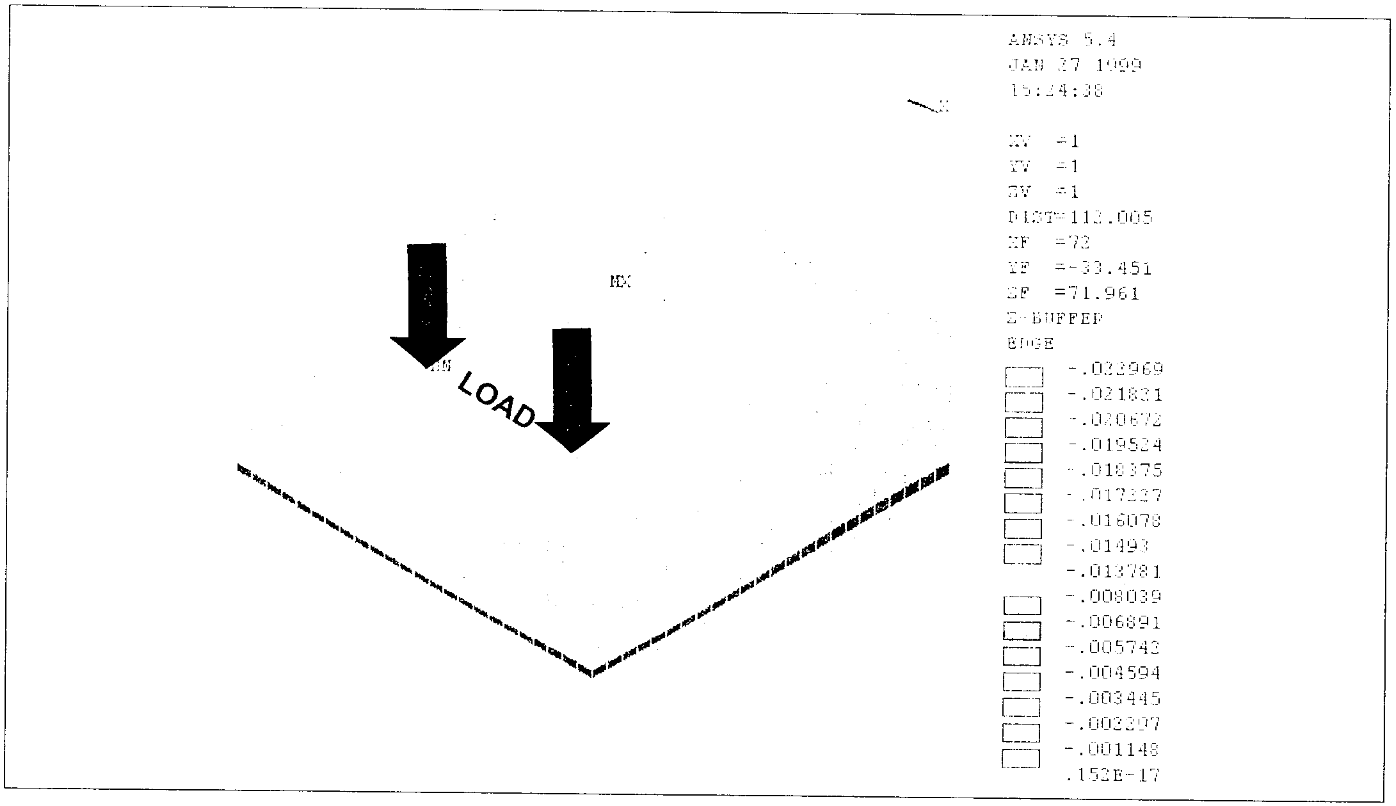



Figure 4.33 Vertical Deflection (in.) onTrack 3, Slab Size - 6' X 6' (Strong Bonding)

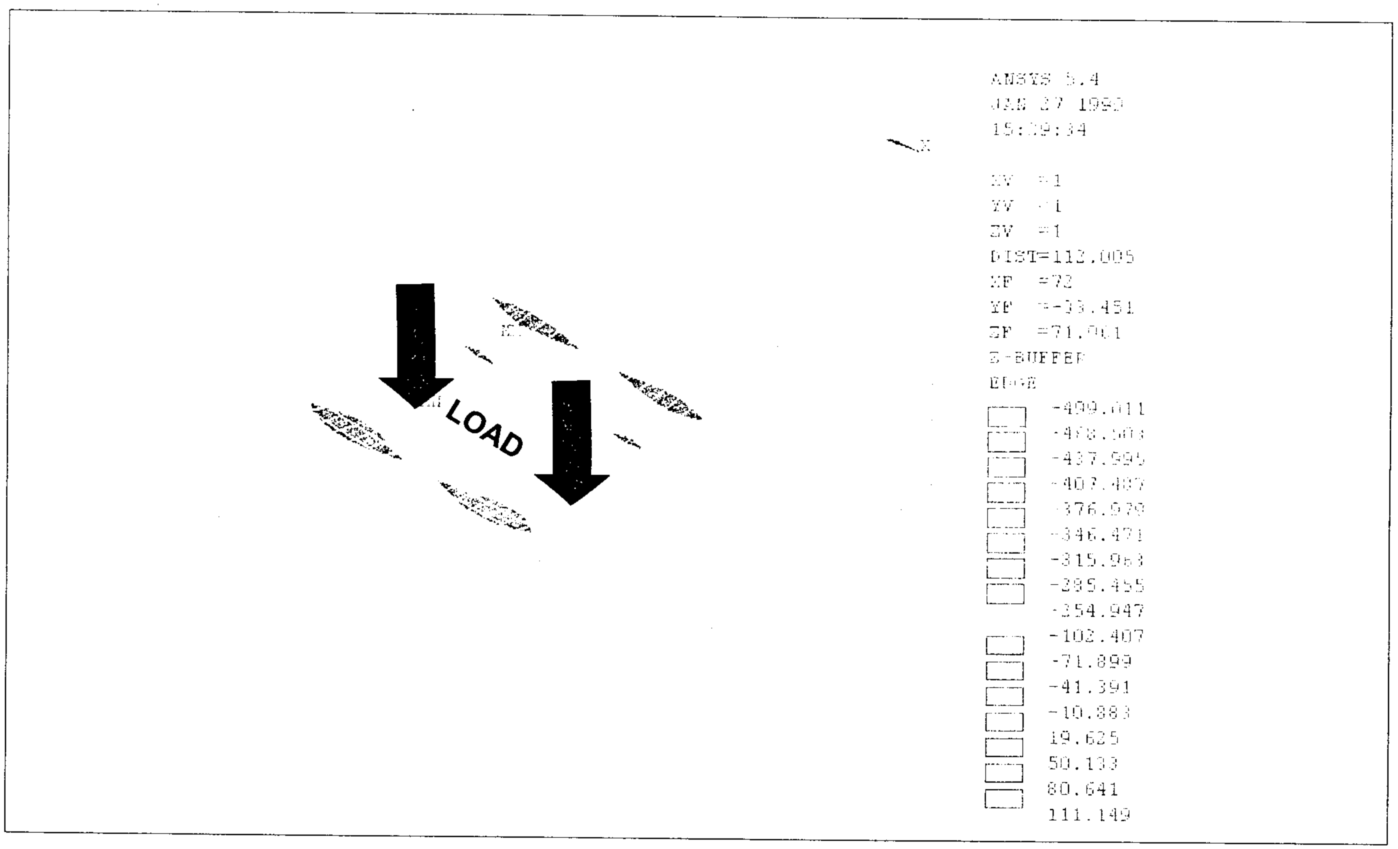



Figure 4.34 Stress (psi) onTrack 3, Slab Size - 6' X 6' (Strong Bonding)

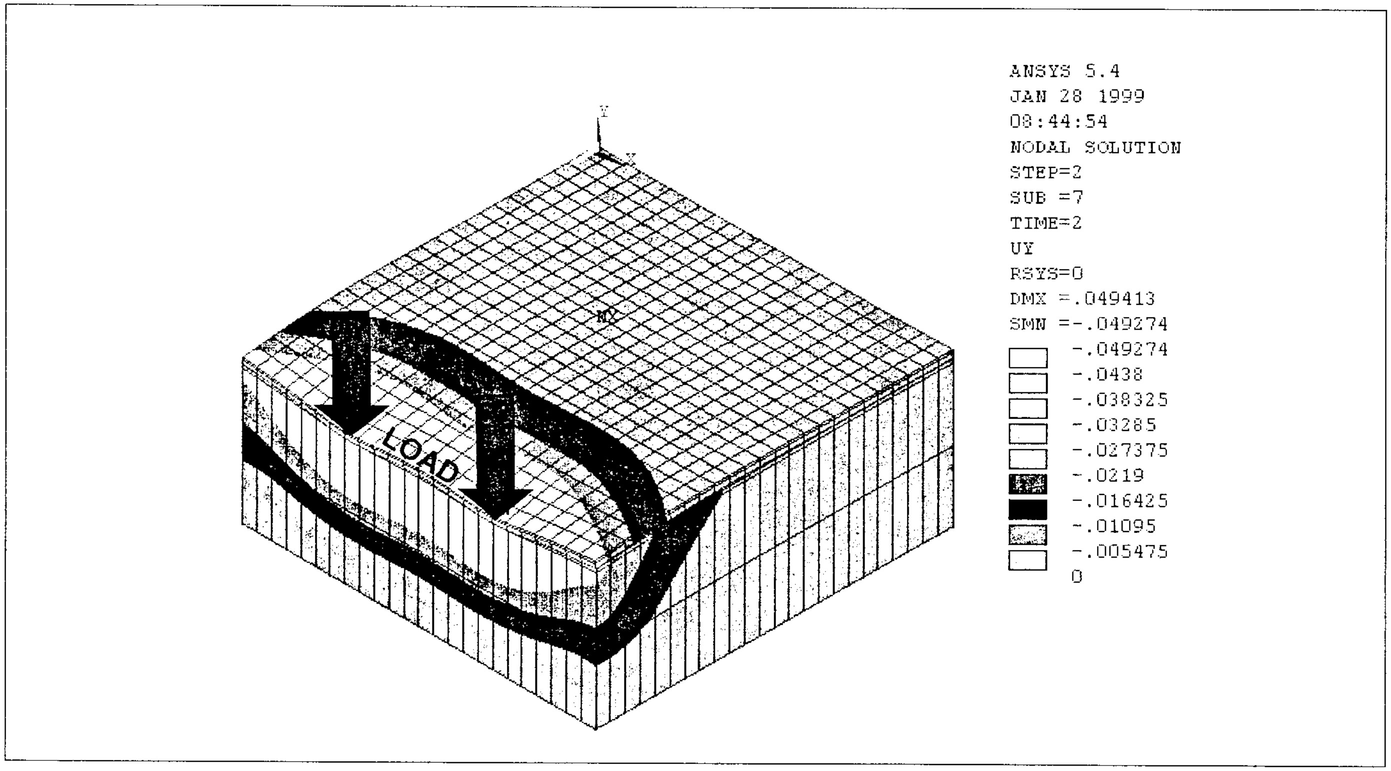



Figure 4.35 Vertical Deflection (in.) onTrack 3, Slab Size - 12' X 12' (Weak Bonding)

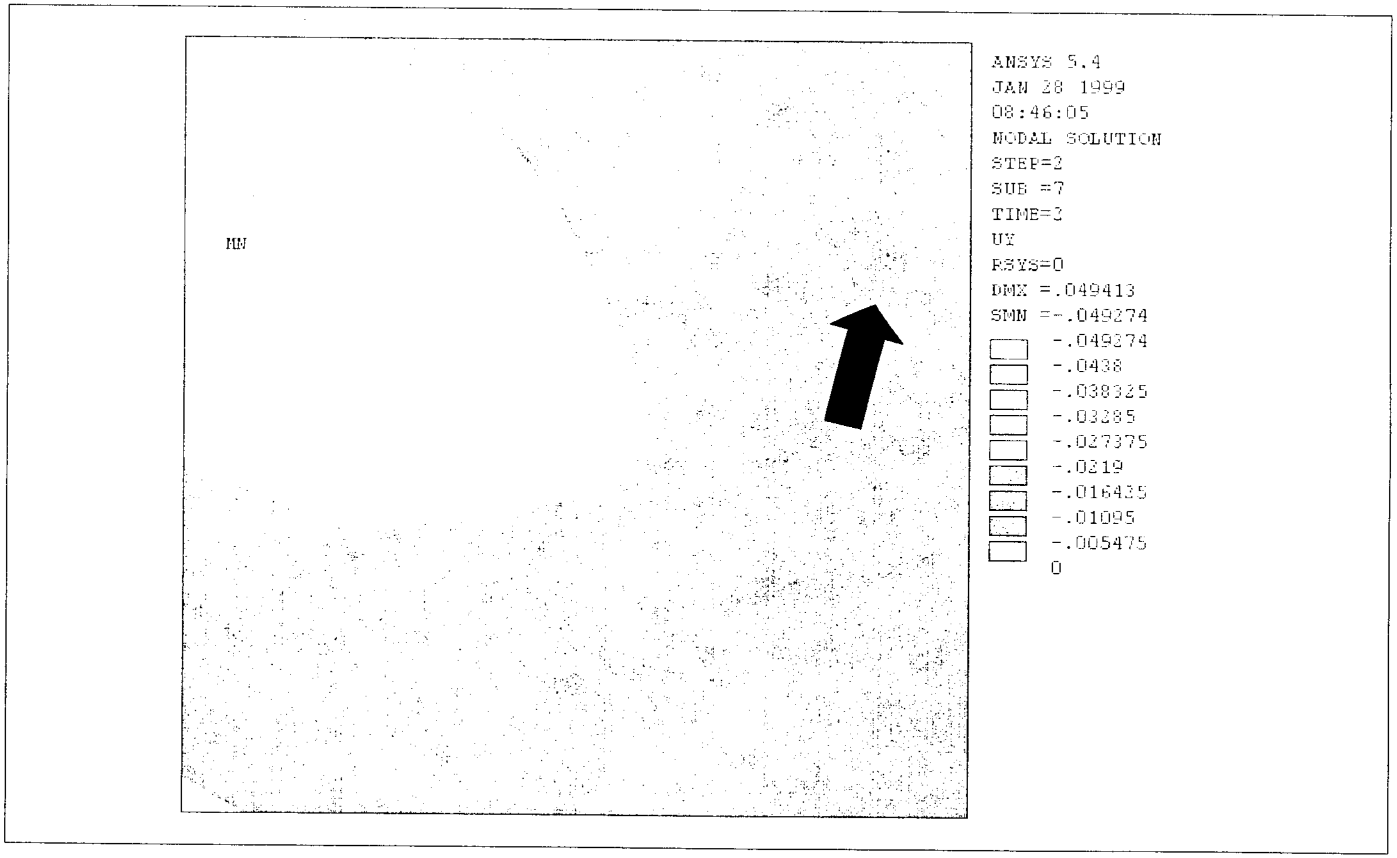



Figure 4.36 Propping Up of Slab Because of Edge Loading Track 3, Slab Size - 12' X 12' (Weak Bonding)

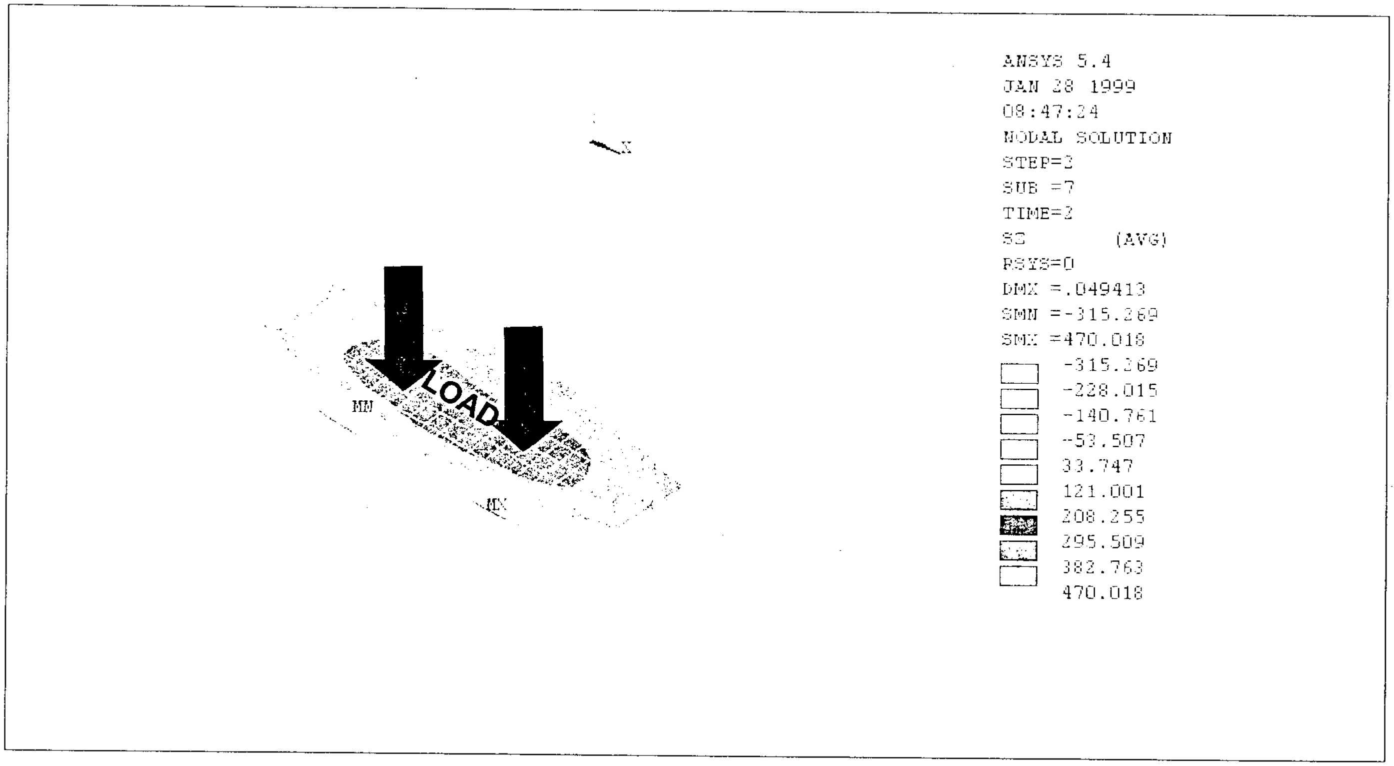



Figure 4.37 Stress (psi) onTrack 3, Slab Size - 12' X 12' (Weak Bonding)

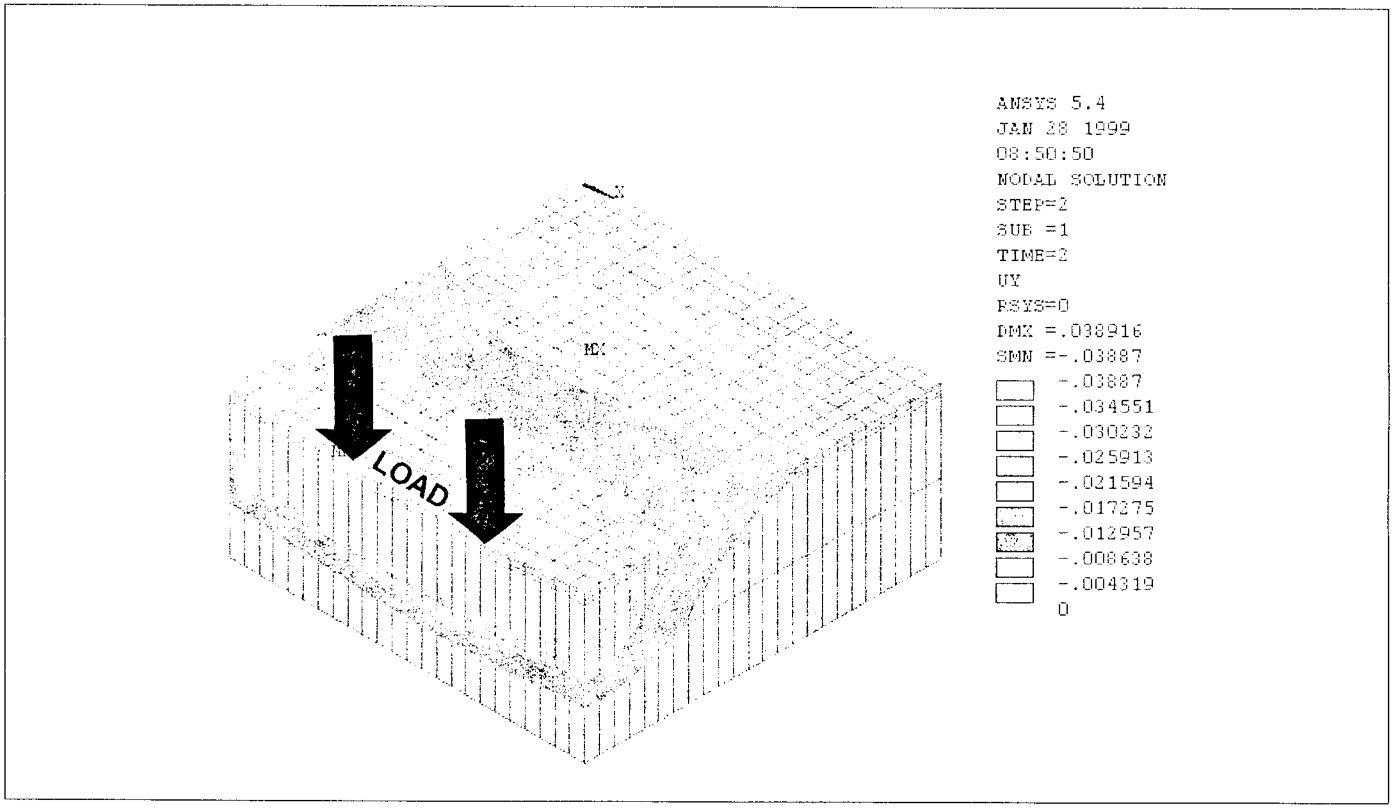



Figure 4.38 Vertical Deflection (in.) onTrack 3, Slab Size - 12' X 12' (Strong Bonding)

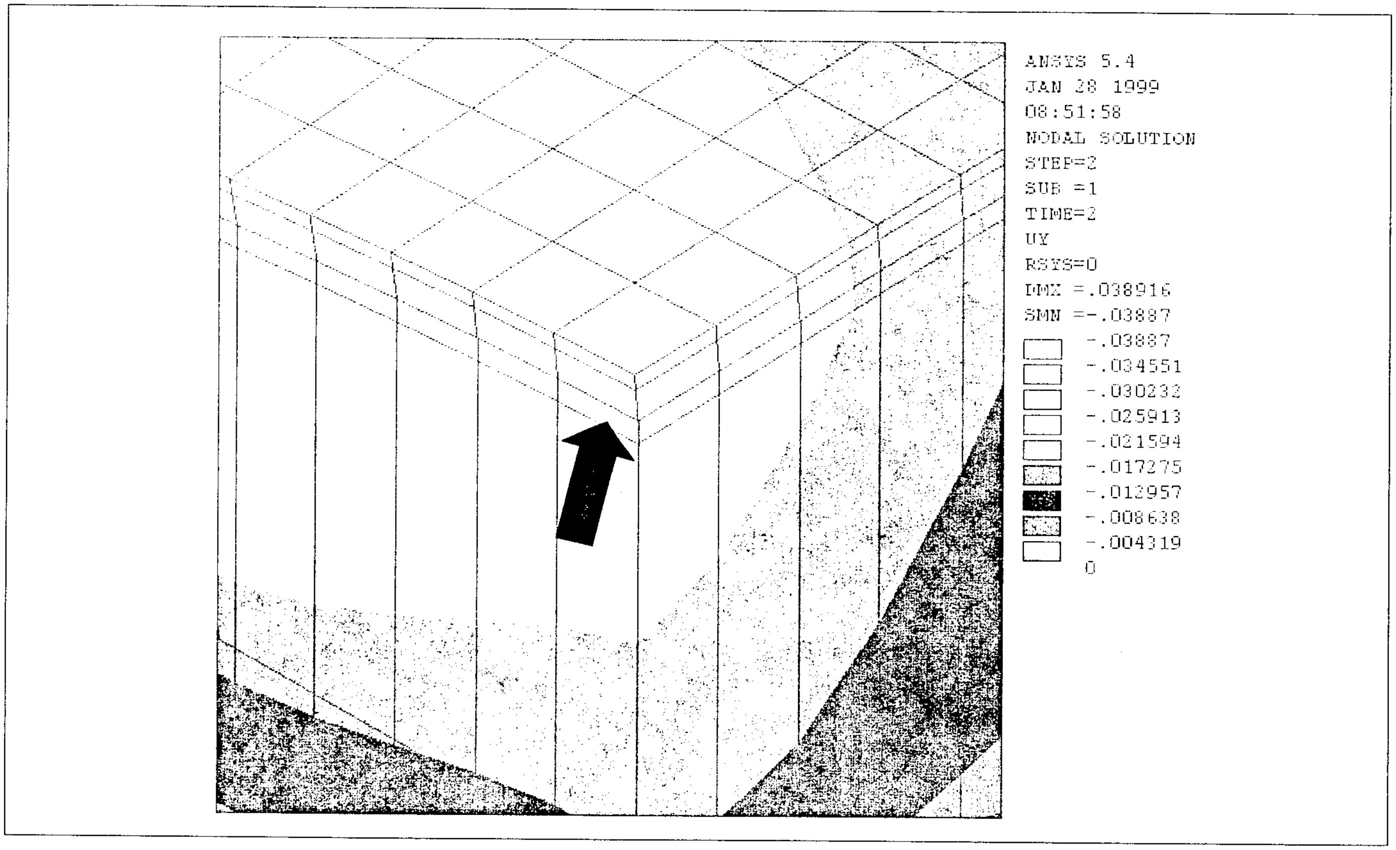



Figure 4.39 No Propping Up of Slab Because of Edge Loading Track 3, Slab Size - 12' X 12' (Strong Bonding)

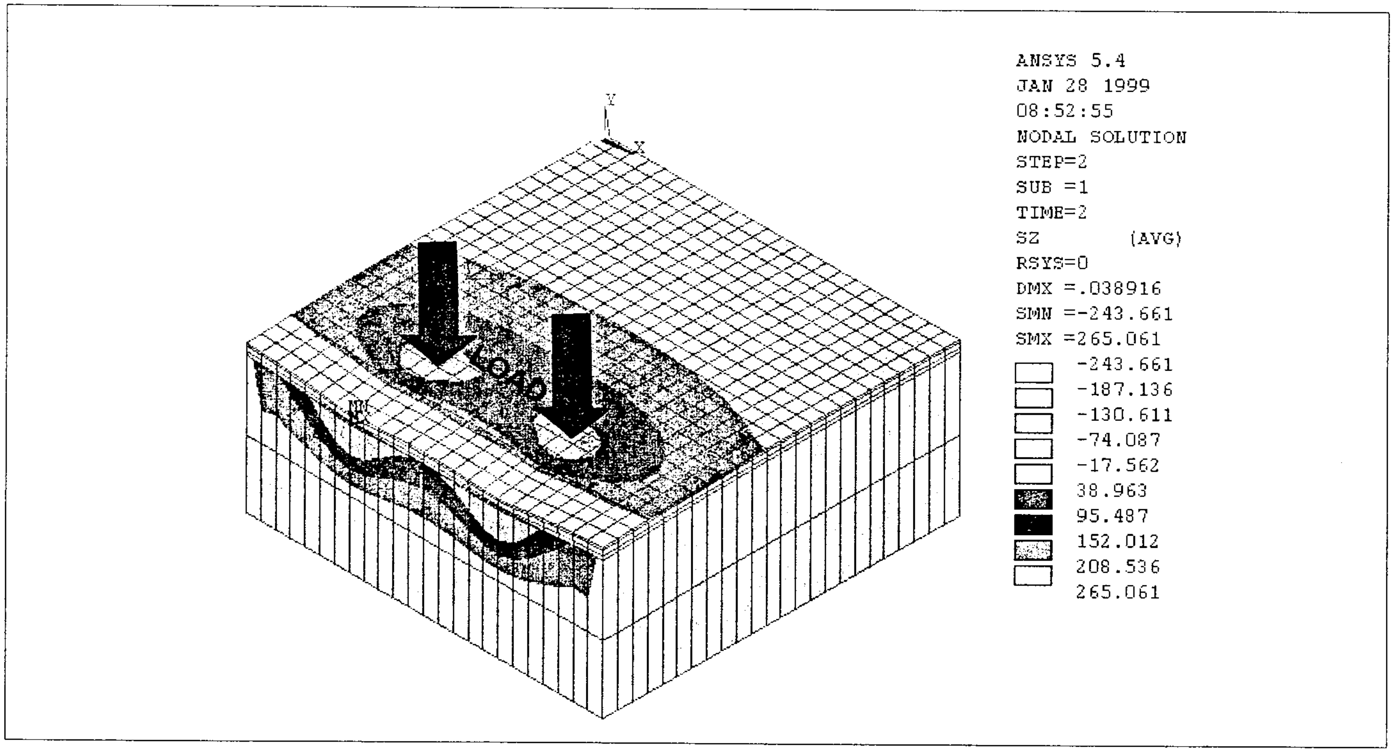



Figure 4.40 Stress (psi) onTrack 3, Slab Size - 12' X 12' (Strong Bonding)

# CHAPTER 5

# FIELD AND ANALYTICAL RESULTS

## 5.1 FIELD TEST RESULTS

Field testing was done in the months of September and October, 1997 and all the data taken from the testing of all three test tracks at the Gainesville FDOT have been analyzed and compared to the results from the computer models. Results have matched very closely in almost all the cases. Strain gauges were placed at different critical positions as shown in the figures for all three test tracks. Four types of gauges were used in the testing. These gauges were for strain as well as deflection measurements. The most efficient gauges used in this phase were the displacement gauges, which can be mounted on the surface immediately before applying the loads and can be removed for the further use after testing. Measurements from the field-testing were used to compare the FE models of the same tracks. The displacement and stress values at those points are measured using a data acquisition system.

The standard truck load was allowed to pass several number of times on all three test tracks and the results were taken. Graphs showing the variation of strain with time are presented for all the passes. Track # 3 showed the closest match of the results with the FEM models in stress and strain values. The strain vs time graphs for all the gauges on all the three tracks were drawn, from which the strain and deflection values were deduced. The deflection was calculated by multiplying the strain value by the length of the gauge which was 15 cm. Strain gauge placement positions are shown in Figures 5.1.1 to 5.1.3 and the strain Vs time results in the form of graphs are shown in Figures 5.1.4 to 5.1.27.

## 5.2 COMPARISON OF FIELD DATA AND COMPUTER MODEL RESULTS

All the above results were compared with those of the computer models and were depicted in bar charts for comparison. As most of the results seemed to be matching well, this comparison was taken as a base to proceed with studying the effect of slab size and bonding for smaller models. The comparison bar charts are shown in Figures 5.2.1 to 5.2.5. 5.3 deflection, stress and strain values.

All the small models have been investigated for the variations in vertical deflection, stress and strain values along the length and width of the tracks with the change in small slab size. The following figure depicts the sections along which the values were taken and the graphs were drawn. The rear tandem of the standard truck load was used to load the small models. The position of loading is shown by the downward pointing arrows. Figures 5.3.2 to 5.3.49 show the variation of vertical deflection, stress and strain along the length and width of tracks. The peaks observed along the length and width can be explained as the sections which have the load on them or they must be nearing the joints. These are clearly explained in Figures 5.3.2 to 5.3.7 on the graphs. Factors were deduced which influence the stress and strain due to change in slab length and thickness as shown in Table 5.13. The percentage difference in the values are also produced in the Tables 5.3.2 through 5.3.4.

**Table 5.3.1** Factors Influencing Stresses and Strains Due to Change in Slab Thickness.
**Case I: General Models**
Effect of slab length on the stresses and strains

| Slab Thickness (in.) | Change in Slab Length |
|---|---|
| 4" | 4' to 6' |
| 2" | 3' to 4' |
|  | 4' to 12' |
| 3" | 4' to 6' |
|  | 6' to 12' |

*Effect of slab thickness on the stresses and strains*

| Slab Size (ft) | Change in Thickness |
|---|---|
| 4' X 4' | 2" to 4" |
|  | 3" to 4" |

| | |
|---|---|
| | 2" to 3" |
| 6' X 6' | 3" to 4" |
| 12' X 12' | 2" to 3" |

**Case II: High Bonding Models**

*Effect of slab length on the stresses and strains*

| Slab Thickness (in.) | Change in Slab Length |
|---|---|
| 4" | 4' to 6' |
| 2" | 3' to 4' |
| | 4' to 12' |
| 3" | 4' to 6' |
| | 6' to 12' |

*Effect of slab thickness on the stresses and strains*

| Slab Size (ft) | Change in Thickness |
|---|---|
| 4' X 4' | 2" to 4" |
| | 3" to 4" |
| | 2" to 3" |
| 6' X 6' | 3" to 4" |
| 12' X 12' | 2" to 3" |

**Effect Of Slab Thickness :**

**Table 5.3.2** Effect of Slab Thickness (From 4" to 2")

| Distance *X* in m | Stress On 4" Slab |
|---|---|
| Along Z | 50.874 |
| | -85.434 |
| Along X | 31.984 |
| | -86.138 |

**Table 5.3.3** Effect of Slab Thickness (on 4' x 4' slabs)

| Distance X in m | Stress On 4" Slab |
|---|---|
| Along Z | 43.961 |
| | -83.49 |
| Along X | 51.01 |
| | 183.18 |

**Table 5.3.4** Effect of Slab Thickness (on 12' x 12' slabs)

| Distance X in m | Stress On 3" Slab |
|---|---|
| Along Z | 163.58 |
| | -243.66 |
| Along X | 218.43 |
| | -700 |

**Effect Of Slab Size**

The Maximum stresses in psi on 4' x 4' , 6' x 6' and 12' x 12' of Track # 3 are tabulated below.

**Table 5.3.5** Effect of Slab Size

| Distance | Stress On 4' x 4' | Stress On 6' x 6' |
|---|---|---|
| Along Z | 92.914 | 155.54 |
| | -200 | -217.48 |
| Along X | 105.99 | 160.63 |
| | -400 | -631.34 |

**Effect of Bonding**

Some modifications on the models were introduced since the analysis was started. These modifications were based on the field results. Among the introduced modifications are the bonding characteristics of the UTW. Two types of bonding were considered namely (1) high bonding and (2) low bonding (friction models). This variation in bonding was realized by having full contact between the overlay and the base layer (models with high bonding), and by having contact elements with certain real constants representing in-plane shear (models with low bonding).

The models with high bonding have shown good improvement in the reduction of deflections, stresses and strains along both the width and length of the tracks. Propping up of slabs was observed at the edges in case of weak bonding.

**Table 5.4.1** Stress Variation Between High And Weak Bonding

| Point | Stresses On Weak Bonding |
|---|---|
| 1 | 143.937 |
| 2 | 128.582 |
| 3 | 113.227 |
| 4 | 97.872 |
| 5 | 82.518 |
| 6 | 67.163 |
| 7 | 51.808 |

**Some suggestions for improving the bond**

1- Apply a crack relief layer (CRL) which consists of a thin coat of asphalt and aggregates to prevent reflective cracking. This is a common practice in Florida when resurfacing asphalt pavement. The CRL will also a level surface with a rough texture to allow a good bond with the concrete overlay.

2- Follow good concrete consolidation, curing and mix design. (These factors offset the ill-effects that drying shrinkage can have on very thin sections).

3- To maintain good bond, joint type, location and width from the existing pavement must be matched in the overlay. If this is not done excessive compressive forces may develop as the underlying pavement expands during increased temperatures. This will de-bond and in the worst case buckle. A cement grout can generate the necessary bond strength and has proven effective.

4- Vibration is another factor, which contributes to the bond strength without using grout. The following pages show the bar chart comparison between the friction models and the full bonding models in the values of vertical deflection, stress and strain along the length and width of the tracks.

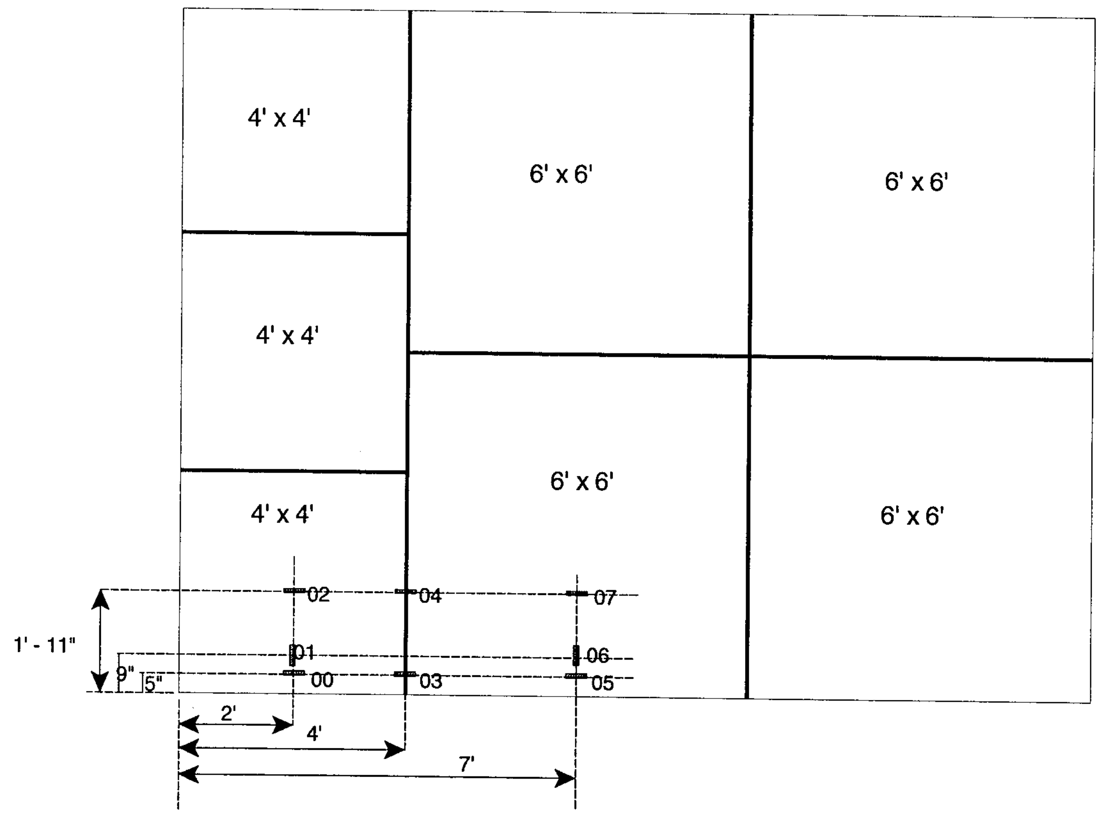



Figure 5.1.1 TEST TRACK # 1 SHOWING GAGE LOCATIONS

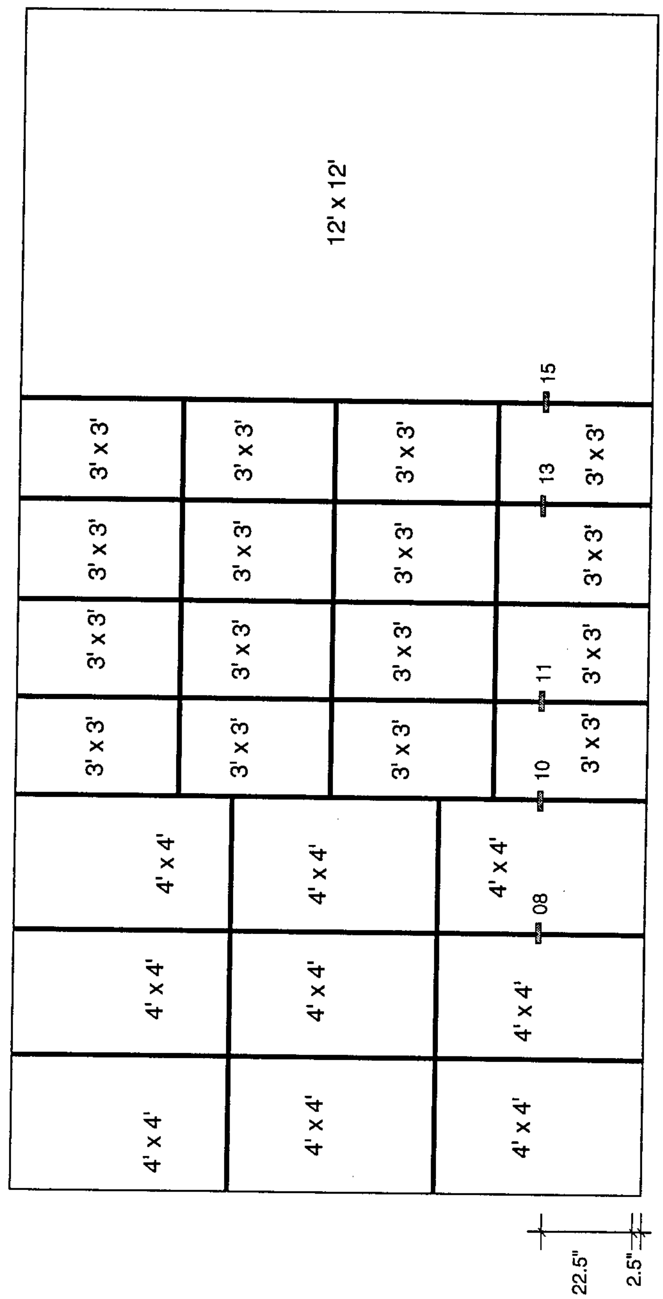



Figure 5.1.2 TEST TRACK # II SHOWING GAGE LOCATIONS

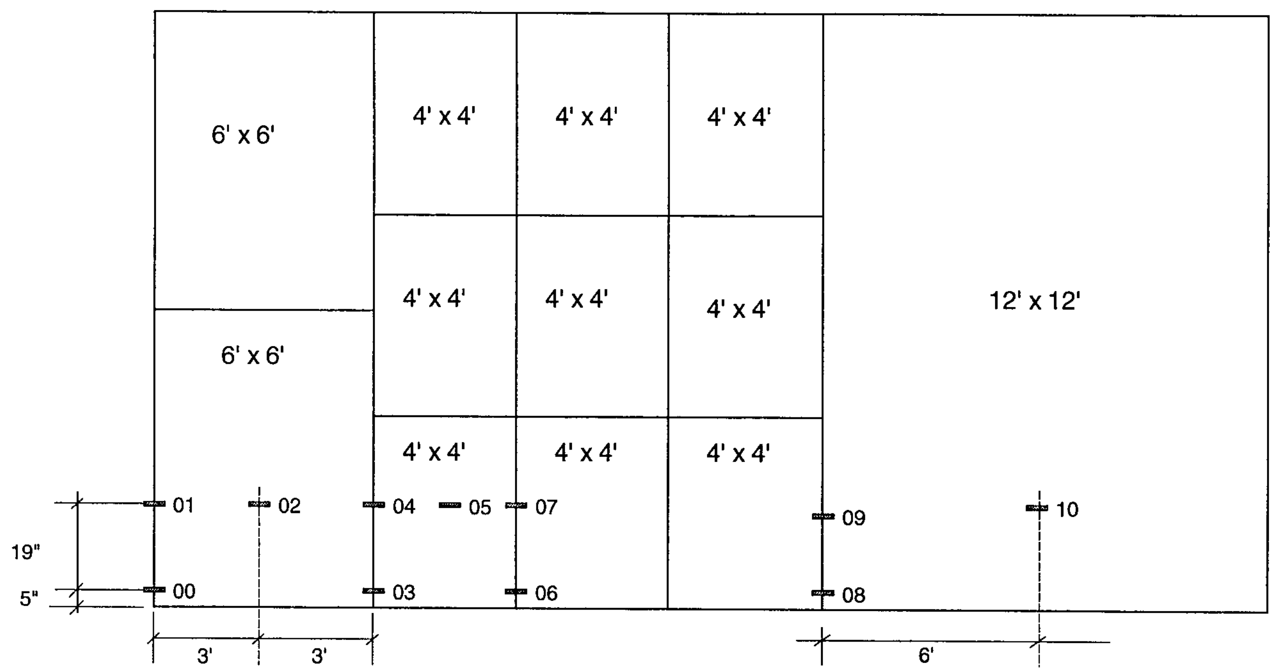



Figure 5.1.3 TEST TRACK # III SHOWING GAGE LOCATIONS

## Track # 1

Comparison of strain values at different gauge locations:

| | Gauge 0 | Gauge 1 | Gauge 2 | Gauge5 | Gauge 6 | Gauge 7 |
|---|---|---|---|---|---|---|
| Field | -4 | 2.5 | -3.13 | -2 | 2 | -5 |
| FEM | -2.30711 | 2.3411 | -4.5163 | -1.96531 | 1.1939 | -3.63989 |

