Technical Report Documentation Page

1. Report No. FHWA/TX-09/0-5551-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle DEPLOYING WEIGH-IN-MOTION INSTALLATIONS ON ASPHALT CONCRETE PAVEMENTS		5. Report Date November 2008 Published: February 2009
		6. Performing Organization Code
7. Author(s) Emmanuel G. Fernando, Dan Middleton, Todd Carlson, Ryan Longmire, Edward Sepulveda, Leonard Ruback, Tom Freeman, and Jeongho Oh		8. Performing Organization Report No. Report 0-5551-1
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. Project 0-5551
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Technical Report: December 2006 – August 2008
		14. Sponsoring Agency Code

15. Supplementary Notes
Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.
Project Title: Alternatives Weigh-in-Motion Plan for Statewide Data
URL: http://tti.tamu.edu/documents/0-5551-1.pdf

16. Abstract
The strategic plan for establishing a vehicle weight monitoring net across Texas calls for deploying two technology types – piezoelectric and bending plate systems, and seeks to install weigh-in-motion (WIM) systems in roadways that are under new construction or re-construction. Because the goal is to provide accurate truck weight data for pavement design, the strategic plan prefers deploying WIM installations on 500-ft continuously reinforced concrete pavements. While these pavements have, from experience, provided suitable stable foundations for WIM sensors, building a continuously reinforced concrete pavement is expensive. This project aimed to find less costly but equally viable alternatives for deploying WIM installations by developing guidelines for finding sections within existing asphalt concrete pavements that provide the level of smoothness, pavement support, and projected service life deemed suitable for weigh-in-motion sites, particularly for installations that use piezoelectric technology. Additionally, the project sought to evaluate the use of solar cells to power WIM systems, and wireless alternatives for data communication. These alternatives become particularly relevant in areas where bringing electrical and telephone wires to the site would add significantly to the cost of the WIM installation.

17. Key Words	18. Distribution Statement
Weigh-in-Motion, Pavement Evaluation, Weigh-in-Motion Sites, WIM Pavement Smoothness, WIM Smoothness Criteria, Inertial Profile Measurement, WIM Pavement Life, WIM Installation, WIM Sensors, WIM Solar Power, WIM Wireless Communication	No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, VA 22161 http://www.ntis.gov

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	300	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

DEPLOYING WEIGH-IN-MOTION INSTALLATIONS ON ASPHALT CONCRETE PAVEMENTS

by

Emmanuel G. Fernando
Research Engineer
Texas Transportation Institute

Dan Middleton
Program Manager
Texas Transportation Institute

Todd Carlson
Associate Transportation Researcher
Texas Transportation Institute

Ryan Longmire
Assistant Research Specialist
Texas Transportation Institute

Edward Sepulveda
Assistant Transportation Researcher
Texas Transportation Institute

Leonard Ruback
Research Scientist
Texas Transportation Institute

Tom Freeman
Program Manager
Texas Transportation Institute

and

Jeongho Oh
Assistant Research Engineer
Texas Transportation Institute

Report 0-5551-1
Project 0-5551
Project Title: Alternatives Weigh-in-Motion Plan for Statewide Data

Performed in cooperation with the
Texas Department of Transportation
and the
Federal Highway Administration

November 2008
Published: February 2009

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report. The engineer in charge of the project was Dr. Emmanuel G. Fernando, P.E. #69614.

ACKNOWLEDGMENTS

The work reported herein was conducted as part of a research project sponsored by the Texas Department of Transportation and the Federal Highway Administration. The authors gratefully acknowledge the steadfast support and guidance of the project director, Mr. Joe Leidy, of the Materials and Pavements Section of TxDOT's Construction Division. Joe Leidy contacted the districts to identify recent flexible pavement projects for developing flexible pavement WIM evaluation guidelines. He facilitated the coordination of the field tests completed on selected projects, and sought and gained the cooperation of the Odessa District in preparing the proposed Balmorhea WIM site along I-10 for installation of the WIM system. In addition, the authors give thanks to members of the Project Monitoring Committee for their support of this project. In particular, Mr. Jim Neidigh of the Transportation Planning and Programming Division conducted the installation of the weigh-in-motion system along I-10 west of Balmorhea and provided WIM data from existing TxDOT WIM sites for evaluating WIM smoothness criteria. The authors are also grateful for the support provided by the Odessa, Lufkin, Bryan, Corpus Christi, and Laredo Districts in the evaluation of flexible pavement projects to determine guidelines for locating WIM installations on asphalt concrete pavements. In particular, the authors acknowledge the contributions made by the following TxDOT engineers and support staff:

- Stephen Smith, Ciro Baeza, Hector Flores, Kelli Williams, K. C. Evans, and Ruben Salcido, Jr. of the Odessa District;
- Richard Boles-Gracia and Kip Smith of the Lufkin District;
- Darlene Goehl and Roy Pilgrim of the Bryan District;
- Pete Stricker, America Garza, and Daniel Gonzalez of the Corpus Christi District; and
- Hector Cantu and Clarissa Sanchez of the Laredo District.

A special note of thanks is given to the Odessa District for completing site preparation work to deploy the Balmorhea WIM installation in this project. Finally, the authors also acknowledge the technical support provided by Gerry Harrison, Lee Gustavus, and David Zeig of the McNew Laboratory in running the field and laboratory tests needed to gather data for developing the flexible pavement WIM evaluation guidelines in this project.

TABLE OF CONTENTS

LIST OF FIGURES

Figure **Page**

LIST OF TABLES

Table **Page**

CHAPTER I. INTRODUCTION

A controlling factor in pavement design is the accuracy of estimates of axle loads that the pavement is expected to experience over its design life. Gross inaccuracies in traffic characterization can result in premature failures due to under-design, or conversely, in inefficient use of limited funds due to over-design. Clearly, an accurate assessment of axle load magnitudes and corresponding numbers of load repetitions is required to determine an optimal pavement design (in terms of material selection and layer thickness) for the expected service (traffic and environmental) conditions during the design period. Crucial to achieving this requirement is an effective weigh-in-motion (WIM) program with instrumented sites strategically placed over the highway network to characterize truck traffic and provide axle load distribution data for cost-effective pavement design; prioritization of maintenance, rehabilitation, reconstruction, or new construction activities; forensic investigations of premature failures; and planning purposes.

Recognizing the need for an effective vehicle weight monitoring network, the Texas Department of Transportation (TxDOT) in cooperation with the Texas Transportation Institute (TTI) established a strategic plan (TxDOT, 2003) for deploying and operating a state network of weigh-in-motion sites. Among the targeted needs that this WIM network would serve are:

- Support implementation of the new mechanistic-empirical pavement design method developed in the National Cooperative Highway Research Program (NCHRP) Project 1-37A. This new method uses the axle load distribution in lieu of 18-kip equivalent single axle loads (ESALs) to determine pavement material and thickness requirements.
- Satisfy federal reporting requirements specified in the Federal Highway Administration's (FHWA's) Traffic Monitoring Guide (TMG).
- Support state and federal initiatives such as the North American Free Trade Agreement (NAFTA) and the Texas Transportation Commission's goal of having 90 percent of state roads and 80 percent of Texas bridges in good or better condition within 10 years.
- Support existing TxDOT programs and activities.

The strategic weigh-in-motion plan calls for deploying two technology types – piezoelectric and bending plate systems and seeks to install WIM systems in roadways that are under new construction or re-construction. Because the goal is to provide accurate truck weight data for pavement design, the plan prefers deploying WIM installations on 500-ft continuously reinforced concrete pavements (CRCPs) that satisfy the smoothness requirements in the American Society for Testing and Materials' (ASTM) E1318 (2006) specification. This pavement type is thought to require the least amount of work to maintain the level of smoothness needed to acquire accurate and reliable weigh-in-motion data over the life of the WIM installation. However, while continuously reinforced concrete pavements have, from experience, provided stable foundations for WIM sensors, building this type of pavement is expensive. Given that CRCPs comprise only about five percent of the total number of lane-miles of the state highway system (TxDOT, 2006), deploying a network of CRCP WIM installations will require new construction of 500-ft CRCP sections within existing lanes of asphalt concrete pavements. Considering that the strategic WIM plan calls for deploying 133 additional weigh-in-motion sites throughout the state, a significant reduction in the cost of establishing the network could be achieved if WIM installations could be placed on newly constructed, re-constructed, or resurfaced asphalt concrete pavements.

RESEARCH OBJECTIVES

Project 0-5551 aimed to support the implementation of TxDOT's strategic weigh-in-motion plan by identifying less costly but equally viable alternatives for deploying WIM installations to cover the state highway network. Recognizing that a major cost component of a WIM installation is the CRCP slab on which the sensors are placed, this project sought to develop guidelines and procedures for finding sections within existing asphalt concrete pavements that provide the level of smoothness, pavement support, and projected service life deemed suitable for weigh-in-motion sites, particularly for installations that use piezoelectric technology. Additionally, this project recognized that TxDOT often incurs extra costs at WIM installations where land lines for electrical and telephone services are not available. Thus, researchers also sought to evaluate the use of solar cells to power WIM systems and wireless alternatives for data transmission. These alternatives become particularly relevant in areas where bringing electrical and telephone wires to the site would add significantly to the cost of the WIM installation. Finally, this project recognized the steady increase in research and development (R&D) of new WIM systems and sensor technologies over the last few years. Driving the R&D efforts are concerns over system accuracy, temperature dependency, speed dependency, signal degradation, and cost effectiveness. Thus, researchers also sought to review recent developments in weigh-in-motion technology to identify alternative sensors for capturing WIM data that could be considered for testing in this project to evaluate their performance and cost-effectiveness.

RESEARCH WORK PLAN

To accomplish the project objectives, researchers carried out a comprehensive work plan that covered the following tasks:

- reviewed current WIM practice that covered an extensive literature search and communications with state departments of transportation (DOTs) and equipment manufacturers,
- conducted field and laboratory tests and data analyses to establish guidelines for evaluating flexible pavements to identify suitable WIM locations,
- investigated the use of solar power and wireless communications, and designed and installed a wireless setup for an actual TxDOT WIM site,
- monitored TxDOT's installation of quartz WIM sensors to document the procedures for placing these load sensors on flexible pavements, and
- conducted roundtable discussions to identify user requirements for truck weight data within the department.

The following chapters of this report document each of the tasks conducted in this project.

CHAPTER II. REVIEW OF CURRENT WIM PRACTICE

LITERATURE REVIEW

Researchers conducted a comprehensive search of the literature and found relevant reports or articles pertaining primarily to tests of new sensors, such as fiber-optic sensors or quartz sensors and pavement smoothness criteria. Researchers also contacted equipment manufacturers to identify candidate WIM sensors for possible evaluation in this project. The primary interest regarding new (low-cost) sensors is in fiber-optic sensors. However, researchers also reviewed research work conducted on TxDOT Project 0-4509 by Liu et al. (2005) that developed a microwave sensor for weigh-in-motion. This chapter presents the findings from the review conducted by researchers to assess the state of WIM practice.

New and/or Low-Cost WIM Sensors

Fiber-optic sensors are a union of the laser electronic industry and the fiber-optic telecommunication industry thus enabling advancements with little incremental cost. TTI found reports or articles claiming that fiber-optic sensors are an excellent candidate for WIM devices and have been proven in measuring bridge stress. They have a versatile range in the fiber Bragg-grating (FBG) design for measuring strain. There are commercial off-the-shelf units for WIM in rail car applications that can measure up to 20 tons. The articles allege that fiber-optic sensors have several advantages over existing sensors—they are not responsive to electromagnetic interference including lightning strikes, they can withstand harsh environments, and they have low power requirements. A fiber-optic sensor has been developed, which consists of either a laser diode (LD) or a light-emitting diode (LED), optical fibers, and a data acquisition module. The LD or LED emits light that travels through the optical fiber cable. When a force from a load causes microbends in the fibers, the light traveling through the fiber cable "leaks" out, causing a loss in light intensity. An appropriate signal processor can measure this loss of intensity and correlate this change to the applied force (Cosentino et al., 1996 and de Vries et al., 1996).

Udd and Kunzler (2003) conducted research on fiber-optic sensors using fiber Bragg-grating technology for use in vehicle classification and possibly weigh-in-motion for the Oregon Department of Transportation (ODOT). Reasons given by the authors justifying the use of FBG sensors is that the demodulation system is external to the sensor, more easily facilitating upgrades after the sensor is installed. These FBG sensors are also compatible with a family of sensors for roadway and civil applications such as humidity, ice, temperature, corrosion, and moisture sensors. The applicable principle for use in WIM is that strain induced in road surfaces from vehicle axle loads is transferred into the FBG traffic sensor housings, straining the sensors in proportion to the weight and speed.

The research involved two phases of sensor installations. The first began in 1999 with construction of two test pads to evaluate FBG sensors, one in asphalt pavement and one in Portland cement concrete. Each of these two pads was 10 ft by 10 ft, and they were placed end to end forming a 10 ft by 20 ft composite test area. Subsequent testing involved eight sensors consisting of two different FBG sensor prototypes that were designed and built by the fall of 2000. The sensors were installed in parallel saw cuts that were 24 inches apart and were loaded by a 3200 lb passenger car driving over them. Udd and Kunzler (2003) concluded from these pad tests that additional tests in a traffic stream were warranted with modifications to the sensor elements.

The second phase of the research involved installation of the FBG sensor on I-84 in Oregon on a section of the freeway where the average daily traffic across six lanes was 57,900 vehicles per day. The initial installation consisted of four sensors that were 4 ft in length placed on the left wheel path of the rightmost through lane. The sealant used for all sensors was a hot bituminous material, which was finished level with the pavement surface. The intent was to primarily measure the strain in the pavement rather than in the sealant. After two weeks of traffic exposure, researchers closely examined signals from all four sensors and concluded that all four had been damaged, possibly by the hot bituminous sealant. After six months of testing, the sensors had not experienced any further deterioration in performance or sensitivity. The authors speculated that it would be theoretically possible for the sensors to last indefinitely although the basis for that conclusion appears to be weak. Analysis of the four sensor responses seemed to indicate the most likely problem being the detachment of the gratings inside the housing. All four of the sensors lost their pre-tension settings, which would indicate a physical break in the sensor line. Since failure in the prototype sensors was likely attributable to sensor overstrain from traffic, succeeding sensor designs focused in this area (Udd and Kunzler, 2003).

Based on the initial installation on I-84, researchers redesigned the FBG traffic sensors, developed four different sensors, and installed them in August 2002. Key criteria in the redesign were sensitivity, durability, ease of installation, and load repeatability. Design 1 involved a composite-reinforced sensor that would protect the sensor by limiting the strain force on the sensor. The composite encasement would protect the tubing and grating by absorbing extremely high strain forces. Design 2 used a spring to dampen the strain effect of traffic; the spring would allow for tension to be released if extremely large strain forces are present. Design 3 enhanced the first generation sensors by adding splice protection, a crimped anchor support, and improved housing strength. Design 4 used direct embedment of the fiber grating area in a composite beam without a protective tube. Freeway installations, again on I-84 near the original sensors, used only Designs 1, 3, and 4. All four of the sensors remained fully functional with no indication of changes in pre-straining as occurred with the original prototype sensors. There are still some unresolved issues related to the performance of the sensors such as time-dependent drift based on temperature change, but the change is slow and predictable. Repeatability (with a given load) seemed to improve with the second generation sensors.

Udd and Kunzler (2003) concluded that the second generation sensors were a significant improvement over the initial sensors. Even though the composite beam approach reduced the recovery time of the sensor, its susceptibility to sudden shock or overstrain was reduced. The newer sensors did not detach from the housing as the original sensors did. Early tests indicate promising results as a vehicle classification sensor. Early 2003 costs for the sensors ranged from $600 to $700 in small quantities, but this price should drop significantly with larger quantities. Use of these sensors also requires an interface unit at a cost (in small quantities) of "several thousands of dollars." There are several statements in the report indicating that the output from the sensors depends on the loading position along the sensor. Therefore, for WIM applications, it will be necessary to modify the design of the sensors and their placement so that the position of the load does not affect the measurement. Future research recommendations made by Udd and Kunzler (2003) include evaluating optimum installation depth, speed-related effects, and materials and procedures for installation.

Research by Cosentino and Grossman (2000) developed a microbend fiber-optic traffic sensor (FOTS), which is sealed from the environment, flexible, and installed below the roadway in an encapsulating material. Microbend sensors function by the fiber being mechanically deformed so that the light guided into the core of the fiber is passed out of the core of the fiber into the cladding layers. This form is the least expensive of the fiber-optic sensor technologies currently available. When a microbend sensor is deflected, the intensity of the light exiting the sensor is less than in the undeflected condition. The report by Cosentino and Grossman (2000) covers Phase III of a cooperative effort between the Florida Institute of Technology and the Florida DOT.

Phase I of the project was a 12-month study to develop a vehicle classification system using fiber-optic sensors. This phase developed four generations of sensors. Data indicated accurate counts of vehicle axles and vehicle classifications in a sample of 250,000 axles within one month of installation. According to the resulting documentation, the analog signal from the generation three and generation four sensors could be correlated directly to axle weights because both the magnitude and duration of the signal varied with load (Cosentino and Grossman, 2000).

Phase II of the project was an 18-month study which subjected microbend sensors to vehicle loading in both flexible and rigid pavements. The optical electronics interface contained LEDs that focus light into the core of the optical fiber and photo-detection equipment that converts light intensity to voltage. This voltage can serve the needs of either WIM or classification. Costs of these sensors are expected to be similar to the cost of piezoelectric sensors. The sensors are non-corrosive and are not affected by power surges. Field studies involved installing five sensors in 1997 where they could be subjected to passage of heavy trucks. Three of these sensors were in Portland cement concrete pavement serving about 1000 trucks per month, and two are in asphalt pavements serving 2000 trucks per month. After about seven months, three of the five sensors had failed, and investigations were underway to determine the cause for failure (Cosentino and Grossman, 2000).

Phase III (Cosentino and Grossman, 2000) tested nearly 50 fiber-optic microbend sensors in five field sites in Florida – four in flexible pavements and one in a rigid pavement. The sensors used in this research were 6 ft in length and used fiber-optic leads. The sensors used relatively soft encapsulation materials that were not temperature dependent. The research tested the sensors in both horizontal and vertical orientations. Two of the sites were Florida DOT telemetry sites used for vehicle classification. The other sites were on access roads to industrial plants that had significant heavy truck traffic. Tests of these sensors included field tests using a falling weight deflectometer (FWD) and lab tests designed to test their behavior under static and pneumatic loading conditions intended to replicate loads and other conditions encountered in pavements.

The authors concluded that the sensors installed in the five field sites had operated long enough to indicate that the technology is marketable. Ten sensors installed at one site in December 1998 continued to work properly through the date the report was written (October 2000). Other sensors installed at entrances to industrial sites and serving heavy truck traffic had also continued to function properly. Four sensors installed at a telemetry site in September 1999 continued to function but signals were weaker than desired for proper classification.

Cosentino and Grossman (2000) concluded that the sensors can be installed either horizontally or vertically, although vertical installation causes less pavement damage and results in longer sensor life. Sensor performance related directly to the material used to seal the sensor, with cellophane-based fiber embedded strapping tape preferred over electrical conduit heat shrink. Of the variables investigated for having an influence on signal generation, the encapsulant material and the sensor orientation had the greatest impact. Differing pavement surface temperatures had minimal impact on signal response.

Research by Szary et al. (1999) attempted to find a better WIM sensor than the typical piezoelectric polyvinyldine flouride (PVDF) polymer sensor. Their research first identified some weaknesses of the current PVDF sensors and therefore a reason to seek better solutions. One is the variability in the voltage output due mainly to temperature fluctuations. Even though built-in temperature corrections and calibration reduce these effects, they are not totally removed. Also, in many cases, this technology cannot correct for the hysteresis that the PVDF sensors experience, causing flawed readings. The other major weakness is that PVDF polymer sensors are more prone to physical damage under heavy loads, leading to premature sensor failure.

The main advantage of the PVDF polymer sensor is its ability to conform to the shape of the roadway. However, these sensors have a low coupling component, are difficult to pole, and have a selected temperature range that may be a constraint to their use for WIM. Their use above 140 °F is not recommended. Lead zirconate titanate (PZT) ceramics have shown the highest promise as a sensor material, but they are brittle, non-flexible, and non-conformable. A solution has been sought and successfully developed with composites of piezoelectric ceramics with flexible inactive polymers. These composites show excellent electromechanical properties while limiting the detrimental properties of products made from single materials. By designing the right structure, one can achieve PZT-like electrical properties and the added flexibility provided by the current polymers (Szary et al., 1999).

This research attempted to develop a composite sensor that would be durable and accurate. As the project progressed, researchers ruled out a number of epoxies for embedment of the sensors. Eventually, they chose G-100 epoxy (E-Bond Epoxies, Inc. of Ft. Lauderdale, FL). The sensor was then placed in an aluminum channel. Testing only occurred in a laboratory setting and not in the field. The lab testing occurred on a 222kN, 810 MTS universal machine with TRS 2000 operating software and an environmental control chamber at temperatures ranging from 23 °C up to 65 °C in 10 °C increments.

Findings indicate that the composite sensor developed in this research offered advantages over PVDF sensors, including a higher voltage output, higher temperature range, resistance to mechanical damage, and tolerance of higher loads. From the laboratory testing, the composite sensors take a longer time to respond to a load compared to PVDF sensors, but they recover quickly for the next loading cycle. Testing indicated that the PZT composite sensors performed as well as or better than commercially available PVDF sensors. Szary et al. (1999) concluded that these sensors show promise for use in practical applications but field testing had not occurred at the time of the report.

In a recent research project, Liu et al. (2005) investigated innovative sensors and techniques for measuring traffic loads on the highway infrastructure. For sensors, this research investigated both existing piezoelectric weigh-in-motion sensors and newer fiber-optic and microwave sensors to determine their viability as a replacement or in addition to the existing WIM sensors. Piezo sensors included in this research were the *Roadtrax BL* sensors by Measurement Specialties, Inc., *Virbracoax* by Thermocoax, Inc., and an encapsulated sensor from ECM. Another part of the research investigated a new algorithm using pavement deflection, comparing it to the integration algorithm. Liu et al. (2005) believe that the pavement deflection algorithm could be used for vehicle weight measurement.

The idea for both the microwave sensor and the fiber-optic sensor was borne out of the need for a less expensive means of measuring traffic loading. Fiber-optic sensors are not new to the realm of WIM detectors, but microwave sensors are. As proposed in this research, the microwave concept is based on microwave cavity theory in which a microwave signal generator sends a signal into one end of a hollow metal pipe, which undergoes elastic deformation under load. The measured variable is the shift in resonant frequency of the microwave energy field. As for the fiber-optic sensor, Liu et al. (2005) seem to offer conflicting results, stating at one point in the project report that this sensor is not practical for current application as a WIM sensor and at another point stating that it is "…a very good candidate for the WIM system." Support for the first statement was not as apparent as for the second. The report goes on to support the use of fiber-optic sensors (fiber Bragg-grating) when compared to piezoelectric sensors, indicating that fiber-optic sensors can utilize a simpler weight algorithm and are expected to last longer than piezoelectric sensors. However, according to these findings, a current limitation of the FBG sensor is that accurate results require loading at the same physical point along the sensor (Liu et al., 2005).

This project installed three sensor types at a Department of Public Safety enforcement site near New Waverly, Texas, along northbound I-45. Again, the sensor technologies were piezoelectric, fiber-optic, and microwave WIM sensors. The report covered only the lab results of the microwave sensor but concluded that with heavy loading, the sensor was able to measure the applied load to within 10 percent of the actual load. The results also indicated satisfactory linearity and uniformity, but again these results are based only on lab tests. The microwave system is new and is not a refined marketable system. It requires a PC in the field for its operation, running under a Linux operating system. Finally, the results suggest that more research is needed with a higher frequency circuit, which might improve measurement accuracy (Liu et al., 2005).

The Connecticut Department of Transportation (ConnDOT) was the first state in the U.S. to install Kistler quartz piezoelectric sensors for weigh-in-motion. A paper presented at the 2000 North American Travel Monitoring and Exposition Conference (NATMEC) described some of the performance and endurance attributes of the new sensors (McDonnell, 2000). The overall objectives of the research effort were to install the sensors and to determine sensor survivability, accuracy, and reliability under actual traffic conditions in the Connecticut environment. At the outset, the evaluation period was to cover a time span of at least three years.

The initial installation occurred in October 1997 in all four lanes of Route 2 in Lebanon, CT. This initial installation covered the full lane width, installing four 1-meter length load sensors end-to-end. There were two of these full-width strips of quartz sensors spaced 16 ft apart in each lane, along with two inductive loops per lane, one upstream and one downstream of the two strips of quartz sensors. The speed limit was initially 55 mph, but it was subsequently raised to 65 mph on October 1, 1998. Moisture penetration into the sensor leads required all load sensors to be replaced in July 1998. Several were reinstalled a second time in September 1998 (reasons not specified). ConnDOT conducted four field validations using trucks of known weights (McDonnell, 2000).

Following calibration of the load sensors in April 2000 using five-axle tractor-semitrailers, results indicated that lanes 1 and 4 (outside lanes) complied with the American Society for Testing and Materials (ASTM) E1318 requirements for gross vehicle weights (i.e., within ±10 percent, 95 percent confidence interval). Steer axles met the ASTM range of ±20 percent on all lanes 95 percent of the time from October 1998 to April 2000. ConnDOT did other analyses of the data to determine shifts or changes in the weight distribution over time. There was a shift in the 1999 data but not the 2000 data. Changes in the 1999 data could have been a result of changes in the load sensors, pavement, environment, or actual changes in the traffic stream. Field inspections revealed cracking between the sensors and the point of connection with cables. Field testing by the manufacturer in August 2000 found one load sensor in lane 1 and another in lane 3 needing replacement (McDonnell, 2000).

In October 1999, ConnDOT had to regrind a load sensor in lane 3 that was protruding ¼ inch above the pavement. ConnDOT replaced this lane 3 sensor and another sensor in lane 1 in November 2000 due to reduced signal strength. ConnDOT found evidence of mice chewing on wires in the lane 3 hand hole during this replacement. This discovery raised suspicions about more widespread damage from mice in connection with previous sensor failures. At the remaining three sensor installation sites, there have been no sensor failures. The sensor design has also been slightly revised since the first installations (Larsen and McDonnell, 1999).

The overall assessment of the load sensors at the end of the second year concluded that the sensors produced good weight data. More work needed to be done to determine why sensors in lanes 2 and 3 were not performing as well as those in lanes 1 and 4. The paper made no firm conclusion as to the expected life of the sensors ((McDonnell, 2000).

Recent research, conducted by TTI and the Center for Transportation Research (CTR) at the University of Texas at Austin included contacting states to determine their experience with the quartz WIM sensors and full-scale field tests (Middleton et al., 2005). The states contacted by the research team included Illinois, Maine, Michigan, Minnesota, Montana, and Ohio to discuss their experiences with the Kistler Quartz sensors. The performance of these sensors in hot-mix asphalt concrete (HMAC) was of particular interest.

Illinois DOT (IDOT) uses Kistler sensors in its Pre-Pass system as a sorter to determine the need for static weighing. IDOT started installing these load sensors around 1999 – 2000. The initial decision to use these sensors considered a quick installation time, along with their accuracy. There are 18 weigh stations that weigh about 2.7 million trucks per year using these Kistlers, bypassing about 2 million of these trucks. The average life of these load sensors, based on the Illinois experience, is about two years. IDOT calibrates the Kistler sensors about three to four times per year, typically based on complaints from Pre-Pass personnel. IDOT uses hydraulic load cells at 17 of its 20 Interstate weigh stations; it uses no bending plate systems at all. Overall, the state prefers load cells because they do not fail as often as the Kistlers or bending plates, and the state does not have to request replacement money as often. For future WIM sensors, in most cases IDOT will replace failed sensors with new Kistlers. However, IDOT will replace some Kistlers with load cell systems. Kistlers installed by IDOT in concrete seem to last longer and perform better than in asphalt (Middleton et al., 2005).

Maine DOT had 13 WIM stations installed with Kistler sensors for a total of 132 load sensors. During a five-year period during which sensors were being installed, there were 18 quartz sensor failures. In all cases, the failures were internal to the sensor; there were no bad connections or sensor lead failures. In all failed sensors, the meter reading indicated low impedance to ground. Kistler representatives also thought that Maine may have received a "bad batch" of sensors. Maine DOT calibrates all sites once a year and checks sites on a weekly basis for discrepancies in the data. Maine DOT has found the Kistler sensors to be "extremely accurate." The department has calibrated sensors to within a 2 percent error compared to test vehicle gross weight and plans to continue using Kistler sensors even though their failure rate is of significant concern. The accuracy of the sensor is the overriding factor causing the agency to continue installing and using them (Middleton et al., 2005).

Michigan Department of Transportation (MDOT) had a total of 30 lanes at eight sites that used Kistler sensors for weigh-in-motion data collection; none are enforcement sites. In 2004, the oldest of the Kistlers was three years old, and the most recent installations occurred in early October 2004. Only six of these lanes were in asphalt pavements with the oldest installed about 1½ years earlier. One of these installations was in a six-inch asphalt overlay with concrete underneath. MDOT had no failures considered to be the fault of the load sensors. The best conditions for installation of the sensors to achieve good cure time is in ambient temperature of

70 °F or higher. The grout will cure in cooler temperatures, but the time required to keep the lanes closed might become an issue (Middleton et al., 2005).

Minnesota Department of Transportation (MnDOT) completed its most recent Kistler installation in September 2004, bringing the total number of asphalt concrete lanes with Kistler sensors to six and the corresponding number of concrete lanes to eight. The installations in asphalt are newer than the ones in concrete, but none of the sensors have been installed for a sufficient length of time to draw strong conclusions. MnDOT installed an additional system on the MnROAD project with one set of sensors in asphalt and one in concrete to test for seasonal drift and durability. MnDOT has not milled the pavement around the sensors because it selected sites with smooth existing pavement. MnDOT has had no problems at all with the Kistler sensors, but the installation process for these sensors requires complete attention to detail. The Kistler sensors are delicate instruments that must be protected and installed properly. The MnDOT experience with these sensors indicates that when the sensors are properly installed by following the detailed instructions from the manufacturer, the sensors have a good bond with the existing pavement and seem to be very durable (Middleton et al., 2005).

Montana Department of Transportation (MDT) discovered a sensor problem at two sites in November 2002, which turned out to be a grounding problem due to a manufacturing defect. Kistler replaced the sensors, and the state reinstalled them in May 2003. Following calibration by MDT in the fall of 2003, the weights agreed closely with static scale data. MDT retested the sensors in December 2003 and found an instability problem in one charge amplifier. Neither weather nor splicing of lead cables were factors. MDT has not performed any in-road maintenance. Prior to replacement, there were no signs of cracks or damage. A recalibration check performed in the spring of 2004 required little or no calibration adjustment. All sites used two sensors per wheel path – a 2.46-ft element and a 3.28-ft element. Each site used the Kistler-supplied grout around the sensors. Based on very limited experience, MDT plans on continuing the use of Kistler sensors as long as their durability is adequate. There is a four-lane installation planned in Rocker, Montana, as part of a Pre-Pass system (Middleton et al., 2005).

The same project by TTI and CTR evaluated the Kistler sensors on SH6 in College Station, and at two other locations in Texas. SH6 is a continuously reinforced concrete pavement overlaid with about 3 inches of HMAC. Through careful analysis of the quartz WIM data, the project reaffirmed what other users had already discovered. When properly installed in pavements that provide adequate structural support, quartz sensors produce accurate vehicle weight measurements that remain stable over time. Furthermore, the quartz sensors in SH6 have not exhibited any signs of physical degradation such as cracks in the sensor and surrounding pavement. One can also infer from the evaluation of the average front axle weight and GVW distributions by week and the follow-up calibration verifications that there is also no significant degradation of the quartz sensor signal (Middleton et al., 2005).

Based on the static versus dynamic weight comparisons collected at both sites over the duration of the study using calibration trucks, and in one case mixed truck traffic selected at random, all weights collected satisfied the ASTM GVW and axle weight accuracy specifications for Type 1 WIM systems. Tabulating the combined data evaluated in this project produced 245 static versus dynamic weight observations from measurements of GVW, steering axles, and axle groups. All 245 dynamic weight measurements fell within ±15 percent of the static weight. Furthermore, 100 percent of the GVW observations satisfied the ±10 percent ASTM criteria; 100 percent of steering axle observations satisfied the ±20 percent criteria (±15 percent was achieved); and 100 percent of all axle group observations satisfied the ±15 percent criteria (Middleton et al., 2005).

It should be noted that 200 of these observations were generated by calibration trucks making multiple controlled passes over the load sensors. Thus, the results are likely skewed somewhat in favor of the sensors. Installation conditions at the other two sites (besides SH6) were near optimum, and both were in Portland cement concrete pavements. One site has 500 ft of Portland cement concrete that was ground smooth to satisfy the ASTM longitudinal roughness specification. Even though installers did not measure the longitudinal roughness at the third site, the relatively slow traffic speed (14.6 mph on average) served to minimize the impact of any roughness that did exist (Middleton et al., 2005).

Based on the results of Kistler tests in Texas and the experience in other states, the Kistler sensors appear to have merit for continued testing in Texas. Middleton et al. (2005) recommended continued monitoring of the Kistler sensors at the three sites tested in TxDOT Project 0-4664. Following up on that project, the Transportation Planning and Programming (TPP) Division of TxDOT installed Kistler quartz sensors on the southbound perpetual pavement lanes along I-35 south of Cotulla as well as on the northbound lanes where the load sensors were placed on CRCP. Researchers tested these WIM sections during the current project and found, among other things, that the sections meet the ASTM E1318 requirements for Type I WIM systems. Chapter III of this report documents the tests conducted at the Cotulla WIM site.

One ongoing research project in WIM technology is WAVE (Weigh-in-motion of Axles and Vehicles for Europe). WAVE is a pan-European project that has specific objectives of improving accuracy and performance of WIM systems. This initiative is investigating multiple sensor systems and bridge systems as a means of improving accuracy. Additionally, it is conducting research into improved durability in colder climates and improving calibration and test procedures to improve performance (Jacob and O'Brien, 1996). Continued research and development of new sensors and systems will provide improvement in WIM costs, portability, and accuracy.

In May 1999, a symposium presented the results of WAVE (1996-1999) in Paris. There were 11 partners representing 10 countries that took part in the WAVE project. The four main areas WAVE focused its research efforts on were (Jacob, 1999):

- accurate estimation of static weights using WIM systems (this area specifically investigated multiple-sensor WIM and bridge WIM);
- quality, management, and exchange of WIM data;
- consistency of accuracy and durability (this area focused on environmental factors such as cold weather operations and calibration procedures); and
- optical WIM systems.

WAVE researchers reported two new theories that were successfully developed to optimize the estimation of static weights. These experimental studies utilized multiple sensor arrays of 11 to 15 sensors that proved to be very accurate (±2 percent of static weight) under controlled conditions (Livingston, 1998). Findings also reported progress in the area of bridge WIM. Both of these areas developed new algorithms to improve accuracy; however, these algorithms have yet to be implemented into marketable WIM systems (Jacob, 1999).

WAVE researchers also reported strides in the development of fiber-optic technology. These include development of prototype systems and beginning of initial testing. The fiber-optic sensor developed in a partnership with the Laboratoire Central des Ponts et Chaussées uses light birefringence in optical fibers, which undergoes a mechanical strain. The sensor design uses a fiber placed between two metal ribbons and embedded in an elastomer material. Initial testing resulted in waveform problems due to high loading. Thus, a second prototype was developed.

The new design, which tested satisfactorily in the laboratory environment and in initial tests, was then prepared and ready for field-testing. No results of the field testing were available (Jacob, 1999).

Pavement Smoothness

The goal of the Long-Term Pavement Performance (LTPP) program is to provide the data necessary to explain how pavements perform (Karamihas, et al. 2004). To accomplish this goal, LTPP has monitored and collected data on 2500 pavement test sections located on in-service highways throughout North America. Data collected at each test section include traffic, climatic, pavement performance monitoring, maintenance, and rehabilitation data. A critical component is the collection of accurate traffic loading data by using weigh-in-motion equipment located near test sections. LTPP established pavement smoothness specifications, both ahead of and immediately beyond the WIM sites. These specifications are intended for the acceptance, verification, and annual checking of potential WIM sites. Pavement smoothness near WIM needed two indices to ensure that background pavement (long wavelength) roughness near the scale was acceptable as well as localized (short wavelength) roughness.

The research project conducted by Karamihas and Gillespie (2002, 2004) developed a virtual fleet of five-axle tractor-semitrailers to represent the heavy truck population. It utilized 9 types of tractors, 18 types of trailers, and 6 loading schemes. Each vehicle was equipped with an appropriate variety of truck tires, with a limited number equipped with wide-based single tires. Loading schemes included loaded to the legal limit, empty, overloaded, intermediate uniform loading, and forward biased loading. Simulation of these vehicles occurred at speeds of 45, 55, and 65 mph, resulting in a total number of 3696 vehicles simulated (Karamihas, et al. 2004).

The four types of pavements researchers considered were asphalt concrete, jointed plain concrete, jointed reinforced concrete, and asphalt overlay on concrete. For each construction type, the research team assembled profiles that spanned as large a range of International Roughness Index (IRI) as possible, with uniform representation over the range.

Truck simulations were confined to rigid body models of behavior in the pitch plane using two-dimensional vehicles. In other words, vehicles have height and length but no width, and they ride in a single wheel path. Equations of motion for the simulation models were written by using the AUTOSIM software program. Overall, 232,848 simulation runs were performed that covered every combination of vehicle parameter with every profile (Karamihas, et al. 2004).

Development of roughness criteria for WIM approaches required a profile-based index that, when applied to the measured profile at a site, could predict the WIM error level. The ASTM error tolerances for Type I WIM systems call for 95 percent confidence that steer axle load will be measured within 20 percent, tandem axle load within 15 percent, and gross vehicle load within 10 percent. Two ranges of pavement roughness stood out as influencing scale error: short-range roughness and long-range roughness. The short-range roughness covered the range of pavement that included about 10 ft preceding the scale, the scale itself, and about 1 ft beyond it. The long-range roughness covered the range of pavement that included about 80 ft preceding the scale, the scale itself, and about 10 ft beyond it. Along roadway segments where a WIM site is being considered, Karamihas et al. (2004) recommend using the roughness profile to determine the best site for the WIM scale. The procedure could also be used to identify candidates for corrective action. Researchers evaluated the WIM smoothness criteria developed by Karamihas and Gillespie (2002, 2004) during the current project. Chapter III of this report presents the findings from this evaluation.

The country of Hungary began using weigh-in-motion equipment for dynamic weight measurements in 1996, primarily relying on piezoelectric sensors and ECM Hestia equipment (Gulyas and Hernadi, 2000). Hungary joined the European COST Action 323 to facilitate international exchange of knowledge on vehicle weight data collection. Officials recognized that WIM data quality strongly depends on roadway surface characteristics, and began a program in 1991 to measure unevenness, rutting, cross profile, and texture by using the Swedish Laser Road Surface Tester (RST). Unevenness utilizes the IRI values and root mean square (RMS) values based on different wavelength ranges.

Data for this study came from measurements performed from 1997 to 1999. Data analysis fitted a linear regression model to the data based on three different unevenness characteristics and on pavement rutting. Unevenness characteristics were the IRI, the RMS1 for short wavelengths (less than 3 m), and the RMS2 for longer wavelengths (longer than 3m). The analysis also looked into some non-linear regression techniques but found no better results. Results indicate that, in some cases, there is no correlation at all while in other cases there is weak but statistically significant correlation. The authors suggest simply considering trends or tendencies when evaluating results. The RMS2 variable (wavelengths between 3 and 30 m) has the strongest effect on WIM accuracy and is therefore preferred over using the IRI. Gulyas and Hernadi (2000) point out that a weak relationship also exists between rutting and axle weight accuracy but was not evident with gross vehicle weight accuracy. They also found that the measured surface characteristics are useful in determining when maintenance is needed near WIM sites.

STATE AND VENDOR CONTACTS

States contacted for information on current weigh-in-motion activities were California, Florida, Minnesota, and Ohio. California and Florida provided information that might be helpful to TxDOT in choosing and installing wireless WIM components. Appendix A presents this information.

California Department of Transportation (CALTRANS)

One of the contacts for information was a retired CALTRANS engineer, who is now doing consulting work related to weigh-in-motion. He had worked for over 20 years with the department and was responsible for supervising the statewide WIM data collection program. In his current role, he works as an independent consultant but still on WIM issues. From a practitioner's standpoint, he is one of the most knowledgeable individuals on issues related to WIM. He indicated that a fiber-optic company had contacted him requesting assistance in developing a problem statement to develop and/or test a fiber-optic WIM sensor. This company was Intelligent Fiber-optic Systems (IFOS). However, the engineer did not know of any actual research that had been funded to evaluate the IFOS sensor. He spoke very candidly, saying that only Type I sensors such as load cell, bending plate, or quartz piezoelectric sensors will actually provide reasonably accurate data. He also said that some of the Specific Pavement Study (SPS) sites have quartz sensors in asphalt pavement.

The second contact was the person who replaced the retired CALTRANS WIM engineer. The department is currently operating 109 WIM sites statewide for data collection and 30 Pre-Pass sites (used for both enforcement screening and data collection). CALTRANS maintains all of the 109 data collection sites and most WIM components of the Pre-Pass sites. Of the currently active WIM sites, the equipment is almost exclusively bending plate equipment; only one six-lane asphalt site has Kistler quartz sensors. The pavement at this site had very little rutting, and the Kistler sensors were performing well. Five of the WIM sites use wireless communication equipment, but at that time, none used solar panels. CALTRANS has had very good experience with the wireless system. The department uses a 200-ft concrete slab at WIM sites to maintain adequate pavement smoothness for bending plate systems. The only type of sensor that might be considered for future WIM sites in asphalt (without a concrete slab) would be quartz. CALTRANS checks the pavement profile by using the standard ASTM E1318 straightedge method. CALTRANS would not consider placing bending plates in asphalt without the concrete slab. In fact, it is unlikely that vendors would provide a warranty for this situation.

For future WIM needs, CALTRANS will continue to rely on bending plate WIMs due to uncertainties about the newer quartz sensors. Even though these sensors have provided good data since their installation in February 2005, there is concern about the time required for replacing a failed sensor and replacing one sensor without damaging an adjacent sensor. For quartz sensors, the only way to replace a failed sensor is to cut it out. Time required for removal and replacement of a failed sensor and curing of grout is anticipated to be longer than the minimum time required for replacing bending plate units. An International Road Dynamics (IRD) bending plate can be replaced in as little as 30 minutes of lane closure time. Replacement of PAT bending plates takes a little longer, but that time may still be less than that required for replacement of a quartz sensor. Given that the warranty period for a bending plate system is five years and the cost is anticipated to be about the same, CALTRANS prefers bending plate systems.

Appendix A presents information on CALTRANS WIM installation procedures. CALTRANS has not installed any other new technology sensors recently for WIM other than the quartz sensors although they have been approached by vendors of new products. For example, IFOS recently brought a fiber-optic sensor to CALTRANS as a potential WIM sensor. IFOS representatives first demonstrated the sensor using an oscilloscope and wanted to borrow the WIM electronics from CALTRANS to conduct other demonstrations. IFOS was promoting the sensor as a high-speed application on the first contact, but subsequently promoted it as a low-speed application. CALTRANS chose not to consider the fiber-optic sensor for WIM beyond the initial demonstration.

Florida DOT

A seasoned Florida DOT (FDOT) representative did not recommend any new (low-cost) sensors that should be considered for WIM applications. Florida DOT discontinued funding of fiber-optic sensor research because the sensors were even less predictable in the roadway than piezoelectric sensors. The research was initially motivated by the need for a reliable axle sensor for vehicle classification although FDOT also tested the sensors to determine applicability to WIM. Lab results were good but, in the roadway, the vendors were not able to develop a satisfactory encapsulation material. It was either too stiff or not stiff enough for the fiber-optic sensors.

Until recently, FDOT philosophy was to install bending plate WIM in concrete (if available) and piezoelectric sensors in asphalt. At one time, FDOT personnel thought that some data was better than no data, but over time they have concluded that no data is better than bad data. FDOT is now replacing all the previous piezoelectric sites in asphalt with Kistler quartz sensors. The installation cost for quartz is about the same as for bending plates, but FDOT hopes the maintenance costs will be less. FDOT has a total of 41 WIM sites around the state with a few bending plates in asphalt. However, FDOT only installs bending plates in asphalt if truck volumes are relatively low. For interstate routes or other high-volume truck routes on asphalt, they only install Kistler quartz sensors. Currently, if FDOT has concrete pavement available, bending plates are installed. FDOT has had quartz sensors on I-10 (asphalt) for four years (as of February 2007) and has had no serious problems. They are considering regrinding due to increased rutting, but the sensors are continuing to provide good data. FDOT has had bending plates in place for as long as 15 years without significant problems but they require periodic maintenance.

The FDOT spokesman alluded to conducting tests on the quartz sensors but did not say there was a report available. Special training is needed, and special care is required during the installation process for Kistler sensors. FDOT does not do any pavement strengthening prior to the installation of the sensors in asphalt and does not build a concrete slab. Ninety percent of the Florida WIM installations are in asphalt. Appendix A presents information on FDOT WIM installation, including information on solar panels and wireless communication.

All of the FDOT WIM systems are using solar panels. There are occasional problems such as trees obscuring the panels, vandalism, and some sites being under-designed. As an example of the wattage that will suffice for WIM, FDOT used two 85 watt solar panels to power one site. For count/classification sites, only one of the total 300 sites has AC power. Of the 300 count/classification sites, 193 are now converted to wireless. FDOT personnel estimate that 20 to 30 additional sites are accessible to wireless services and will be converted at some future date.

Verizon has the state contract for service in Florida, so the state is using Raven modems at most of its sites. One problem with wireless communication in general is that service is not available everywhere. In Florida, most of the interstate is covered but there are some remote areas, even along the coast where service is not available. Overall, FDOT is pleased with wireless communication. One exception is a recent (early 2007) problem where 35 sites simultaneously did not communicate data. Troubleshooting the sites did not determine a conclusive cause of the problem, but 33 of the 35 sites had a common chip in the modem that Florida personnel suspected might have been the problem. The modems were all under warranty, but there was a delay of five weeks in getting them all repaired. The cost of wireless communication is almost always less than wired telephone service. For count and classification sites, the cost is about half what it was before with phone lines. For WIM sites with fairly low truck volumes, the cost is less with wireless. However, on higher truck volume sites, FDOT personnel are not sure which method is less expensive, but they still believe wireless is the better choice. One final point from the FDOT spokesman was that we are unlikely to find a “low-cost weigh-in-motion sensor” that will work satisfactorily.

Minnesota DOT

MnDOT is now installing only the Kistler Lineas quartz sensors but almost exclusively in concrete pavements. The few they have installed in HMAC pavements were on new or rehabilitated pavements that were very smooth. The state removed all bending plate WIM systems in 1998. MnDOT uses the ASTM standard methodology for checking pavement profile. MnDOT does not generally use wireless systems for WIM although one wireless communication system was being tested in July 2007 at an enforcement site. Solar panels are used at some classification sites but not at WIM sites.

Ohio DOT

The section head of the Ohio Department of Transportation (ODOT) who is responsible for traffic data collection provided the following information. ODOT currently has 45 total WIM sites statewide; 35 of these sites use piezoelectric sensors, and 10 use hydraulic load cells (Mettler Toledo). ODOT does not use a concrete slab for piezoelectric WIM sensors installed in asphalt. However, ODOT does build a 300-ft section of reinforced concrete for its load cell WIM sites. Otherwise, there is no special pavement treatment at the WIM sites to accommodate the WIM sensors. There was some initial pavement milling at the concrete sites when first installed but nothing since. ODOT uses the standard ASTM method for checking the pavement profile and does not use any non-destructive testing to check for pavement structural anomalies.

ODOT has not experimented with any new sensors for WIM but is anxious to try the Kistler quartz sensors. The department intended to purchase a limited number of these sensors but have not done so. ODOT does not have any wireless communication at WIM sites but has used it successfully for some time now at vehicle classification sites. One reason ODOT has hesitated in using wireless communications at WIM sites is due to the fact that a larger amount of data will be sent, although the concept has worked well at less demanding sites. One other reason wireless has not been expanded is that when these systems were installed several years ago, wireless was not as reliable as it is today.

International Road Dynamics

International Road Dynamics is one of the largest suppliers of weigh-in-motion systems and components in the U.S. and around the world. TTI has worked with IRD on multiple levels, both within the state of Texas related to the TxDOT WIM program and within professional organizations and conferences such as the Transportation Research Board and the NATMEC conference. In conversations with TTI researchers on the topic of new and promising WIM sensors, an IRD engineer had expressed an interest in FBG sensors and possible collaboration with TTI to install and conduct an evaluation of these sensors. Since that initial expression of interest, TTI has made more recent contact with the same IRD engineer with a somewhat different outcome.

The overall IRD experience suggests that fiber-optic sensors can work reasonably well as axle sensors but not as weigh-in-motion sensors. The difference is primarily due to the grout that is currently used for WIM applications. The grout material is critical to the success of fiber-optic WIM, and the proper material has not been found. IRD favors urethane materials over epoxies because epoxy materials result in too much signal loss. IRD has tested fiber-optic sensors for WIM by using a loop of fiber in the sensor and using a transmitter at each end, varying the signal. IRD has become disinterested in the Bragg-Grating fiber-optic sensor. IRD is now considering the use of sensors similar to those used on bridges on its bending plate WIM systems.

IRD was also involved in testing of a highly promising silicone rubber compound, for which Bridgestone has a patent. However, the current outlook indicates that Bridgestone will not be producing WIM sensors using this compound due primarily to the number of sensors that Bridgestone would need to produce to make the venture worthwhile. A visit by IRD personnel to Bridgestone offices in Japan led to the conclusion that the company was not interested in pursuing a deal using their silicone rubber compound. Apparently, Bridgestone would have to sell 100 million of these 0.25-inch diameter sensors to consider a deal with IRD.

There is a fiber-optic product from MSI, called Sensor Line, that has been tested as well, but it has not held up as a WIM sensor. It is not produced in the correct lengths for U.S. roadways anyway; it is either 3.0 or 3.5 m in length. Roadways in Europe experience very little deflection, so these sensors last much longer in Europe than on U.S. roads. There is also the Optical Sensors and Switches product from Florida, but the lack of activity from this company in recent years may be indicative of the state of fiber-optic sensors for highway applications.

The IRD engineer also indicated that some jurisdictions use HMAC pavement overlays in the vicinity of WIM sites to increase the life of the sensors and the WIM itself. Oklahoma and others have added a 3-inch to 4-inch hot-mix overlay with reasonably good success. Some Canadian provinces have done this such as New Brunswick, Nova Scotia, and Prince Edward Island. The idea of precast vaults in precast panels has also been tried with some success.

CHAPTER III. PAVEMENT EVALUATION TO IDENTIFY FLEXIBLE PAVEMENT WIM LOCATIONS

INTRODUCTION

This project covered field and laboratory testing, and analyses of test data to establish pavement evaluation criteria for identifying suitable WIM locations on flexible pavement projects. Factors considered in establishing these criteria included:

- pavement smoothness,
- support conditions as reflected in measured pavement deflections with the falling weight deflectometer,
- subsurface uniformity as inferred from ground penetrating radar (GPR) measurements,
- predicted pavement life as determined from FWD, GPR, and laboratory test results from Hamburg and overlay tests done on hot-mix asphalt concrete specimens, and
- highway geometric conditions.

This chapter presents the investigations conducted by researchers to establish guidelines for locating suitable WIM locations on flexible pavements.

EVALUATION OF HIGH-SPEED SMOOTHNESS CRITERIA

Weigh-in-motion systems need to provide good estimates of static loads for pavement design. However, because of the interaction between the vehicle and the road profile, WIM measurements will exhibit deviations from static vehicle loads. For this reason, it becomes important to have a smooth surface to minimize dynamic load variability on the WIM section. Given the current use by TxDOT of inertial profilers for network-level inventory of pavement smoothness and for quality assurance testing on paving projects under Item 585, researchers investigated methods for evaluating the surface smoothness at proposed WIM sites based on inertial profile measurements. In this regard, this task investigated WIM smoothness criteria based on surface profile criteria developed in a study conducted by Karamihas and Gillespie (2004) for the Federal Highway Administration. The recommended criteria were developed to ensure that the expected WIM scale error at a given site would be within the 95 percent tolerance limits for measurement of axle and gross vehicle weights specified in the American Society for Testing and Materials' (ASTM) E1318 (2006) specification for Type I WIM systems. Table 3.1

shows the ASTM performance requirements for Type I WIM systems. Note that the FHWA project sought to establish smoothness indices for predicting the 95th percentile error level in steering axle, tandem axle, and gross vehicle weights. Thus, wheel loads were not considered directly in this development.

Table 3.1. ASTM E1318 Functional Performance Requirements for Type I WIM Systems.

Function	Tolerance for 95% Conformance
Wheel load	±25%
Axle load	±20%
Axle-group load	±15%
Gross vehicle weight (GVW)	±10%

The FHWA project recommended two roughness indices computed from inertial profile measurements. As discussed in Chapter II, these indices were originally developed based on results from simulations of truck dynamic loading over measured road profiles on LTPP sections that included asphalt concrete, jointed plain concrete, and jointed reinforced concrete pavements, as well as asphalt overlays on concrete pavements (Karamihas and Gillespie, 2002). The first index, called the long range index or LRI, characterized the roughness for a relatively long distance ahead of the WIM sensor and a short distance beyond it, while the second index characterized the roughness directly at the WIM sensor location (short range index or SRI). Table 3.2 presents the limits over which the indices are calculated from the surface profiles, as well as the short and long wavelength cutoff values for the 4-pole Butterworth filter used in calculating the indices.

Table 3.2. Profile Interval and Filter Cutoff Values for Computing WIM Roughness Indices.

Criterion	Profile Interval (ft)		Filter Cutoff Values (ft)	
	Start[1]	End[2]	Short	Long
Long range	-84.6	10.5	3.5	37.3
Short range	-9.0	1.5	5.1	54.0

[1]Distance ahead of WIM sensor
[2]Distance beyond WIM sensor

The original project established a tolerance of about 50 inches per mile for both LRI and SRI. In a follow-up project, Karamihas and Gillespie (2004) later modified the criteria to include lower and upper threshold values and to screen for localized roughness. Table 3.3 shows the modified criteria from this follow-up project. According to Karamihas and Gillespie (2004), WIM sites with indices below the lower thresholds are very likely to perform within ASTM E1318 Type I functional performance requirements. Sites with profiles that give indices above the upper thresholds are very unlikely to perform within the ASTM requirements.

Table 3.3. Modified WIM Smoothness Criteria (Karamihas and Gillespie, 2004).

Index	Lower threshold (inches/mile)	Upper threshold (inches/mile)
LRI	31.7	133.0
SRI	31.7	133.0
Peak SRI[1]	47.5	183.7
Peak LRI[2]	31.7	133.0

[1]Refers to the highest value of SRI from 8.0 ft ahead of the scale to 4.9 ft past the scale.
[2]Refers to the highest value of LRI over the 98.4-ft interval ahead of the scale.

The WIM smoothness criteria given in Table 3.3 have been adopted in a provisional standard published by the American Association of State Highway and Transportation Officials (AASHTO MP-14, 2007). For this task, researchers made an attempt to verify the WIM smoothness criteria given in Table 3.3. Note that these criteria were developed based on results of vehicle simulations that covered a diverse range of trucks. For this verification, it would be of interest to check the criteria using actual WIM test data. However, as Karamihas and Gillespie (2004) recognized, it is impractical to perform tests that cover the same range of trucks used in the simulations made to develop the criteria. Thus, for the tests performed herein, researchers collected WIM data using a tractor-semitrailer (3S2) loaded at two different nominal gross vehicle weights of 55 and 78 kips. Figure 3.1 shows the test vehicle used for verifying the LTPP WIM smoothness criteria.

Researchers used concrete blocks to load the trailer and vary the vehicle axle loads. Once the blocks were loaded, researchers used a static scale to measure the reference loads as illustrated in Figure 3.1. For a given load level, researchers then ran the test vehicle at three different speeds over a selected TxDOT WIM site, with five repeat runs made at each test speed. Table 3.4 identifies the WIM sites that TxDOT's Transportation Planning and Programming Division recommended for verifying the WIM smoothness criteria given in Table 3.3. During the truck testing, researchers recorded the date and time of each run so that later the WIM measurements can be extracted from the WIM data file provided by TPP.

Figure 3.1. Test Vehicle Used for Verifying LTPP WIM Smoothness Criteria.

Table 3.4. TxDOT WIM Sites Tested.

TxDOT WIM site ID[1]	County	Highway	Pavement type	Sensor type
531	La Salle	I-35	CRCP[2]/HMAC[3]	Quartz
530	Archer	US277/US82	CRCP	Bending plate
528	Wilbarger	US287	CRCP	Bending plate
506	Wichita	US287	CRCP	Bending plate

[1]Four instrumented lanes per site with two sensors per lane
[2]500-ft continuously reinforced concrete pavement on northbound lanes
[3]Hot-mix asphalt concrete pavement on southbound lanes

To account for the fuel consumed during the runs, researchers measured the static axle loads four to five times during the course of testing at a given site depending on the turnaround times for the different runs.

In addition to the truck tests, researchers collected inertial profile measurements on the instrumented lanes of the 500-ft WIM section using TTI's inertial profiler. Following AASHTO

MP-14, researchers collected elevation measurements at least 400 ft prior to the WIM scale sensor and at least 100 ft beyond it. Appendix B provides plots of the profile data taken at the WIM sites shown in Table 3.4. In these plots, the WIM scale location is at 956 ft from the start of the inertial profiler runs on the given lane. Since each site has two sensors per instrumented lane, researchers defined the location of the scale to be midway between the sensors placed on each wheel path, following AASHTO MP-14. For these measurements, researchers collected profile elevations at 0.95-inch intervals, which is slightly less than the 25 mm (≈1 inch) sampling interval specified in the AASHTO MP-14 specification. No moving average filter was applied at the time of profile measurements. Researchers note that a 300 mm moving average filter is applied as a post-processing step in the procedure to calculate LRI and SRI according to Karamihas and Gillespie (2004).

Researchers later used the profile data with LTPP's WIM index software (FHWA-LTPP Technical Support Services Contractor, 2005) to compute and verify the WIM smoothness criteria presented previously. Tables 3.5 and 3.6 show the WIM smoothness indices computed from the profile data taken at TxDOT's WIM site on I-35 south of Cotulla in La Salle County. These indices are based on profiles for the southbound travel lanes where the WIM sensors are on flexible pavement. On the northbound travel lanes, the sensors are on a 500-ft segment of the 1-mile 9-inch CRCP whitetopping placed on those lanes. Appendix B provides tables of the WIM smoothness indices computed from profiles measured at the TxDOT WIM sites.

In Tables 3.5 and 3.6, indices that are above the corresponding lower thresholds but below the corresponding upper thresholds are shaded grey. These cases fall within the undetermined or "grey" region of the LTPP WIM smoothness criteria. Note the wide gap between the lower and upper thresholds of the indices given in Table 3.3. From the experience with verifying the LTPP WIM smoothness criteria in this project, most of the computed indices fall within the grey region of the requirements given in Table 3.3. This observation is seen in the tabulations of computed WIM smoothness indices presented in Appendix B where shaded cells show indices falling within the grey region. The other observation noted from Tables 3.5 and 3.6 is that not all of the smoothness criteria are satisfied for both wheel paths of the instrumented lanes. Researchers made this same observation for all four WIM sites where verification data were collected. Thus, the main finding from this investigation is that the LTPP WIM smoothness criteria generally tend to produce inconclusive determinations of whether a WIM installation classifies as Type I or not for the WIM sites tested. Researchers found that in most cases, the computed indices fall within the undetermined region of the proposed criteria. Moreover, it was difficult to find locations where all of the criteria are met on both wheel paths.

Table 3.5. LTPP WIM Smoothness Indices for I-35 Southbound Outside Lane.

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	39.296	34.768	40.470	38.178
	Peak LRI	39.296	39.929	40.470	39.898
	SRI	21.035	20.168	17.898	19.700
	Peak SRI	27.141	34.282	43.853	35.092
Right	LRI	31.158	32.346	31.575	31.693
	Peak LRI	31.643	32.556	33.623	32.607
	SRI	27.141	26.410	19.907	24.486
	Peak SRI	28.520	29.163	23.514	27.066

Table 3.6. LTPP WIM Smoothness Indices for I-35 Southbound Inside Lane.