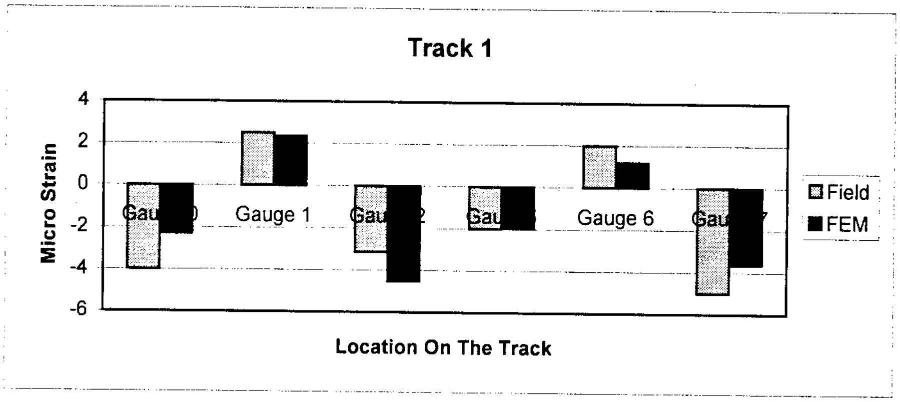



Figure 5.2.1 Comparison of Field and FEM Strain Values on Track # 1

Comparison of displacement values at different gauge locations:

| | Gage 4 | Gage 3 |
|---|---|---|
| Field | 2.95E-04 | 1.77E-04 |
| FEM | 3.12E-04 | 1.97E-04 |

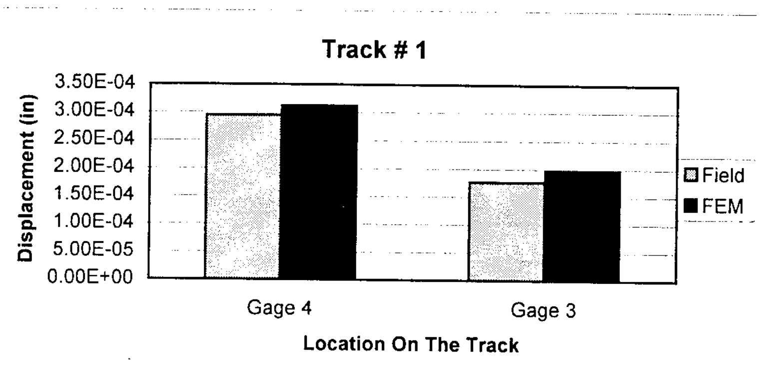



Figure 5.2.2 Comparison of Field and FEM Displacement Values on Track # 1

## Track # 2

Comparison of displacement values at different gauge locations:

| | Gauge 8 | Gauge10 | Gauge 11 | Gauge 13 | Gauge 15 |
|---|---|---|---|---|---|
| Field | 2.66E-04 | 2.07E-04 | 1.18E-04 | 1.18E-04 | 2.36E-04 |
| FEM | 3.04E-04 | 3.04E-04 | 1.76E-04 | 1.76E-04 | 2.49E-04 |

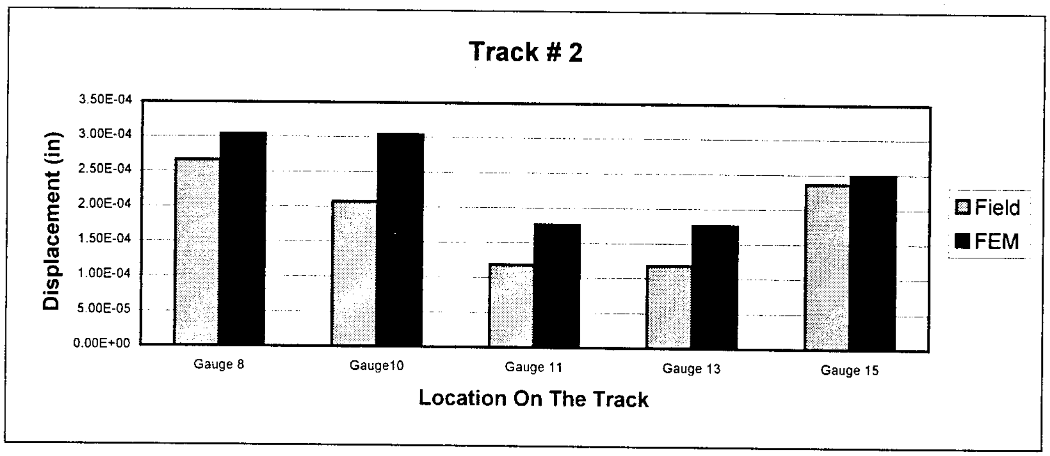



Figure 5.2.3 Comparison of Field and FEM Displacement Values on Track # 2

# Track # 3

Comparison of displacement values at different gauge levles:

| | Gauge 0 | Gauge 1 | Gauge 3 | Gauge 4 | Gauge 6 | Gauge 7 | Gauge 8 | Gauge 9 |
|---|---|---|---|---|---|---|---|---|
| Field | 1.48E-04 | 1.77E-04 | 8.86E-05 | 1.48E-04 | 5.91E-05 | 1.18E-04 | 8.86E-05 | 1.18E-04 |
| FEM | 1.31E-04 | 1.79E-04 | 8.10E-05 | 1.81E-04 | 8.10E-05 | 1.81E-04 | 9.97E-05 | 8.10E-05 |

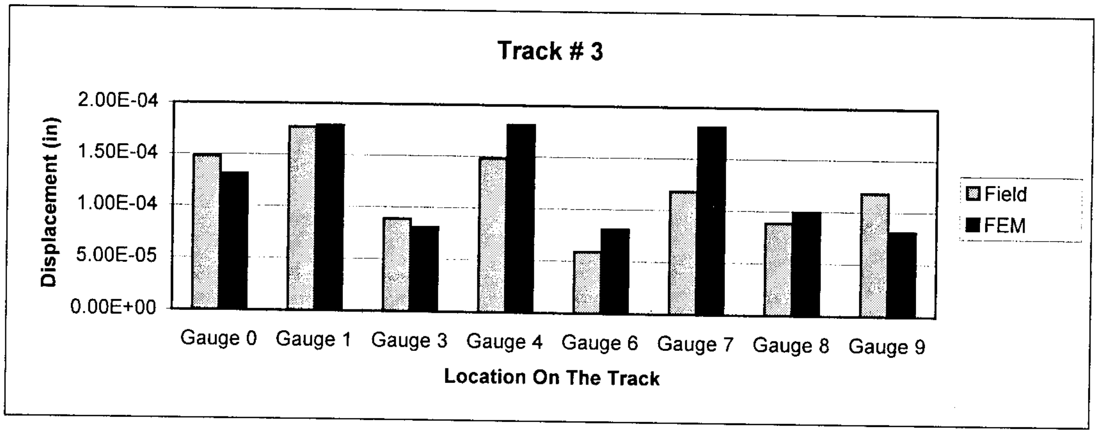



Figure 5.2.4 Comparison of Field and FEM Displacement Values on Track # 3

Comparison of strain values at different gauge levles:

| | Gauge 2 | Gauge 5 | Gauge 10 |
|---|---|---|---|
| Field | 1.50E+00 | 3.00E+00 | 2.25E+00 |
| FEM | 1.34 | 3.9 | 2.06 |

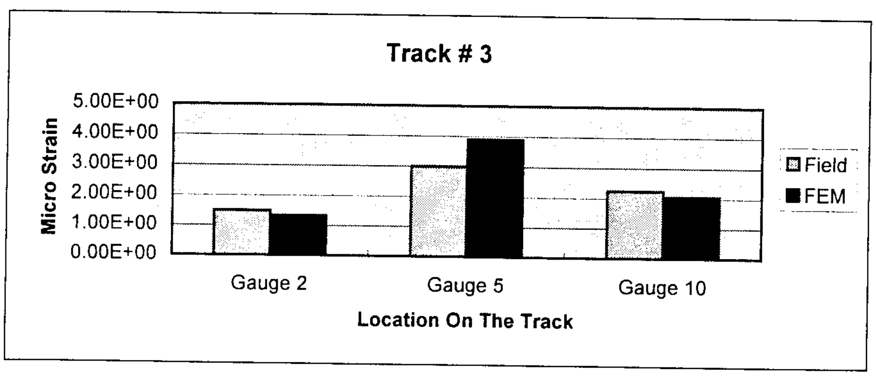
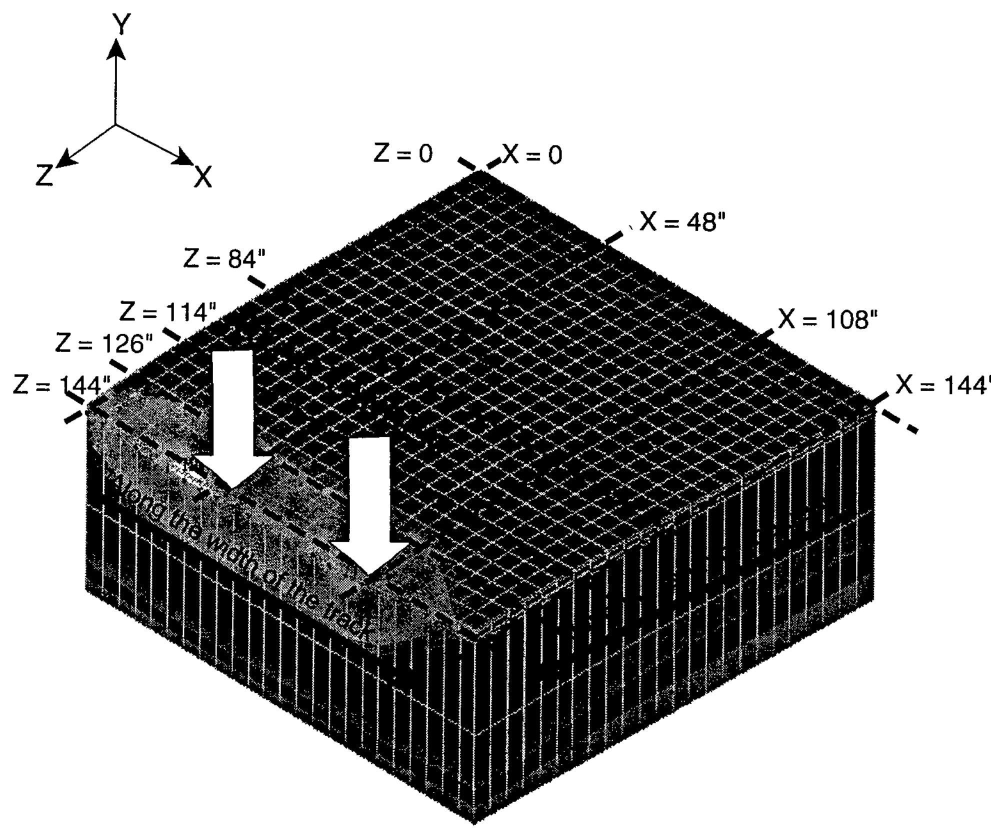



Figure 5.2.5 Comparison of Field and FEM Strain Values on Track # 3




Figur 5.3.1 UTW Cross Sections Along X and Z Directions

Figure 5.3.2. Vertical Deflection At Different Sections Along The Length Of The Track # 1 (Friction), Panel Size 4' x 4'

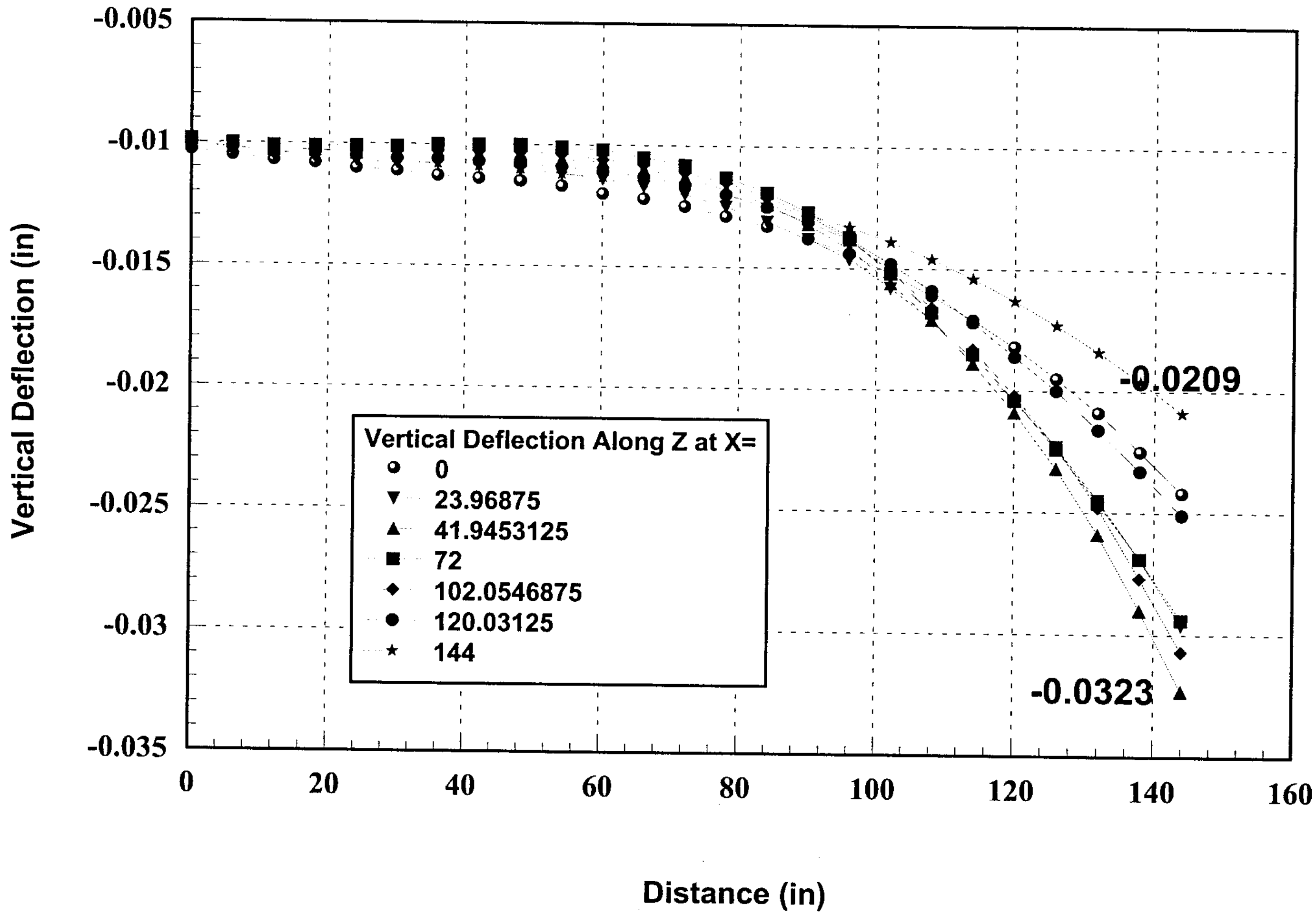


Figure 5.3.3 Stress At Different Sections Along The Length Of The Track # 1 (Friction), Panel Size 4' x 4'

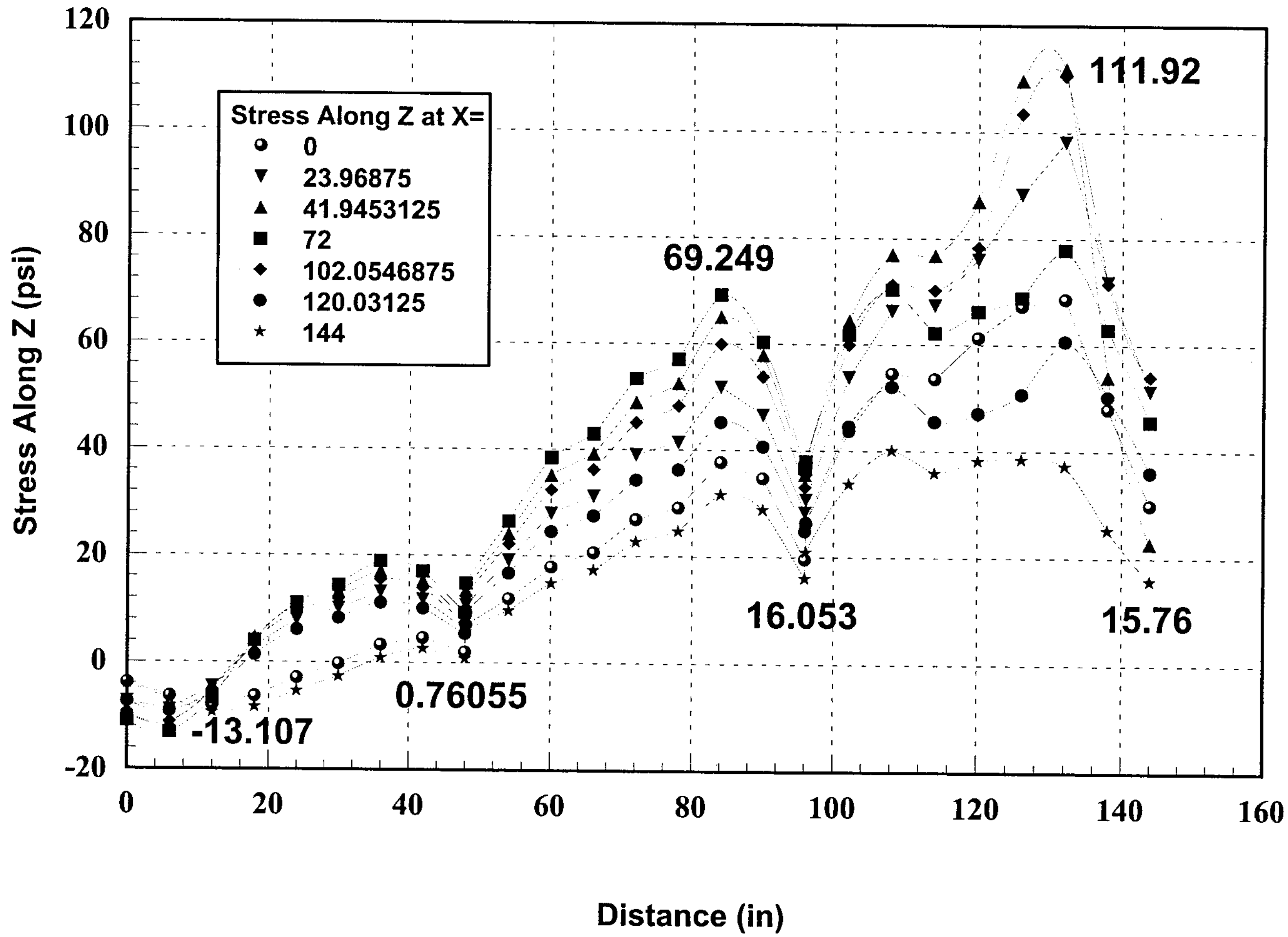


Figure 5.3.4. Strain At Different Sections Along The Length Of The Track # 1 (Friction), Panel Size 4' x 4'

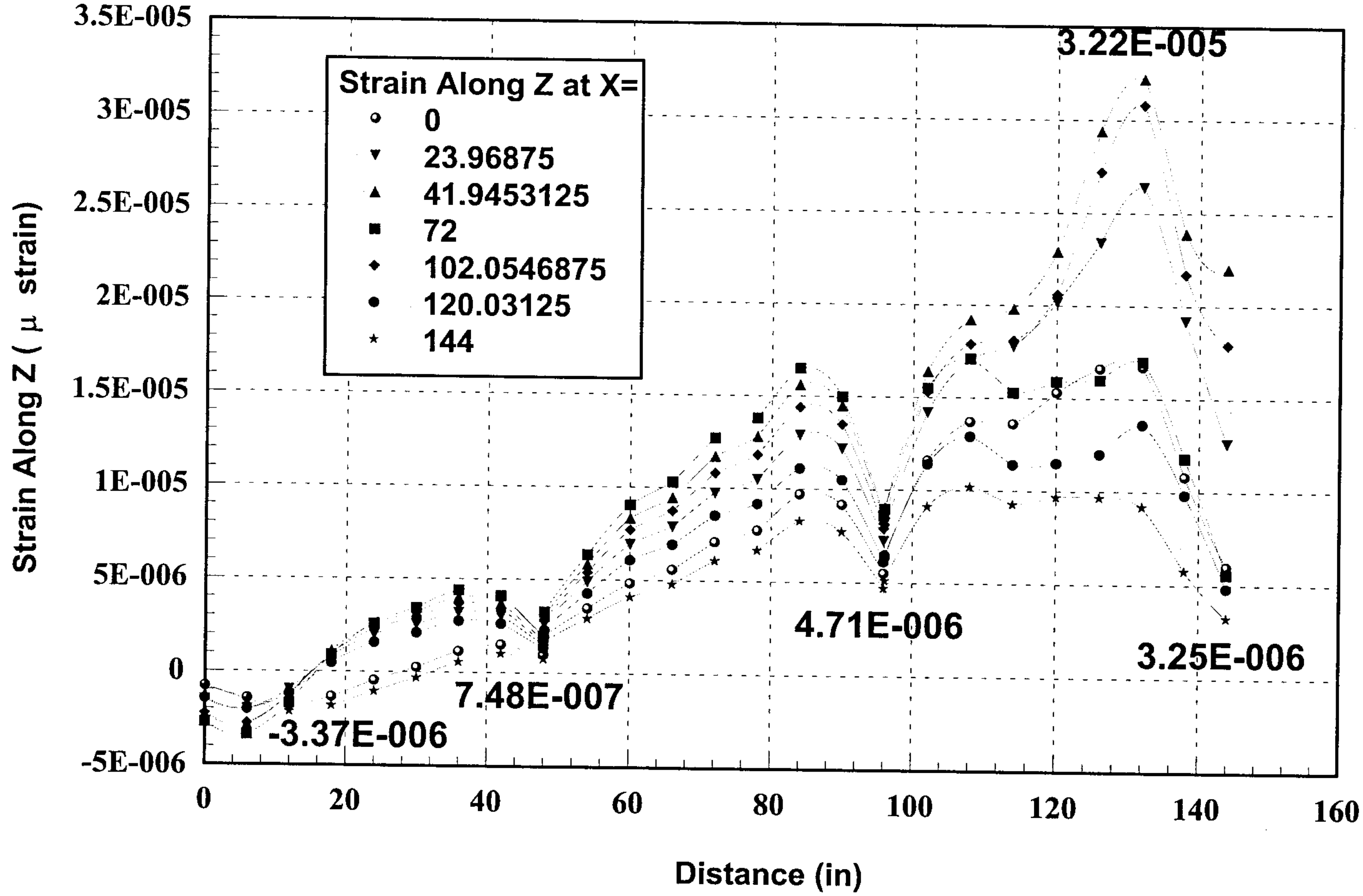


Figure 5.3.5. Vertical Deflection At Different Sections Along The Width Of The Track # 1 (Friction), Panel Size 4' x 4'

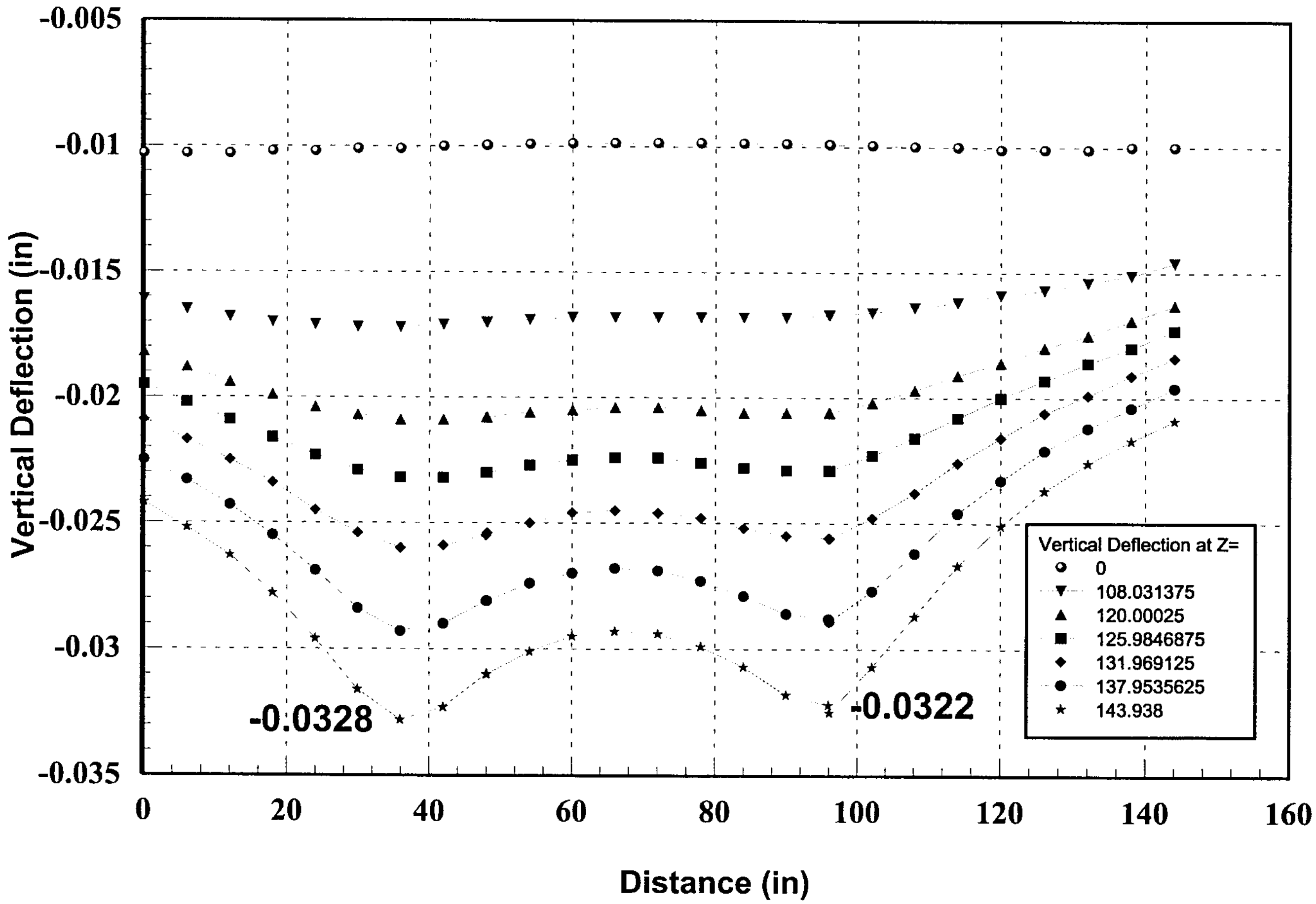


Figure 5.3.6. Stress At Different Sections Along The Width Of The Track # 1 (Friction), Panel Size 4' x 4'

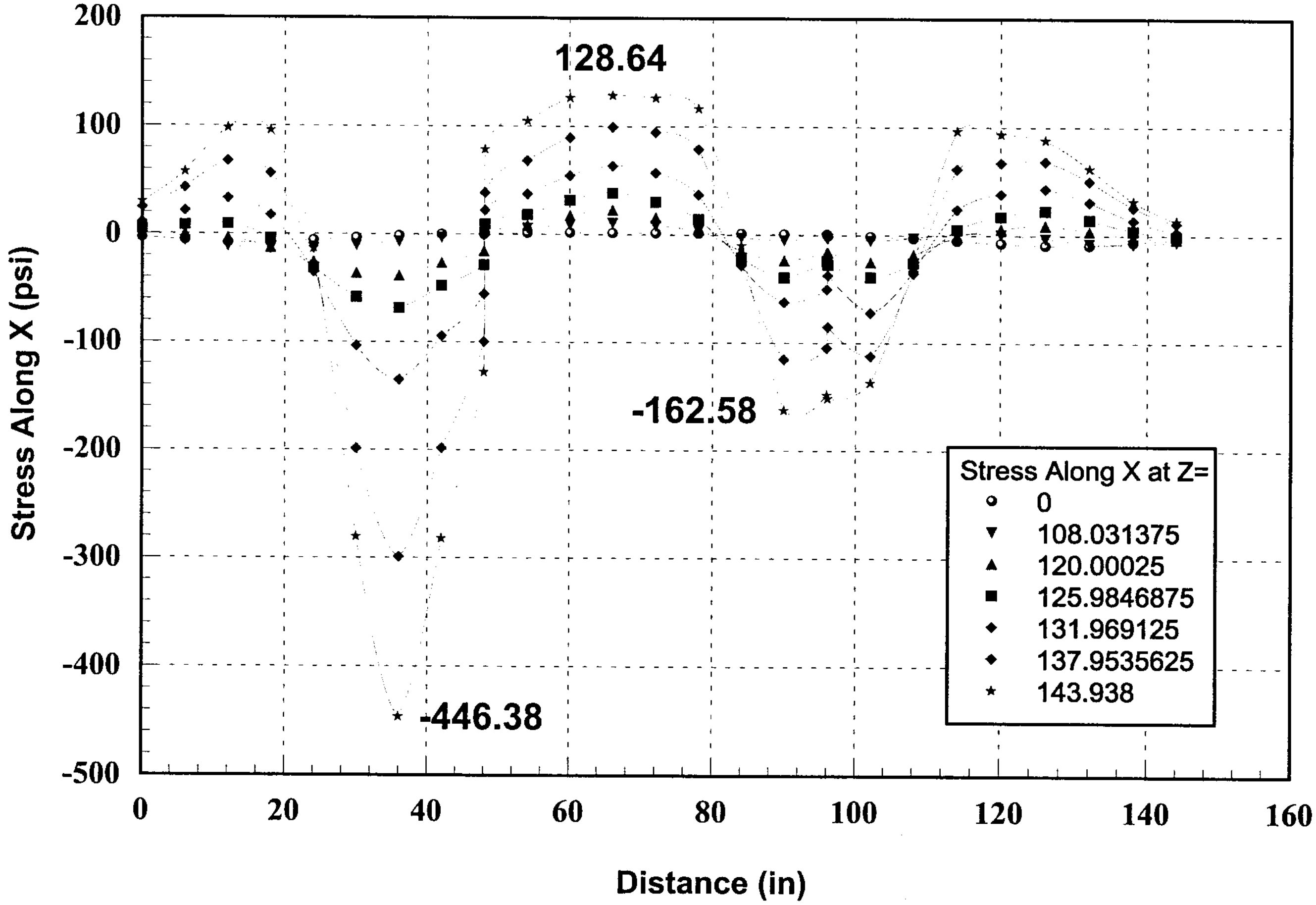


Figure 5.3.7. Strain At Different Sections Along The Width Of The Track # 1 (Friction), Panel Size 4' x 4'

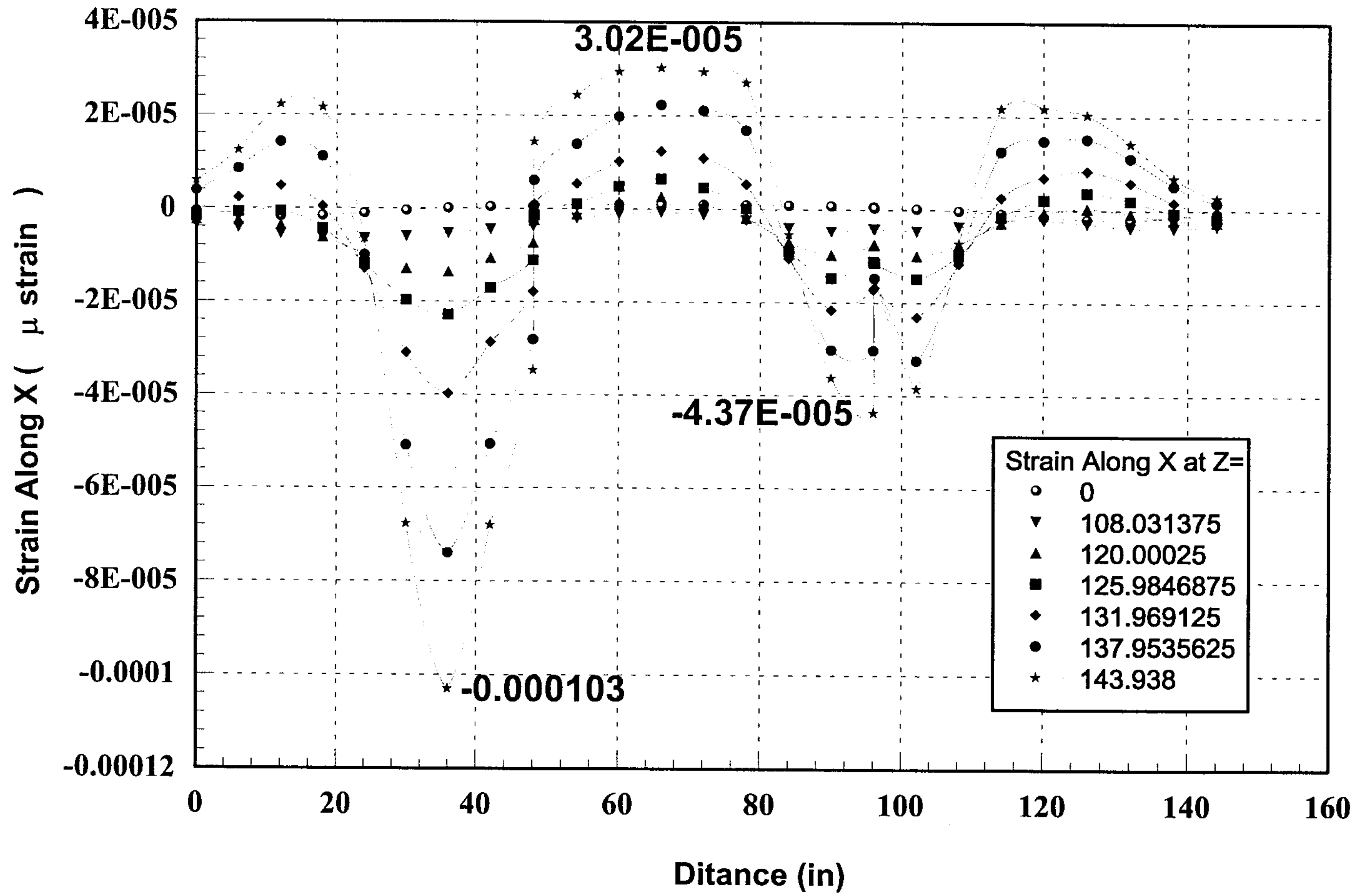


Figure 5.3.8. Vertical Deflection At Different Sections Along The Length Of The Track # 1 (Friction), Panel Size 6' x 6'

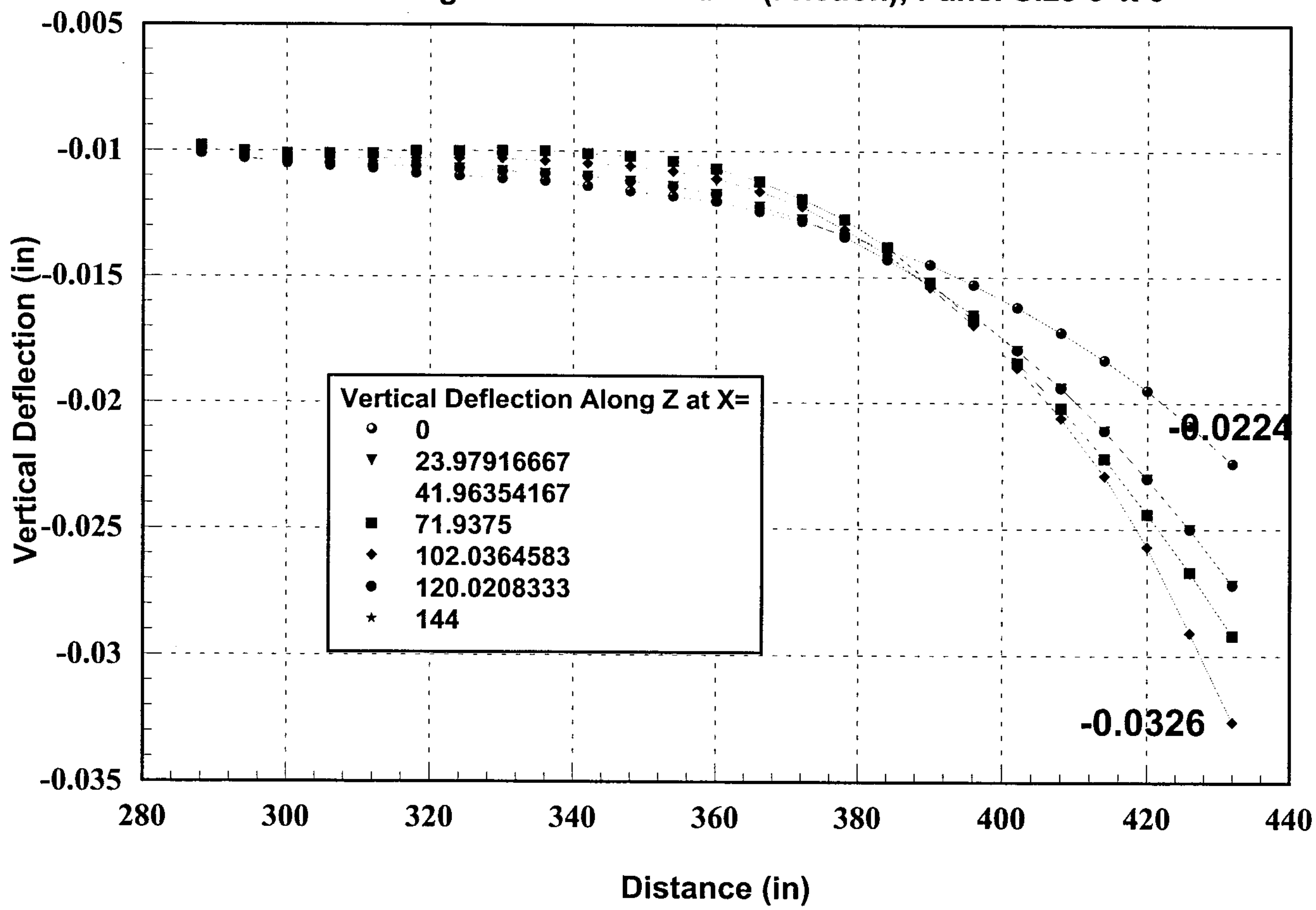


Figure 5.3.9. Stress At Different Sections Along The Length Of The Track # 1 (Friction), Panel Size 6' x 6'

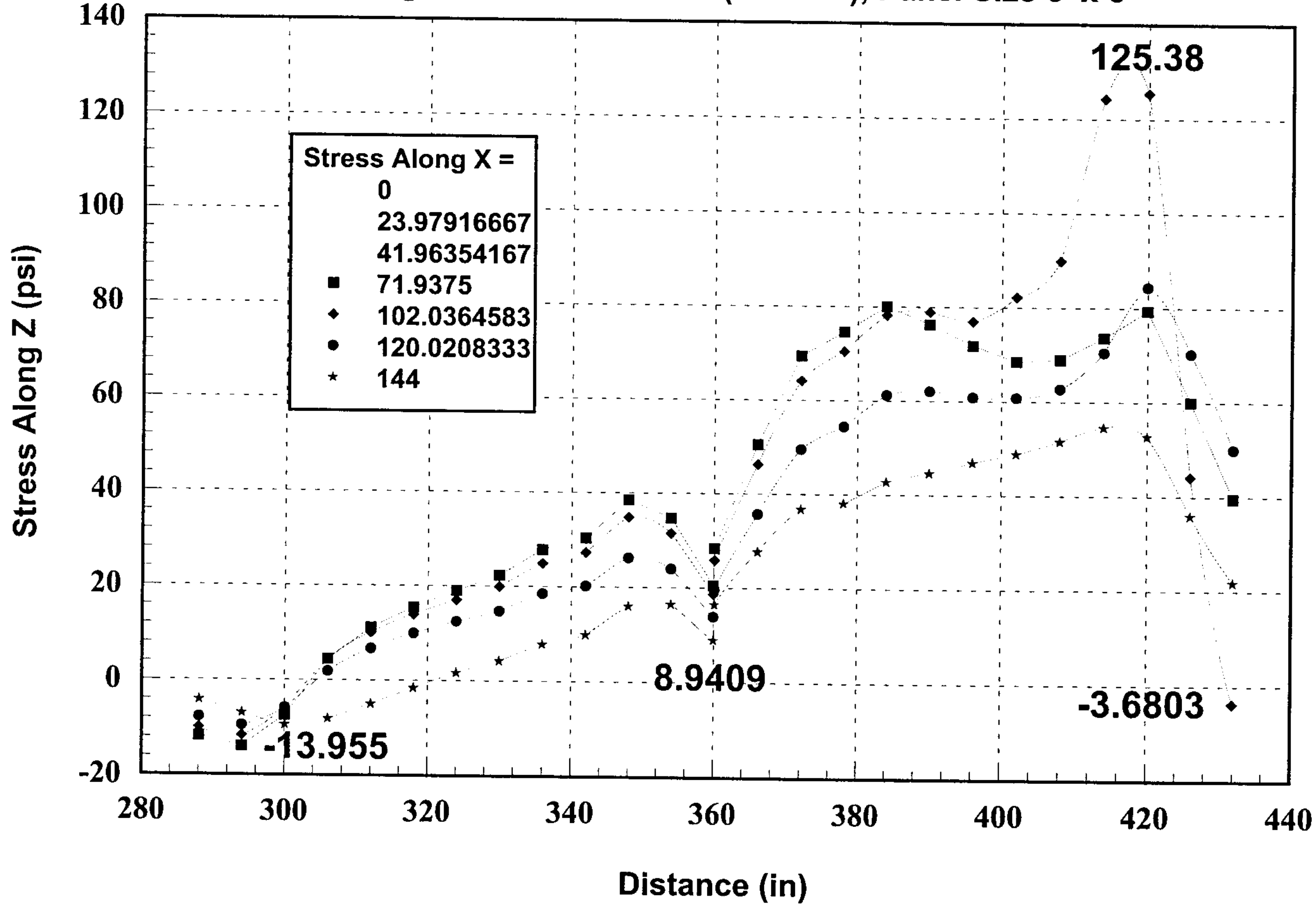


Figure 5.3.10. Strain At Different Sections Along The Length Of The Track # 1 (Friction), Panel Size 6' x 6'

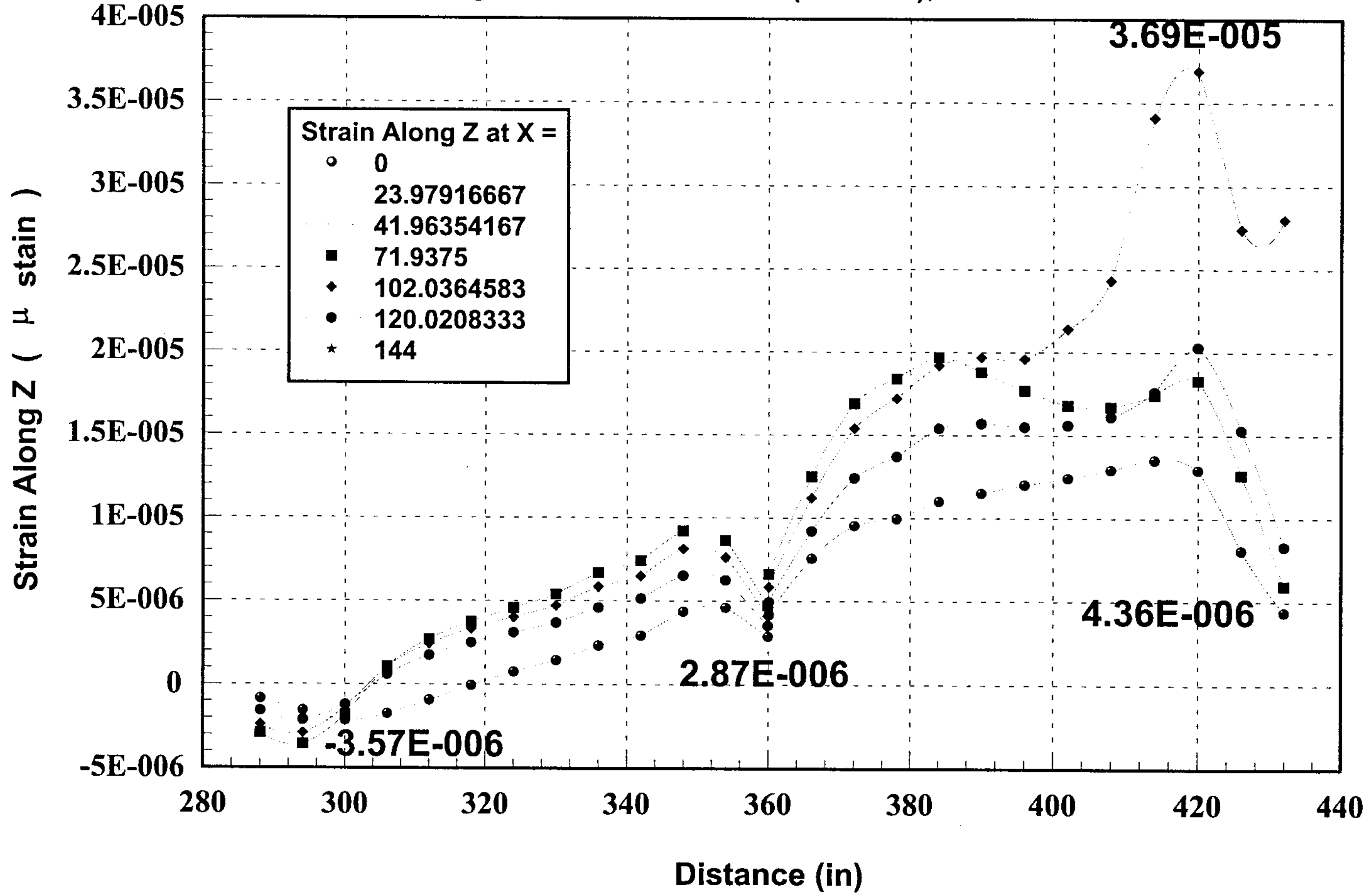


Figure 5.3.11. Vertical Deflection At Different Sections Along The Width Of The Track # 1 (Friction), Panel Size 6' x 6'

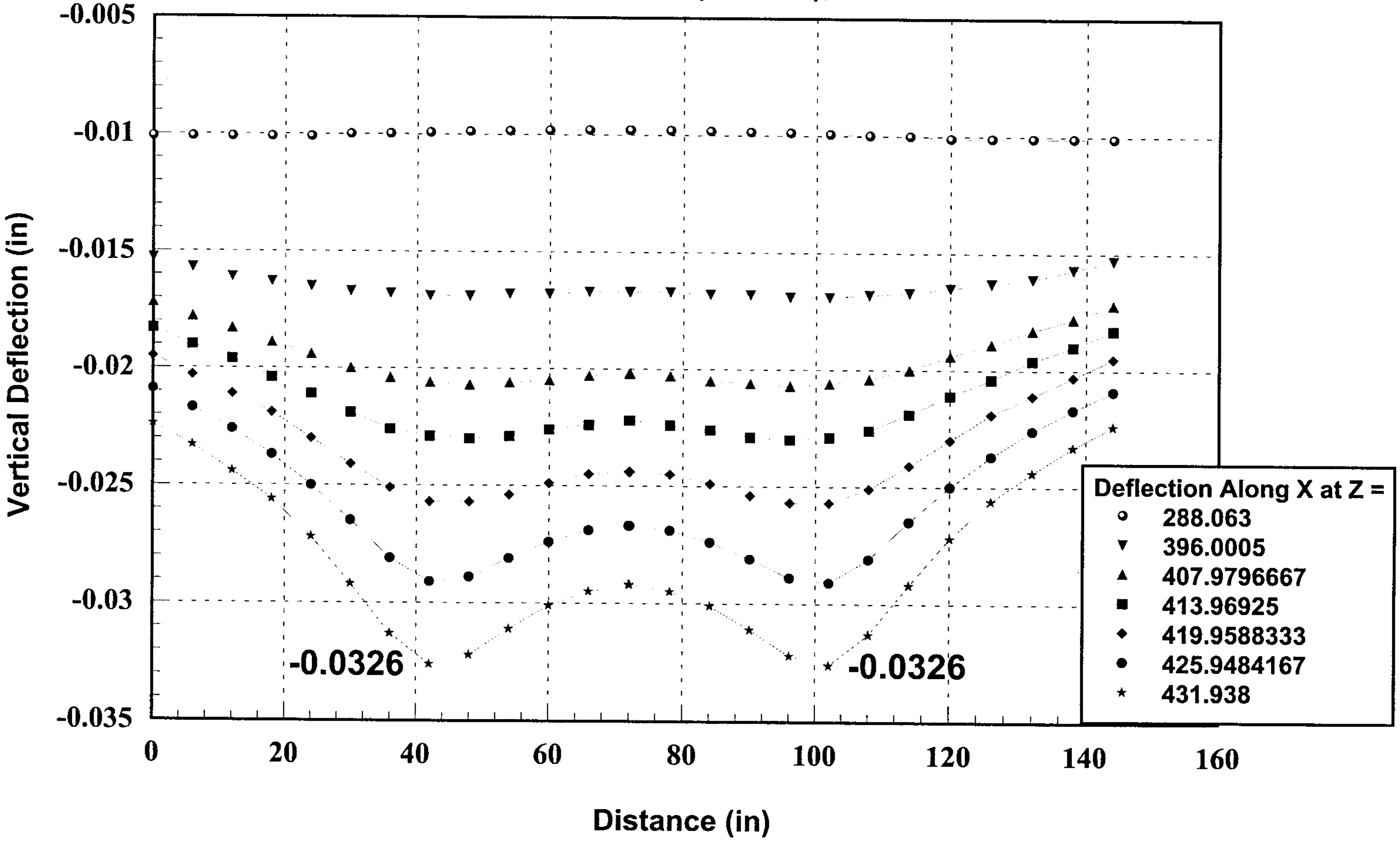


Figure 5.3.12. Stress At Different Sections Along The Width Of The Track # 1 (Friction), Panel Size 6' x 6'

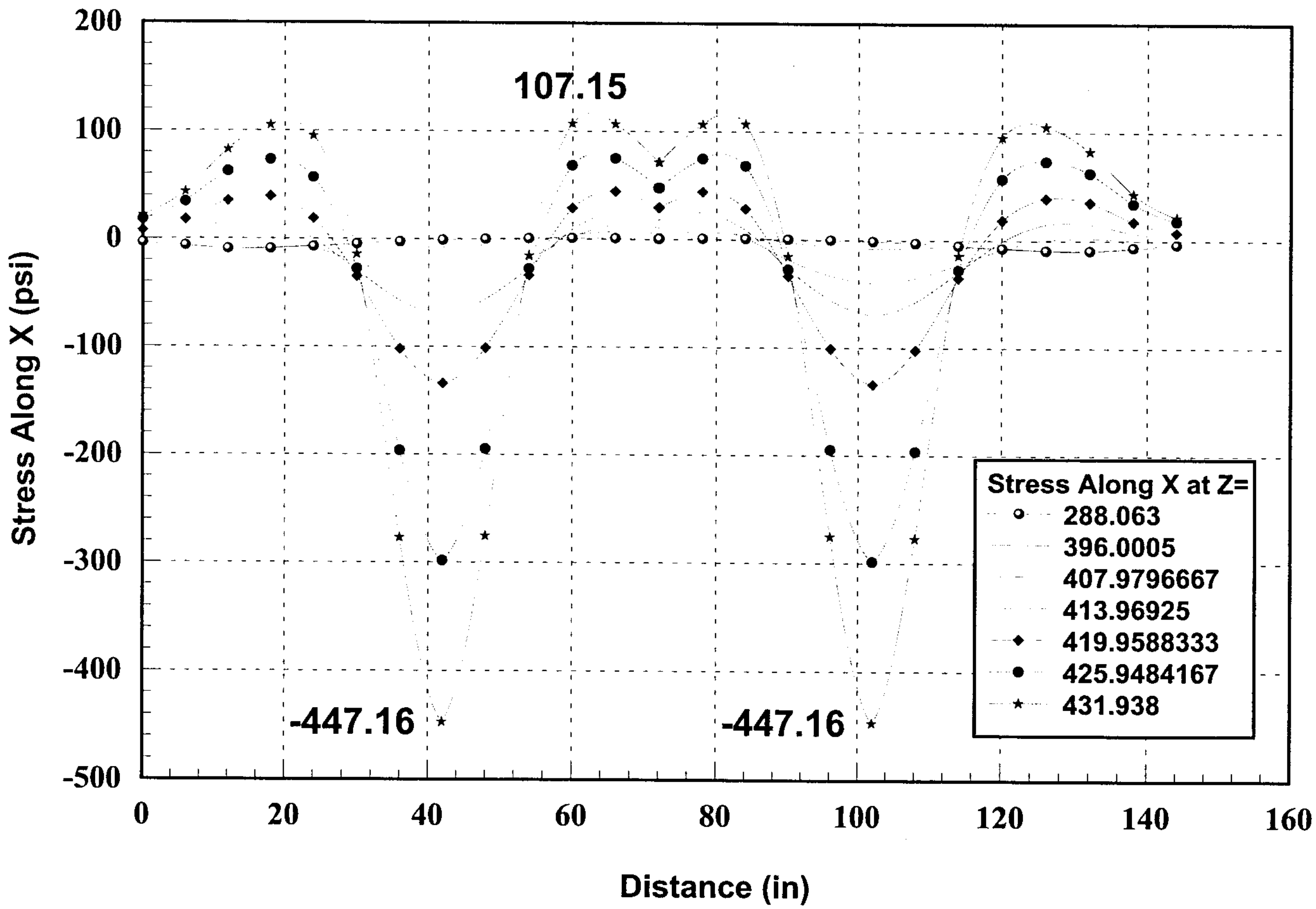


Figure 5.3.13. Strain At Different Sections Along The Width Of The Track # 1 (Friction), Panel Size 6' x 6'

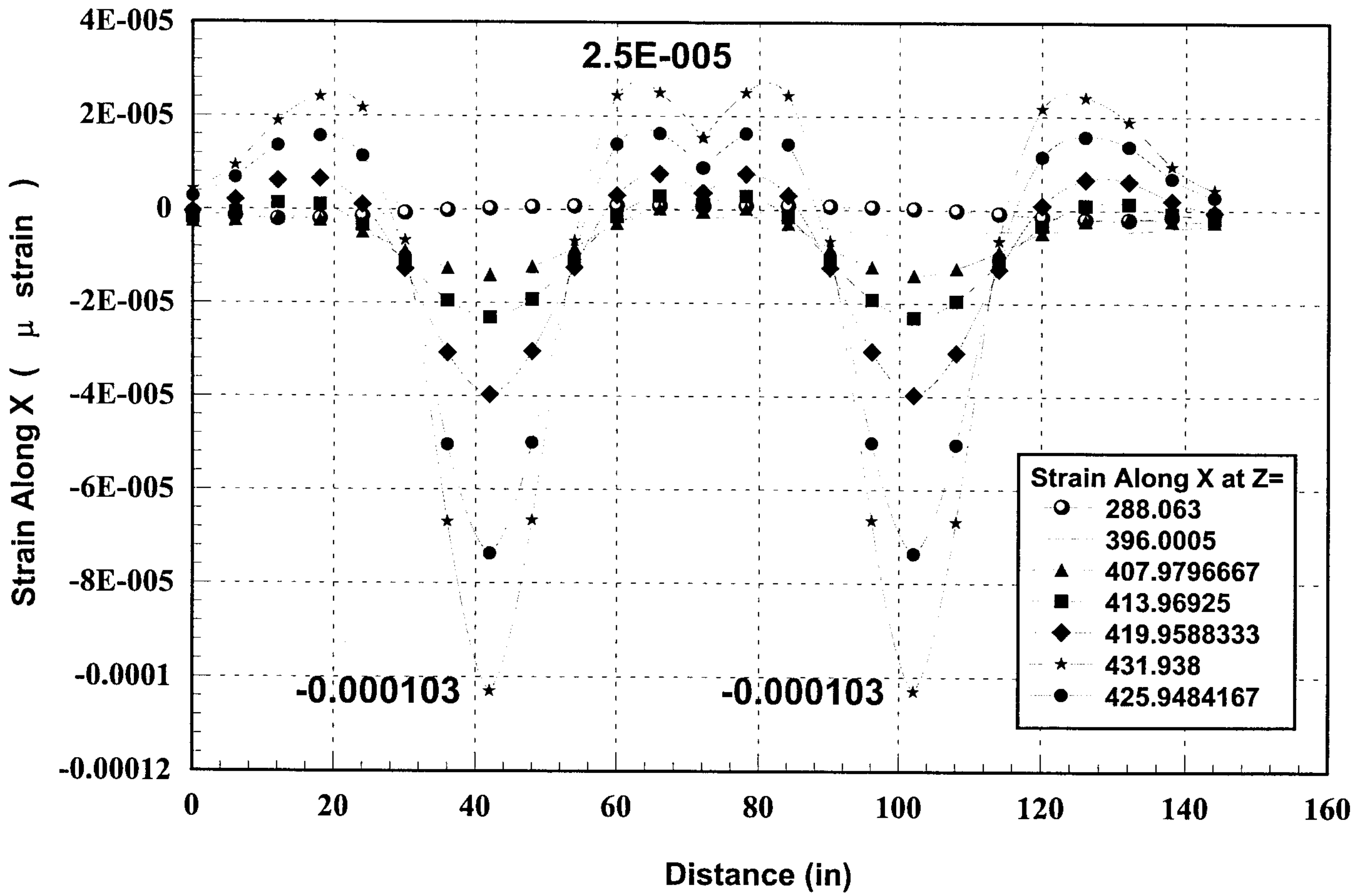


Figure 5.3.14. Vertical Deflection At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 3' x 3'

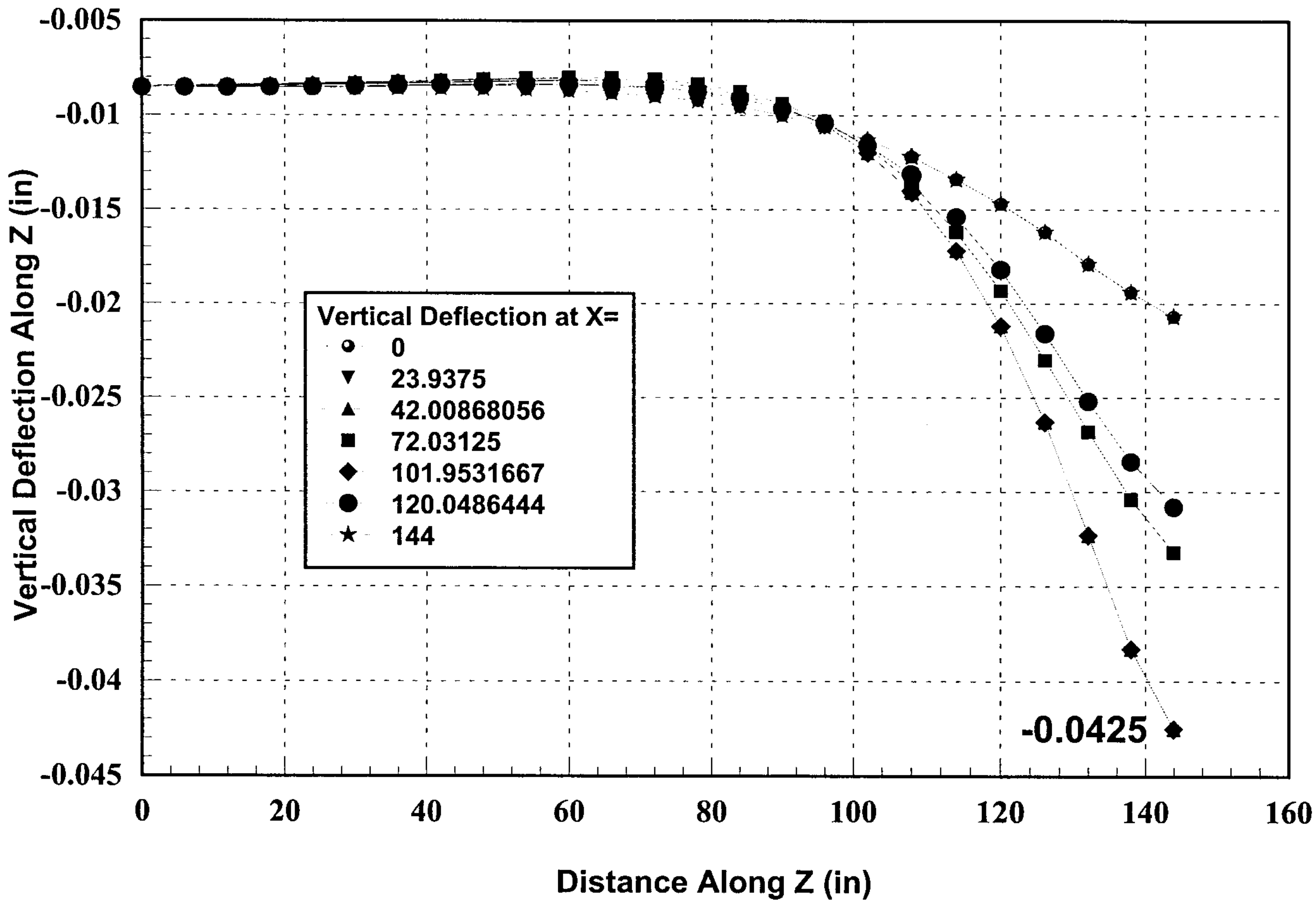


Figure 5.3.15. Stress At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 3' x 3'

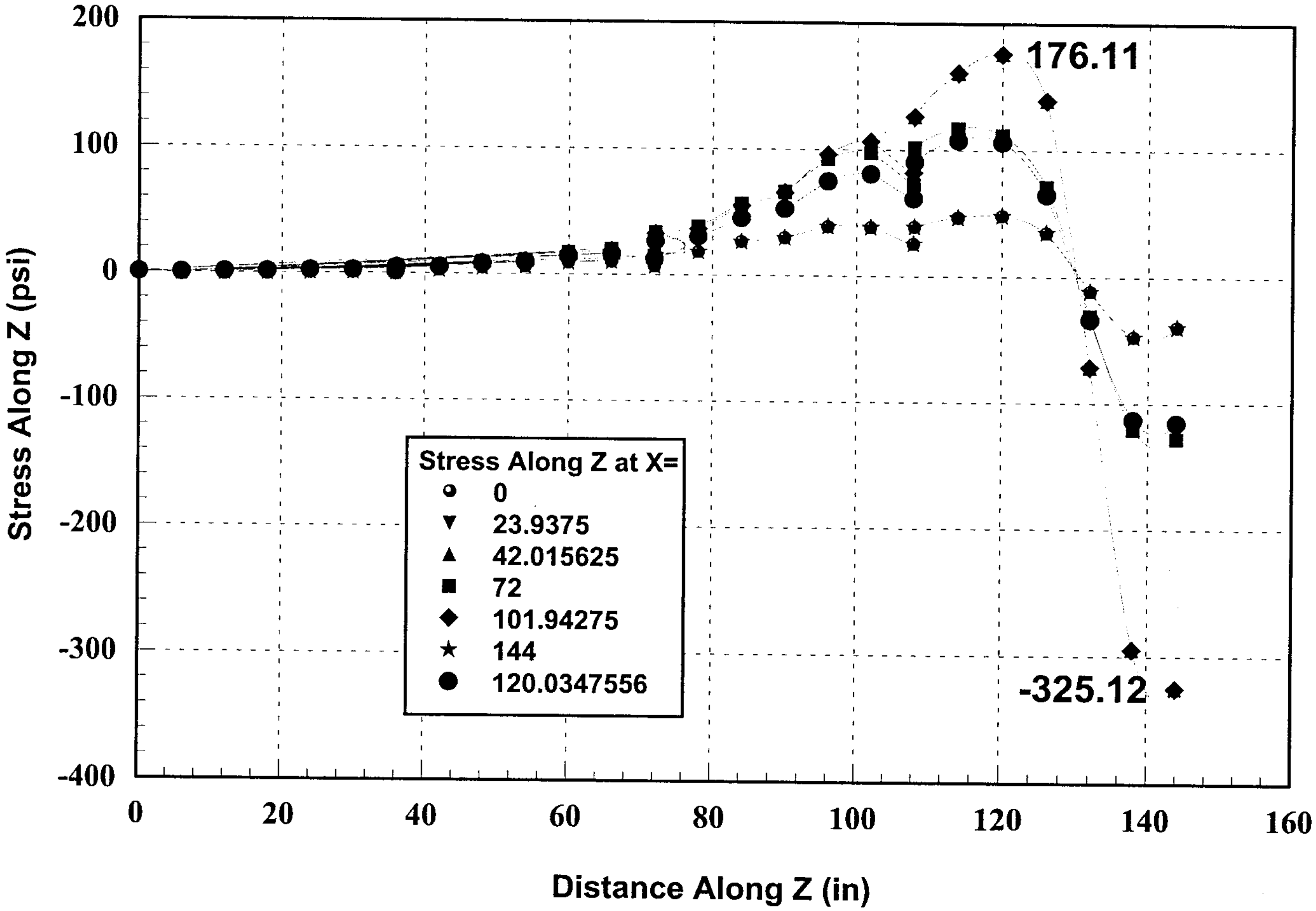


Figure 5.3.16. Strain At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 3' x 3'

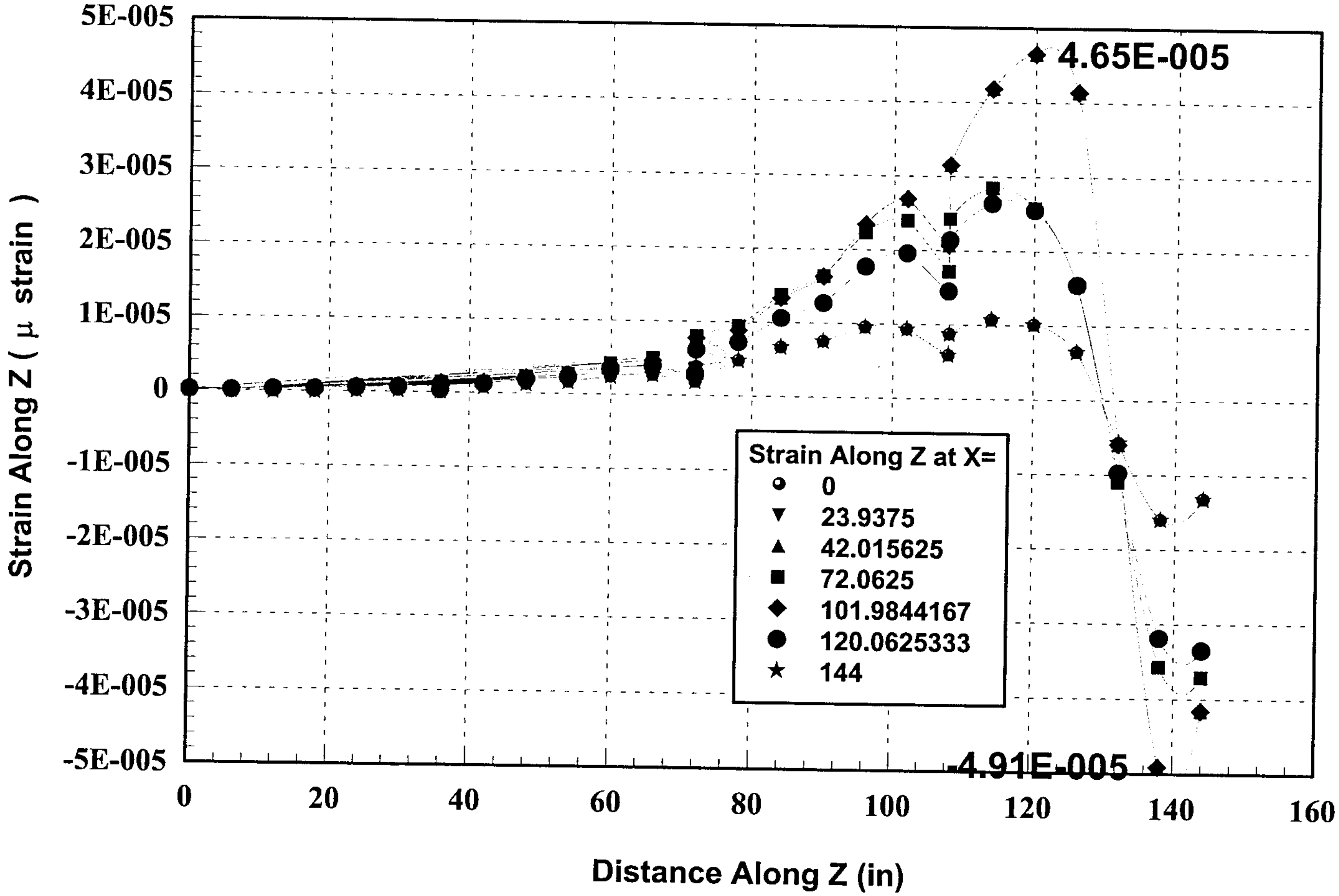


Figure 5.3.17. Vertical Deflection At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 3' x 3'

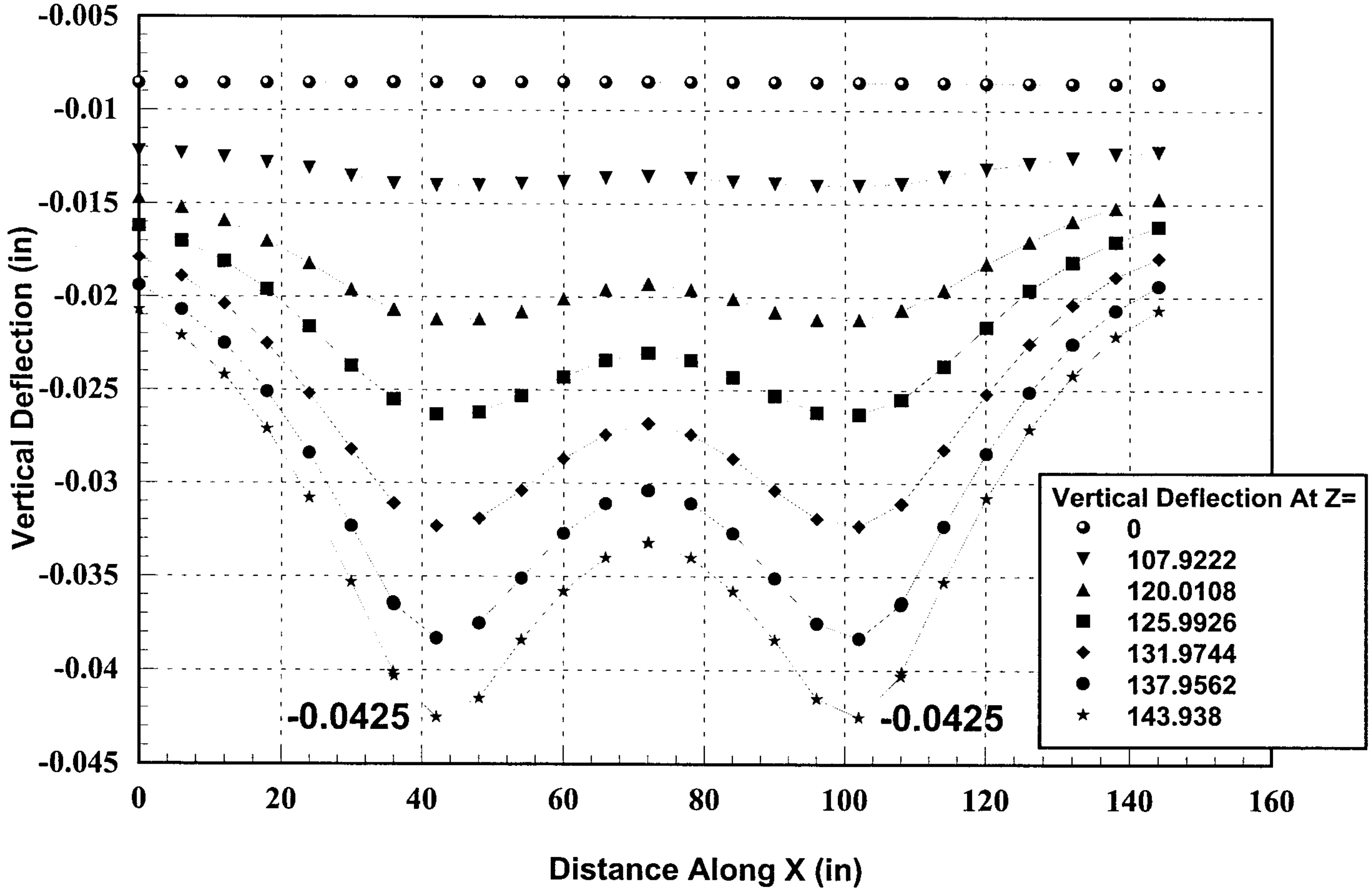


Figure 5.3.18. Stress At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 3' x 3'

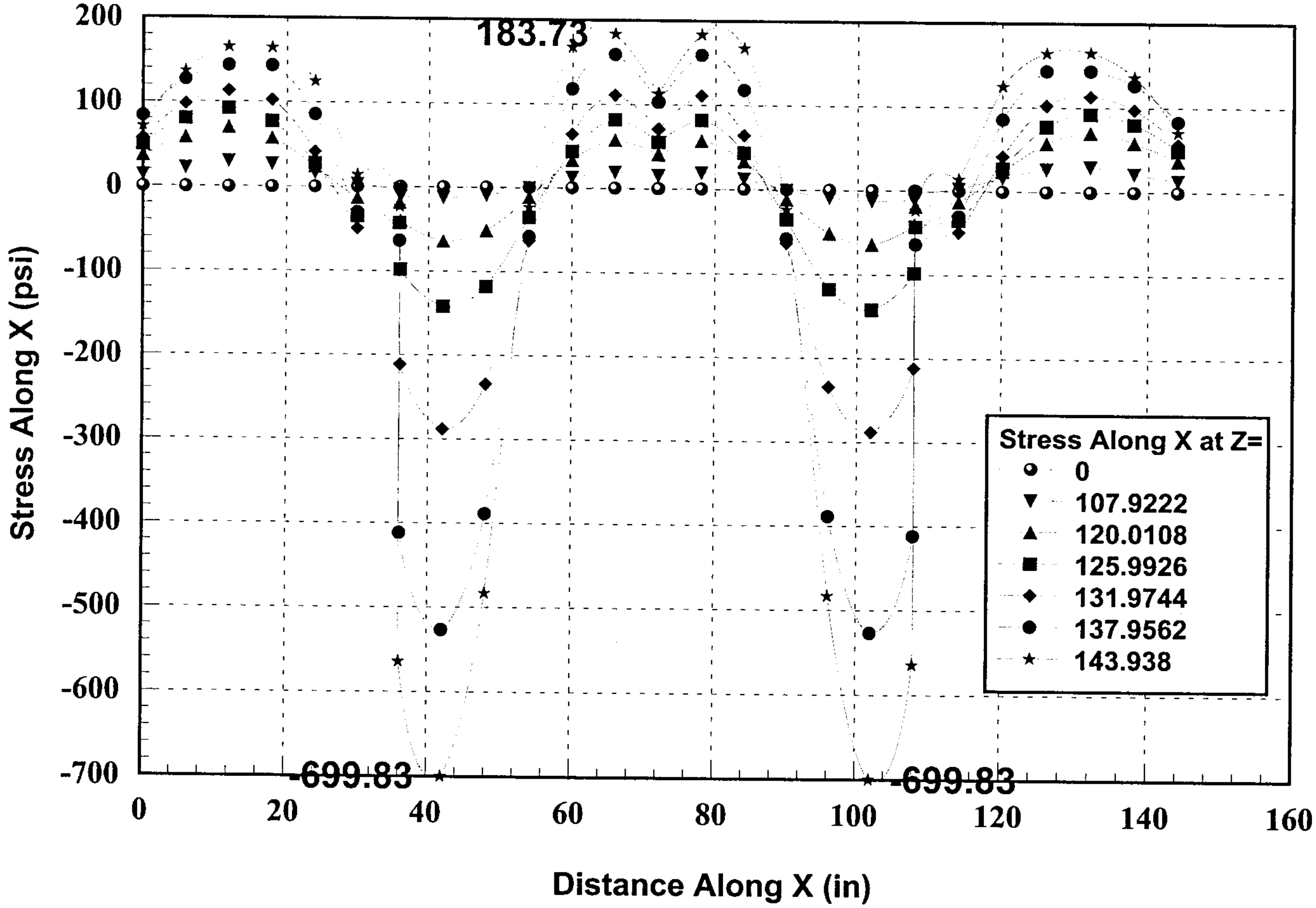


Figure 5.3.19. Strain At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 3' x 3'

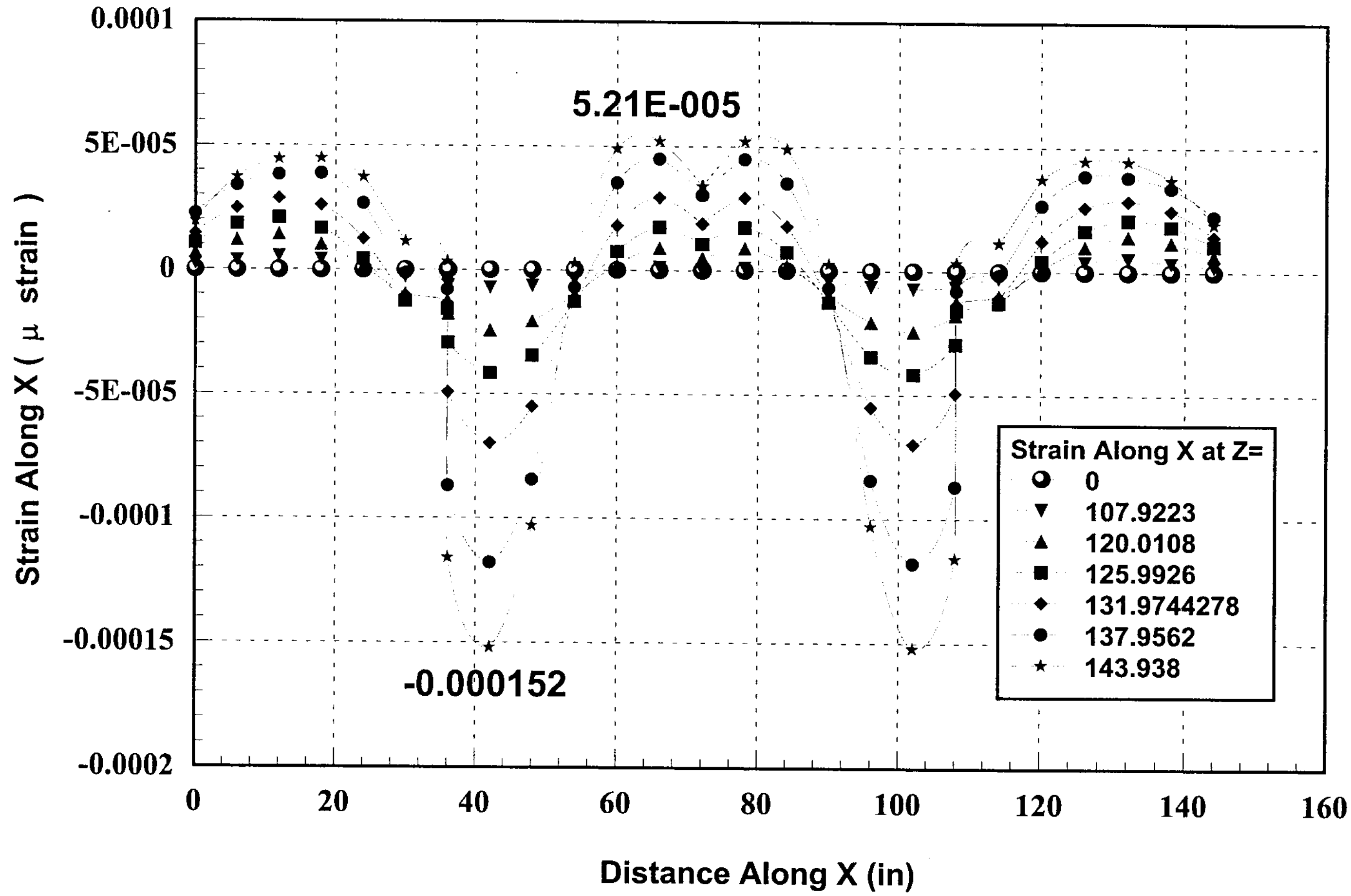


Figure 5.3.20. Vertical Deflection At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 4' x 4'

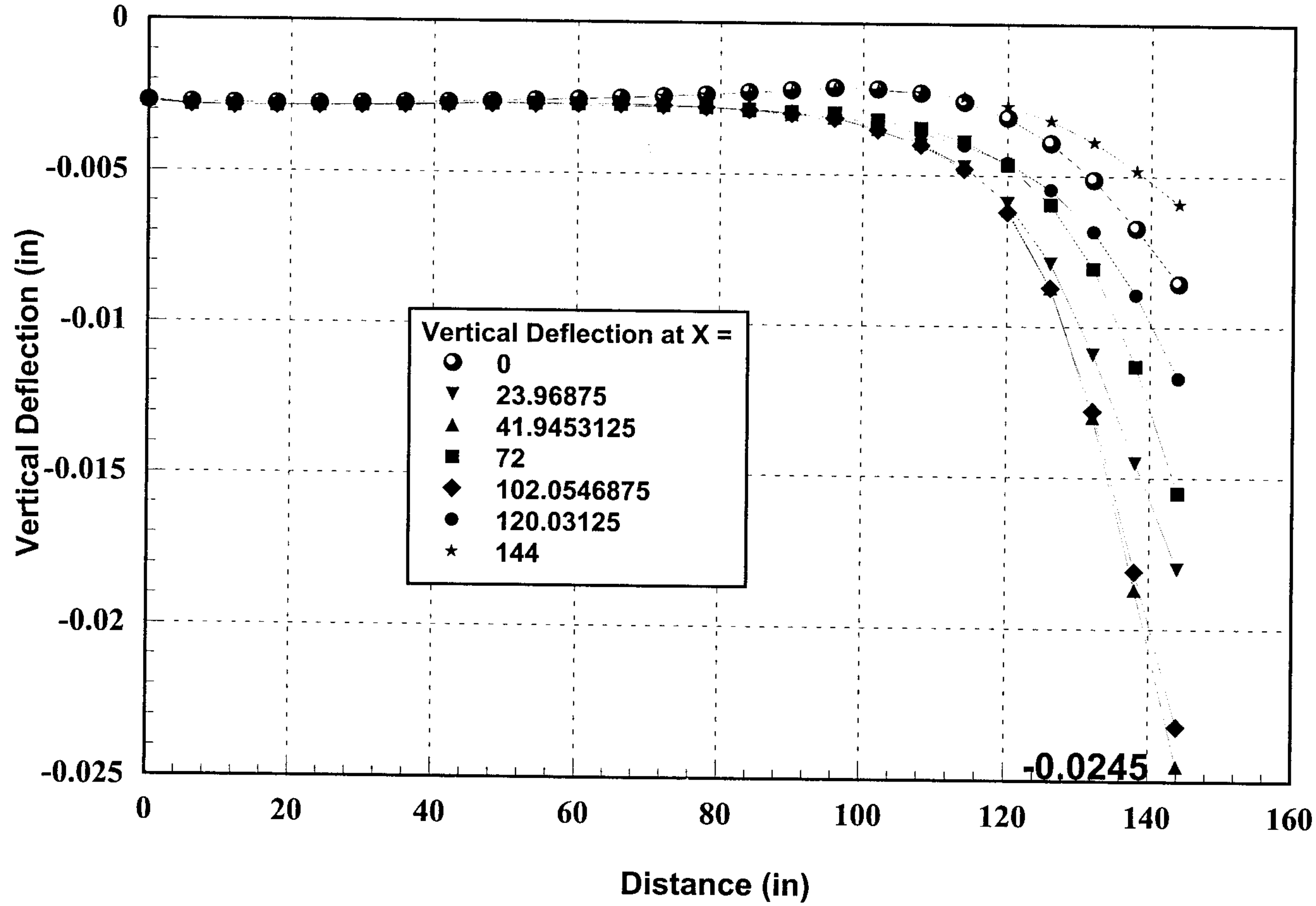


Figure 5.3.21. Stress At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 4' x 4'