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	51.378	56.584	53.384	53.782
	Peak LRI	54.571	66.396	59.864	60.277
	SRI	31.801	27.266	20.796	26.621
	Peak SRI	49.675	52.288	49.309	50.424
Right	LRI	44.708	41.784	42.492	42.995
	Peak LRI	44.738	41.817	42.557	43.037
	SRI	42.125	45.98	44.24	44.115
	Peak SRI	55.515	57.698	52.979	55.397

Test data from two other evaluations also show a general tendency for the computed WIM smoothness indices to be within the lower and upper thresholds of the AASHTO MP-14 WIM smoothness criteria for Type I WIM systems. MACTEC (2004) reported results from a field evaluation of the smoothness criteria on four WIM sites (three in Texas and one in Ohio). The field tests in Texas made use of two tractor-semitrailers (3S2s) and two 3-axle straight trucks, while the tests in Ohio included a third 3S2. The MACTEC study found that most of the computed LRIs and SRIs on the lanes tested fell between the lower and upper limits of the corresponding thresholds for these statistics. Thus, based on the data collected, none of the proposed statistics were found to be definitive predictors of the effects of pavement smoothness on WIM scale performance. The criteria could not conclusively determine whether a WIM system is Type I or not. In terms of the actual differences between the WIM weights measured on the test vehicles and the corresponding static (reference) weights, MACTEC examined the differences for each instrumented lane. Of the 13 lanes tested, only one classified as a Type I system based on comparing the 95 percent confidence intervals of the WIM weight errors against the ASTM E1318 functional requirements for steering, tandem axle, and gross vehicle weights given in Table 3.1. However, if the possible bias in the WIM data due to calibration issues is considered, two more lanes were found to classify as Type I bringing the total number to three. The MACTEC report recommended that additional tests be performed on pavements exceeding the upper thresholds as well as on pavements falling below the lower thresholds to substantiate the proposed WIM smoothness criteria.

In another contract, MACTEC (2006) evaluated the performance of a WIM system installed on a Texas LTPP SPS-1 section (480100) located along US281 near Edinburg, Texas. This section, located along the southbound outside lane of US281, is instrumented with bending plates installed on a 500-ft continuously reinforced concrete slab. The field evaluation used three test trucks, two 3S2s and one 3S3, which were driven along the LTPP section at test speeds ranging from 49 to 72 miles per hour. The 3S2s were loaded to cover heavy (78,200 lb GVW) and light (56,500 lb GVW) conditions, while the 3S3 was loaded to a GVW of 75,900 lb. Based on the 95 percent confidence intervals of the differences between the measured WIM and static (reference) weights, MACTEC found that the LTPP WIM section met the ASTM E1318 Type I functional requirements for steering, axle group, and gross vehicle weights. MACTEC did not provide comparisons between measured WIM and static wheel loads since LTPP does not validate WIM performance with respect to wheel load.

MACTEC also collected profile measurements on the LTPP SPS-1 section and used the data to assess the expected WIM scale performance based on the provisional AASHTO MP-14 WIM smoothness criteria. Considering the computed LRIs, SRIs, peak LRIs, and peak SRIs, it was not possible to conclusively determine whether the LTPP WIM section classifies as Type I or not. All indices fell between the corresponding lower and upper thresholds specified in the provisional AASHTO MP-14 specification.

As part of verifying the LTPP WIM smoothness criteria in this TxDOT project, TTI researchers compared the WIM readings with corresponding measured static weights from the truck runs made at the Cotulla WIM site. Since static weight measurements were made at different times during the course of verification testing at a given site, researchers compared the WIM measurements to the corresponding static weight measurements that matched closest in time to the WIM data. This comparison followed the procedure given in ASTM E1318 for calculating the percent of nonconformance. Specifically, the percent difference d in corresponding WIM and reference values for a given functional item is determined as follows:

$$d = \frac{100 \times (C - R)}{R} \qquad (3.1)$$

where,

C = WIM reading for a given functional item, and

R = corresponding reference value for the same functional item.

Then, the number of calculated differences P_{de} exceeding the specified tolerance shown in Table 3.1 for the given functional item is determined and expressed as a percent of the total number of observed values according to the following equation:

$$P_{de} = \frac{100 \times n}{N} \qquad (3.2)$$

where,

n = number of calculated differences that exceed the specified tolerance value for the given functional item, and

N = total number of observed values for the same functional item.

The ASTM specification requires that no more than five percent of the calculated differences exceed the specified tolerance shown in Table 3.1 for the given functional item. Table 3.7 shows the results from the above calculations for the flexible pavement WIM sections on I-35 south of Cotulla. Based on the data from the truck runs, it is observed that the WIM sections classify as Type I. However, the calculated smoothness indices shown in Tables 3.5 and 3.6 cannot definitively classify these lanes as Type I.

Appendix B provides tables of the calculated differences for all WIM sites tested while Table 3.8 provides a summary of the classification results based on the truck tests conducted in this project. Researchers observed that TxDOT WIM site 531 passed the Type I ASTM E1318 requirements for steering, tandem axle, and gross vehicle weights on all four lanes, while on TxDOT WIM site 530, all lanes failed to classify as Type I. On WIM sites 528 and 506, all lanes met the Type I requirements except for the northbound outside lanes. Once more, the provisional AASHTO MP-14 smoothness criteria could not conclusively determine whether a given lane classifies as Type I or not on all lanes tested.

Table 3.7. Cotulla WIM Site (531) Classification Using Type I Criteria.

Lane	Functional Item	Tolerance for 95% Conformance	P_{de} %[1]	Number of observations	Result
Southbound inside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	1.67	60	Passes Type I
	GVW	±10%	0.00	30	Passes Type I
Southbound outside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	0.00	60	Passes Type I
	GVW	±10%	0.00	30	Passes Type I

[1]P_{de} should not exceed 5%.
[2]Drive and trailer tandem axles were combined in determining the result shown.

Table 3.8. WIM Site Classifications Based on Truck Verification Tests.

TxDOT WIM Site	Lane	Functional Item	Result
531	Southbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Southbound inside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Northbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Northbound inside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
530[2]	Southbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Fails Type I
		GVW	Fails Type I
	Southbound inside lane	Steering axle weight	Fails Type I
		Tandem axle weight[1]	Fails Type I
		GVW	Fails Type I
	Northbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Fails Type I
		GVW	Fails Type I

[1]Drive and trailer tandem axles were combined in determining the result shown.
[2]Northbound inside lane found inoperative due to damaged cables.

Table 3.8. WIM Site Classifications Based on Truck Verification Tests (continued).

TxDOT WIM Site	Lane	Functional Item	Result
528	Southbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Southbound inside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Northbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Fails Type I
		GVW	Fails Type I
	Northbound inside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
506	Southbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Southbound inside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I
	Northbound outside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Fails Type I
	Northbound inside lane	Steering axle weight	Passes Type I
		Tandem axle weight[1]	Passes Type I
		GVW	Passes Type I

[1]Drive and trailer tandem axles were combined in determining the result shown.

In an internal TxDOT study, Chen and Hong (2008) evaluated the WIM smoothness criteria based on calibration records provided by TPP for 19 TxDOT WIM sites. They collected profile data on these 19 sites and assessed the proposed criteria based on whether a given site passed calibration or not. Chen and Hong (2008) noted that the WIM scales are calibrated by TxDOT using a typical tractor-semitrailer loaded to approximately 80,000 lb GVW. In these field calibrations, four repeat runs of the calibration vehicle are made at a test speed based on the posted speed limit. The WIM scale is regarded as passing when the difference between the WIM and static gross vehicle weights is within three percent of the static GVW.

From their study, Chen and Hong (2008) concluded that the only critical WIM criterion is the long-range roughness index. Based on examining the distributions of the WIM indices computed from the inertial profile measurements, they proposed a threshold value of 110 inches/mile on LRI for the 85-ft distance leading up to the WIM scale. Researchers note that this conclusion is based on the calibration records of the WIM sites Chen and Hong investigated and, specifically, on the GVW measurement error. No actual truck tests were conducted on the selected WIM sites as was done in this research project.

If the proposed LRI criterion of 110 inches/mile is used on the WIM sites tested in this project, the lanes that failed the ASTM E1318 Type I requirements in Table 3.8 will go undetected. In fact, all of the lanes meet this LRI criterion, which appears to be too lenient based on the LRIs computed from the profiles collected at the four WIM sites identified in Table 3.8. It is noted that these four sites are included among the 19 weigh-in-motion sites that passed calibration in the study conducted by Chen and Hong (2008). In the researchers' opinion, more verification tests are needed to establish WIM smoothness criteria that better differentiate between Type I and Type II WIM systems based on ASTM E1318 functional performance requirements.

LIMITED INVESTIGATION OF USING THE FWD TO TEST WIM SENSORS

During the course of testing WIM sites to establish criteria for identifying candidate flexible pavement WIM sections, researchers initiated a small study to determine if the FWD can be used to check WIM sensors. The FWD load plate has a rubber pad, and the applied load is a close simulation of the load imparted by a truck to the highway. The load pad could simply be placed over the WIM sensor, and FWD drops at a given height could be made to check the loads from the FWD load cell against the WIM station output. Ten FWD drops can be applied in about two to three minutes. If the findings look promising, this application of the FWD could be a highly efficient alternative method for a district to verify if the WIM system is operating normally. Researchers recognize that the FWD will not replace tests with loaded trucks for calibrating WIM systems as these truck tests incorporate the interactions between road smoothness, and vehicle mass, geometric, and suspension characteristics. However, the FWD might be used, at the very least, to determine if there is a problem with the WIM hardware.

Initial tests were conducted on the flexible pavement WIM site south of Cotulla. Researchers conducted these tests during one of the planned FWD visits to monitor the flexible pavement WIM sections during this project. Figure 3.2 shows a picture of the FWD being used to test the right wheel path WIM sensor on the southbound outside lane. Prior to testing, researchers placed a call to TPP to synchronize the clock of the FWD computer with the internal clock of the WIM system at the site so that the WIM data can be requested for the specific period during which the FWD tests on the sensor were conducted.

Figure 3.2. FWD WIM Sensor Testing.

However, after the tests were conducted, and after receiving the WIM data file from TPP, researchers found that the data from the FWD tests were not recorded by the WIM system. After discussions between researchers, it was determined that the problem was due to the loop detector timing out during the test. In other words, the loop detector has to be triggered before the WIM system begins to monitor wheel loads. Thus, researchers identified the need to set up the test such that the WIM system can manually be triggered to collect data for each FWD drop.

The research team conducted subsequent tests on the SH6 WIM test bed operated by TTI. This test bed is located near the junction of FM60 and the SH6 bypass in College Station, Texas. For these tests, TTI staff wired a loop detector with the WIM control box to enable the operator to manually activate the system just prior to the FWD drop. This setup proved successful, but if this process of using the FWD for WIM calibration proves to be promising, WIM manufacturers should add a feature that allows these tests with a simple switching mechanism. Figure 3.3

shows the data collected from these tests. In this figure, the WIM readings shown are from the right wheel path Kistler quartz WIM sensor placed on the southbound outside lane of the SH6 WIM test bed.

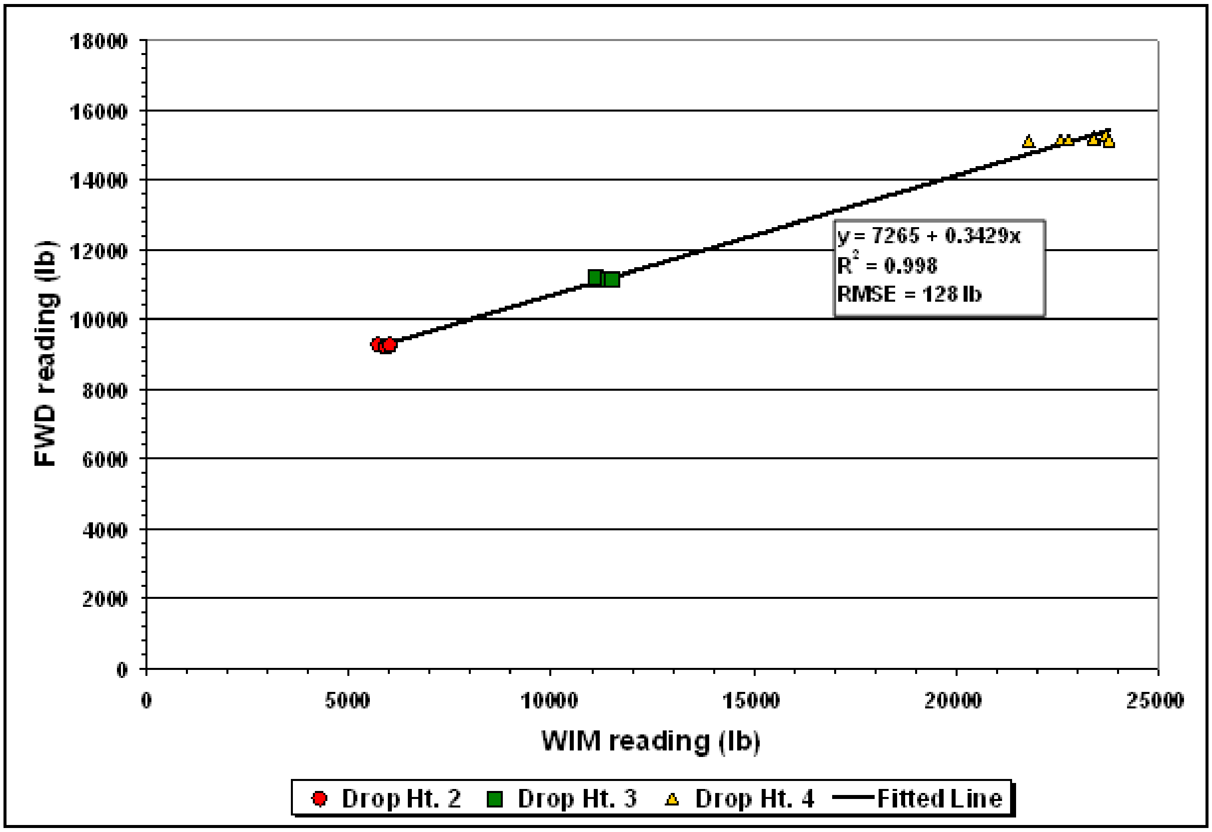


Figure 3.3. Comparison of FWD and WIM Load Readings from SH6 WIM Test Bed.

Researchers note that prior to the FWD tests on the SH6 site, an electronics board in the control box had to be replaced because of a power surge. Since the system had not undergone calibration with TPP's calibration truck, this circumstance provided researchers with the opportunity to play with the WIM calibration settings during the FWD tests. It is unlikely that these settings would have been changed had the calibration with the reference truck been made before the FWD tests.

Researchers initially calibrated the WIM system with the FWD load cell readings corresponding to drop height #3. This calibration involved a trial and error process since the WIM software installed with the system is not set up to accept input of reference loads. Instead, the operator has to manually vary the calibration settings until an acceptable agreement between the WIM and FWD load readings are obtained. Once this calibration was completed, data were then collected at two other drop heights.

Figure 3.3 shows a strong linear relationship between the FWD and WIM load readings from the tests conducted on SH6. Thus, one possible application of the FWD might be as a tool for checking the linearity of the WIM response to applied loads. In practice, one could run the FWD at different drop heights and compare the FWD load cell readings with the corresponding WIM readings for the current WIM calibration factors. Note that these factors would not be changed during the FWD test. The objective is simply to check the linearity of the WIM system response. It would not be appropriate anyway to calibrate based totally on the FWD because this calibration would not capture the effects of interactions between road smoothness and vehicle characteristics. Again, the FWD might serve as an interim calibration tool until the TPP truck could fully calibrate the WIM system.

Another possible application of the FWD might be to check the consistency in the WIM readings from repeat measurements at a given drop height. This check would be harder to perform with a calibration truck because factors such as driver variability and the interactions between road smoothness and vehicle characteristics can affect the variability in the WIM measurements. The FWD permits one to conduct a more controlled experiment to check the consistency of the WIM readings. Figure 3.3 shows that the WIM readings from the SH6 tests are fairly consistent. This consistency is indicated in the standard deviations and coefficients of variation of the WIM measurements that are given in Table 3.9.

Table 3.9. Consistency of WIM Measurements at Various FWD Drop Heights.

Drop Height No.	Average WIM Load (lb)	Standard Deviation of WIM Loads (lb)	Coefficient of Variation (percent)
2	5,850	131	2.24
3	11,357	181	1.60
4	23,038	667	2.90

In summary, this limited investigation has identified a couple of applications by which the FWD might be put to use. However, more work is needed, in the researchers' opinion, before any definitive conclusions can be made for using the FWD to perform checks on existing weigh-in-motion systems. In particular, more FWD tests are needed to cover the types of WIM systems deployed by TxDOT on both concrete and flexible pavements. Additionally, communications with WIM manufacturers and vendors is recommended to provide hardware and software modifications that permit the FWD to be more readily used for checking existing WIM systems.

ESTABLISHING ACCEPTABLE PAVEMENT CRITERIA FOR WIM SITES

TxDOT's WIM site on I-35 south of Cotulla was already operational at the time this project began in December 2006. This site afforded researchers the opportunity to study the performance of a flexible pavement WIM site instrumented with the latest generation of quartz sensors that are reported to have improved temperature stability than earlier piezoelectric WIM systems. Thus, researchers conducted tests on this site during the course of the project as part of establishing criteria for identifying candidate flexible pavement WIM sites. In this regard, one factor researchers considered is the uniformity of the pavement support, particularly on flexible pavements with thick HMAC layers such as the full-depth perpetual pavement built at the Cotulla WIM site. HMAC stiffness can vary significantly with pavement temperature. Thus, the variations in mix stiffness with pavement temperature and the consequent changes in pavement support conditions need to be considered in establishing acceptable pavement criteria for flexible pavement WIM installations.

The flexible pavement WIM site in Cotulla provided an opportunity to check the uniformity in support conditions on a full-depth HMAC perpetual pavement section. The pavement cross-section at the site consists of 17 inches of structural mixes over an eight-inch cement-stabilized subgrade. Researchers note that, for flexible pavement installations, TPP needs a minimum four inches of HMAC thickness to embed quartz WIM sensors into the pavement. The asphalt mat on the Cotulla project certainly provided ample thickness for installation of these sensors. Construction on this perpetual pavement project was completed in the summer of 2005. During and after construction, TTI researchers conducted tests at the site as part of TxDOT Project 0-4822. Researchers reviewed information from field and laboratory evaluations reported by Scullion (2006) on Project 0-4822. FWD data taken on this project in the summer of 2005 showed sensor 1 deflections ranging from 4.41 to 5.96 mils with a mean deflection of 5.11 mils and a standard deviation of 0.40 mils. These measurements were taken over the four-mile length of the perpetual pavement project and show that the pavement tested is very stiff, especially when one considers that the data were collected during the summer where the average temperature of the mat was over 100 °F. Scullion (2006) noted that on this and other perpetual pavement projects he tested, the measured deflections were observed to be similar to those obtained on thick concrete pavements, which is the pavement preferred by TPP for installation of WIM sensors. Researchers also saw this same observation from FWD test data collected during this project on the 500-ft perpetual pavement WIM sections established in Cotulla. Tables 3.10 and 3.11 show the FWD deflections measured along these sections in July 2008. On the outside lane (L1), the sensor 1 deflections under drop height 2 range from 4 to 5 mils over a range of pavement temperatures from 86 to 113 °F during testing. On the inside lane (L2), the sensor 1 deflections range from 3.8 to 6.6 mils over a range of pavement temperatures from 114 to 130 °F. Given these measurements, it does not appear that the uniformity and degree of pavement support under the WIM sensors would be an issue at the Cotulla WIM site.

To check the consistency of the readings from the WIM sensors, TPP conducted tests at the site in April 2007 with its calibration truck. During these tests, TPP made multiple runs on the WIM sensors. TPP staff then compared the gross vehicle weight measurements from the WIM with the reference GVW of the calibration truck. The results of these comparisons are given in Table 3.12, which shows that the errors (as percentages of the reference GVW) are all within the 10 percent tolerance on GVW specified in ASTM E1318 for Type I WIM systems. Table 3.12 shows that pavement temperatures during the tests ranged from 79 to 127 °F. Based on these results, there does not appear to be a discernible effect of pavement temperature on the readings obtained from the quartz WIM sensors placed. This observation is clearly reflected in Figure 3.4, which shows that the percent errors in GVW measurements exhibit no significant correlation with pavement temperature.

In addition to the uniformity in pavement support, researchers also considered the uniformity in subsurface conditions to assess acceptable pavement criteria for candidate WIM sites. Scullion (2006) used the GPR to assess the uniformity in subsurface conditions on a number of perpetual pavement projects. He explained that the presence of strong reflections in intermediate layers of the asphalt mat might indicate pockets of segregation during HMAC placement that exhibit positive or negative reflections depending on whether or not moisture is trapped within the voids. Figure 3.5 presents an example of GPR data that Scullion (2006) considered as ideal. The data shown were taken along another perpetual project on I-35 in the Laredo District that was completed in early 2003. Note that no strong intermediate reflections are observed within the asphalt mat. Scullion (2006) noted that all cores taken from this project were solid, i.e., the cores did not break during coring.

Table 3.10. FWD Deflections on 500-ft WIM Section along I-35 Southbound Outside Lane.

Station (ft)	Load (lb)	R1 (mils)	R2 (mils)	R3 (mils)	R4 (mils)	R5 (mils)	R6 (mils)	R7 (mils)	SCI (mils)	BCI (mils)	Pavement Temp. (°F)
0	10519	4.00	2.17	1.48	1.05	0.83	0.67	0.56	1.83	0.69	86
10	10499	4.17	2.43	1.74	1.26	1.00	0.79	0.69	1.74	0.69	86
20	10491	4.47	2.76	1.96	1.42	1.07	0.83	0.67	1.71	0.80	86
30	10495	4.37	2.55	1.76	1.24	0.91	0.77	0.63	1.82	0.79	86
41	10456	4.40	2.57	1.83	1.27	0.98	0.77	0.64	1.83	0.74	86
50	10436	4.22	2.53	1.75	1.24	0.90	0.70	0.58	1.69	0.78	92
60	10531	4.35	2.70	1.88	1.30	1.00	0.78	0.66	1.65	0.82	92
72	10480	4.51	2.87	2.10	1.53	1.20	0.98	0.82	1.64	0.77	92
80	10511	4.71	3.07	2.26	1.67	1.30	1.04	0.87	1.64	0.81	92
90	10460	5.00	3.28	2.39	1.76	1.37	1.07	0.89	1.72	0.89	92
100	10491	4.98	3.31	2.46	1.77	1.39	1.08	0.86	1.67	0.85	95
110	10484	4.85	3.24	2.41	1.74	1.37	1.09	0.89	1.61	0.83	95
120	10487	4.74	3.07	2.29	1.74	1.37	1.10	0.89	1.67	0.78	95
130	10464	4.57	2.81	2.08	1.58	1.27	1.03	0.82	1.76	0.73	95
140	10487	4.44	2.59	1.88	1.37	1.09	0.84	0.71	1.85	0.71	95
150	10444	4.34	2.67	1.96	1.42	1.10	0.87	0.70	1.67	0.71	99
160	10428	4.02	2.46	1.81	1.31	1.04	0.81	0.67	1.56	0.65	99
170	10468	4.34	2.52	1.83	1.34	1.05	0.83	0.68	1.82	0.69	99
180	10408	4.55	2.81	2.05	1.43	1.07	0.82	0.63	1.74	0.76	99
190	10503	4.19	2.54	1.84	1.27	0.94	0.70	0.58	1.65	0.70	99
200	10468	4.62	2.93	2.24	1.69	1.34	1.05	0.87	1.69	0.69	103
210	10408	5.02	3.34	2.57	1.88	1.48	1.17	0.97	1.68	0.77	103
221	10436	5.03	3.32	2.56	1.96	1.55	1.20	1.04	1.71	0.76	103
230	10460	4.80	3.06	2.36	1.74	1.44	1.13	0.93	1.74	0.70	103
240	10420	4.91	3.11	2.32	1.68	1.30	1.01	0.79	1.80	0.79	103
252	10400	5.02	3.07	2.27	1.68	1.30	1.05	0.85	1.95	0.80	105
260	10444	4.84	2.89	2.12	1.56	1.22	0.98	0.81	1.95	0.77	105
270	10468	4.65	2.94	2.13	1.58	1.24	0.97	0.83	1.71	0.81	105
280	10456	4.91	2.92	2.14	1.62	1.32	1.01	0.87	1.99	0.78	105
290	10464	4.91	3.00	2.28	1.70	1.40	1.12	0.93	1.91	0.72	105
300	10472	4.81	2.88	2.16	1.65	1.38	1.11	0.96	1.93	0.72	106
310	10444	4.67	3.02	2.36	1.80	1.46	1.19	0.99	1.65	0.66	106
320	10444	4.96	3.11	2.36	1.78	1.44	1.17	0.98	1.85	0.75	106
330	10428	4.64	2.91	2.20	1.69	1.39	1.15	0.97	1.73	0.71	106
340	10424	4.54	2.89	2.18	1.65	1.36	1.09	0.93	1.65	0.71	106
350	10420	4.91	3.10	2.35	1.79	1.52	1.19	0.98	1.81	0.75	109
360	10464	4.64	2.67	2.01	1.56	1.30	1.06	0.89	1.97	0.66	109
370	10440	4.20	2.76	2.13	1.63	1.33	1.08	0.89	1.44	0.63	109
380	10412	4.72	3.03	2.30	1.78	1.44	1.15	0.99	1.69	0.73	109

Table 3.10. FWD Deflections on 500-ft WIM Section along I-35 Southbound Outside Lane (continued).

Station (ft)	Load (lb)	R1 (mils)	R2 (mils)	R3 (mils)	R4 (mils)	R5 (mils)	R6 (mils)	R7 (mils)	SCI (mils)	BCI (mils)	Pavement Temp. (°F)
390	10432	4.55	2.93	2.28	1.75	1.41	1.16	0.96	1.62	0.65	109
400	10464	4.45	2.85	2.21	1.70	1.40	1.09	0.90	1.60	0.64	112
410	10396	4.49	2.94	2.25	1.69	1.37	1.07	0.90	1.55	0.69	112
420	10424	4.52	2.82	2.14	1.63	1.31	1.07	0.91	1.70	0.68	112
430	10416	4.40	2.78	2.09	1.61	1.31	1.05	0.90	1.62	0.69	112
440	10380	4.68	3.03	2.31	1.77	1.43	1.17	0.99	1.65	0.72	112
450	10400	4.72	3.04	2.29	1.74	1.40	1.12	0.94	1.68	0.75	113
460	10436	4.37	2.89	2.22	1.68	1.37	1.11	0.94	1.48	0.67	113
470	10376	4.62	2.98	2.30	1.80	1.44	1.16	0.98	1.64	0.68	113
480	10293	4.62	3.11	2.41	1.81	1.48	1.19	1.00	1.51	0.70	113
490	10372	4.75	3.12	2.41	1.85	1.56	1.23	1.04	1.63	0.71	113
500	10344	5.01	3.28	2.47	1.92	1.56	1.28	1.06	1.73	0.81	112
	Minimum	4.00	2.17	1.48	1.05	0.83	0.67	0.56	1.44	0.63	
	Maximum	5.03	3.34	2.57	1.96	1.56	1.28	1.06	1.99	0.89	
	Average	4.61	2.90	2.16	1.61	1.28	1.02	0.85	1.72	0.74	
	Std. dev.	0.27	0.26	0.24	0.21	0.19	0.16	0.14	0.12	0.06	
	CV%	5.87	8.84	10.96	12.87	14.78	15.40	16.04	7.20	7.99	

Table 3.11. FWD Deflections on 500-ft WIM Section along I-35 Southbound Inside Lane.

Station (ft)	Load (lb)	R1 (mils)	R2 (mils)	R3 (mils)	R4 (mils)	R5 (mils)	R6 (mils)	R7 (mils)	SCI (mils)	BCI (mils)	Pavement Temp. (°F)
0	10329	5.11	2.73	2.07	1.48	1.17	0.92	0.75	2.38	0.66	114
10	10372	5.15	2.61	2.03	1.51	1.20	0.92	0.70	2.54	0.58	114
20	10388	4.93	2.57	1.91	1.34	1.08	0.80	0.64	2.36	0.66	114
30	10376	4.97	2.64	2.02	1.50	1.18	0.89	0.69	2.33	0.62	114
40	10368	5.02	2.70	2.12	1.60	1.24	0.98	0.78	2.32	0.58	114
50	10356	5.42	2.89	2.29	1.70	1.33	1.04	0.82	2.53	0.60	117
60	10388	5.30	2.77	2.00	1.39	1.07	0.80	0.66	2.53	0.77	117
70	10428	4.24	2.28	1.71	1.23	0.96	0.75	0.63	1.96	0.57	117
80	10341	4.26	2.28	1.68	1.23	0.94	0.72	0.61	1.98	0.60	117
90	10376	4.36	2.13	1.54	1.13	0.87	0.67	0.53	2.23	0.59	117
100	10416	3.83	1.89	1.40	0.96	0.74	0.55	0.47	1.94	0.49	124
110	10396	3.93	1.93	1.41	1.01	0.79	0.63	0.51	2.00	0.52	124
120	10428	4.00	2.09	1.57	1.15	0.89	0.70	0.57	1.91	0.52	124
130	10392	4.09	2.20	1.65	1.18	0.89	0.68	0.57	1.89	0.55	124
140	10396	4.23	2.15	1.55	1.08	0.83	0.63	0.50	2.08	0.60	124
150	10344	4.09	2.02	1.45	1.01	0.76	0.58	0.48	2.07	0.57	125
160	10428	3.98	2.01	1.43	0.97	0.72	0.55	0.44	1.97	0.58	125
171	10428	4.17	2.11	1.55	1.10	0.86	0.65	0.54	2.06	0.56	125
180	10408	4.47	2.33	1.77	1.26	1.02	0.73	0.59	2.14	0.56	125
190	10396	4.35	2.20	1.65	1.18	0.95	0.72	0.60	2.15	0.55	125

Table 3.11. FWD Deflections on 500-ft WIM Section along I-35 Southbound Inside Lane (continued).

Station (ft)	Load (lb)	R1 (mils)	R2 (mils)	R3 (mils)	R4 (mils)	R5 (mils)	R6 (mils)	R7 (mils)	SCI (mils)	BCI (mils)	Pavement Temp. (°F)
200	10392	4.88	2.49	1.90	1.39	1.11	0.87	0.70	2.39	0.59	126
210	10356	4.81	2.52	1.94	1.43	1.12	0.92	0.76	2.29	0.58	126
220	10364	4.94	2.64	2.06	1.56	1.25	1.02	0.84	2.30	0.58	126
230	10372	4.85	2.59	2.00	1.53	1.24	0.99	0.82	2.26	0.59	126
240	10388	4.74	2.61	2.07	1.55	1.21	0.94	0.76	2.13	0.54	126
250	10372	5.01	2.67	2.11	1.60	1.21	0.95	0.80	2.34	0.56	128
260	10360	5.18	2.82	2.20	1.61	1.26	0.96	0.78	2.36	0.62	128
270	10309	5.31	3.13	2.46	1.86	1.46	1.14	0.91	2.18	0.67	128
280	10364	5.15	3.11	2.46	1.90	1.51	1.17	0.96	2.04	0.65	128
290	10364	5.40	3.19	2.60	2.03	1.65	1.31	1.05	2.21	0.59	128
300	10305	5.48	3.44	2.76	2.13	1.73	1.38	1.10	2.04	0.68	129
310	10364	6.03	3.61	2.89	2.23	1.76	1.39	1.15	2.42	0.72	129
320	10297	6.25	3.93	3.17	2.41	1.94	1.53	1.22	2.32	0.76	129
330	10261	6.31	3.89	3.15	2.43	1.96	1.54	1.25	2.42	0.74	129
340	10321	6.39	3.93	3.11	2.45	1.93	1.55	1.26	2.46	0.82	129
350	10297	6.60	3.93	3.21	2.53	2.04	1.64	1.31	2.67	0.72	130
360	10301	6.39	3.94	3.15	2.50	2.01	1.63	1.31	2.45	0.79	130
370	10317	6.15	3.92	3.15	2.46	2.02	1.62	1.33	2.23	0.77	130
381	10297	6.36	3.83	3.12	2.39	1.94	1.52	1.26	2.53	0.71	130
390	10364	6.10	3.76	3.00	2.32	1.85	1.43	1.19	2.34	0.76	130
400	10321	5.22	3.22	2.65	2.09	1.73	1.39	1.11	2.00	0.57	129
410	10325	5.31	3.15	2.60	2.05	1.68	1.37	1.14	2.16	0.55	129
420	10305	5.42	3.21	2.65	2.12	1.75	1.40	1.14	2.21	0.56	129
430	10301	5.20	3.20	2.60	2.07	1.71	1.35	1.10	2.00	0.60	129
440	10333	5.20	3.13	2.51	1.98	1.63	1.33	1.10	2.07	0.62	129
450	10325	5.24	3.03	2.52	1.98	1.63	1.30	1.10	2.21	0.51	128
460	10273	5.37	3.05	2.50	1.93	1.60	1.30	1.03	2.32	0.55	128
470	10257	5.69	3.22	2.63	2.02	1.65	1.32	1.11	2.47	0.59	128
480	10293	5.55	3.04	2.55	2.04	1.67	1.35	1.11	2.51	0.49	128
490	10337	5.64	3.44	2.84	2.27	1.86	1.51	1.24	2.20	0.60	128
500	10273	6.08	3.42	2.81	2.25	1.82	1.46	1.23	2.66	0.61	130
	Minimum	3.83	1.89	1.40	0.96	0.72	0.55	0.44	1.89	0.49	
	Maximum	6.60	3.94	3.21	2.53	2.04	1.64	1.33	2.67	0.82	
	Average	5.14	2.89	2.28	1.73	1.38	1.09	0.89	2.25	0.62	
	Std. dev.	0.75	0.62	0.56	0.49	0.41	0.34	0.28	0.20	0.08	
	CV%	14.51	21.30	24.61	28.13	29.78	31.39	31.47	9.04	13.25	

Table 3.12. Results from TPP Calibration Truck Tests on Cotulla Perpetual Pavement WIM Sections.

Lane	Time	Pavement Temperature (°F)	GVW (kips)	Absolute % Error[1]
L1	9:11	79	77.2	0.77
L1	9:25	81	75.3	3.21
L1	9:41	84	75.4	3.08
L1	9:56	88	76.0	2.31
L1	10:11	90	74.0	4.88
L1	10:26	93	76.1	2.19
L2	10:41	95	78.5	0.90
L2	10:56	96	77.9	0.13
L2	11:11	103	79.1	1.67
L2	11:26	105	76.9	1.16
L2	11:46	107	75.0	3.60
L1	12:02	109	75.9	2.44
L1	12:27	112	78.6	1.03
L1	12:46	117	75.2	3.34
L1	3:12	121	76.0	2.31
L1	3:27	127	75.6	2.83
L1	3:42	125	76.7	1.41
L1	3:56	122	73.7	5.27

[1]GVW of TPP calibration truck is 77.8 kips.

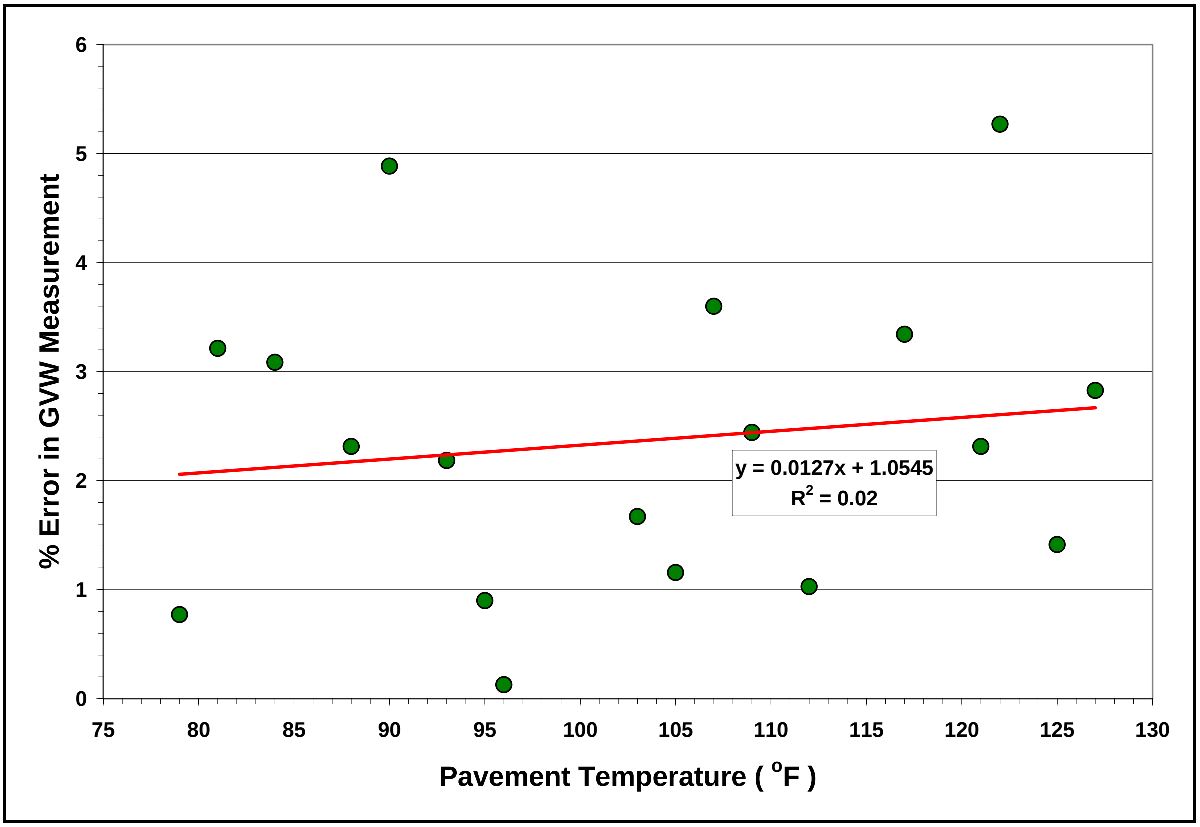


Figure 3.4. Correlation of Error in GVW Measurement with Pavement Temperature.

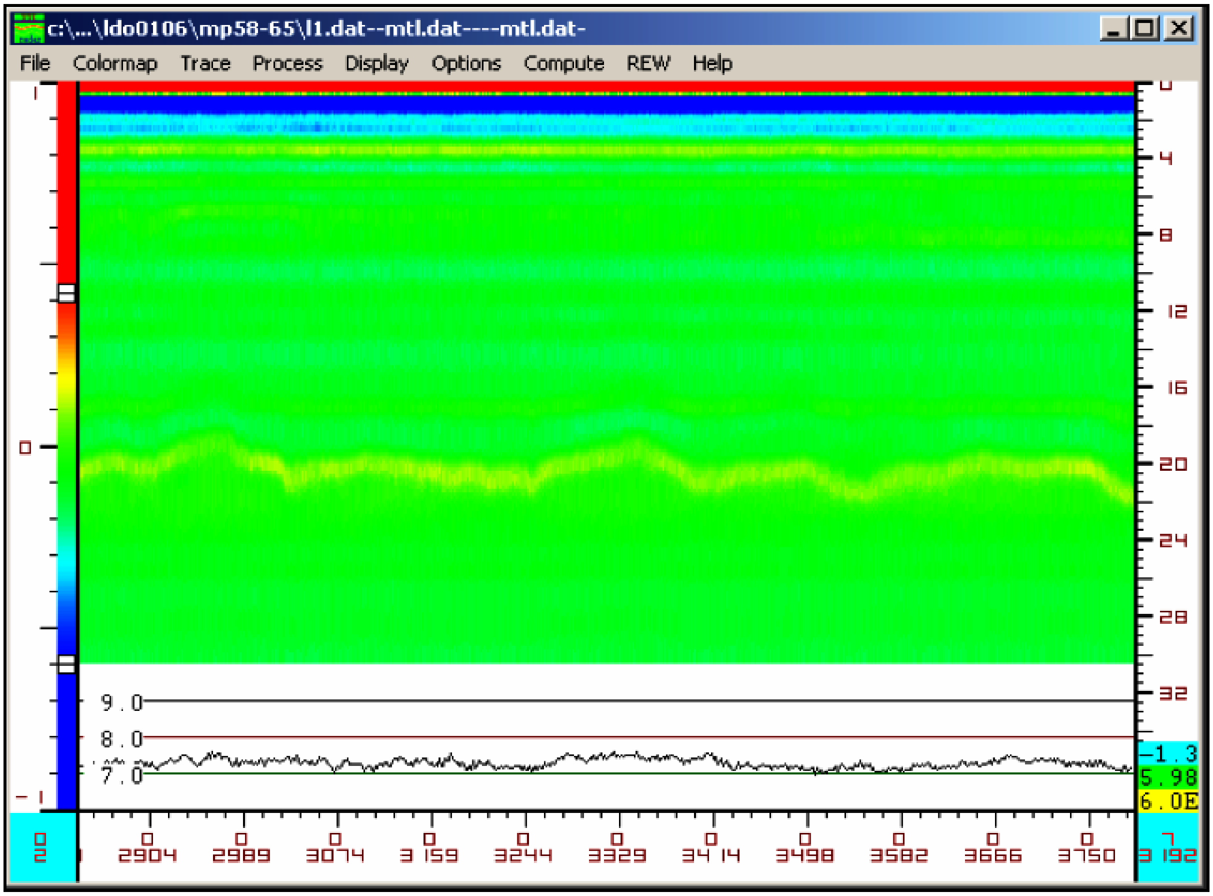


Figure 3.5. GPR Data from I-35 Perpetual Pavement Project (CSJ 0018-01-063).

Figure 3.6 shows GPR data taken along the perpetual pavement project where the Cotulla WIM site was established. It is observed that the GPR data look clean, with no major intermediate reflections coming from within the asphalt mat. The surface dielectric plot at the bottom of the figure also looks uniform, with no significant peaks or dips that would indicate wet or low density areas, respectively. The only defect Scullion (2006) noted on this project is that the cores all debonded in the middle of the one-inch (nominal aggregate size) stone-filled layer. This debonding appears to be due to insufficient tack coat placed between compacted lifts.

In addition to the GPR data, researchers also examined results obtained from Hamburg and overlay tests completed in Project 0-4822 on HMAC samples taken from the Cotulla WIM project. The Hamburg test is described in TxDOT Test Method Tex-242-F, which is normally specified in the plans for a given construction or rehabilitation project. For mix design and acceptance purposes, TxDOT normally requires less than 0.5-inch rutting after 20,000 passes of the 158-lb wheel load at the standard test temperature of 122 °F that HMAC samples are tested with the Hamburg. This test provides a measure of the rutting resistance of the asphalt mix.

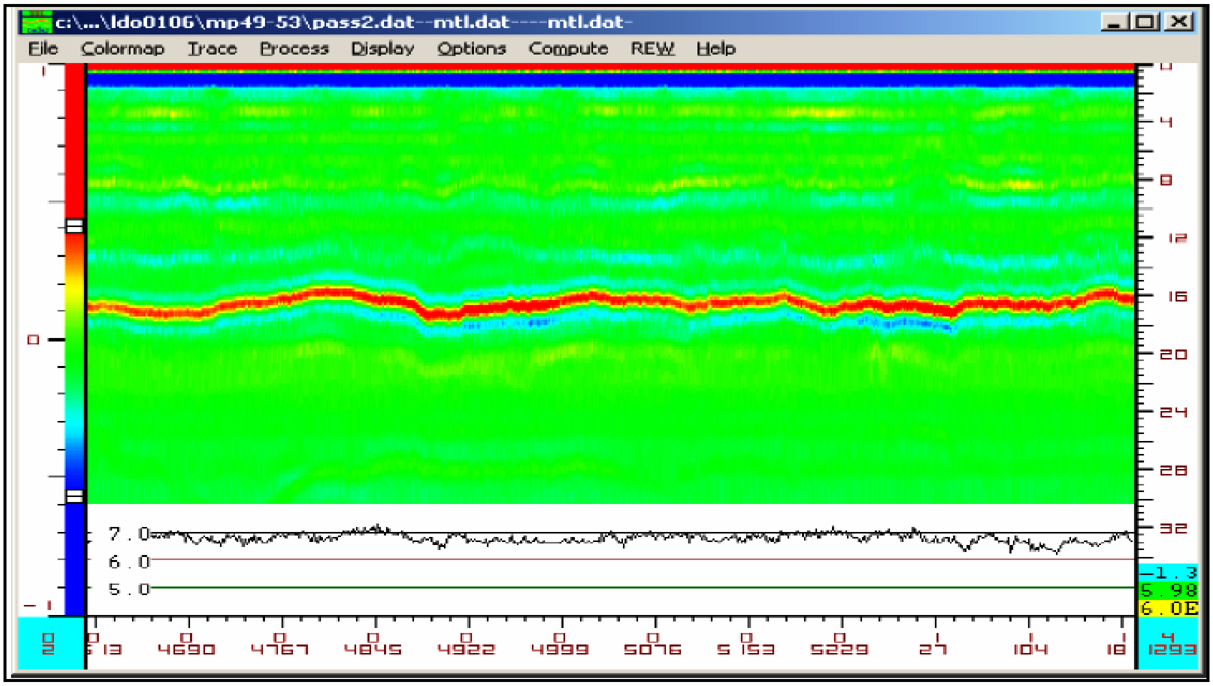


Figure 3.6. GPR Data from I-35 Perpetual Pavement Project Where WIM Site Is Located (CSJ 0018-02-049).

Researchers note that the stone matrix asphalt (SMA) and stone-filled (SF) layers tested from the Cotulla WIM project passed the Hamburg test. Since this test is typically specified on TxDOT projects, one item to check when evaluating candidate WIM sites are the Hamburg test results from the district laboratory engineer. However, the Hamburg only provides an indication of rutting resistance. To evaluate candidate WIM sites, it is also important, in the researchers' opinion, to consider the fatigue resistance of the structural HMAC layers. From practical experience, it is possible to get a mix with acceptable rutting resistance but unacceptable fatigue performance and vice-versa. The need for resiliency or resistance to cracking significantly increases with thinner HMA structures as the level of strain increases at the bottom of the HMA mass. Lean asphalt concrete mixtures meeting the low end of the specified range for binder content have been observed to pass the Hamburg test but perform poorly in terms of cracking during service. For this reason, researchers also examined laboratory data collected with the overlay tester on HMAC samples taken from Cotulla. Figure 3.7 shows a schematic illustration of the overlay tester while Figure 3.8 shows a picture of the test equipment.

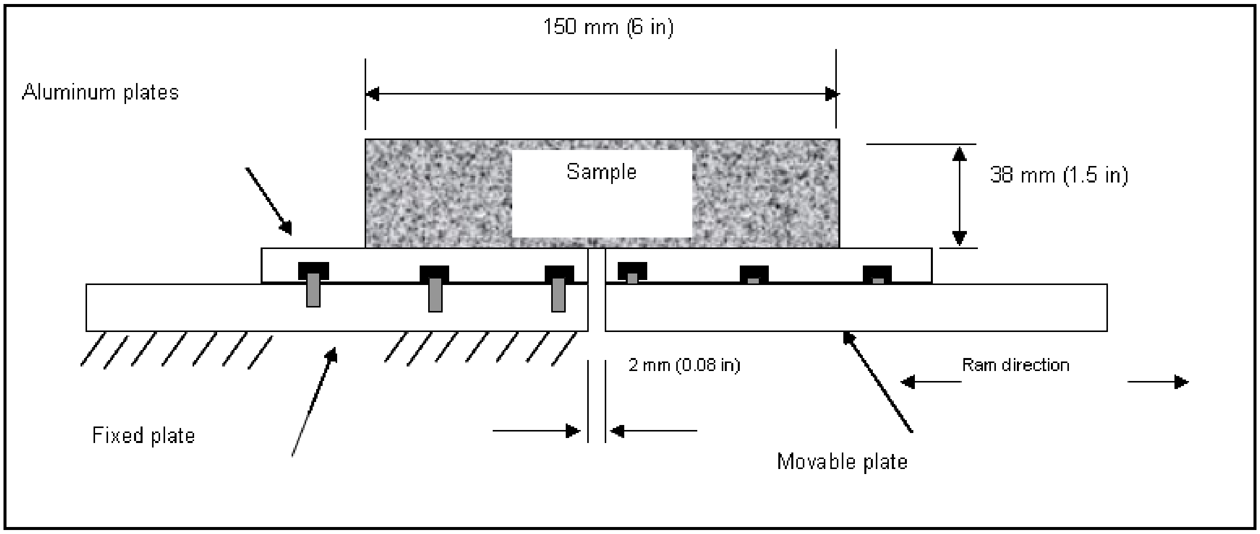


Figure 3.7. Schematic Illustration of Overlay Tester.

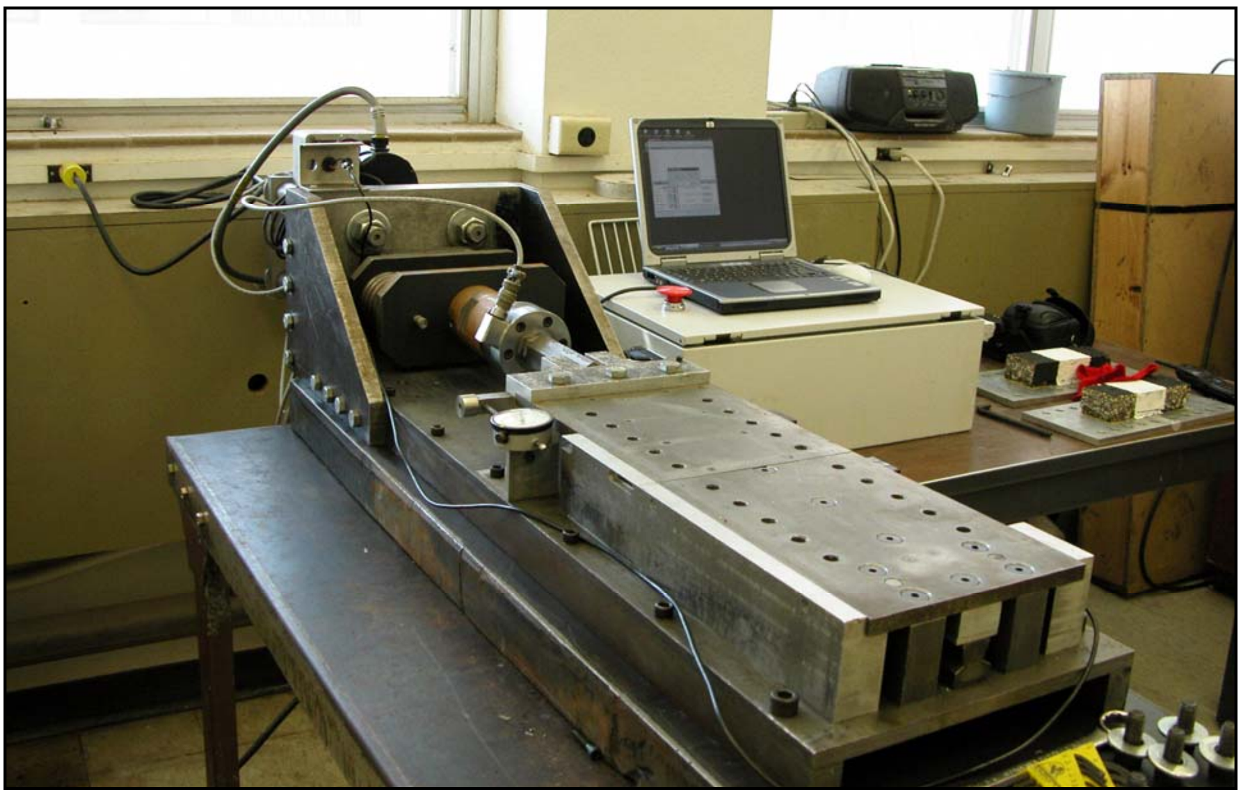


Figure 3.8. Picture of TTI Overlay Tester.

The overlay tester simulates the horizontal opening and closing of cracks. In this test, the HMAC specimen is glued to two platens, one fixed and the other movable. Specimens for testing are cut from six-inch diameter cores taken from the field or molded in the laboratory using the Superpave gyratory compactor. Specimen dimensions are 6 inches long × 3 inches wide × 1.5 inches deep. Tests are typically conducted at room temperature (77 °F). During testing, specimens are subjected to a constant displacement loading where an opening displacement of 25 mils is maintained per cycle. The load required to produce this displacement is monitored during the test. From this test, the number of load cycles to fracture the specimen, or to reach a 93 percent reduction in the initial loading is reported, whichever occurs earlier. This number is used as a measure of the material's crack resistance, i.e., the higher the number, the greater the crack resistance.

Based on experience with testing TxDOT surface mixtures, a criterion of 300 load cycles has been proposed for differentiating between good and poorly performing mixtures. For the SMA and rich bottom layers placed at the Cotulla WIM project, Scullion (2006) reports overlay test results of 47 and 227 load cycles, respectively, which are below the proposed criterion of 300 load cycles. Scullion (2006) commented that these results, particularly for the SMA mix, are disappointing. However, researchers note that the Cotulla flexible pavement WIM sections appear to be performing well, with no observed distresses on the sections as of the July 2008 visual survey performed on this research project. Appendix C presents the visual survey data from this project that researchers collected in July 2007 and July 2008. It should be pointed out that the FWD deflections on these sections are relatively small, similar in magnitude to the deflections of a thick concrete pavement slab. This observation should be considered in interpreting the overlay test results from this perpetual pavement project as the small pavement deflections would tend to retard the development of cracks within the pavement.

Scullion (2006) also noted in his review of perpetual pavements that there appears to be room to increase the binder content and still achieve a mix that would meet both Hamburg and overlay test requirements. Researchers note that a procedure for balanced mix design based on the Hamburg and overlay tester is being developed in TxDOT Project 0-5123. The criteria from this project should prove useful in identifying candidate flexible pavement WIM sites that can reasonably be expected to provide adequate service life. In this regard, researchers note that TPP aims to have a minimum 10-year service life from its weigh-in-motion sites. Thus, candidate sites should provide this minimum predicted service life for the projected truck traffic.

ENGINEERING EVALUATION OF CANDIDATE WIM SITES

This section presents guidelines for evaluating flexible pavements to identify candidate WIM sites. The guidelines presented are meant to be used on new construction, reconstruction, or resurfacing projects, or on pavements that have recently been rehabilitated or at the early stages of their life cycles. Researchers make this recommendation to minimize the likelihood that the WIM installation will be lost due to construction work being scheduled within 10 years after placement of the load sensors.

- High level of initial smoothness. In the researchers' opinion, the 500-ft WIM sections should be located within intervals of a project that fall within the bonus provisions of TxDOT's Item 585 smoothness specification as applied to individual wheel path profiles. This recommendation requires that defects be determined by wheel path. For the purpose of identifying candidate WIM sites, 500-ft sections showing no defects (bumps or dips) per wheel path should be established based on inertial profile measurements. This recommendation is based on findings from Project 0-4863 (Fernando, Harrison, and Hilbrich, 2007), where researchers found that using the average profile tends to mask defects that exist along the individual wheel paths. This project also found that occurrences of high dynamic load variability are associated with defects found on the pavement surface. Thus, defect locations should be avoided so as not to negatively affect the accuracy of the WIM measurements at a proposed location. Researchers also recommend checking a candidate site against the WIM smoothness criteria in the provisional AASHTO MP-14 specification. Certainly, a 500-ft section that meets the Type I criteria would provide a suitable candidate location for placing a WIM installation on the basis of pavement smoothness. Flexible pavement sections with WIM smoothness indices falling between the lower and upper thresholds of the AASHTO MP-14 provisional specification would still make suitable candidates for WIM installations provided that no defects are found on each wheel path based on the current TxDOT ride quality bump template. However, researchers recommend avoiding flexible pavement sections that exceed one or more of the upper thresholds of the WIM smoothness criteria for Type I systems given in AASHTO MP-14.
- Uniformity and degree of pavement support. Researchers recommend collecting FWD data on candidate WIM sections and assessing the uniformity and degree of pavement

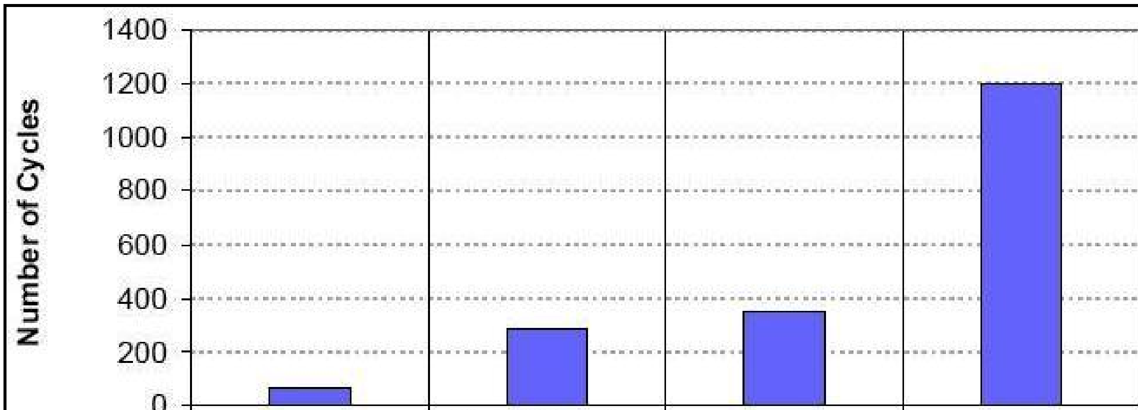
support on the basis of predicted service life. For this purpose, the engineer will use the MODULUS program (Liu and Scullion, 2001) to backcalculate pavement layer moduli at each FWD station on a given lane. The engineer will then use the output from MODULUS to predict pavement life using the computer program for overweight truck route analysis (OTRA) developed in TxDOT Project 0-4184 (Fernando and Liu, 2004). This approach permits the engineer to assess the uniformity and degree of pavement support based on the predicted reliability for the user-prescribed design period. The variability of the measured FWD deflections, as well as the magnitudes of these deflections, will factor into the predicted reliability of the pavement structure along the candidate WIM section. During FWD testing, researchers recommend that pavement temperatures be collected on asphalt concrete WIM sections, at least once at the beginning and again at the end of the given section. These temperatures will permit the engineer to correct the backcalculated asphalt concrete modulus to a reference temperature of 75 °F in the OTRA analysis.

- Subsurface uniformity. GPR data on a candidate WIM section should show uniformity within the asphalt mat as indicated by the absence of strong intermediate reflections from within the asphalt layers and consistency in the surface dielectric values over the range of the GPR data.
- Good indications of material durability. Candidate WIM sections should pass both Hamburg and overlay test criteria. Since the Hamburg test (Tex-242-F) is normally required on TxDOT projects, the engineer can check with the district laboratory to obtain the Hamburg test results on a given project where a WIM installation is being considered. A rut depth of 12.5 mm (0.5-inch) at 50 °C (122 °F) is used as a limiting criterion to identify rut or moisture susceptible mixtures tested according to the following number of load cycles:
  - PG76-*xx*: 20,000
  - PG70-*xx*: 15,000
  - PG64-*xx*: 10,000

  Researchers also recommend running the overlay test (Tex-248-F) to check the cracking resistance of HMAC mixtures placed on a given project. This test takes on added importance with thinner HMAC surfaces. If the total HMAC thickness is six inches or less, researchers recommend running the overlay test to check the crack resistance of each asphalt lift. Since this test only became effective in 2007, there are yet no standard criteria unlike the Hamburg to identify acceptable mixtures other than the minimum 750 load cycles used for crack attenuating mixtures (CAMs). Figures 3.9 to 3.13 show data compiled by Rand (2007) from overlay tests completed on different asphalt concrete mixtures.