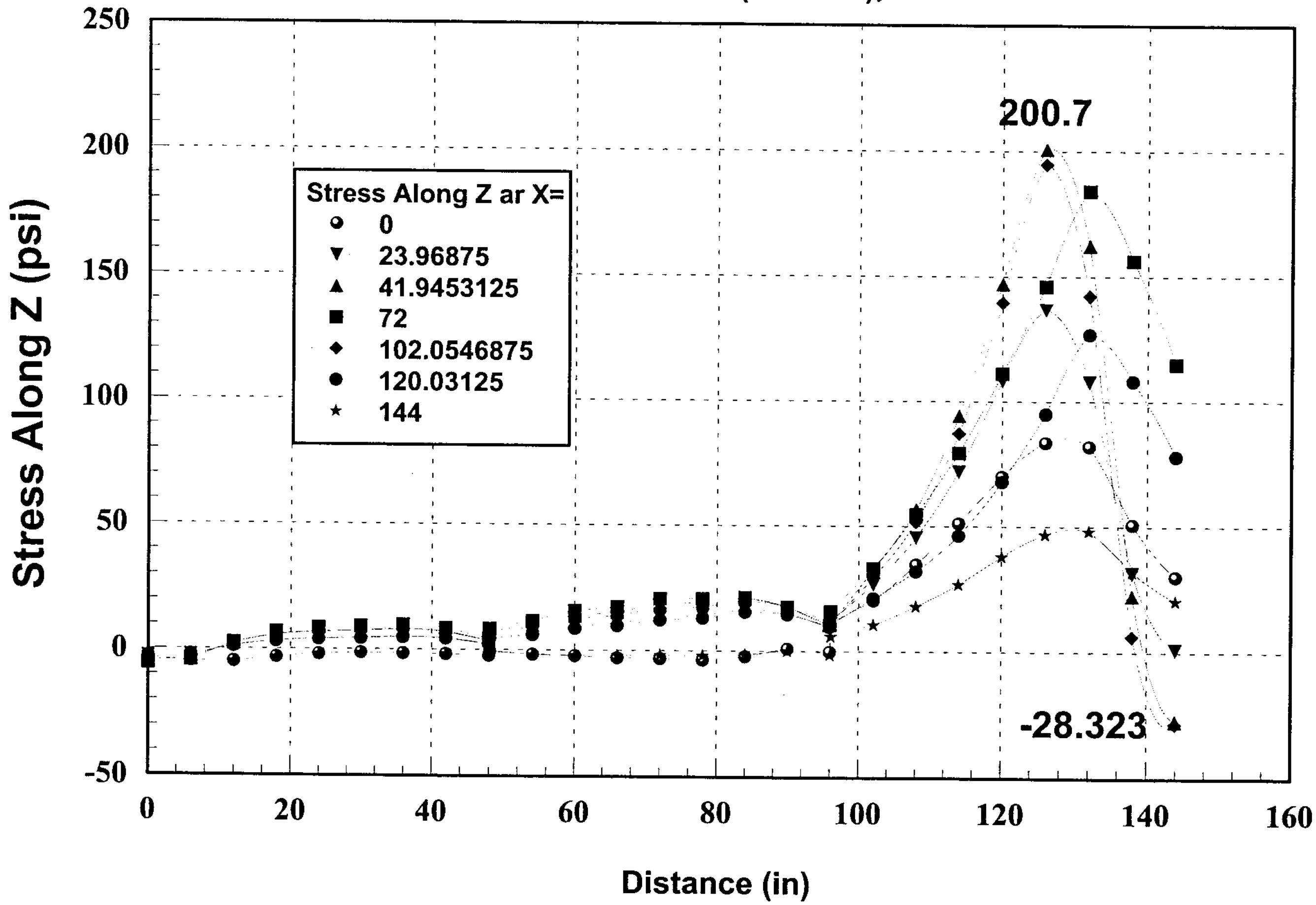


Figure 5.3.22. Strain At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 4' x 4'

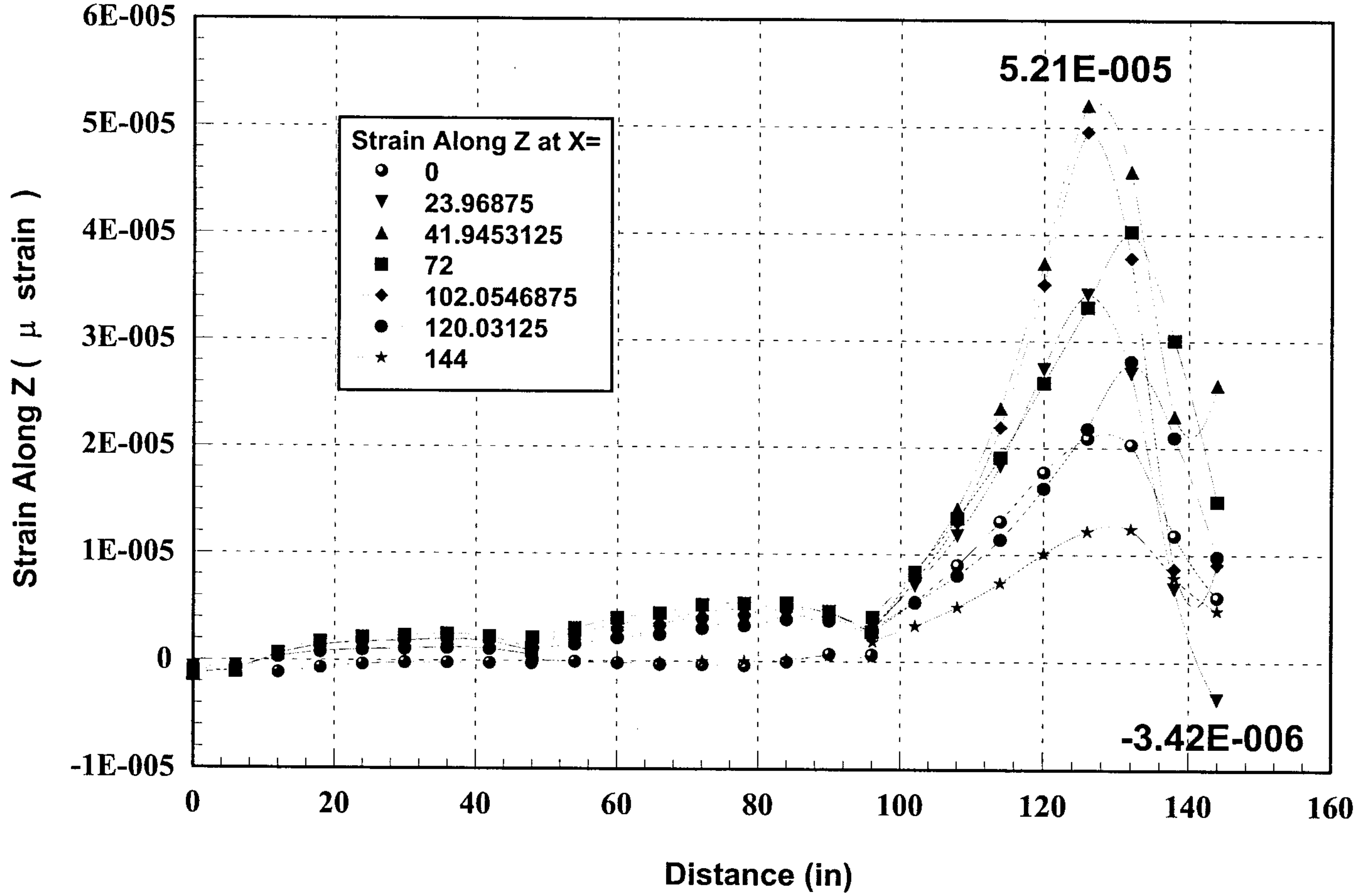


Figure 5.3.23. Vertical Deflection At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 4' x 4'

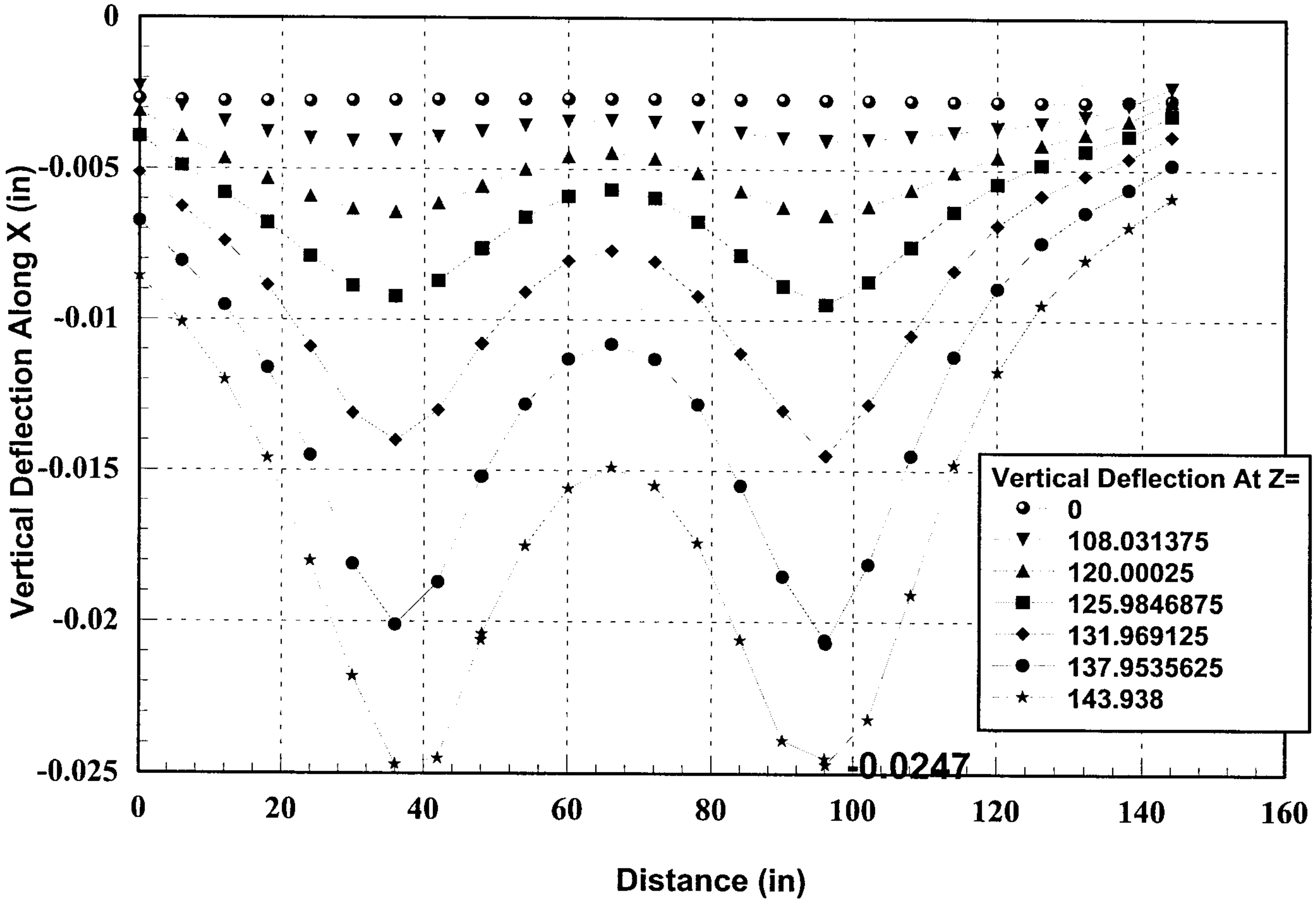


Figure 5.3.24. Stress At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 4' x 4'

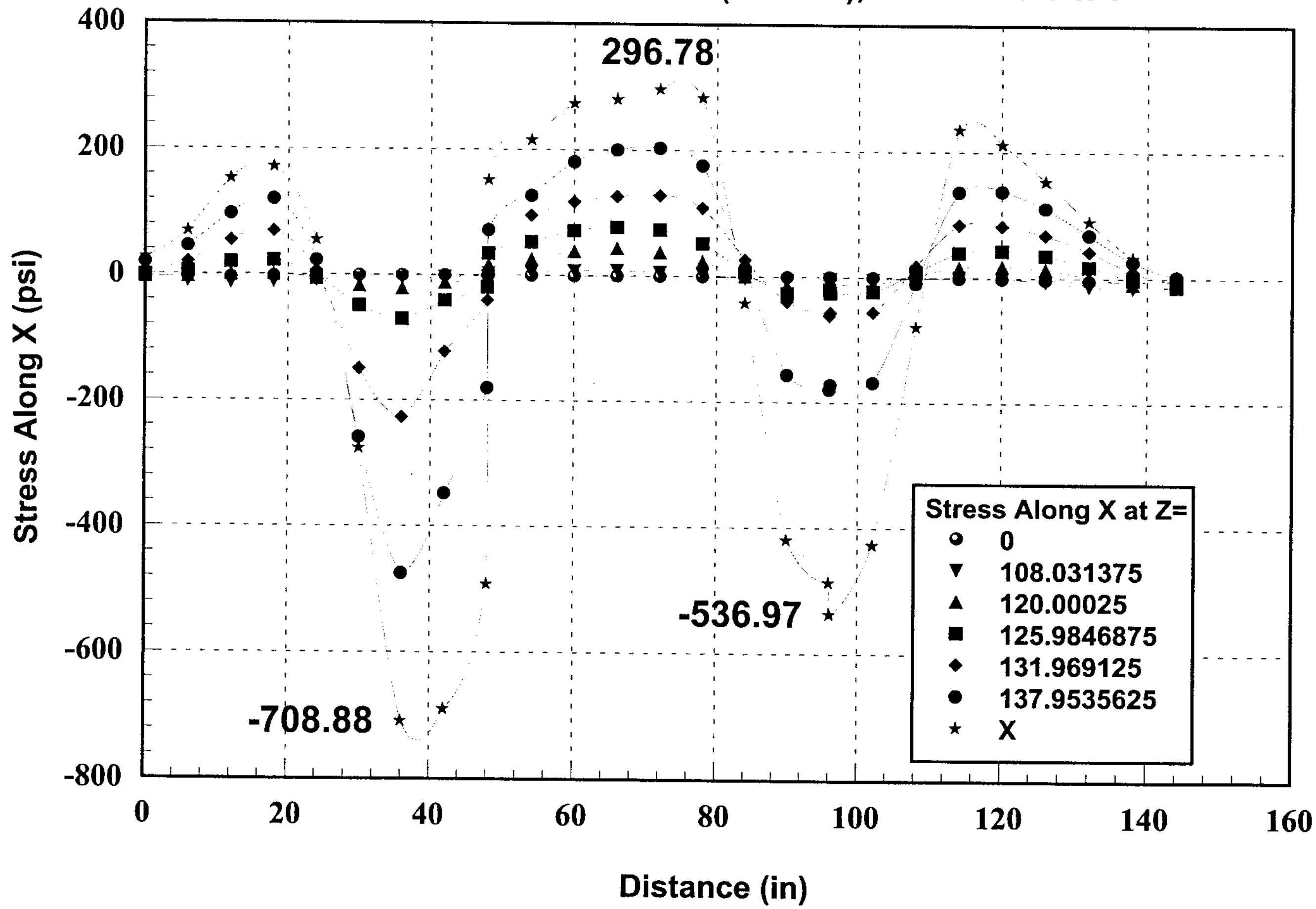


Figure 5.3.25. Strain At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 4' x 4'

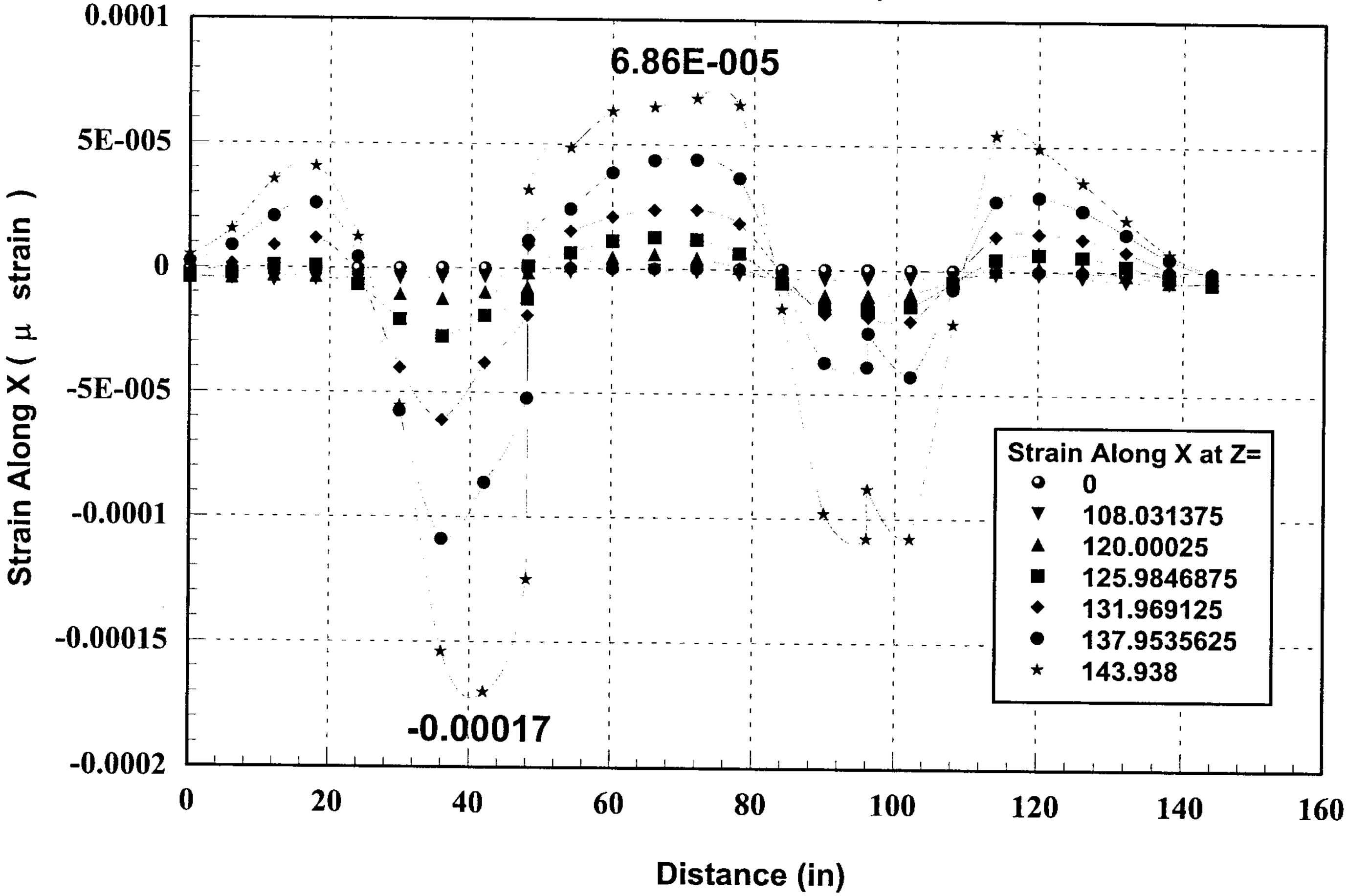


**Figure 5.3.26. Vertical Deflection At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 12' x 12'**

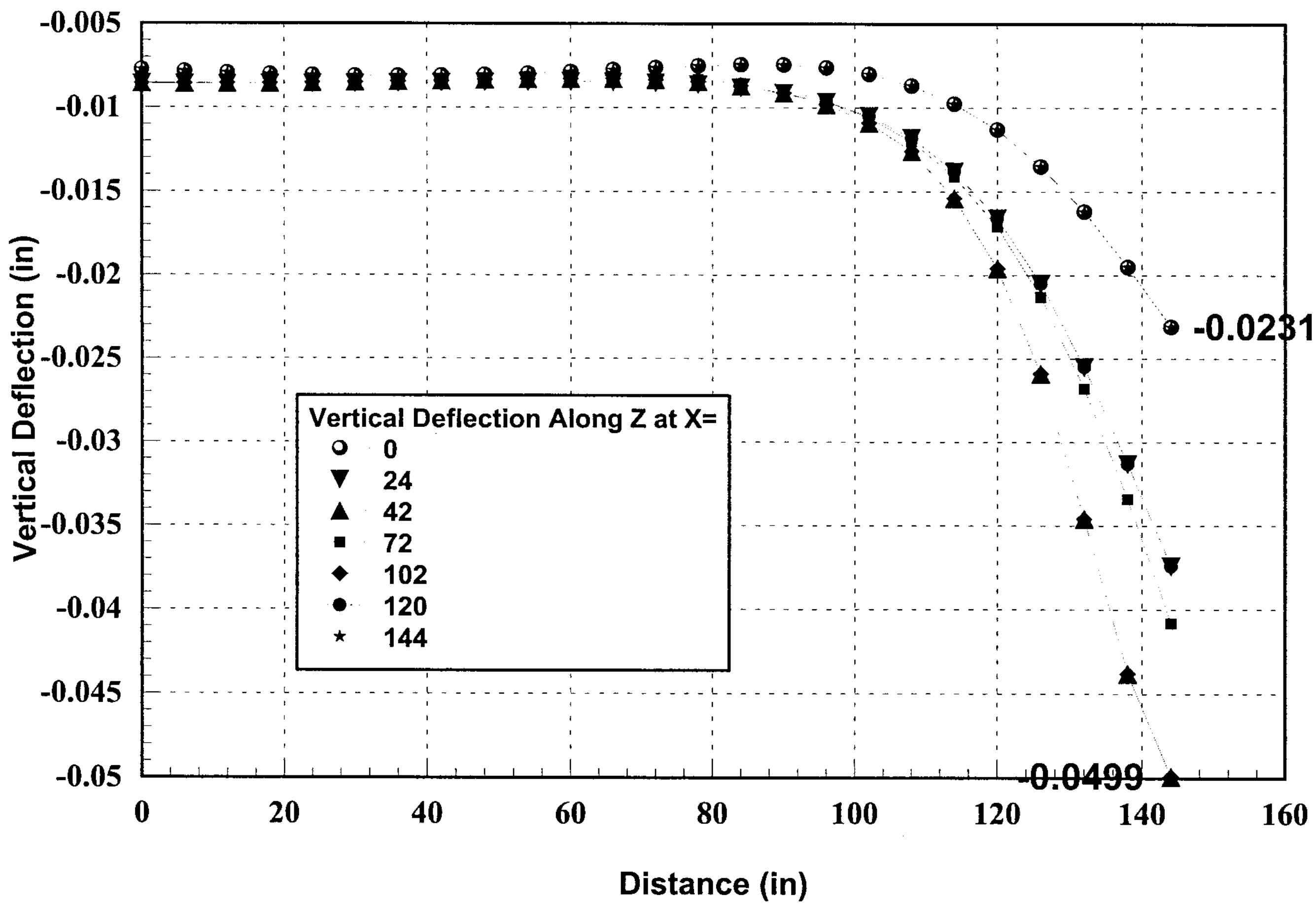


Figure 5.3.27. Stress At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 12' x 12

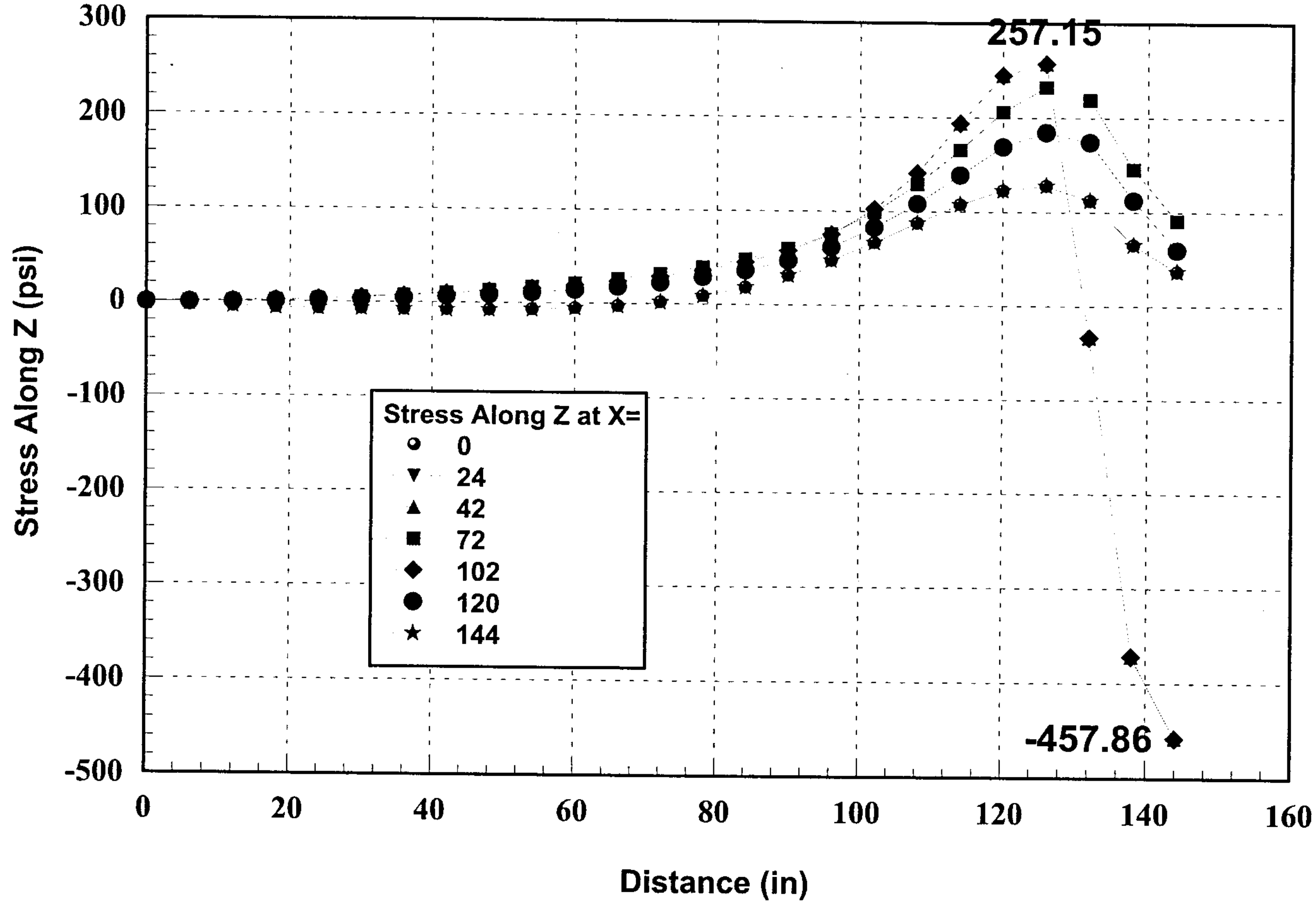


Figure 5.3.28. Strain At Different Sections Along The Length Of The Track # 2 (Friction), Panel Size 12' x 12

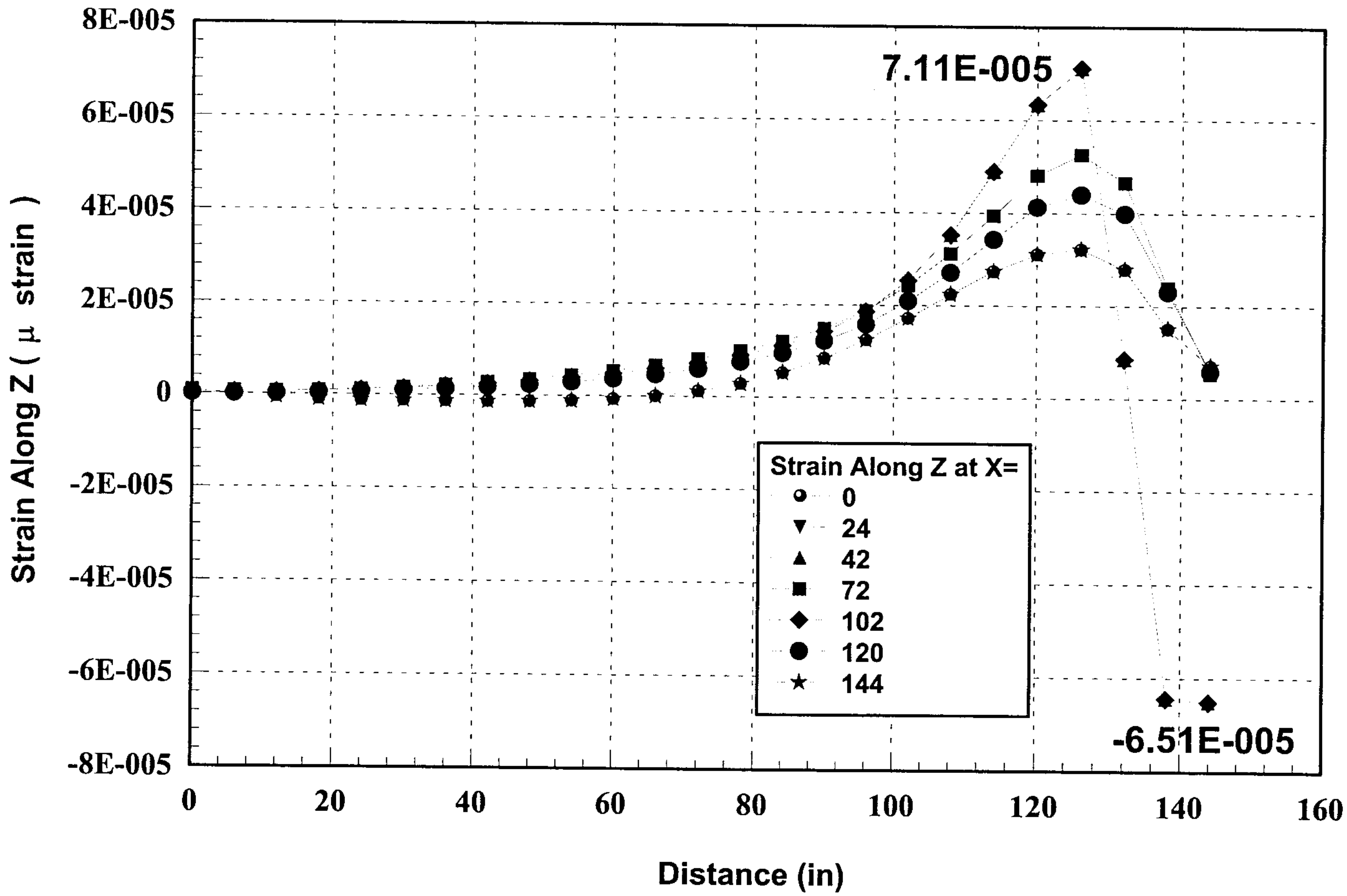


Figure 5.3.29. Vertical Deflection At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 12' x 12

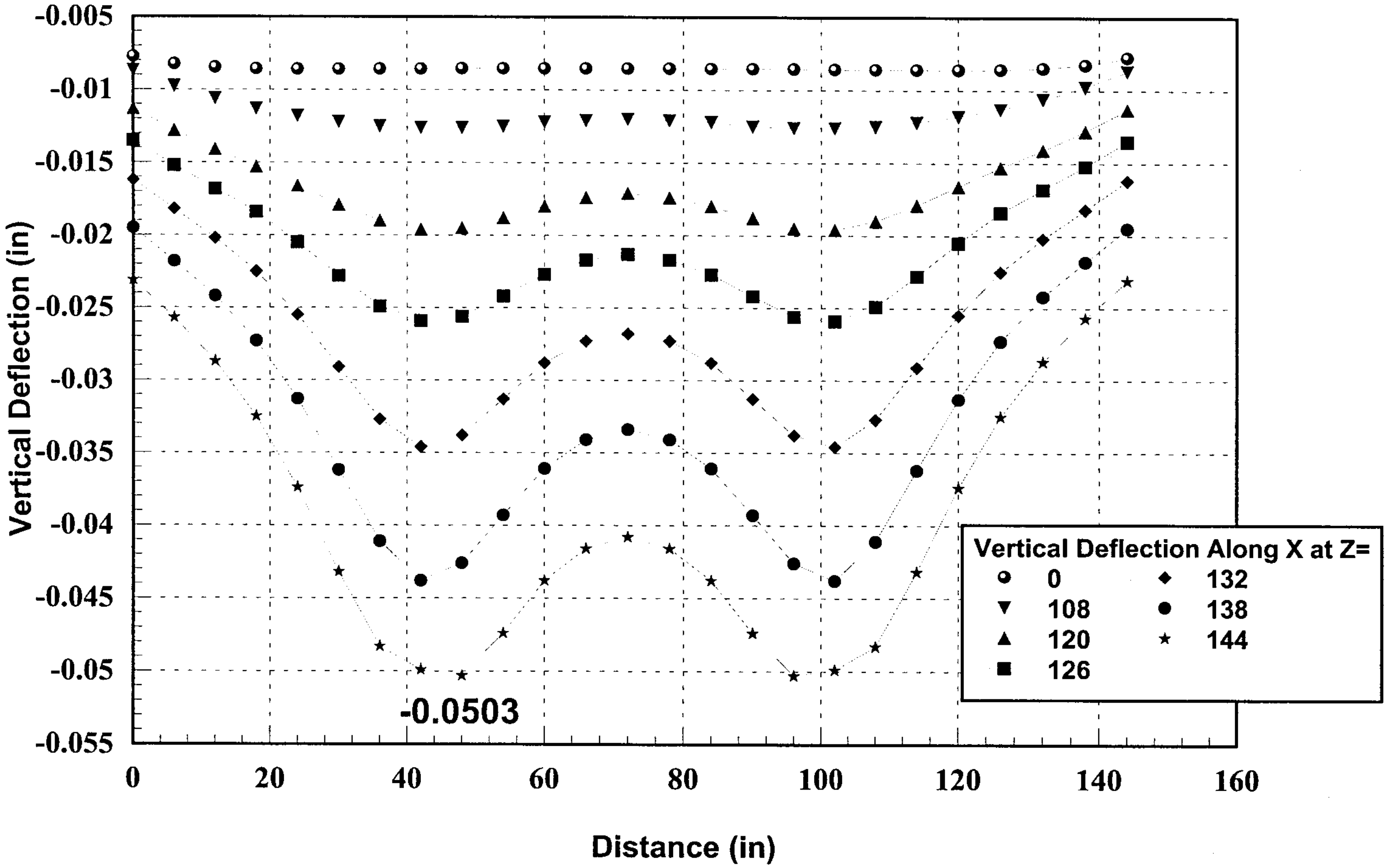


Figure 5.3.30. Stress At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 12' x 12

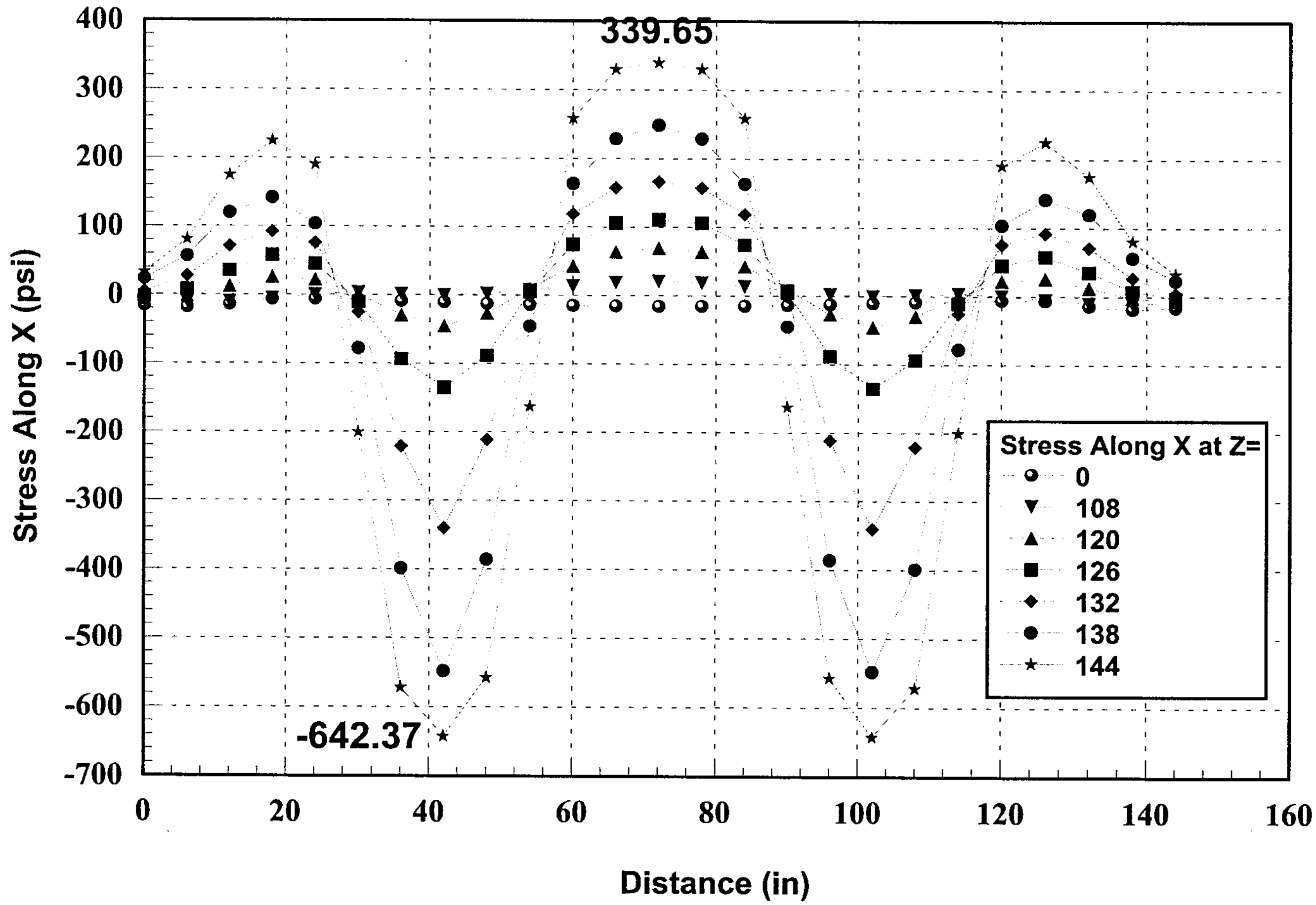


Figure 5.3.31. Strain At Different Sections Along The Width Of The Track # 2 (Friction), Panel Size 12' x 12

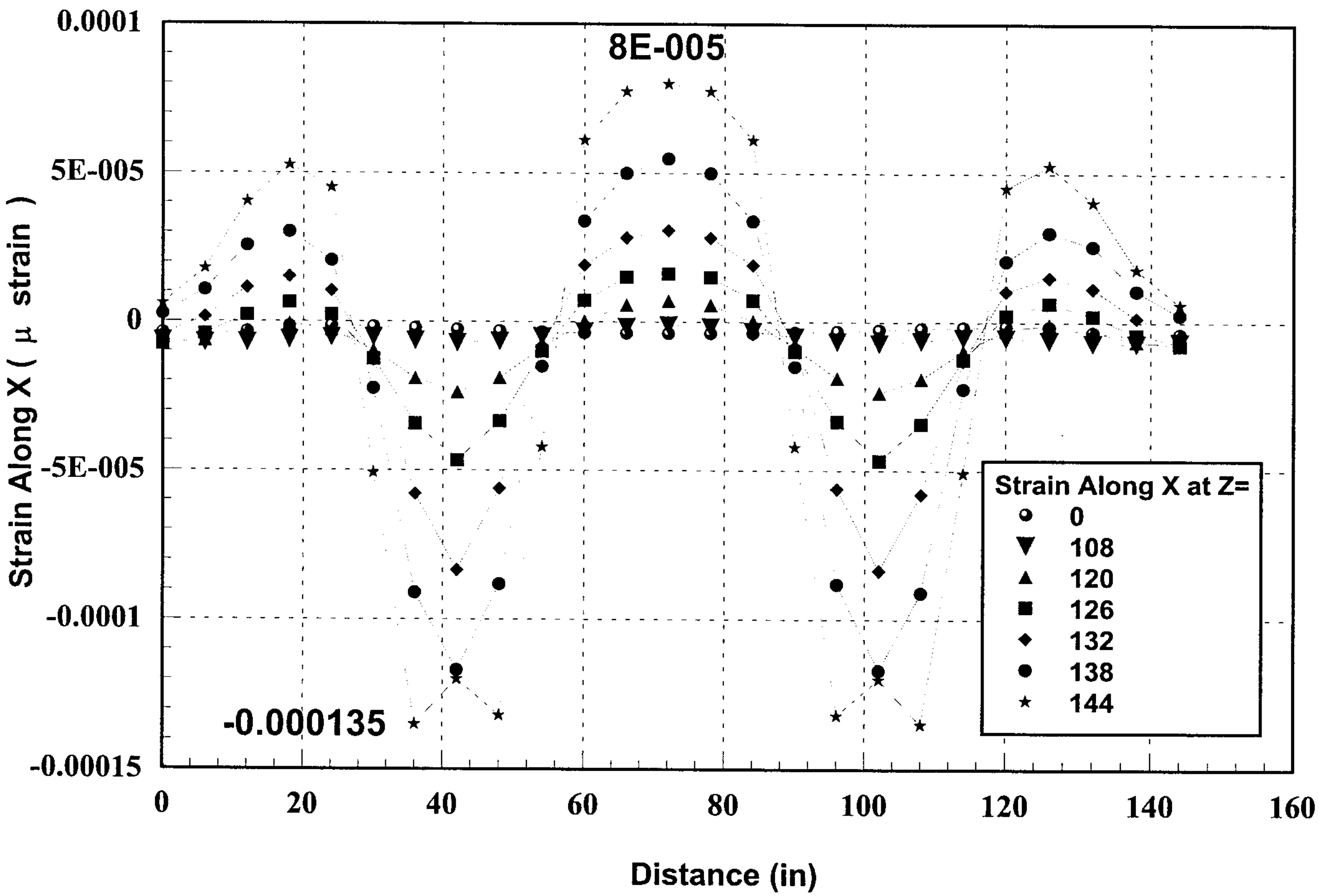


Figure 5.3.32. Vertical Deflection At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 4' x 4'

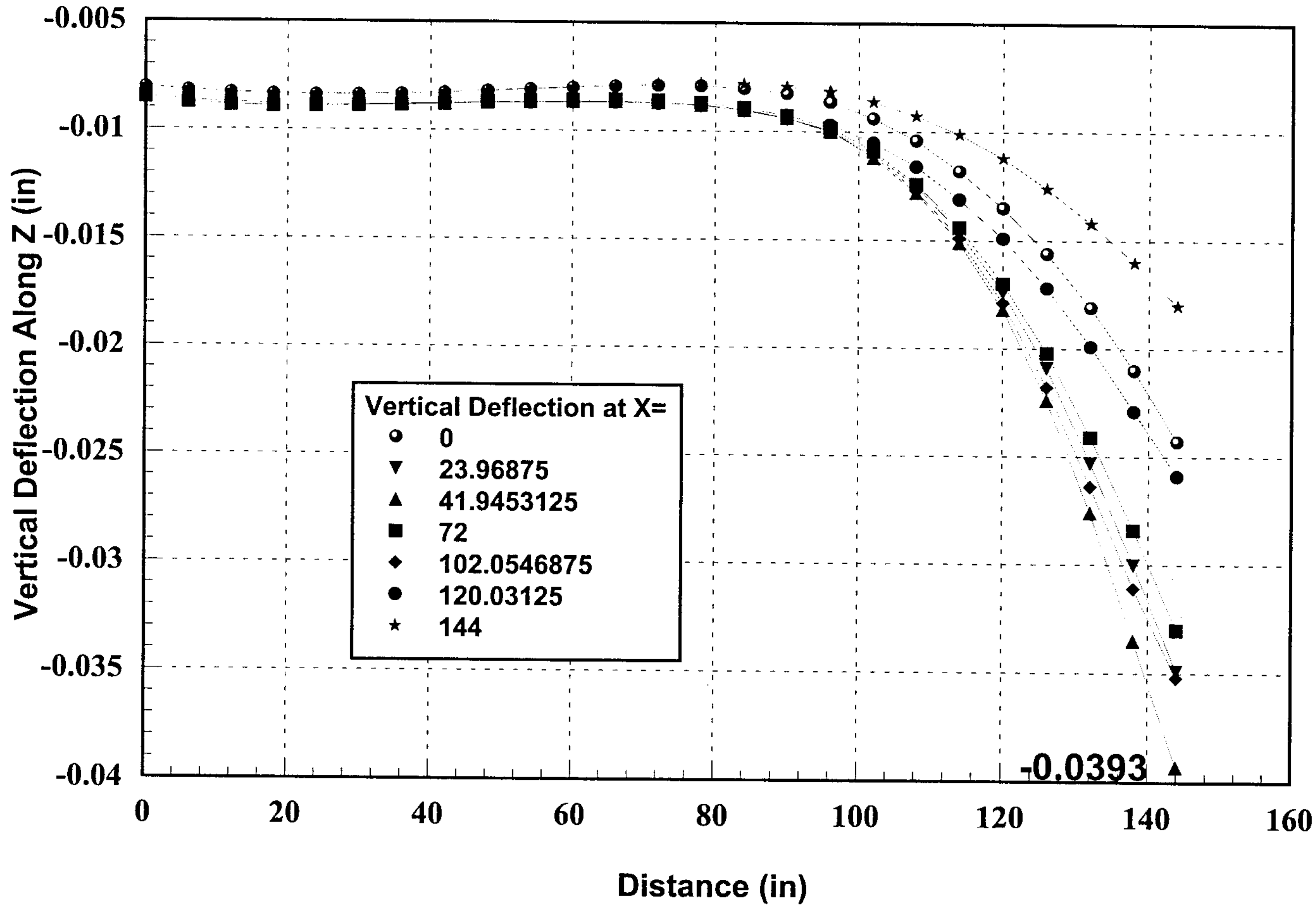


Figure 5.3.33. Stress At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 4' x 4'