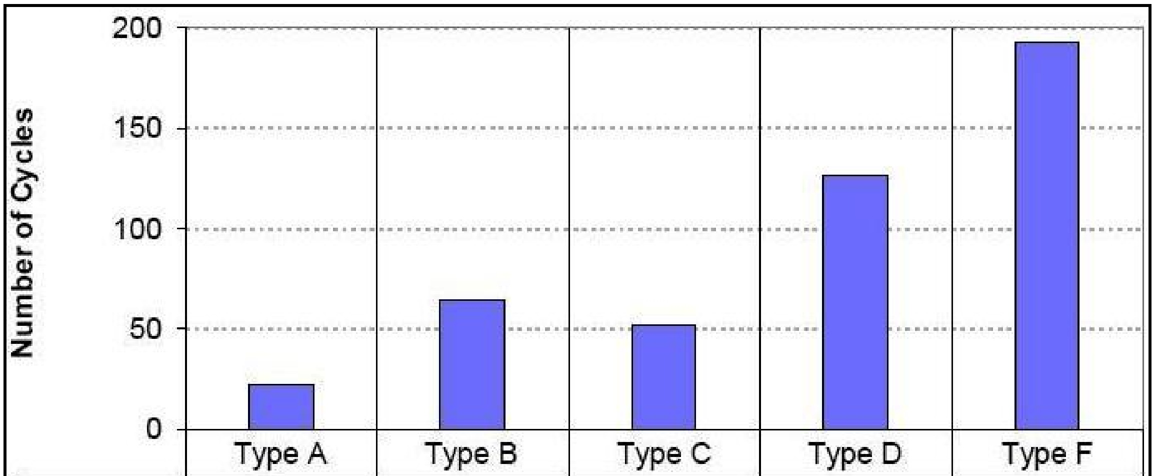
- Minimum 10-year predicted pavement life. Researchers recommend that the engineer run an analysis to predict pavement life on a given project where TxDOT plans to place a WIM installation. Since this candidate project would have been designed using the flexible pavement system (FPS) program, the engineer should first check the FPS design. In addition, nondestructive tests with the FWD and GPR should be run to check the predicted pavement life. For this purpose, researchers recommend using the OTRA program to assess the predicted reliability of the given pavement to sustain the expected traffic on the route for the minimum required 10-year pavement life. In this analysis, the engineer specifies the vehicle class distribution considered representative of the truck traffic on the candidate WIM section. In addition, the engineer inputs the beginning and ending average daily traffic (ADT) values, directional factor, and percent trucks, which are obtained from TPP's *Traffic Analysis for Highway Design* sheets. The user's guide prepared by Fernando and Liu (2004) gives more detailed instructions on using OTRA. For the purpose of assessing the capacity of the existing pavement to sustain the expected truck traffic, researchers recommend using a minimum reliability level of 95 percent for the minimum required 10-year pavement life.
- Geometric requirements. A minimum four-inch HMAC thickness is required to embed quartz WIM sensors into the pavement. In addition, TPP guidelines require the longitudinal slope of the WIM section to be within ±2 percent. It is also recommended that the WIM section be located on a straight tangent of the project tested and have four travel lanes with paved shoulders to facilitate WIM sensor installation. Setting up WIM installations on two-lane roadways is not advisable. Passing on two-lane roadways can result in crossing the loops in reverse order. Neither PAT nor the IRD WIM systems correctly classify these vehicles.

- Traffic conditions. To collect good WIM data, traffic on the instrumented lanes need to be traveling at constant speed, with vehicles tracking along the middle of each lane. Thus, engineers should avoid locations where vehicles tend to accelerate and/or decelerate (stop-and-go traffic) and change lanes to pass other vehicles. Accelerations or decelerations compromise accuracy, and lane changing can result in partially or totally missing one or more sensors.



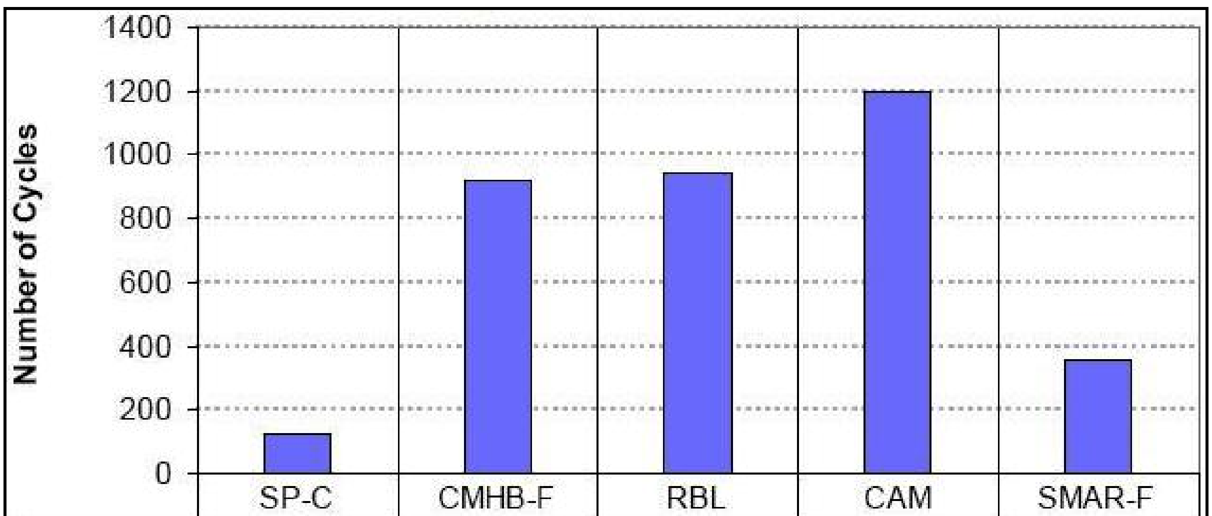
	Dense Graded	Performance-Designed Mixtures	Stone-Matrix Asphalt	RBL/CAM
Median	70	286	353	1200
Count	166	30	5	21
Minimum	2	14	258	147
Maximum	1200	1200	696	1200

Figure 3.9. Summary of Overlay Test Data on Various HMAC Mixtures (Rand, 2007).



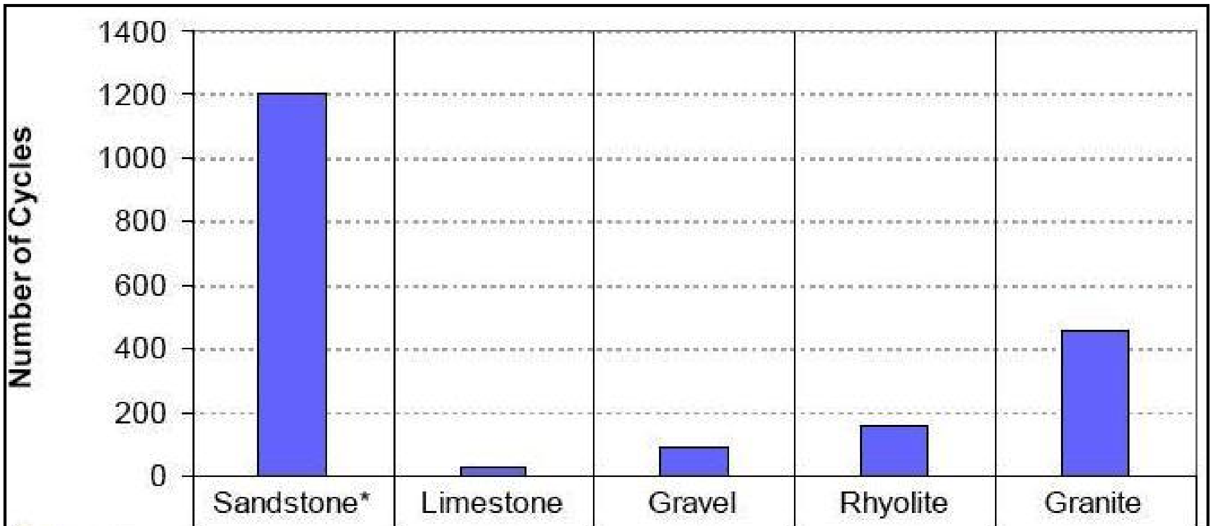
	Type A	Type B	Type C	Type D	Type F
Median	22	65	52	127	193
Count	14	33	49	59	10
Minimum	6	5	2	3	14
Maximum	91	1200	880	1200	1200

Figure 3.10. Summary of Overlay Test Data on Dense-Graded Mixtures (Rand, 2007).



	SP-C	CMHB-F	RBL	CAM	SMAR-F
Median	122	919	944	1200	353
Count	11	16	10	11	5
Minimum	84	14	147	202	258
Maximum	387	1200	1200	1200	696

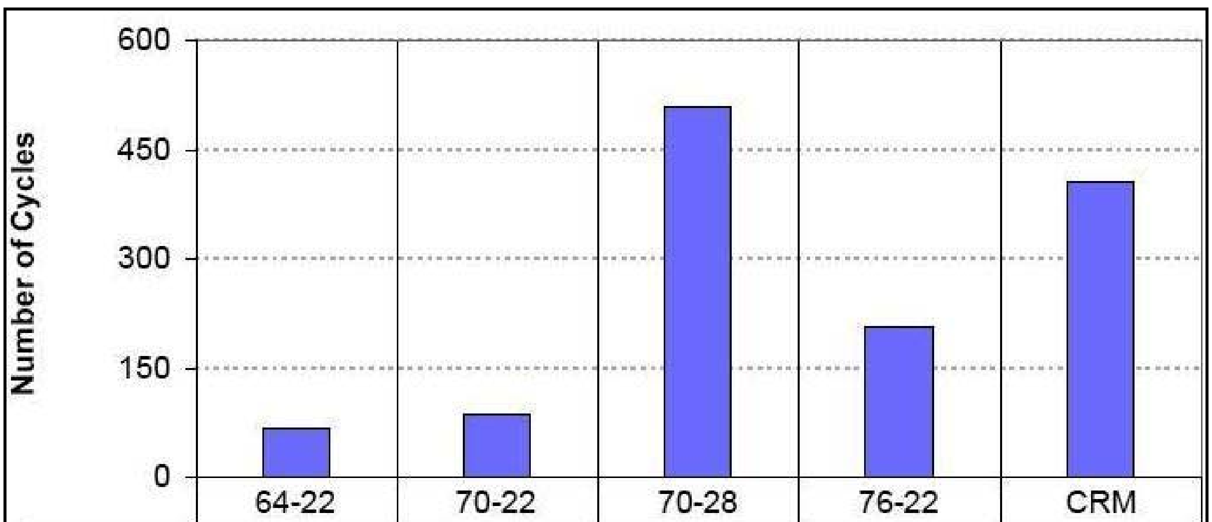
Figure 3.11. Summary of Overlay Test Data on Performance-Designed, CAM, and Stone Matrix Asphalt Mixtures (Rand, 2007).



	Sandstone*	Limestone	Gravel	Rhyolite	Granite
Median	1200	32	90	155	455
Count	6	65	32	9	49
Minimum	1200	2	2	86	6
Maximum	1200	1200	1200	387	1200

*Only CAM mixes tested

Figure 3.12. Influence of Aggregate Type on Overlay Test Results (Rand, 2007).



	64-22	70-22	70-28	76-22	CRM
Median	66	86	508	207	404
Count	40	59	34	57	10
Minimum	3	2	6	2	14
Maximum	1200	1200	1200	1200	1200

Figure 3.13. Influence of Binder Grade on Overlay Test Results (Rand, 2007).

With TxDOT's assistance, TTI researchers conducted tests on a number of flexible pavement projects to identify potential WIM sites. These projects included the US77 project in Robstown, the SH19 project in Trinity County, the SH105 project in Liberty County, and the Balmorhea project along I-10 in Reeves County. A preliminary survey of the SH105 project revealed that most of this project consists of two travel lanes with no paved shoulders. Where four travel lanes are found, these sections are primarily in urban areas with stop-and-go traffic. Because of these conditions, the SH105 project was not considered for further testing. In addition to these completed projects, two other on-going projects were identified. One project is along the SH89 bypass in Sinton, and the other project is located on SH6 north of Calvert in the Bryan District. However, these projects have not been completed at the time this research project terminated. Researchers suggest that these sites be tested in a follow-up project. In the following, the researchers present the results from pavement evaluations of potential WIM sections that were conducted during this project.

US77 Project in Robstown

TPP brought the US77 project in Robstown to the researchers' attention as it is a project where TPP and the Corpus Christi District had already selected a WIM location and where preliminary groundwork to instrument the site had already been made. This ground-work included a ground bore for routing wires beneath the highway and the placement of controller boxes to house the WIM electronics. Researchers collected profile, FWD, and GPR data along the US77 project that covered the selected 500-ft WIM site. The findings from analyses of the test data are presented in the following.

Road Smoothness. The road smoothness on the proposed 500-ft WIM site is acceptable based on existing TxDOT ride quality criteria. This assessment is based on the IRIs computed from the profiles collected on the lanes tested (see Table 3.13). Researchers also did not detect any bumps or dips on the individual wheel paths of each lane using the current TxDOT ride quality bump template. Examination of the WIM smoothness indices given in Table 3.14 showed that no lanes met all of the criteria specified in AASHTO MP-14 for Type I WIM systems. However, all indices are within the upper thresholds of the provisional WIM smoothness criteria.

Table 3.13. IRIs on 500-ft WIM Sections along US77 in Robstown.

Lane	Direction	Wheel Path IRI (inches/mile)	
		Left	Right
Outside (L1)	Northbound	57.7	61.3
Inside (L2)	Northbound	47.6	49.0
Outside (R1)	Southbound	35.2	37.3
Inside (R2)	Southbound	37.7	43.7

Table 3.14. WIM Smoothness Indices on 500-ft WIM Sections along US77 in Robstown*.

Wheel Path	Index (inches/mile)	Lane			
		Northbound outside	Northbound inside	Southbound outside	Southbound inside
Left	LRI	35.808	35.094	34.208	27.57
	Peak LRI	45.036	40.047	38.545	34.629
	SRI	20.685	29.342	34.787	35.272
	Peak SRI	29.594	37.345	38.946	35.902
Right	LRI	48.336	40.489	27.004	33.374
	Peak LRI	56.104	41.547	33.766	41.295
	SRI	30.931	47.899	34.663	27.037
	Peak SRI	44.484	50.405	40.271	47.104

*Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Predicted Pavement Life. The pavement structure at the proposed WIM site consists of a Type C surface mix overlying a Type B mix with nominal thicknesses of 2.2 and 5.5 inches, respectively. These HMAC lifts sit on top of an 8-inch flexible base overlying a 10-inch cement-stabilized subbase. The FWD maximum deflections at drop height 2 were generally under 10 mils on all four lanes with coefficients of variation under 10 percent except for the northbound outside lane where this statistic is about 14 percent. Backcalculations of layer moduli showed that the average base stiffness on this lane is quite a bit lower than on the other three lanes. Tables D1 to D4 in Appendix D present the measured FWD deflections on all four lanes while Tables D5 to D8 present the backcalculated layer moduli. Researchers used these modulus values in OTRA to predict the pavement life expectancy on each lane tested. From this analysis, researchers determined the pavement design reliability, which is the estimated probability that the given lane will last 10 years for the predicted traffic during that period.

To characterize the truck traffic for determining reliability, researchers obtained information from TPP's *Traffic Analysis for Highway Design* sheets that TxDOT engineers use for pavement design. Table 3.15 presents relevant traffic information on the candidate WIM sites evaluated during this research project. The *Traffic Analysis for Highway Design* sheets typically show the beginning and ending ADTs corresponding to a 20-year design period. Given this information, researchers estimated the traffic growth rate for a given site using the following equation:

$$ADT_t = ADT_0 (1 + g)^t \quad (3.3)$$

where,

ADT_0 = initial ADT,

ADT_t = ADT after t years, and

g = growth rate.

Given the estimated traffic growth rate, the researchers computed the ADT at year 10 (ADT_{10}) for the OTRA analysis. This program also requires the average number of axles per truck and the percentages of single, tandem, and triple axle groups. To determine these input variables, researchers assumed the truck traffic distribution given in Table 3.16. The distribution shown is taken from the default truck traffic classifications (TTCs) incorporated into the mechanistic-empirical pavement design guide (M-E PDG) program developed by Applied Research Associates (2004). This pavement analysis program provides a set of 17 default truck traffic distributions determined from analyzing traffic data collected on over 180 LTPP test sections. For the performance evaluation of the candidate WIM sites tested in this project, researchers selected TTC group 1 of the M-E PDG default truck traffic classifications. This TTC group is representative of major single-trailer truck routes. Based on this distribution and the percent of truck traffic, OTRA determines the average number of axles per truck, and the percentages of single, tandem, and triple axle groups. These variables are also given in Table 3.15.

Table 3.15. Traffic Information for Reliability Analysis of Candidate WIM Sections.

Traffic Variable	WIM Site Project		
	US77 at Robstown	SH19 at Trinity	I-10 at Balmorhea
ADT_0 [1]	20,340	8,800	3,700
ADT_{20} [1]	30,000	12,900	4,921
Traffic growth rate (%) [2]	1.96	1.93	1.44
ADT_{10} [2]	24,697	10,654	4,269
Directional factor (%) [1]	50	50	50
Percent trucks in ADT [1]	19.7	9.1	28.1
Average axles/truck [3]	2.96	2.96	2.96
Percent single axles [3]	43.7	43.7	43.7
Percent tandem axles [3]	55.8	55.8	55.8
Percent triple axles [3]	0.5	0.5	0.5

[1] From *Traffic Analysis for Highway Design* sheet for given project
[2] Calculated using equation (3)
[3] Based on truck traffic distribution given in Table 3.16

Table 3.16. Truck Traffic Distribution Used in Pavement Design Reliability Analysis.

Truck Category	Percent of Truck Distribution
Buses (3B)	1.3
2-axle single unit truck (2D)	8.5
3-axle single unit truck (3A)	2.8
4-axle single unit truck (4A)	0.3
2-axle tractor, 1-axle trailer (2S1) 2-axle tractor, 2-axle trailer (2S2) 3-axle tractor, 1-axle trailer (3S1)	7.6
3-axle tractor, 2-axle trailer (3S2)	74.0
3-axle tractor, 3-axle trailer (3S3)	1.2
5-axle multi-trailer (2S1-2)	3.4
6-axle multi-trailer (2S2-2)	0.6
7 or more axle trucks	0.3
Total	100.0

Table 3.17 shows the estimated pavement design reliability levels for the US77 500-ft WIM sections. In this analysis, all trucks were assumed to be running at the legal axle load limits of 20, 34, and 42 kips for single, tandem, and triple axle groups, respectively. It is observed that three of the four lanes tested meet the minimum recommended reliability level of 95 percent based on both the Asphalt Institute (1982) fatigue and rutting performance models incorporated in OTRA. On the northbound outside lane, there are a couple of FWD test locations where the predicted service life based on fatigue cracking is less than the minimum

recommended 10-year target life. This result, coupled with the observation that a longitudinal crack had already formed on this lane, raised reservations about installing a weigh-in-motion system at the proposed location. Examination of GPR data and results from overlay tests also raised concerns about the uniformity and durability of the HMAC material placed at the site.

Table 3.17. Results from Pavement Design Reliability Analysis on US77 WIM Sections.

Lane	Predicted Reliability (%)		Number of FWD stations with less than 10 years predicted life*	
	Fatigue	Rutting	Fatigue	Rutting
Northbound outside	93	≈100	2	0
Northbound inside	≈100	≈100	0	0
Southbound outside	≈100	≈100	0	0
Southbound inside	≈100	≈100	0	0

* 51 FWD test locations per lane

Subsurface Uniformity and Material Durability. The GPR data from the US77 WIM sections revealed multiple reflections within the hot-mix layer that suggest possible areas of moisture infiltration. This observation is seen in Figures 3.14 to 3.17, which illustrate the GPR data from the 500-ft WIM sections. This apparent non-uniformity within the asphalt mat was a concern among researchers and members of the project monitoring committee. While the WIM section exhibits good ride quality, the GPR data suggest that the pavement could become rougher in the near future due to possible moisture infiltration in the asphalt layer and the heavy truck volume on this route. In addition, a longitudinal crack (40 ft long) had already formed in the middle of the northbound outside lane about 140 ft south of the proposed WIM sensor location. This crack is within the 350-ft approach to the WIM sensor on that lane.

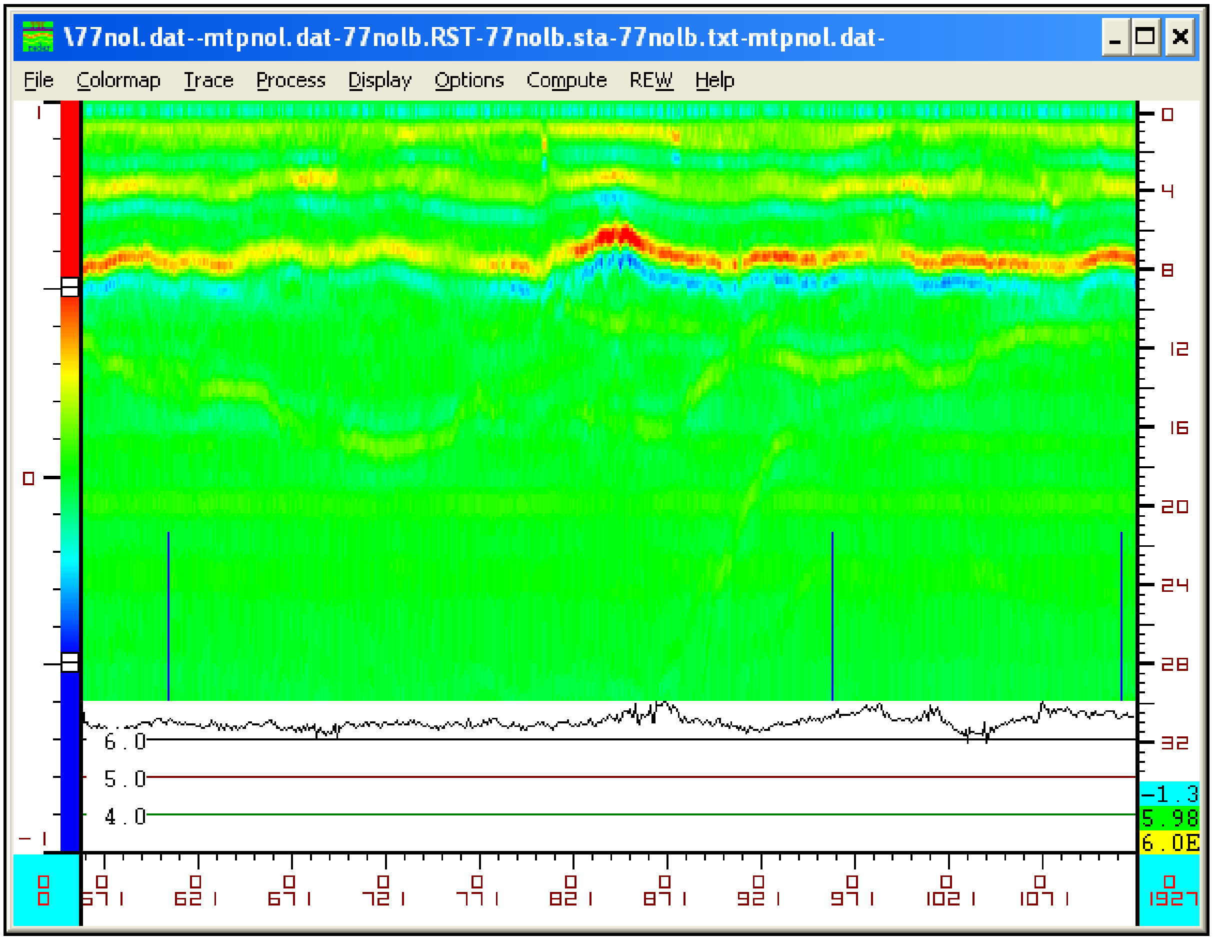


Figure 3.14. GPR Data on US77 Northbound Outside Lane.

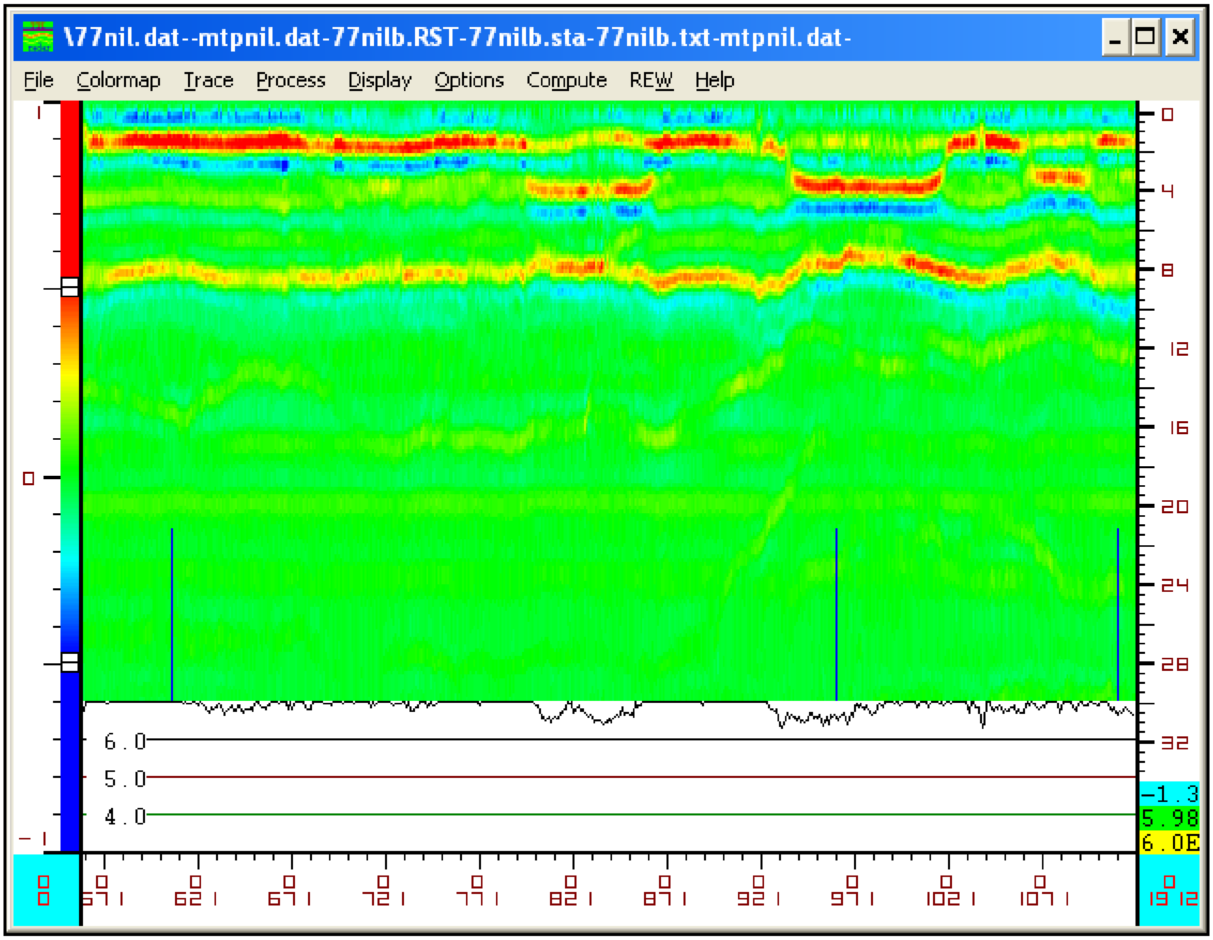


Figure 3.15. GPR Data on US77 Northbound Inside Lane.

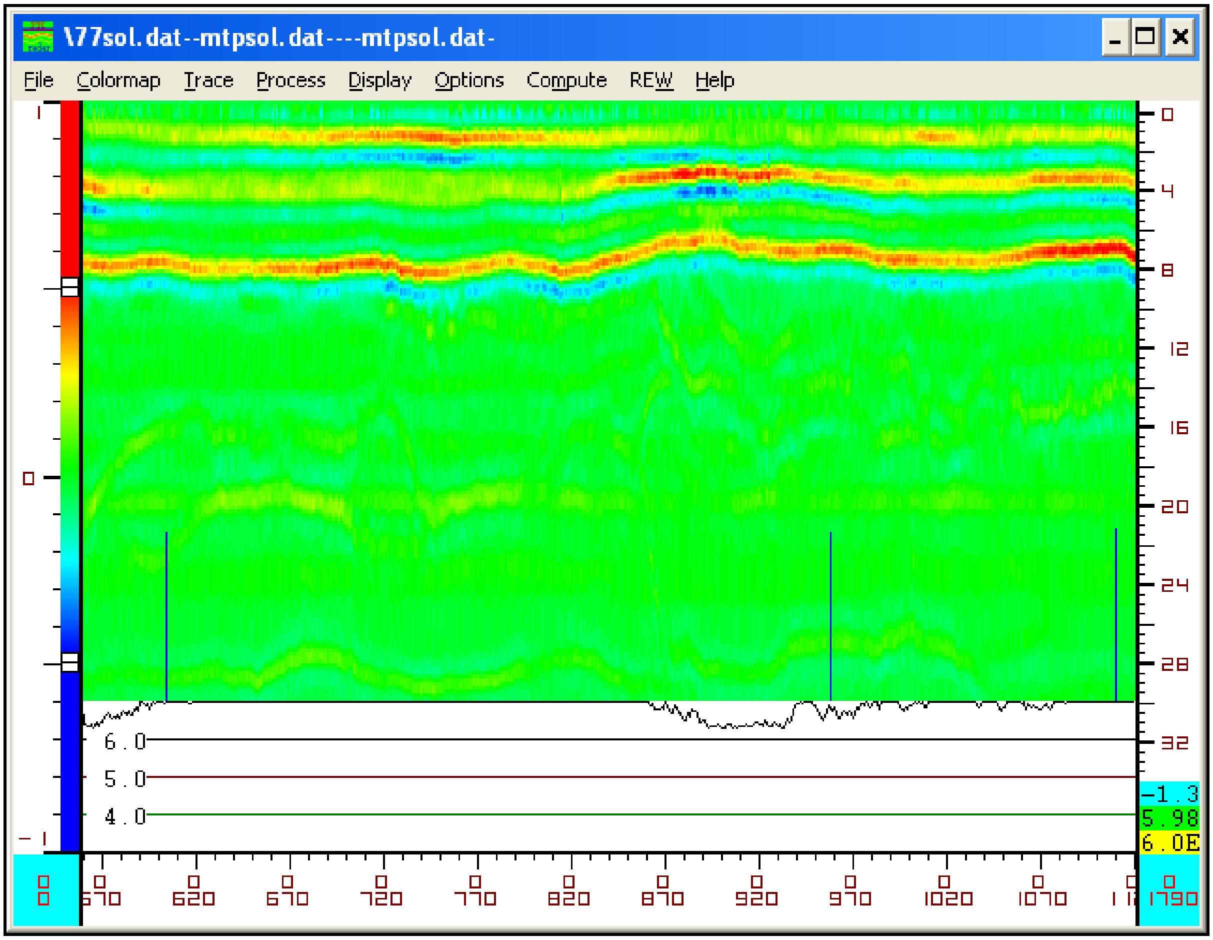


Figure 3.16. GPR Data on US77 Southbound Outside Lane.

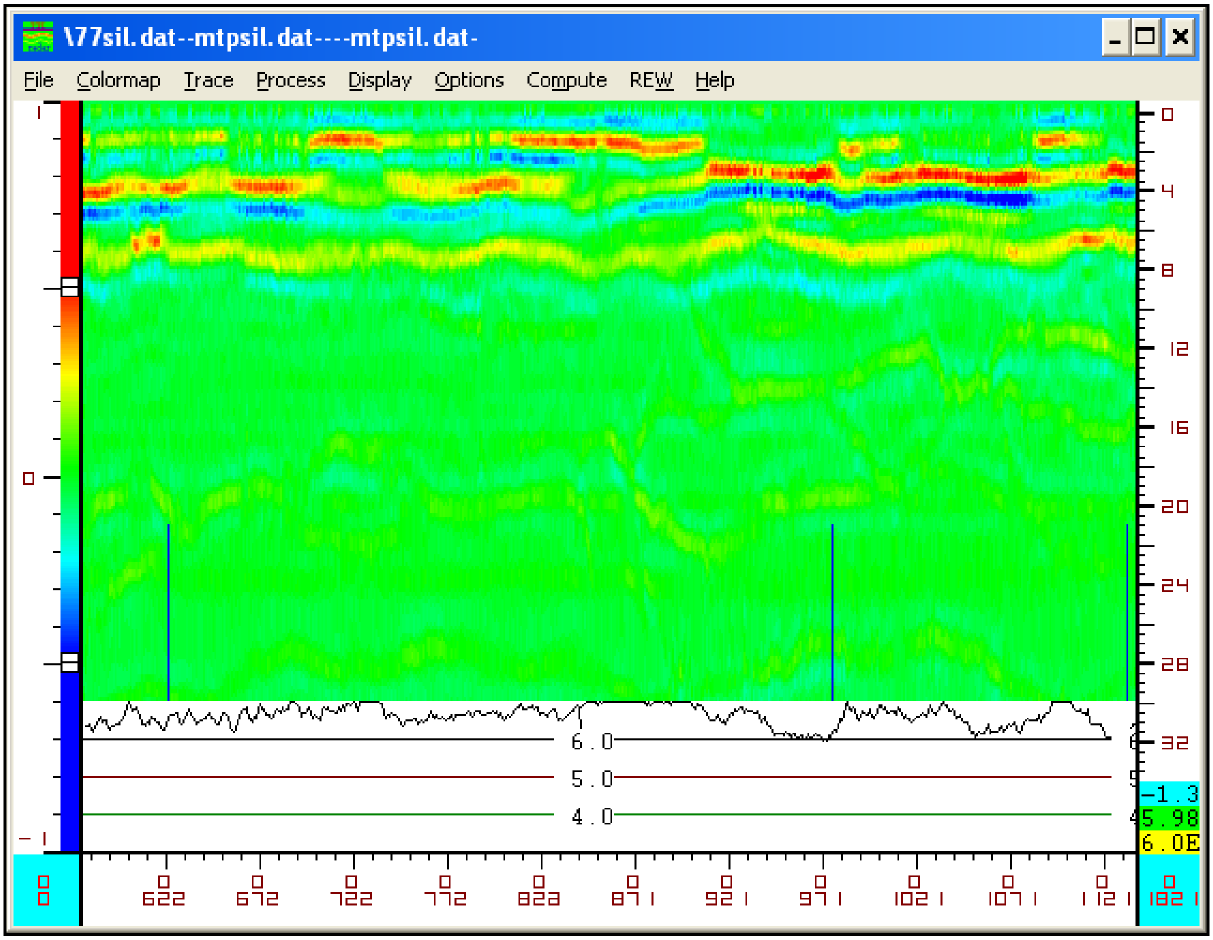


Figure 3.17. GPR Data on US77 Southbound Inside Lane.

Researchers note that the Corpus Christi District took cores at the vicinity of the selected WIM location along US77. After receiving these cores, TTI technicians cut the cores into the three different lifts that comprised each core. From the typical cross-section sheets provided by the district, the top lift was identified as a Type C, while the middle and bottom lifts were identified as Type B mixes. TTI technicians ran the overlay test on a sample of material cut from each lift and found that each sample took just one load application to fracture. This result is far below the desired minimum number of 300 load cycles. However, researchers recognize that testing was conducted on field samples that already have undergone some aging. In practice, it would be important to run the overlay test on field cores taken just after placement of the asphalt mix or on specimens molded from samples of the mix taken at the plant or the job site. Nevertheless, given the poor performance of the cores on the overlay tests and the fact that cracking has already occurred, there is justification to consider placing the WIM system at an alternative location.

Of interest from the laboratory tests are the measured asphalt contents for the three lifts. For this determination, researchers used the broken samples from the overlay tests to determine the asphalt content using TxDOT Test Method Tex-236F. From these tests, the asphalt contents were found to be 3.960, 3.647, and 3.598 percent, respectively, for the top (Type C), middle (Type B), and bottom (Type B) lifts. These values are low and are consistent with the observation made by the District Pavement Engineer who commented that the asphalt content for the Type B mix appears to be on the dry side after examining the cores taken from the proposed WIM location.

Based on the above results, the district came up with an alternative location along US77 from which additional cores were taken and tested. However, researchers obtained results similar to those on the original proposed WIM location along the route. The district decided to do a full-depth repair on the original site and to have the WIM sensors placed at that location. The test data collected along US77 provide baseline information and can prove useful in a follow-up study to assess the performance of the pavement placed on that project over the current life cycle.

From a site visit made in August 2008, researchers observed that more cracking has developed on the northbound outside lane as shown in Figure 3.18. There is visible rutting where crack development has progressed on the lane. Researchers note that only loop detectors were installed on the northbound lanes. The quartz WIM sensors were installed only on the southbound lanes.

SH19 Project at Trinity

This project is located along SH19 between reference marker limits 414 +1.979 and 418 +0.376. As was done on the US77 project, researchers collected profile, FWD, and GPR data along the SH19 project to determine a suitable location for a WIM installation. The following sections present the findings from analyses of the test data collected.

<u>Road Smoothness</u>. Figures 3.19 to 3.22 show the IRIs computed at 0.1-mile intervals along the lanes of the project surveyed by researchers. These charts show that within a 0.5-mile interval south of the intersection of SH19 with FM3453, smooth sections are found with wheel path IRIs lower than 60 inches/mile. These sections fall within the bonus provisions of TxDOT's Item 585 ride specification and thus merit further testing.


Figure 3.18. Cracking on US77 Northbound Outside Lane (from August 2008 Visit).

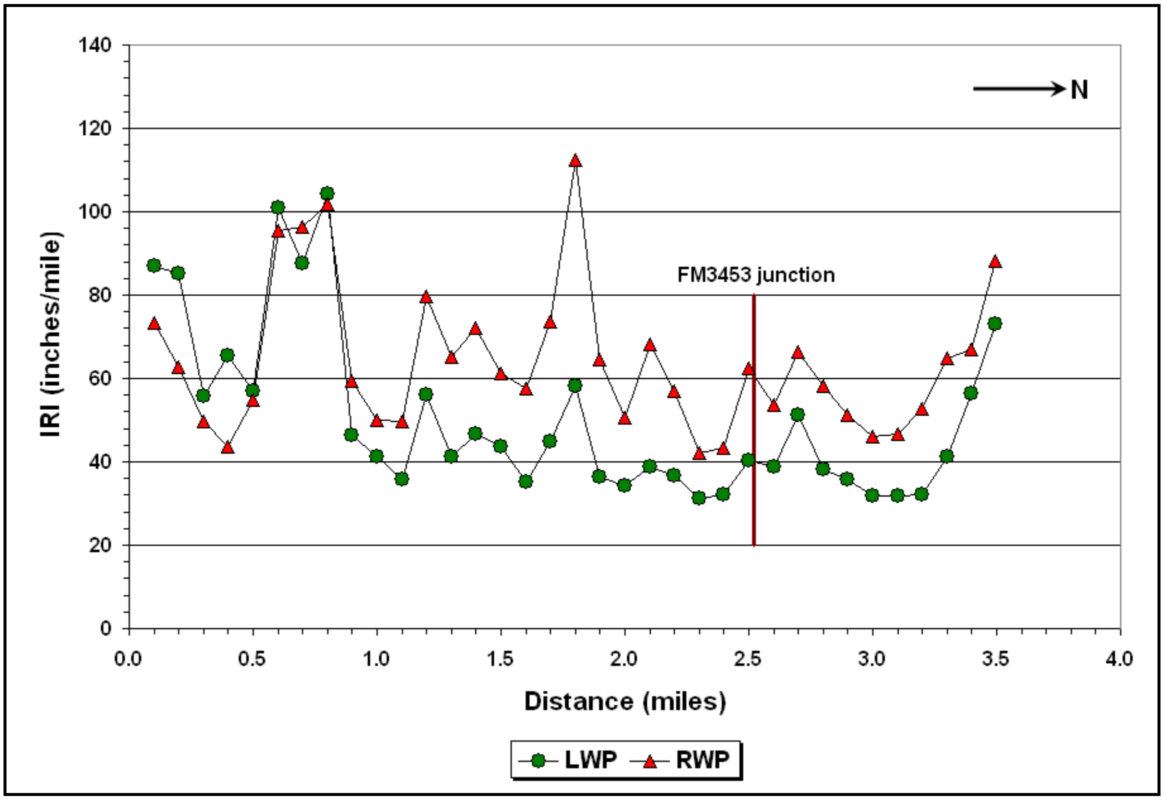


Figure 3.19. IRIs at 0.1-mile Intervals on SH19 Northbound Outside Lane.

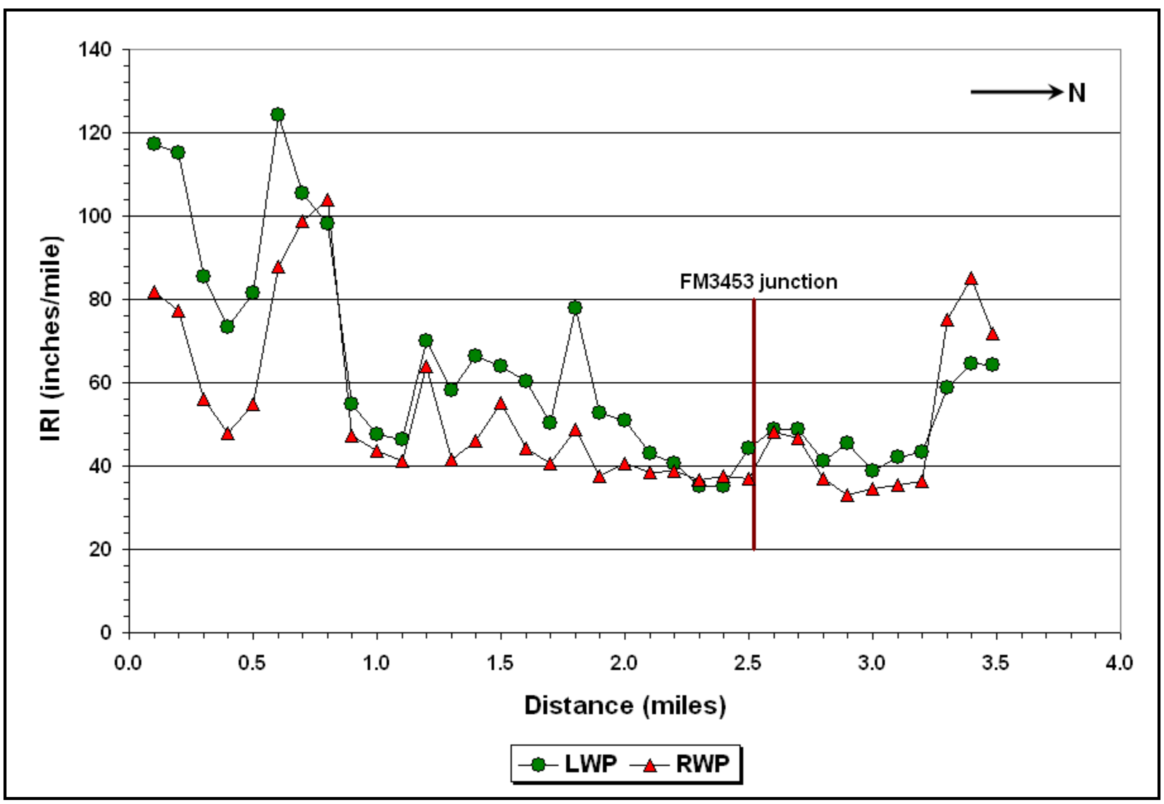


Figure 3.20. IRIs at 0.1-mile Intervals on SH19 Northbound Inside Lane.

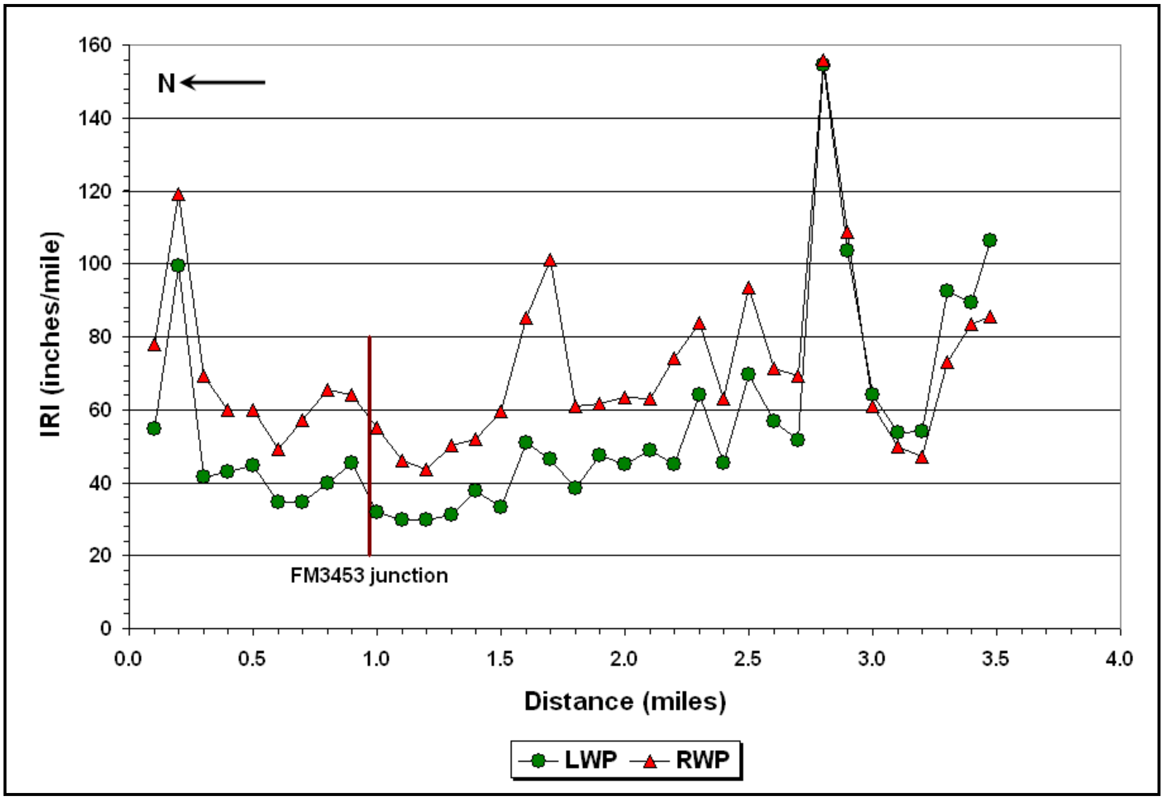


Figure 3.21. IRIs at 0.1-mile Intervals on SH19 Southbound Outside Lane.

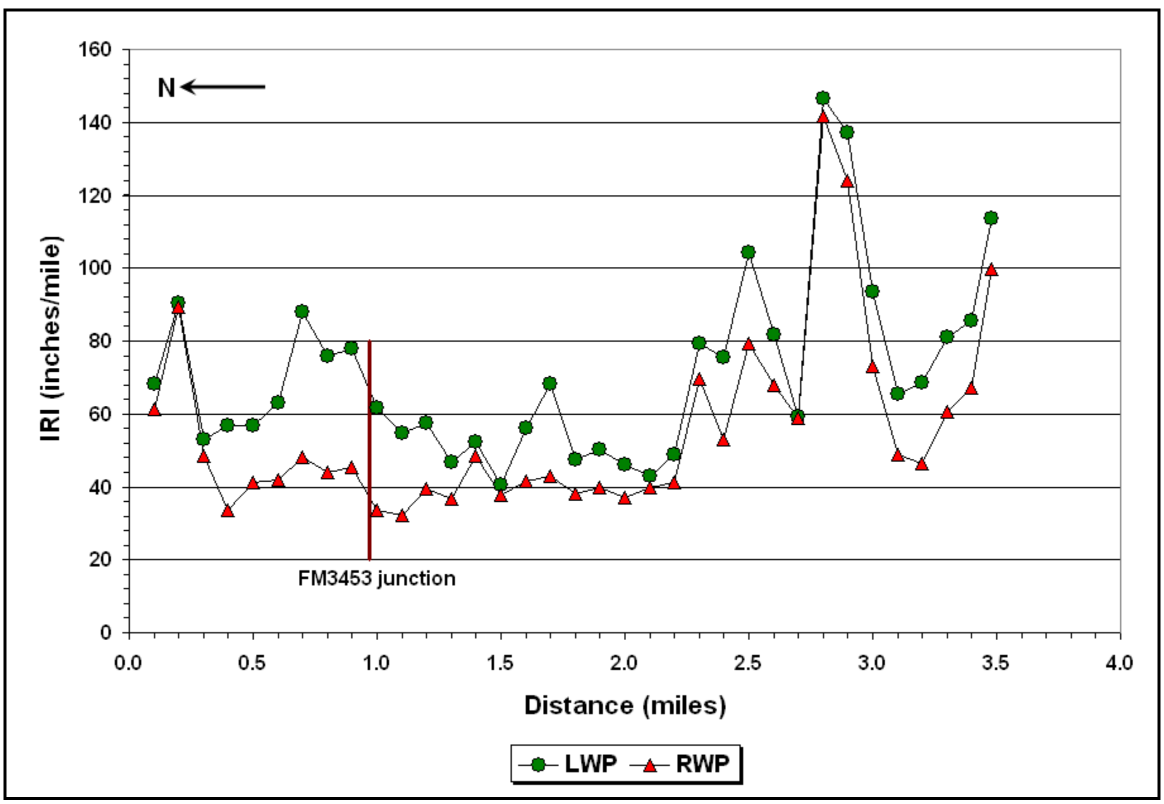


Figure 3.22. IRIs at 0.1-mile Intervals on SH19 Southbound Inside Lane.

Consequently, researchers marked a 0.5-mile interval along SH19 (with the FM3453 junction as its northern terminus) on which additional profile measurements were taken. This interval is located along a straight tangent of the project. Researchers then analyzed the profile data on each lane to compute 500-ft moving IRIs and WIM smoothness indices and to check for defects. These analyses were completed on the individual wheel path profiles for each lane tested.

Figure 3.23 shows the results of the profile analysis on the data collected along the northbound outside lane. Plotted on this chart are the moving IRIs on the left and right wheel paths computed over a moving base length of 500 ft, the standard length of a TxDOT WIM section. Beginning at the first 500 ft of profile data, the IRI is computed per wheel path, and the resulting statistic is plotted at the middle of that interval. The process steps forward a distance of one sample interval (corresponding to the distance between two successive profile elevations). The IRI is then computed for the next 500 ft interval, and the calculations are repeated until the last 500-ft of profile data are processed. The resulting plot of moving IRIs helps in identifying smooth 500-ft sections along the lane tested. In Figure 3.23, the moving IRIs on each wheel path are given by the solid lines labeled LWP (for left wheel path) and RWP (right wheel path) in the figure legend. Also shown in the figure are the locations of the defects found in each wheel path using the existing TxDOT ride quality bump template except that the analysis is applied to the individual wheel path profiles instead of the average profile. The magnitudes of the defects are given on the secondary ordinate axis shown on the right side of Figure 3.23. Positive defects identify bumps while negative defects denote dips. Note that the defects are located toward the north end of the 0.5-mile segment near the FM3453 junction.

Researchers also computed the WIM smoothness indices in the provisional AASHTO MP-14 specification using LTPP's WIM index software. In this analysis, the indices were computed continuously along the lane tested, similar in concept to the calculation of moving IRIs. In this way, the WIM indices corresponding to any possible placement of the WIM sensor along the measured profile can directly be obtained from the program output. From this analysis, researchers found no locations along the lane that meet the MP-14 thresholds for a Type I WIM system. However, there are locations near the FM3453 junction where one or more of the indices exceed the upper thresholds specified in the MP-14 specification.

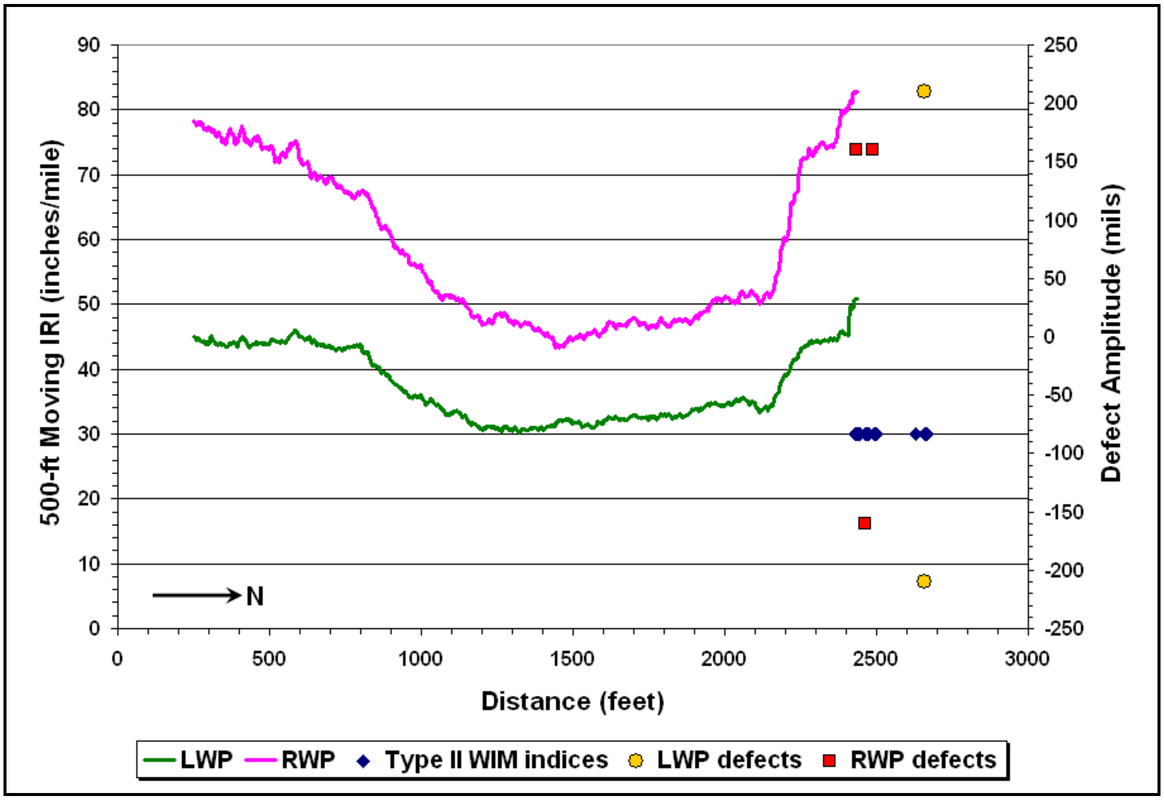


Figure 3.23. Ride Quality Evaluation Results on SH19 Segment (Northbound Outside Lane).

These locations are identified by the blue diamond symbols in Figure 3.23, which are labeled as Type II WIM indices in the figure legend. Note that these symbols are not associated with any of the two vertical axes in the figure. The symbols are only tied to the horizontal scale to show the locations where one or more of the WIM indices exceed the upper thresholds given in Table 3.3. As recommended previously, placing WIM sensors at these locations should be avoided. Thus, based on the information presented in Figure 3.23, the interval between 1000 and 2000 ft appears to provide a suitable section within which a WIM installation may be placed.

Researchers conducted a similar profile analysis on each of the other lanes. The results from these analyses are presented in Figures 3.24 to 3.26. On the northbound inside lane, Figure 3.24 shows that the interval between 1000 and 2000 ft also provides a suitable section (based on road smoothness criteria) within which a WIM installation may be placed. On the southbound lanes, this interval corresponds to a segment with limits at 640 and 1640 ft. Examination of Figures 3.25 and 3.26 shows that between these limits, a suitable section can also be found within which the WIM sensors can be installed on each lane.

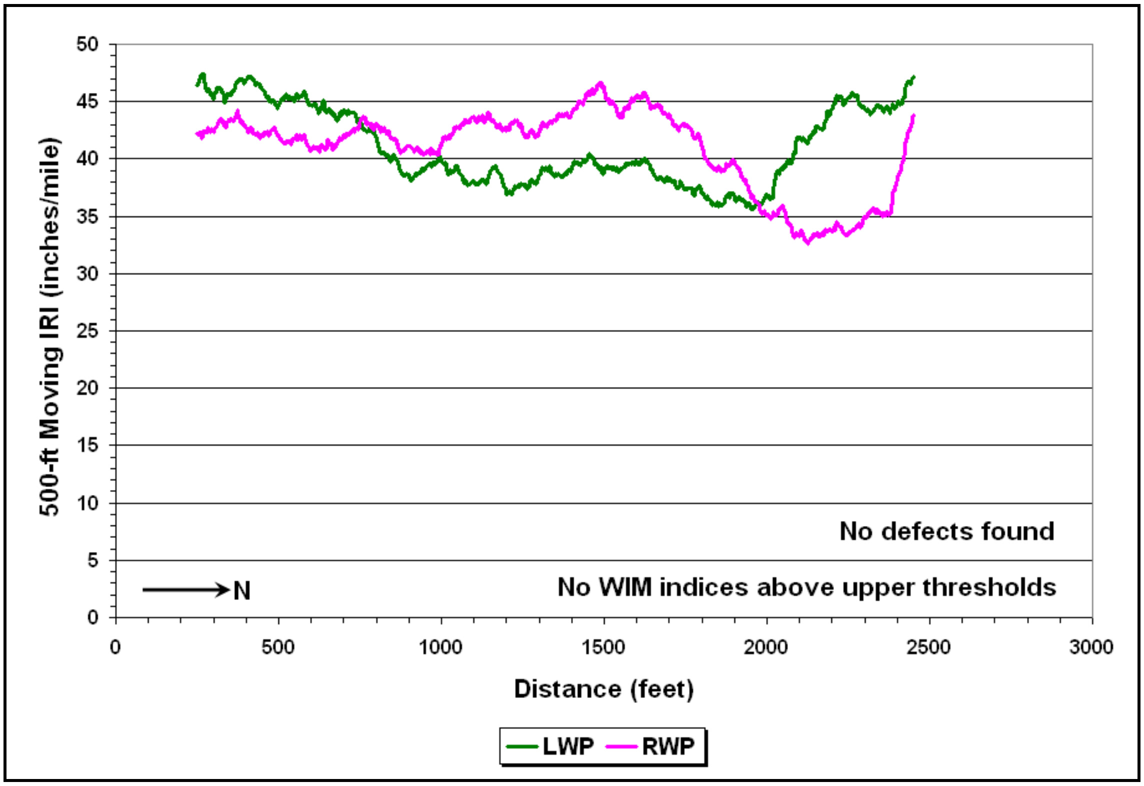


Figure 3.24. Ride Quality Evaluation Results on SH19 Segment (Northbound Inside Lane).

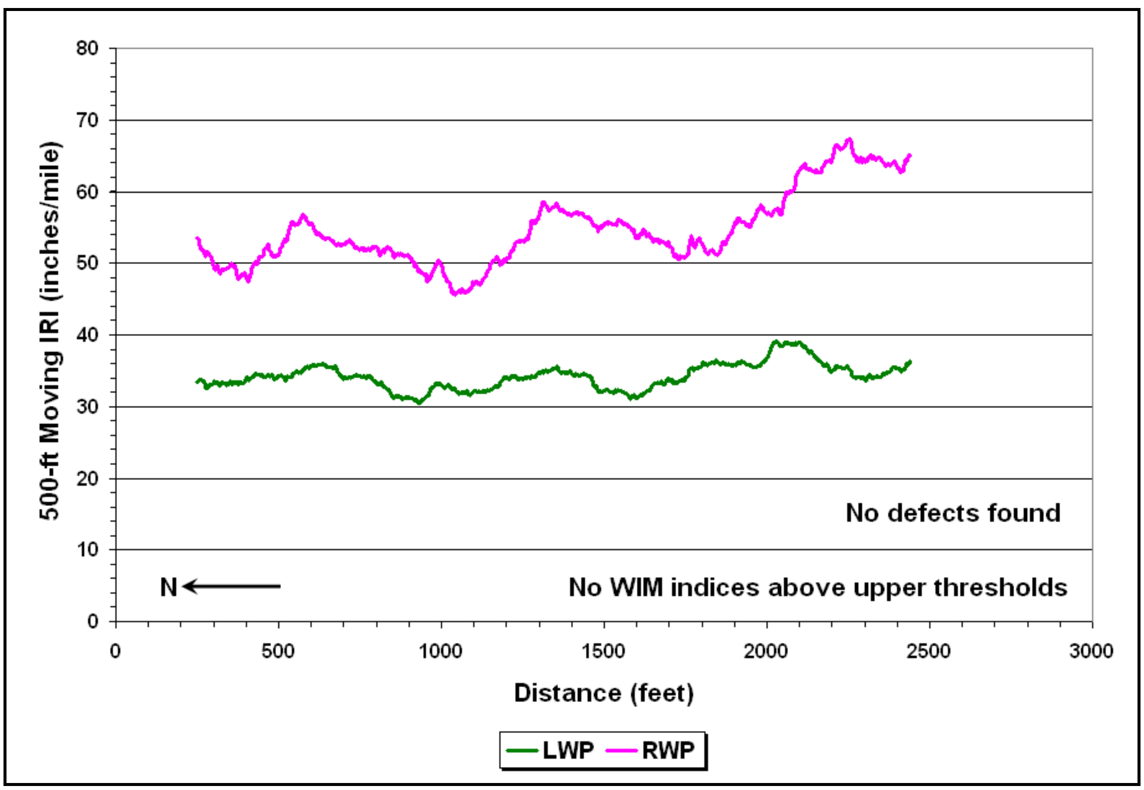


Figure 3.25. Ride Quality Evaluation Results on SH19 Segment (Southbound Outside Lane).