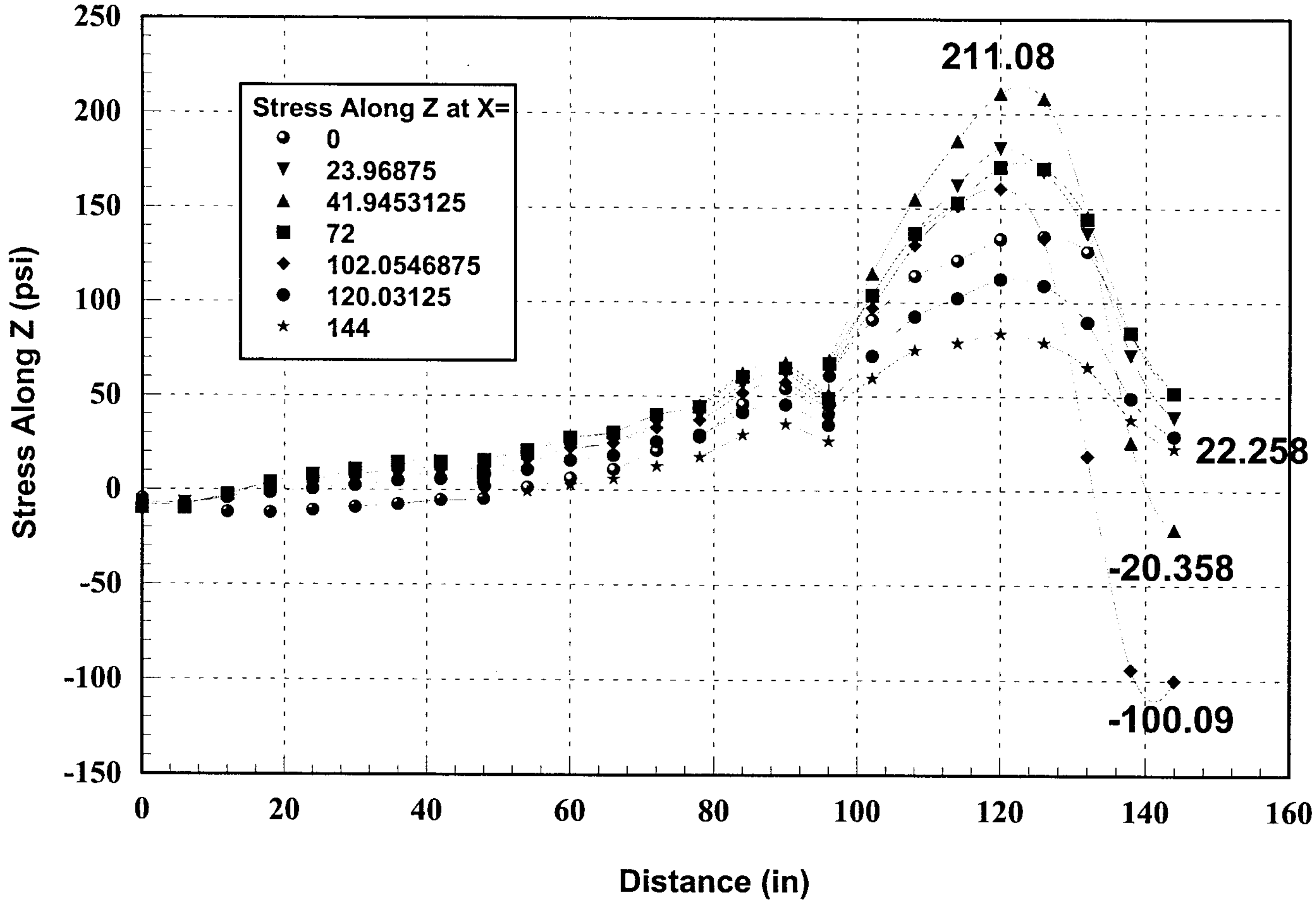


Figure 5.3.34. Strain At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 4' x 4'

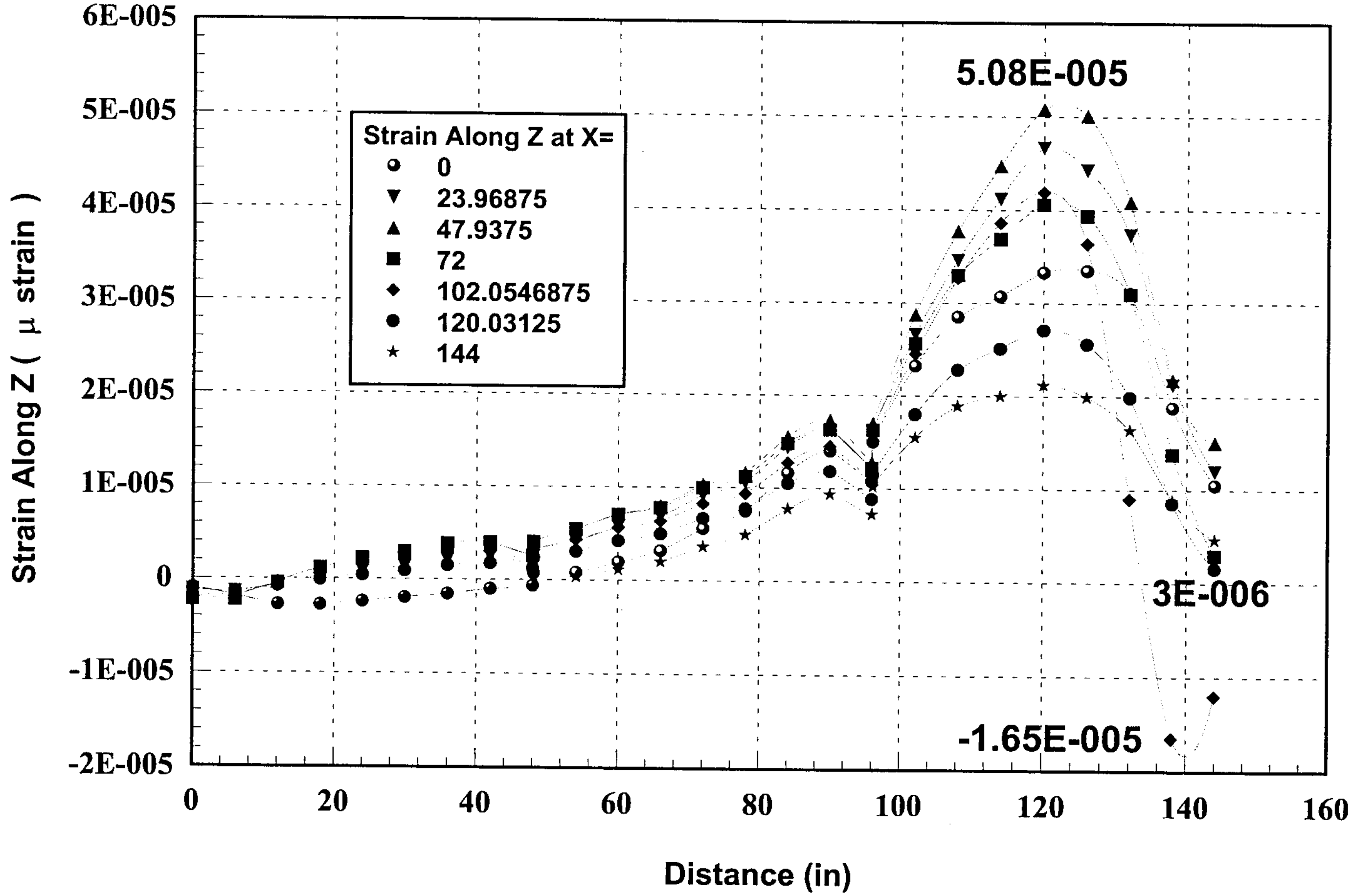


**Figure 5.3.35. Vertical Deflection At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 4' x 4'**

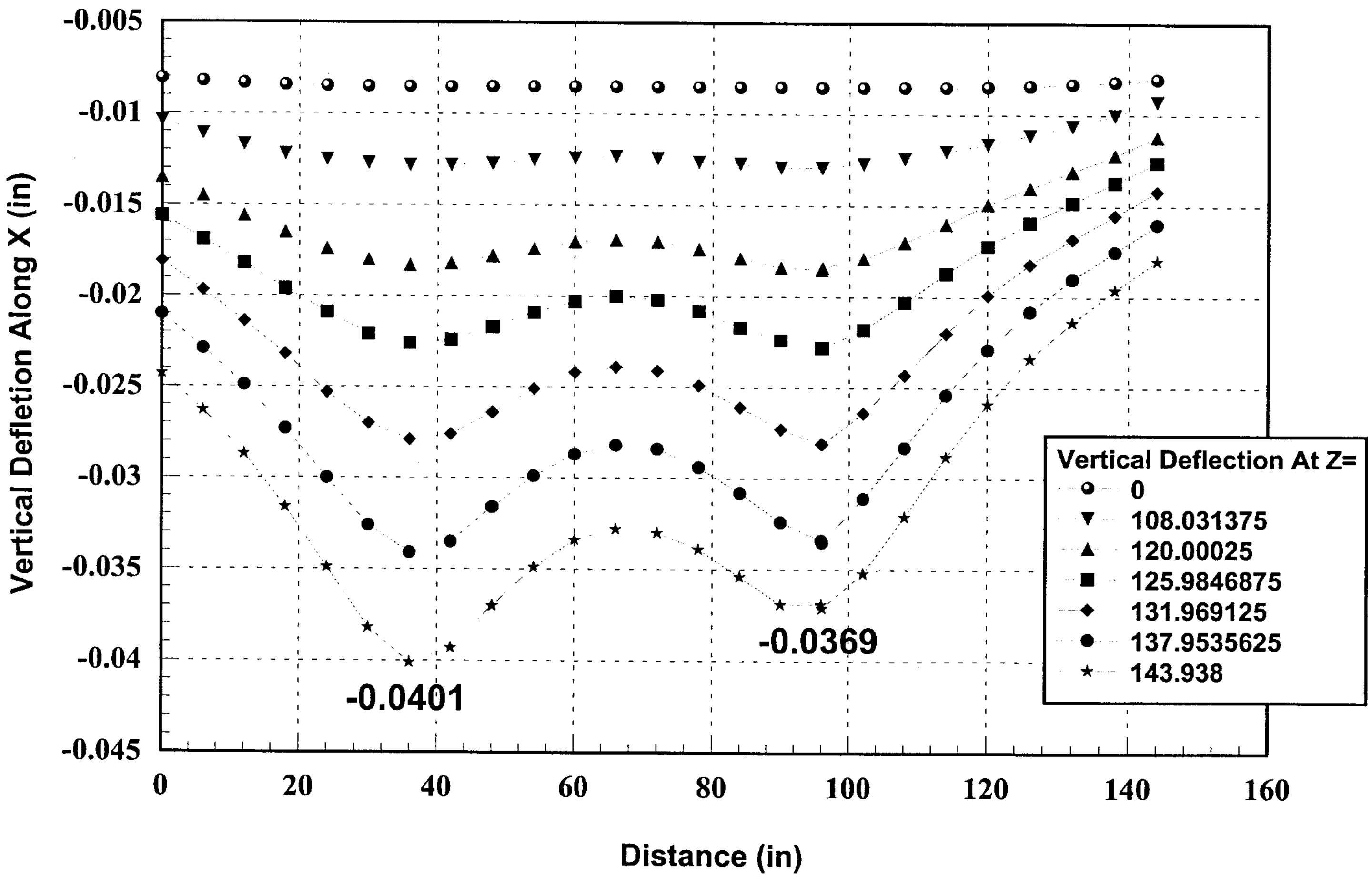
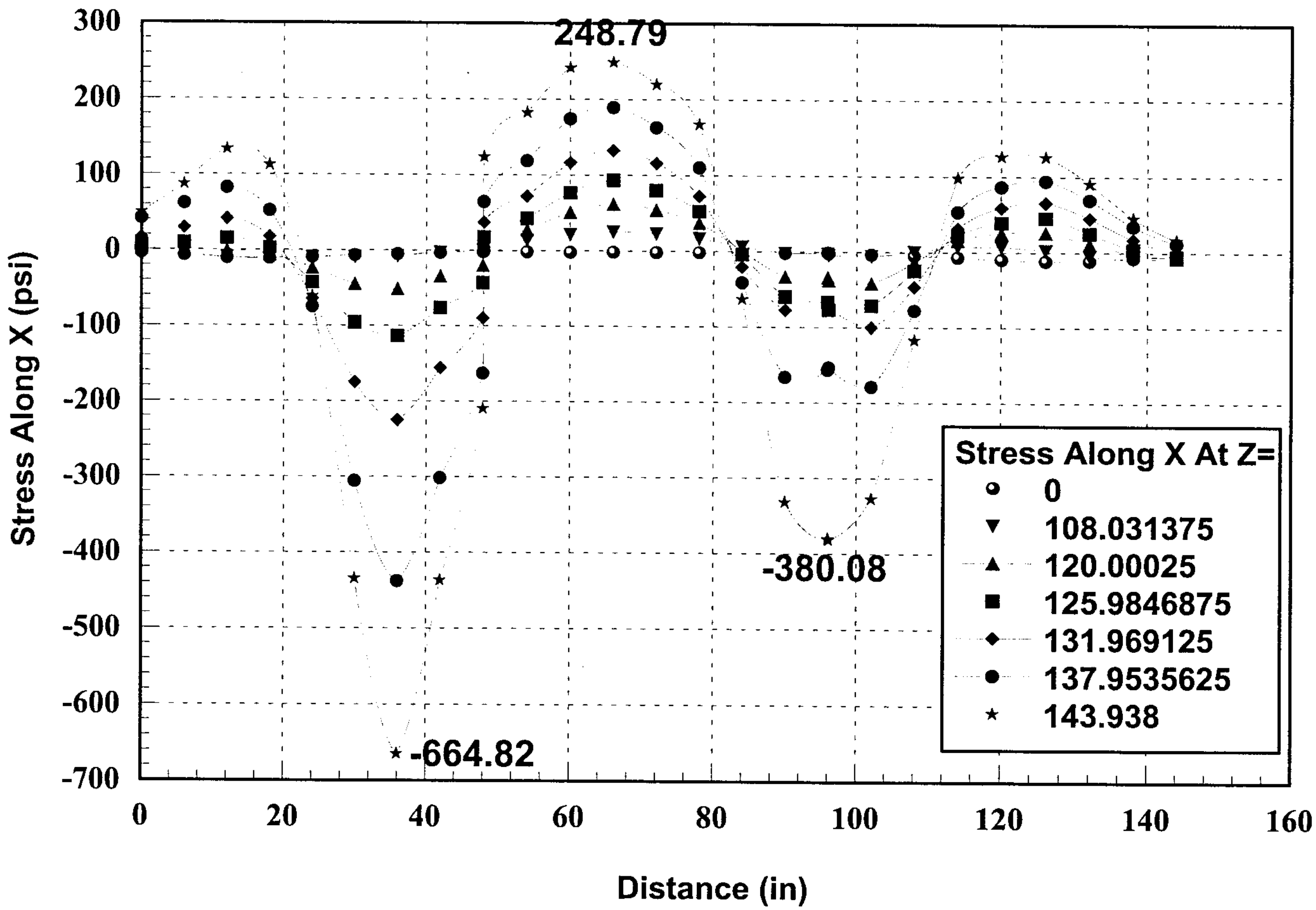


Figure 5.3.36. Stress At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 4' x 4'



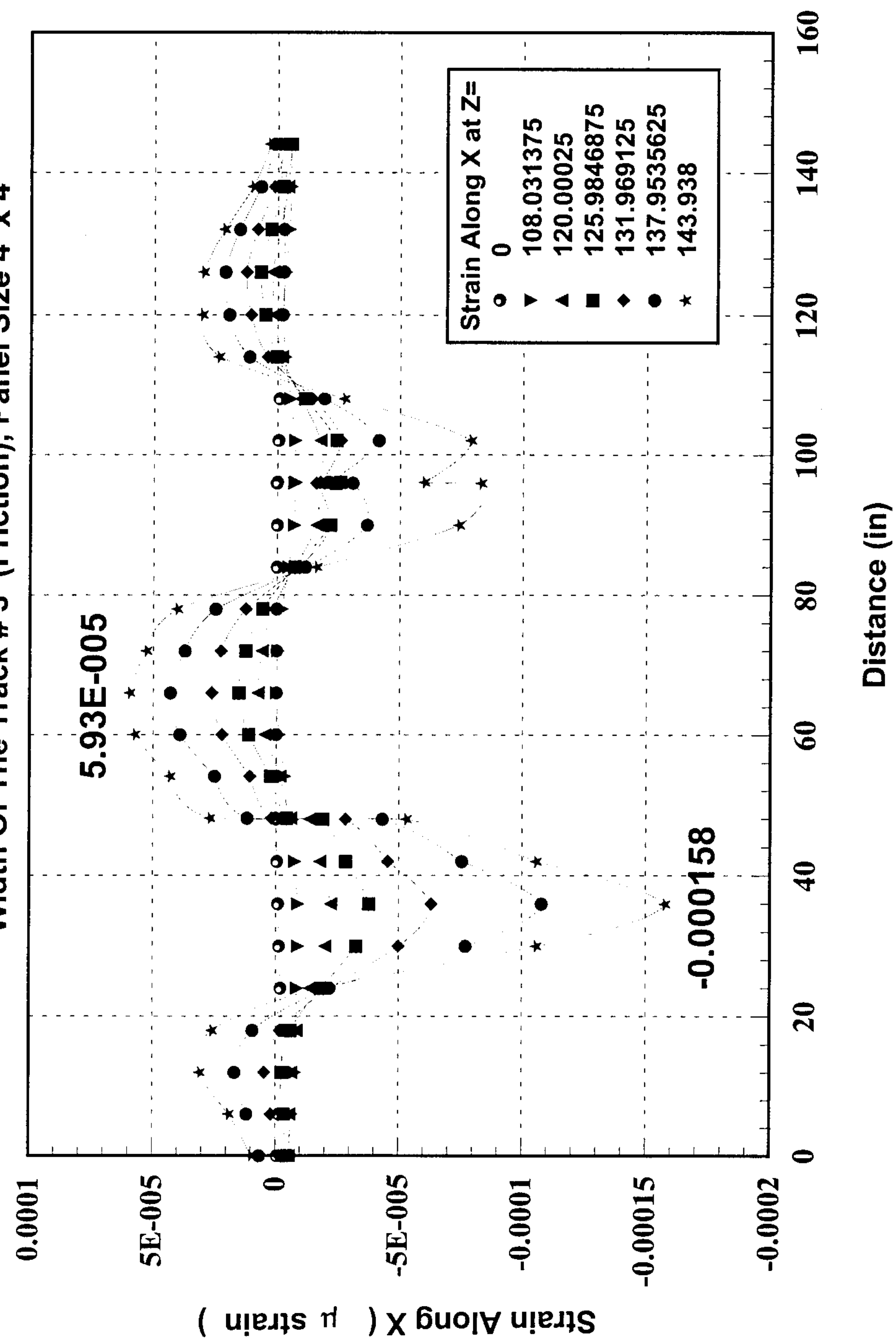



Figure 5.3.37. Strain At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 4' x 4'

Figure 5.3.38. Vertical Deflection At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 6' x 6'

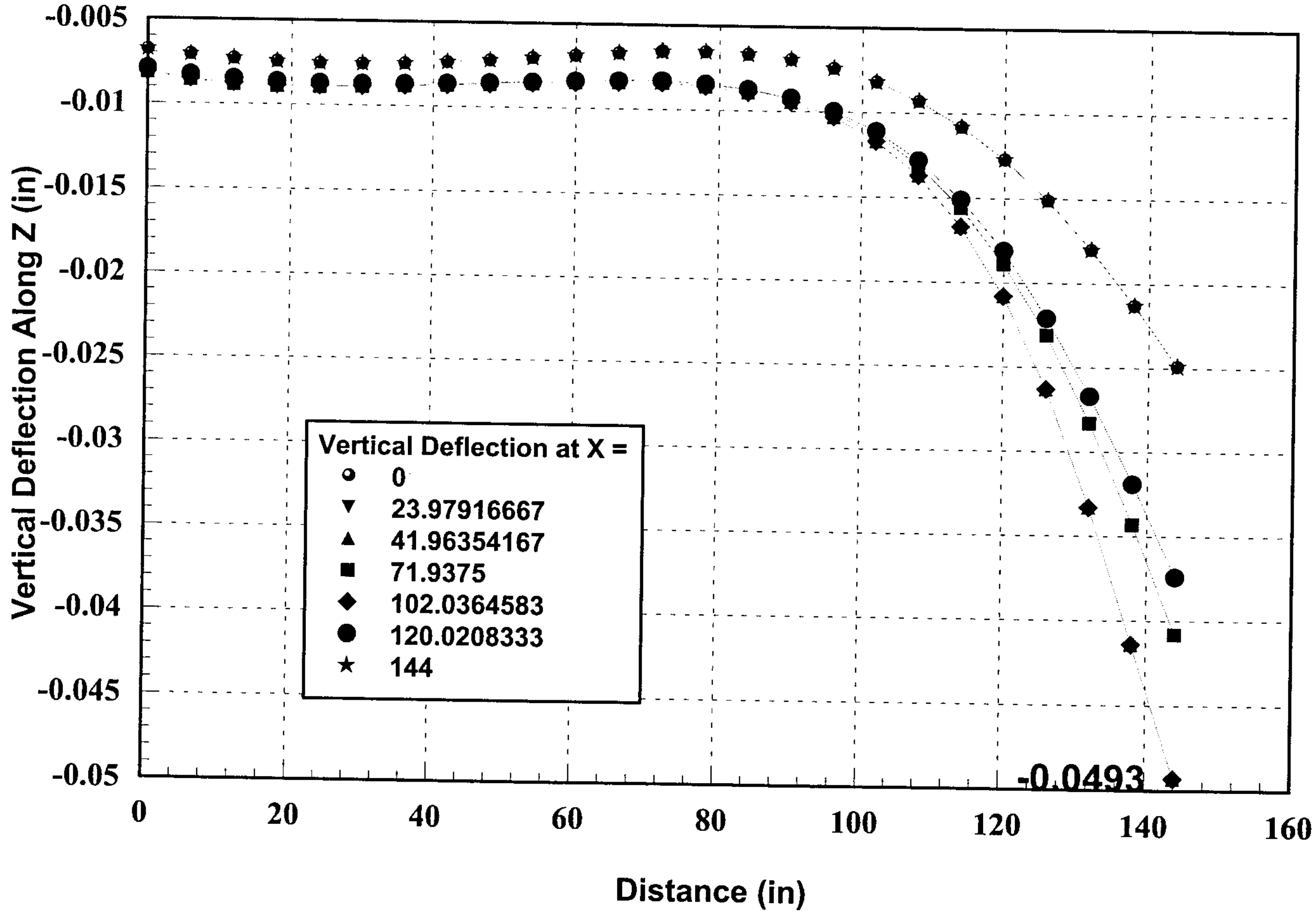


Figure 5.3.39. Stress At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 6' x 6'

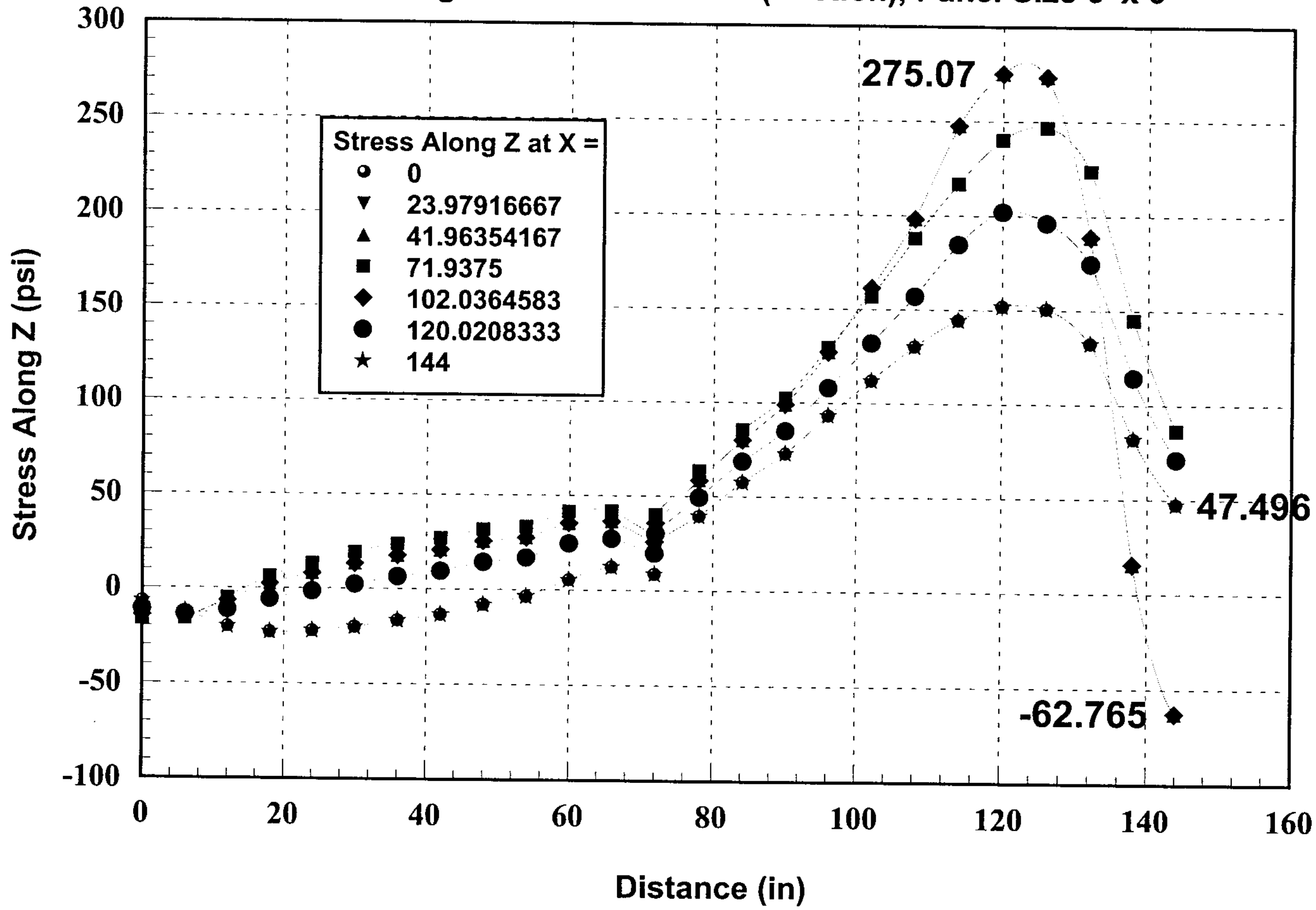


Figure 5.3.40. Strain At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 6' x 6'

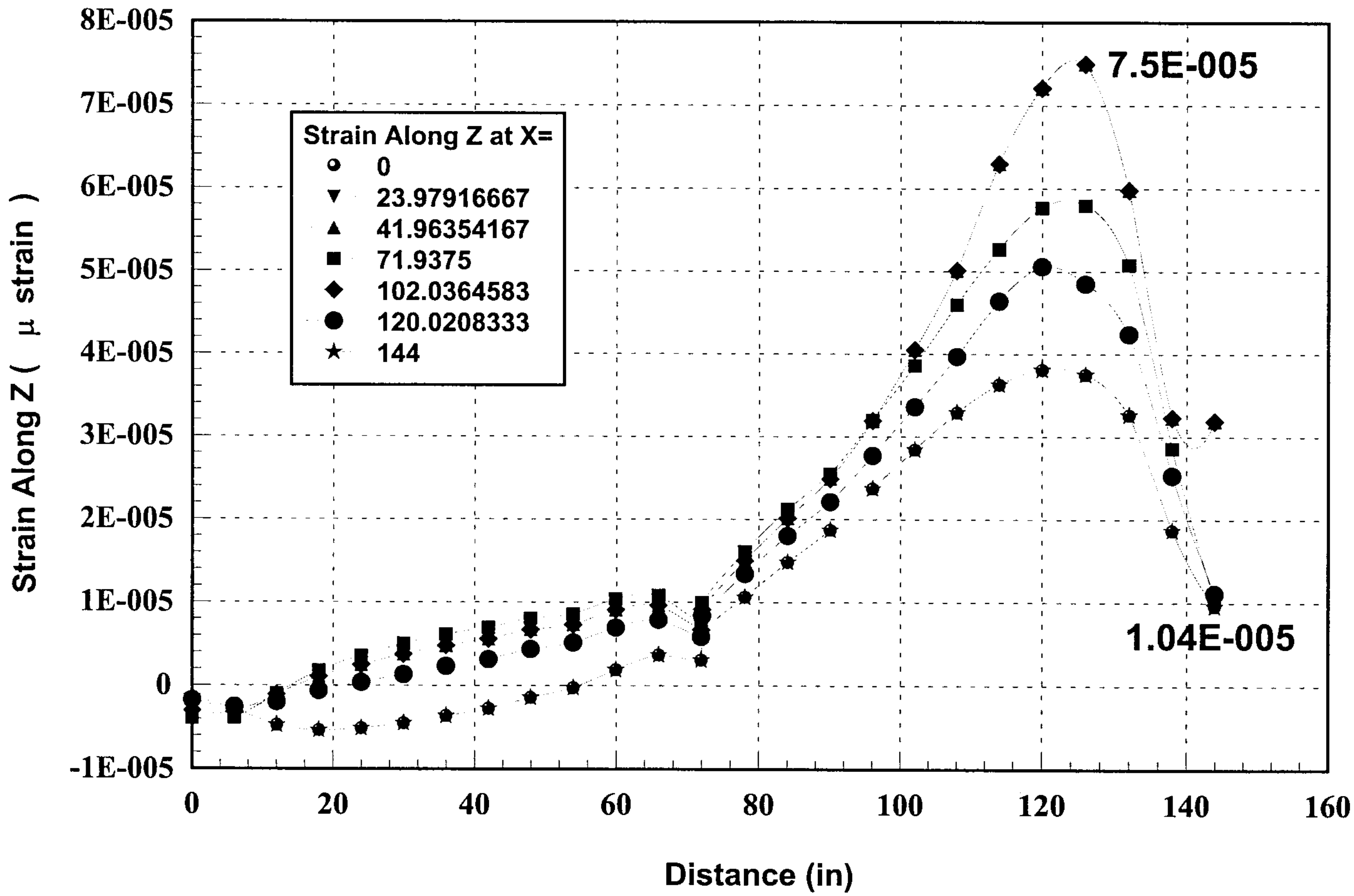


Figure 5.3.41. Vertical Deflection At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 6' x 6'

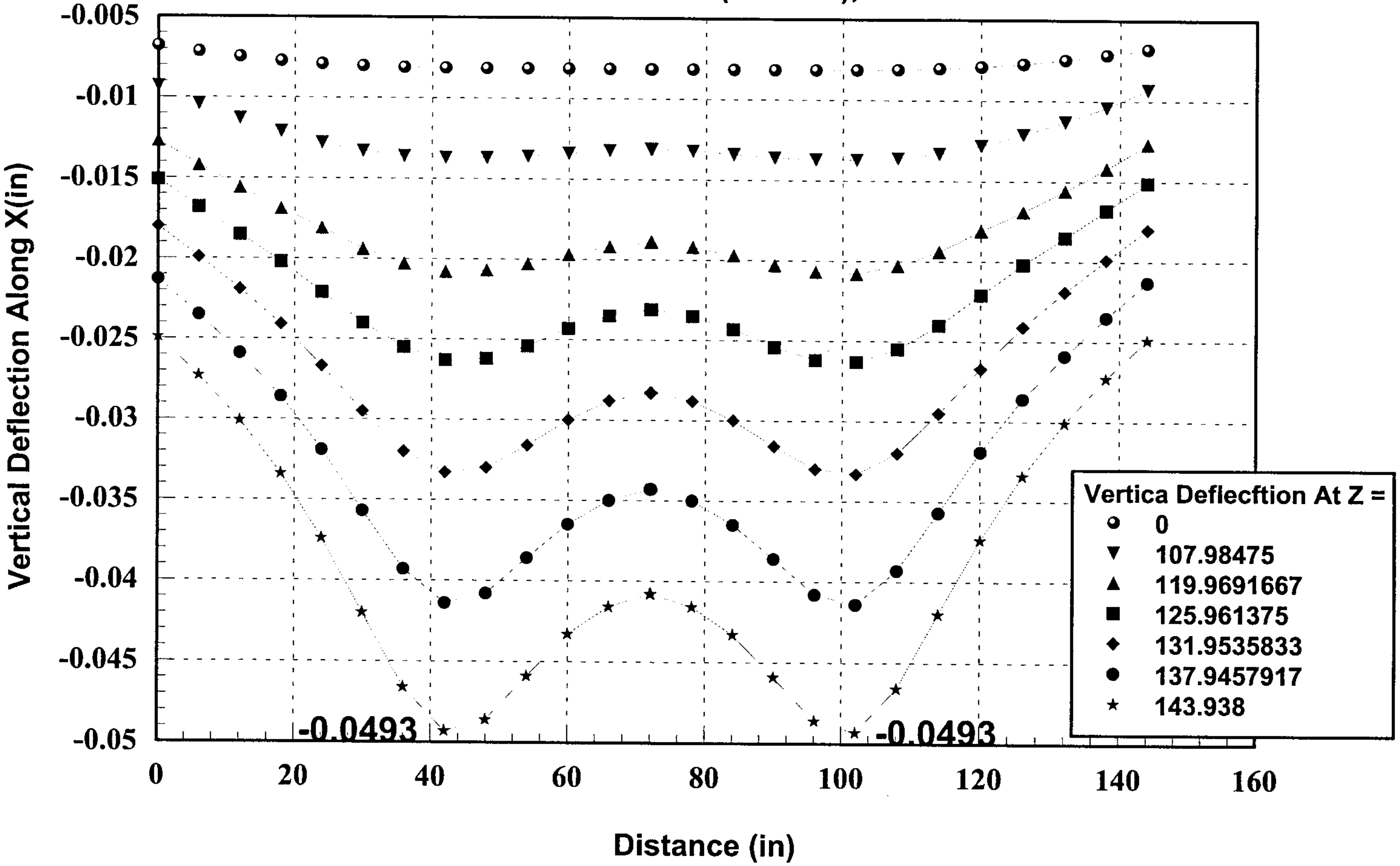


Figure 5.3.42. Stress At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 6' x 6'

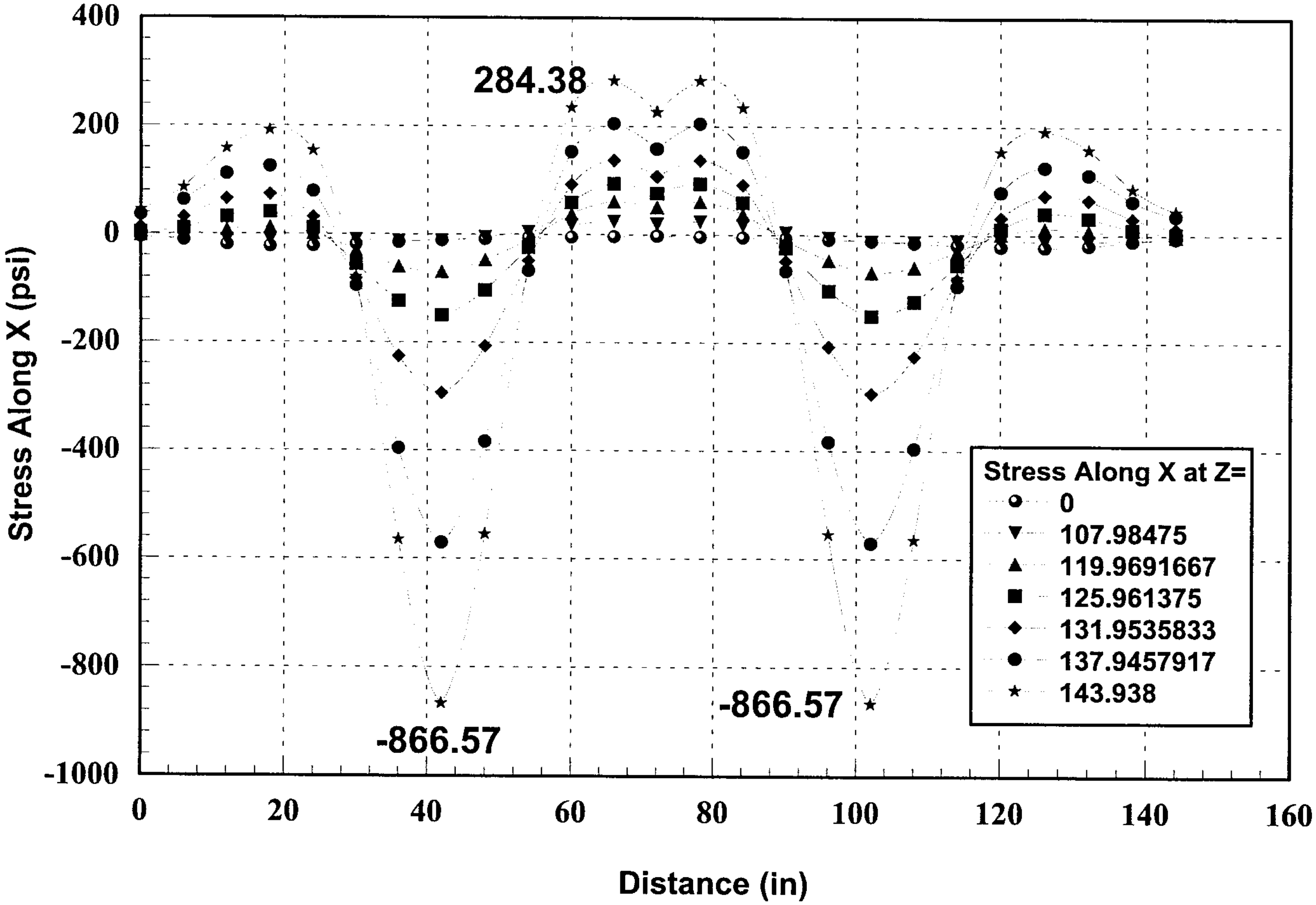


Figure 5.3.43. Strain At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 6' x 6'

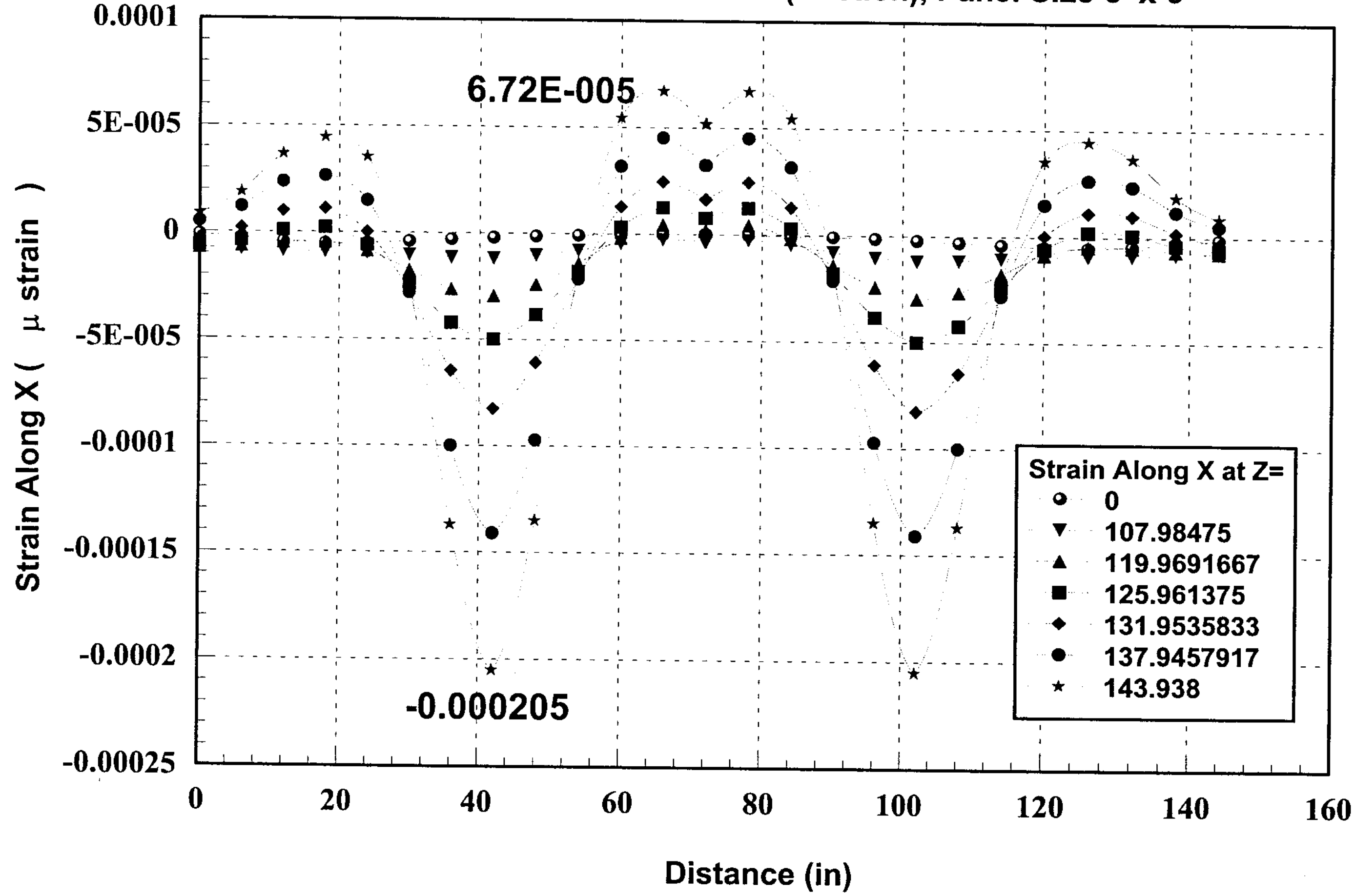


Figure 5.3.44 Vertical Deflection At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 12' x 12'