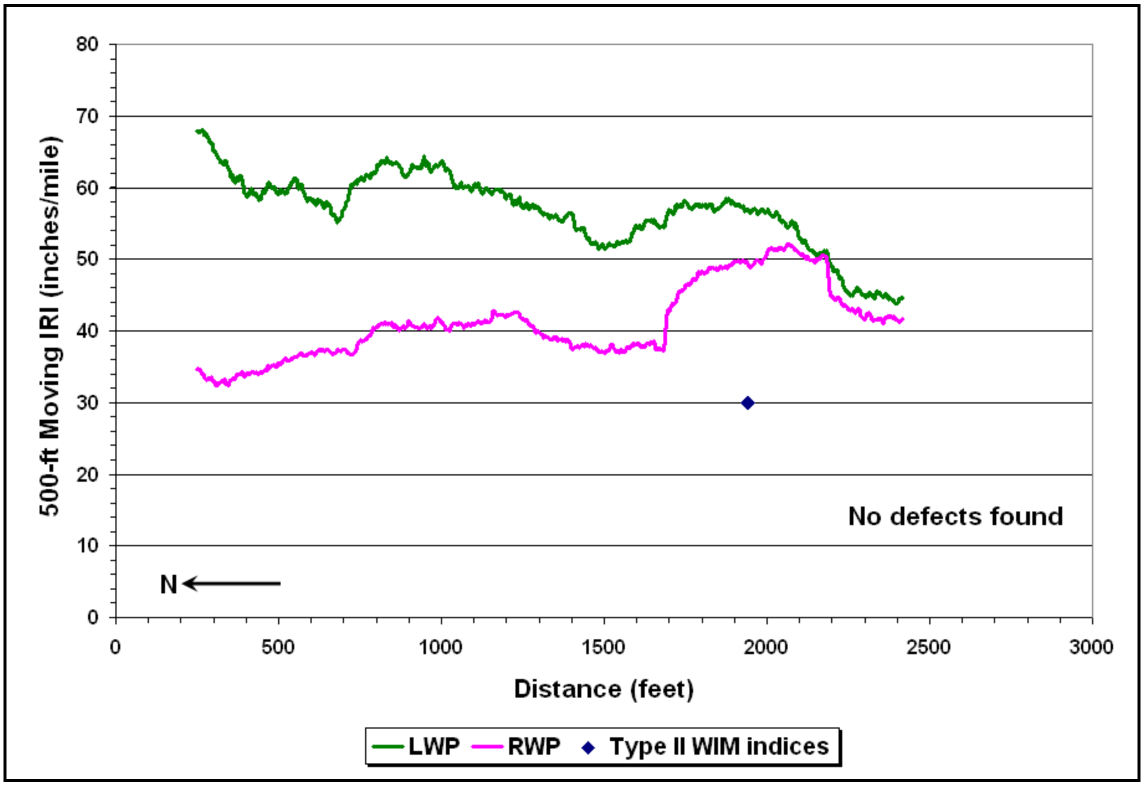


Figure 3.26. Ride Quality Evaluation Results on SH19 Segment (Southbound Inside Lane).

Predicted Pavement Life. With assistance from the Lufkin District, researchers collected FWD data along SH19 from the FM3453 junction to a distance of 0.5 miles south of this intersection. Within this interval, the typical cross-section sheets from the project plans show a pavement structure consisting of a 2-inch Type C surface mix, 3.5-inch Type C base, an underseal of MC-30 prime coat and one-course surface treatment, 9 inches of Type A, Grade 2 flexible base, and 12 inches of lime-treated subgrade. On this pavement structure, the FWD sensor 1 deflections at drop height 2 varied from about 4 to 15 mils over a range of pavement temperatures from 95 to 136 °F. Tables D9 to D12 in Appendix D present the measured FWD deflections on all four lanes while Tables D13 to D16 present the backcalculated layer moduli. Researchers used these modulus values in OTRA to estimate the pavement design reliability as explained previously. This analysis showed the estimated reliabilities to be approximately 100 percent on all four lanes for both fatigue and rutting criteria, with all FWD test locations predicted to last more than the minimum required 10-year pavement life for the projected traffic at the site. Thus, based on the FWD deflections, the pavement at the site appears to have enough structural capacity to provide the minimum service life required of a WIM installation.

Subsurface Uniformity and Material Durability. Figures 3.27 to 3.30 show GPR data taken within the limits of the FWD surveys along SH19. The GPR data on the northbound lanes (Figures 3.27 and 3.28) show intermittent reflections that appear to be coming from the underseal between the bottom of the Type C base and the top of the flexible base. The blue-colored reflections indicate a material with a lower dielectric constant than the Type C base. On the southbound lanes, the reflections from the underseal appear as red bands (see Figures 3.29 and 3.30), which suggest the possible presence of moisture at this depth. In addition, these figures show intermittent blue-colored bands that appear at a depth of about 2 inches, which coincides with the interface between the Type C surface and the Type C base. This interface is visible on the cores taken from the southbound lanes as illustrated in Figure 3.31. Note the two air pockets at the interface between the lifts, which might explain the blue-colored bands observed in the GPR data. These observations suggest possible potential problems due to moisture filled voids within the asphalt bound layers, particularly if water gets trapped within the HMAC material.

TTI technicians also ran the overlay tester on eight cores taken from the site. For these tests, each core was cut into two lifts so that both the Type C surface and the Type C base materials could be tested. The Type C surface specimens generally lasted only 2 load cycles while the Type C base specimens lasted from 2 to 13 cycles. These results are far below the recommended minimum of 300 load cycles to failure, and led to reservations among researchers on the suitability of using the SH19 project as a site for a weigh-in-motion installation.

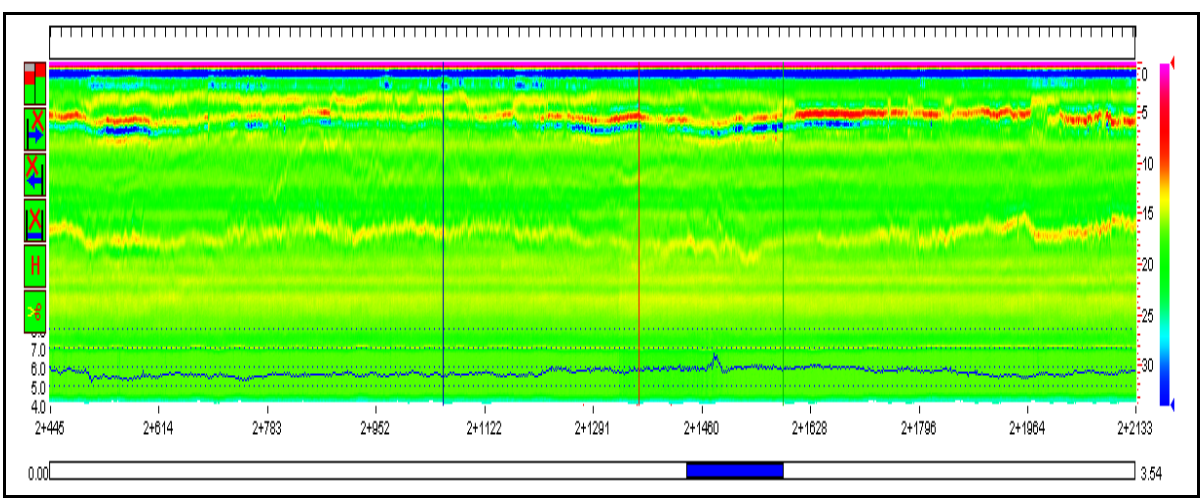


Figure 3.27. GPR Data on SH19 Test Segment (Northbound Outside Lane).

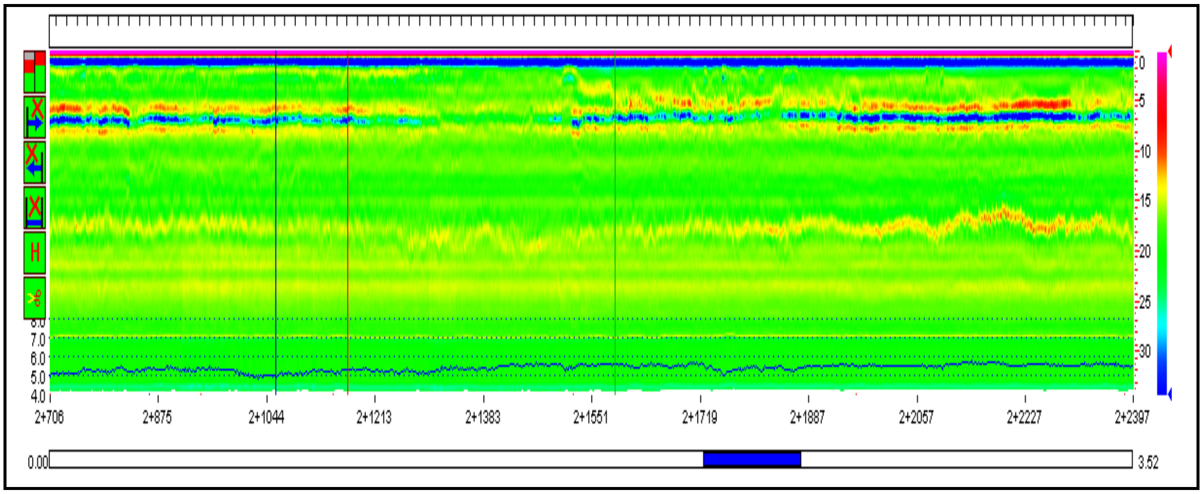


Figure 3.28. GPR Data on SH19 Test Segment (Northbound Inside Lane).

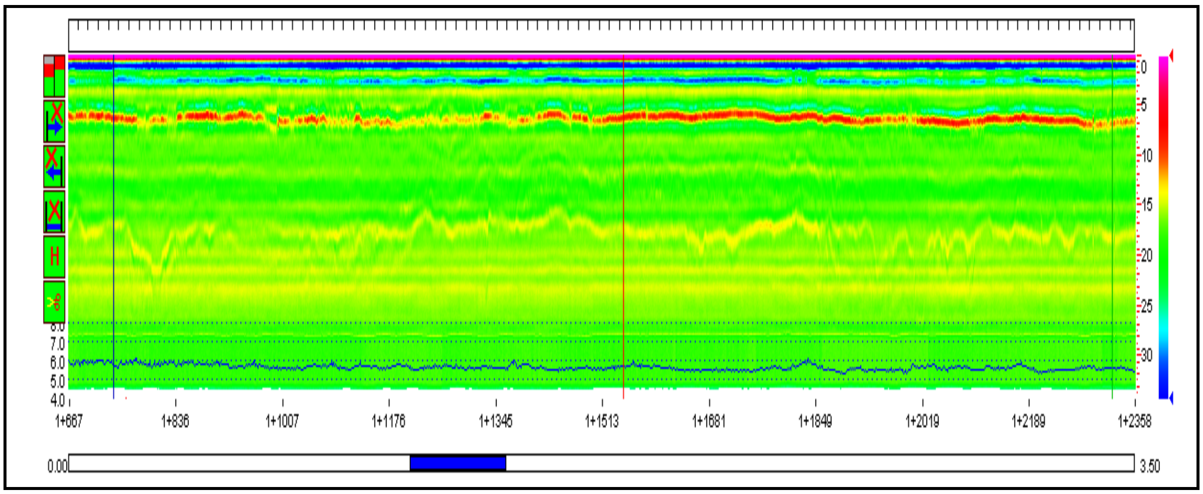


Figure 3.29. GPR Data on SH19 Test Segment (Southbound Outside Lane).

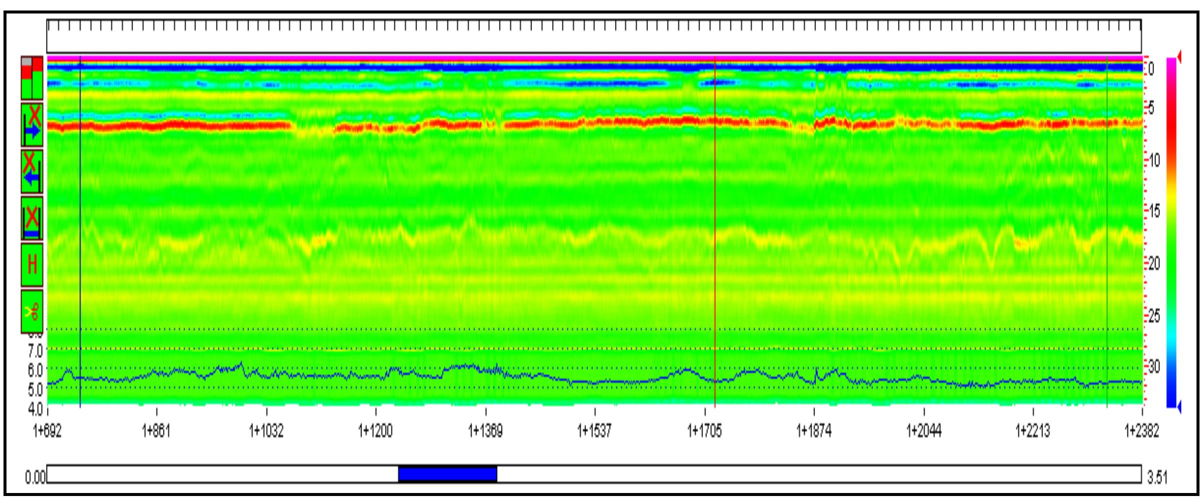


Figure 3.30. GPR Data on SH19 Test Segment (Southbound Inside Lane).

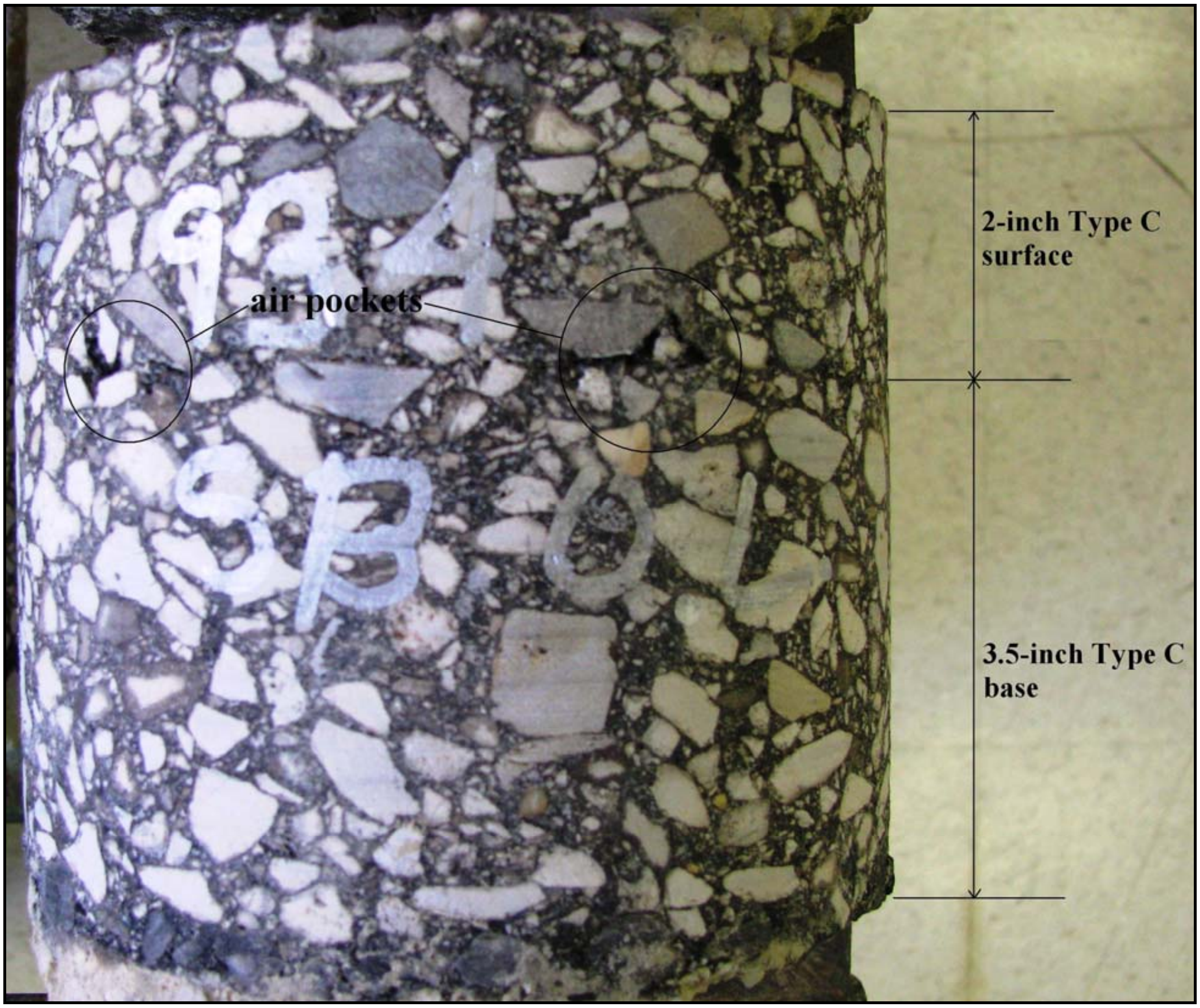


Figure 3.31. Core from Southbound Outside Lane Taken 934 ft South of FM3453 Junction.

Balmorhea Project along I-10 in Reeves County

This project is located on I-10 between reference marker limits 202 +0.447 and 204 +0.431. Over this interval, researchers initially collected profile measurements with TTI's inertial profiler. Researchers then analyzed these measurements to identify a candidate location for a WIM installation following the same procedure as described for the SH19 project at Trinity. This procedure involved calculation of 500-ft moving IRIs, identification of defects on each wheel path using the current TxDOT ride quality bump template, and point-by-point calculation of the WIM smoothness indices in the provisional AASHTO MP-14 specification. Figure 3.32 shows the results of the profile analysis on the eastbound outside lane of the I-10 project at

Balmorhea. This chart is similar to the one described earlier in Figure 3.23. The calculation of WIM smoothness indices showed locations along the eastbound outside lane that meet the Type I criteria proposed in AASHTO MP-14. These locations are identified by the triangular symbols in Figure 3.32. Note that these symbols are not associated with any of the two vertical axes in the figure. The symbols are tied to the horizontal scale to show the locations where all Type I thresholds are met. These locations make good candidates for placement of WIM sensors.

In a similar manner, researchers identified locations where one or more of the computed WIM indices exceed the upper thresholds of the AASHTO MP-14 provisional specification. As before, these locations are identified by the diamond symbols in Figure 3.32. Researchers ran the same analyses on the other lanes of the Balmorhea project and plotted the results in Figures 3.33 to 3.35. Researchers then examined these figures to identify candidate 500-ft WIM sections on all travel lanes that satisfy the recommended guidelines on pavement smoothness given earlier in this chapter. Furthermore, researchers looked for sections that satisfy the geometric requirements for a WIM installation and permit the installation of WIM sensors at the same longitudinal position across all four lanes of the highway. From these considerations, researchers came up with candidate 500-ft WIM sections on the Balmorhea project. For each lane, this section is identified by the vertical line labeled *WIM* in Figures 3.32 to 3.35. This vertical line denotes the proposed location of the WIM sensor on the given lane.

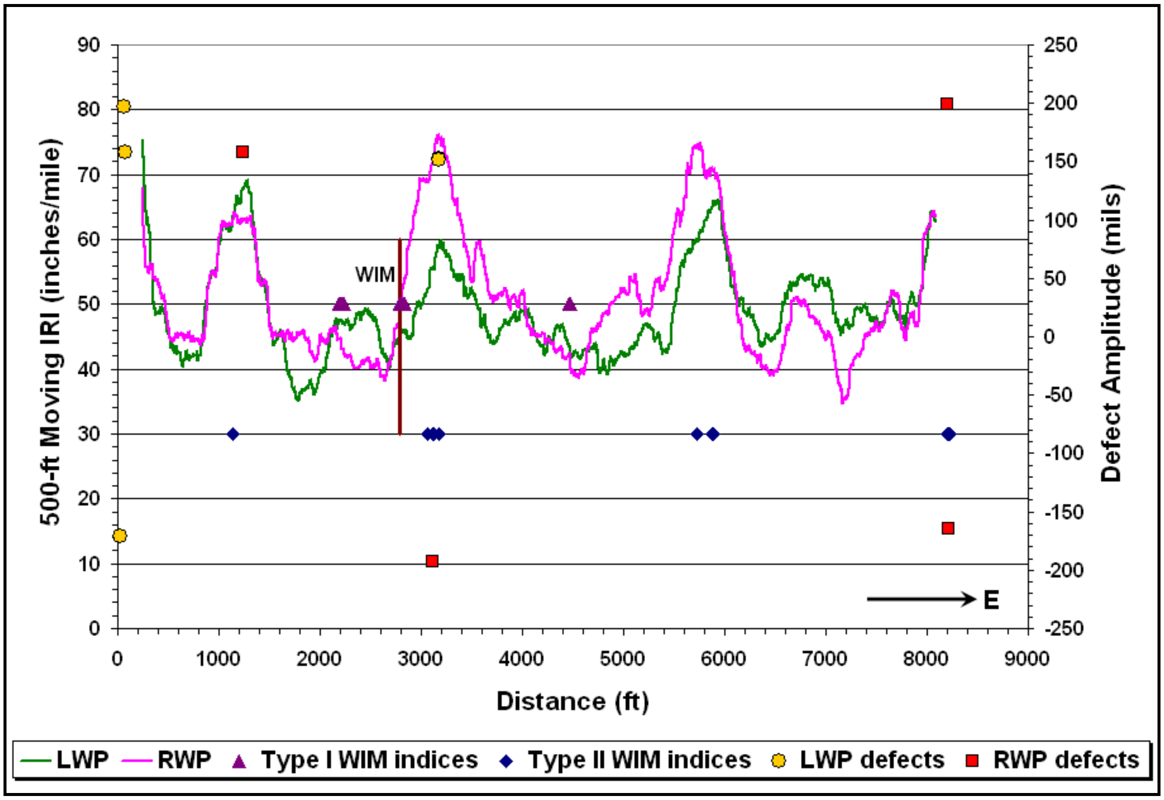


Figure 3.32. Results of Profile Analysis on Eastbound Outside Lane of Balmorhea Project.

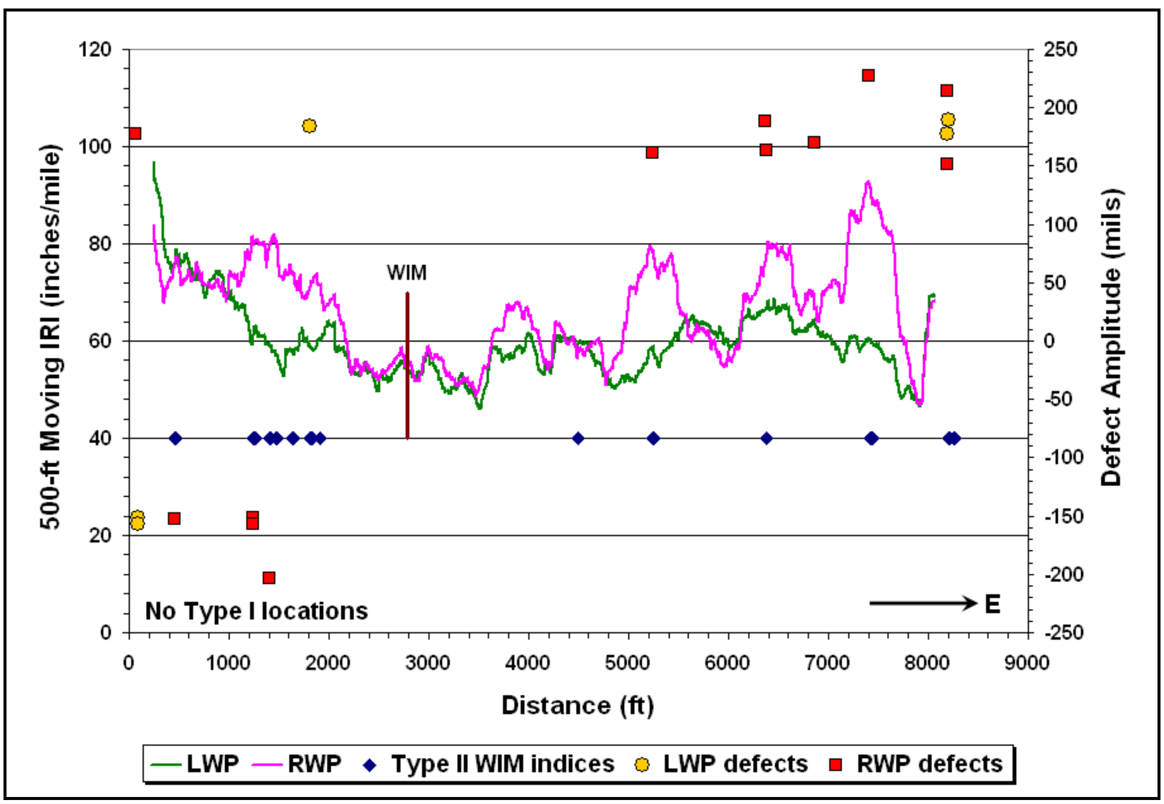


Figure 3.33. Results of Profile Analysis on Eastbound Inside Lane of Balmorhea Project.

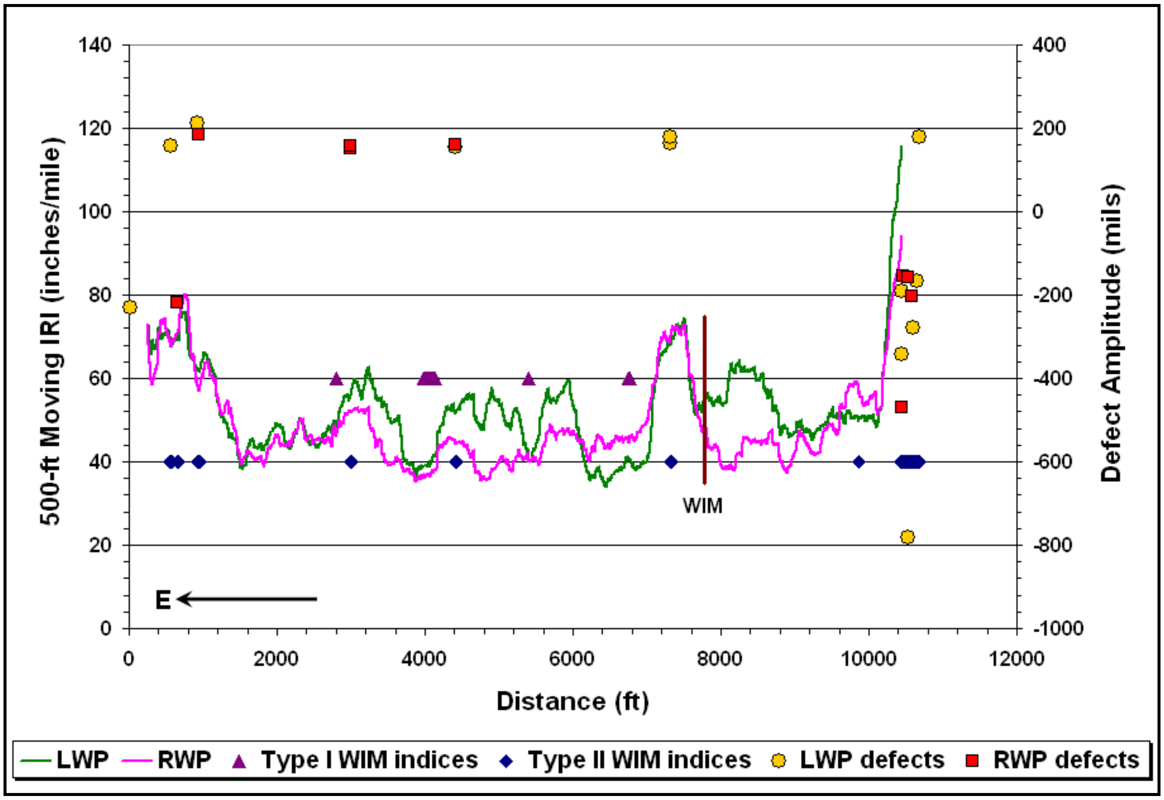


Figure 3.34. Results of Profile Analysis on Westbound Outside Lane of Balmorhea Project.

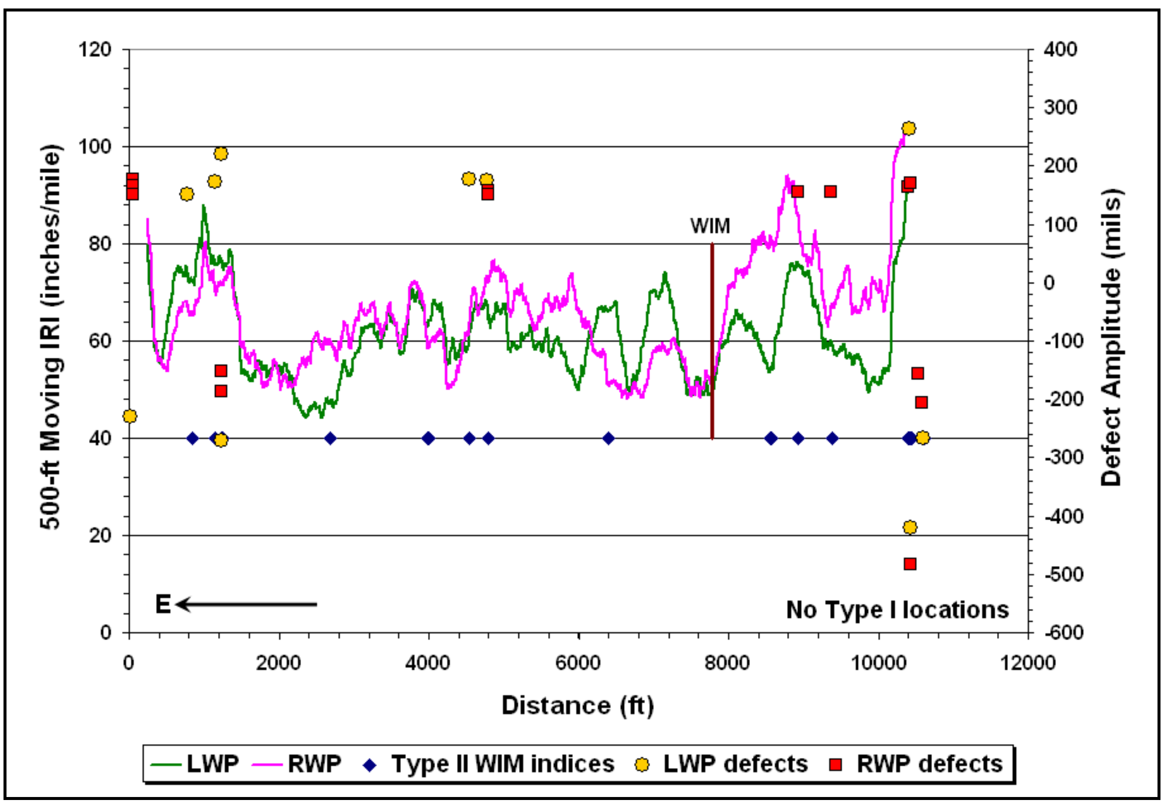


Figure 3.35. Results of Profile Analysis on Westbound Inside Lane of Balmorhea Project.

In general, researchers selected sections with wheel path IRIs less than 60 inches/mile, with no defects found on each wheel path based on the existing TxDOT ride quality bump template. In addition, locations that meet the AASHTO MP-14 Type I criteria make good candidates for installation of WIM sensors. An example can be found on the eastbound outside lane of I-10 where researchers identified a candidate section as illustrated in Figure 3.32. However, when Type I locations are found, researchers recommend checking each wheel path for the presence of defects over the 350-ft approach to the candidate WIM sensor location and the 150-ft leave-out past that location. If defects are found within this 500-ft interval, researchers recommend that the engineer consider another WIM sensor location. In addition to these guidelines, the engineer is advised to avoid sections where placement of the WIM sensor would coincide with locations where one or more of the WIM smoothness indices exceed the upper thresholds of the AASHTO MP-14 specification.

Based on the above smoothness considerations, researchers identified potential 500-ft WIM sections on all four lanes of I-10. On the eastbound travel lanes, the sections begin at about 2440 ft from the west end of the Balmorhea project. On these 500-ft sections, the left and right wheel path IRIs are 40.3 and 40.5 inches per mile on the eastbound outside lane and 54.7 and 57.0 inches per mile on the inside lane. On the westbound lanes, researchers located the start of the 500-ft WIM sections at about 7435 ft from the east end of the project. On these sections, the left and right wheel path IRIs are 51.4 and 51.6 inches per mile on the westbound outside lane, and 49.1 and 50.4 inches per mile on the inside lane. Within each 500-ft section, no defects were found on each wheel path based on the current TxDOT ride quality bump template. All candidate sections are located along a straight tangent of I-10. On all sections, researchers located the WIM sensors such that the sensors line up across all four lanes of the highway.

Researchers also examined GPR data collected over the Balmorhea project and found the GPR data to be clean with no intermediate reflections coming from within the HMAC layer. Figures 3.36 and 3.37 illustrate the GPR data for the eastbound (R1) and westbound (L1) outside lanes. For each lane, the Odessa District collected GPR data on each wheel path and along the lane centerline. In Figures 3.36 and 3.37, the reflection at about 4 inches down from the surface is the bottom of the HMAC layer or the top of the cement-treated base. As noted, the GPR data for the hot-mix layer look clean.

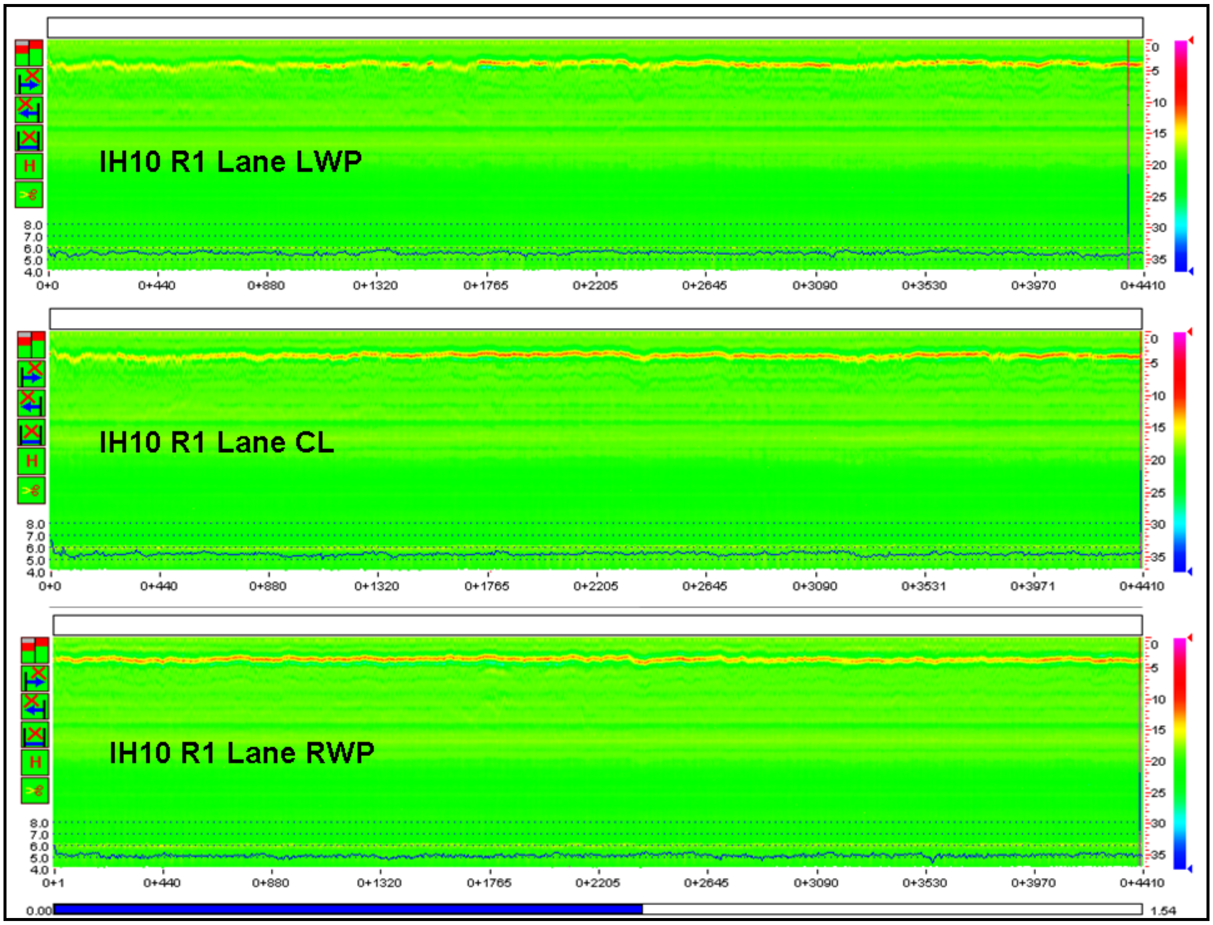


Figure 3.36. GPR Data on Eastbound Outside Lane of I-10 at Balmorhea.

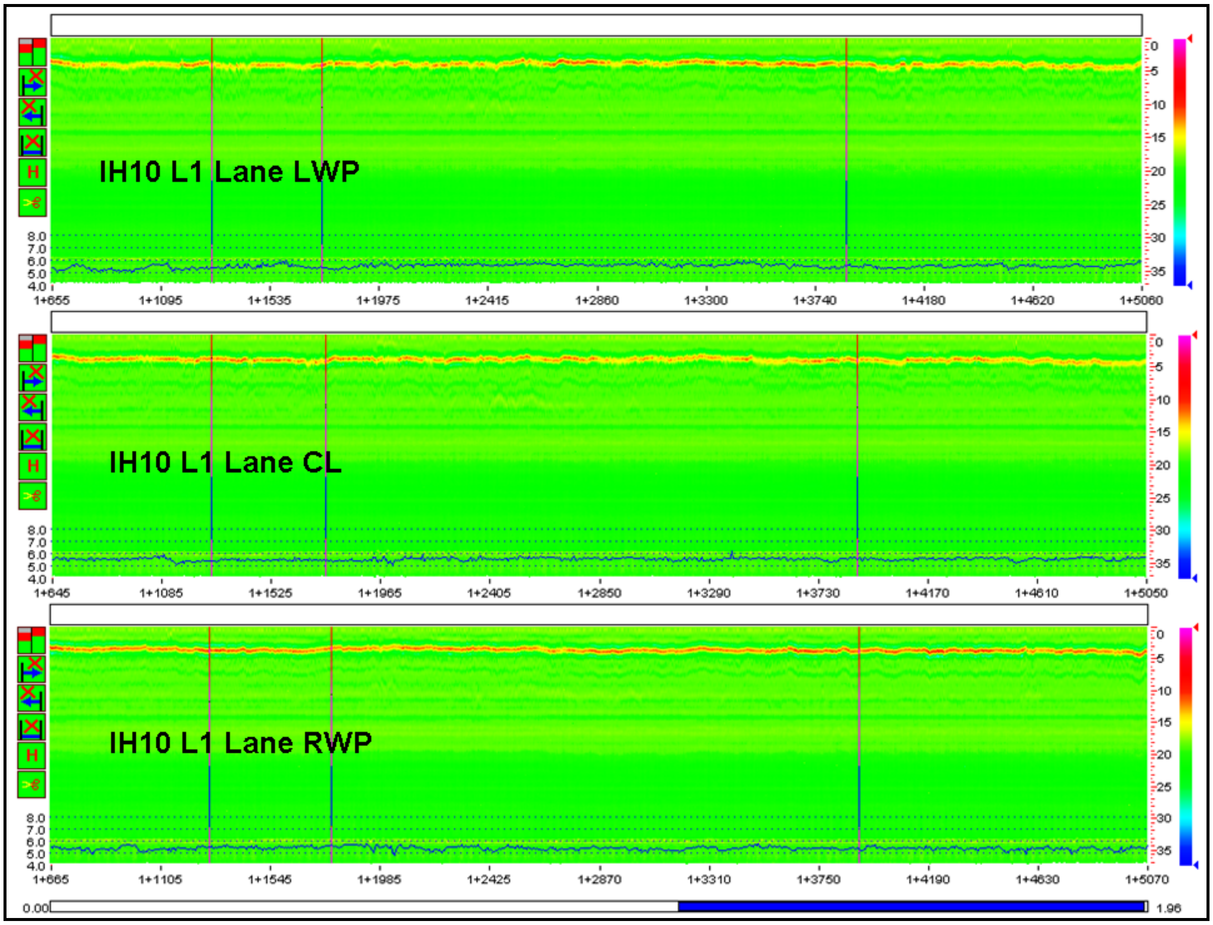


Figure 3.37. GPR Data on Westbound Outside Lane of I-10 at Balmorhea.

With assistance from the Odessa District, researchers also collected cores at a number of locations on all four lanes of the Balmorhea project. Researchers then ran the overlay tester on selected cores to check the crack resistance of the SPHMACP C mix placed on the project. Table 3.18 shows the number of load cycles to failure from these tests. It is encouraging to see that three of the four specimens met the proposed minimum criterion of 300 load cycles. Researchers note that cores were taken away from the proposed WIM sensor locations so as not to introduce features that could create roughness, particularly within the 350-ft approach to the WIM sensor on each lane. In addition to the overlay test results, researchers also verified from the Odessa District Laboratory that the mix placed on the project passed the Hamburg test.

Table 3.18. Results from Overlay Tests of Specimens Cut from I-10 Cores.

Lane	WIM Section Limits		Core Location (ft)	Number of Load Cycles to Failure
	From	To		
Westbound outside lane	7435	7935	7900	315
Westbound inside lane	7435	7935	7900	304
Eastbound outside lane	2440	2940	2300	373
Eastbound inside lane	2440	2940	2100	231

With assistance from district staff, researchers also collected FWD data on the proposed 500-ft WIM sections. The typical cross-section sheets from the project plans showed a pavement structure consisting of four inches of SPHMACP C, overlying eight inches of cement-treated base, over six inches of the original flexible base material. On this pavement structure, the FWD sensor 1 deflections at drop height 2 varied from about 4 to 17 mils over a range of pavement temperatures from 79 to 113 °F. Tables D17 to D20 in Appendix D present the measured FWD deflections on all four lanes while Tables D21 to D24 present the backcalculated layer moduli. Researchers used these modulus values in OTRA to estimate the pavement design reliability. The results, given in Table 3.19, show that all four lanes meet the minimum recommended reliability of 95 percent for both fatigue and rutting criteria, with all FWD test locations predicted to last more than the minimum required 10-year pavement life for the projected traffic at the site. Thus, based on the FWD deflections, the pavement structure on the candidate WIM sections appears to have enough structural capacity to provide the minimum service life required of a WIM installation. This finding is also consistent with the results from a remaining life analysis made with the MODULUS program. This analysis showed that the proposed sections exhibit good to very good layer strengths on the basis of the FWD deflections as summarized in Table 3.20. In addition, the sections are predicted to have over 10 years of remaining life.

Table 3.19. Results from Pavement Design Reliability Analysis of I-10 WIM Sections.

Lane	Estimated Reliability (%)		Number of FWD stations with less than 10 years predicted life*	
	Fatigue	Rutting	Fatigue	Rutting
Westbound outside lane	98	99	0	0
Westbound inside lane	≈100	≈100	0	0
Eastbound outside lane	96	≈100	0	0
Eastbound inside lane	96	≈100	0	0

* 51 FWD test locations per lane

Table 3.20. Remaining Life Analysis Results from MODULUS Program.

Lane	Layer Strength			Remaining Life (years)	
	UPR	LWR	SGR	Rut	Crack
Westbound outside lane	VG	GD	GD	10+	10+
Westbound inside lane	VG	VG	VG	10+	10+
Eastbound outside lane	VG	VG	GD	10+	10+
Eastbound inside lane	VG	VG	GD	10+	10+

Researchers also verified the longitudinal grade at the proposed WIM sections. Initially, researchers obtained and reviewed the original alignment on this I-10 project. The historical information showed that the original centerline grades are all within ±2 percent throughout the project. At the suggestion of the project director, researchers conducted a Walking Profiler/rod and level survey to verify the longitudinal grades at the proposed WIM sections. These measurements were made in the direction of traffic along the centerlines of the eastbound and westbound outside lanes. Using the measured profiles, researchers determined the variation of the slopes on each lane surveyed. On the eastbound outside lane, the slopes determined range from -0.89 to 0.08 percent with an average slope of -0.39 percent. On the westbound outside lane, the slopes range from -0.39 to 0.59 percent with an average slope of 0.19 percent. These slopes are all within ±1 percent, indicating that the longitudinal grade at the proposed WIM location is acceptable.

In view of the preceding test results, researchers recommended that TPP establish a WIM installation on the Balmorhea project. For this purpose, researchers marked the proposed WIM sensor locations as well as the limits of the 500-ft WIM section on each lane. Appendix E provides information on where the proposed WIM sections are located along I-10 of the Balmorhea project. The project director and the researchers also worked with the Odessa District to provide the ground bore for routing wires underneath the highway, to build the slab for the controller box, to place ground boxes, and to erect the utility pole for the solar panel to be used for powering the WIM system at the site. For this WIM installation, research funds allocated in FY08 were used to purchase a 4-lane Kistler quartz WIM system, following specifications provided by TPP staff who installed the WIM system at the recommended locations in August 2008. Since this is the first TxDOT WIM site that uses solar power and wireless communications for data transmission, researchers propose a follow-up project to monitor the performance of the WIM system.

CHAPTER IV. EVALUATE WIM, POWER, AND COMMUNICATION TECHNOLOGY

INTRODUCTION

One task of the research work plan aimed to select, install, and test a new weigh-in-motion sensor, and then to install and test wireless communication and power for the selected system. Early in this selection process, TxDOT asked TTI to evaluate a microwave WIM sensor and accompanying software that had been developed at the University of Houston (UH) in an earlier research project. TTI agreed to conduct the tests as long as UH could provide the system and provide staff support during installation and testing of the system on an as-needed basis. However, UH could not provide the microwave WIM system, so TTI finally recommended choosing another sensor technology. During this decision process, and while waiting for UH to provide the WIM system, TTI continued developing specifications for purchasing a solar panel and wireless modem to be used with the selected WIM system. Assuming that the UH sensor would be used, TTI ordered the solar panel to match the potential power requirements of the PC that would be required to operate the UH system. Therefore, the solar panel was over-designed for its final use with a standard WIM system, but that was not an issue. The final decision was to work with the Kistler Lineas quartz WIM sensors, considering that TxDOT had already installed the same types of sensors along the perpetual pavement lanes of I-35 south of Cotulla. In addition, TPP has developed confidence in these sensors in terms of accuracy. In fact, they were the only sensor respected by TPP from both an accuracy standpoint and performance on asphalt concrete pavements. The same sensors had also been installed in other DOTs with reasonably good results.

KISTLER LINEAS WEIGH-IN-MOTION SENSORS

The Kistler Lineas is a quartz sensor used for measuring wheel and axle loads under moving vehicle conditions. The specific sensor installed at the Balmorhea site during this project is a Type 915E. Kistler sensors consist of a light metal material fitted with quartz discs which are under preload. When an external force is applied to the surface of the sensor, the load causes the quartz discs to yield an electrical charge proportional to the applied force through the piezoelectric effect. A charge amplifier converts the electric charge into a proportional voltage that can be measured and correlated with the applied force.

Other States' Use of Kistler WIM Sensors

Results from other states that have used Kistler sensors for weigh-in-motion were helpful in deciding whether or not to install them in Texas. The review of current WIM practice presented in Chapter II documents the experience of several state DOTs with using these sensors. To recap, some of the earliest known tests of Kistler WIM sensors occurred in Connecticut. The overall assessment of the sensors at the end of the second year of testing concluded that the sensors produced good weight data. However, more work needed to be done to determine why sensors in some lanes were not performing as well as those in other lanes. The source of this result made no firm conclusion as to the expected life of the sensors (McDonnell, 2000).

Illinois DOT has used Kistler sensors in its Pre-Pass system as a sorter to determine the need for static weighing. IDOT began installing these sensors around 1999 or 2000. The department's experience with these sensors is rather mixed. The average life of the sensors based on the Illinois experience was about two years. IDOT calibrates the Kistler sensors about three to four times per year, typically based on complaints from Pre-Pass personnel. IDOT uses hydraulic load cells at 17 of its 20 interstate weigh stations. The literature review revealed that the department uses no bending plate systems. Overall, IDOT prefers load cells because from experience these sensors did not fail as often as the Kistlers or bending plates, and the department did not have to request replacement money as often. However, it is interesting to note that for future WIM installations, IDOT plans to replace failed sensors with new Kistlers in most cases. IDOT plans to replace some Kistlers with load cell systems. Kistlers installed by IDOT in concrete pavements seem to last longer and perform better than in asphalt pavements (Middleton et al., 2005).

Maine DOT had 13 WIM stations installed with Kistler sensors for a total of 132 sensors. During a five-year period during which sensors were being installed, there were 18 quartz sensor failures. However, on a positive note, Maine DOT has found the Kistler sensors to be "extremely accurate." The department has calibrated sensors to within two percent error compared to the test vehicle gross weight. Maine DOT plans to continue using Kistler sensors even though their failure rate is of significant concern (Middleton et al., 2005).

The Michigan Department of Transportation had a total of 30 lanes at 8 sites that used Kistler sensors for weigh-in-motion data collection. In 2004, the oldest of the Kistlers were three years old, and the most recent installations occurred in early October 2004. Only six of these lanes were in asphalt pavements with the oldest installed about 1.5 years earlier. One of these installations was in a six-inch asphalt overlay with concrete underneath. MDOT had no failures considered to be the fault of the sensors (Middleton et al., 2005).

Balmorhea WIM Installation

The elements to be evaluated in this task included wireless communication and solar power components, combined with Kistler Lineas WIM sensors. For the permanent installation of these components, the research team recommended a site on I-10 just west of Balmorhea, Texas, near mile marker 203. This site has asphalt concrete pavement on all travel lanes, is located on a tangent section of the roadway, and has a slight constant longitudinal grade that is within ±1 percent at the selected WIM site. The site met these and other criteria discussed in Chapter III, and had sufficient truck volume to serve as a TPP WIM site as well as the research project needs. Researchers had considered other sensors but none were found to be as promising as the Kistler sensors. The FY 08 project budget paid for a four-lane Kistler WIM system for this installation. TxDOT installed a new cabinet which housed a new PAT DAW 100 weigh-in-motion unit. The Type 6 cabinet also had sufficient space in the bottom for all six of the batteries purchased with project funds to power the WIM and modem system. TTI had also purchased a smaller cabinet, which was originally intended to hold the batteries and to be attached to the timber pole supporting the solar panel. However, placement of batteries in the larger cabinet provided by TxDOT simplified wiring and reduced the wiring runs significantly.

Figure 4.1 shows the layout of the Kistler sensors as installed at the Balmorhea site. In addition to the sensors shown on this drawing, TPP also installed one 6-ft × 6-ft inductive loop upstream of the WIM sensors in each of the four lanes. These four loops will serve as independent count detectors for use by TPP as an automatic traffic recorder (ATR) site. In case problems and/or data loss occur with the WIM system, TPP will still have vehicle counts from the inductive loops. The loops will also provide vehicle counts that can serve as a comparison with counts from the WIM system.

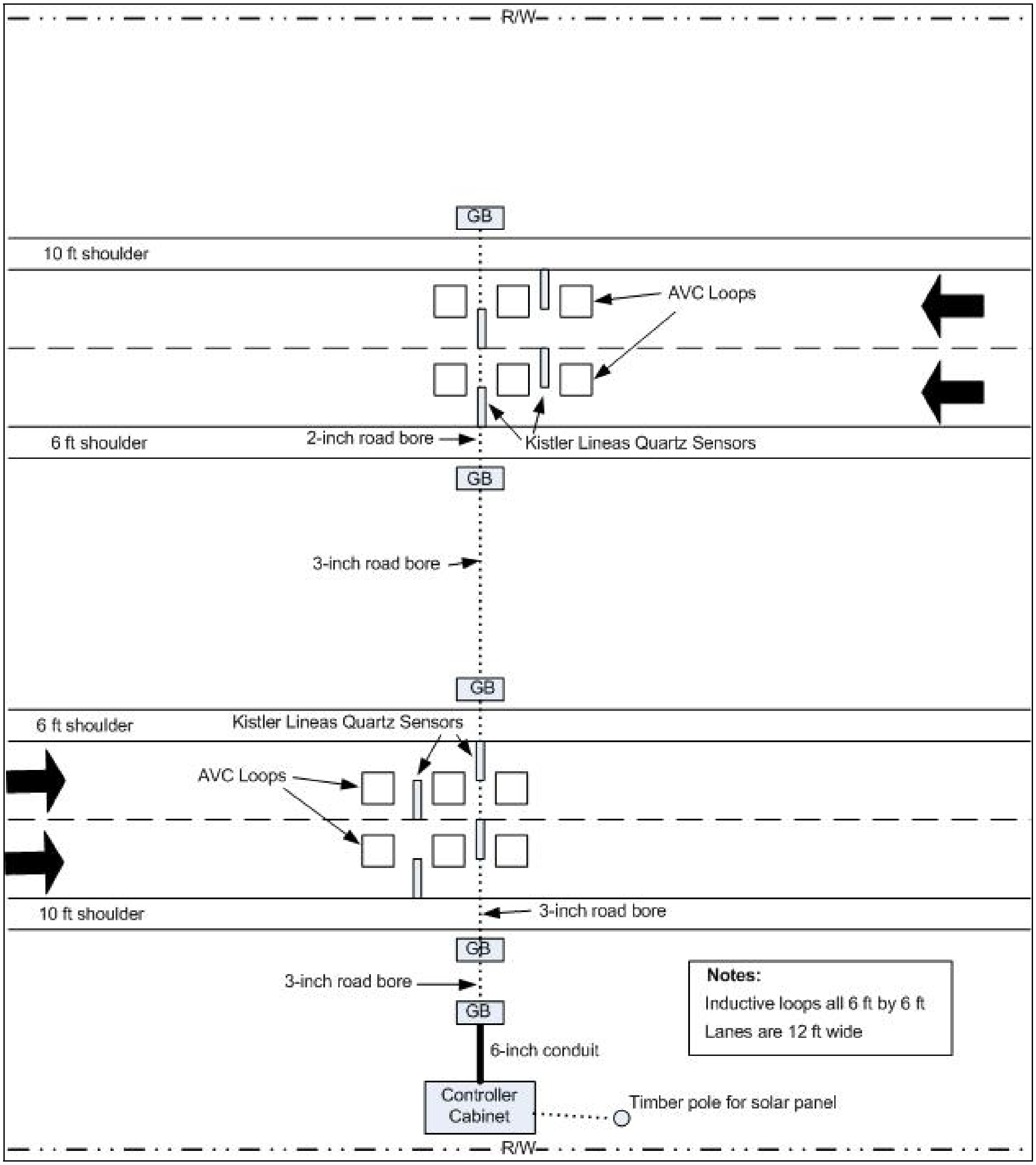


Figure 4.1. Schematic of WIM Sensors Installed on I-10 near Balmorhea.

On August 12, 2008, TPP began installation of the quartz WIM site at mile marker 203 near Balmorhea, Texas. This section of I-10 has a total of four lanes with two in each direction. It has a wide depressed earth median that is about 100 ft wide. Figure 4.2 is a photo of this section. TxDOT traffic control from the Odessa District closed the right westbound lane beginning at about 8:00 a.m. TPP completed all work on the right lane before closing the left westbound lane later that same day. However, TPP did not complete all of the work in the left lane until the following day. They were able to mark all saw cuts and lay all the loop wire for that lane on the first day, but did not cut the quartz sensor slots until the third day (August 14).

Figure 4.2. Photo of Balmorhea WIM Installation.

The sequence for installing all lanes was generally as follows, although some activities occurred simultaneously:

- Mark the positions of all sensors with red paint using a straightedge to guide the pavement saw operator.
- Cut all inductive loop cuts.
- Cut the quartz slots (3 inches wide by 2.25 inches deep).
- Clean the saw cuts with a water pressure washer.
- Blow dry the cleaned saw cuts using compressed air.
- Place four turns of loop wire in the loop saw cuts.

- Backfill cuts and wire with loop sealant.
- Connect two Kistler quartz sensors end-to-end (1.0 m plus 0.75 m lengths).
- Record the serial numbers of each sensor.
- Mix and pour epoxy into the sawed slot ensuring uniform depth.
- Place the two connected sensor elements into the epoxy such that the top of the sensors are flush with the pavement surface.
- Allow time for epoxy to cure.
- Grind the epoxy and sensors to be flush with the road surface.

Checking the sensors in accordance with pre-installation checks might have occurred prior to installing them in the roadway, but no special testing was observed.

The pavement surface in the vicinity of the newly installed WIM sensors was smooth and was not noticeably or measurably rutted from heavy wheel loads. Only lane 4 (closest to the cabinet) had a slight dip between the shoulder and the outside edge of the through lane, but it was not due to rutting. It appeared to be simply a difference due to final rolling of the hot mix surface. In addition, analysis of the site profiles presented in Chapter III did not show defects in any of the wheel paths of the travel lanes.

TPP installed the sensor in the right wheel path so that it matched the pavement cross-section perfectly at all points except the very end of the sensor nearest the outside lane line. When they poured the epoxy into the slot and forced the sensor downward into the epoxy for curing, the rightmost end of that sensor set was slightly lower than the pavement surface for a distance of 3 to 4 inches from the end of the sensor. Matching the pavement surface for the entire sensor length left a thin layer of epoxy over this final 3 to 4 inches of the sensor's length. Since the lateral position of the sensor placed the end of the sensor at the lane line, the normal outside wheel path of passing vehicles was beyond the end of the sensor by more than 3 to 4 inches. Therefore, researchers do not think this issue to be problematic.

The only discrepancy noted between the installation by TPP personnel and that recommended by Kistler was the use of water during the saw cutting process. Kistler recommends using a dry cut process. The reasons for using water are well founded and include reduction of asphalt "dust", a known carcinogen, and increased life of saw blades. With either the dry or wet cut process, the installation crew must be careful to clean all surfaces of dust and debris for a proper bond between pavement and epoxy. Using the wet cutting process leaves water standing in saw cuts, which requires blowing saw cuts with compressed air for an extended period of time to clean and dry the saw cuts. For inductive loop cuts, some literature sources claim that sufficient drying requires both the compressed air procedure and a 4-hour wait time for additional air drying before applying epoxy. However, the practicality of this long wait must be questioned, especially on high-volume facilities.

The installation in Balmorhea took the TPP crew 2.5 days. The pace increased after installing the first lane because the crew had not installed a Kistler WIM system in several months and needed the first lane to re-familiarize themselves with the process. In the time period spent on site, the crew was able to install all sensors and reseal the pavement over the saw cuts. With support from a TxDOT bucket truck operator and his assistant from Odessa, TTI was able to install the solar panel, wireless modem, batteries, and related hardware on the south side of the site during this same time period. Upon leaving the site, TPP still had to return later and hook up all wiring to the PAT WIM and do the WIM calibration.

For the initial calibration of the sensors in Balmorhea, TPP intended to use the auto-calibration feature of the PAT system. For auto-calibration, TPP will probably set the PAT system to use steering axles of Class 9 trucks due to their consistency under various loading scenarios. The consistency of the new quartz sensors over time will be important as well as its initial accuracy. The data collection should include periodic samples of data (e.g., once monthly) for several months to evaluate any degradation in sensor performance. Testing of the fluctuation of sensor performance against temperature is recommended, just like what was done at the Cotulla WIM site.

Final calibration of the Balmorhea site will require the use of the TPP calibration truck. It will enhance the evaluation since its static axle loading is known. However, this final calibration had not been completed near the end of this research project so any further evaluation of sensor performance by TTI would require a modification to extend the project. Ongoing tasks following installation and calibration of the Kistler sensors would include documenting all detectable changes in sensor performance. The primary data to be recorded from the sensor on a continuous basis would be axle weights for all vehicles. The research team and TPP would need to decide the most appropriate additional parameters to monitor. This record could be anything from simple voltage readings as vehicles pass, to oscilloscope plots or others. One of the best ways to detect degradations in sensor signals is to record axle load applications using very high sampling rates of sensor signals and storing the generated plots for subsequent comparison. Under highly controlled conditions, one can identify any discrepant signals, which could indicate sensor degradation. In this case the best control must ensure speed and load consistency, suggesting the use of a calibration truck of known axle loads and controlled speeds.