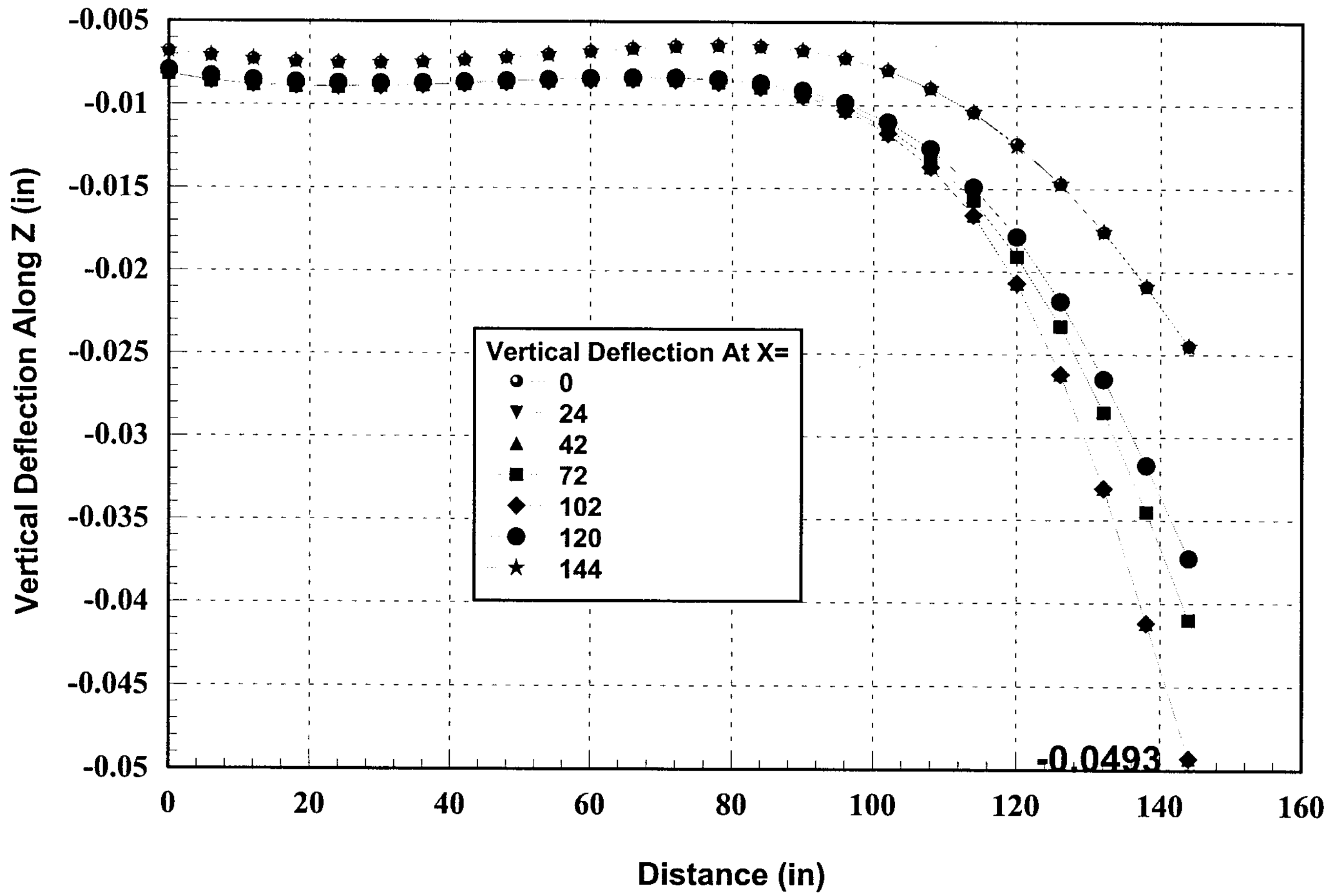


Figure 5.3.45. Stress At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 12' x 12'

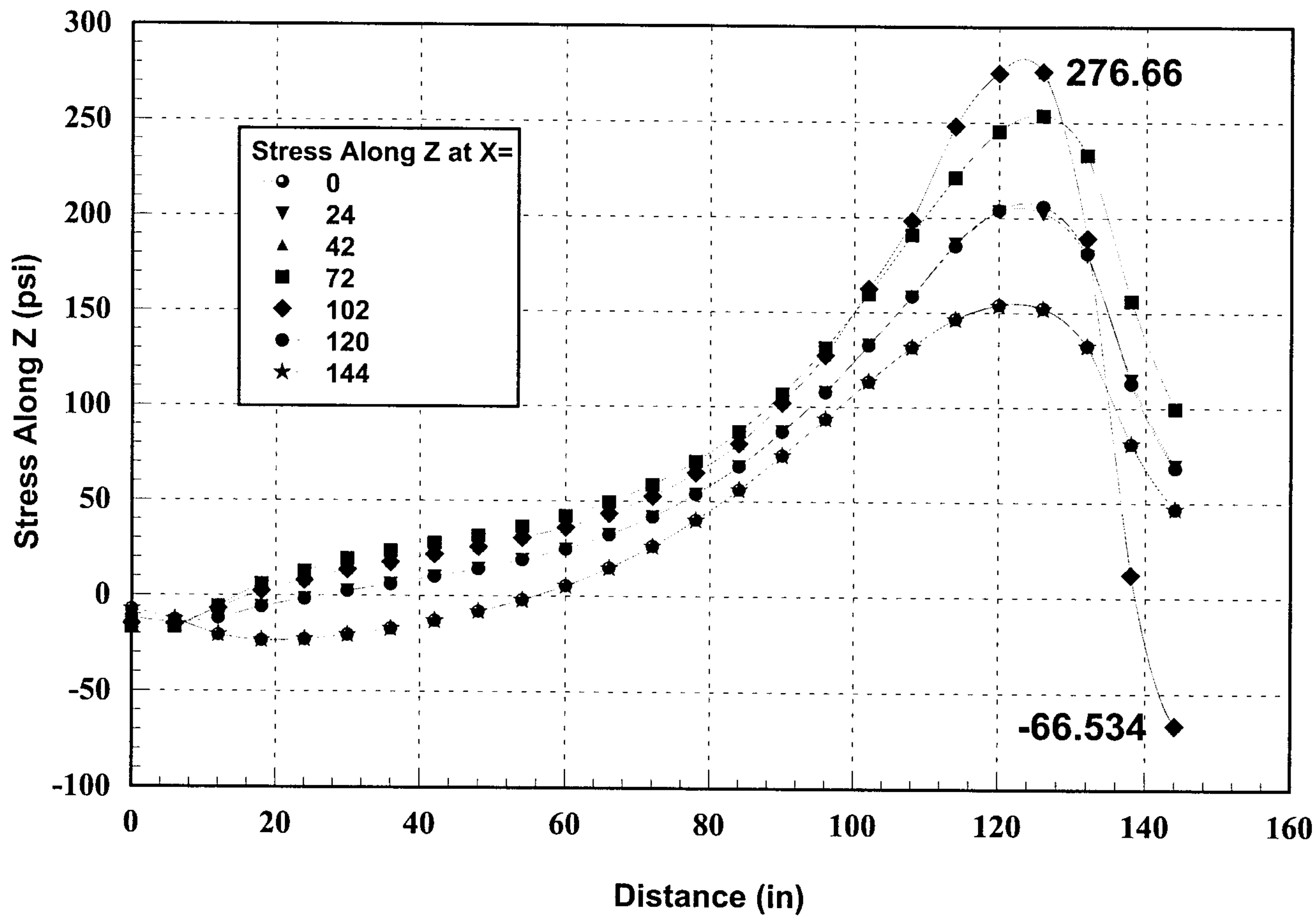


Figure 5.3.46. Strain At Different Sections Along The Length Of The Track # 3 (Friction), Panel Size 12' x 12'

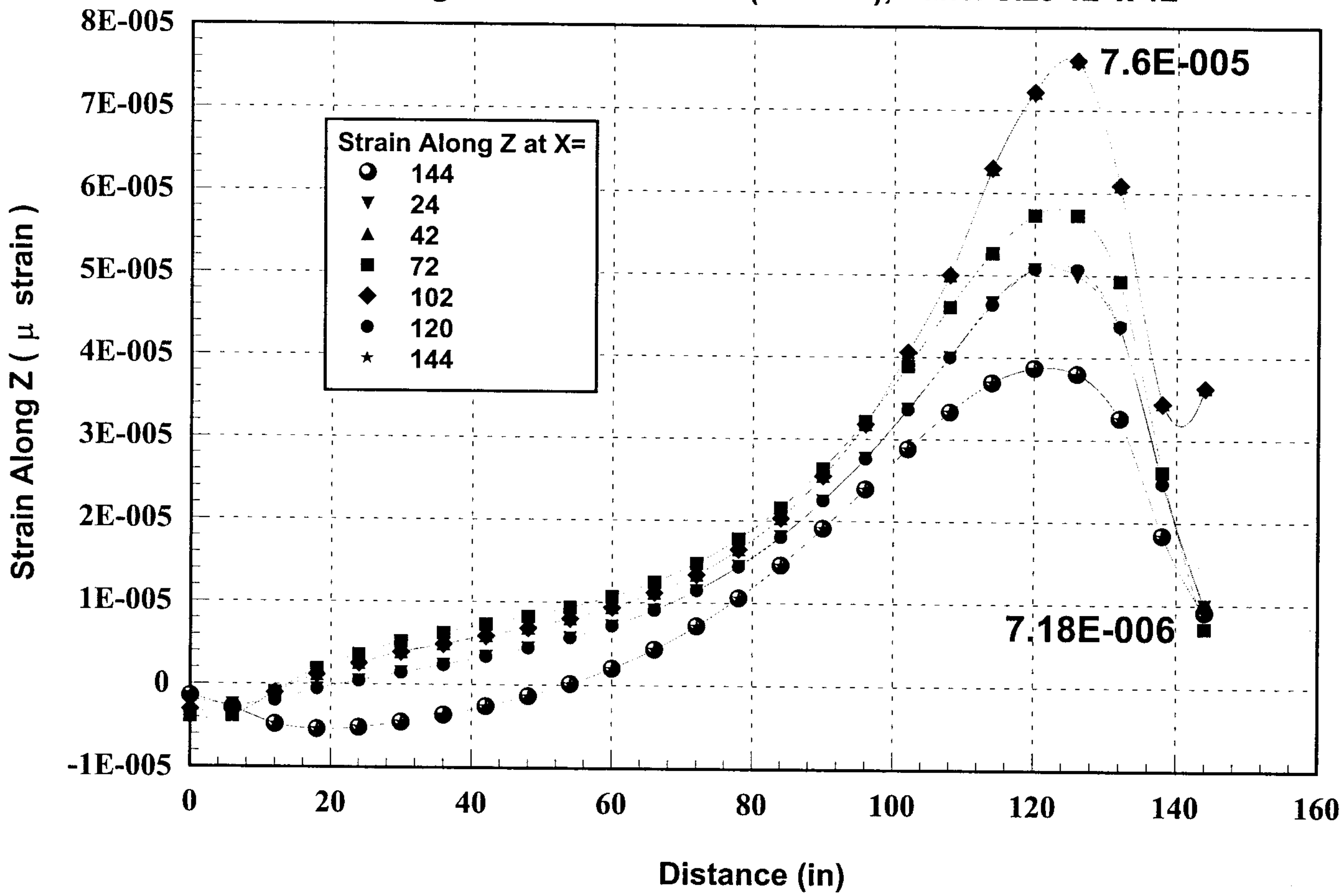


Figure 5.3.47. Vertical Deflection At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 12' x 12'

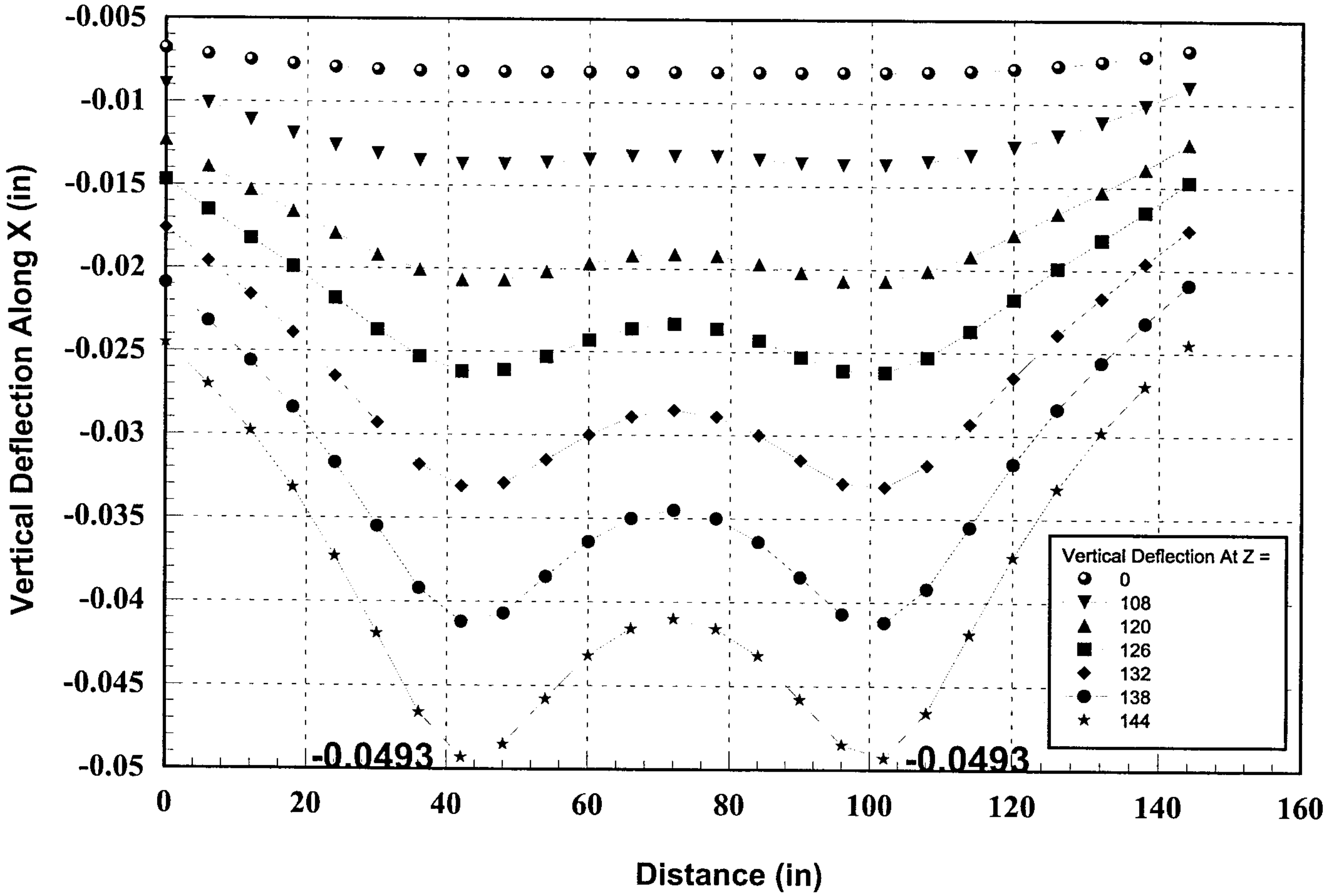


Figure 5.3.48. Stress At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 12' x 12'

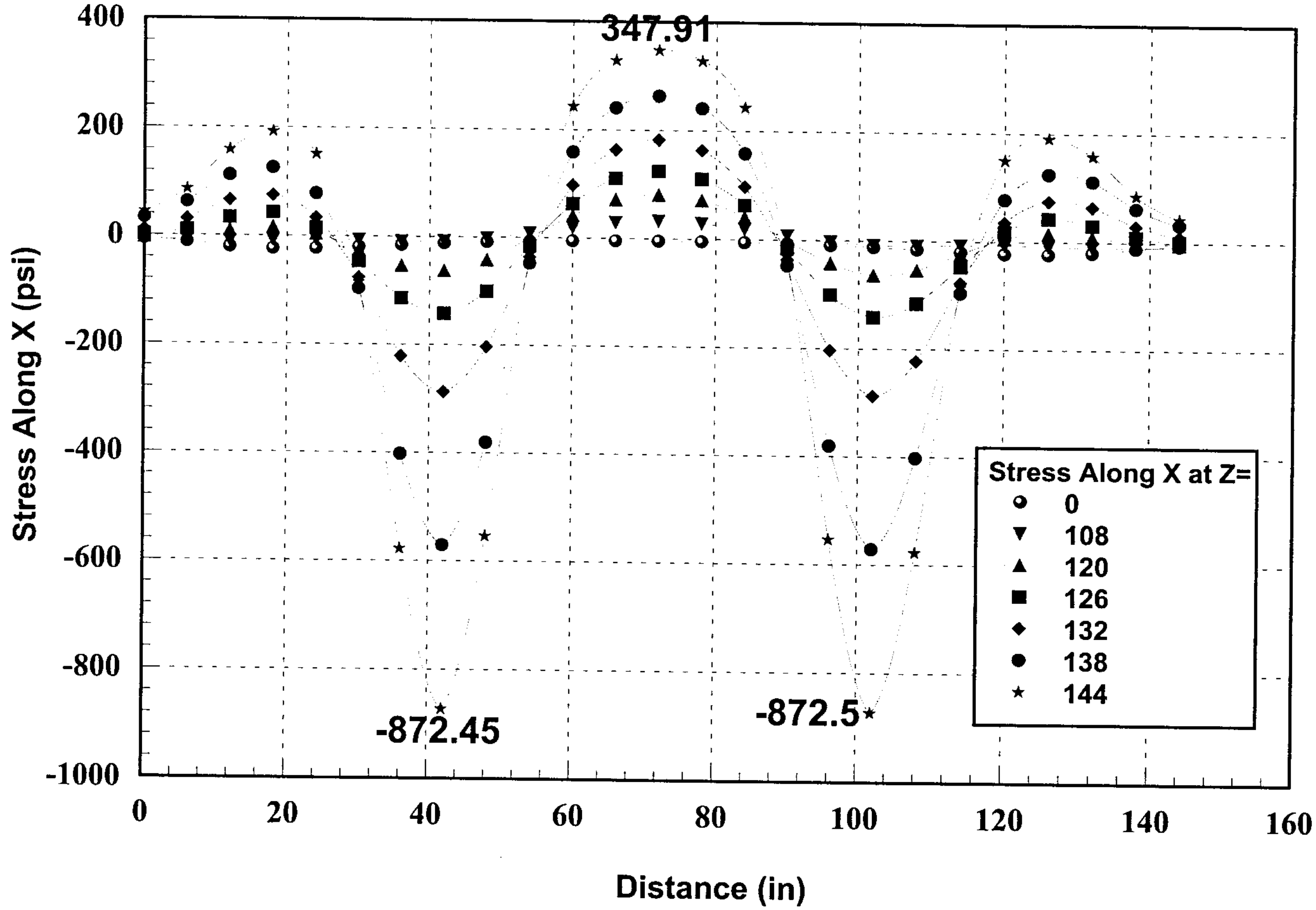
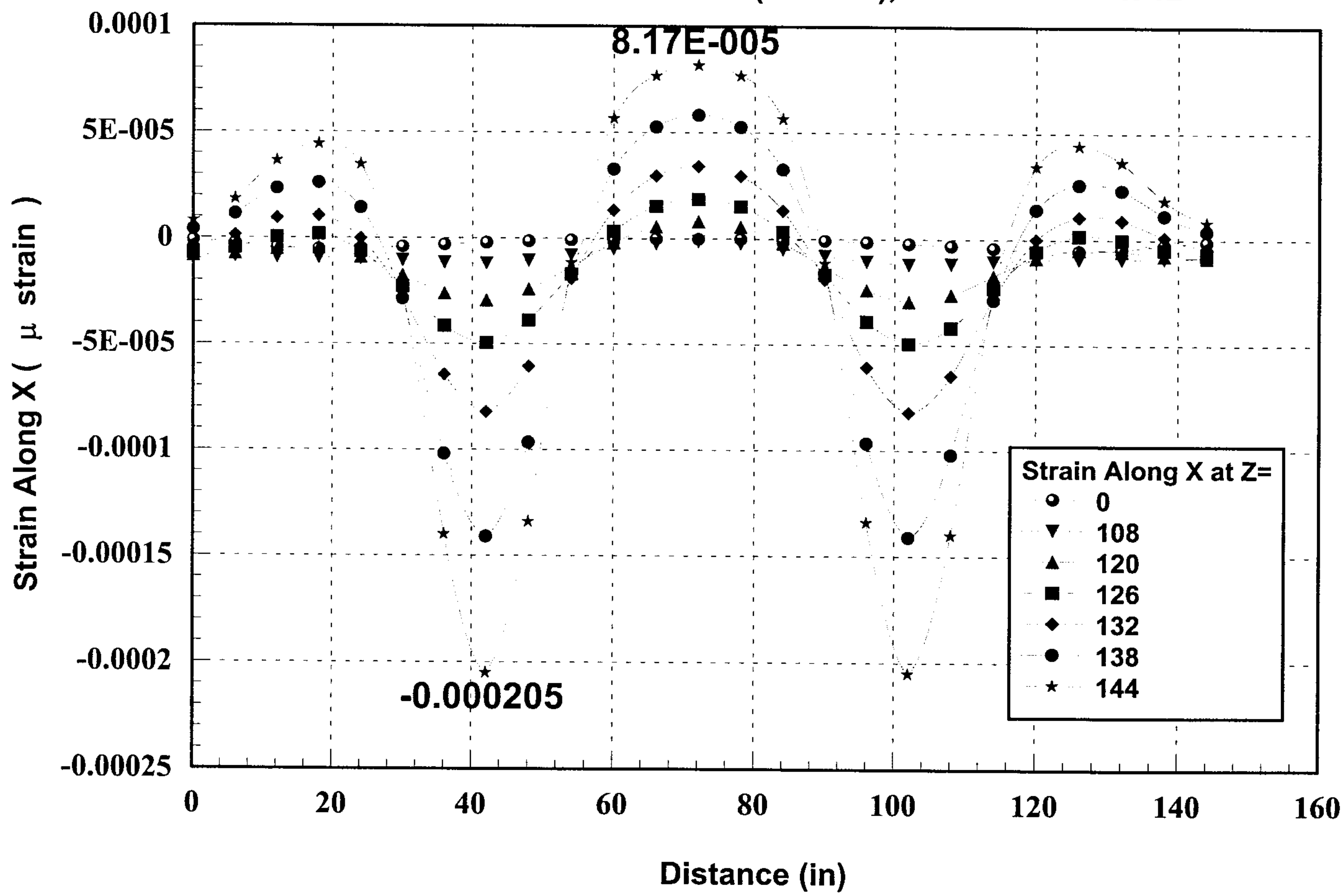


Figure 5.3.50. Strain At Different Sections Along The Width Of The Track # 3 (Friction), Panel Size 12' x 12'



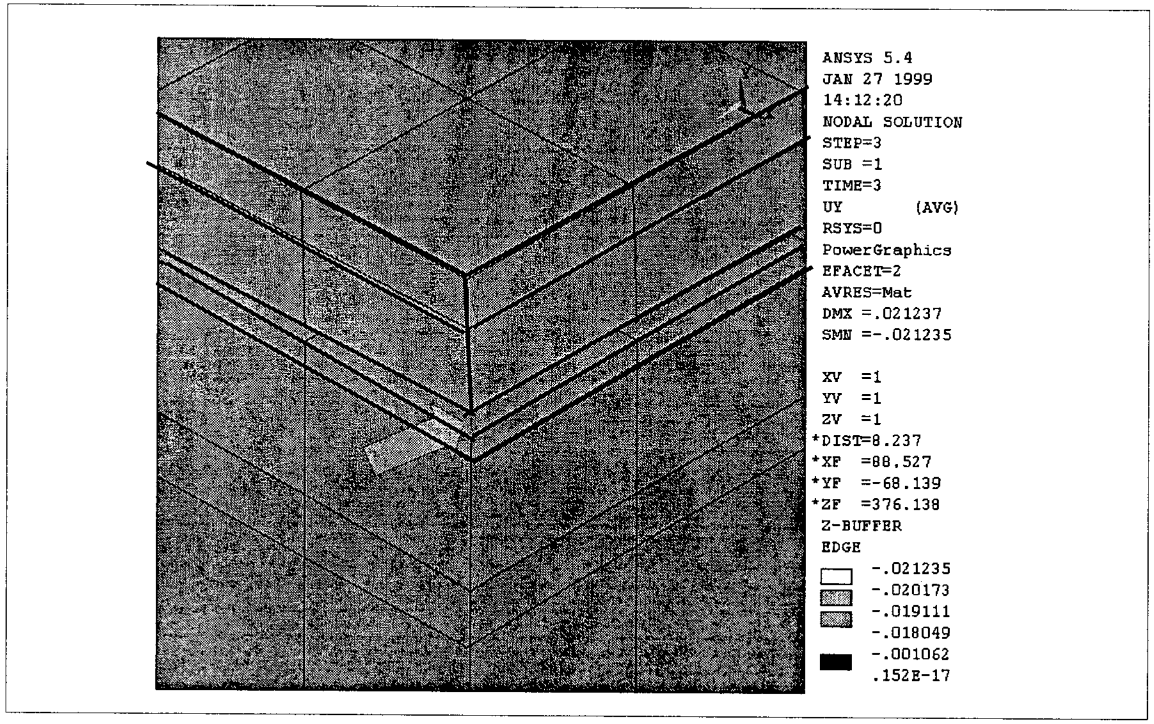



Figure 5.4.1 Prooping up of slab because of edge loading, Track # 1, 6' x 6' (Weak Bonding)

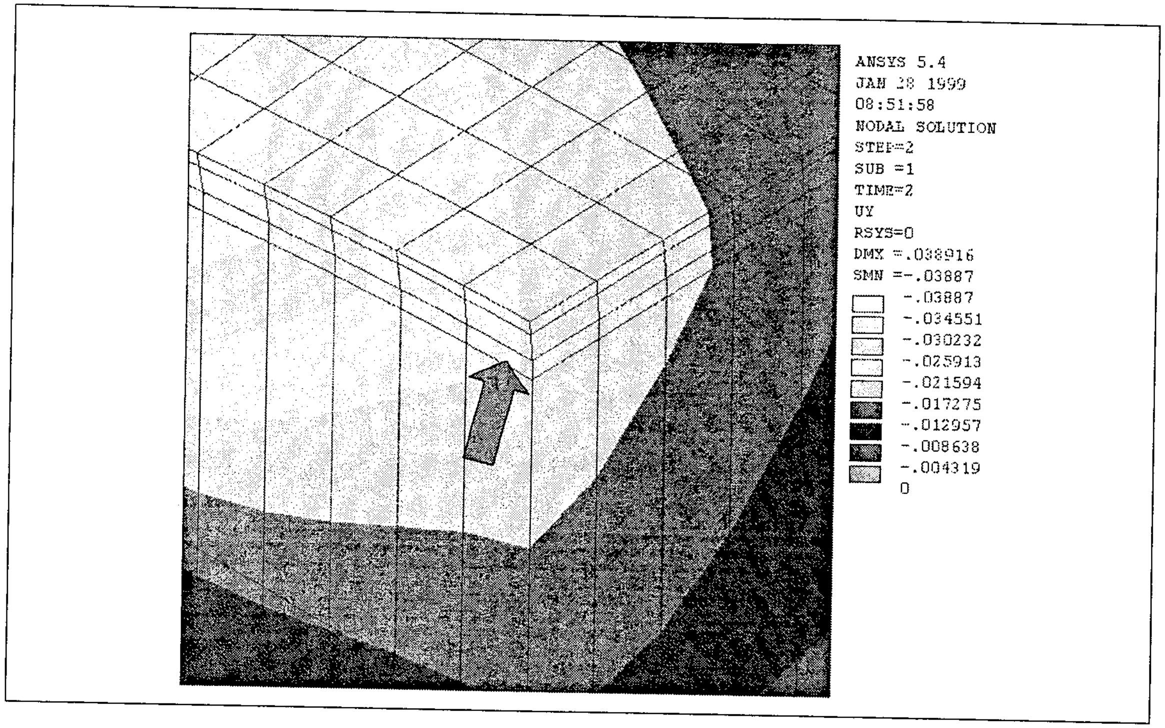



Figure 5.4.2 No Propping Up of Slab Because of Edge Loading Track 3, Slab Size - 12' X 12' (Strong Bonding)

Figure 5.4.3 Track # 1, Panel Size 4' x 4'

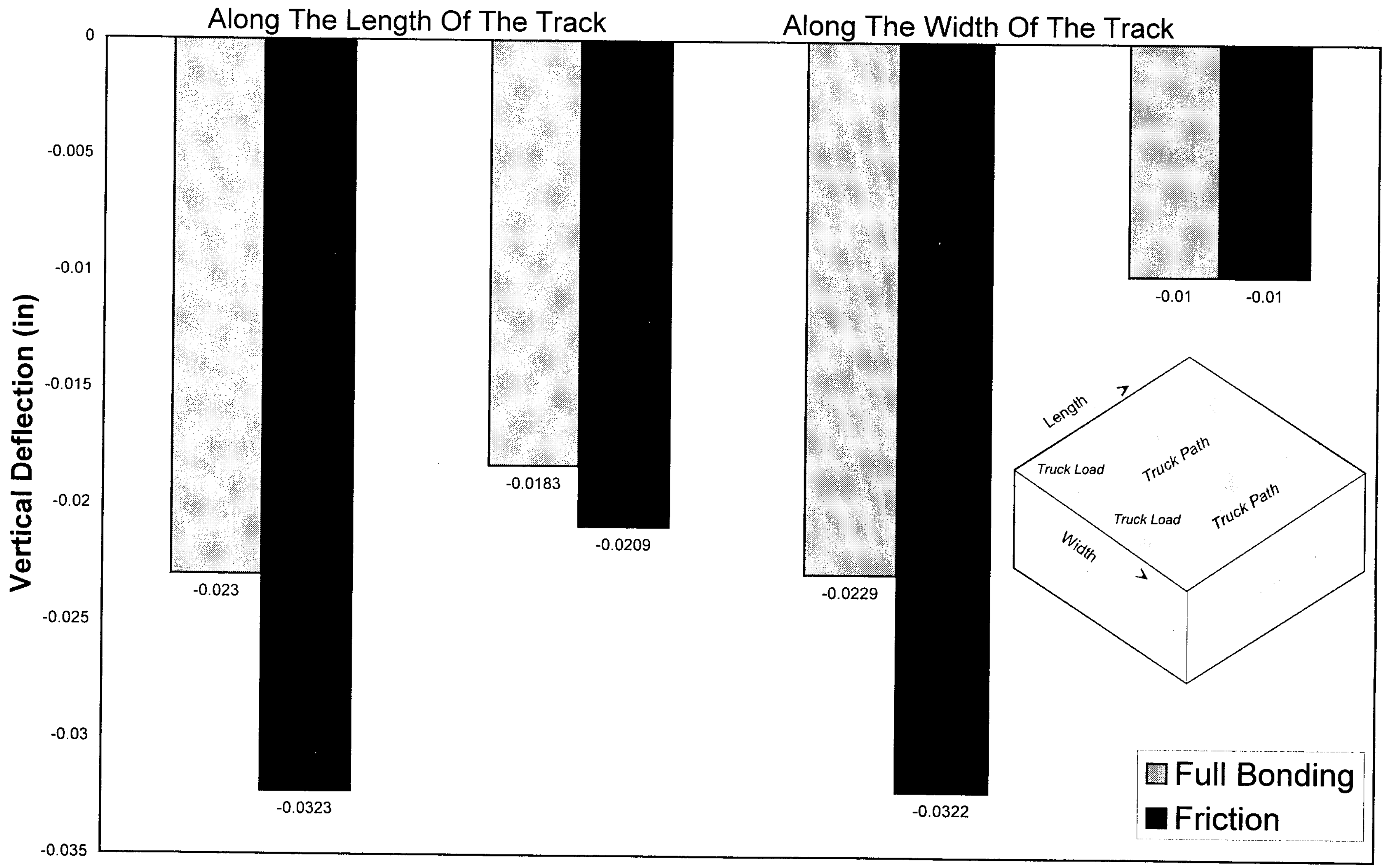


Figure 5.4.4 Track # 1, Panel Size 4' x 4'

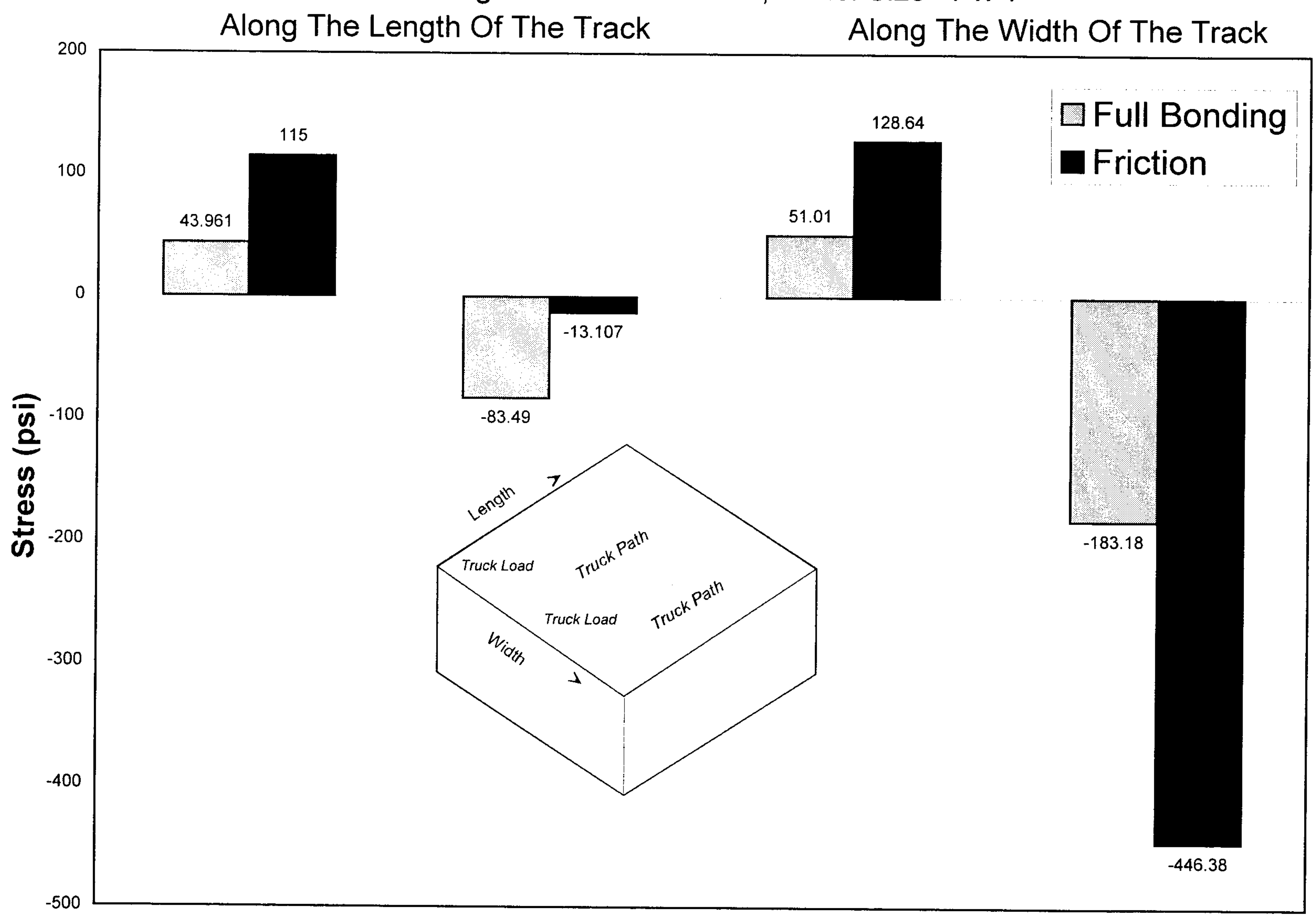


Figure 5.4.5 Track # 1, Panel Size 4' x 4'

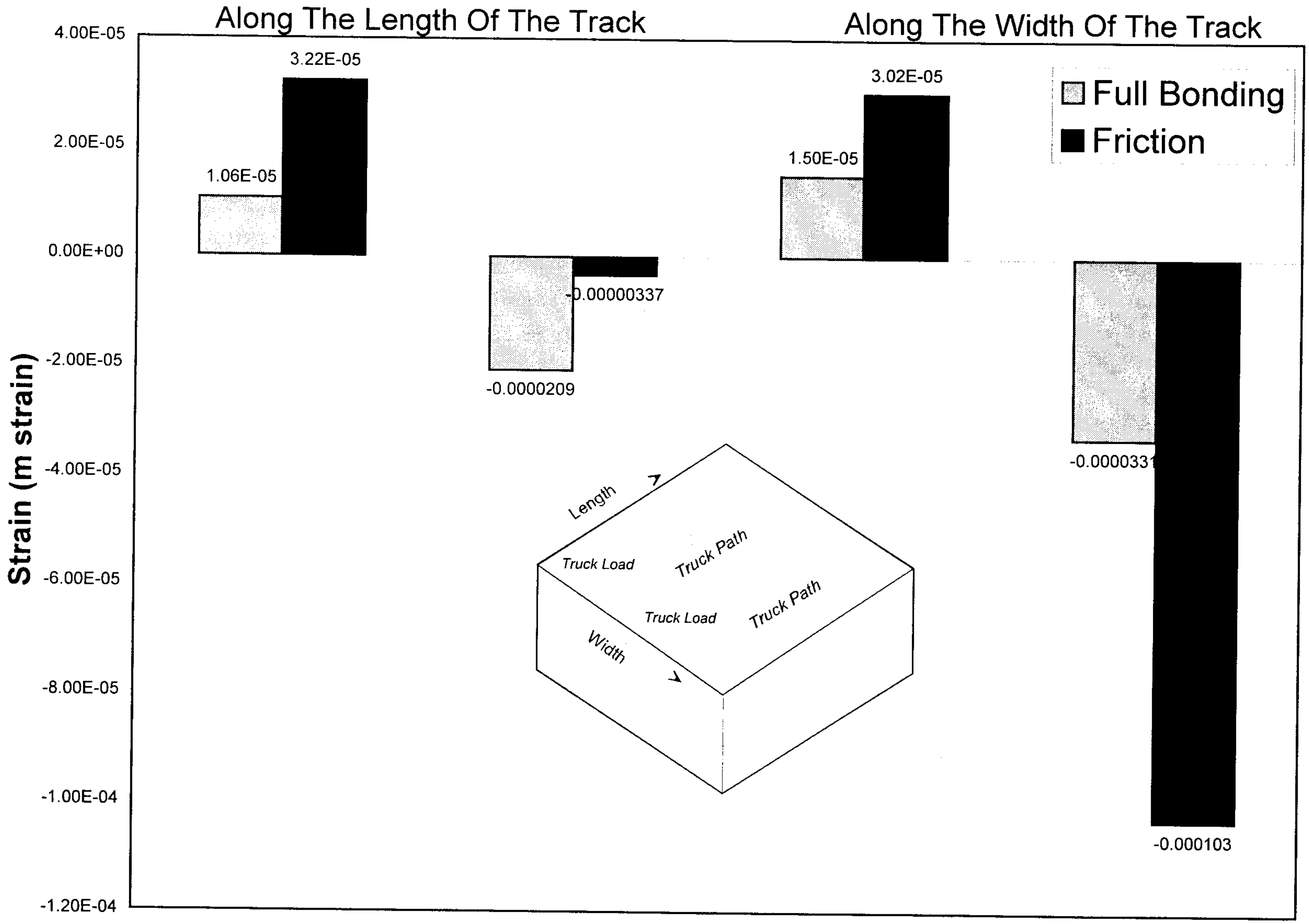


Figure 5.4.6 Track # 1, Panel Size 6' x 6'

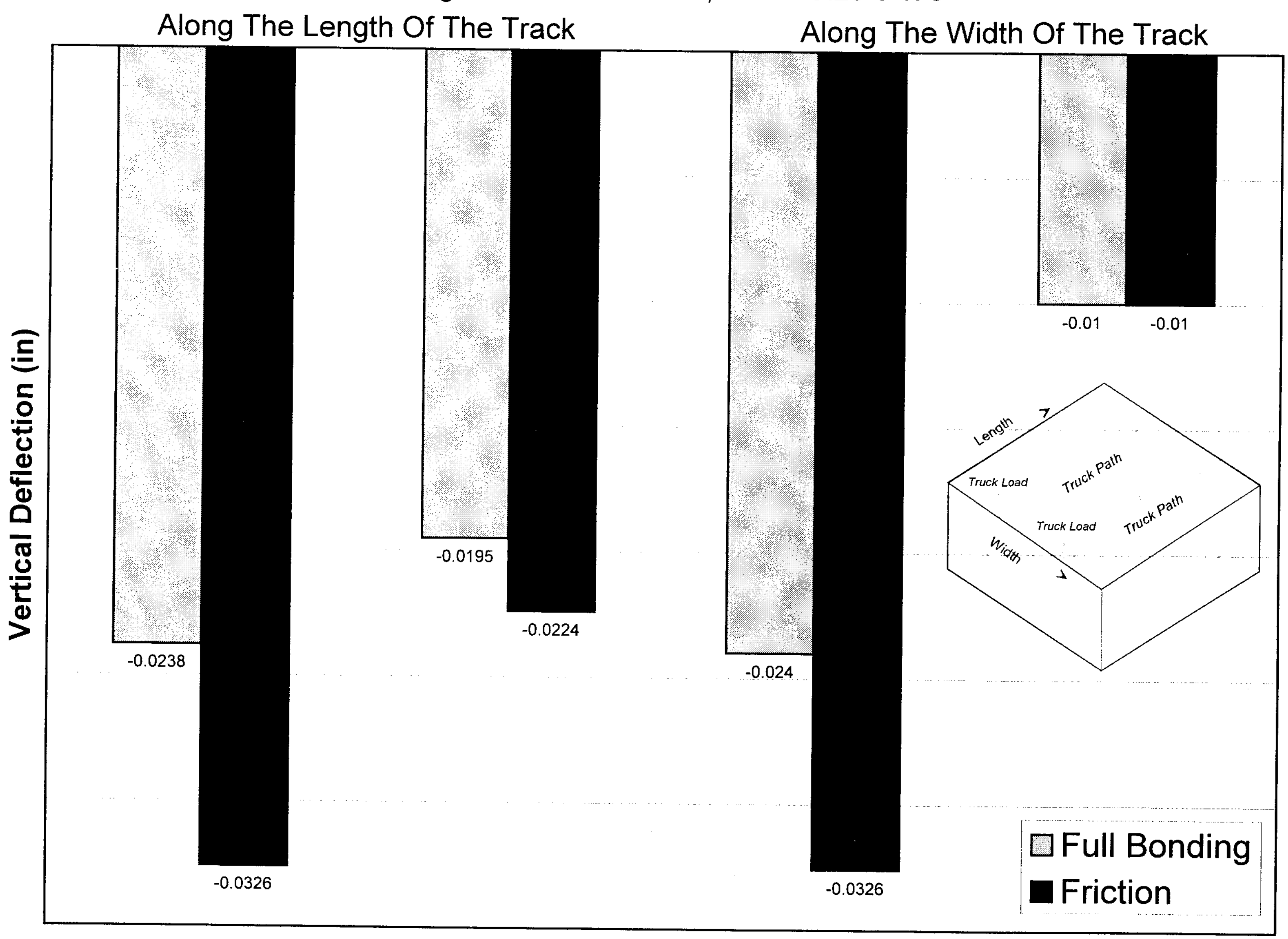


Figure 5.4.7 Track # 1, Panel Size 6' x 6'

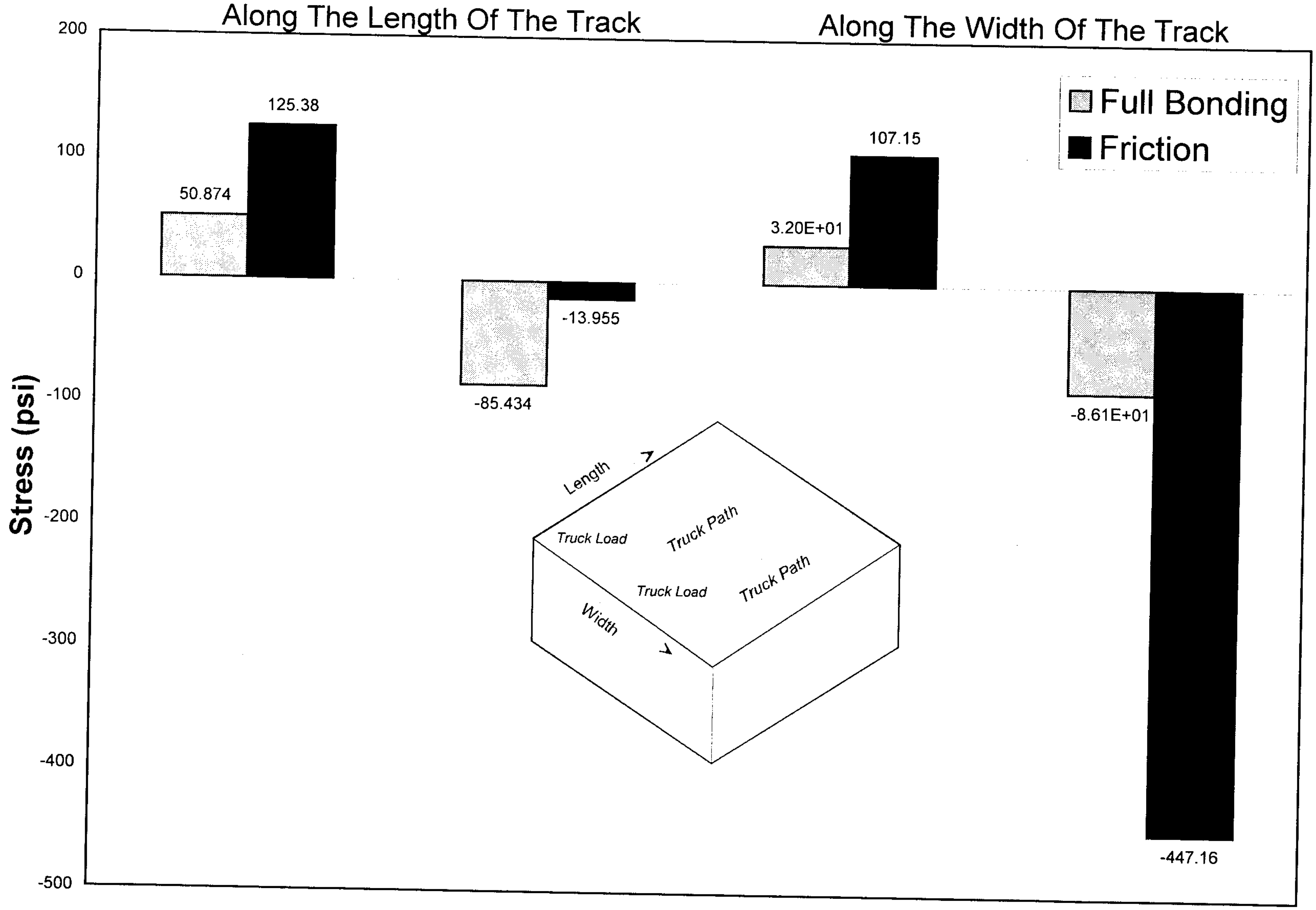


Figure 5.4.8 Track # 1, Panel Size 6' x 6'

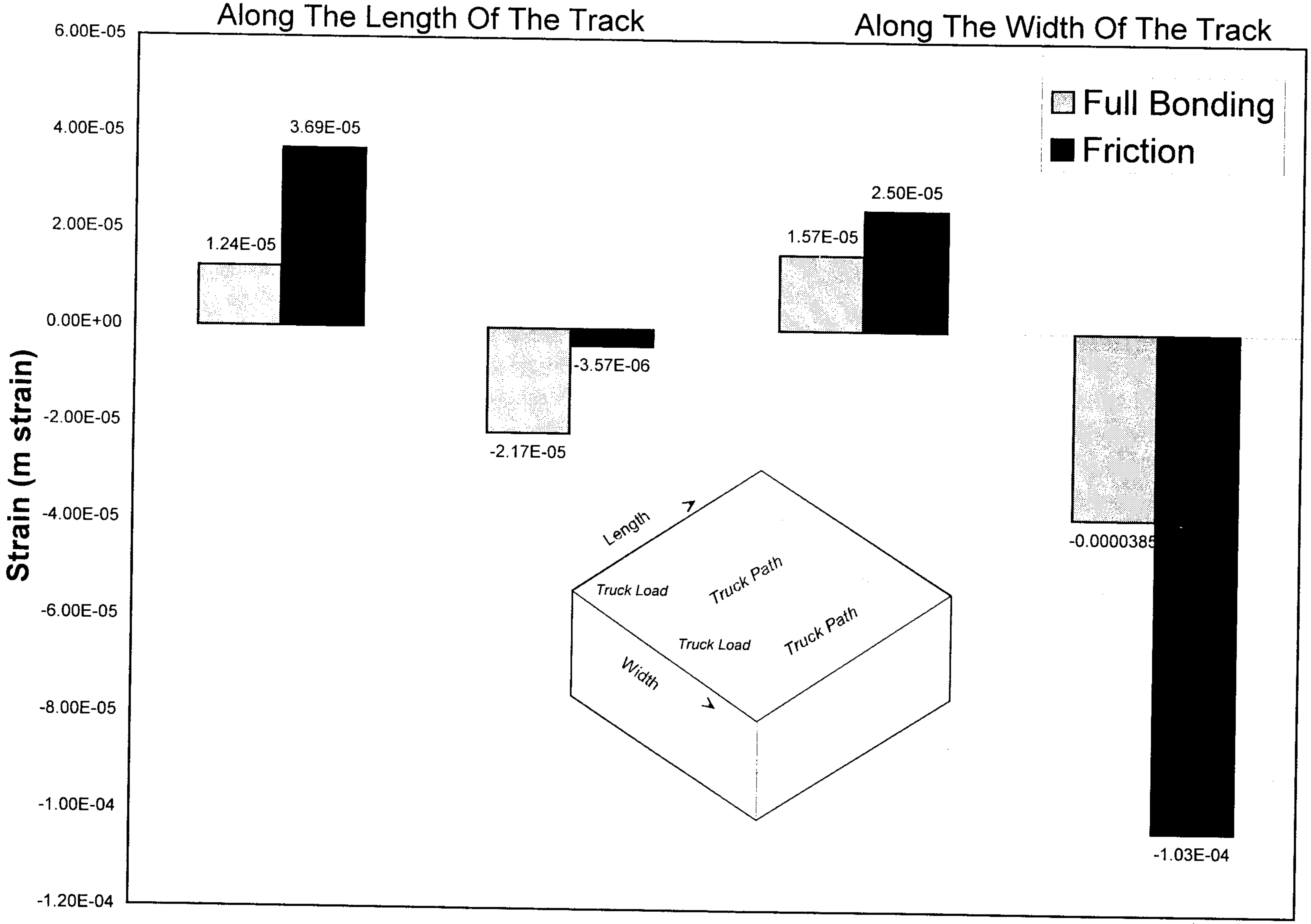


Figure 5.4.9 Track # 2, Panel Size 3' x 3'

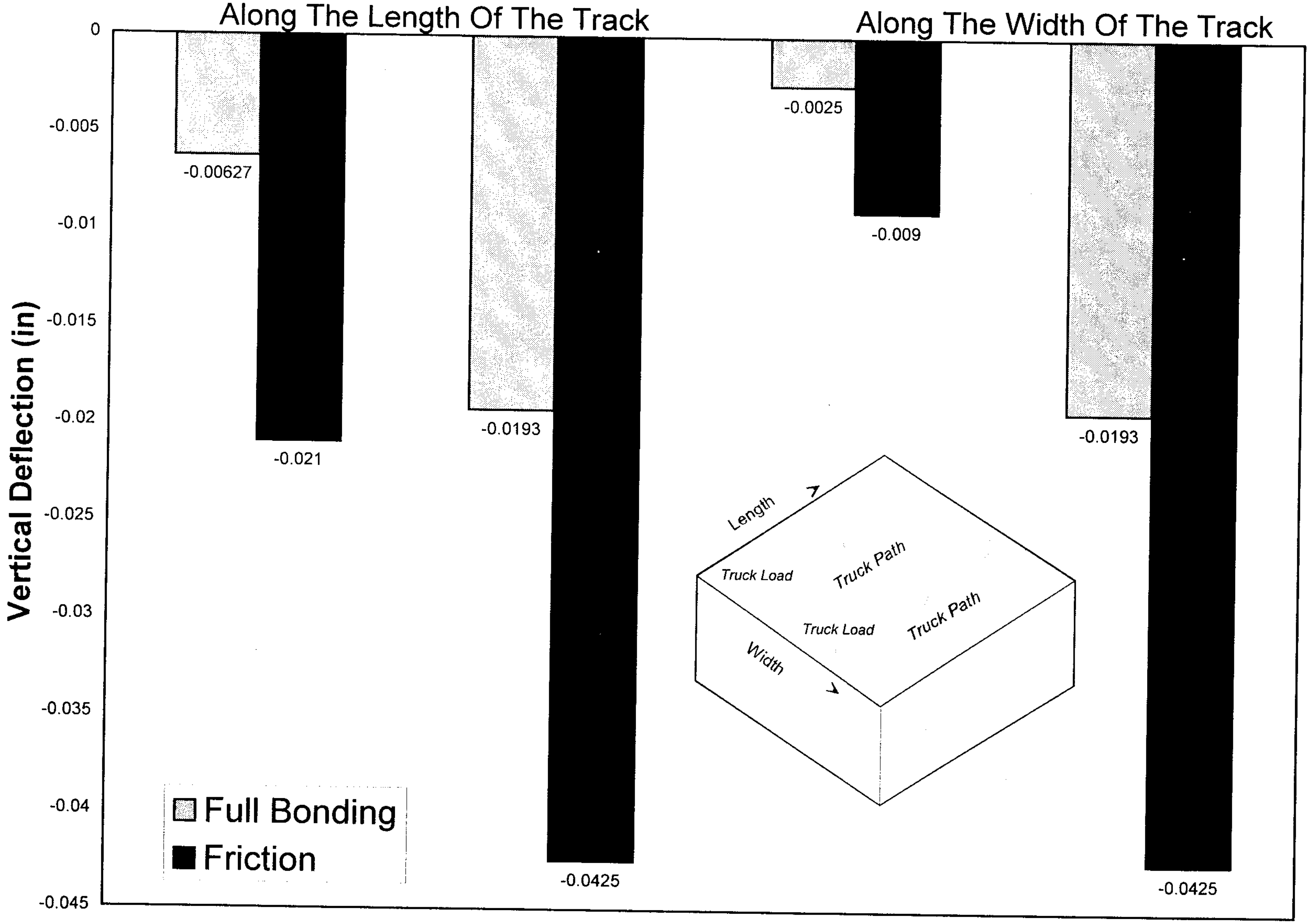


Figure 5.4.10 Track # 2, Panel Size 3' x 3'

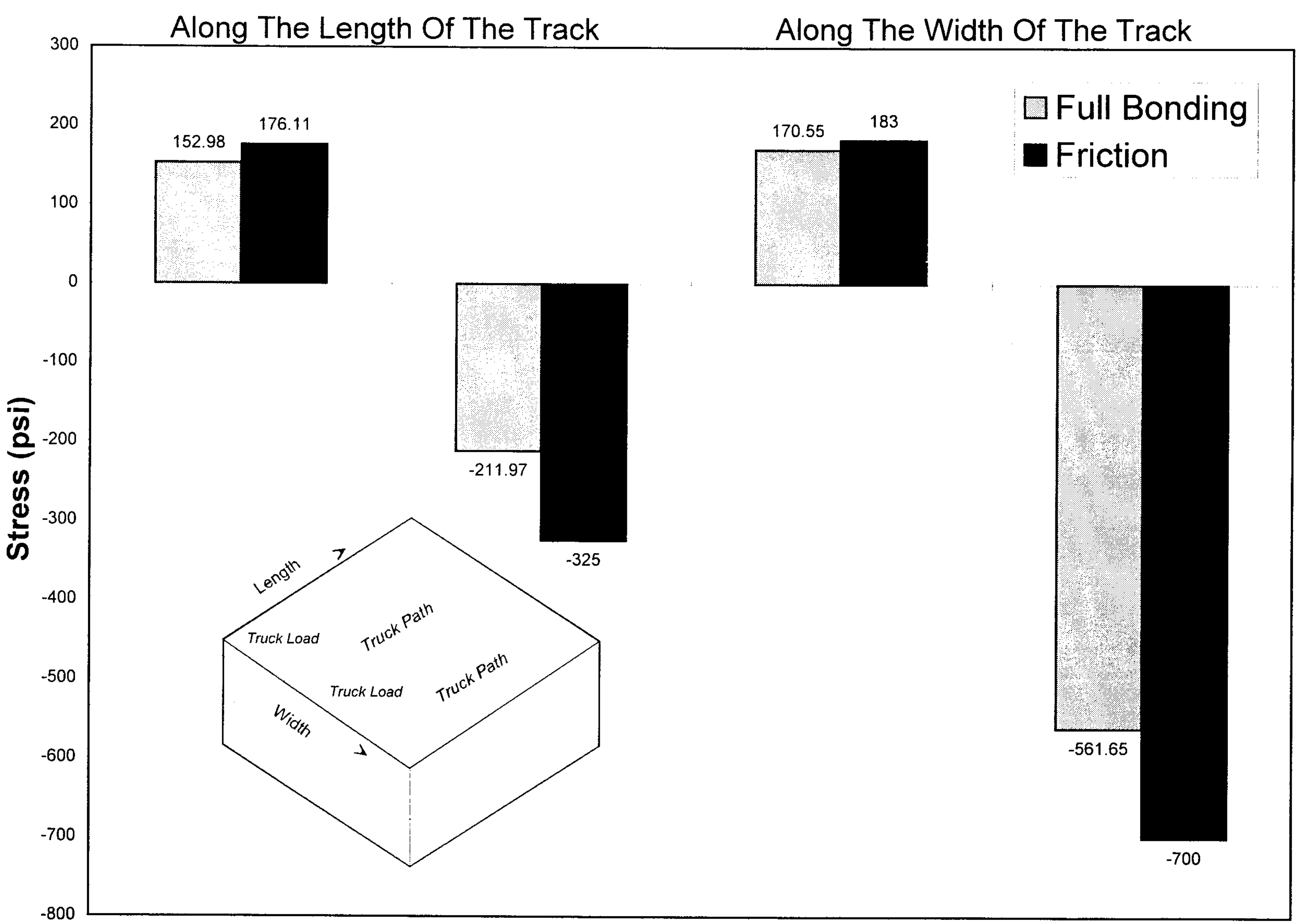


Figure 5.4.11 rack # 2, Panel Size 3' x 3'

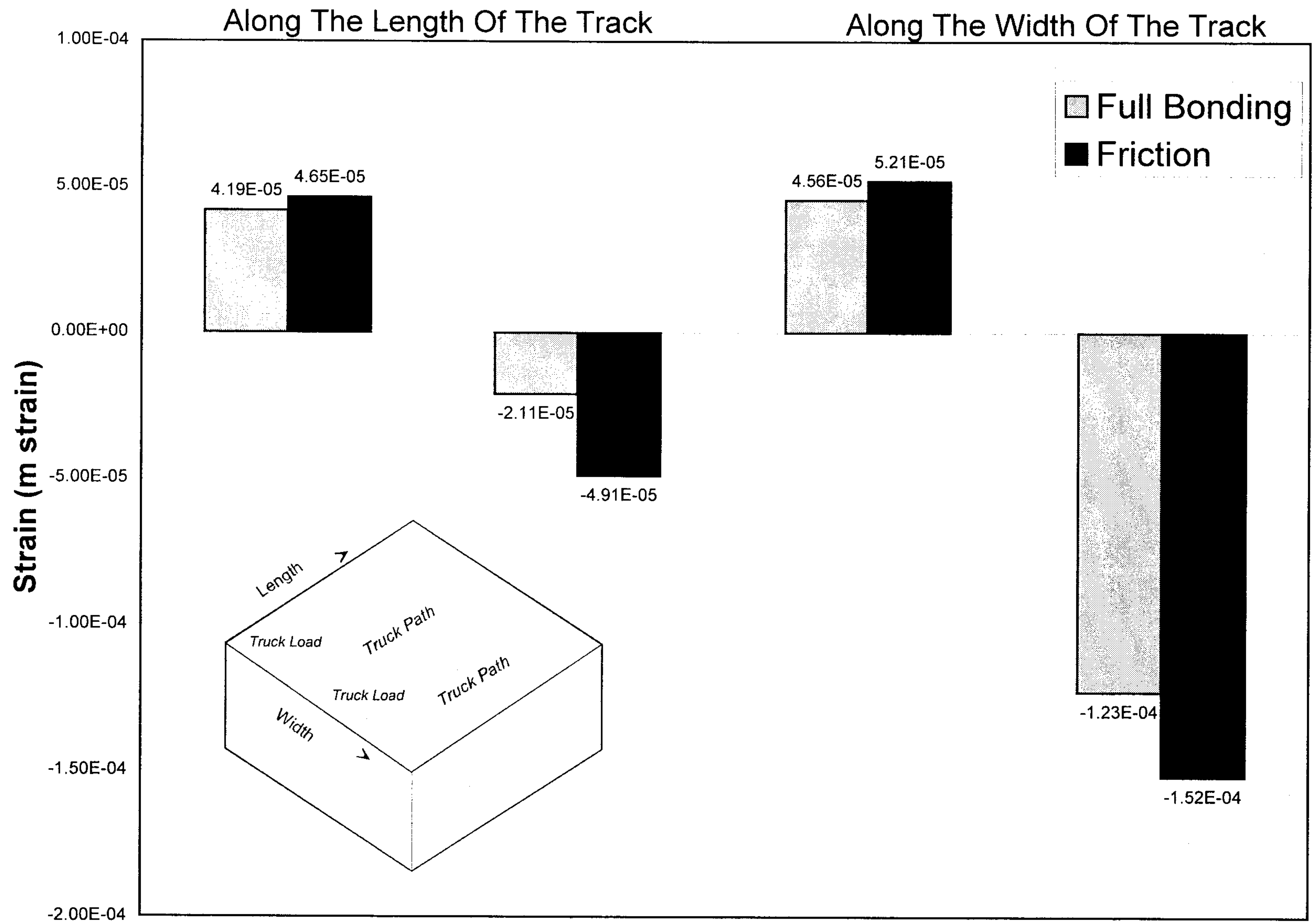


Figure 5.4.12 Track # 2, Panel Size 4' x 4'

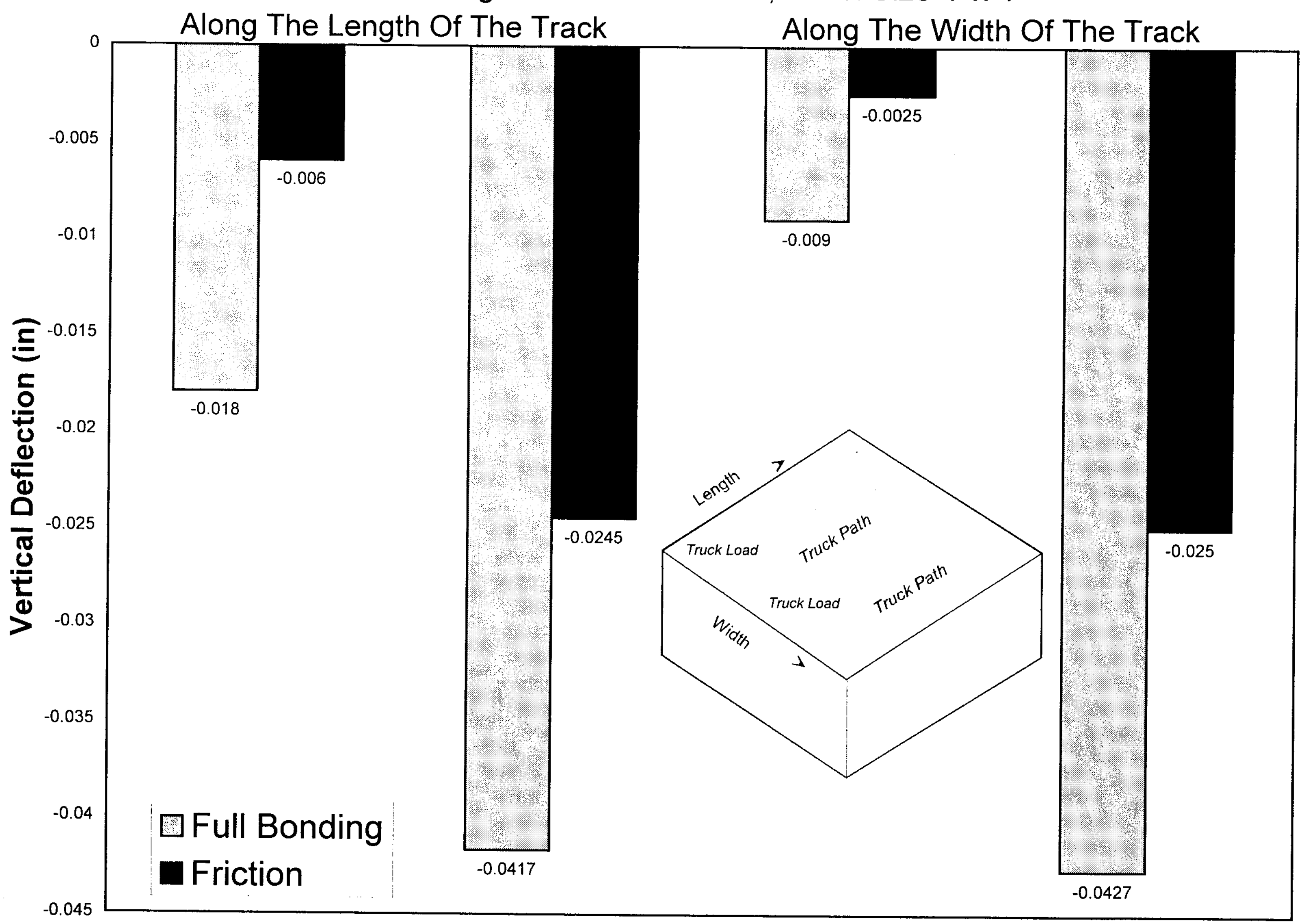


Figure 5.4.13 Track # 2, Panel Size 4' x 4'

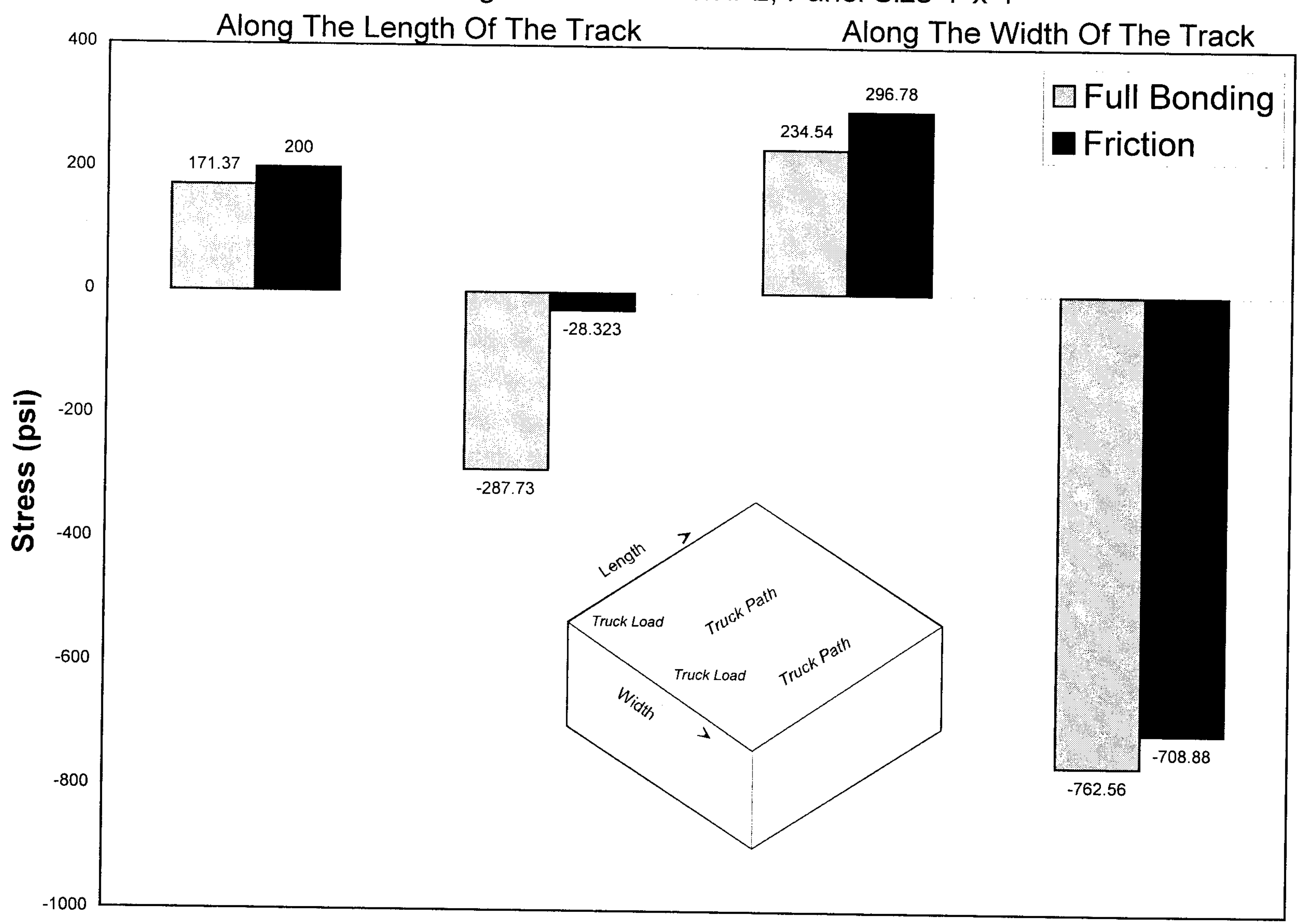


Figure 5.4.14 Track # 2, Panel Size 4' x 4'

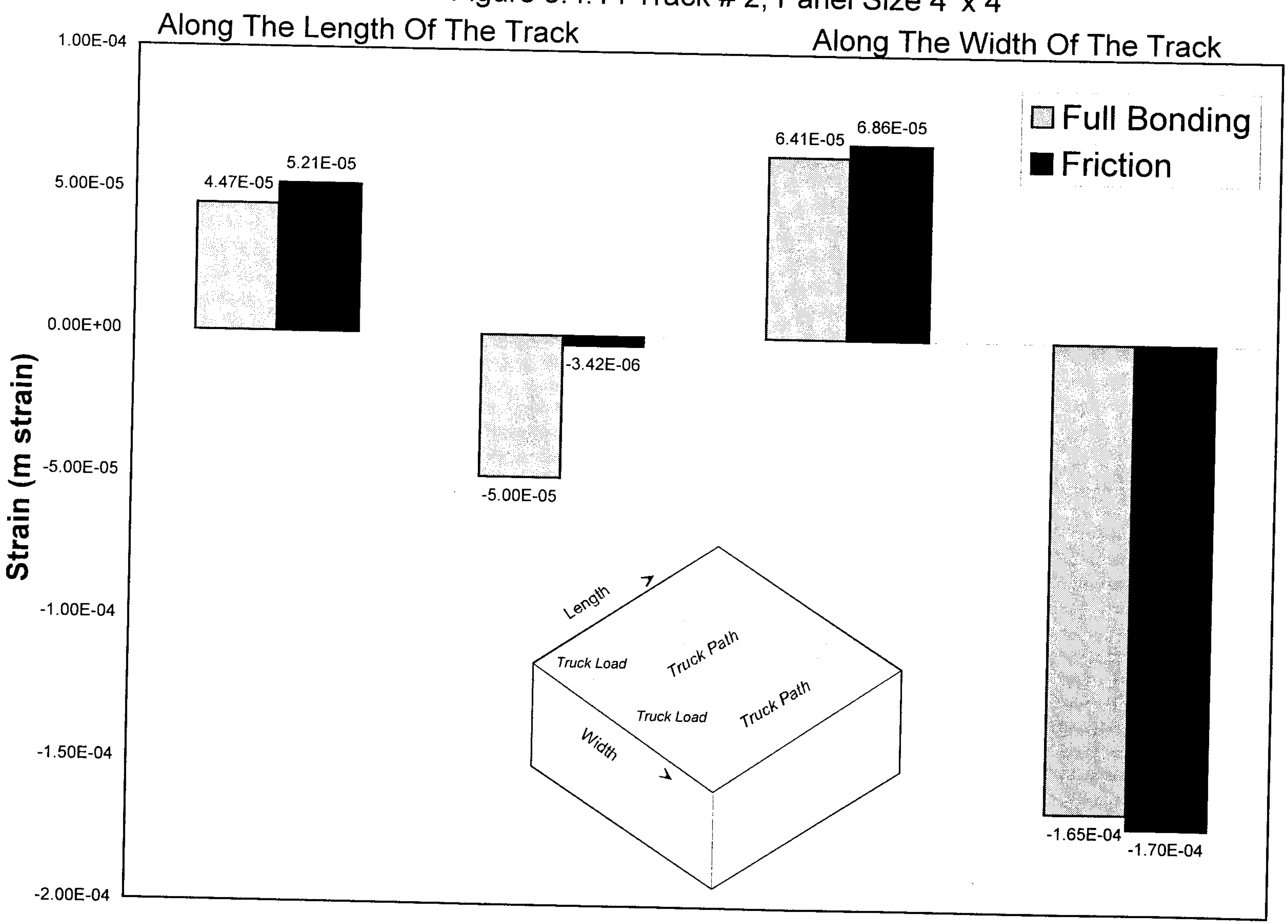


Figure 5.4.15 Track # 2, Panel Size 12' x 12'

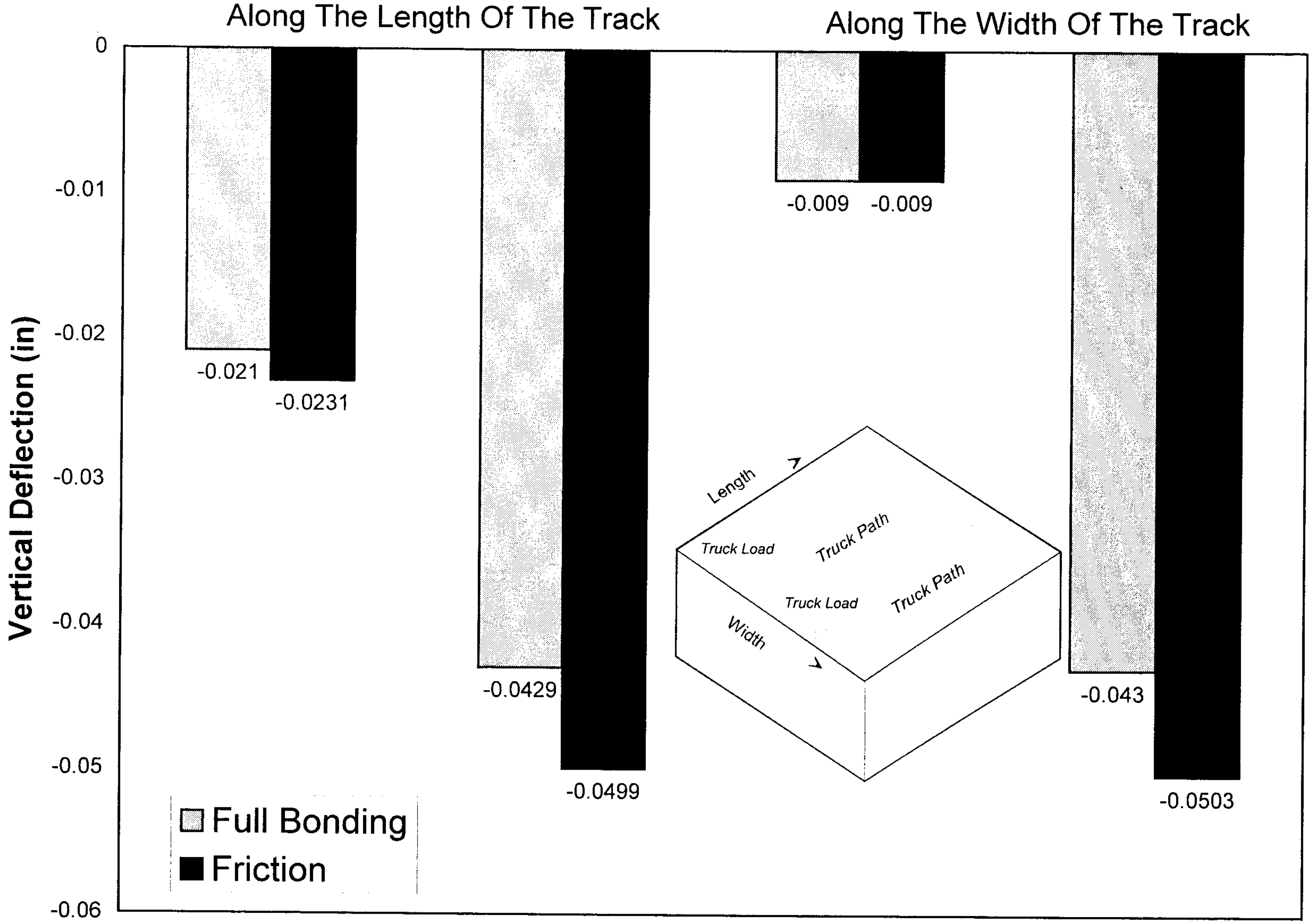


Figure 5.4.16 Track # 2, Panel Size 12' x 12'

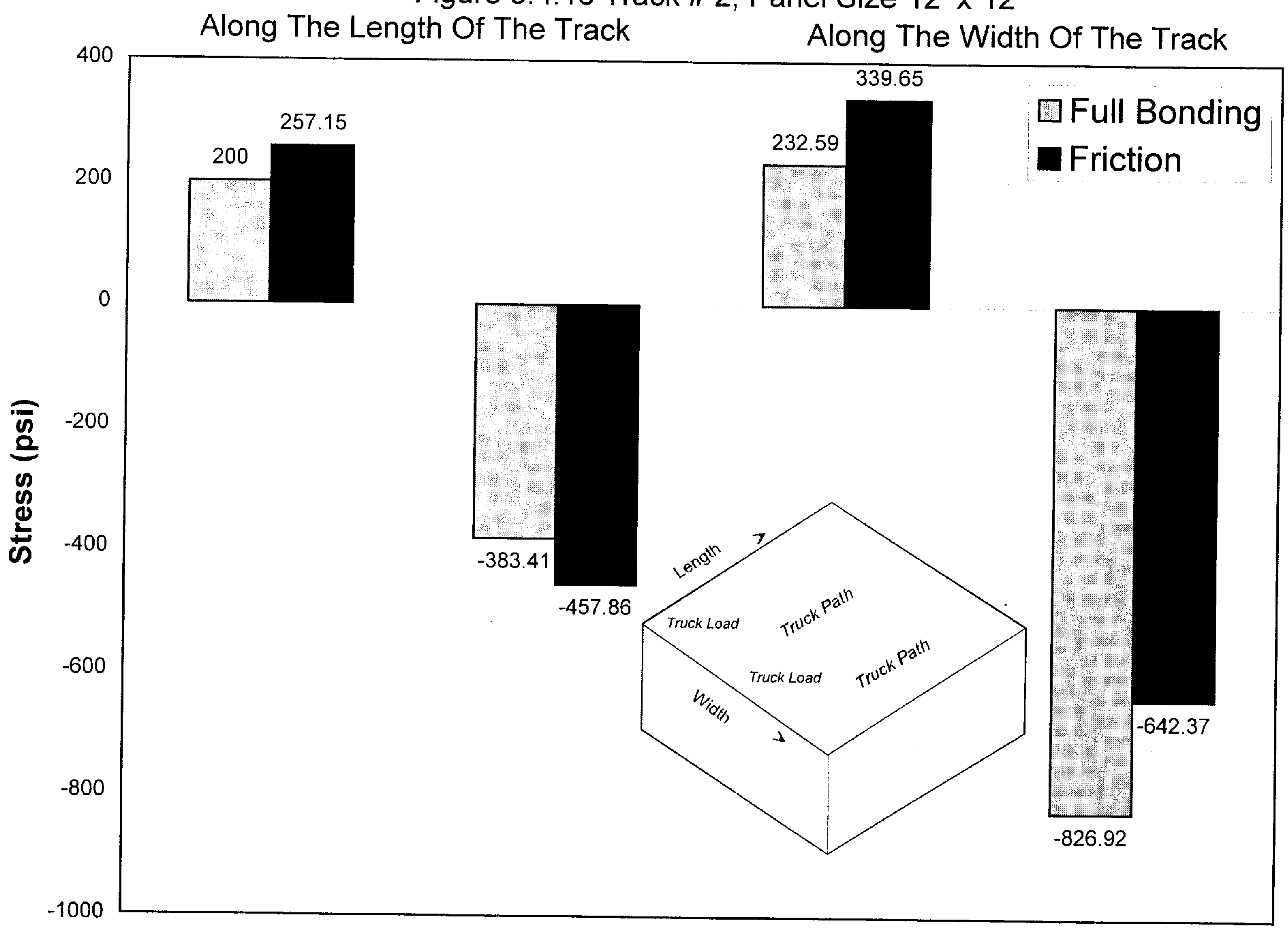


Figure 5.4.17 Track # 2, Panel Size 12' x 12'

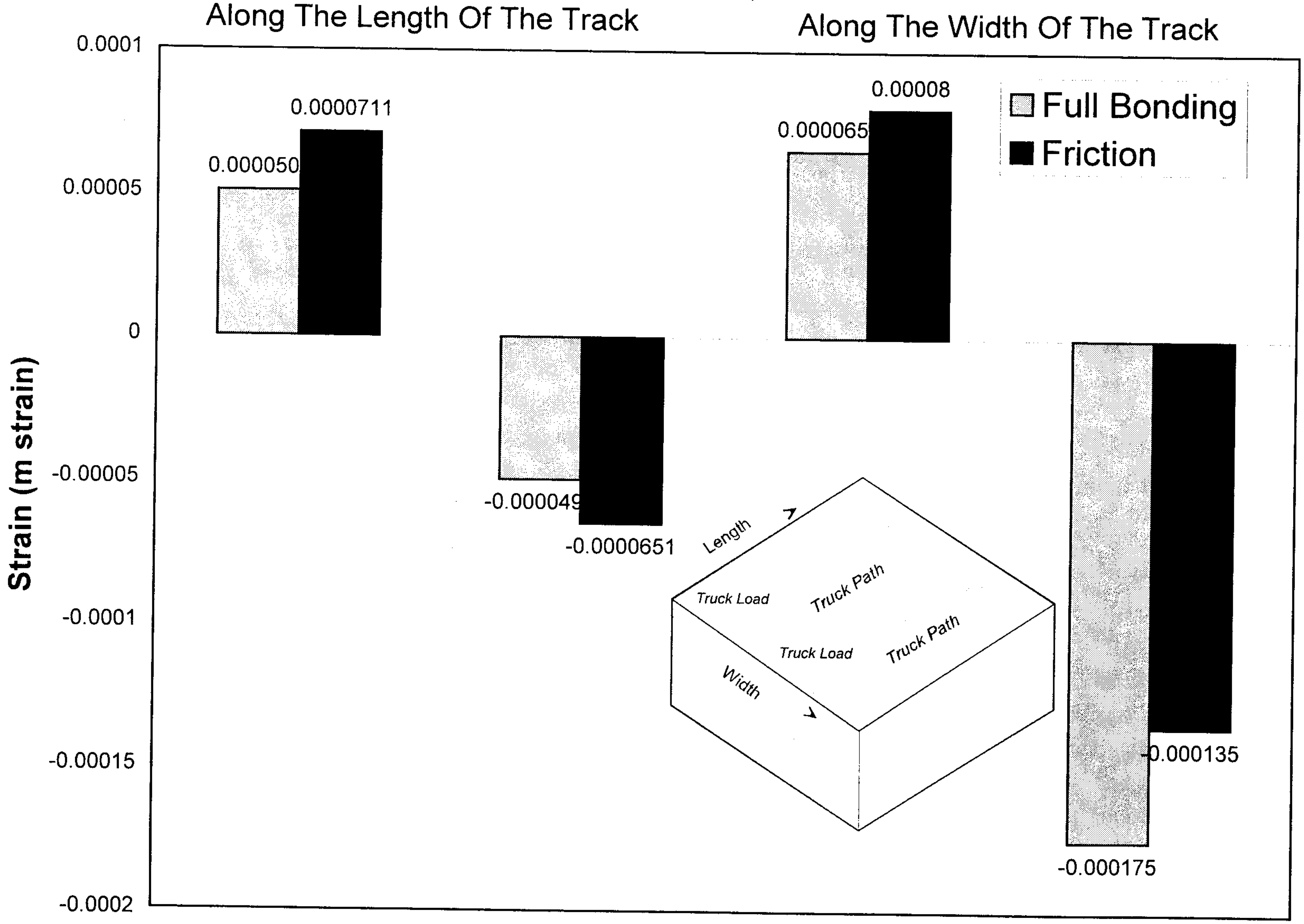


Figure 5.4.18 Track # 3, Panel Size 4' x 4'