POWER AND COMMUNICATIONS FOR WIM

Introduction

The traditional solution for power and communication at WIM sites has been to gain power from the commercial electric grid and to obtain communications via land line dial-up service. Although this solution is sound and is still widely used, other options are now available to complement the current practice. Purchased power from a nearby provider typically results in a reasonably high level of service reliability, but many times the cost for bringing power to the specific spot from the grid is significant. The same situation exists for telephone line service. The local telephone company may charge to bring their service to the WIM site. Additionally, telephone service outside of metro areas may not provide the same level of reliability as that inside metropolitan areas.

Solar is a common alternative power solution for rural deployments. Solar deployments are common across the state for devices such as school zone flashers and warning signs. Solar is a viable option given reasonable power needs from the system it is supplying. Obviously, solar is not a good choice for a high power draw system or even a system which requires medium levels of power continuously 24 hours per day. Based on past experience and equipment specifications, WIM systems tend to be low power consumption equipment and thus lend themselves to solar applications. The overall power needs for a WIM site (including the WIM equipment, communications, and support devices or hardware) have to be reviewed and precisely calculated.

Today, numerous communication options could have been applied to support a WIM site, but wireless cellular technology was considered the most attractive alternative to traditional dial-up. Current cellular systems (Sprint, Verizon, AT&T, T-Mobile, etc.) are digital in nature. Being digital, the ability to handle true data across a modern cellular network has increased significantly over the capabilities of older analog cellular networks. Cellular deployment is widespread throughout the state with most major roadways covered in their entirety. Data only cellular services are now available that can deliver the bandwidth required by a WIM installation.

Given the choices reviewed for WIM, the most reasonable alternate to wire line dial-up appeared to be digital cellular. Standalone cellular modems are sold by several vendors and are designed to survive in a harsh environment. The devices are used in the oil and gas pipeline industry, an application with very similar environmental requirements. The modem supports an Ethernet interface, which is by far the most flexible interface in the marketplace. A terminal server, a device which converts one or multiple serial channels to Ethernet, can be connected to the cellular system. Terminal servers today have been designed to be a drop in replacement for a telephone modem (modem emulation). They are designed to respond to commands that would normally be sent to dial-up modems. The advantage lies in integration with the WIM back office system. In general, the terminal server over network solution will appear to the back office equipment as just another landline dial-up.

At the outset of this research project, the research team did not anticipate needing to demonstrate the adequacy of solar power in being able to power a WIM system. However, TTI investigated the costs of the components necessary to provide solar power including the panel size typically used and other major components such as batteries. The solar panel for this WIM project costs about $1000, although as noted previously, it was somewhat over-designed for the PAT system installed in Balmorhea. As a general rule, this cost will usually be much less than hookup to the electrical grid. Based on discussions with the project director, TTI conducted a test of the solar system at its SH6 test site to power an ECM WIM system installed there. The project budget included the cost of the solar panel and communication equipment.

The use of digital cellular systems for this application were not as well understood as solar power, so TTI conducted field tests to demonstrate its use and to better understand its strengths and weaknesses. TTI determined the details needed for an adequate test based on information gathered from other states, the team's expertise, and guidance from the project director. Early issues surrounding the request for a static network address [known as a static Internet Protocol (IP) address] caused significant delay and limited the amount of testing that could be done. However, the outcome of this activity included information to help TxDOT make decisions regarding the technology. For example, the monthly cost associated with the service is higher than that for standard telephone lines, but its bandwidth is significantly greater with no build-out costs. There may be ways to couple multiple data transmission needs within a focused geographic area to more fully utilize the bandwidth and make this an overall better choice for TxDOT in some cases.

Identifying and Selecting the Cell Modem

TTI spent a considerable amount of time investigating cell modems that might adequately serve the needs of this project. One of the attributes that was deemed to be critical was that the cell modem had to have a static IP address. Otherwise, the modem could go off-line, and the system could assign a different IP address than the one that was available for setup. The data retrieval software must know where to find the WIM station on the network (its address), and therefore the address must not change unexpectedly. Given that the future WIM site where this modem would be installed might be at a considerable distance from TxDOT or TTI headquarters, the system had to be robust and predictable. Acquiring a fixed IP address from the selected provider turned out to be a more time-consuming and arduous task than anticipated.

The modem selected for this project was a Digi Connectport WAN VPN HSDPA (Model CP-WAN-B302-A) with a wireless module for AT&T service. The modem cost $950 from CDW.com. The CDW website describes the modem as follows:

> *The ConnectPort WAN VPN is an upgradeable, commercial-grade 3G cellular router that provides secure high-speed wireless connectivity to remote sites and devices. It can be used for primary wireless broadband network connectivity to equipment at remote locations, as well as for a backup to existing landline communications. Applications include utilities, industrial automation, POS/retail, financial (ATMs), traffic, medical, video surveillance and more. With an upgradeable wireless network platform, customers are able to quickly migrate to future 3G platforms and beyond with supported Type 2 PCMCIA Card slots or PCI Express modules. Also included are two RS-232 serial ports for connecting legacy COM devices and a built-in four-port 10/100 Ethernet switch for connecting additional TCP/IP network devices.*
>
> *ConnectPort WAN supports both CDMA EvDO and GSM HSDPA/UMTS 3G networks. Digi Connectware® Manager, an enterprise remote device management software, enables users to easily view, configure and monitor one or thousands of remote devices, receive email alerts, integrate with DNS server, or incorporate with your network management software. For applications requiring secure connections, ConnectPort WAN VPN offers an available integrated IPsec VPN client/server for true end-to-end data protection.*

Other States' Use of Wireless Components

At this juncture, it is worth recalling the experience of other state DOTs with using wireless components in their WIM installations. To recap from Chapter II, five of the total 109 WIM sites operated by the California Department of Transportation use wireless communication equipment, although none used solar panels. Contacts with CALTRANS revealed the department's good experience with the wireless systems.

At the Florida DOT, all WIM systems are powered by solar panels. The department has had occasional problems with trees obscuring the panels, vandalism, and under-estimation of power requirements at some sites. As an example of the wattage that will suffice for WIM, FDOT used two 85 watt solar panels to power one site. For count/classification sites, FDOT has only one out of 300 sites that is tied to AC power. Of the 300 count/classification sites, 193 have now been converted to use wireless communication. FDOT personnel estimate that 20 to 30 additional sites are accessible to service and will be converted at some future date.

Verizon has the state contract for service in Florida, so the state is using Raven modems at most of its sites. One common problem with wireless communication is that service is not available everywhere. In Florida, most of the interstate is covered but there are some remote areas, even along the coast where service is not available. Overall, FDOT is pleased with wireless communication.

Minnesota Department of Transportation does not generally use wireless systems for WIM although one wireless communication system was under test in July 2007 at one enforcement site. Solar panels are used at some classification sites but not at WIM sites.

Ohio Department of Transportation does not have any wireless communication at WIM sites but has used it successfully for some time at automatic vehicle classification (AVC) sites. One reason the department has hesitated in using wireless communications at WIM sites is the larger amounts of data that would be sent, although the concept has worked well at less demanding sites. One other reason wireless has not been expanded is that when these systems were installed several years ago, wireless was not as reliable as it is today.

College Station Installation

TTI temporarily installed the solar panel at TTI's SH6 WIM test bed in College Station in January 2008 and monitored its operation by connecting it to an ECM WIM system located there. Its operation continued until early August, so the total observation time at the site was almost eight months. TTI observed no problems in powering the WIM with the solar panel while this test was underway. The unknowns when specifying the size of the solar panel included the exact type of load sensor and which WIM electronics would be used. One of the load sensors being considered at the time of purchase would have required the use of a PC at the field location, which would require a larger panel than a standard off-the-shelf WIM. For that reason, TTI purchased a larger panel size than was actually needed once the final sensor and electronics were decided. The solar panel was a 115 watt panel. The SH6 setup was intended to be temporary until the project team identified a WIM site for a more permanent installation.

Table 4.1 is a log of events related to testing the cell modem at the SH6 WIM test bed. Problems within Texas A&M University in acquiring a static IP address resulted in delays of a few months before service was actually established. The DigiConnectPort typically self-detects that it has trouble and attempts to reconnect (log entries show that unit identifies error state). The reconnects were not always successful, but a simple power cycle returns the unit to service. Researchers concluded that the DigiConnectport did not demonstrate through this test that it would be reliable on a long-term basis without an occasional reboot – either remotely or by an on-site automated system.

Table 4.1. Cell Modem Activities during SH6 Test.

Date	Activity
05/21/2008	Placed Digi Connectport into service at SH6. TTI connected modem to record per-vehicle records for several hours.
05/27/2008	System was inoperable.
05/28/2008	Digi internal software detected the system inoperable but was unable to return it to service online with the cellular network. Internal soft reboot via laptop on the LAN side was ineffective in returning unit to service.
06/02/2008	Morning check showed unit not responding to ping or Wavetronics software.
06/03/2008	Unable to get to WAN from LAN – cycled power.
06/06/2008	Checked at 10:12 am, and the system responded to pings.
06/17/2008	Checked at 9:30 am, and system did not respond to pings.
08/01/2008	Local vendor uploaded new firmware to the DigiConnectPort .

TTI installed the modem in College Station in May 2008 and monitored its operation through mid-August 2008, a period of almost three months. During that period of initial observation in College Station, the only problem with the modem was the need to travel to the site two or three times to “cycle power” to revive the modem and return it to normal operation. To cycle power simply means powering the unit down and then back up. Researchers concluded that, for full remote operation, there would need to be a solution to automatically cycle power on the modem on at least a daily basis. Before installing the modem in Balmorhea, TTI investigated the available options and ordered a set of relays that would operate on 12 volts of direct current (DC) supplied through a solar panel/battery system. TTI had used alternating current units in earlier research but found that previously used solutions would not work with DC. The solution for DC required two timer relays purchased from B&B Electronics at a cost of $44 each. The model number for these relays is 821TD10H-UNI.

Balmorhea Installation

The power and communication equipment evaluated in College Station was removed and reinstalled near Balmorhea, Texas, in the Odessa District. The installation utilized all the equipment from the College Station evaluation except the standalone battery cabinet. The larger WIM cabinet at the site was installed on a prefabricated base. The interior of the base presented an excellent location to house the solar batteries. The Odessa district staff indicated they use the interior of prefabricated bases for battery storage at other sites. The batteries were not placed directly on the concrete pad as per direction by the WIM installation crew. Installers placed the batteries on a piece of wood that raises each battery approximately 1.5 inches above the concrete pad. Figure 4.3 is a photograph taken of the inside of the cabinet, showing access at the bottom for batteries. Cables pulled from WIM sensors and connections to the nearby pole-mounted solar panel also come through this opening.

The solar equipment was tested upon completion of the solar panel and battery mounting. The charge controller contains a built in testing meter which alternately shows the electrical current inbound from the solar panel, the charge voltage on the battery bank, and the amount of current delivered to the load (the cabinet equipment). Each item showed expected results (i.e., panel current approximately 4 amps, battery voltage 12.5 volts, load current 0.0 amps). The panel current varies with the charge on the battery bank and the amount of light incident on the panel face. At the time of the power system tests, there was no cabinet equipment hooked up and therefore no significant current draw.

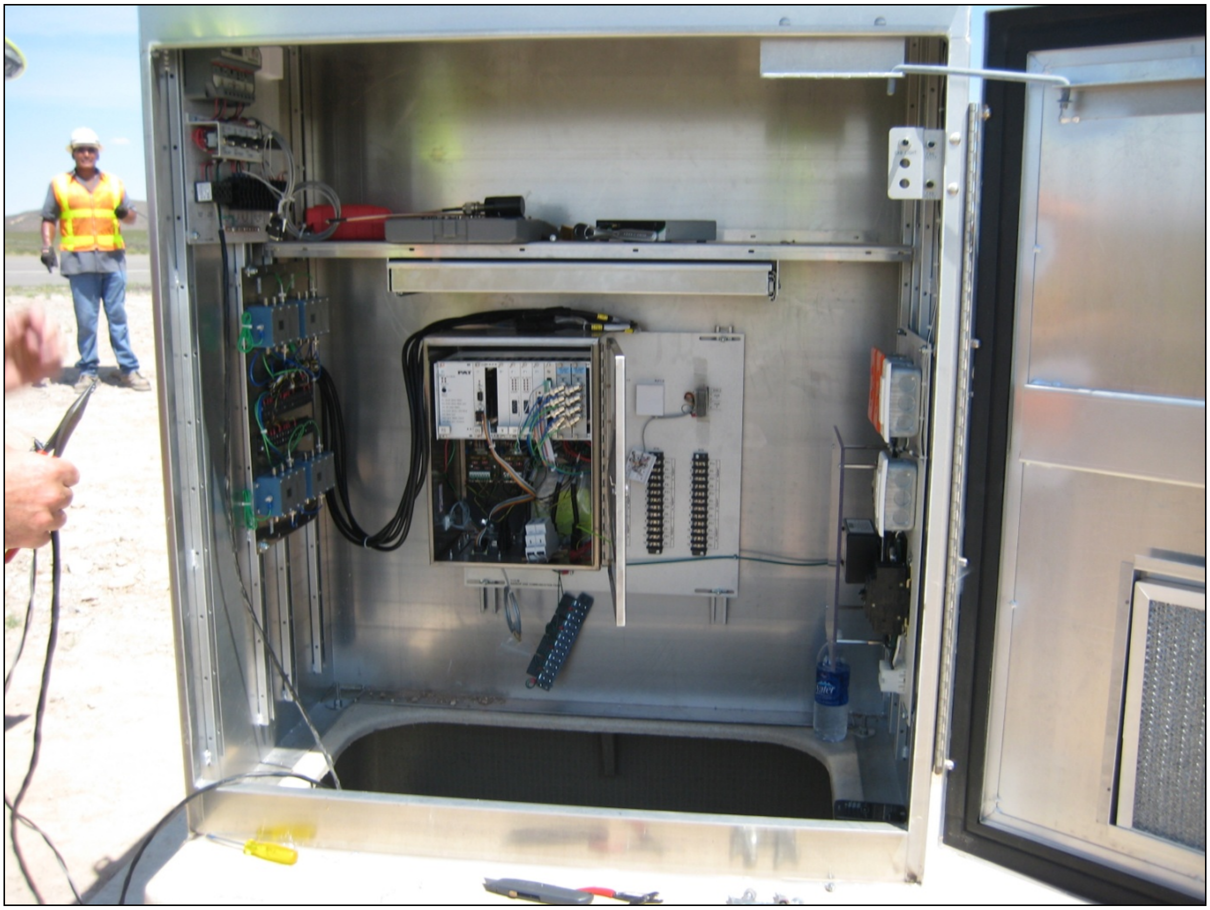


Figure 4.3. Inside the Cabinet Showing PAT WIM Electronics.

An outside antenna was selected for use at the Balmorhea location to ensure the best possible signal strength for the wireless modem. The antenna was mounted on the side of the cabinet by enlarging one of the lifting lug bolt holes. The use of the lifting lug hole allowed for a quick and clean installation without drilling on the top of the cabinet. The small antenna securely snugs down to the side of the cabinet and is nondescript.

For the DigiConnectport modem installed in Balmorhea, the bandwidth up/down is entirely dependent on the cell network it is connected to. The modem is running on the AT&T EDGE network at the Balmorhea site. AT&T states on its website that the EDGE network is available in more than 13,000 cities and towns, and in areas along 40,000 miles of highways. It provides average data speeds from 75 to 135 Kbps.

TTI ran a download test at the Balmorhea site and observed 120 Kbits per second, which was an acceptable number, especially given the remote location. Signal strength was very good (-73dBm) with the external antenna. After returning to College Station, TTI checked the modem and the signal strength was still -73dBm (i.e., very good) and the modem was cycled around the midnight timeframe (i.e., the relay system was working).

The service plan selected by TTI for this service costs $50 per month. This monthly charge was paid through the research project, but TxDOT needs to pick up the service after the project terminates (August 2008). If TxDOT chooses to continue using the modem for the long term, the data upload needs should be determined (megabytes per month) and the best plan selected based on usage. Again, the modem is configured with a static IP for field use. TPP has indicated that it will work with TxDOT's Information Systems Division/Technology Services Division (ISD/TSD) to have the wireless service continued.

CHAPTER V. DEVELOP GUIDELINES FOR WIM INSTALLATION, MAINTENANCE, AND CALIBRATION

INTRODUCTION

The research work plan included a task to develop guidelines for WIM installation, maintenance, and calibration that are applicable to a newly installed WIM system on asphalt concrete pavement or to future WIM systems that TPP plans to install on such pavements. The requirements of this task were determined primarily by the types of WIM systems presently operated by TxDOT and other state DOTs, particularly with respect to the types of WIM sensors that are placed on asphalt concrete pavements, and the methods used to power the system and provide data communication. In the final analysis, TxDOT and TTI were limited on which sensors could be selected for this task simply because no new and innovative sensor from the literature review appeared viable and ready to be used. However, Kistler quartz sensors were still relatively new to TxDOT and not widely installed at the time of the selection. This decision is actually fortuitous in that TPP was already considering the Kistler sensors as part of its ongoing implementation of TxDOT's strategic WIM plan. Therefore, the decision to monitor an installation of these sensors represented both a logical choice and probably the best use of research resources. TPP was well qualified to install the sensors, so monitoring the installation was not expected to be the same as a first-time installation of some unknown sensor as originally anticipated. Besides, TPP had a certified installer who supervised the installation at the Balmorhea site and is well qualified to oversee the installation to ensure success. Having a certified installer is a Kistler requirement.

There were delays in hiring a contractor to prepare the site for installing the WIM system in Balmorhea, so TPP's part of the installation had to wait until site preparation was completed. The Odessa District processed the hiring and monitoring of the contractor during this period. The responsibilities of the contractor at the WIM site included directional boring underneath the roadway, placing the concrete pad for the cabinet, placing ground boxes, and installing the timber pole for mounting the solar panel.

The original intent of this task was to provide guidelines that would apply to both installation, and maintenance and calibration of a newly installed WIM sensor. Given that the field installation did not occur until the month of August on the final year of the research project, it was not possible to evaluate maintenance and calibration of the WIM system installed at the Balmorhea site. However, some general guidelines are presented on maintenance and calibration of the Kistler sensors. Researchers recommend a follow-up project to monitor the Balmorhea WIM site as well as other sites on which TxDOT has installed or plans to install Kistler quartz sensors so that more data may be gathered for assessing maintenance and calibration needs based on observed WIM system performance and pavement performance.

SITE INSTALLATION IN BALMORHEA

TTI accompanied TPP staff to Balmorhea to observe and document the installation of the WIM system. Scheduling of the installation depended on the longer than anticipated process to set up the site preparation work for competitive bid, the selected contractor's ability to get the site preparation work done, and TPP's other work assignments and scheduled activities. TPP began the site installation within a few days of the contractor completing that portion of the work. Aspects of interest relating to the installation of the selected Kistler sensors included:

- time required to complete the installation,
- extent of compromise to pavement structural integrity,
- overall process,
- required crew size,
- required equipment support (compressors, pavement saws, water trailers, etc.), and
- traffic control to facilitate installation.

The first two items on the above list are pivotal in the selection of the WIM technology since installation techniques differ by WIM technology used. One of the key variables in installing any WIM system is the amount of time required on the roadway. This factor affects both traffic control costs and delays to motorists. At the Balmorhea site, traffic control was provided by the district and the delay to motorists was negligible due to the relatively low traffic volume (about 4000 total vehicles per day or 2000 vehicles per day per direction). During the installation, there was always at least one lane open. In comparison with time required to install a bending plate system, the time taken to install the Kistler sensors was about the same. Therefore, the staff cost and delay to motorists would be about the same for either system. Compromise to the pavement structure is less with the Kistler system compared to a bending plate. Also, installation of bending plate systems in HMAC has been discontinued. Kistlers, on the other hand, are considered more appropriate for HMAC because they can be ground to match surface undulations. Bending plates use a rigid frame that cannot be modified to match a pavement profile that might change.

INSTALLATION GUIDELINES

Kistler has published a thorough set of guidelines for installation of the sensors used in Balmorhea, Texas. For purposes of this document, TTI simply compared the installation by the TPP field crew to that described in the Kistler installation manual (Kistler, 2008). Comments that follow are organized according to the sections in the Kistler instructions. Researchers have provided comments on a few selected sections of the manual where minor discrepancies or differences were observed between the installation procedures followed in Balmorhea and the manufacturer’s instructions. These discrepancies could either be perceived or real but are deemed insignificant overall.

Section 3. General

- Sensor slots have to be 55 $\pm$ 3 mm deep and 72 $\pm$ 3 mm wide, perpendicular to the roadway, and just over 1.75 m in length (enough to allow grout to penetrate around the ends of the sensors).

TTI monitored the depths and widths of all sensor saw cuts and all met the prescribed dimensions.

- Once sensors are installed, grout should be allowed a cure time of 72 hours prior to calibration.

TTI and the TPP crew left the site after installation was complete. TPP did not do the final hook-up of the WIM electronics until several days later, so the 72 hour minimum cure time was met.

- Sensor layout: Kistler diagrams all show detection loops leading (quartz) sensors.

TPP used a different sequence in each lane compared to the manufacturer’s instructional drawings. TPP used the following sequence: load sensor→loop→load sensor→loop. However, for installations in asphalt, this sequence should not be critical as long as the PAT DAW-100 can accept that sequence. Even if it does not, TPP installed an AVC loop upstream of the WIM saw cuts, so it could be used if needed. TTI researchers do not consider this discrepancy to be a problem.

Section 4. Preassembling Sensors in Workshop

- Kistler recommends that the two sensor elements to be connected and placed in each wheel path be preassembled in a workshop or laboratory to ensure a clean and dry environment.

TPP connected the sensor elements in the field on the tailgate of a pickup. The method used by TPP worked well and allowed the individual sensors to be hauled to the site in original packing, making them less likely to be damaged en route. This discrepancy is not considered to be a problem.

Section 4.2 Insulation Check

- Kistler recommends measuring the insulation resistance of each sensor and provides the measurement range that is acceptable.

TTI researchers did not notice this test being conducted at the site, but it could have been carried out in the office or laboratory before the crew arrived on site.

Section 4.5 Assembling Row of Sensors

- Kistler recommends removing the black strip of foam from one of the sensor elements before connecting the two elements together.

TTI did not observe the foam being removed. Not removing one of the foam layers is not considered a serious problem.

Section 5. Installation

Section 5.3 Cutting Slot

- Kistler strongly recommends using a dry cutting pavement saw with a dust extraction system or preferably forming the slot with a pavement milling machine.

TPP cut the slot using a wet cut process since it does not use a dust extraction system or a milling machine for this task. TPP's saw can cut both the loops and the sensor slot by changing the saw blades. This method appears to be a more efficient. The wet cut causes some concern but only if the cuts and standing water are not removed properly. Some sources recommend air drying for a few hours following the pressurized removal of water in saw cuts. Such delays would have serious consequences in terms of crew productivity and potential delay to motorists. The west

Texas heat and low humidity were clear justification for not needing to air dry. Besides, the TPP crews have installed these sensors before using the same grout and if their procedures caused problems at these earlier sites, they would have developed a different and better technique. TTI checked all saw cuts prior to installation of loop wire or Kistler sensors and found no moisture whatsoever.

Section 5.6 Grounding Lineas Sensors

- Kistler recommends a separate ground for each row of load sensors, with connection to an earth ground in or at the equipment cabinet.

TPP did not run a separate ground cable from the sensor rows to the cabinet. However, there have been no known problems due to the grounding offered by the coaxial cable connecting sensors to the cabinet. The additional ground cables would require larger conduit than installed at the Balmorhea site, since the loop wire and Kistler cables alone resulted in very full conduits.

Section 5.11 Cleaning Sensors

- Kistler recommends cleaning the outside of each sensor with a dry cloth to ensure bonding by the grout.

TPP did not wipe the sensors, but they were all clean and did not appear to need cleaning. The only time the sensors could have gotten dirty is when they are placed on the pavement beside the saw cut just before the grout is poured. Even then, any dirt or debris that might have stuck to the bottom would be minimal.

CALIBRATION OF WIM SENSORS

Aspects of interest relating to calibration include procedures followed, vehicle types, axle loadings, suspension type, equipment and staff availability, time to complete, and physical requirements to facilitate the calibration process. The research team is aware of trends in some states to monitor sites for an indication of when calibration is required versus current TxDOT practice using scheduled quarterly calibration. State practice in the last several years has monitored combination truck steering axles for signal drift, as these axle loadings are typically within a narrow range and not as affected by cargo loadings as other axles. Quarterly calibration using known axle loadings (calibration truck) appears to work well and should be continued. Other innovative techniques to check the need for WIM system maintenance and calibration, such as the potential application of the FWD that researchers presented in Chapter III, should be further explored.

MAINTENANCE OF WIM SENSORS

Maintenance aspects of interest for a WIM system include type and frequency of sensor maintenance, frequency of sensor and pavement monitoring, frequency of preventive or reactive maintenance, and costs relating to maintenance actions. Sensor maintenance varies by WIM technology, as do maintenance strategies by pavement type.

One of the primary advantages anticipated with the Kistler Lineas WIM sensors is their low maintenance needs. Of course, one of the maintenance issues is calibration. If calibration frequency is similar or perhaps less than the frequency for bending plate systems, then the sensor's life cycle cost will be reduced accordingly. The other factor that must be considered with this sensor is its mean-time-between-failures. Information from other states has been mixed in terms of sensor life. Some states report failures averaging about every two years. When one sensor fails in the two-detector set, it will be difficult to extract one sensor without causing failure in the other one. To date, TPP has only installed a few lanes using these sensors so statistics to establish their life expectancy, and therefore their full maintenance requirements in Texas, are not available. A follow-up project is recommended to collect more data for assessing the expected service life of WIM systems that use Kistler quartz load sensors.

PAVEMENT MONITORING AT WIM SITES

Pavement monitoring should also be part of a maintenance program for WIM sites since the error in the load sensor measurement is influenced by vehicle-pavement interaction. Thus, researchers recommend periodic monitoring to collect pavement condition data on each 500-ft instrumented section consisting of the 350-ft approach to the load sensor and the 150-ft leave-out from the sensor on the given travel lane. Note that this information may not be provided in TxDOT's pavement management information system (PMIS) data base due to the difference in reporting interval. If data on each 500-ft WIM section cannot be extracted from the annual PMIS data, specific pavement condition surveys on the WIM sections would need to be conducted. These surveys could be conducted on one of the scheduled WIM calibration visits by TPP. In this way, TPP can assess the performance of the WIM sensors against the pavement condition measurements. For example, TPP can check whether the differences between the

WIM weight readings and the static loads of the calibration vehicle are increasing due to increased roughness in the WIM section.

Pavement monitoring should include profile measurements on the 500-ft WIM sections conducted at least annually, on or close to one of the scheduled calibration visits. On each 500-ft WIM section on a travel lane, researchers recommend checking the pavement smoothness as follows:

- Run the TxDOT ride quality bump template on each wheel path profile to check for the presence of bumps or dips and assess the need for and type of corrective work to smooth out localized roughness.
- Compute the WIM smoothness indices to check that all indices are below the corresponding upper thresholds prescribed in the provisional AASHTO MP-14 specification.
- Compute the wheel path IRIs over the 500-ft WIM section to compare against previous measurements and to check the variation of pavement smoothness over time.

In addition to collecting profile data on the WIM sections, researchers recommend monitoring the progression of rutting and cracking on each 500-ft section. Conduct surveys annually, on or reasonably close to one of the scheduled WIM calibration visits. The data collected can be used to assess the need for and type of maintenance work to retard the development of cracking and rutting that could lead to increased pavement roughness. In the final analysis, this effort to monitor the pavement condition at WIM sites will, over time, provide TxDOT with the performance data it needs to assess the cost-effectiveness of WIM installations on asphalt concrete pavements.

CHAPTER VI. DEVELOP RECOMMENDATIONS FOR TRANSFERRING AND PROCESSING WIM DATA

During this project, researchers also conducted two roundtable discussions with TxDOT personnel regarding truck weight data collection, processing, analysis, and reporting. These meetings took place in May 2008 at TxDOT's Camp Hubbard Facility (Building #1) and in June 2008 at the TxDOT Riverside Campus. Participants were from various sections of the department including TPP's Traffic Analysis Section, the Construction Division's Materials and Pavements Section, the Bridge Division, and the Motor Carrier Division (MCD).

Specific focus was given in the discussions to the requirements of other users of truck weight data within the department. Representatives from TxDOT divisions and sections were queried about their use of truck weight data and desired data over and above their current usage. The discussions also included incorporation of the AASHTO mechanistic-empirical pavement design guidelines to TxDOT traffic data collection and analysis, and current internal truck weight data processes. Part of this work included reviewing reports on the TrafLoad traffic data analysis software from NCHRP Project 1-39, previous work done on truck weight data by TxDOT, and the M-E PDG. This chapter presents the findings from the roundtable meetings and the review of related literature.

CURRENT WIM DATA COLLECTION

The roundtable discussions brought up the following points regarding the current TxDOT WIM program:

- The department currently has 22 WIM sites, an increase of six from 2007. Most of these sites use bending plates, but a few have Kistler quartz sensors.
- All WIM sites collect data 365 days per year, but TPP only uses one week of this WIM data every quarter to create weight tables.
- WIM data does not reflect seasonal variations after data are averaged.
- TPP polls the WIM stations daily and fixes equipment failures quickly. TPP has a goal of keeping 100 percent of systems operating 90 percent of the time.
- Having a limited number of data collection sites means that data from each site is assigned to many segments of roadway around the state. Every section of on-system

roadway in the state is assigned a vehicle classification and a weight station as representative in characterizing traffic on specific sections.

- For quality control, TPP calibrates each WIM site at least once a year, but some sites require more frequent calibration (e.g., piezoelectric sites).
- TPP uses the mainframe to process WIM data using its RDTEST68 program. Another program from FHWA that WIM data are used for is the Vehicle Travel Information System (VTRIS) program.
- The WIM data are stored on a local network drive within TPP. Historical data are available as far back as the 1970s, but data reliability was not as good in earlier years. Historical data can be used to analyze seasonal variations.
- The WIM strategic plan was developed considering regions of the state, but its implementation is advancing slowly. It may appear that TPP is not collecting data on a regional basis, but as the strategic plan is implemented, regional coverage will become clear.
- Meeting TMG requirements calls for a minimum of 130 WIM sites as proposed in the strategic plan. Accomplishing a higher degree of accuracy would be desirable, but it would require more sites.
- The number of other continuous data collection sites is as follows: volume – 109 (33 are LTPP – all lanes) and classification – 136. These numbers are subject to change since volume sites are being upgraded to classification sites, and other changes may occur as well.

WIM SITE INSTALLATION

TPP is continuing to implement the department's strategic WIM plan. However, this implementation is moving slower than desired. TPP recently installed a WIM site on I-45 south of Corsicana using Measurement Specialties, Inc. "BL" piezoelectric sensors in concrete pavement. The BL sensors yield reasonably good WIM data when installed in smooth concrete.

It is not TPP's intent to rely heavily on any piezoelectric sensors over the long term because they are temperature sensitive and require a constant stream of trucks for acceptable self-calibration. TPP used a PAT DAW 190 to check the data from the I-45 Corsicana site and found good results.

TPP is not encouraging widespread installation of piezoelectric sensors simply because they are cheaper. TPP staff emphasized that collecting some weight data was not the intent, and that no data is better than bad data, an opinion also shared by some other state DOTs. TPP has installed a few lanes of the relatively new Kistler quartz WIM sensors and conveyed that the results to date are good. TPP is relying on districts for installation (mostly as part of construction projects) or funding through research projects, such as the project (0-5551) that is documented in this report. In general, systems using bending plates and Kistler quartz sensors have similar costs. However, bending plates installed in HMAC sections require a 500-foot concrete pavement section to ensure long-term site stability, which is expensive. Examples of quartz installations are the I-35 site in Cotulla, the WIM installation on the southbound lanes of US77 in Robstown, and the new Balmorhea site on I-10 that was set up in this project. In addition, TPP plans to install Kistler quartz sensors on SH89 in Sinton and on SH6 north of Calvert.

Considering the high price of WIM equipment, TPP is careful to locate sites based on high volumes of trucks. TPP also considers the roadway type, whether construction is being planned, and specific commodity movements served. For example, the Rio Grande Valley needs more WIM sites for roads used for hauling produce, which would have significant seasonal variation. Other considerations for WIM sites include the minimum pavement thickness necessary to ensure stability of the WIM load sensors, proximity of turn-around space for a five-axle calibration truck, pavement smoothness over a 500-foot tangent section, and grades of less than two percent.

Providing the right data for meeting the intent of the M-E PDG is also critical. Participants of the roundtable discussions agreed that TPP is collecting the correct data but needs to increase the number of sites. TPP also realizes that more classification data is needed and has upgraded some of TxDOT's count sites to classification sites. TPP has also upgraded LTPP sites to include both directions of traffic flow. However, pavements at some candidate sites have deteriorated to the point that they do not adequately serve WIM needs. Thus, these sites have been downgraded to classification sites.

INTERNAL PROCESSING OF WIM DATA

The basic procedures used by TxDOT for processing WIM data were noted in previous TTI research work on data flow mapping (Project SPR-0420, *Assisting TxDOT in Transportation Monitoring Systems*). When field data come in, the data go to analysts for initial checks and smoothing, after which the data go to traffic analysts. Figure 6.1 graphically shows the overall process. Traffic Analysis Section staff believe that the data processing is effective and accurate. Appendix F shows the WIM data screening procedures in greater detail based on interviews with Traffic Analysis Section staff in the work noted above.

There was discussion among participants about distinguishing bad data from good data. TPP staff noted that the "Average 10 Heaviest Wheel Loads" have shown an upward trend and that a recent value is 19,700 lb. This increase may be a result of increasing numbers of spread tandems. TPP used to consider some of these loads as outliers, but now they believe the high values are accurate. However, in a subsequent communication, the project director commented that these high values appear unrealistic considering that wheel load magnitudes of 19,700 lb exceed the manufacturers' load ratings on highway truck tires.

In discussions about the accuracy of current weight data, TPP staff defended the data saying that TxDOT WIM meets the ASTM 1318 standard for accuracy, so TPP believes that the data being supplied are within the expected accuracy. It was also noted that IRD and other equipment vendors also provide similar outputs that could be used to verify TPP numbers if data users need further verification.

Regarding calibration of WIM for extra-legal loads, TPP staff does not attempt to calibrate the WIM equipment for heavier than legal loads since it would require overloading the calibration truck.

As the number of WIM sites increase, TxDOT personnel at Riverside have to make adjustments in the overall program to choose the most representative new site to represent other roadways around the state.

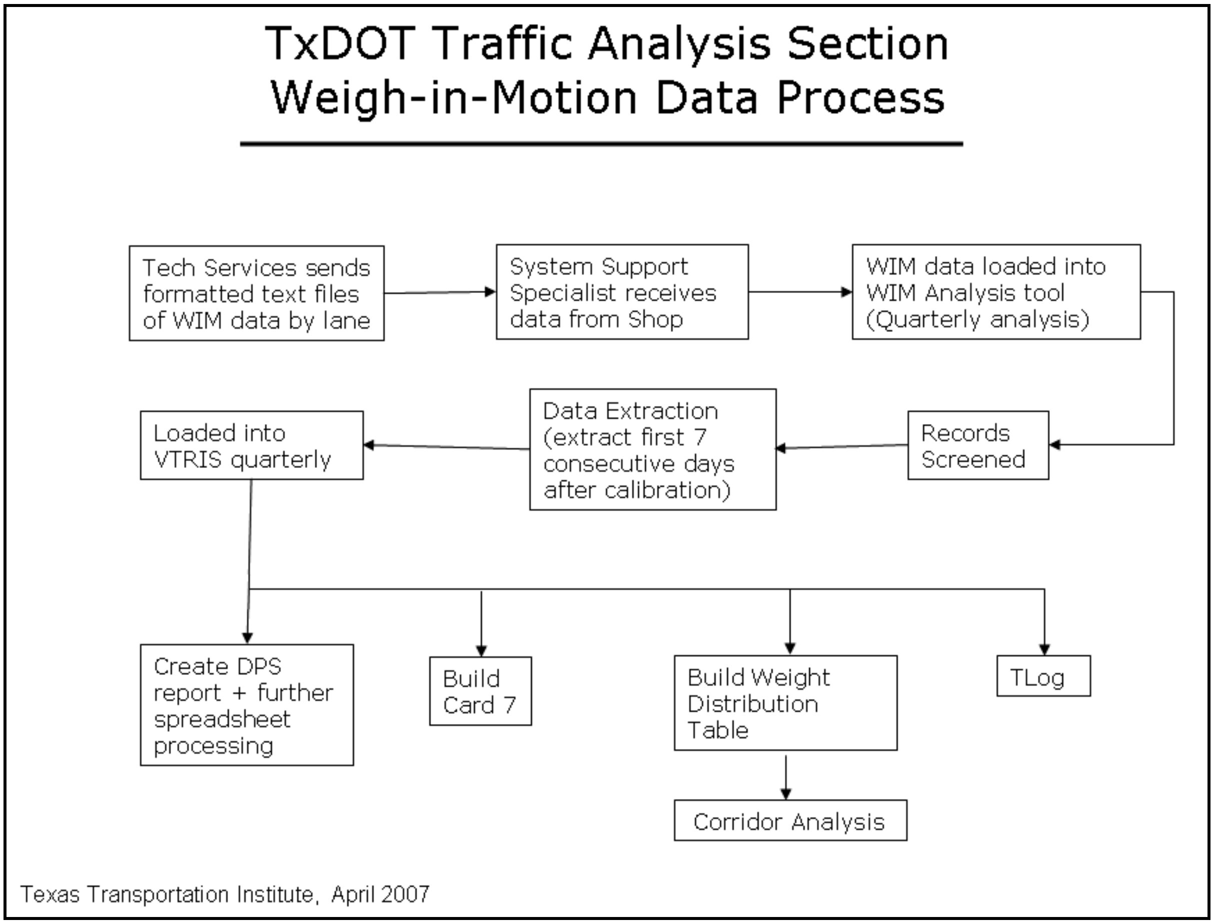


Figure 6.1. Flow Chart of Procedure for WIM Data Processing.

USER DATA REQUIREMENTS

TPP staff meets with the Department of Public Safety (DPS) four times per year to discuss vehicle weight data and any significant trends that need to be addressed by enforcement. Some districts have recognized the need to increase monitoring of truck traffic and are requesting additional WIM sites. For example, the Lubbock District is concerned about truck traffic around cattle feedlots.

Motor Carrier Division

The Motor Carrier Division needs to evaluate the availability and application of vehicle weight and classification data because data from WIM sites could possibly be beneficial for MCD’s operations. One use of the data might be determining compliance with the oversize/overweight (OS/OW) permit program through the use of transponders as a means of identifying specific trucks with a non-divisible load permit. Some trucks in Texas have transponders, but enforcement sites would have to be equipped with transponder readers to identify the truck as one having a permit (based on MCD records made available to DPS). No such readers currently exist for this purpose. TPP staff had provided the MCD with a map showing results of truck weight data collection, which might be useful in identifying possible areas of excessive weight violation. If enforcement sites had other technology that could measure dimensional elements of permit loads, that too could be helpful in automatically monitoring compliance. Again, this equipment is not available.

Other types of permits, such as the one that permits a 10 percent allowance over the legal limit, would also be difficult to detect with WIM equipment. Again, without an automated means of identifying trucks with permits (e.g., transponders with permit ID stored for communication with the roadside), DPS would not be very effective in tracking compliance of permits.

The MCD coordinates with DPS occasionally in the movement of OS/OW loads, but this coordination has been limited and mainly focused on super-heavy loads. This activity has not been very effective according to MCD staff. WIM systems would not be effective either for permit loads on specialized equipment that is wider than the lane width since it would not be accurately detected and would probably be misclassified. MCD might benefit from DPS feedback on number of citations issued and on "hot spots" where DPS might be targeting truck activities. MCD coordinates with various DPS captains in these efforts.

The MCD does not have a means of checking axle loads on the trucks they permit. The Division has to take the word of the truckers on their axle loads. TPP could not provide verification either even if the permitted load happened to cross a WIM system. Some permit loads are not assigned a route anyway (e.g., envelop vehicles). There is also no way to monitor over-dimensional loads at the present time at WIM sites. DPS can check the axle loads if they happen to stop a permitted truck.

Bridge Division

The Bridge Division has only used WIM data on a limited basis but has been able to request and obtain data from TPP anytime data are needed. Bridge engineers cannot easily get data for a specific bridge site, but when they need statewide data, they submit a request to TPP. The data from TPP could be useful for design instead of using AASHTO loads; for design verification; and for the future design of "truck corridors." The TPP data could be useful on a route-specific basis to determine the number of fatigue cycles for inspection purposes and could also help identify trouble spots.

The Bridge Division needs a means of getting WIM data without having a special program. Division staff mentioned the need for data sharing so that the users do not have to contact TPP all the time. In the meantime, TPP can provide some support, but it will probably be limited. It was noted that the WIM data are filtered since TPP only uses one week of data per month for reporting purposes.

Pavement Management

PMIS Section staff indicated that in August 2008 their office will update the PMIS database. The section does not currently use TPP data very much, but it does get data from TRM. PMIS Section staff occasionally get requests to provide data for legislative impact statements, so data on equivalent single axle loads (ESALs) would be useful for this purpose. One of the issues that recently came up was the NAFTA impact on selected roadways such as I-35 and US83. Findings indicate that these two roadways were not impacted as much by NAFTA as originally thought. Some of the less prominent roadways are apparently taking the brunt of NAFTA traffic. PMIS data is not sufficient to support network-level decision-making regarding truck load trends, but districts have immediate needs where TPP data might be useful.

TPP can install counters to get site-specific volume data as requested by districts. However, districts often need the data earlier than TPP can accommodate them, so districts must provide more lead time if they want TPP to provide such data. There is a need to educate users on how to request information from TPP.

M-E PDG and TrafLoad

Participants noted that there needs to be a way to automatically input data from TPP WIM sites for designing pavements using the M-E PDG. Pavement designers need to be able to access the axle loads and use the data in the design process. The TrafLoad program from NCHRP Project 1-39 was originally written to fill this need for processing the raw WIM data to generate axle load distribution tables in a format that can directly be used with the M-E PDG analysis program. However, example applications of TrafLoad reported in the literature and in practice indicate that the current program is not ready to serve this purpose due to problems in the software. At least two states have reported problems using the program. Researchers with the University of Arkansas led a project sponsored by the Arkansas State Highway and Transportation Department to establish axle load spectra for pavement design. They reported problems using TrafLoad with Card 7 data and had to write an Excel program to generate axle load spectra (Tran and Hall, 2007). Florida DOT staff also tried using TrafLoad in a project to implement the M-E PDG program. They were able to run the program with WIM weight data but had to edit the program output manually to convert the output into the M-E PDG format. They also commented that the program documentation was difficult to use.

TPP staff noted that TxDOT Project 0-4510 found that the WIM and AVC sites being used were sufficient to meet the requirements of the M-E PDG. They added that TPP is upgrading many sites to classification sites for two reasons. One reason is the need to do more classification (based on M-E PDG), and the other is that the federal government is providing 100 percent of the cost needed to upgrade to classification sites (Prozzi and Feng, 2006).

TPP staff suggested that if TxDOT would still need ESALs a few years from now, it would not be wise to abandon that process for the short term and possibly for the longer term. Pavements staff responded that TxDOT will not abandon ESALs in the short term. Participants noted that analysts and designers need to realize that the procedure for M-E PDG is more challenging and that we probably want to continue to use ESALs, perhaps indefinitely. ESALs still serve TxDOT's needs, and there is some apprehension regarding this new process. TPP intends to continue to provide the traditional data. Furthermore, all of the vendors proposing on the Statewide Traffic Analysis and Reporting System 2 (STARS 2) traffic data analysis software have said they intend to continue to provide ESAL data. TPP staff noted that STARS 2 will probably eliminate the need for TrafLoad anyway. In either case (using either TrafLoad or STARS 2), TPP plans on providing the necessary data to meet pavement design traffic data requirements, which will be ESALs in the short term, and perhaps eventually, axle load spectra.

IMPROVEMENTS TO CURRENT PROCEDURES

The STARS 2 program provides the main opportunity to improve current internal procedures for summarizing and reporting truck weight data. TPP is now working to obtain a vendor to purchase off-the-shelf software to complete this effort. The effort has been funded and will move TPP away from the current mainframe system, probably via an Oracle database.

There should be some web access available as long as security can be adequately addressed. User requirements have been established.

According to TPP staff, the STARS 2 process will cover all the data procedures and should be sufficient for what other groups within TxDOT and elsewhere will need. Bridge Division staff noted that STARS 2 may solve the data accessibility and timeliness concerns expressed during the roundtable discussions.

Approvals to move forward on STARS 2 have been granted, and a vendor should be selected by January 2009. This should position TPP to begin using the program within one year of signing a contract. Data input using the new process may happen as early as late FY2009 or early FY2010. TPP wants the program to be based on a geographic information system platform to more easily track each dataset. TPP wants to use a simple process in the new program to scan each site and then scan the data from the site. TPP collects 70,000 to 90,000 counts per year so the division needs a better way to track each site and the data collected to minimize confusion or loss of data.

RECOMMENDATIONS

Based on the information obtained from the roundtable discussions, researchers offer the following recommendations to address user requirements for WIM data within the department and the research community:

- TxDOT should continue to promote the installation of new WIM sites on roadways as opportunities are presented to the department. This objective may be accomplished by coordinating with the districts on new construction, reconstruction, or resurfacing projects, coordinating with the TxDOT research program where opportunities for collaborative efforts to meet the needs of research or implementation projects could result in new WIM sites or improvements to existing sites, and working with districts that have expressed an interest and need for WIM data.
- TPP should continue its efforts to install cost-effective WIM site technology that does not result in degradation of data quality. In particular, technology that performs to standard in asphalt pavements should be encouraged.
- TPP should make efforts to include divisions and sections of the department and other state agencies that use or desire to use WIM data in the development of STARS 2. Close

coordination with these entities during development will ensure that other user data requirements will be addressed.

- TxDOT should consider funding research that compares the values derived from using ESALs with load spectra using the M-E PDG. TPP staff and researchers are of the opinion that ESALs could provide as accurate data for pavement design as axle load spectra. The findings from the proposed research will factor into development of STARS 2. The concept of ESALs incorporates both the effects of load magnitude and number of load applications and has proven to be a useful parameter for quantifying truck traffic in pavement design.
- TPP staff should note the future findings from TxDOT Research Project 0-6095 in FY2009. The research will evaluate the concept of instrumentation using onboard sensors to monitor axle weights and trucks equipped with transponders. This research will address the concern voiced by MCD with its use of WIM data.

CHAPTER VII. SUMMARY OF FINDINGS AND RECOMMENDATIONS

Project 0-5551 aimed to support the implementation of TxDOT's strategic weigh-in-motion plan by identifying less costly but equally viable alternatives for deploying WIM installations to cover the state highway network. To accomplish this objective, researchers followed a three-pronged approach that comprised the following elements:

- Provide an alternative to WIM installations on CRC pavements by developing guidelines and procedures for finding sections within existing asphalt concrete pavements that provide the level of smoothness, pavement support, and projected service life deemed suitable for weigh-in-motion sites, particularly for installations that use piezoelectric technology.
- Reduce the cost of WIM installations in areas where land lines for electrical and telephone services are not available by evaluating the use of solar cells to power WIM systems and wireless alternatives for data transmission.
- Review recent developments in weigh-in-motion technology to identify alternative sensors for capturing WIM data that could be considered for testing in this project to evaluate their performance and cost-effectiveness.

In line with the above strategy, researchers carried out a comprehensive work plan that covered the following tasks:

- Review of current WIM practice that included an extensive literature search and communications with state DOTs and equipment manufacturers,
- Field and laboratory tests and data analyses to establish guidelines for evaluating flexible pavements to identify suitable WIM locations,
- Investigation of WIM applications of solar power and wireless communications, and design and installation of a wireless setup for an actual TxDOT WIM site,
- Monitoring installation of quartz WIM sensors to document the procedures for placing these load sensors on flexible pavements, and
- Roundtable discussions to identify user requirements for truck weight data within the department.

Based on the research conducted, the following findings are noted:

- The review of WIM practice identified that most DOTs use bending plates, load cells, piezoelectric sensors, and Kistler quartz sensors at WIM installations. Some states have started replacing the traditional piezoelectric sensors with the newer Kistler quartz sensors because of the reported greater accuracy achieved with the quartz sensors as well as their low maintenance needs. However, greater usage of the quartz sensors has probably been hampered by the mixed experience reported by states on their expected service life. Proper installation on pavements that provide adequate support appears to be a key factor in getting good sensor performance and service life.
- To date, the limited experience with Kistler quartz WIM sensors in Texas has generally been good. Current TxDOT WIM installations that use these load sensors are the I-35 site south of Cotulla, the WIM installation on the southbound lanes of US77 in Robstown, and the new Balmorhea site on I-10 that was set up in this project. Additional installations of quartz sensors on flexible pavements are planned.
- The review of WIM practice did not identify any new and innovative low-cost WIM sensors that could be recommended for practical use in a state WIM network at this time. The results reported to date on fiber optic and microwave sensors are at best inconclusive in the opinion of the researchers.
- Some state DOTs are already using solar power and/or wireless communications at their WIM installations. Tests conducted in this project showed that solar panels provide a viable alternative in the absence of land lines when properly matched with the expected power requirements at a WIM site. Based on past experience and equipment specifications, WIM systems tend to be low power consumption equipment and thus lend themselves to solar applications. With respect to data communications, researchers reviewed the available options as well as the practice in other states and concluded that current wireless cellular technology provides the most reasonable alternative to traditional wire line dialup. Researchers configured and tested a wireless communication setup for the Balmorhea WIM installation along I-10 and found the download speed and signal strength to be good. However, the long-term performance of this wireless communication system still needs to be determined.

- Researchers evaluated the WIM smoothness criteria prescribed in the provisional AASHTO MP-14 specification and found that the criteria generally tend to produce inconclusive determinations of whether a WIM installation classifies as Type I or not for the TxDOT WIM sites tested in this project. In most cases, the computed indices fell within the undetermined region of the proposed criteria. Moreover, it was difficult to find locations where all of the criteria were met on both wheel paths.
- During the course of testing WIM sites to establish criteria for identifying candidate flexible pavement WIM sections, researchers also initiated a small study to determine if the FWD can be used to check WIM sensors. This limited investigation identified two potential applications of the FWD:
  - first, as a tool to check the linearity of the WIM response to applied loads and
  - second, as a tool to check the consistency in the WIM readings from repeat measurements at a given drop height.

  However, more work is needed, in the researchers' opinion, before any definitive conclusions can be made for using the FWD to perform checks on existing weigh-in-motion systems.
- FWD deflections taken on the 500-ft perpetual pavement WIM sections south of Cotulla show the uniformity and degree of pavement support to be comparable to that provided by CRC pavements. On the outside lane, the FWD maximum deflections under drop height 2 ranged from 4 to 5 mils over a range of pavement temperatures from 86 to 113 °F during testing. On the inside lane, the maximum deflections varied from 3.8 to 6.6 mils over a range of pavement temperatures from 114 to 130 °F. Given these measurements, the concern over the consistency of WIM readings due to the variation of asphalt concrete stiffness with pavement temperature does not appear to be an issue, at least for the perpetual pavements tested at the Cotulla WIM site.
- During this project, TPP also conducted tests with its calibration truck to check the consistency of the readings from the quartz WIM sensors placed in Cotulla. TPP compared the gross vehicle weight measurements from multiple runs made on the WIM sensors with the reference GVW of the calibration truck. These comparisons showed that the errors (as percentages of the reference GVW) are all within the 10 percent tolerance on

gross vehicle weight specified in ASTM E1318 for Type I WIM systems. Further, the errors showed no significant correlation with pavement temperature.

- The engineering evaluation of the proposed WIM site along US77 in Robstown showed evidence of potential pavement performance problems as reflected in the results of laboratory tests done on cores and the observed premature cracking at the site. All asphalt mixture specimens cut from the cores took just one load cycle to fracture under the overlay test, which is far below the recommended minimum number of 300 load cycles. The laboratory tests also found the asphalt content of the mix to be low, which verified the observation made by the Corpus Christi District Pavement Engineer who commented that the asphalt content appears to be on the dry side after examining the cores taken from the proposed WIM location. This finding, and the fact that the mix passed the Hamburg test, indicates the need for a procedure to achieve a balanced mix design based on rutting and cracking criteria. In this regard, TxDOT Project 0-5123 is developing a balanced mix design method based on the Hamburg and overlay tester. The criteria from this project should prove useful in identifying candidate flexible pavement WIM sites that can reasonably be expected to provide adequate service life.
- Using the pavement evaluation criteria proposed in Chapter III, researchers found an acceptable flexible pavement WIM site located within the limits of a recently completed project along I-10 west of Balmorhea in the Odessa District. However, performance monitoring of the newly installed WIM site is needed to collect data with which to assess the effectiveness of the proposed criteria based on long-term observations of WIM system performance and pavement performance.
- Based on the roundtable discussions conducted with TxDOT, researchers are of the opinion that the STARS 2 program provides the main opportunity to improve current internal procedures for summarizing and reporting truck weight data. This new program will cover all data procedures and is expected to provide for the traffic-related data needs of various users within TxDOT. Chapter VI identified several of the needs expressed by the TxDOT divisions, such as axle load distribution data for pavement design and data accessibility and timeliness. The latter concern is expected to be addressed via the Oracle database platform of STARS 2. Approvals to move forward on STARS 2 have been

granted, and TPP expects to select a vendor by January 2009. This should position TPP to begin using the program within one year of signing a contract.

Considering the findings from the investigations presented in this report, researchers offer the following recommendations on TxDOT's continuing implementation of its strategic WIM plan:

- TxDOT should implement the guidelines presented in Chapter III to locate suitable asphalt concrete pavement sections for WIM installations. Chapter III included detailed project case studies that illustrated the applications of these guidelines, which are based on existing pavement evaluation tools and computer programs within TxDOT. Thus, researchers are of the opinion that the guidelines can readily be put into practice within the department. Considering that TxDOT incurs a cost of about $500,000 to build 500-ft CRCP slabs for a 4-lane WIM installation, the department stands to realize significant cost savings by implementing the flexible pavement WIM evaluation guidelines from this project. Based on the cost for the 4-lane Kistler quartz WIM system installed in Balmorhea, four 500-ft CRCP slabs are equivalent to about eight 4-lane Kistler quartz WIM systems. Given this perspective, TxDOT has good reason to find sections within existing asphalt concrete pavements that provide the level of smoothness, pavement support, and projected service life deemed suitable for weigh-in-motion sites, particularly for installations that use piezoelectric technology.
- TxDOT should consider funding a contract to support implementation of the flexible pavement WIM evaluation guidelines as part of its ongoing effort to establish a state highway WIM network. This contract can be executed as an implementation project or an inter-agency agreement. Its objective is to provide implementation support in the following areas:
  - pavement testing services such as inertial profiler and GPR testing on flexible pavement projects to locate candidate WIM sites;
  - laboratory testing of asphalt cores taken from candidate WIM sites to assess material durability under the Hamburg and overlay tester;
  - analyses of field and laboratory test data to identify candidate WIM sections on flexible pavement projects;
  - testing of installed WIM systems to verify WIM site classification based on ASTM E1318 criteria;

    - monitoring of flexible pavement WIM systems to provide TxDOT with performance data for assessing maintenance and calibration needs of WIM systems that use Kistler quartz load sensors, verify quartz sensor service life, determine the reliability of solar power and wireless communications at WIM installations, and assess the cost-effectiveness of flexible pavement WIM installations; and
    - testing of new WIM sensors or system components on the TTI SH6 WIM test bed to help TxDOT make informed decisions on use of new WIM technology.
- TxDOT should consider funding a follow-up research project to further investigate the potential applications of the FWD to serve WIM system maintenance and calibration needs. The limited investigation conducted in this project identified a couple of applications by which the FWD might be put to use. However, more work is needed before any definitive conclusions can be made for using the FWD to perform checks on existing weigh-in-motion systems. In particular, more FWD tests are needed to cover the types of WIM systems deployed by TxDOT on both concrete and flexible pavements. Additionally, communications with WIM manufacturers and vendors is needed to provide hardware and software modifications that permit the FWD to be more readily used for checking existing WIM systems.
- TPP should continue its efforts to install cost-effective WIM site technology that does not result in degradation of data quality. In particular, technology that performs to standard in asphalt pavements should be encouraged.
- TPP should make efforts to include divisions and sections of the department and other state agencies that use or desire to use WIM data in the development of STARS 2. Close coordination with these entities during development will ensure that user data requirements are addressed.

REFERENCES

American Association of State Highway and Transportation Officials. *Standard Specification for Smoothness of Pavement at the Approaches to Weigh-in-Motion (WIM) Scales – AASHTO Designation: MP14-05*. AASHTO Provisional Standards, Washington, D.C., 2007, pp. MP 14-1 to MP 14-9.

American Society for Testing and Materials. *Standard Specification for Highway Weigh-in-Motion (WIM) Systems with User Requirements and Test Methods – ASTM Designation: E1318-02*. Annual Book of ASTM Standards, Section 4, Construction, Vol. 04.03, ASTM International, West Conshohocken, PA, 2006, pp. 1083-1099.

Applied Research Associates. *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*. Final Report, National Cooperative Highway Research Program Project 1-37A, Transportation Research Board, Washington, D.C., 2004.

Asphalt Institute. *Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition*. Research Report No. 82-2, Asphalt Institute, Lexington, KY, 1982.

CDW website. *http://www.cdw.com/shop/products/default.aspx?EDC=1185578*, accessed August 29, 2008.

Chen, D. and F. Hong. *Field Verification of Smoothness Requirements for Weigh-in-Motion Approaches*. Report DHT 48, Construction Division, Texas Department of Transportation, Austin, TX, 2008.

Cosentino, P. and B. Grossman. *Optimization and Implementation of Fiber Optic Sensors for Traffic Classification and Weigh-in-Motion Systems (Phase 3).*
Report No. FL/DOT/RMC/06650-7754, Florida Institute of Technology, Melbourne, FL, 2000.

Cosentino, P., B. Grossman, C. Taylor, W. Eckroth, R. Tongta, and T. Zhao. *Fiber Optic Traffic Sensor*. Presented at NATDAC96 hosted by Alliance for Transportation Research, Federal Highway Administration, and the New Mexico State Highway and Transportation Department, Albuquerque, NM, 1996.

de Vries, M. J., V. Arya, and R. O. Claus. *Fiber Sensors Guard Highways of the Future*. Vol. 32, Laser Focus World, Pennwell Publishing, Tulsa, OK, 1996.

Fernando, E. G., and W. Liu. *User's Guide to the Overweight Truck Route Analysis (OTRA) Program*. Product 0-4184-P3, Texas Transportation Institute, The Texas A&M University System, College Station, TX, 2004.

Fernando, E. G., G. Harrison, and S. Hilbrich. *Evaluation of Ride Specification Based on Dynamic Load Measurements from Instrumented Truck*. Research Report 0-4863-2, Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2007.

FHWA-LTPP Technical Support Services Contactor. *LTPP WIM Index Software Manual, Version 1.1*., MACTEC Engineering and Consulting, Inc., Beltsville, MD, 2005.

Gulyas, A. and P. Hernadi. *Effects of Road Surface Characteristics on Dynamic Axle Weight Measurements*. Proceedings of Quatrie'me Symposium International sur Caracte'ristiques de Surface des Chausse'es Routie'res et Ae'ronautiques, May 2000, pp. 413-422.

Jacob, B. A. *Weigh-in-Motion of Road Vehicles*. Proceedings of the Final Symposium of the Project WAVE (1996-1999), European Commission DG VII-Transport, Paris, France, 1999.

Jacob, B. A. and E. J. O'Brien. *WAVE - A European Research Project on Weigh-in-Motion*. Presented at NATDAC96 hosted by Alliance for Transportation Research, Federal Highway Administration, and the New Mexico State Highway and Transportation Department, Albuquerque, NM, 1996.

Karamihas, S. M. and T. D. Gillespie. *Smoothness Criteria for WIM Scale Approaches*. Final Report 2002-37, The University of Michigan Transportation Research Institute, Ann Arbor, MI, 2002.

Karamihas, S. M. and T. D. Gillespie. *Advancement of Smoothness Criteria for WIM Scale Approaches*. Final Report 2004-12, The University of Michigan Transportation Research Institute, Ann Arbor, MI, 2004.