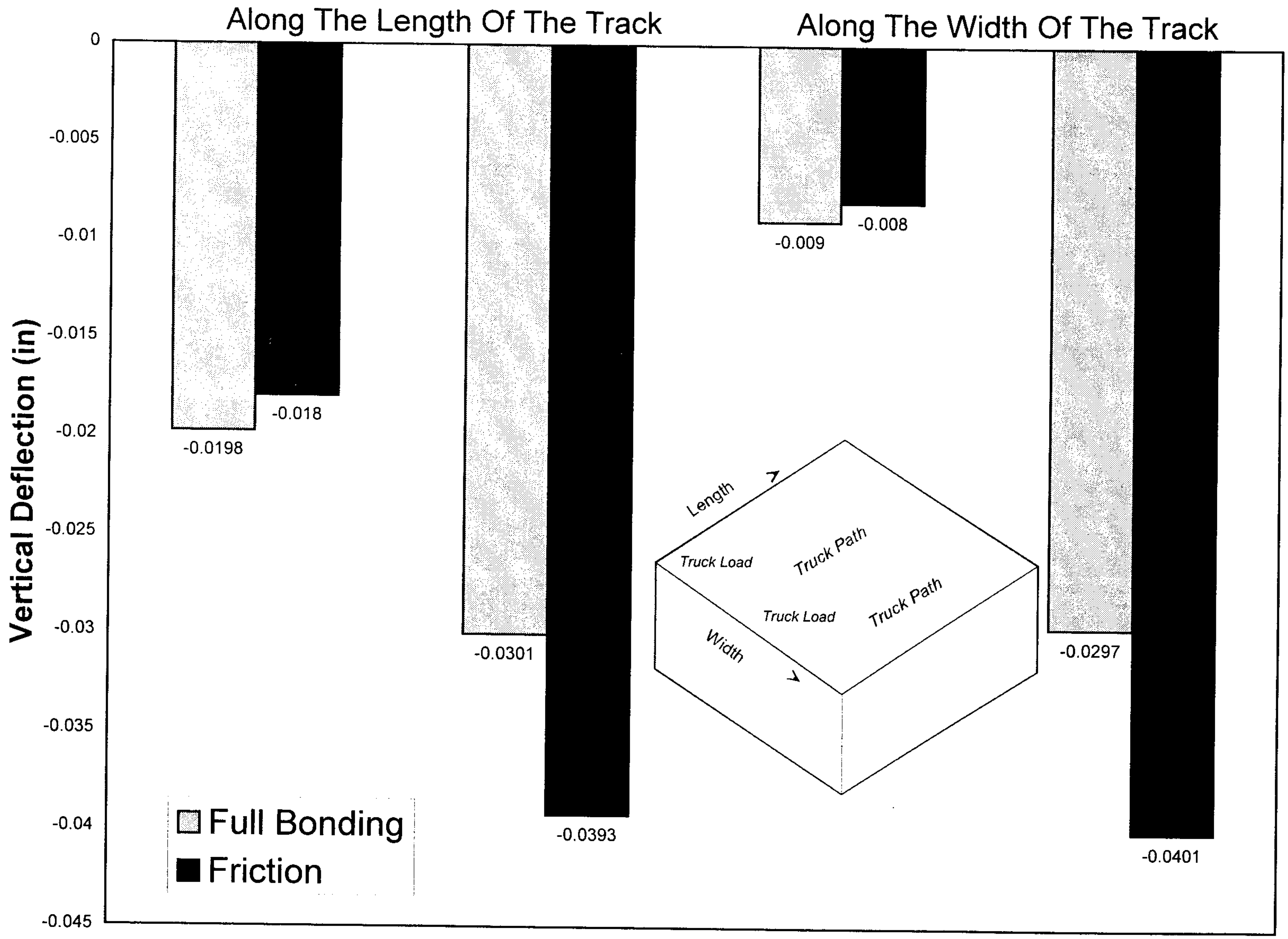


Figure 5.4.19 Track # 3, Panel Size 4' x 4'

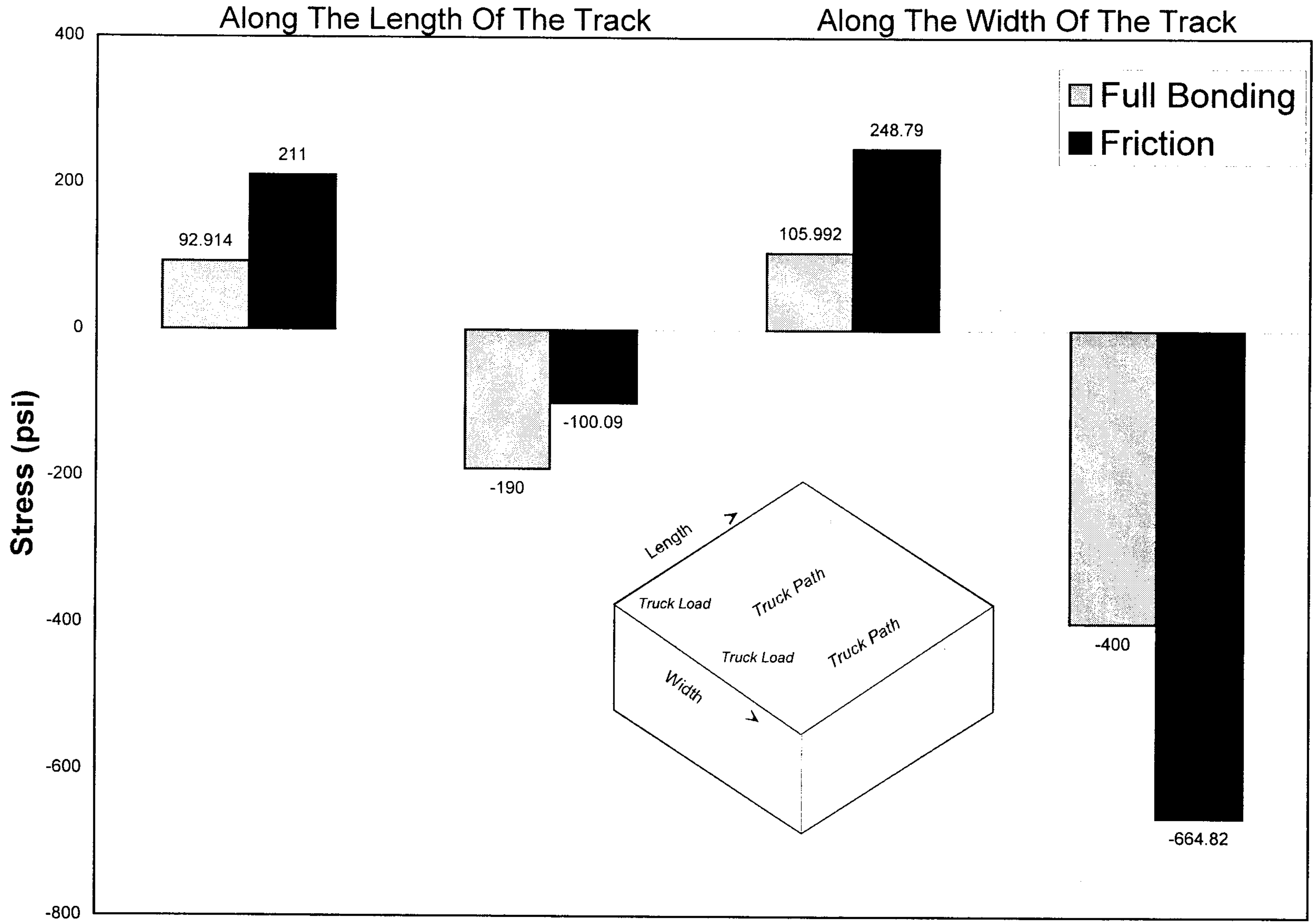


Figure 5.4.20 Track # 3, Panel Size 4' x 4'

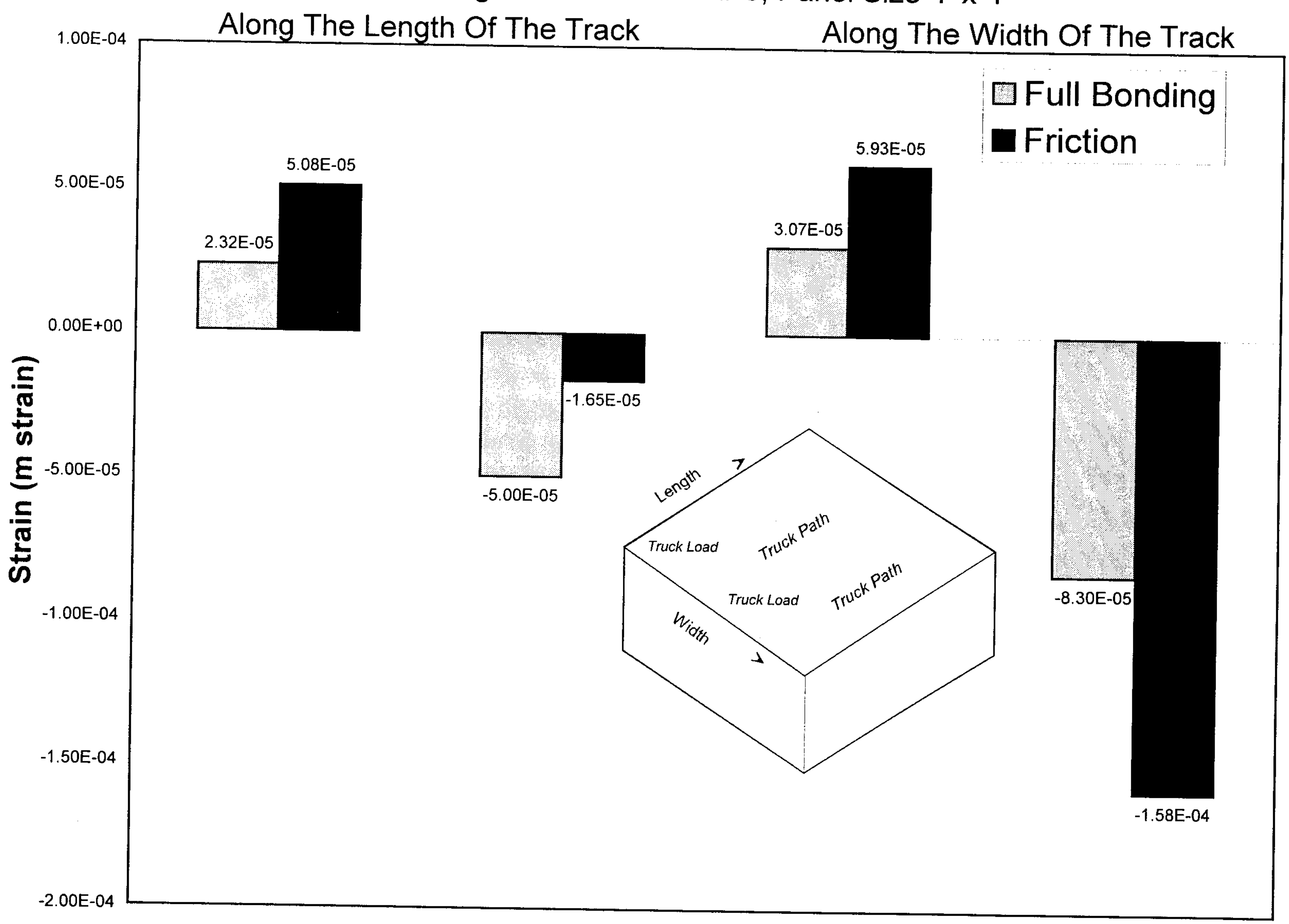


Figure 5.4.21 Track # 3, Panel Size 6' x 6'

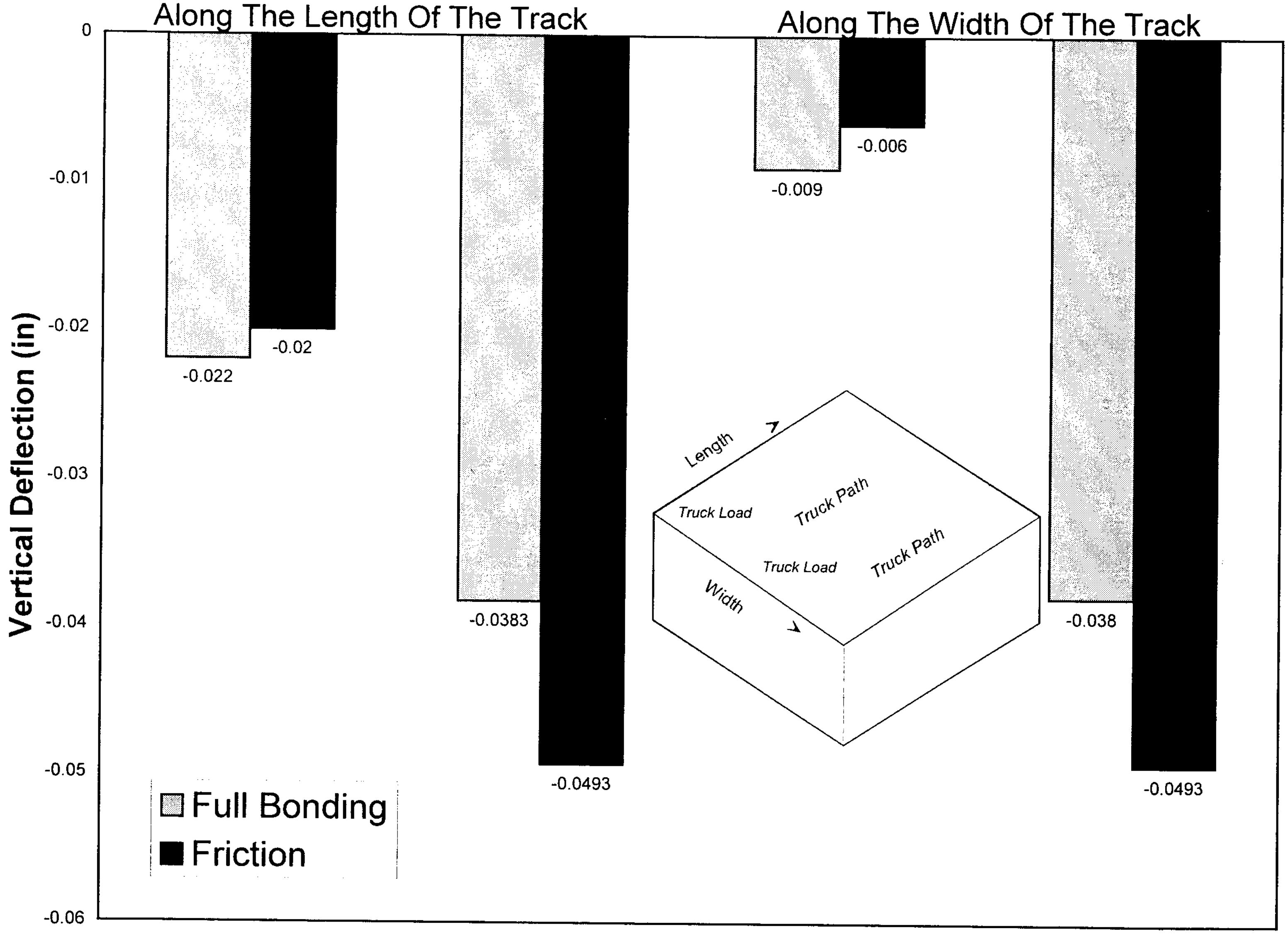


Figure 5.4.22 Track # 3, Panel Size 6' x 6'

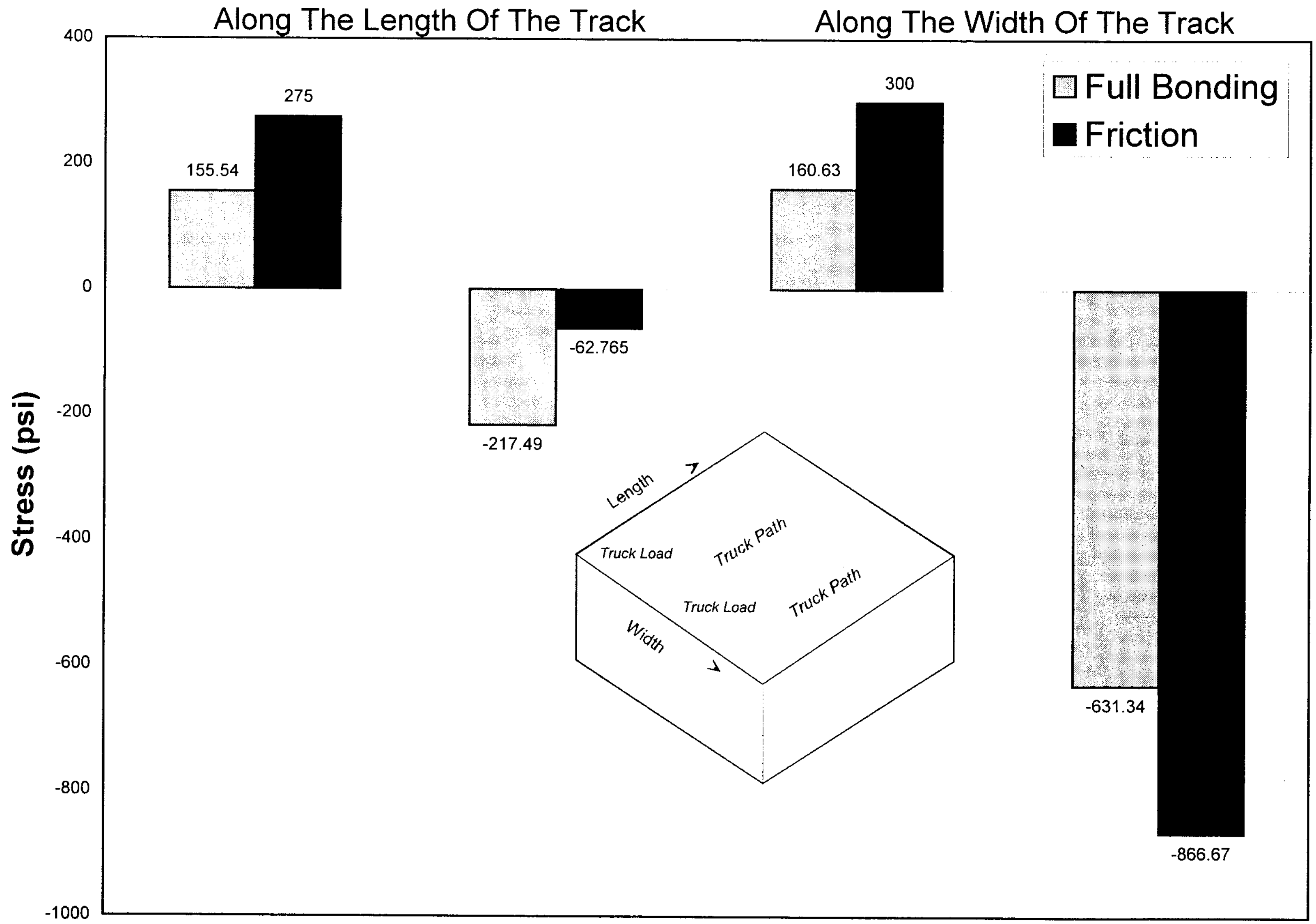


Figure 5.4.23 Track # 3, Panel Size 6' x 6'

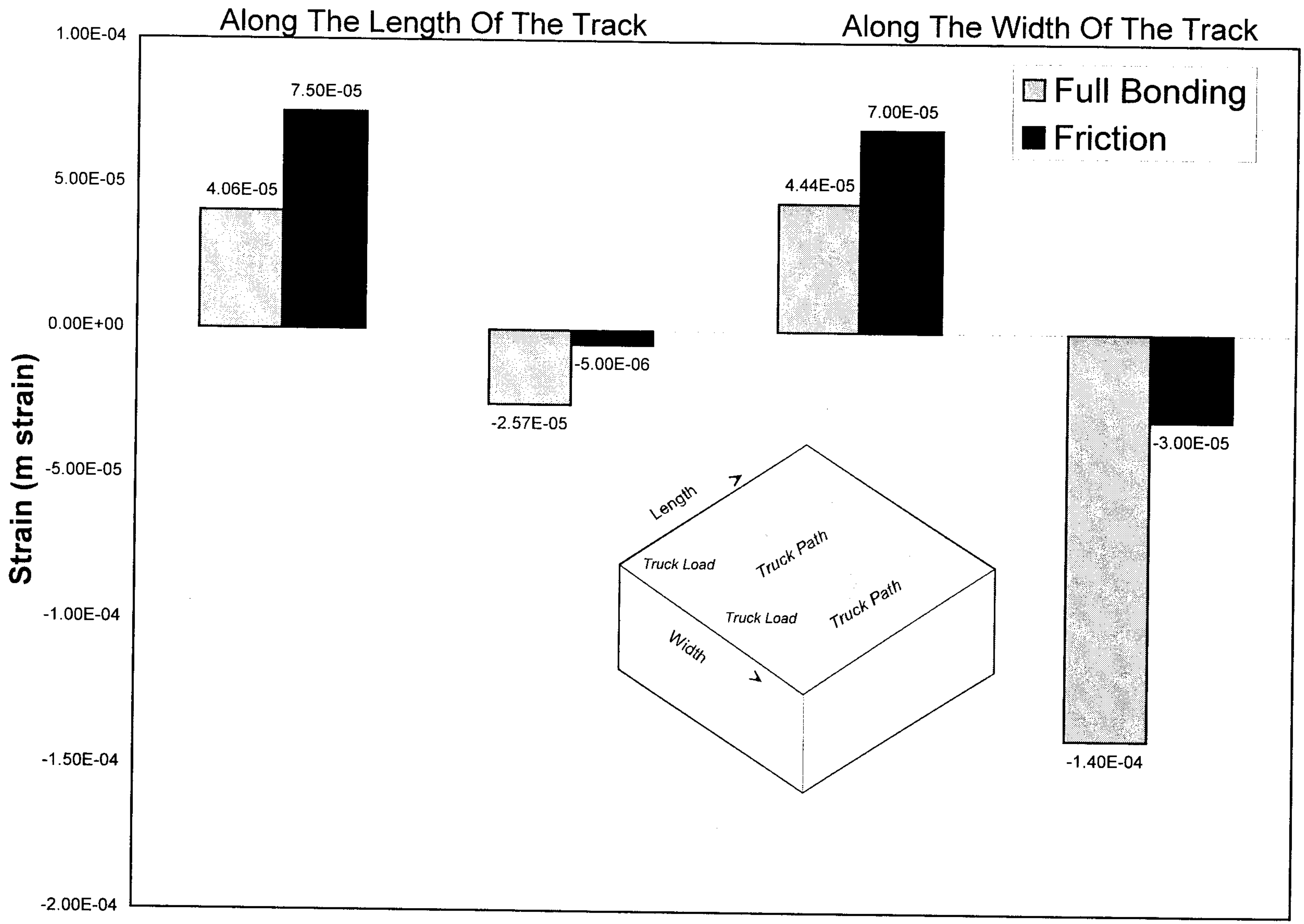


Figure 5.4.24 Track # 3, Panel Size 12' x 12'

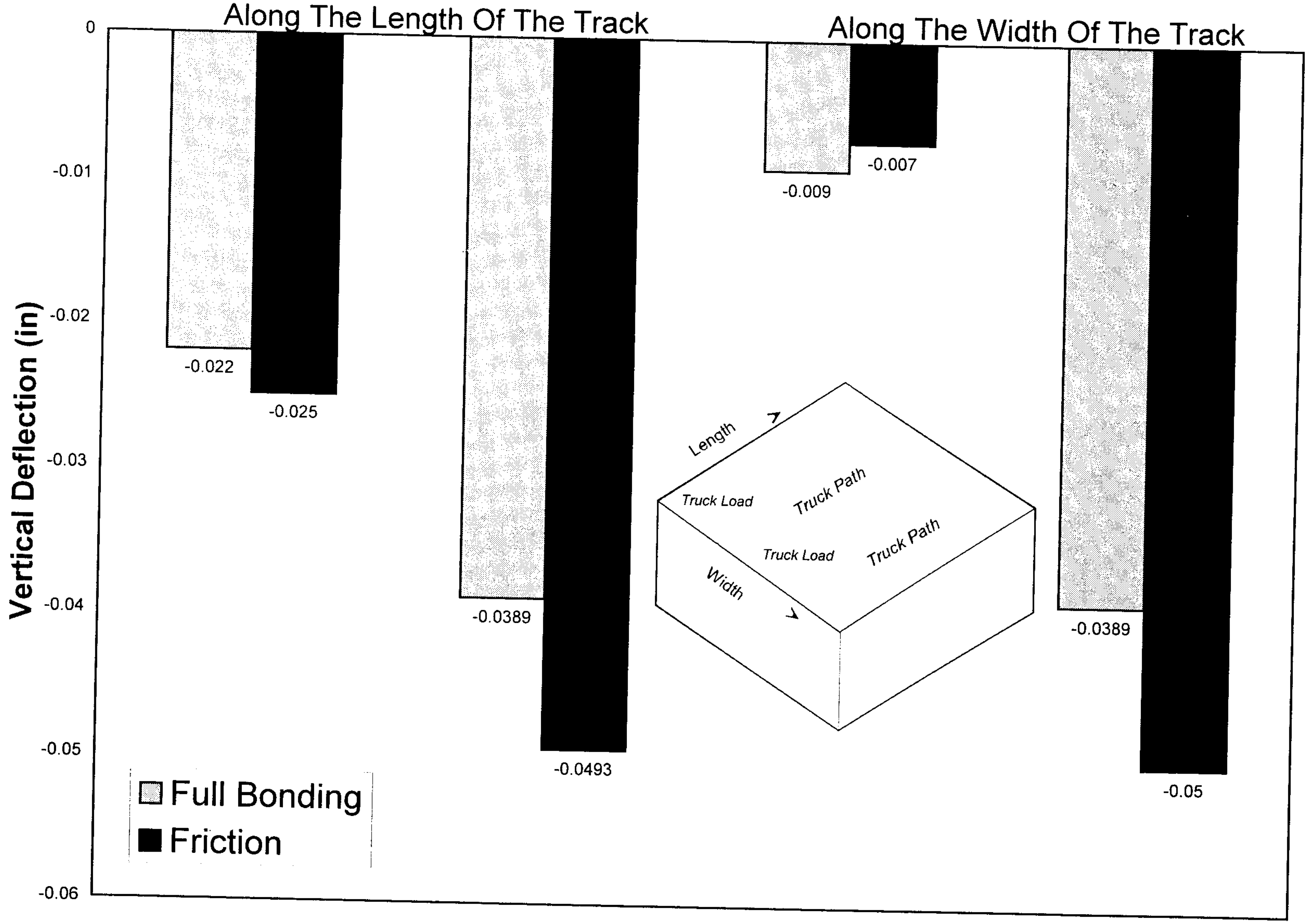


Figure 5.4.25 Track # 3, Panel Size 12' x 12'

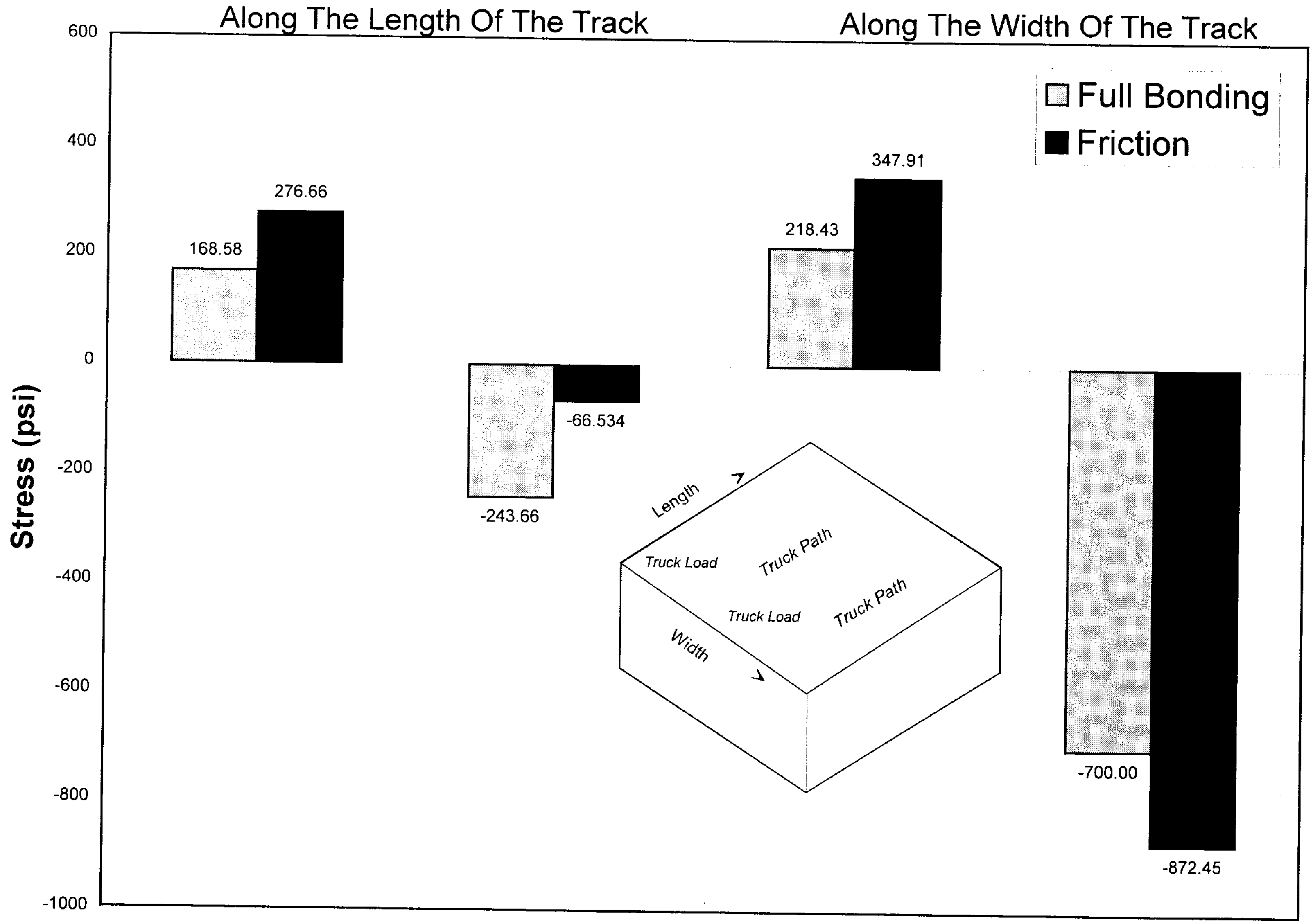
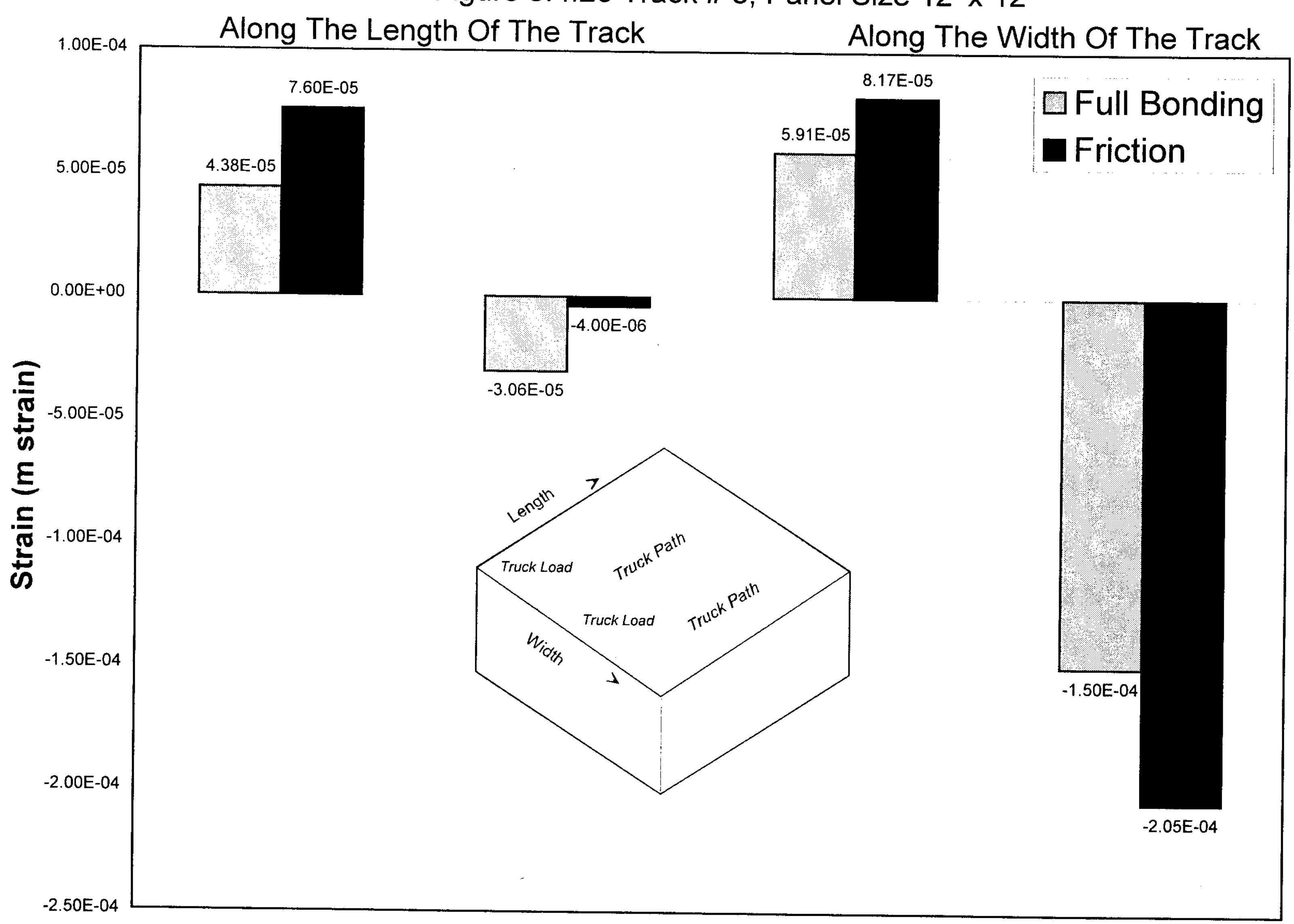


Figure 5.4.26 Track # 3, Panel Size 12' x 12'



# CHAPTER 6

# DESIGN SOFTWARE

## 6.1 FDOT UTW DESIGN SOFTWARE (FUTW)

An attempt has been made in this investigation to develop a user friendly application to ease off the iterative process involved in designing the ultra thin whitetopping. The computer software was named FDOT Ultra thin Whitetopping (FUTW). The main purpose of this package was to use the pavement conditions and analyze the data to determine the minimum thickness needed for the concrete overlay. The methodology followed was the same as the one used by FDOT for UTW design. The reason for this adaptation was the practicality of the method itself. It is easy to use and a sensitivity analysis could be performed in a short period of time. The only drawback in this method was the bonding strength which could not be taken into account since the added overlay is considered to have a monolithic effect with the underlaying layers. This assumption is an oversight of the actual condition. However, there is no closed form solution that could count for such parameters. Some FEA driven software may claim to count for this effect, but in reality they do not. The UTW overlay bonding behavior should be considered in its three dimensional effect. This means the effects of the in-plane friction/adhesion resistance of the shear forces as well as adhesional resistance of the out-of-plane forces (pullout). A thorough finite element analysis conducted in this study showed that the two mechanisms acted simultaneously and one should not ignore the out-of-plane resistance which was measured by any existing laboratory or field testing. Therefore, the principal investigator. has placed the demand for the need of developing a testing procedure to determine out-of-plane resistance first before any UTW bonding consideration could be taken.

**Software design parameters:** The following are the parameters needed for the data of the software. These parameters include the relevant properties of the existing asphalt pavement and the properties of the concrete overlay.

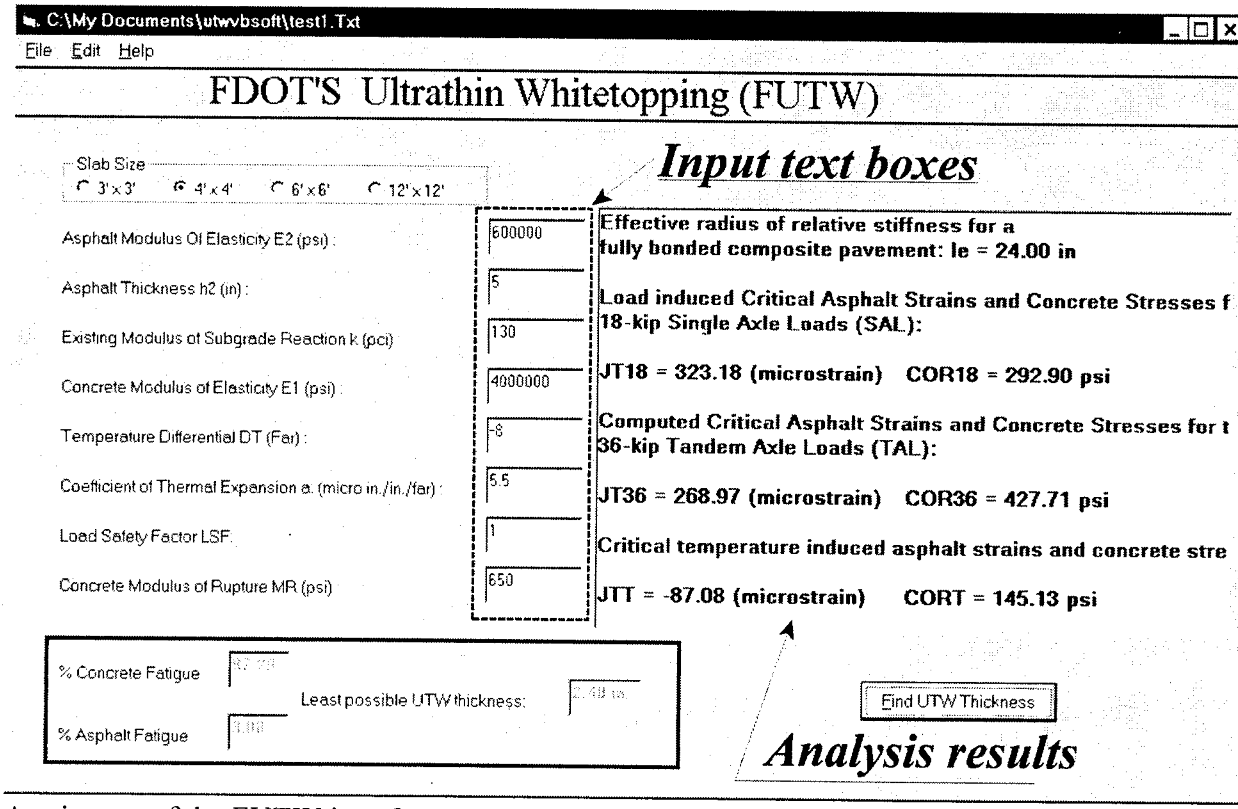
**Input Parameters:**

| | |
|---|---|
| Asphalt Modulus Of Elasticity, | E2 |
| Asphalt Thickness, | h2 |
| Existing Modulus Of Subgrade Reaction, | K |
| Concrete Modulus Of Elasticity, | E1 |
| Temperature Differential, | DT |
| Coefficient of Thermal Expansion, | a |
| Joint Spacing desired, | L |
| Load Safety Factor, | LSF |
| Concrete Modulus of Rupture, | $M_R$ |

All the input values should be entered in the text boxes provided in graphic user interface of the software, except for the choice of slab size which should be selected in advance from the options provided. Units could also be chosen. All the calculations for the stresses and strains in concrete and asphalt respectively are accomplished internally by the program and when the "Find UTW thickness" button is pressed, the least possible thickness needed for the whitetopping is given by the program with the condition that the total concrete/asphalt fatigue should be less than 100%.

Since UTW is characterized by overlay thickness that range between 2 to 4 in., the program automatically starts with a trial thickness of 2 in. and keeps iterating the procedure until the condition of the total fatigue of the concrete/asphalt layers is less than 100%. To facilitate the data, a typical example file was provided from which the user could start with modified measurements.

Once the solution is completed, the critical stresses and strains in the 18 kip axles for single axial loads case and 36 kip axles for tandem axle loads case are presented in a separate window. The file can be saved as a text file which can also be retrieved later for further analysis.



A print out of the FUTW interface

# CHAPTER 7

# CONCLUSIONS

UTW overlay is a thin concrete layer over an asphalt layer. Being very thin, the UTW should be given extra care in the analysis and design including the short and long term performance. Although this is true for all pavements, the UTW has its unique characteristics because of its inherent sensitivity to different structural and environmental distress factors.

The current study focused on several issues related to the design, construction, and performance of various layouts of UTW. The main design parameters of the UTW used in the study included, slab thickness, joint spacing, type of base, and subbase layers. Three test tracks were constructed at the FDOT materials laboratory in Gainesville, Florida. These test tracks were subjected over long time to repetitive truck loads simulating actual field conditions. Strain and deflection measurements were recorded using electronic strain gauges and LVDTs to determine the stress variations upon load application. Structural layer stiffness of the UTW sections were assessed using FWD tests before and after construction. To have an insight on the stress distribution at different slab thicknesses and panel sizes, a detailed finite element modeling was conducted considering the three Gainesville test tracks. Three nonlinear finite element analysis models were developed with a simulated moving truck loads similar to the one used in the field testing.

To facilitate the design and analysis of future UTW sections, a stand alone software named FUTW was compiled utilizing the methodology used by the FDOT. The purpose of this selection was the simplicity of the method, and the various parameters considered in the design. The graphic user interface (GUI) of the FUTW was set in a way that all the design parameters appeared on one screen with text boxes indicating the choices of the design parameters. An assisted window appears on the screen to show the process of the analysis. The final result of the FUTW is manifested by the suitability of the selected panel size and the slab thickness.

The main findings of the current study can be summarized as follows:

1- Like any other stiff layer, the addition of a thin concrete overlay manifested by the UTW significantly improved the load carrying capacity of the asphalt pavement used in the study. Accordingly, the UTW could be considered as viable option for rehabilitation and surface restoration of asphalt pavements.

2- Several design and construction parameters had to be considered to achieve the successful performance of the UTW overlays. Among these parameters were (i) slab thickness, (ii) joint spacing, (iii) stiffness of underlaying layers, and (iv) bonding characteristics of the concrete-asphalt interface.

3- UTW may not be suitable for some particular field conditions. For instance, the UTW constructed on Interstate I-10 leading to a weigh station has experienced cracking at a place where underground drainage existed. The failure could be due to the settlement of layers underneath due to seepage. Similarly, studies should be directed in order to throughly understand the effect of other environmental conditions such as excessive temperature changes, and vulnerability to chemical attacks.

4- Durability is one of the important factors to be considered, as one of the goals of whitetopping is to ensure long life for the pavements. Therefore, long term monitoring of the overlays is essential. Tests should have been carried out in excess of a two year period to understand the effect of the actual structural and environmental fatigue cycles on the UTW.

5- One of the major conclusions of the study was the fact that bonding between the asphalt layer and the UTW plays a significant and vital role in deciding the short and long performance of the pavement. The UTW slabs were found to have out-of-plane resistance in when high bonding occurred. Interestingly, there was no separation between the whitetopping and the asphalt layer in case of high bonding. They are rather intact. Results clearly indicated that the stresses and deflections in the high bonding sections were lower than those with weak bonding.

6- The Iowa shear test only determines the in-plane frictional shear which is not enough to predict the bonding strength as propping up of the slabs was noticed in when weak bonding occurred.

7- No methods are available to measure the amount of bonding required and also to achieve the right amount of in-plane and out-of plane bonding while constructing, proving to be a real disadvantage in this investigation.

8- As of now, the shearing strength at the concrete-asphalt interface should be at least 1.4 MPa. (200 psi), when the Iowa shearing test is used. This is based on test results from the Gainesville test tracks.

9- The method of analyzing and designing UTW could best be realized by using the FEM approach. However, such simulation may not be successful if certain parameters are clearly defined. These parameters include bonding, fatigue characteristics of concrete and asphalt layers, and temperature changes. Thus, the existing semi empirical approach was considered adequate until such factors were well defined.

10- The UTW should be designed using the short joint spacings. Based on results of this study, the preferred spacing for 75 mm. to 100 mm. (3 in. and 4 in.) thick UTW should be 1.20 to 1.80 m. (4 to 6 ft). For a 50 mm (2 in.) thick UTW, the joint spacing should not be greater than 1.2 m. (4 ft).

11- The UTW performed well on 38-mm. (1.5 in.) asphalt base. This finding allows the use of the UTW in Florida where the asphalt layers are relatively thin compared to the northern states.

12- The effect of fibers on performance of the UTW could not be determined. Sections using plain concrete and those using fiber concrete performed equally well. A possible explanation for that performance, was the fact that fibrillated plastic fiber was used. As oppose to the monofilament fiber, the fibrillated is used to control shrinkage cracking while the 2.5 in. monofilament fiber is utilized for structural cracking.

13- The software package developed in this study (FUTW) was aimed at facilitating the selection of a suitable UTW section for an existing asphalt pavement. It eases off the laborious iterative process through a user friendly GUI designed especially to display the data, results, and the developed internal strains all on one screen. The user can run a sensitivity analysis by changing the design parameters and monitoring the outcomes immediately.

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