Karamihas, S., G. Rada, B. Ostrom, A. Simpson, and L. Wiser. *Pavement Smoothness at Weigh-in-Motion Sites – Developing Specifications for the Long-Term Pavement Performance Program*. Transportation Research Record: Journal of the Transportation Research Board, No. 1870, National Research Council, Washington, D.C., 2004, pp. 116-123.

Kistler. *Installation Instructions – Lineas Sensors for Weigh-in-Motion Type 9195E*. Kistler Instrumente AG. Winterthur, Switzerland, 2008.

Larsen, D. and A. H. McDonnell. *Second Interim Report on the Installation and Evaluation of Weigh-in-Motion Utilizing Quartz-Piezo Sensor Technology*. Connecticut Department of Transportation, Hartford, CT, 1999.

Liu, W. and T. Scullion. *MODULUS 6.0 for Windows: User's Manual*. Research Report 1869-2, Texas Transportation Institute, The Texas A&M University System, College Station, TX, 2001.

Liu, R., X. Chen, J. Li, L. Guo, and J. Yu. *Evaluate Innovative Sensors and Techniques for Measuring Traffic Loads*. Report No. TxDOT 0-4509, University of Houston, 4800 Calhoun Road, Houston, TX, 2005.

Livingston, R. A. *Gauging Strain on New Design Swiss Box-Girder Bridge*. Web pages for the Turner-Fairbank Highway Research Center, http://www.tfhrc.gov/trnsptr/apr98.htm, 1998.

MACTEC, Inc. *Field Evaluation of the UMTRI Smoothness Criteria for WIM Scale Approaches*. FHWA-LTPP Technical Support Services Contractor, Beltsville, MD, 2004.

MACTEC, Inc. *Validation Report: Texas, SPS 480100, Task Order 13, CLIN 2*. FHWA-LTPP Technical Support Services Contractor, Beltsville, MD, 2006.

McDonnell, A. H. *Evaluation of Quartz-Piezoelectric WIM Sensors: Second Year Study*. Presented at the North American Travel Monitoring and Exhibition Conference, Madison, WI, 2000.

Middleton, D., R. White, J. Crawford, R. Parker, J. Song, and C. Haas. *Investigation for Traffic Monitoring Equipment Evaluation Facility*. Report No. FHWA/TX-06/0-4664-3, Texas Transportation Institute, The Texas A&M University System, College Station, TX, 2005.

Prozzi, J. A. and F. Hong. *TxDOT Project 0-4510: Evaluate Equipment, Methods, and Pavement Design Implications for Texas Conditions of the AASHTO 2002 Axle-Load Spectra Technology*. Texas Department of Transportation, Austin, Texas, 2006.

Rand, D. *TxDOT's Use of the Overlay Tester and Hamburg Wheel Test*. Presentation made at the Louisiana Transportation Engineering Conference, Baton Rouge, LA, 2007.

Scullion, T. *Perpetual Pavements in Texas: State of the Practice.*Research Report 0-4822-1, Texas Transportation Institute, The Texas A&M University System, College Station, TX, 2006.

Szary, P., A. Maher, R. Panda, and A. Safari. *Implementation of Advanced Fiber Optic and Piezoelectric Sensors: Fabrication and Laboratory Testing of Piezoelectric Ceramic-Polymer Composite Sensors for Weigh-in-Motion Systems*. Report No. FHWA 1999-029. Rutgers University, Trenton, NJ, 1999.

Texas Department of Transportation. *A Strategic Plan for Weigh-in-Motion Compliance*. Texas Department of Transportation, Austin, TX, 2003.

Texas Department of Transportation. *Condition of Texas Pavements: Pavement Management Information System (PMIS) Annual Report FY 2003 – 2006*. Texas Department of Transportation, Construction Division, Materials and Pavements Section, Austin, TX, 2006.

Tran, N. H. and K. D. Hall. *Development and Influence of Statewide Axle Load Spectra on Flexible Pavement Performance*. Transportation Research Record No. 2037, Transportation Research Board, Washington, D.C., 2007, pp. 106-114.

Udd, E. and M. Kunzler. *Development and Evaluation of Fiber Optic Sensors*. FHWA-OR-RD-30-14, Blue Road Research, Gresham, OR, 2003.

APPENDIX A

CALTRANS AND FLORIDA DOT WIM PRACTICES

CALTRANS WIM INSTALLATION PROCEDURES

CALTRANS uses the following guidelines to locate the controller cabinet:

- not subject to being hit by errant vehicles,
- easily and safely accessible, with adjacent vehicular parking,
- provides full view of the roadway in which the WIM sensors are installed,
- not subject to flooding during heavy rains or from near irrigation systems, and
- does not require long conduit runs for the required sensors.

Bending plate systems must have adequate drainage of water from under the plates. Ideally, the lanes to be instrumented should all slope to the outside in a roadbed on an embankment to easily remove outflow. Crown section roadbeds need drains on both sides of the roadway. Roadways in flat or cut sections should not be considered for bending plates unless the WIM drain pipes can be tied into existing drainage facilities.

Traffic conditions are another critical consideration. The best WIM performance occurs with all traffic traveling at a constant speed with vehicles staying near the middle of each lane. Tangent sections of roadway with little or no grade in rural areas normally best meet this condition unless there are only two lanes, and passing is significant. Conditions to avoid include stop-and-go traffic, slow moving traffic, lane changing, and passing. A condition where vehicles stop over the sensors results in useless data. The problem with slow-moving traffic is that WIM systems cannot compensate for accelerations or decelerations, compromising accuracy. Lane changing can result in partially or totally missing one or more sensors. Passing on two-lane roadways can result in crossing the loops in reverse order. Neither PAT nor the IRD WIM systems correctly classify these vehicles. For roadways with two or more lanes in each direction, passing is only a problem if passing vehicles are changing lanes over the WIM system.

Roadway geometrics are also critical for optimized WIM performance. Installers should only consider tangent (straight) sections of roadway. Lane width is a consideration in that weigh pads in a side-by-side configuration must be able to fit. Being too close to interchanges and intersections may increase lane changing and speed change and may be a factor in controlling traffic during setup or maintenance operations.

Grade is an important determinant in the accuracy of WIM. Anything in excess of 1 percent grade results in weight transfer from the steer axle to the drive axle of loaded trucks. Weight transfer can easily exceed 1500 lb, with resultant errors in the WIM's reporting of gross vehicle weight. The higher recording of weight for the drive axle will often result in a weight violation for the drive axle group. Other problems that may occur as a result of grades involve initial calibration and calibration monitoring. The grade may decrease the number of faster moving trucks, and adequate calibration requires the entire range of speeds. For CALTRANS, which uses a software program to track truck weights by speed distribution, a larger speed range makes the weight/speed analysis much more difficult. Finally, grades that result in slow-moving trucks will result in increased passing within the WIM system by faster vehicles.

The pavement profile and condition are critically important for WIM accuracy. The goal of the installation process is to minimize the dynamic effects induced by pavement roughness and profile, and to approach what would be measured statically. CALTRANS avoids areas where major roadway reconstruction would be required to achieve the desired WIM performance. However, pavement resurfacing and/or grinding are considered appropriate items of a WIM installation contract. CALTRANS recommends that a potential WIM site have a minimum 1000 ft of approach roadway with even profile. Pavement should be stable, with the thought that roadways settle around bridge and drainage structures.

If the roadway profile and overall pavement condition are acceptable, engineers should evaluate the pavement in the immediate vicinity of the WIM system. CALTRANS criteria require that the pavement be absolutely smooth for 150 ft in advance of and 75 ft beyond the bending plates. The pavement type is important to CALTRANS as well; the department only uses concrete pavements. CALTRANS considers roadway improvements in terms of a “strategic importance” scale. For sites with high truck volumes on the upper end of this scale, the pavement should be improved to the highest quality that is affordable in terms of cost. Lower volume sites would justify less pavement improvement. Pavement preparation criteria used by CALTRANS are as follows:

- For existing PCC pavement:
    - If in excellent condition (stable and smooth), grind 150 ft in advance of and 75 ft beyond the bending plates.
    - If in less than excellent condition, replace existing pavement with seven sack concrete as follows:
        - Remove existing PC pavement and first level base but no less than 12 inches in depth. Replace a minimum 50 ft preceding and 25 ft beyond the

bending plates; consider a longer replacement based upon the condition of the existing pavement and importance of the truck weight data. CALTRANS' longest replacement to date is 200 ft.

    - Grind existing and new PCC pavement, starting 100 ft upstream of the new pavement and end 50 ft beyond the new pavement.

- For existing AC pavement, replace existing pavement with seven sack concrete as described above for PCC pavement replacement. Grind existing AC pavement and new PCC pavement beginning 25 ft upstream of new pavement and end 25 ft downstream of new pavement.

The CALTRANS document recommends that the reviewer observe the traffic flow at various times of the day, watching for undesirable traffic conditions when reviewing a potential WIM site. The observer should carefully watch trucks passing through the site to determine if they are traveling through the site at a fairly constant speed and that they are not bouncing due to pavement roughness or profile. The document also recommends contacting traffic engineers and maintenance personnel who are familiar with the traffic characteristics at the site for their knowledge and observations. It is also very important to confirm that there are no plans to widen or reconstruct the roadway soon after the WIM system is installed.

FLORIDA DOT TECHNICAL SPECIAL PROVISIONS FOR A WEIGH-IN-MOTION SITE

The following information is reprinted directly from the specifications set forth by the Florida Department of Transportation for the installation of a weigh-in-motion system along State Road 55 in Levy County. The financial project identification number for this WIM installation is 210431-1-52-01.

WEIGH-IN-MOTION SYSTEM INSTALLATION

I. Scope

This project includes the installation of a telemetered weigh-in-motion (WIM) system at the location shown in the Plans. This contract includes furnishing and installing a WIM electronics unit, Type V cabinet, solar power unit with new pole, Class I piezoelectric weight sensors, inductive loops, conduit, pull boxes, related hardware, and removal of some existing traffic monitoring site equipment. It also includes testing and calibrating the WIM equipment following installation. The completed WIM system shall be fully functional and responsive to the Transportation Statistics Office's polling computer.

II. General Requirements

The accuracy for this system shall comply with the requirements of ASTM E1318 for a Type II WIM system. The Weigh-In-Motion System Electronics unit shall be installed in a California Department of Transportation (CALTRANS) Type 334 cabinet with a solar power supply and a data communications modem and receive input from roadway embedded loop detectors and Class I piezoelectric weight sensors.

Work to be accomplished under this Contract is highly technical in nature. The Contractor shall adhere to the installation instructions provided by the WIM electronics manufacturer and shall engage the services of a field representative of that manufacturer for the purpose of obtaining technical supervision of all work contained herein. The field representative shall be on site during the installation of the Class I piezoelectric weight sensors, temperature sensors and inductive loops in the roadway, the connection of the WIM electronics unit, and calibration and testing of the completed system.

The Contractor shall provide a fully functional weigh-in-motion installation that is compatible with the Department's existing polling computer and software. There is currently only one approved manufacturer that provides WIM equipment compatible with the Department's polling software. The following individual may be contacted for any information concerning approval of equipment and materials:

Attention: Mr. Mulder Brown
Florida Department of Transportation
Transportation Statistics Office
605 Suwannee Street, MS 27
Tallahassee, Florida, 32399-0450, Ph. (850) 414-4716

The Contractor shall supply three (3) copies of all manuals that document installation, operation and maintenance of all equipment supplied.

III. Referenced Specifications

The Florida Department of Transportation (FDOT) Standard Specification for Road and Bridge Construction, 2000 Edition, and Evaluation Criteria for Traffic Control Devices are incorporated by reference and amended by these Technical Special Provisions for this project.

Additional Applicable Specifications:

- FHWA 1P-78-16, 170 Traffic Control System Hardware Specification for Type 334 cabinets;
- American Society of Testing Materials (ASTM), E1318 Type II Weigh-In-Motion Standards; and
- Federal Highway Administration (FHWA) - Traffic Monitoring Guide.

IV. Material and Installation Requirements

Directional Bore

Conduits shall be placed 1.0 meters beneath existing roadway by the directional bore method in accordance with the Utility Accommodation Manual. All conduits shall be intermediate metal conduit (IMC). Associated fittings shall be the same as specified for conduit underground.

Grounding Electrode (F&I)

The grounding system shall consist of the following:

1. For the cabinet, a minimum of 15 meters of 16mm diameter ground rod **and** a maximum measurement of 25 ohms to ground, and
2. For the solar power pole, 6 meters of 16mm diameter ground rod.

Included with the grounding is all associated hardware and wire in accordance with Section 620, as amended and supplemented. All connections to ground rods shall be by exothermic weld.

Conduits (F&I) (Underground)

The conduit shall be furnished and installed in accordance with the requirements of Section 630, as amended and supplemented, except as modified in the Plans. All conduits shall be intermediate metal conduit (IMC, 50mm dia. unless otherwise specified) and terminated with grounding bushings. Ground bushings at conduit terminations shall be interconnected with #8 AWG solid bare copper wire. All metallic conduit connections, including elbows, couplings, ground bushings, etc., shall be of threaded design and coated with a conductive lubricant in the threaded areas during installation to ensure the integrity of the connection with respect to grounding.

Pull and Junction Boxes (F&I) (Pull Box)
Pull boxes shall be furnished and installed in accordance with Section 635. Only concrete pull boxes with cast iron covers are allowed. Metallic pull box covers will not require grounding.

Remove Miscellaneous Signal Equipment
The Contractor shall remove the items referenced in the plan note on the Weigh-In-Motion Site Plan View sheet of the plans. These items, Type IV base mounted cabinet, concrete base, weighplates, weighplate frames, lighting mast arm, pole and base are located at station 669+40 and 669+52 approximately. The Transportation Statistics Office will have a technician present while equipment is being removed.

The Contractor will remove and dispose of the lighting mast arm, pole and base.

Note: There are four weighplates and weighplate frames at the existing weigh-in-motion site (two in each of the northbound lanes). Weighplates shall be salvaged by the Contractor and retained until Transportation Office technicians can transport them to the office in Tallahassee. Weighplate frames will be removed and disposed of by the Contractor. The volume of excavation for each frame is estimated to be 0.09 cubic meters. Total fill requirements should be 0.4 to 0.5 cubic meters.

Removal of weighplates and frames from the roadway should be conducted as follows:

1. Disconnect or cut all weighplate leads from the pull box adjacent to the roadway;
2. Remove nuts and lock washers from the weighplate frame studs and remove the weighplate retaining bars and shims;
3. Pry up the end of the weighplate that does not have a lead extending from it. As the weighplate is lifted feed lead-in cable from the pull box to avoid stressing the cable connection to the weighplate. Set the weighplate down on blocks (or short pieces of 2x4 wood) and pull the remainder of the lead through the frame's lead-in tube. Coil and tie the lead-in cable and place it on top of the weighplate;
4. Steel anchors are embedded in the roadway to hold down the weighplate frames. The top of these anchors must be cut with an acetylene torch or straightened to allow removal of the weighplate frame. The embedded end of the anchors do not need to be removed;
5. Because weighplate frames are coated with a bituminous material, heating may be required in addition to prying in order to remove them. Care must be taken to minimize damage to the existing pavement while prying. Weighplate frames will be disposed of by the Contractor; and
6. After removal of the weighplate frames, the excavated area shall be backfilled with an approved patching material. The Contractor will follow the manufacturer's recommended procedure for site preparation and filling as approved by the Engineer.

Weigh-In-Motion Electronics Assembly (F&I)

The weigh-in-motion electronics assembly consists of the following major components:

1. Electronics unit(s) with housing, loop and weight sensor detector boards, and data storage;
2. Class I weight sensor and epoxy grout;
3. Inductive loops and loop sealant;
4. Lightning suppression for all weight sensor inputs; and
5. All associated installation hardware.

The Contractor shall furnish and install the following approved WIM electronics units:

Model DAW190 manufactured by PAT America, Inc. at:
1665 Orchard Drive
Chambersburg, Pa. 17201

or equivalent with prior approval as stipulated in Section II.

The electronics unit shall include the appropriate number of loop detector boards. The unit(s) shall be capable of weighing all vehicles moving over four lanes of a highway at speeds of from 10 to 70 mph. They shall be able to measure the axle weights, axle spacing(s), and speed of all vehicles having up to and including nine axles, with axle weights up to 35,000 pounds and gross weights up to 200,000 pounds, and through use of this data determine the vehicle's Type Code in accordance with Scheme F as described in the Federal Highway Administration's *Traffic Monitoring Guide.* The unit shall be able to store this data, along with the site number, lane number, time and date, and be able to display and/or print out this data as requested.

The system will normally operate in an unattached mode, accumulating data for later retrieval by either a direct connection of a PC to the electronics unit, or by downloading via polling by the Transportation Statistics Office's polling computer.

Note: Payment for technical assistance for the installation of this complete WIM system by the electronics equipment manufacturer, shall be included in this pay item number.

Installation requirements:

The WIM electronics unit shall be rack mounted in the cabinet. All cable terminations and connections to this unit will be as directed by the manufacturer's field representative.

System installation and calibration shall be in accordance with the manufacturer's standard, the manufacturer's representative and the specifications of ASTM E1318.

Class I Piezoelectric Weight Sensors:

The Piezoelectric weight sensors shall be supplied by the weigh-in-motion electronics unit manufacturer, and shall be of unencapsulated design.

Installation Requirements:

The Contractor shall install the piezoelectric weight sensors in accordance with the manufacturer's recommended instructions supplied by the field representative. The plans detail typical placement of all major components.

In general, piezoelectric sensors shall be installed in a channel cut into the asphalt pavement. The piezoelectric sensors shall be installed in the roadway seven millimeters below the pavement surface. The channel shall be perpendicular to the flow of traffic. Seven millimeter (7mm) holes shall be drilled at a 45 degree angle downward to the side if the channel to a depth of twenty-five millimeters. The holes shall be spaced every 0.3 meters on both sides of the channel. These holes will be filled with the epoxy grout to secure the piezoelectric sensor mechanically to the pavement.

A slot shall be cut into the pavement from the end of the sensor to the edge of the road nearest the splice pull box as shown in the plans. Axle sensor cables shall be of a sufficient continuous length to reach the cabinet without splicing. Field splicing of sensor cables is prohibited except upon specific approval by the Engineer. The sensor cable shall be run in a conduit as shown in the Plans from the edge of the shoulder to the pull box. All sawcuts, channels, drilled holes and cable lead-in slots shall be cleaned thoroughly of all dirt, water and dust prior to applying the epoxy.

Plastic standoffs or similar devices shall be attached at intervals along the length of the sensor elements to assure placement at uniform depth. Epoxy shall be poured into the channel to cement the sensor into the roadway. The epoxy shall be flush with the surface of the roadway at completion of the installation.

Each lead-in cable shall be labeled in each pull box and in the cabinet with a permanent identification tag denoting lane number and sensor number.

Temperature Sensors

The Contractor shall furnish and install the type and number of temperature sensors required by the manufacturer of the WIM equipment used for this project. Location of these sensors shall be as specified by the manufacturer's field technician. Sensor lead-in cables shall be of a sufficient continuous length to reach the cabinet without splicing. Field splicing of sensor lead-in cables is prohibited except upon specific approval by the Engineer.

Installation requirements:

The Contractor shall follow the manufacturer's recommended procedure for installing the temperature sensors, with the assistance of the manufacturer's field representative.

Grout

The epoxy grout shall be as specified by the piezoelectric weight sensor manufacturer.

Loop Assembly

The inductive loop assemblies, loop wire, shielded lead-in cable, loop sealant and associated materials shall be furnished and installed in accordance with Section 745, as amended and supplemented or as modified in the plans.

Installation requirements:

Inductive loops will be installed by the Contractor in each lane in accordance with the manufacturer's instructions, or as modified by the weigh-in-motion electronics manufacturer's field representative and approved by the Engineer. The Contractor shall furnish all equipment and materials, and all necessary labor to perform this work. Each loop shall be centered in its respective lane and shall have three turns of wire.

Lightning Protection System (F&I)

The Contractor shall furnish and install in the cabinet lightning suppression consisting of the following:

1. 8 - EDCO Model SRA-6LCA-716 suppressors (one for each loop lead-in);
2. 1 - EDCO Model FAS-TEL suppressor for the phone line; and
3. 1 - EDCO Model MOVP-047 suppressor for the solar power leads.

V. Testing and Inspection

General

It shall be the Contractor's responsibility to ensure that upon completion of the installation, the equipment is properly calibrated, fully functional, and responsive to the Department's Transportation Statistics Office's polling computer.

Piezoelectric Sensor Threshold Test

The Class I sensor shall have the flexibility necessary to conform to the roadway profile while maintaining a uniform distance from the roadway surface. The voltage generation of the sensor shall be measured. The signal produced when the sensor is contacted by a wheel impact shall be a positive output and shall have an inherent 10:1 rejection of road noise due to bending, adjacent lanes and bow waves of approaching and receding vehicles. Signal output may vary no more than 7% across the length of the sensor. Operating temperature range of the sensor shall be from -40 to +160 degrees Fahrenheit. Capacitance of the weight sensor and cable lead-in shall be measured before installation to assure they are within the range allowed by the WIM equipment manufacturer. Capacitance shall be measured after the sensor is grouted in the roadway and leads pulled to the cabinet, to verify that the sensor and lead-in were not damaged during installation. Sensors shall be provided with lead-in cables of sufficient length to reach the cabinet without splicing. The lead-in cable shall be made of high-density polyethylene. The insulation resistance of the extension cable shall exceed 500 megohms.

Inductive Loop Tests
The inductive loop assemblies and their respective lead-in cables shall be tested in accordance with the requirements of Section 660-5 as amended and supplemented. These tests shall include insulation resistance and series resistance.

System Tests
Prior to acceptance, the Contractor shall demonstrate in the presence of the Engineer that the equipment supplied and installed for the system is in full compliance with the plans and specifications herein. Any equipment or labor, which is found to be defective prior to Final Acceptance, shall be replaced or corrected to suitable specifications at the Contractor's expense.

The Contractor shall ensure, and so demonstrate, that the systems are fully compatible with the Department's polling system, and is capable of transmitting and receiving data to and from that system.

The Department will operate the complete system for 30 consecutive days without failures prior to final acceptance. The Department will poll the site and statistically check data from historical data, field collected data and field observations. In the event of failures, the Contractor shall correct the problem(s) and restart the 30-day test. Final acceptance will be made upon the successful completion of the 30-day test.

All equipment and material furnished and all work performed shall be subject to inspection by the Engineer. The Engineer shall have free access during normal working hours to any local facility or area in which the associated work is occurring. The Contractor shall ensure all information is made available to the Engineer upon request.

Warranty
The Contractor shall obtain from the manufacturer and assign to the Department a manufacturer's warranty on all system components, which is equal to those provided as customary trade practice in the manufacturer's industry.

FINANCIAL PROJ. I.D.	SHEET NO.
210431-1-52-01	190

GENERAL NOTES

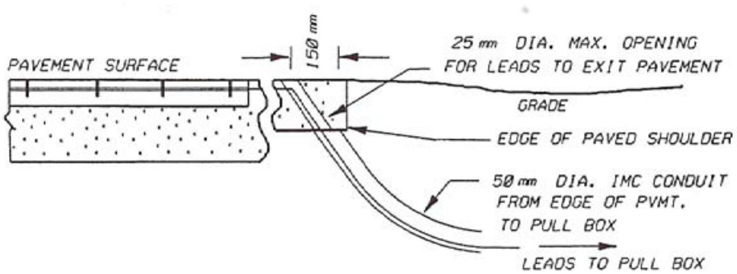
1.) THE CONTRACTOR WILL NOTIFY CHARLES GOFF OF THE FDOT, TRANSPORTATION STATISTICS OFFICE, (850)488-2855, TEN WORKING DAYS PRIOR TO REMOVAL OF EXISTING TRAFFIC MONITORING EQUIPMENT AND TEN DAYS PRIOR TO INSTALLATION OF NEW EQUIPMENT.

2.) THE FDOT TRANSPORTATION STATISTICS OFFICE SHALL BE RESPONSIBLE FOR CONTACTING THE TELEPHONE COMPANY AND OBTAINING SERVICE TO THE SOLAR POWER POLE. THE CONTRACTOR SHALL PROVIDE CONNECTION TO THE J-BOX MOUNTED BY THE TELEPHONE COMPANY AND EXTEND THIS LINE INTO THE CABINET, PERFORMING ALL CONNECTIONS TO THE MODEM AND ELECTRONICS UNIT.

3.) REFERENCE THE TECHNICAL SPECIAL PROVISIONS FOR THE DISPOSITION OF ITEMS TO BE REMOVED.

4.) A TYPE II POST MOUNTED OBJECT MARKER SHALL BEINSTALLED AT EACH PULL BOX.

5.) SOLAR POWER UNIT WILL REQUIRE 2 - 50-WATT SOLAR PANELS



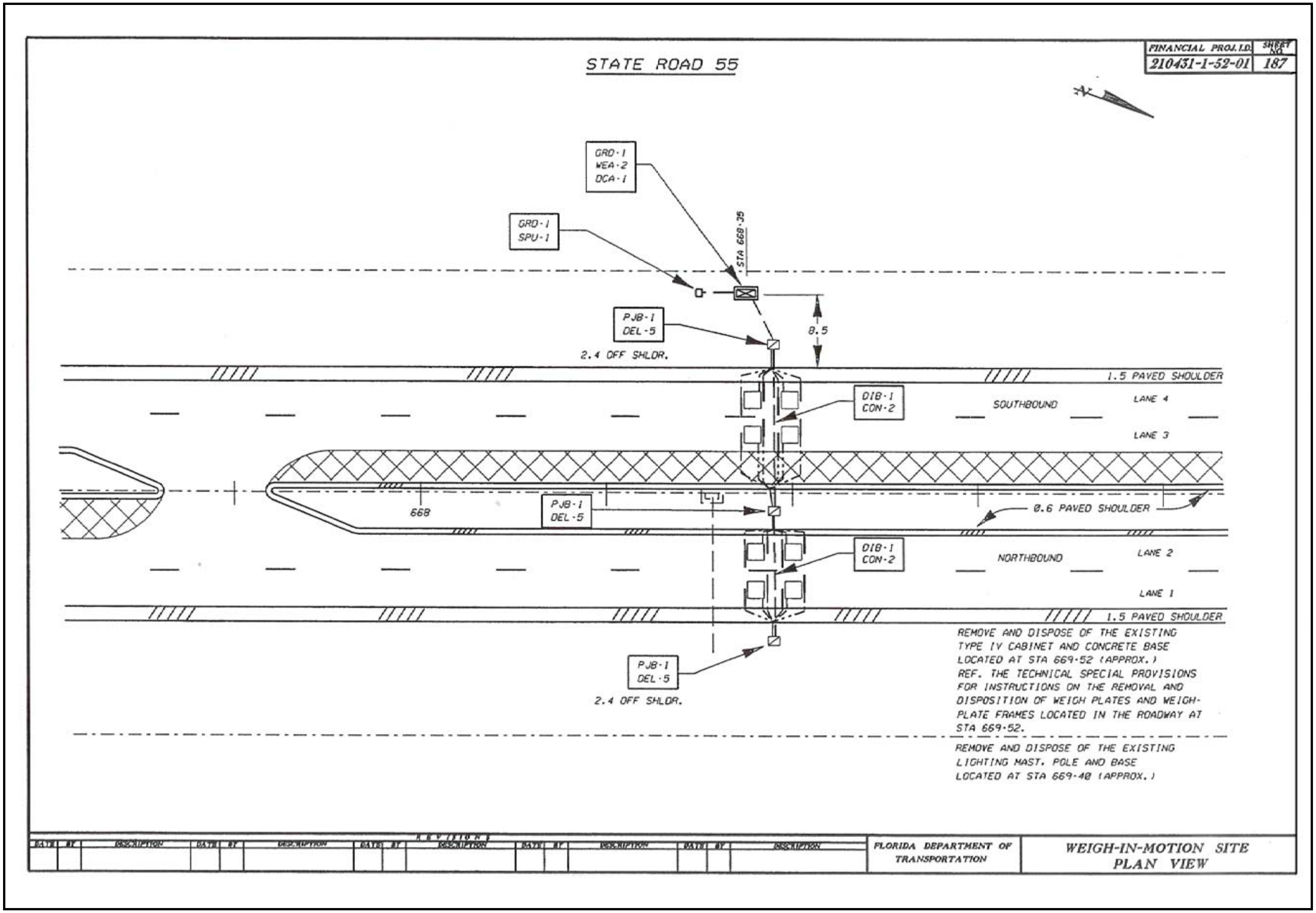
LEAD ROUTING TO PULL BOX
(NTS)

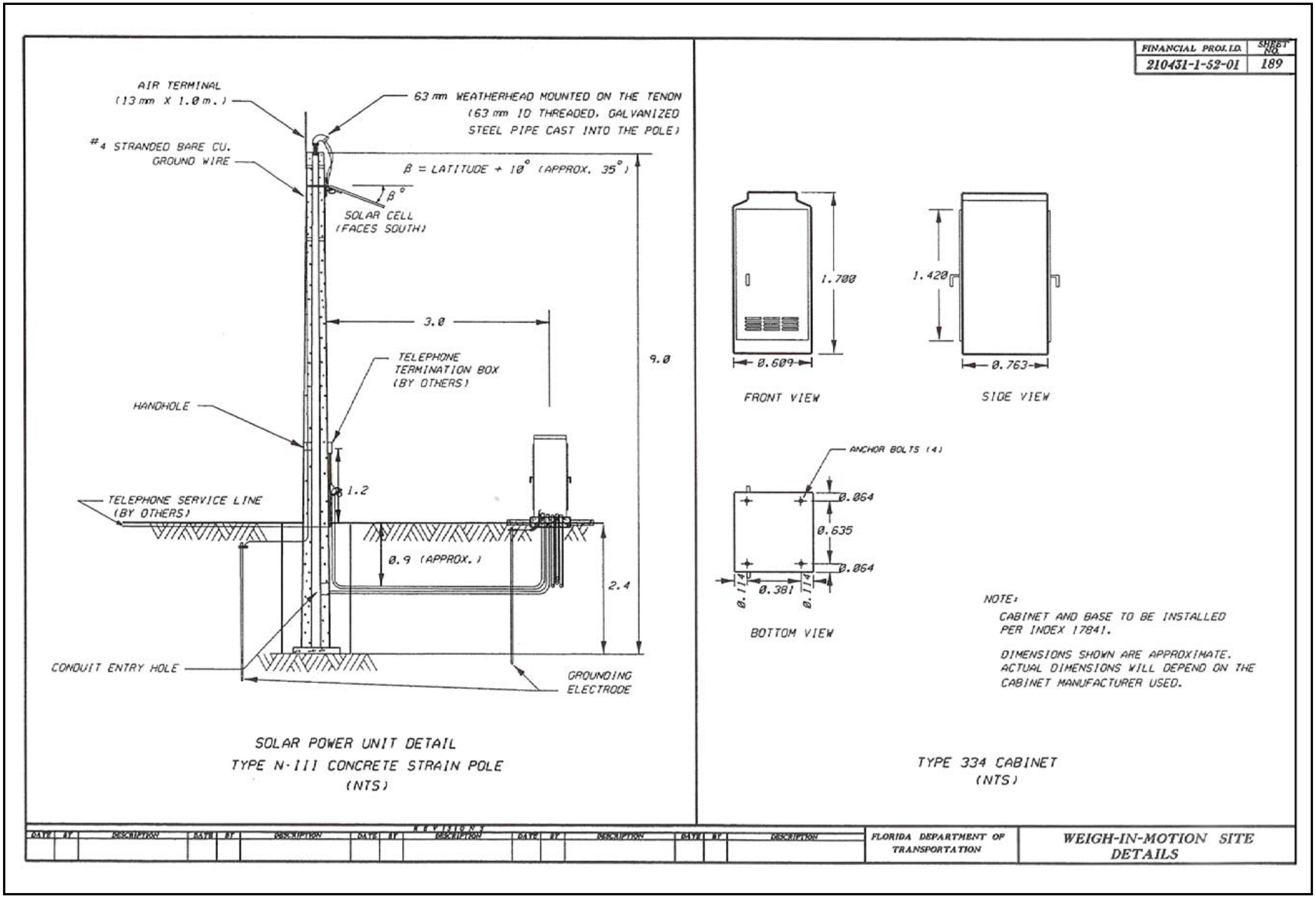
ITEM IDENTIFICATION NUMBERS
THROUGHOUT THIS PLAN SET THE FOLLOWING APPLIES:
F & I - FURNISH AND INSTALL

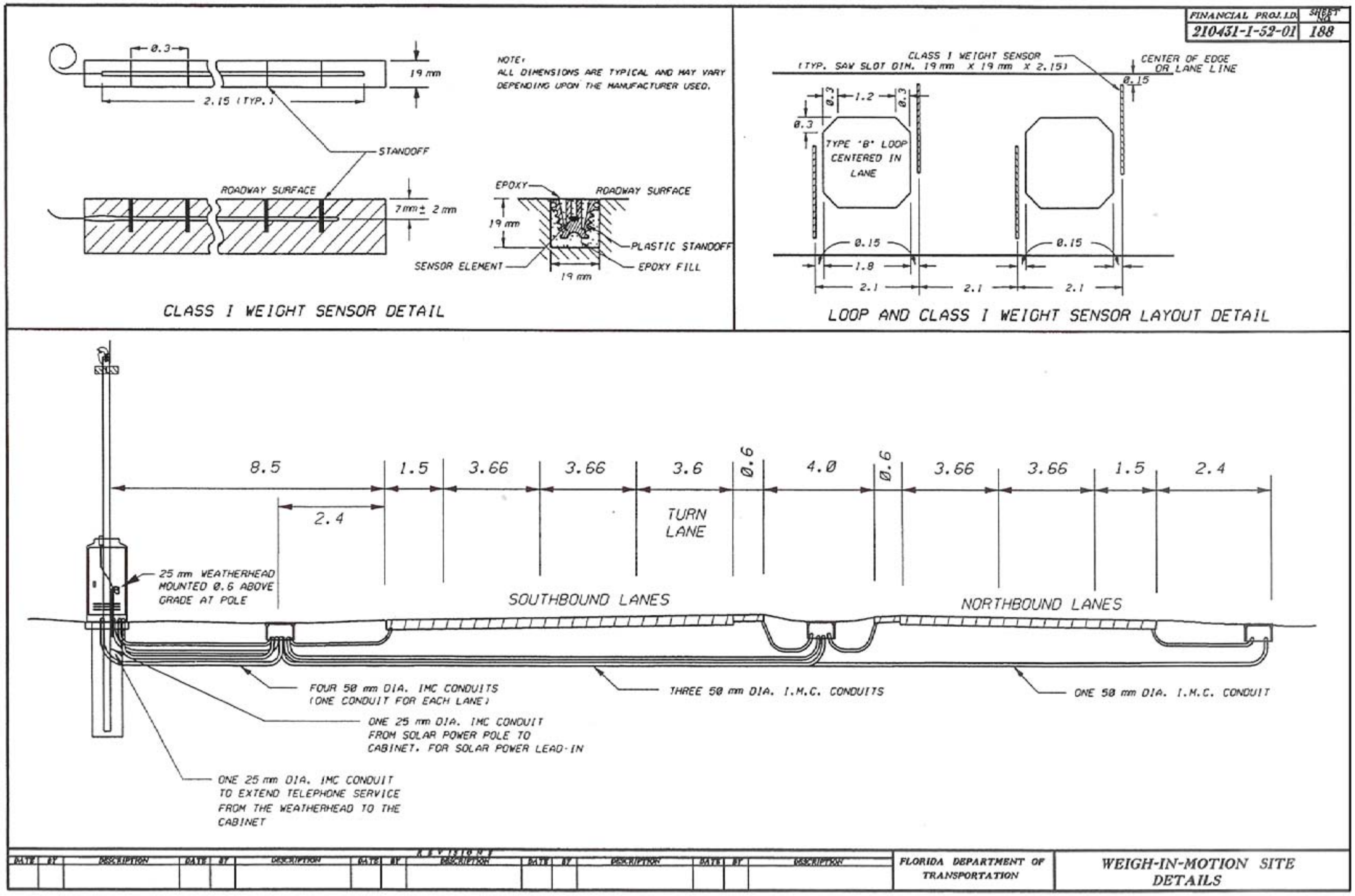
DIB-1	F & I - DIRECTIONAL BORE
GRD-1	F & I - GROUNDING
CON-2	F & I - CONDUIT UNDERGROUND
PJB-1	F & I - PULL & JUNCTION BOXES
DEL-5	F & I - OBJECT MARKER
WEA-2	F & I - WIM ELECTRONICS ASSEMBLY
SPU-1	F & I - SOLAR POWER UNIT
DCA-1	F & I - DETECTOR CABINET

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	FLORIDA DEPARTMENT OF TRANSPORTATION	WEIGH-IN-MOTION SITE DETAILS


STATE ROAD 55
FINANCIAL PROJ.I.D. 210431-1-52-01
SHEET NO. 187
GRD-1
WEA-2
DCA-1
GRD-1
SPU-1
STA 668+35
PJB-1
DEL-5
2.4 OFF SHLDR.
8.5
1.5 PAVED SHOULDER
DIB-1
CON-2
SOUTHBOUND
LANE 4
LANE 3
668
PJB-1
DEL-5
0.6 PAVED SHOULDER
DIB-1
CON-2
NORTHBOUND
LANE 2
LANE 1
1.5 PAVED SHOULDER
REMOVE AND DISPOSE OF THE EXISTING TYPE IV CABINET AND CONCRETE BASE LOCATED AT STA 669+52 (APPROX.)
REF. THE TECHNICAL SPECIAL PROVISIONS FOR INSTRUCTIONS ON THE REMOVAL AND DISPOSITION OF WEIGH PLATES AND WEIGH-PLATE FRAMES LOCATED IN THE ROADWAY AT STA 669+52.
PJB-1
DEL-5
2.4 OFF SHLDR.
REMOVE AND DISPOSE OF THE EXISTING LIGHTING MAST, POLE AND BASE LOCATED AT STA 669+40 (APPROX.)
REVISIONS
DATE BY DESCRIPTION
FLORIDA DEPARTMENT OF TRANSPORTATION
WEIGH-IN-MOTION SITE PLAN VIEW


FINANCIAL PROJ. I.D.
SHEET NO.
210431-1-52-01
189
AIR TERMINAL (13 mm X 1.0 m.)
63 mm WEATHERHEAD MOUNTED ON THE TENON (63 mm ID THREADED, GALVANIZED STEEL PIPE CAST INTO THE POLE)
#4 STRANDED BARE CU. GROUND WIRE
β = LATITUDE + 10° (APPROX. 35°)
β°
SOLAR CELL (FACES SOUTH)
3.0
9.0
TELEPHONE TERMINATION BOX (BY OTHERS)
HANDHOLE
1.2
TELEPHONE SERVICE LINE (BY OTHERS)
0.9 (APPROX.)
2.4
CONDUIT ENTRY HOLE
GROUNDING ELECTRODE
SOLAR POWER UNIT DETAIL
TYPE N-III CONCRETE STRAIN POLE
(NTS)
1.700
0.609
FRONT VIEW
1.420
0.763
SIDE VIEW
ANCHOR BOLTS (4)
0.064
0.635
0.064
0.114
0.381
0.114
BOTTOM VIEW
NOTE:
CABINET AND BASE TO BE INSTALLED PER INDEX 17841.
DIMENSIONS SHOWN ARE APPROXIMATE. ACTUAL DIMENSIONS WILL DEPEND ON THE CABINET MANUFACTURER USED.
TYPE 334 CABINET
(NTS)
REVISIONS
DATE BY DESCRIPTION
FLORIDA DEPARTMENT OF TRANSPORTATION
WEIGH-IN-MOTION SITE DETAILS


FINANCIAL PROJ. I.D.
210431-1-52-01
SHEET NO.
188
NOTE:
ALL DIMENSIONS ARE TYPICAL AND MAY VARY DEPENDING UPON THE MANUFACTURER USED.
0.3
19 mm
2.15 (TYP.)
STANDOFF
ROADWAY SURFACE
7 mm ± 2 mm
EPOXY
ROADWAY SURFACE
19 mm
PLASTIC STANDOFF
SENSOR ELEMENT
EPOXY FILL
19 mm
CLASS I WEIGHT SENSOR DETAIL
CLASS I WEIGHT SENSOR
(TYP. SAW SLOT DIM. 19 mm X 19 mm X 2.15)
CENTER OF EDGE OR LANE LINE
0.15
0.3
1.2
0.3
0.3
TYPE "B" LOOP CENTERED IN LANE
0.15
0.15
1.8
2.1
2.1
2.1
LOOP AND CLASS I WEIGHT SENSOR LAYOUT DETAIL
8.5
1.5
3.66
3.66
3.6
0.6
4.0
0.6
3.66
3.66
1.5
2.4
2.4
TURN LANE
25 mm WEATHERHEAD MOUNTED 0.6 ABOVE GRADE AT POLE
SOUTHBOUND LANES
NORTHBOUND LANES
FOUR 50 mm DIA. IMC CONDUITS (ONE CONDUIT FOR EACH LANE)
THREE 50 mm DIA. I.M.C. CONDUITS
ONE 50 mm DIA. I.M.C. CONDUIT
ONE 25 mm DIA. IMC CONDUIT FROM SOLAR POWER POLE TO CABINET. FOR SOLAR POWER LEAD-IN
ONE 25 mm DIA. IMC CONDUIT TO EXTEND TELEPHONE SERVICE FROM THE WEATHERHEAD TO THE CABINET
REVISIONS
DATE BY DESCRIPTION
FLORIDA DEPARTMENT OF TRANSPORTATION
WEIGH-IN-MOTION SITE DETAILS

APPENDIX B

DATA FROM EVALUATION OF WIM SMOOTHNESS INDICES

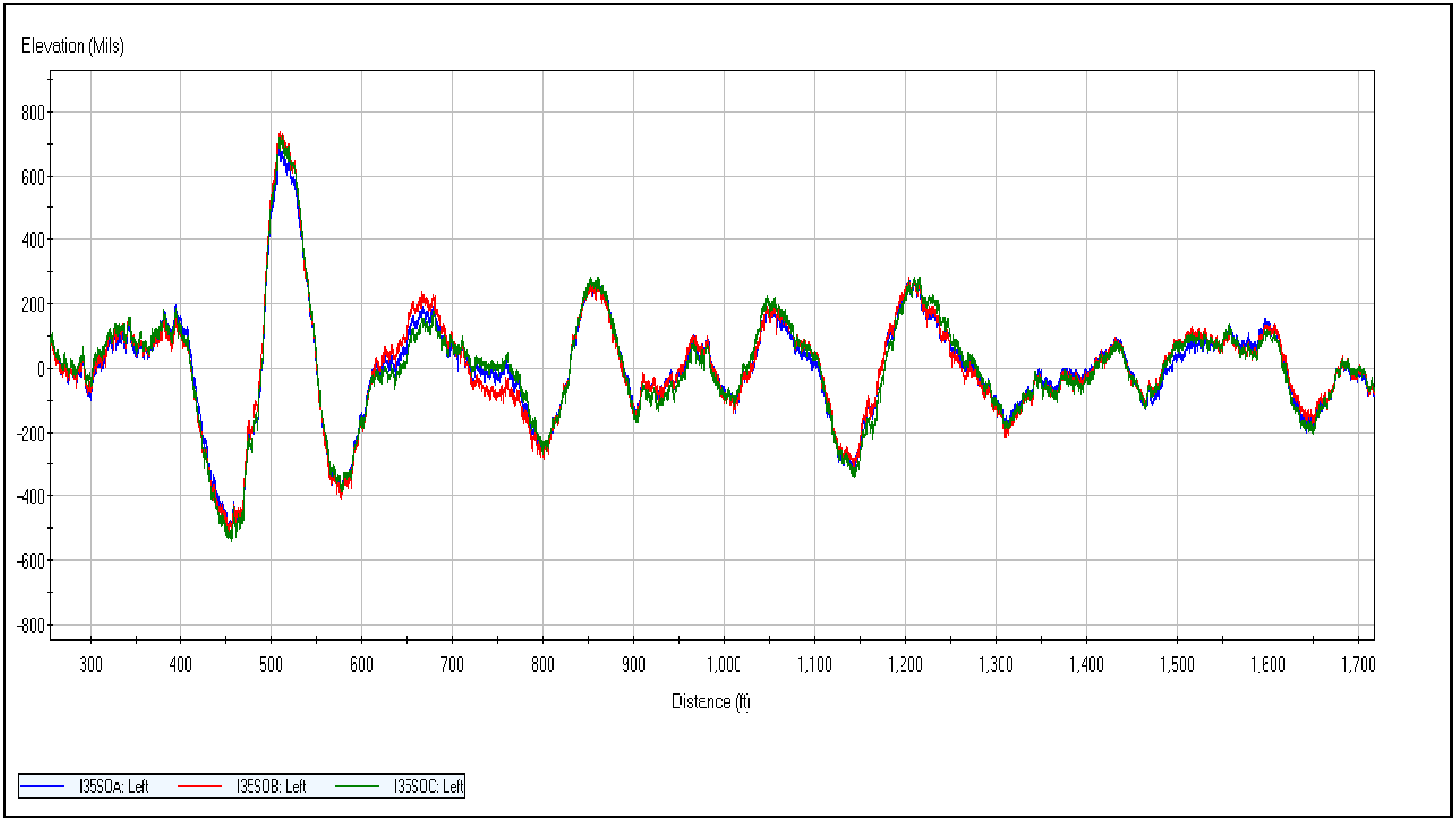


Figure B1. LWP Profiles on I-35 Southbound Outside Lane of TxDOT WIM Site 531.

Note: WIM location is at 956 ft on all charts in this appendix.

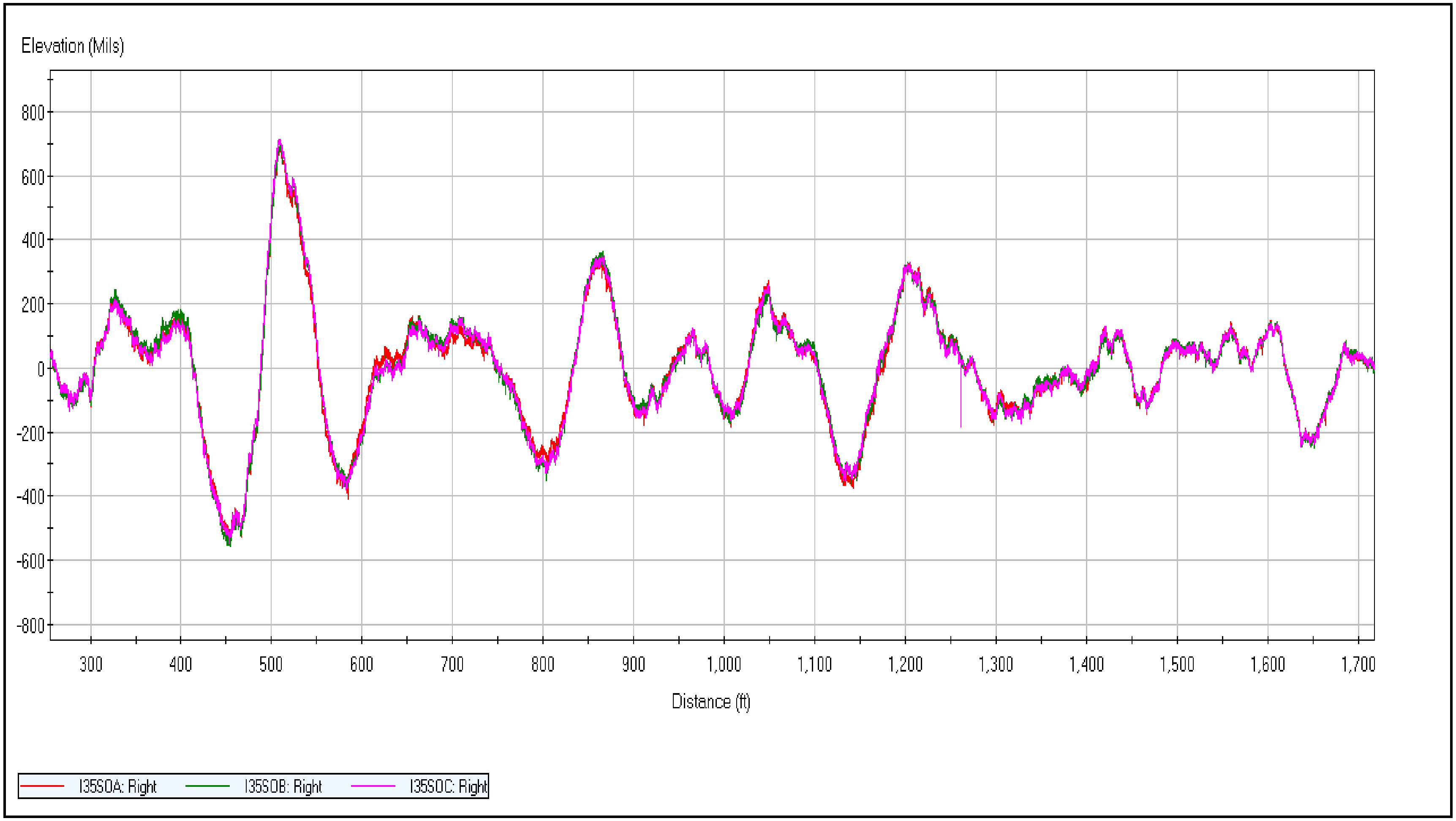


Figure B2. RWP Profiles on I-35 Southbound Outside Lane of TxDOT WIM Site 531.

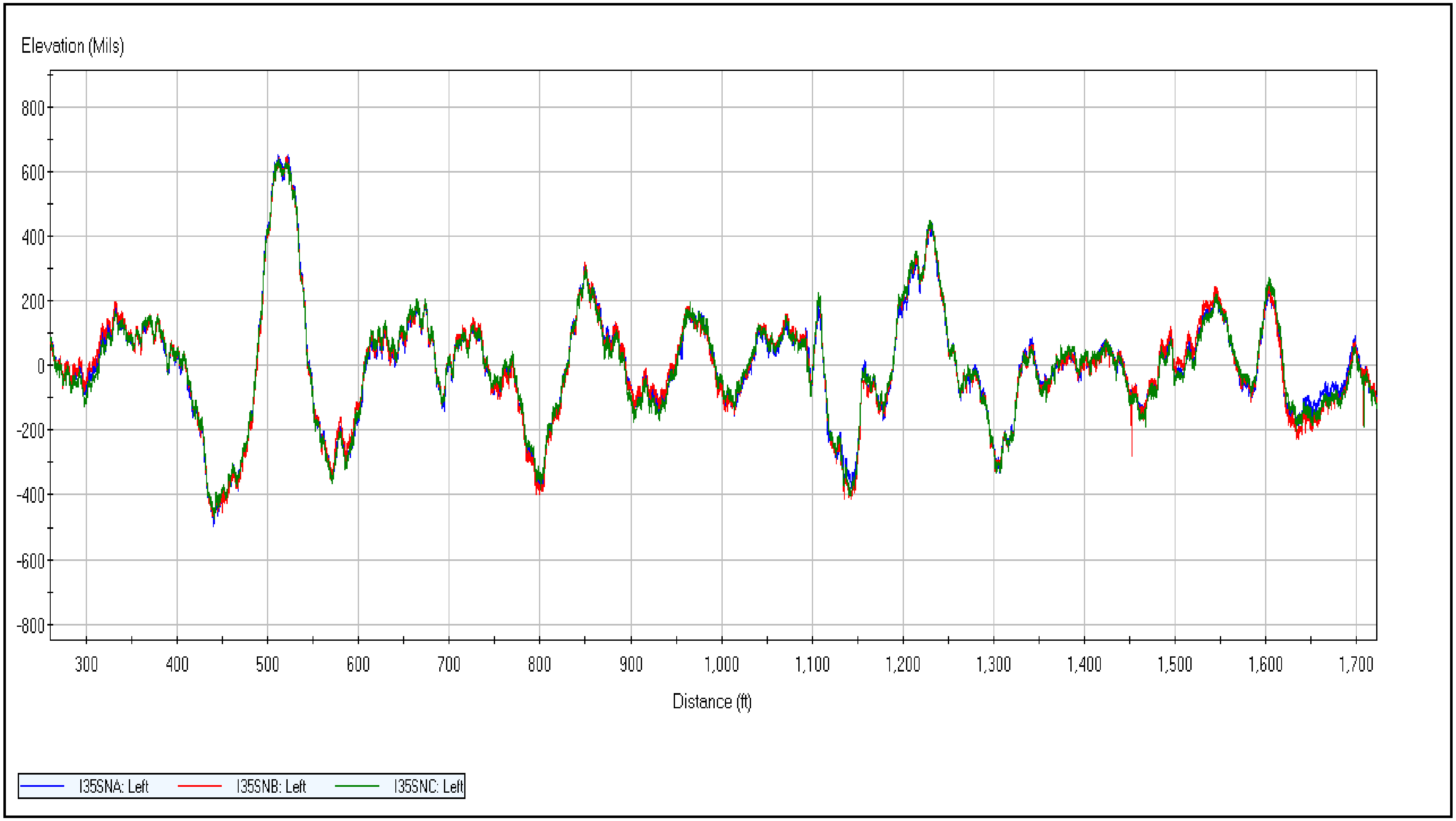


Figure B3. LWP Profiles on I-35 Southbound Inside Lane of TxDOT WIM Site 531.

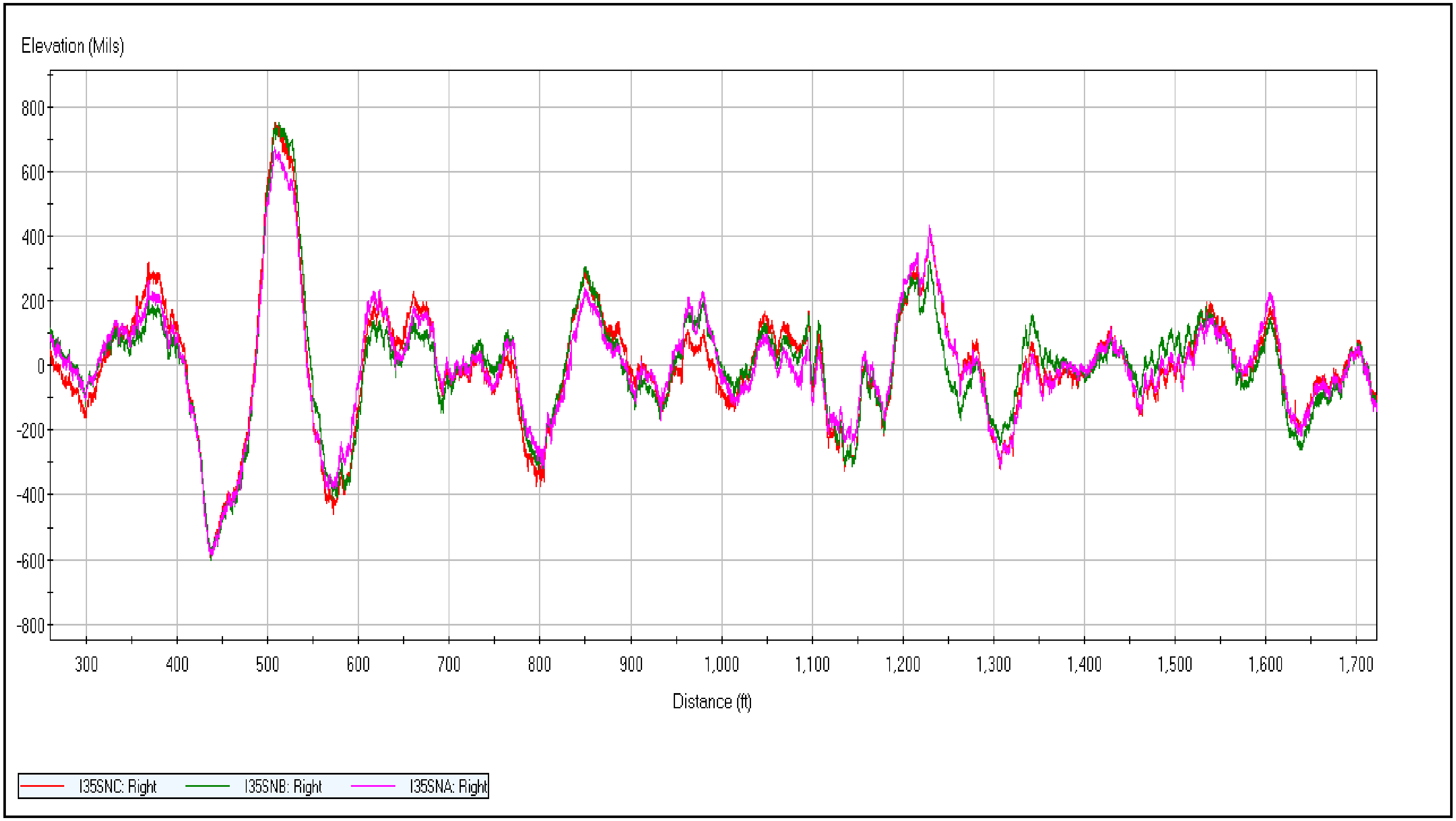


Figure B4. RWP Profiles on I-35 Southbound Inside Lane of TxDOT WIM Site 531.

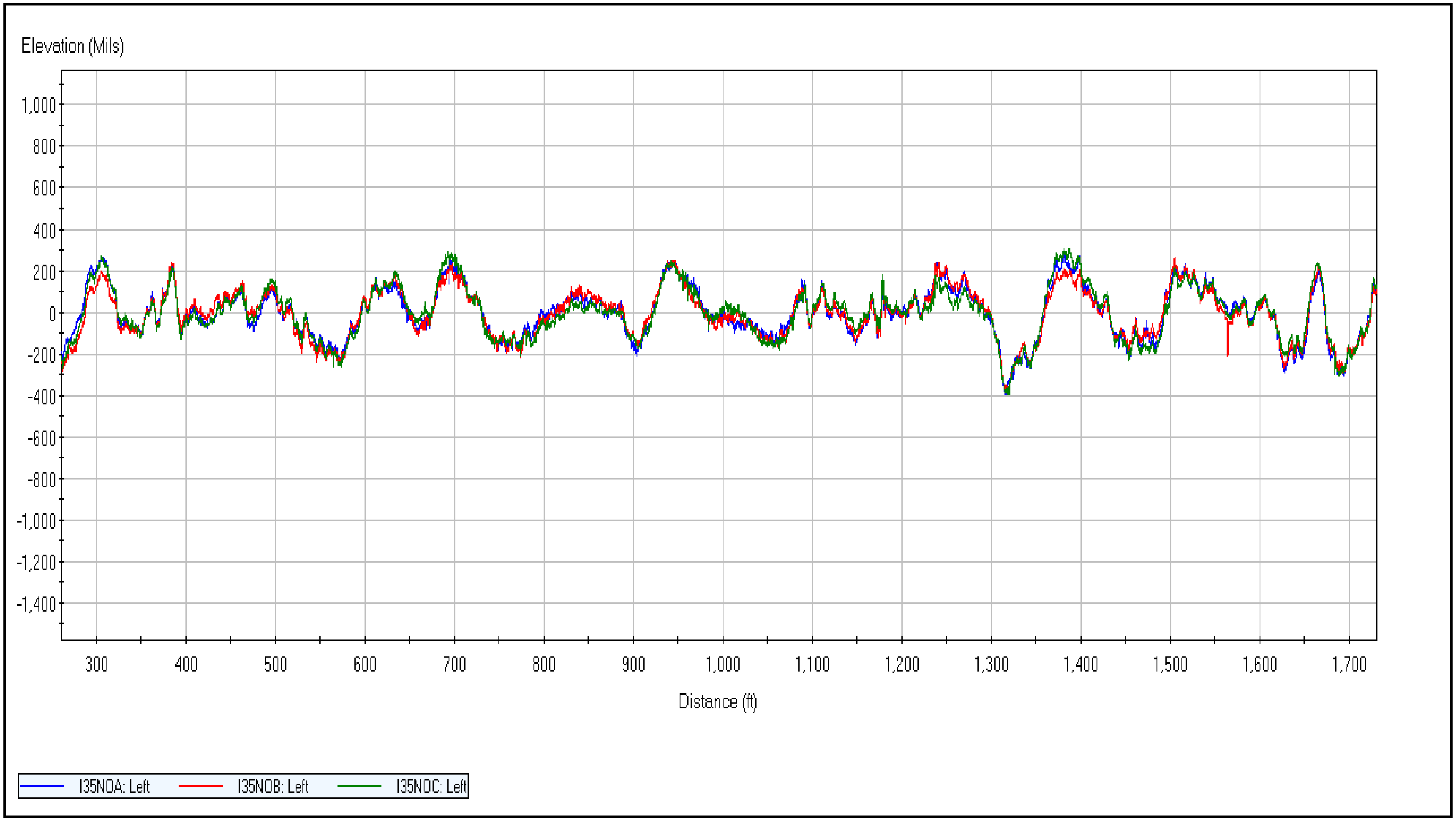


Figure B5. LWP Profiles on I-35 Northbound Outside Lane of TxDOT WIM Site 531.

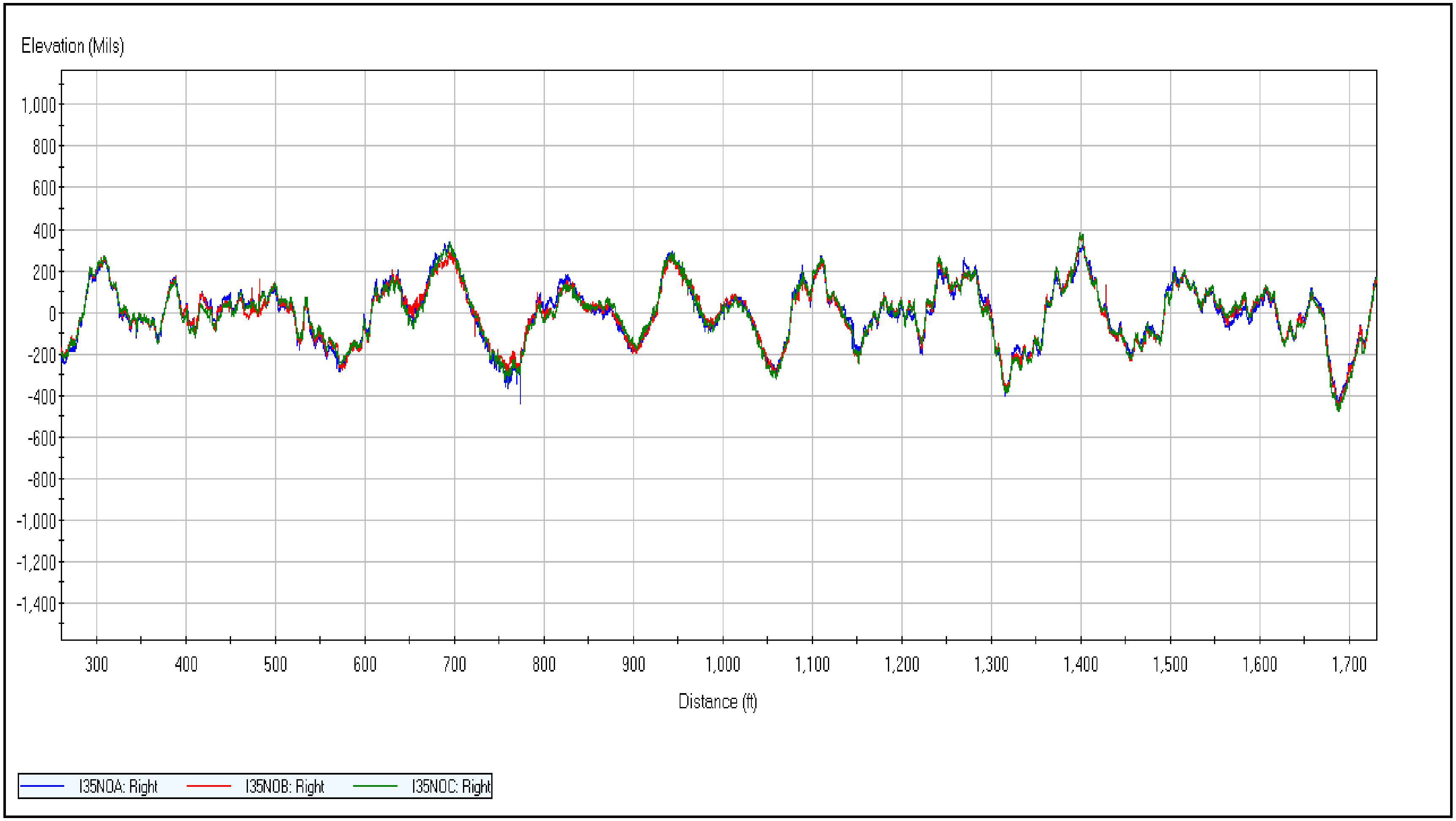


Figure B6. RWP Profiles on I-35 Northbound Outside Lane of TxDOT WIM Site 531.

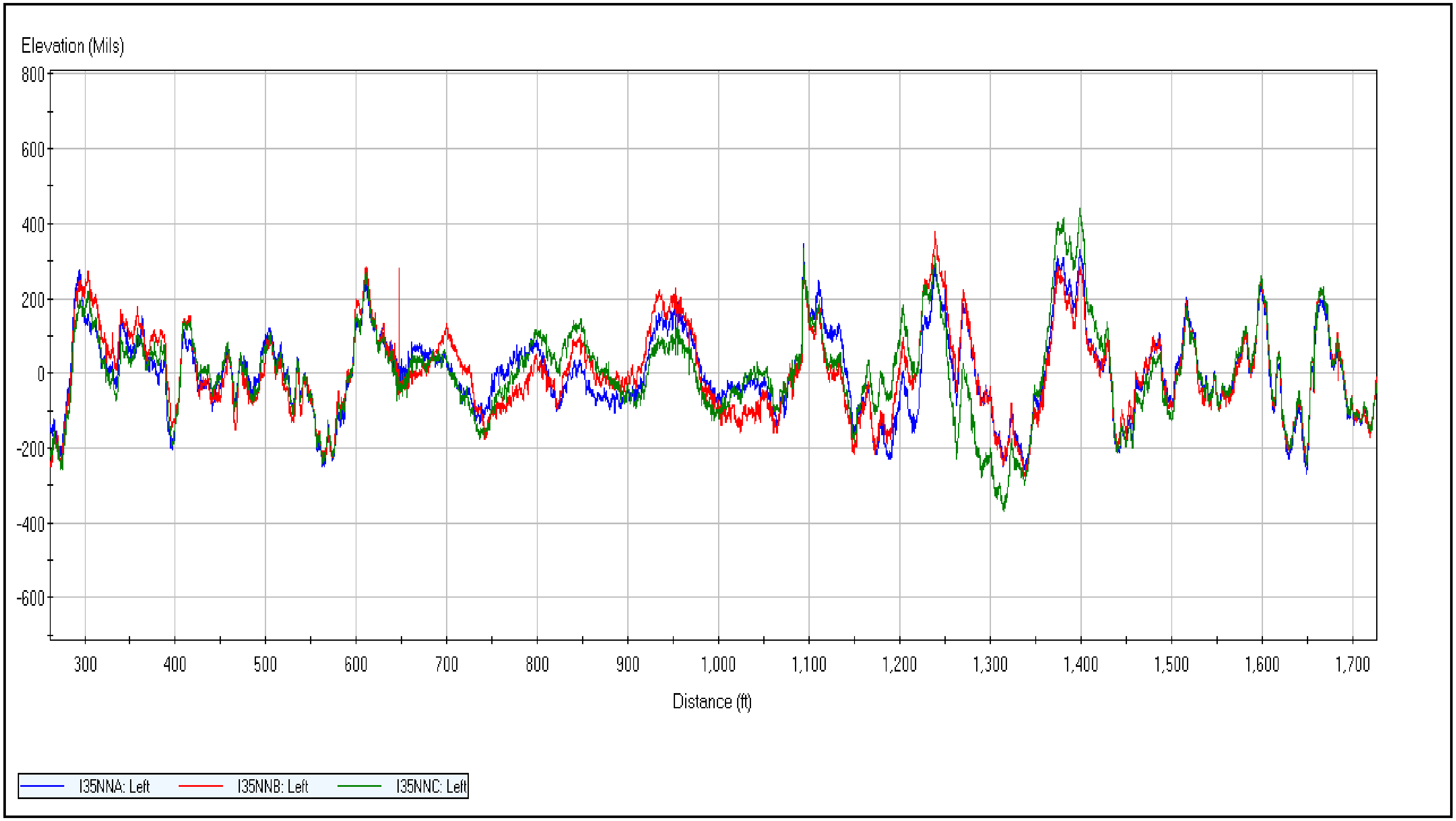


Figure B7. LWP Profiles on I-35 Northbound Inside Lane of TxDOT WIM Site 531.

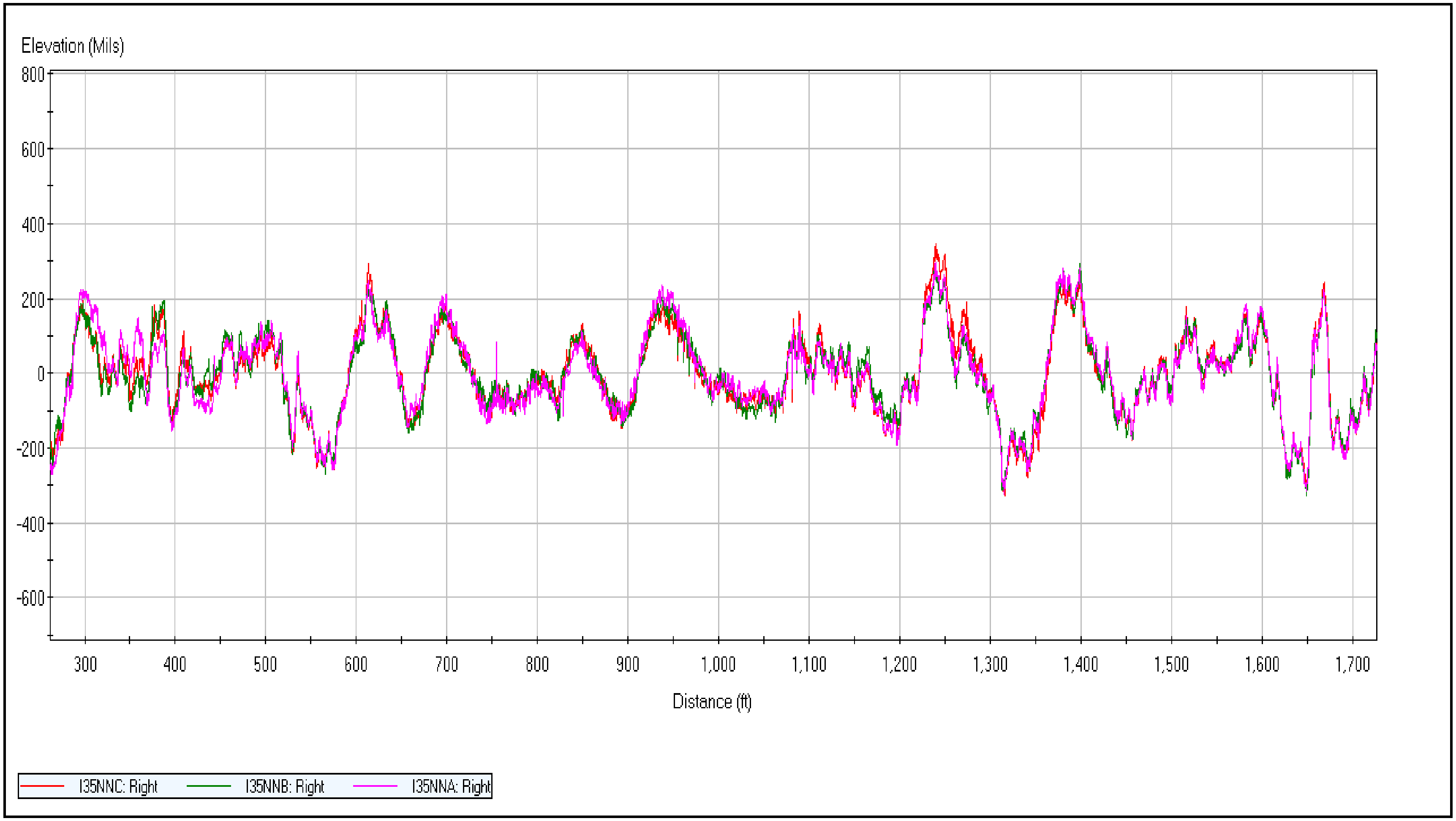


Figure B8. RWP Profiles on I-35 Northbound Inside Lane of TxDOT WIM Site 531.

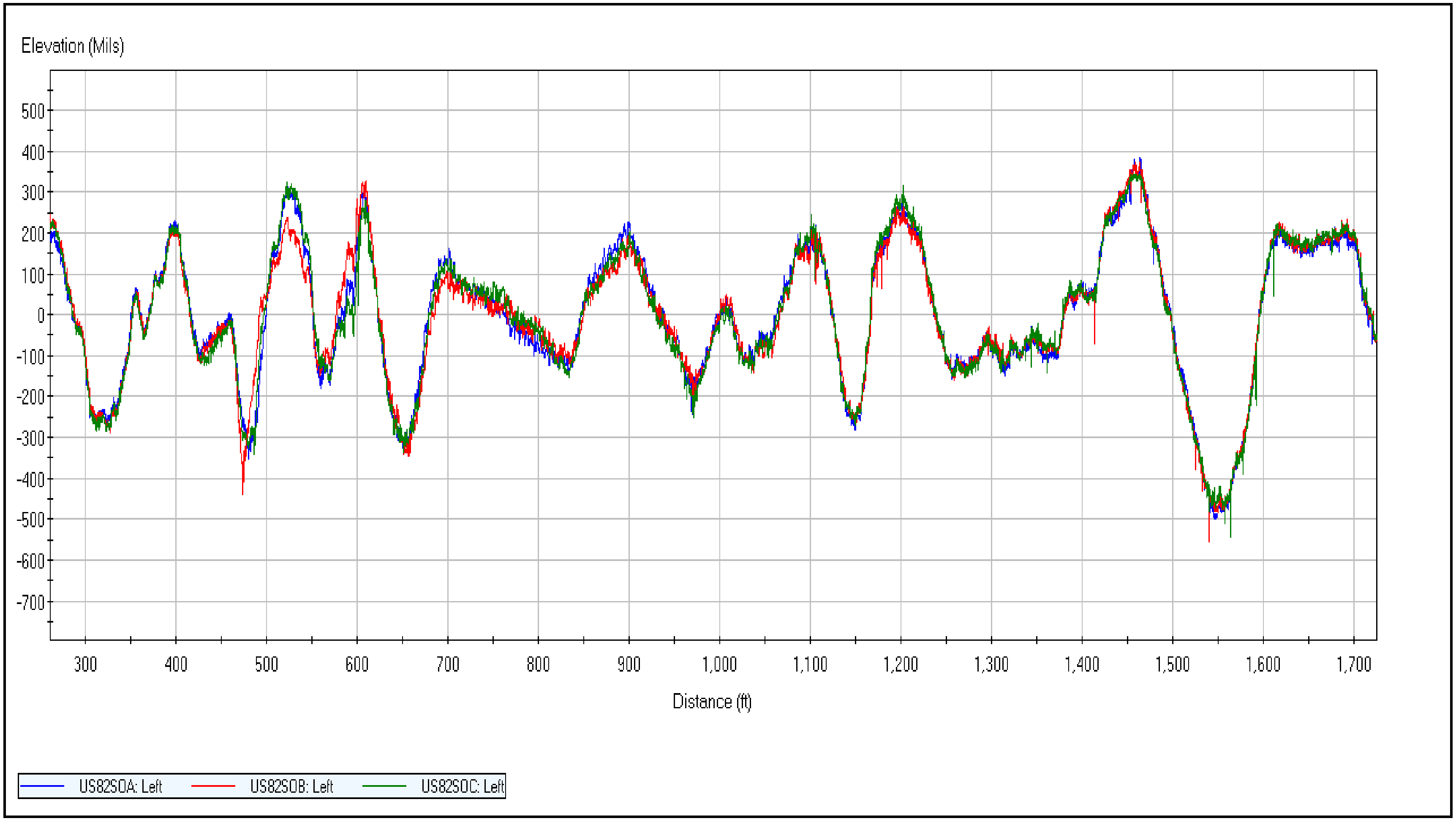


Figure B9. LWP Profiles on US277/US82 Southbound Outside Lane of TxDOT WIM Site 530.

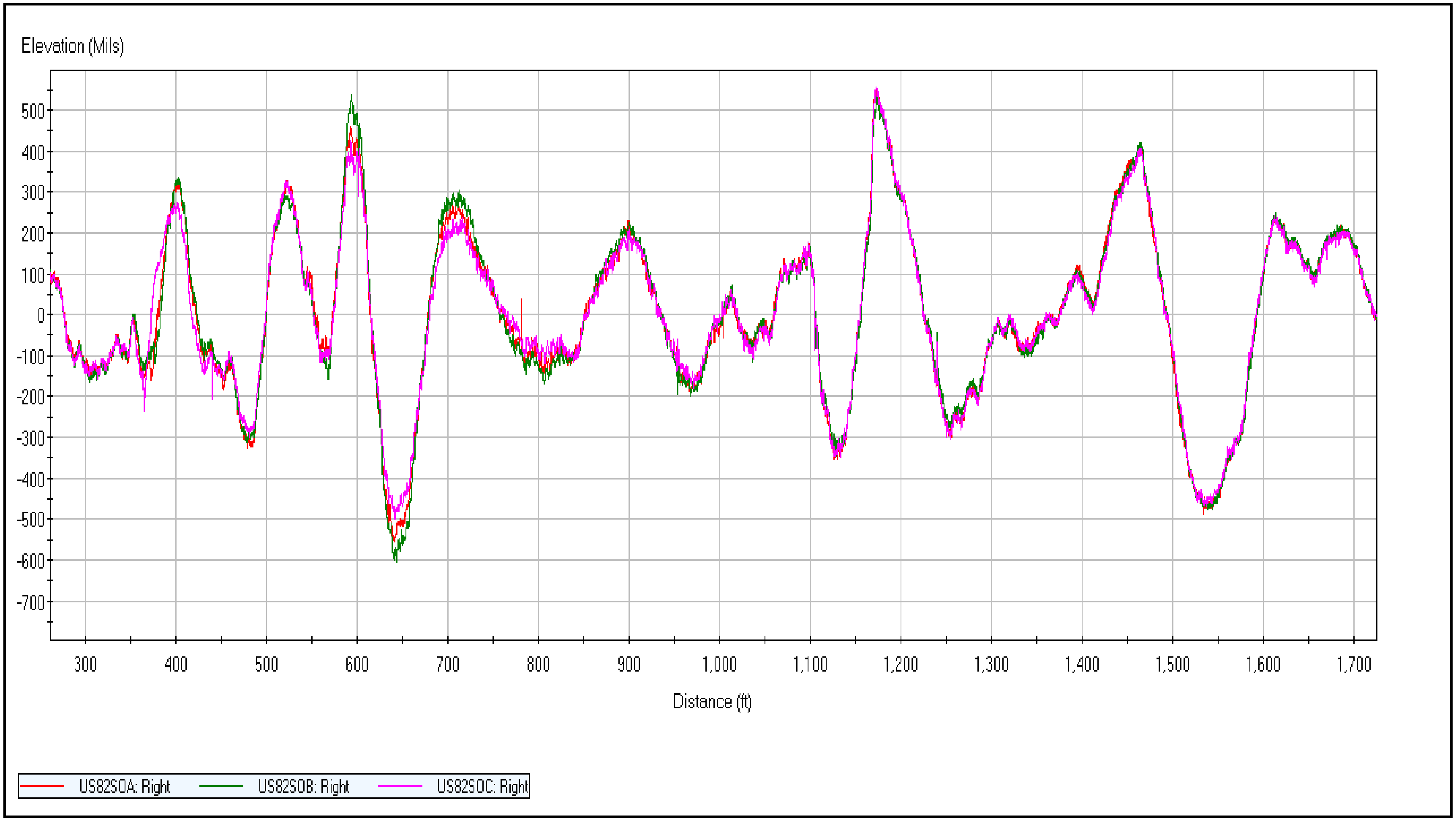


Figure B10. RWP Profiles on US277/US82 Southbound Outside Lane of TxDOT WIM Site 530.

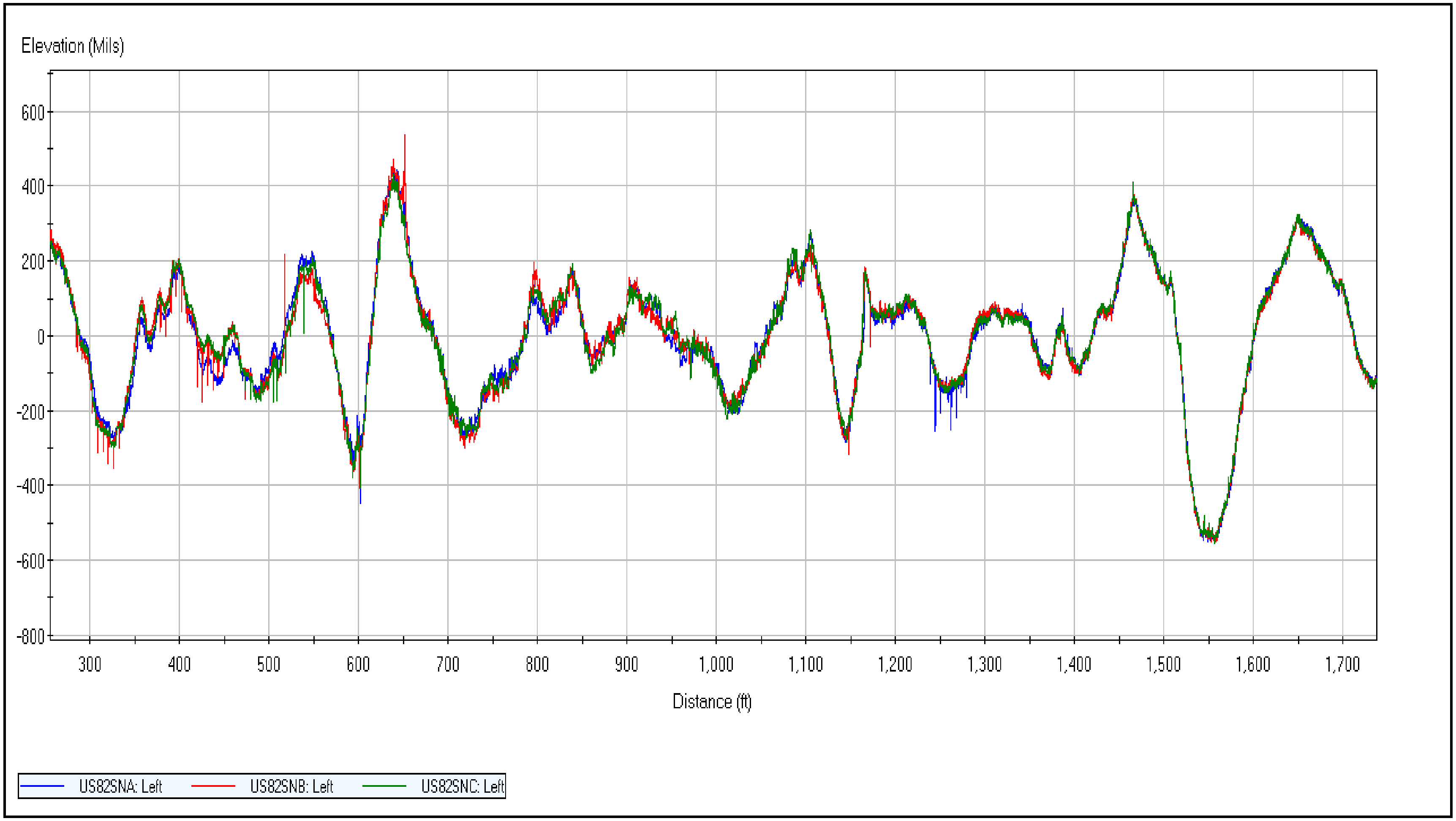


Figure B11. LWP Profiles on US277/US82 Southbound Inside Lane of TxDOT WIM Site 530.

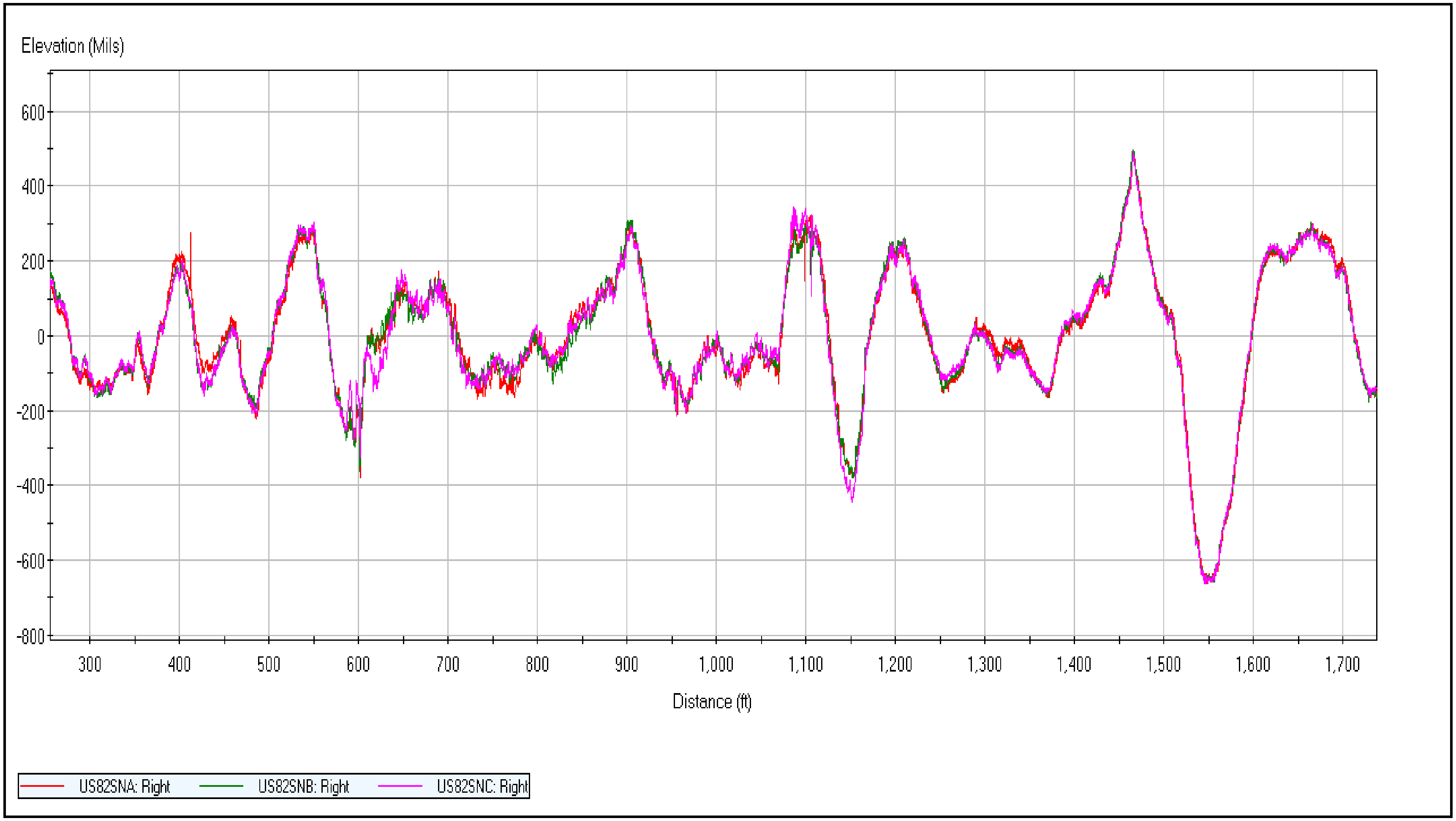


Figure B12. RWP Profiles on US277/US82 Southbound Inside Lane of TxDOT WIM Site 530.

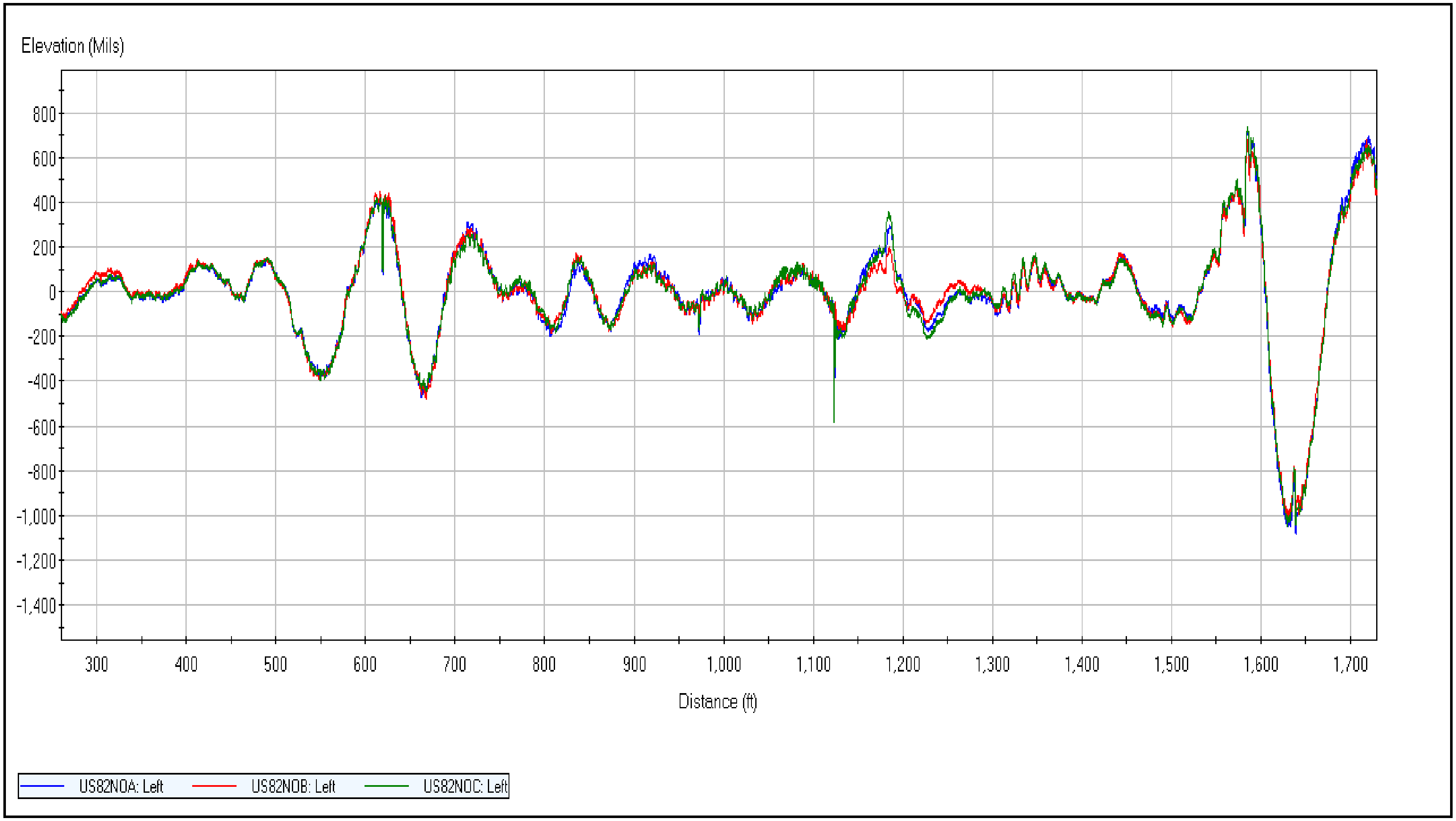


Figure B13. LWP Profiles on US277/US82 Northbound Outside Lane of TxDOT WIM Site 530.

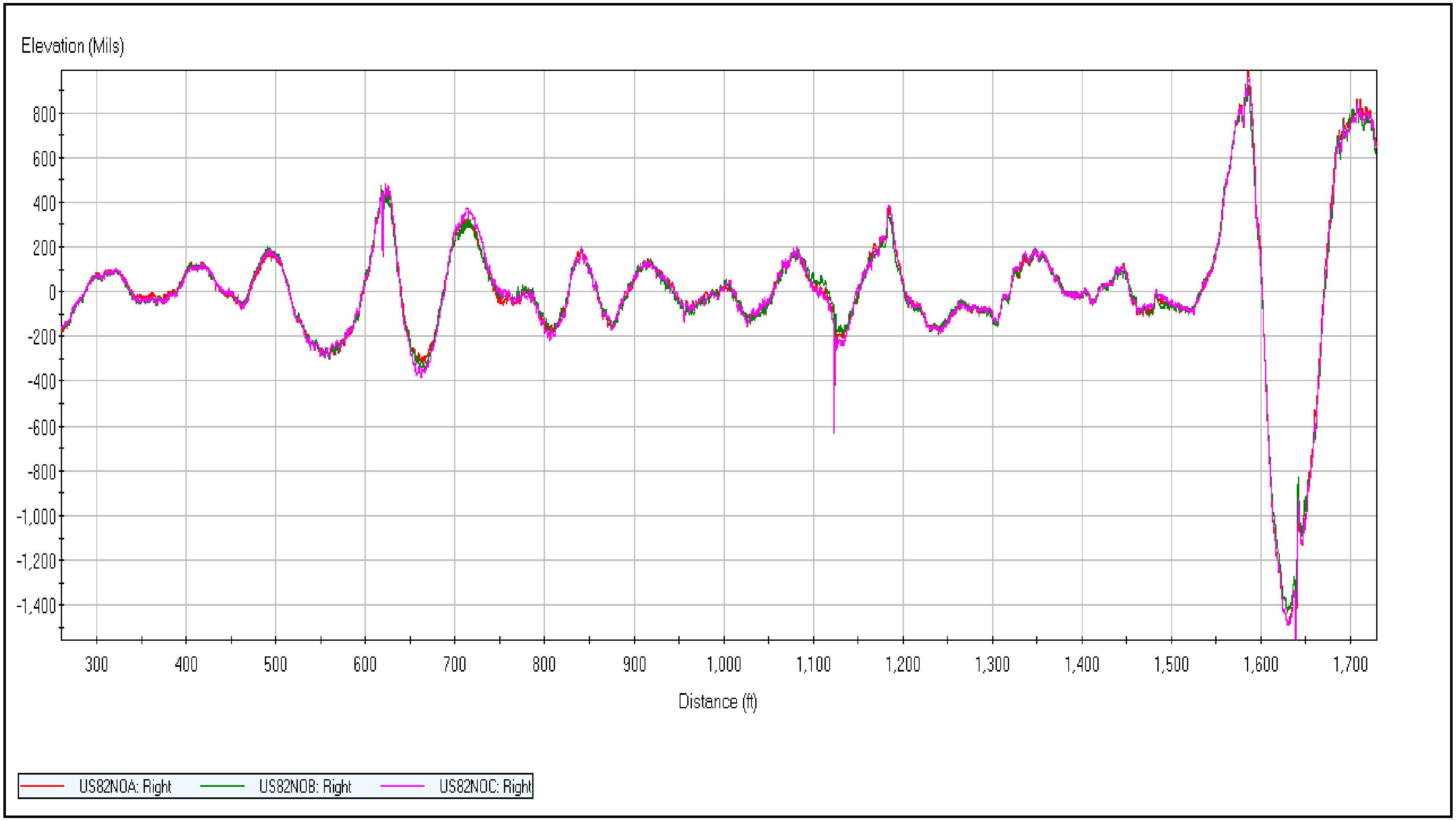


Figure B14. RWP Profiles on US277/US82 Northbound Outside Lane of TxDOT WIM Site 530.

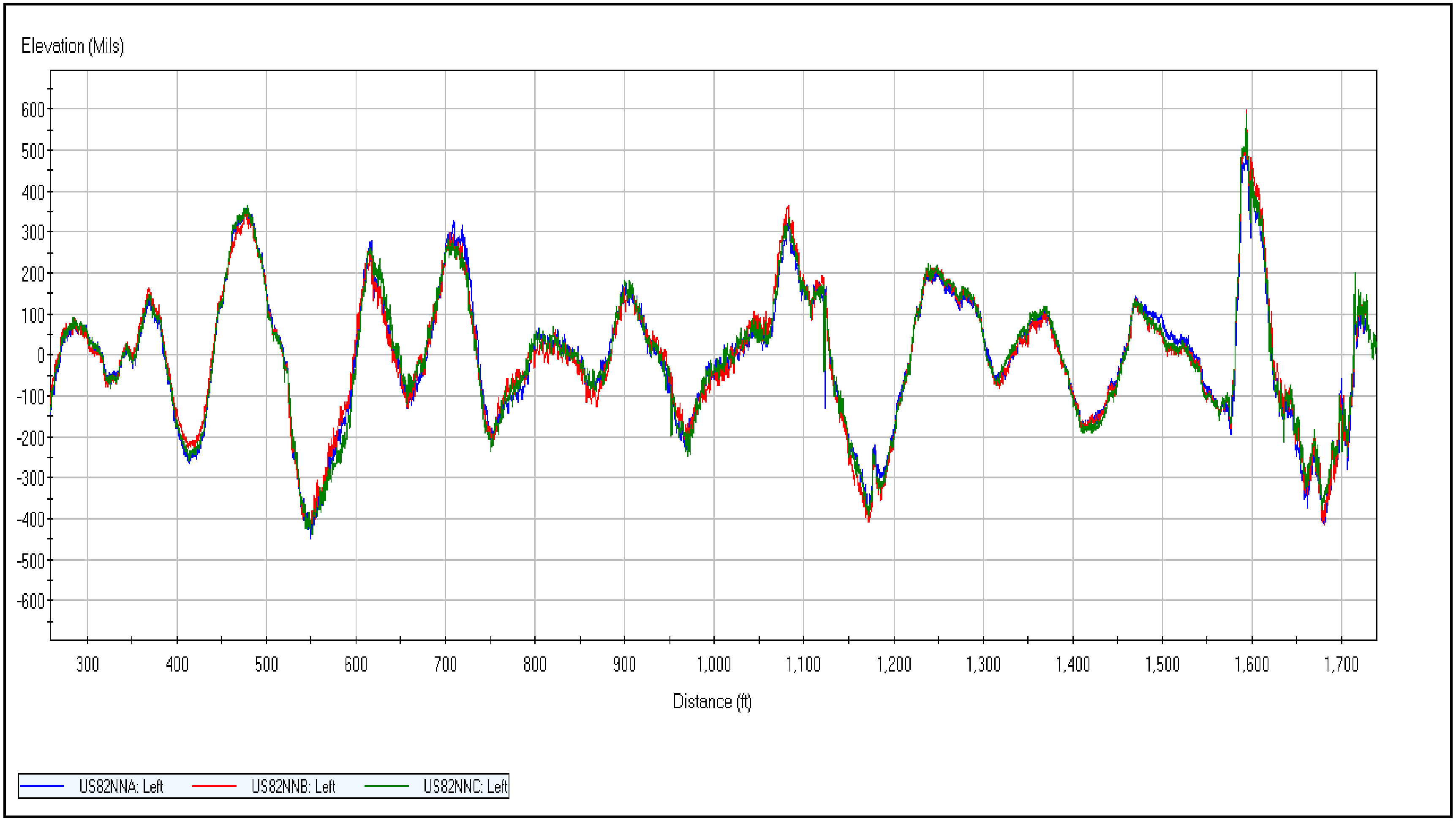


Figure B15. LWP Profiles on US277/US82 Northbound Inside Lane of TxDOT WIM Site 530.

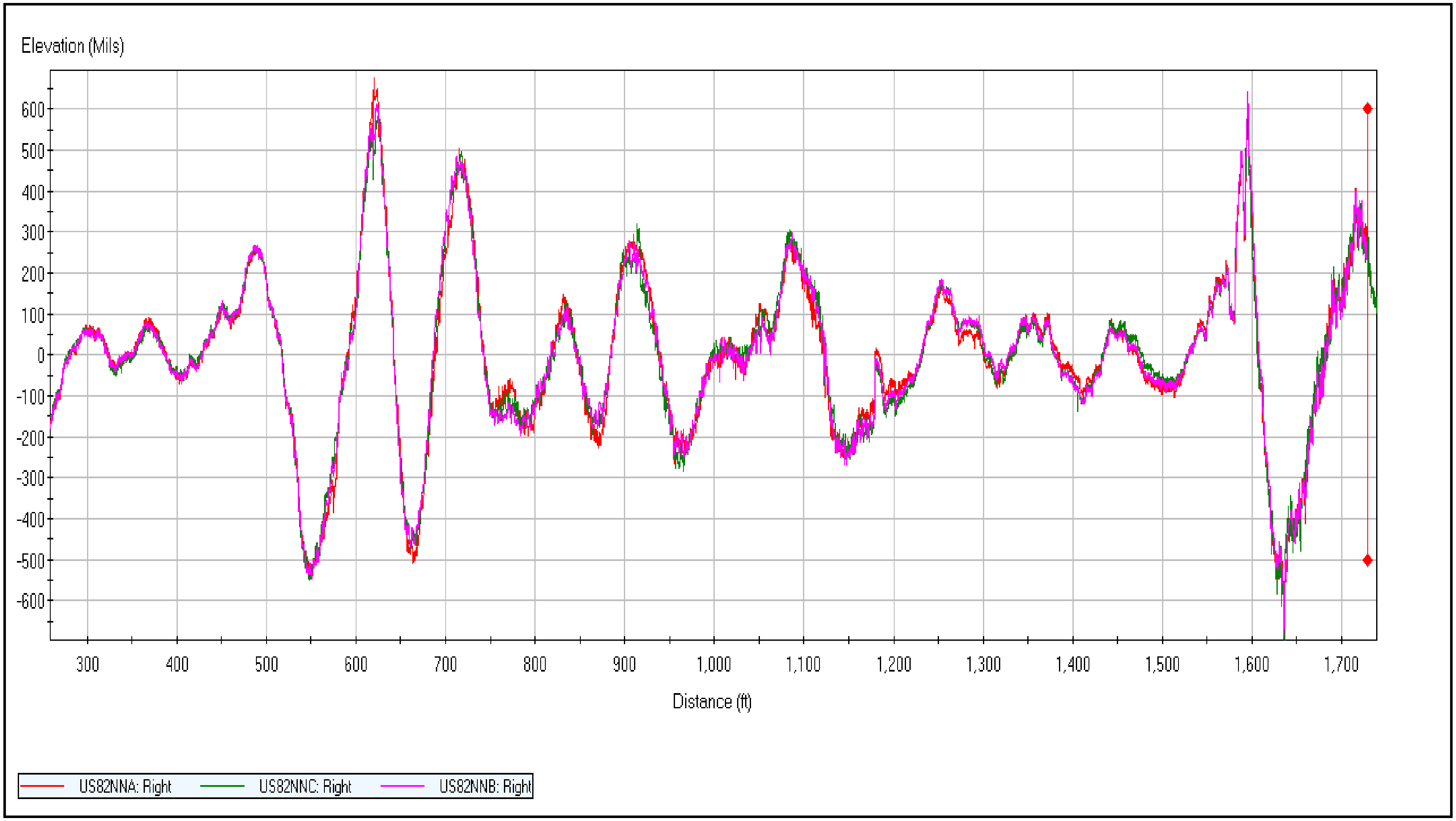


Figure B16. RWP Profiles on US277/US82 Northbound Inside Lane of TxDOT WIM Site 530.

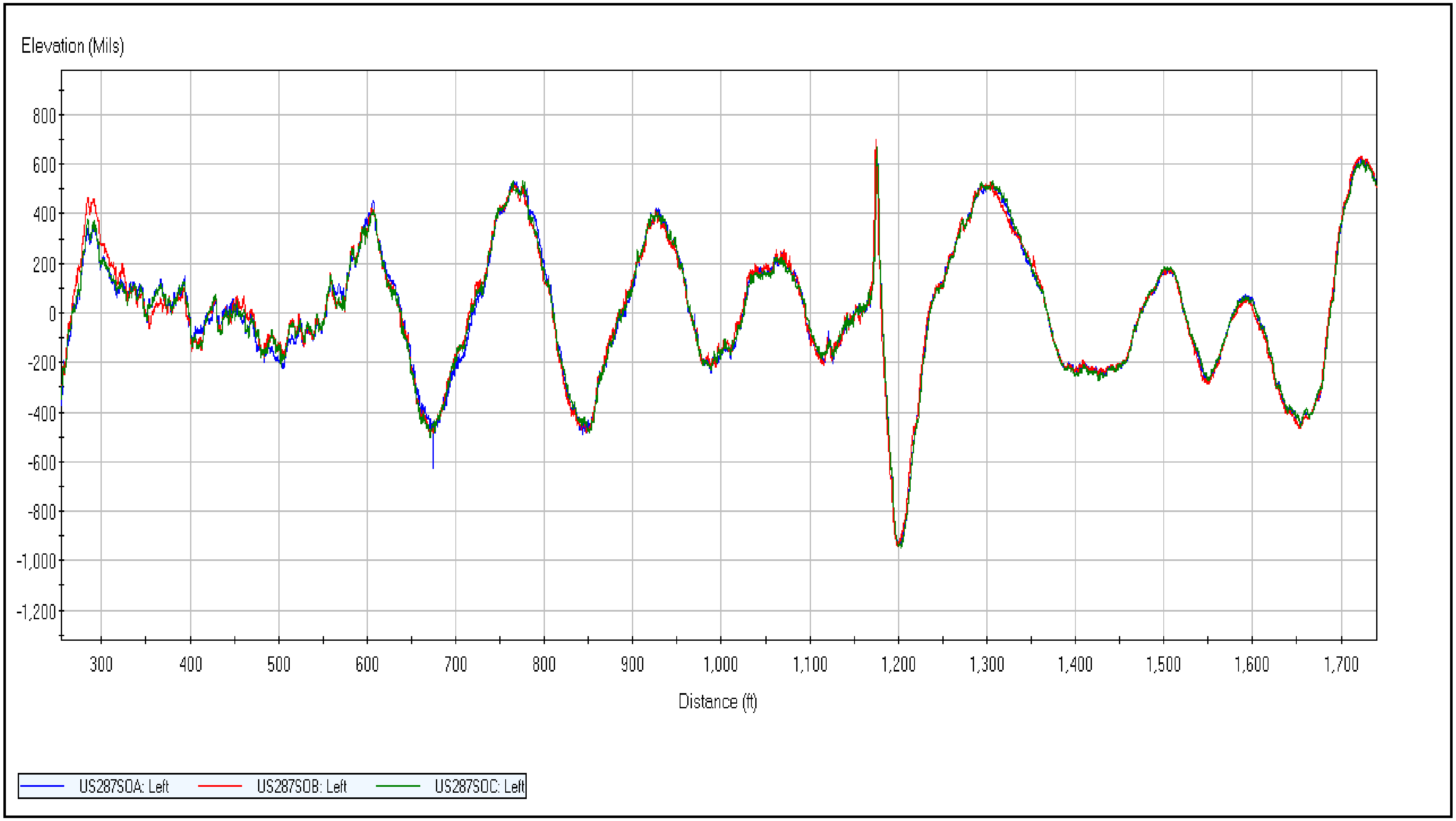


Figure B17. LWP Profiles on US287 Southbound Outside Lane of TxDOT WIM Site 528.

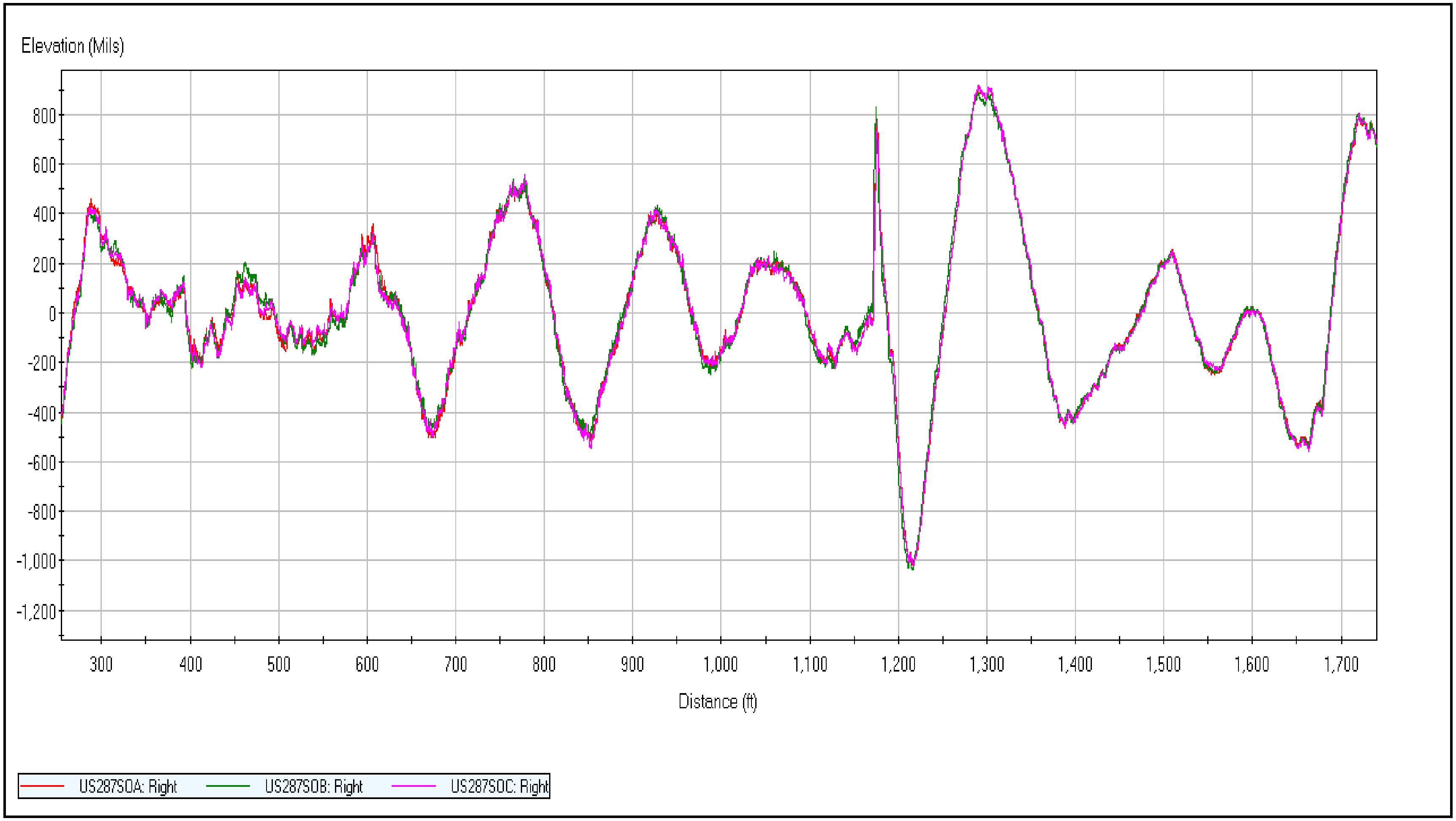


Figure B18. RWP Profiles on US287 Southbound Outside Lane of TxDOT WIM Site 528.

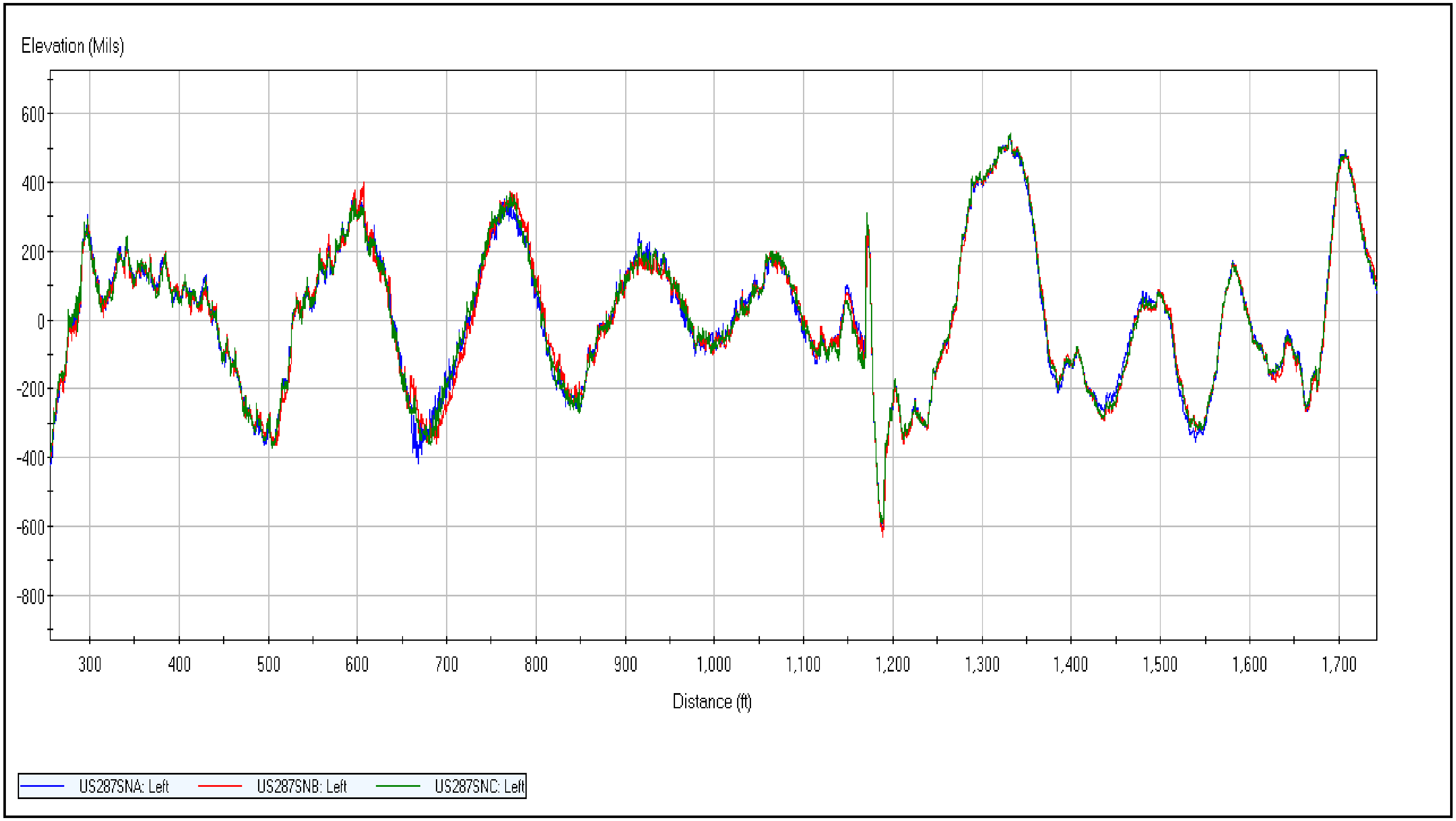


Figure B19. LWP Profiles on US287 Southbound Inside Lane of TxDOT WIM Site 528.

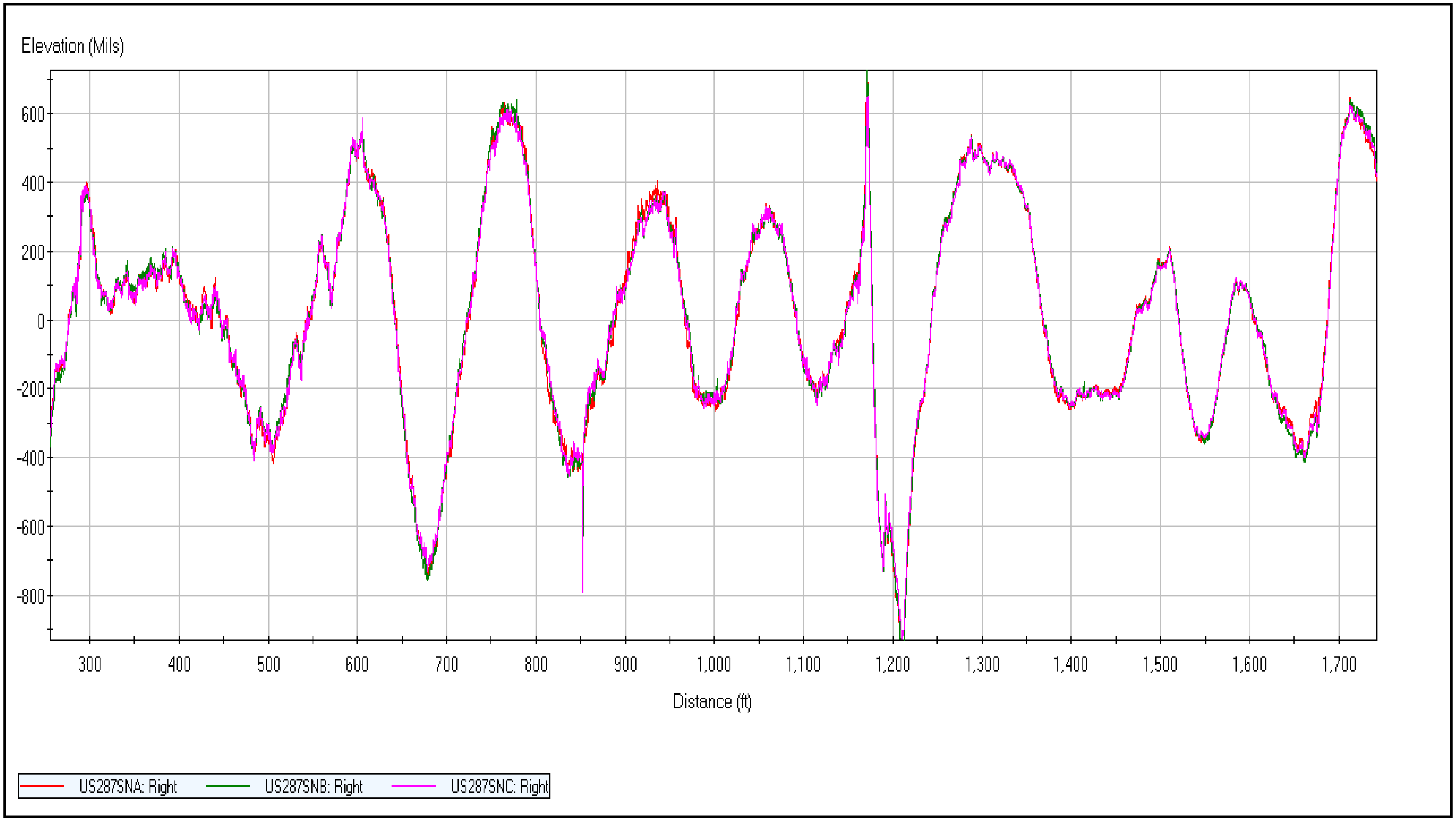


Figure B20. RWP Profiles on US287 Southbound Inside Lane of TxDOT WIM Site 528.

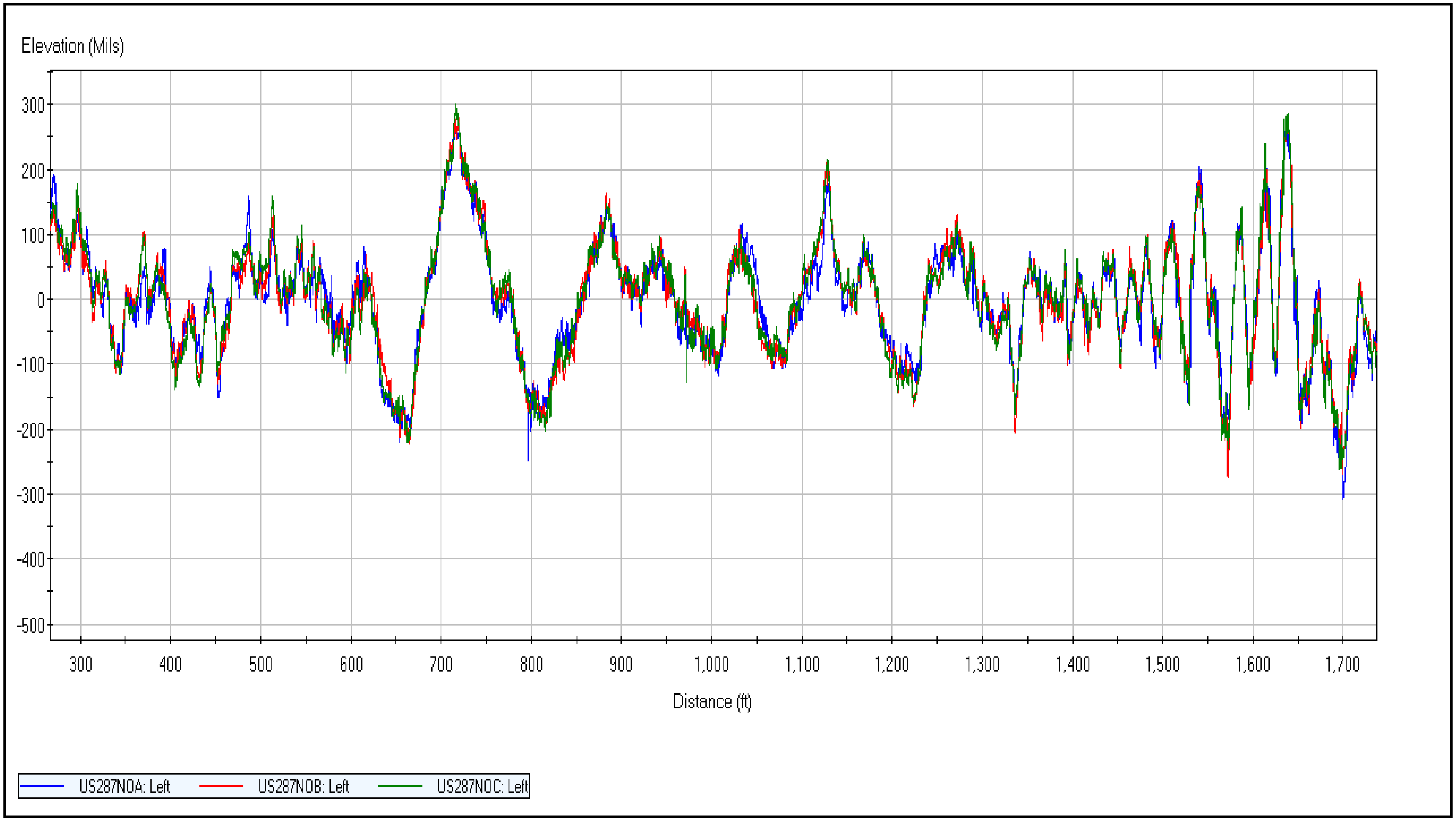


Figure B21. LWP Profiles on US287 Northbound Outside Lane of TxDOT WIM Site 528.

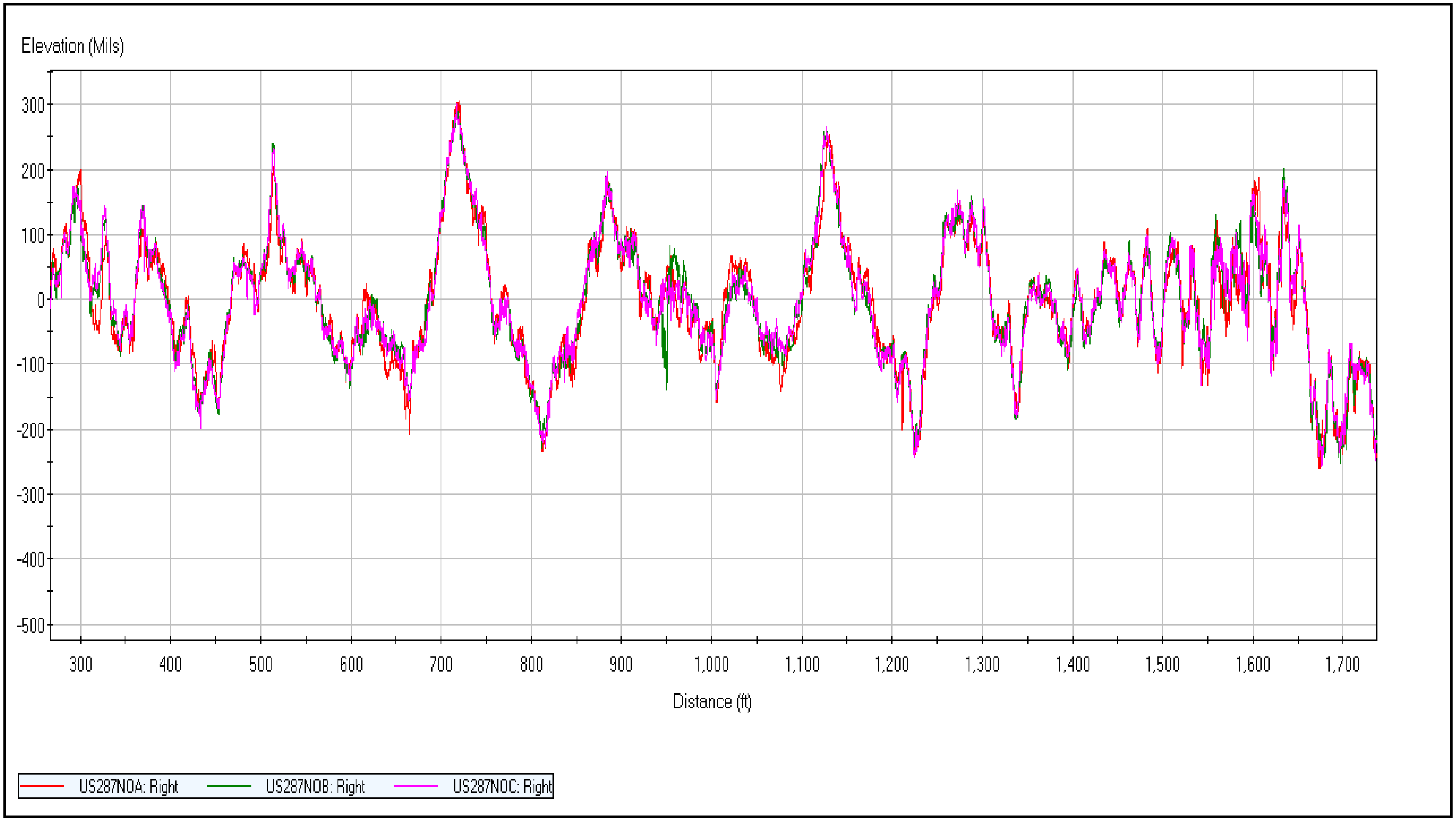


Figure B22. RWP Profiles on US287 Northbound Outside Lane of TxDOT WIM Site 528.

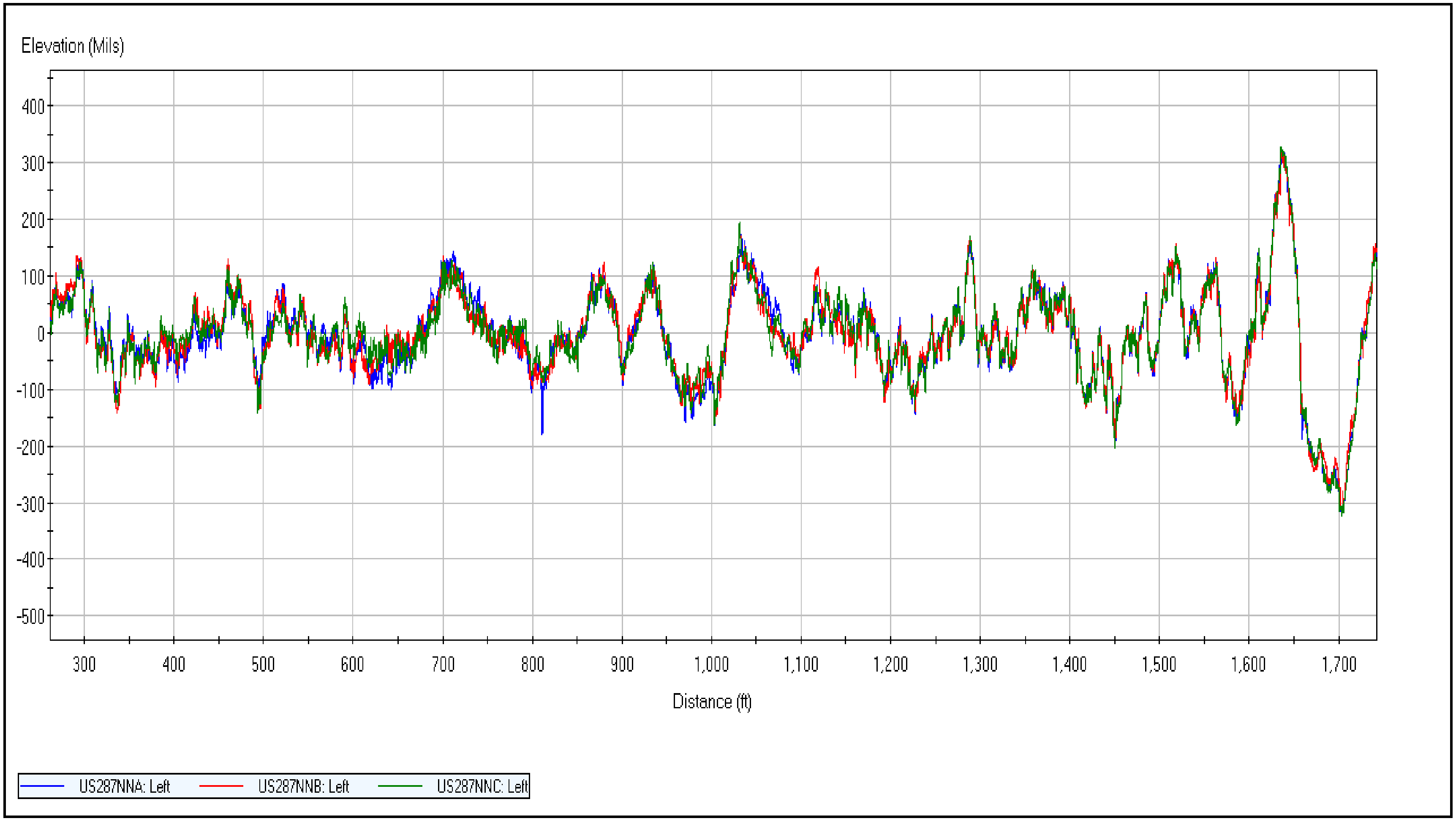


Figure B23. LWP Profiles on US287 Northbound Inside Lane of TxDOT WIM Site 528.

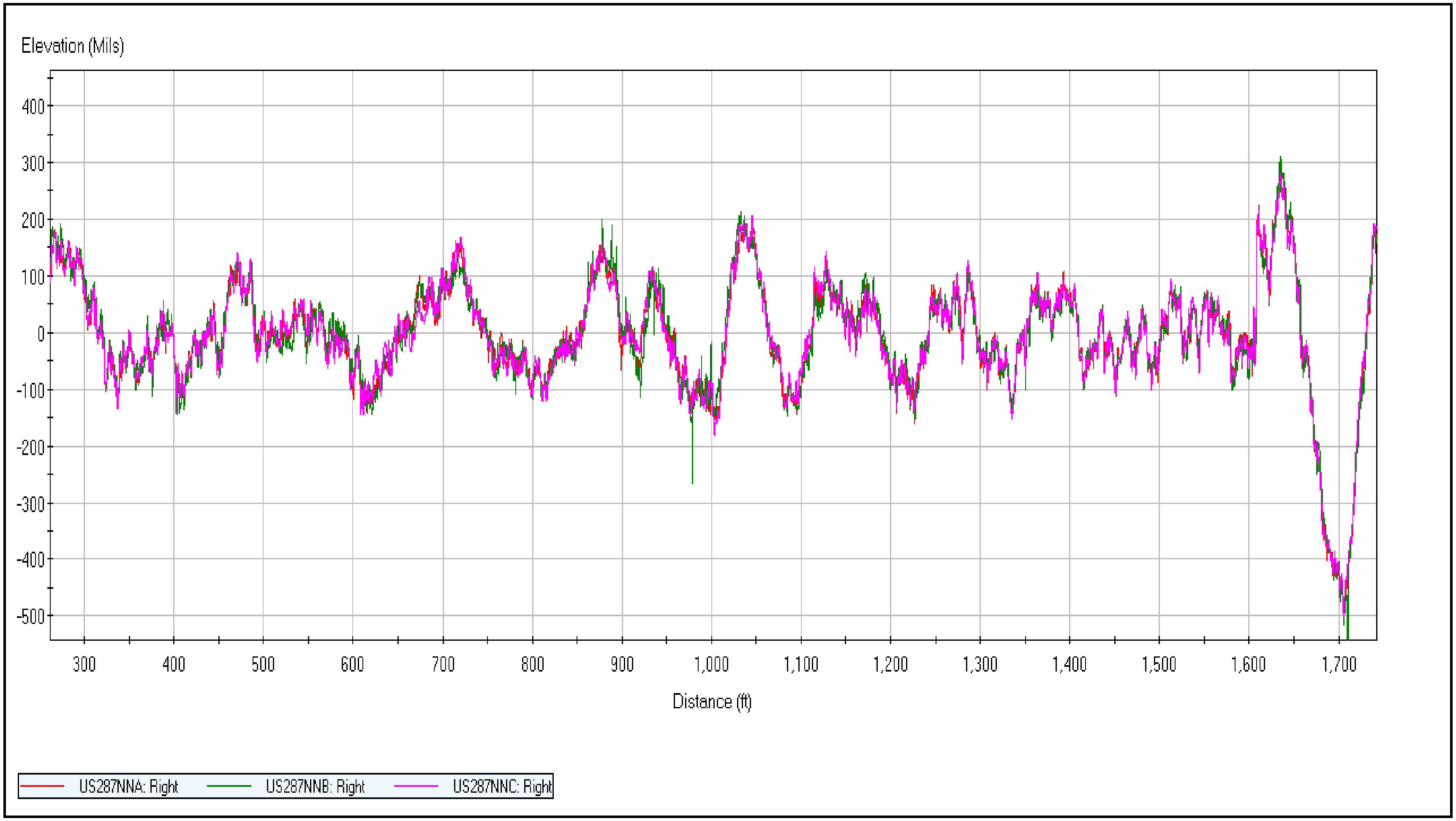


Figure B24. RWP Profiles on US287 Northbound Inside Lane of TxDOT WIM Site 528.

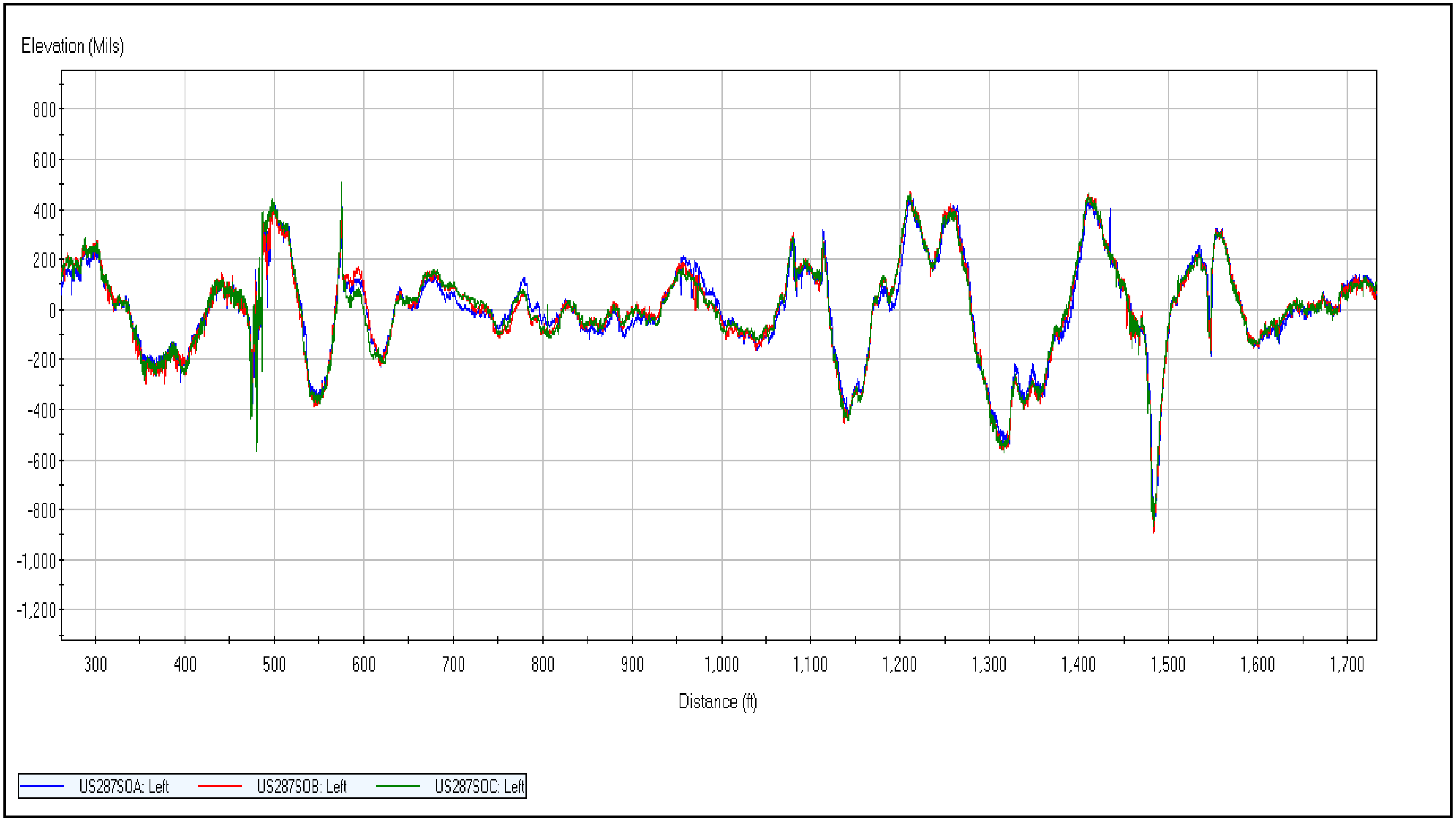


Figure B25. LWP Profiles on US287 Southbound Outside Lane of TxDOT WIM Site 506.

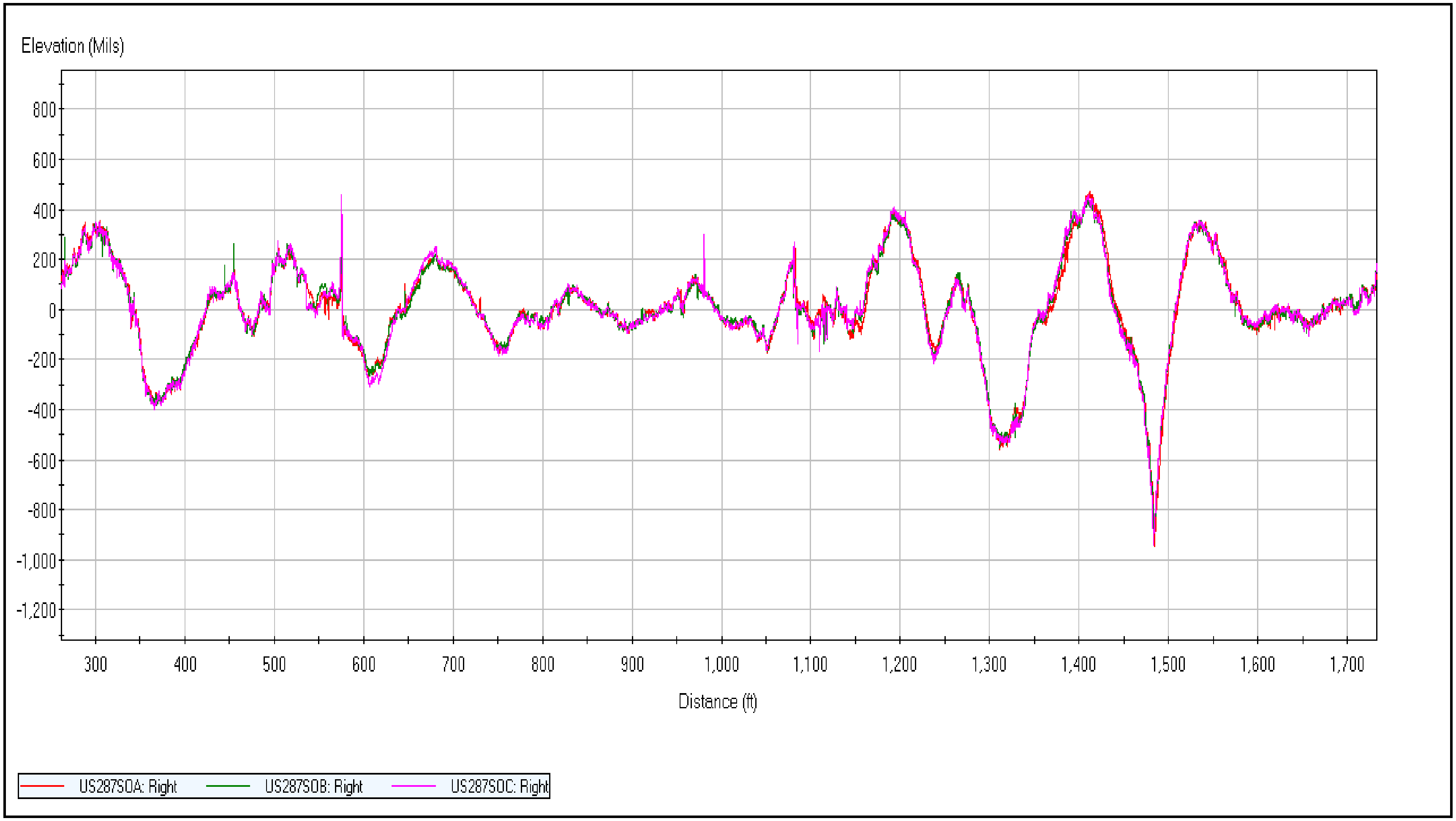


Figure B26. RWP Profiles on US287 Southbound Outside Lane of TxDOT WIM Site 506.

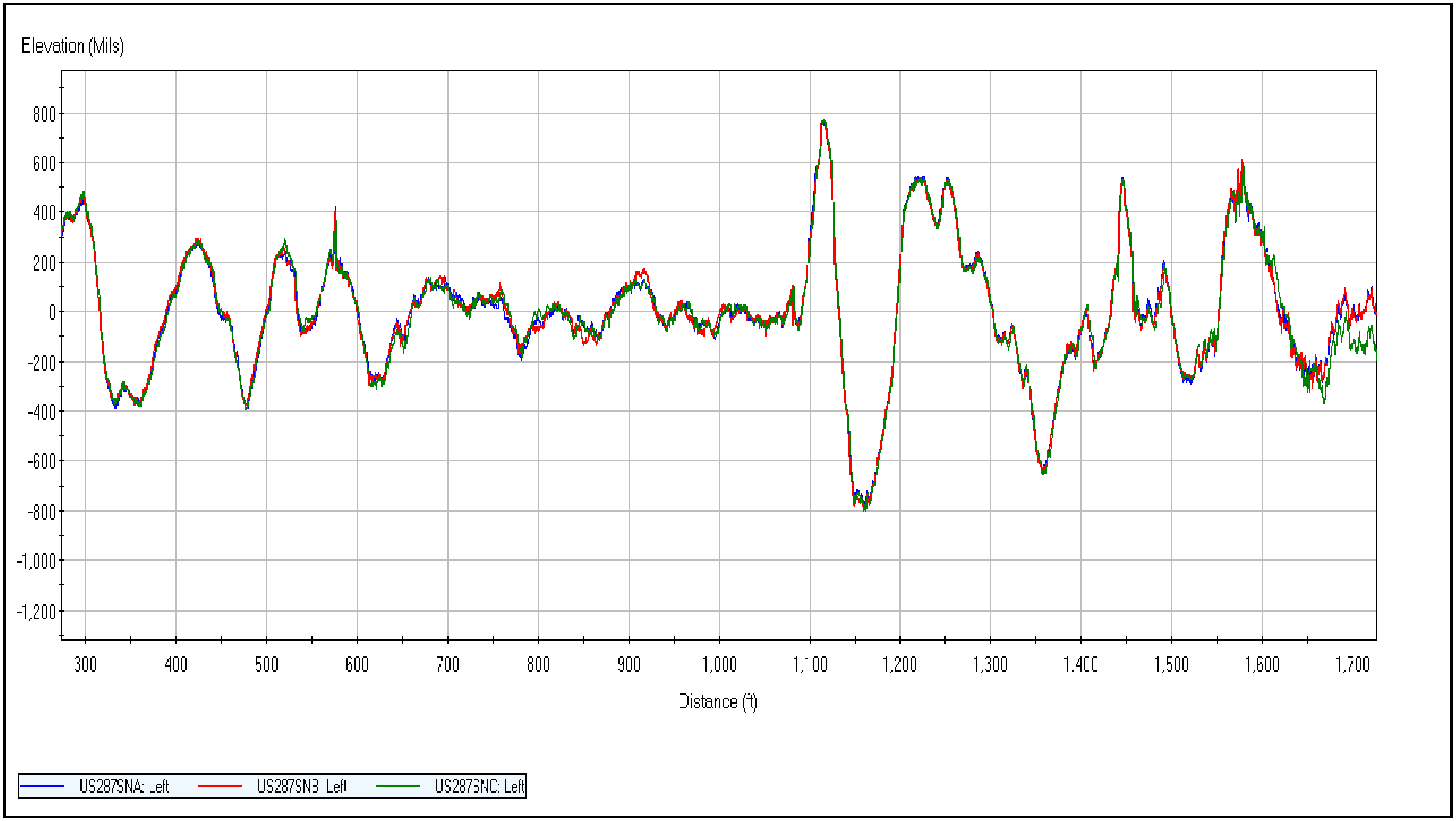


Figure B27. LWP Profiles on US287 Southbound Inside Lane of TxDOT WIM Site 506.

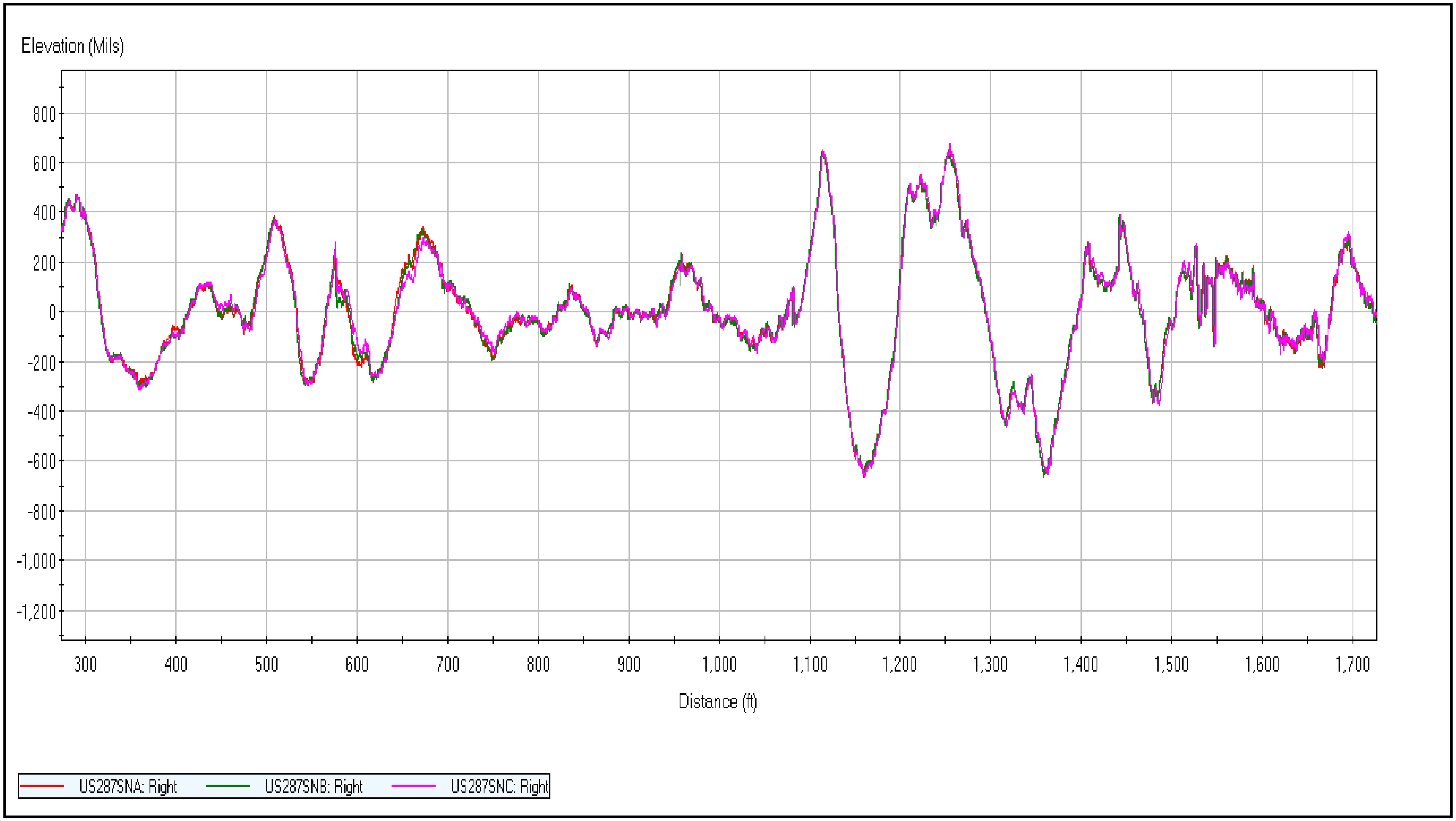


Figure B28. RWP Profiles on US287 Southbound Inside Lane of TxDOT WIM Site 506.

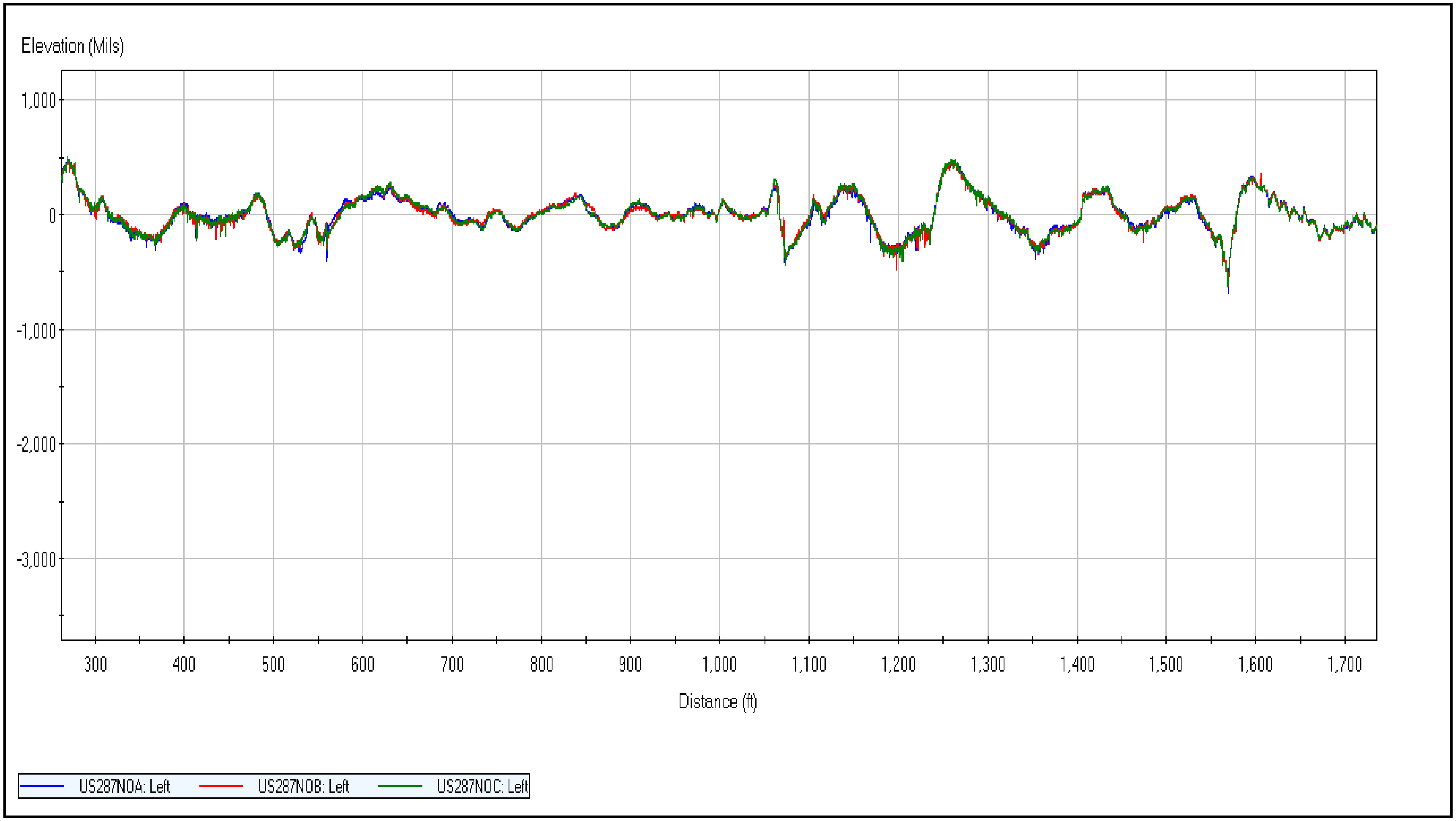


Figure B29. LWP Profiles on US287 Northbound Outside Lane of TxDOT WIM Site 506.

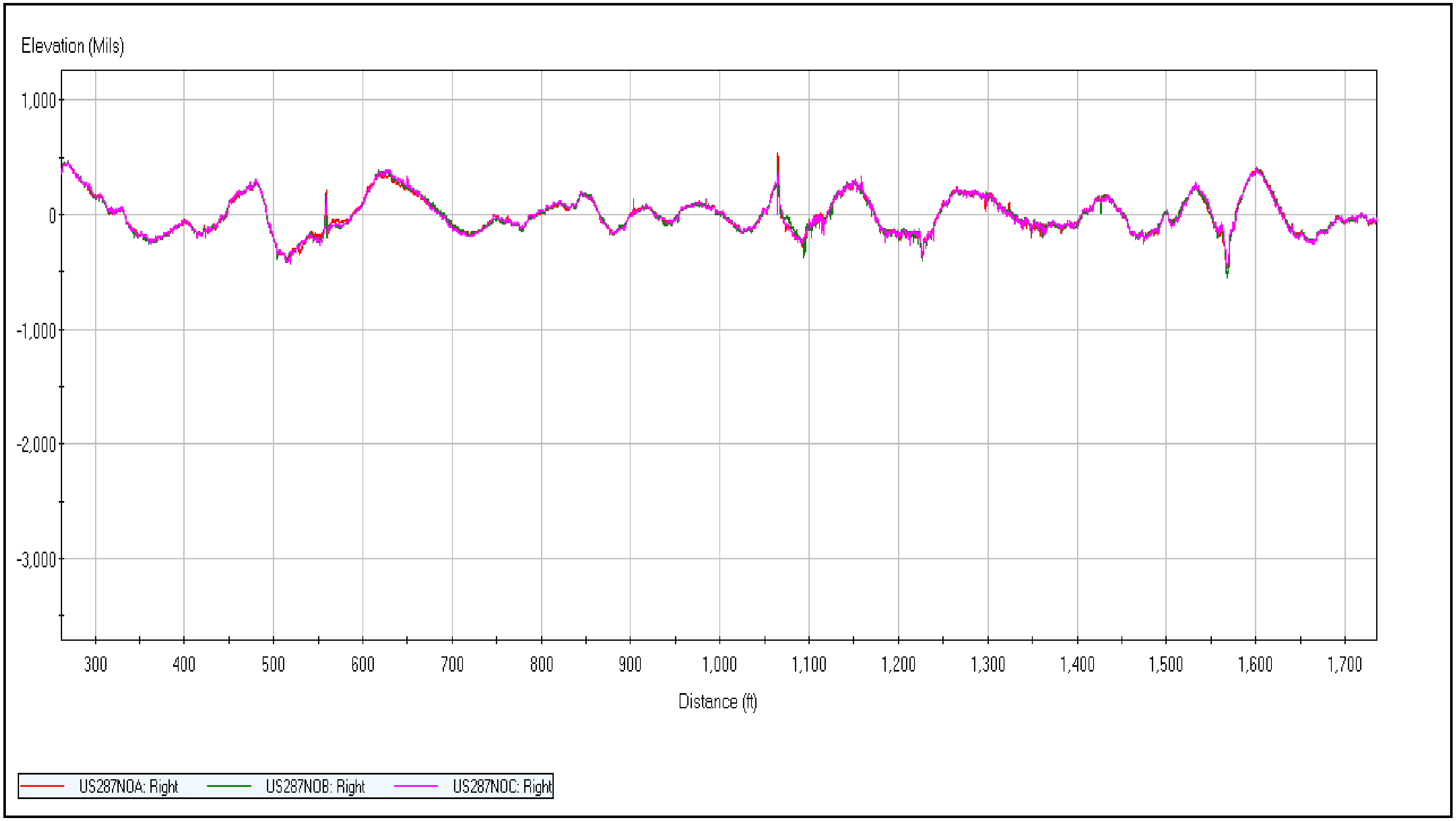


Figure B30. RWP Profiles on US287 Northbound Outside Lane of TxDOT WIM Site 506.

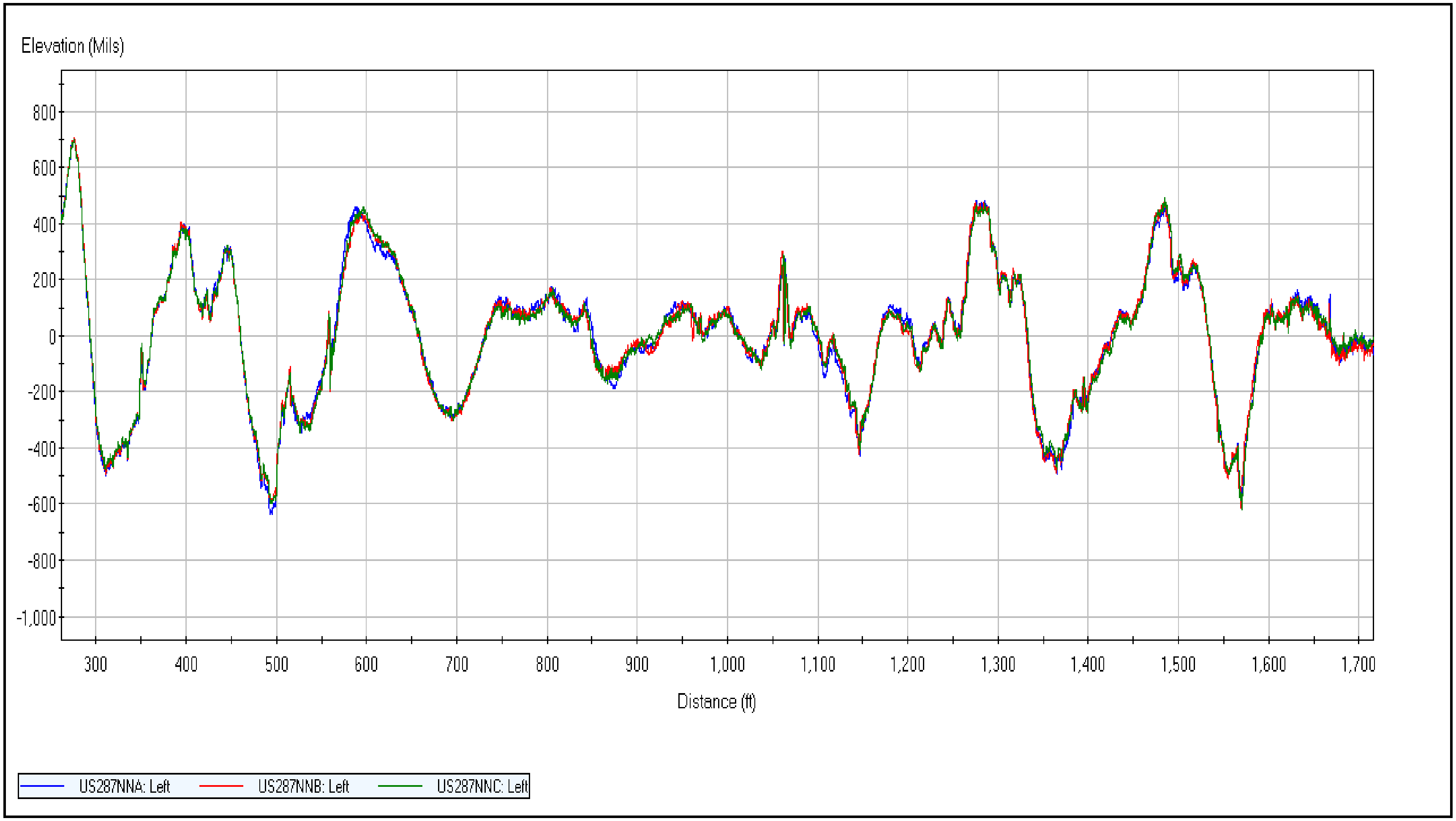


Figure B31. LWP Profiles on US287 Northbound Inside Lane of TxDOT WIM Site 506.

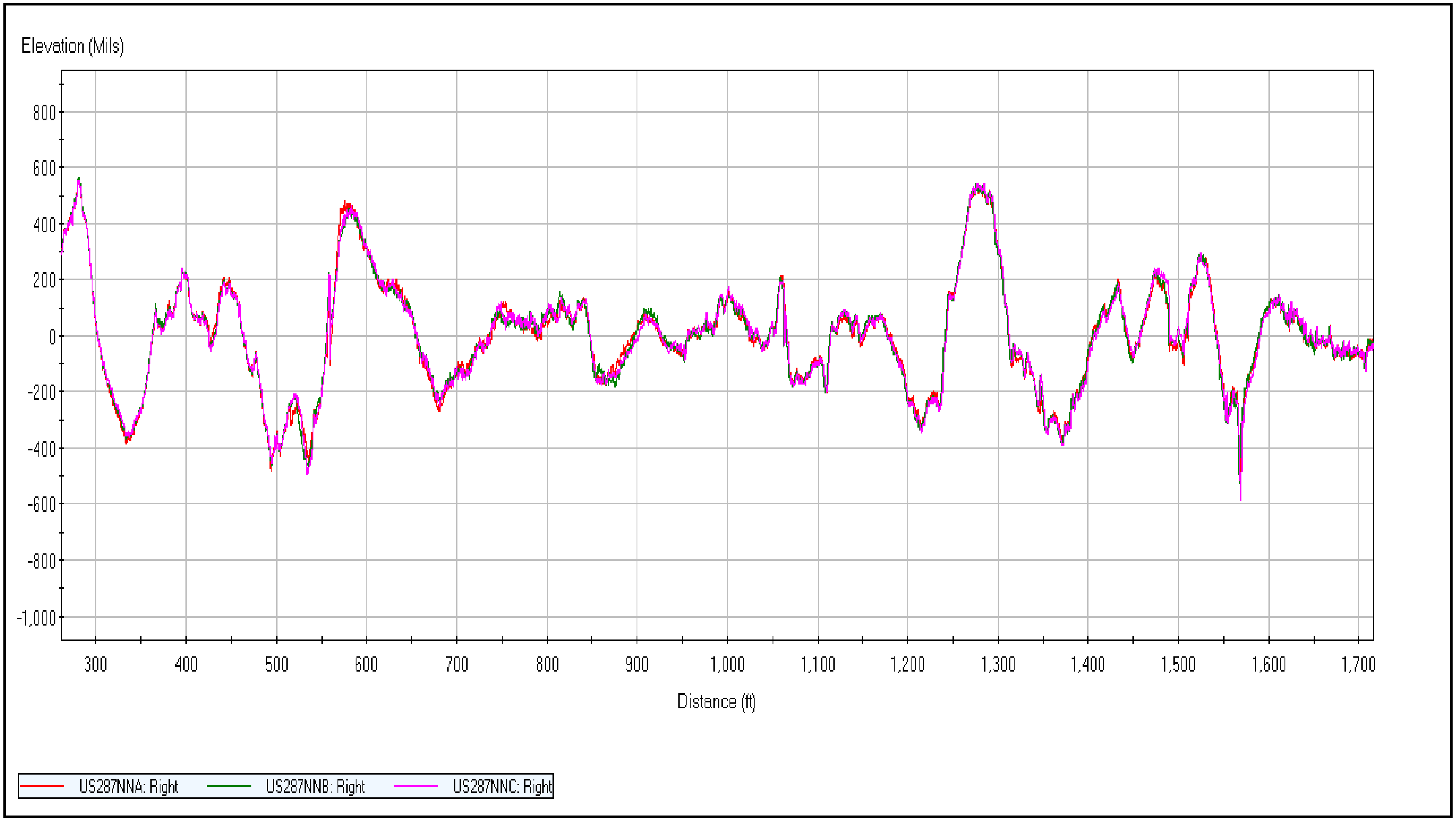


Figure B32. RWP Profiles on US287 Northbound Inside Lane of TxDOT WIM Site 506.

Table B1. WIM Smoothness Indices on Site 531 (I-35 Southbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	39.296	34.768	40.470	38.178
	Peak LRI	39.296	39.929	40.470	39.898
	SRI	21.035	20.168	17.898	19.700
	Peak SRI	27.141	34.282	43.853	35.092
Right	LRI	31.158	32.346	31.575	31.693
	Peak LRI	31.643	32.556	33.623	32.607
	SRI	27.141	26.410	19.907	24.486
	Peak SRI	28.520	29.163	23.514	27.066

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B2. WIM Smoothness Indices on Site 531 (I-35 Southbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	51.378	56.584	53.384	53.782
	Peak LRI	54.571	66.396	59.864	60.277
	SRI	31.801	27.266	20.796	26.621
	Peak SRI	49.675	52.288	49.309	50.424
Right	LRI	44.708	41.784	42.492	42.995
	Peak LRI	44.738	41.817	42.557	43.037
	SRI	42.125	45.98	44.24	44.115
	Peak SRI	55.515	57.698	52.979	55.397

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B3. WIM Smoothness Indices on Site 531 (I-35 Northbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	49.540	44.895	41.613	45.349
	Peak LRI	52.770	52.467	52.553	52.597
	SRI	43.727	37.637	33.276	38.213
	Peak SRI	51.591	40.485	45.766	45.947
Right	LRI	40.887	43.849	39.527	41.421
	Peak LRI	57.417	65.787	50.011	57.738
	SRI	37.913	32.030	27.900	32.614
	Peak SRI	72.060	55.404	48.622	58.695

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B4. WIM Smoothness Indices on Site 531 (I-35 Northbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	44.330	48.557	42.002	44.963
	Peak LRI	45.108	51.999	44.346	47.151
	SRI	29.601	37.575	31.924	33.033
	Peak SRI	48.065	43.479	38.303	43.282
Right	LRI	52.158	42.841	55.136	50.045
	Peak LRI	55.950	54.717	55.571	55.413
	SRI	27.290	30.605	49.242	35.712
	Peak SRI	54.770	45.330	54.345	51.482

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B5. TxDOT WIM Site 531 Classification Based on ASTM E1318 Type I Criteria.

Lane	Functional Item	Tolerance for 95% Conformance	P_{de} %[1]	Number of observations	Result
Southbound outside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	0.00	60	Passes Type I
	GVW	±10%	0.00	30	Passes Type I
Southbound inside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	1.67	60	Passes Type I
	GVW	±10%	0.00	30	Passes Type I
Northbound outside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	5.00	60	Passes Type I
	GVW	±10%	3.33	30	Passes Type I
Northbound inside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	0.00	60	Passes Type I
	GVW	±10%	0.00	30	Passes Type I

[1]P_{de} should not exceed 5%.
[2]Drive and trailer tandem axles were combined in determining the result shown.

Table B6. WIM Smoothness Indices on Site 530 (US277 Southbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	43.951	47.462	46.940	46.118
	Peak LRI	52.332	47.492	46.952	48.925
	SRI	28.363	25.010	30.835	28.069
	Peak SRI	32.663	35.037	42.497	36.732
Right	LRI	34.803	37.137	45.869	39.270
	Peak LRI	48.066	37.781	46.226	44.024
	SRI	25.362	22.747	49.406	32.505
	Peak SRI	31.940	33.824	49.735	38.500

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B7. WIM Smoothness Indices on Site 530 (US277 Southbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	44.587	52.370	50.148	49.035
	Peak LRI	53.178	67.349	50.327	56.951
	SRI	38.829	37.323	48.338	41.497
	Peak SRI	43.245	54.935	66.856	55.012
Right	LRI	52.580	56.000	59.971	56.184
	Peak LRI	53.922	56.788	60.814	57.175
	SRI	61.625	85.711	71.651	72.996
	Peak SRI	85.996	96.319	91.388	91.234

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B8. WIM Smoothness Indices on Site 530 (US277 Northbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	43.389	53.905	46.296	47.863
	Peak LRI	50.319	56.771	48.564	51.885
	SRI	33.569	45.452	44.828	41.283
	Peak SRI	45.969	64.706	53.334	54.670
Right	LRI	46.035	45.144	41.987	44.389
	Peak LRI	49.345	47.535	57.124	51.335
	SRI	33.438	36.990	41.307	37.245
	Peak SRI	74.195	49.671	65.164	63.010

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B9. WIM Smoothness Indices on Site 530 (US277 Northbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	50.772	48.137	50.405	49.771
	Peak LRI	50.772	57.092	51.809	53.224
	SRI	49.397	44.367	85.491	59.752
	Peak SRI	51.034	48.640	87.963	62.546
Right	LRI	50.102	65.288	60.658	58.683
	Peak LRI	50.102	65.981	61.185	59.089
	SRI	48.528	61.089	50.291	53.303
	Peak SRI	62.737	72.643	56.017	63.799

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B10. TxDOT WIM Site 530 Classification Based on ASTM E1318 Type I Criteria.

Lane	Functional Item	Tolerance for 95% Conformance	P_{de} %[1]	Number of observations	Result
Southbound outside lane	Steering axle weight	±20%	3.33	30	Passes Type I
	Tandem axle weight[2]	±15%	38.33	60	Fails Type I
	GVW	±10%	100.00	30	Fails Type I
Southbound inside lane	Steering axle weight	±20%	17.24	29	Fails Type I
	Tandem axle weight[2]	±15%	51.72	58	Fails Type I
	GVW	±10%	93.10	29	Fails Type I
Northbound outside lane	Steering axle weight	±20%	3.45	29	Passes Type I
	Tandem axle weight[2]	±15%	12.07	58	Fails Type I
	GVW	±10%	65.52	29	Fails Type I
Northbound inside lane[3]	Steering axle weight	±20%			
	Tandem axle weight[2]	±15%			
	GVW	±10%			

[1]P_{de} should not exceed 5%.
[2]Drive and trailer tandem axles were combined in determining the result shown.
[3]Lane found to be inoperative due to damaged cables.

Table B11. WIM Smoothness Indices on Site 528 (US287 Southbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	47.591	44.032	46.706	46.110
	Peak LRI	51.863	45.973	48.785	48.874
	SRI	21.349	37.235	51.587	36.724
	Peak SRI	42.518	48.866	54.372	48.585
Right	LRI	52.453	53.423	49.200	51.692
	Peak LRI	52.718	53.444	59.215	55.126
	SRI	28.943	45.635	46.810	40.463
	Peak SRI	42.518	67.977	68.678	59.724

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B12. WIM Smoothness Indices on Site 528 (US287 Southbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	52.569	53.305	55.173	53.682
	Peak LRI	56.101	58.108	56.233	56.814
	SRI	52.582	46.176	26.345	41.701
	Peak SRI	55.120	50.258	52.562	52.647
Right	LRI	61.876	56.627	54.560	57.688
	Peak LRI	62.964	73.241	82.281	72.829
	SRI	55.817	56.333	69.862	60.671
	Peak SRI	68.870	62.350	90.092	73.771

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B13. WIM Smoothness Indices on Site 528 (US287 Northbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	39.578	46.851	43.733	43.387
	Peak LRI	41.944	48.493	44.090	44.842
	SRI	34.551	44.410	22.128	33.696
	Peak SRI	43.704	53.535	38.093	45.111
Right	LRI	47.734	56.464	48.131	50.776
	Peak LRI	50.943	56.464	48.131	51.846
	SRI	33.554	96.035	44.610	58.066
	Peak SRI	42.311	110.388	48.337	67.012

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B14. WIM Smoothness Indices on Site 528 (US287 Northbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	41.657	54.455	43.435	46.516
	Peak LRI	45.664	56.256	48.271	50.064
	SRI	44.936	34.653	46.180	41.923
	Peak SRI	45.927	49.031	54.239	49.732
Right	LRI	48.292	56.583	49.897	51.591
	Peak LRI	51.023	57.622	51.586	53.410
	SRI	51.576	49.600	30.984	44.053
	Peak SRI	65.985	51.532	48.845	55.454

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B15. TxDOT WIM Site 528 Classification Based on ASTM E1318 Type I Criteria.

Lane	Functional Item	Tolerance for 95% Conformance	P_{de} %[1]	Number of observations	Result
Southbound outside lane	Steering axle weight	±20%	0.00	13	Passes Type I
	Tandem axle weight[2]	±15%	0.00	26	Passes Type I
	GVW	±10%	0.00	13	Passes Type I
Southbound inside lane	Steering axle weight	±20%	0.00	15	Passes Type I
	Tandem axle weight[2]	±15%	0.00	30	Passes Type I
	GVW	±10%	0.00	15	Passes Type I
Northbound outside lane	Steering axle weight	±20%	0.00	15	Passes Type I
	Tandem axle weight[2]	±15%	6.67	30	Fails Type I
	GVW	±10%	6.67	15	Fails Type I
Northbound inside lane	Steering axle weight	±20%	0.00	15	Passes Type I
	Tandem axle weight[2]	±15%	0.00	30	Passes Type I
	GVW	±10%	0.00	15	Passes Type I

[1]P_{de} should not exceed 5%.
[2]Drive and trailer tandem axles were combined in determining the result shown.

Table B16. WIM Smoothness Indices on Site 506 (US287 Southbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	43.738	37.615	41.764	41.039
	Peak LRI	43.738	38.627	48.651	43.672
	SRI	59.643	25.593	27.799	37.678
	Peak SRI	69.388	38.923	40.251	49.521
Right	LRI	42.530	42.856	41.688	42.358
	Peak LRI	42.557	42.862	41.778	42.399
	SRI	52.274	50.782	42.743	48.600
	Peak SRI	74.599	71.245	67.430	71.091

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B17. WIM Smoothness Indices on Site 506 (US287 Southbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	33.057	35.524	40.509	36.363
	Peak LRI	40.956	43.132	44.890	42.993
	SRI	21.967	21.901	22.325	22.064
	Peak SRI	35.668	44.985	44.753	41.802
Right	LRI	36.886	47.982	45.263	43.377
	Peak LRI	39.437	48.131	47.164	44.911
	SRI	39.391	38.314	35.205	37.637
	Peak SRI	50.531	50.587	64.437	55.185

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B18. WIM Smoothness Indices on Site 506 (US287 Northbound Outside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	37.788	38.136	40.698	38.874
	Peak LRI	37.788	40.963	41.200	39.984
	SRI	48.152	34.585	38.178	40.305
	Peak SRI	49.988	48.338	43.147	47.158
Right	LRI	35.321	43.716	37.758	38.932
	Peak LRI	39.579	44.985	42.610	42.391
	SRI	44.677	71.704	42.431	52.937
	Peak SRI	48.727	72.617	54.982	58.775

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B19. WIM Smoothness Indices on Site 506 (US287 Northbound Inside Lane)[1].

Wheel Path	Index (inches/mile)	Run 1	Run 2	Run 3	Average
Left	LRI	26.662	43.730	31.643	34.012
	Peak LRI	50.066	43.730	40.891	44.896
	SRI	18.927	30.658	23.405	24.330
	Peak SRI	25.221	50.834	30.017	35.357
Right	LRI	30.141	37.030	34.819	33.997
	Peak LRI	53.602	56.073	53.375	54.350
	SRI	30.770	44.613	56.547	43.977
	Peak SRI	42.858	48.945	63.720	51.841

[1]Shaded cells show indices between lower and upper thresholds of WIM smoothness criteria.

Table B20. TxDOT WIM Site 506 Classification Based on ASTM E1318 Type I Criteria.

Lane	Functional Item	Tolerance for 95% Conformance	P_{de} %[1]	Number of observations	Result
Southbound outside lane	Steering axle weight	±20%	0.00	30	Passes Type I
	Tandem axle weight[2]	±15%	0.00	60	Passes Type I
	GVW	±10%	0.00	30	Passes Type I
Southbound inside lane	Steering axle weight	±20%	0.00	15	Passes Type I
	Tandem axle weight[2]	±15%	1.67	60	Passes Type I
	GVW	±10%	0.00	15	Passes Type I
Northbound outside lane	Steering axle weight	±20%	0.00	15	Passes Type I
	Tandem axle weight[2]	±15%	5.00	60	Passes Type I
	GVW	±10%	6.67	15	Fails Type I
Northbound inside lane	Steering axle weight	±20%	0.00	15	Passes Type I
	Tandem axle weight[2]	±15%	0.00	60	Passes Type I
	GVW	±10%	0.00	15	Passes Type I

[1]P_{de} should not exceed 5%.
[2]Drive and trailer tandem axles were combined in determining the result shown.

APPENDIX C

VISUAL SURVEY DATA COLLECTED ON I-35 PERPETUAL PAVEMENT WIM SECTIONS

Table C1. Summary of Distress Survey on WIM Site 531 along I-35 (July 20, 2007).

Lane	PMIS* Distress Score	LTPP Distress Score
Northbound outside, R1 (CRCP)	100	62 (low severity transverse cracks). 8-ft average crack spacing
Northbound inside, R2 (CRCP)	100	63 (low severity transverse cracks). 8-ft average crack spacing
Southbound outside, L1 (HMAC)	100	
Southbound inside, L2 (HMAC)	100	

* Pavement management information system distress score of 100 indicates no distresses.

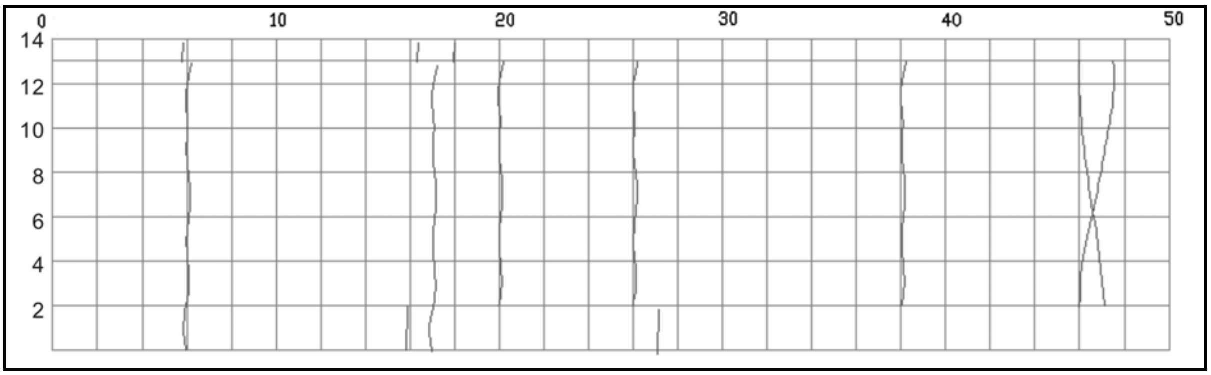


Figure C1. Visual Survey Chart for I-35 R1 Lane (0 to 50 ft, 07/20/2007 survey).

Note: On the survey charts for the CRCP sections, the edge stripe is at 1 ft on the vertical axis, the edge joint at 2 ft, centerline stripe at 13 ft, and the centerline joint at 13 ft.

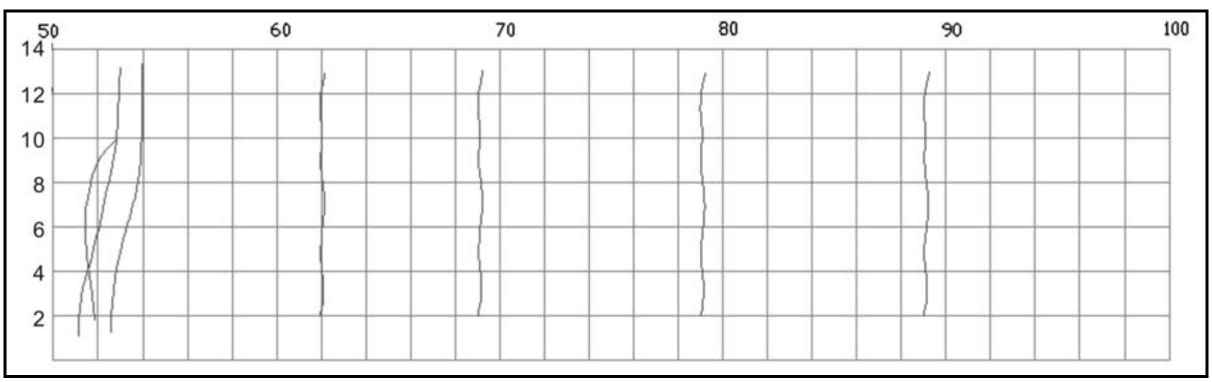


Figure C2. Visual Survey Chart for I-35 R1 Lane (50 to 100 ft, 07/20/2007 survey).

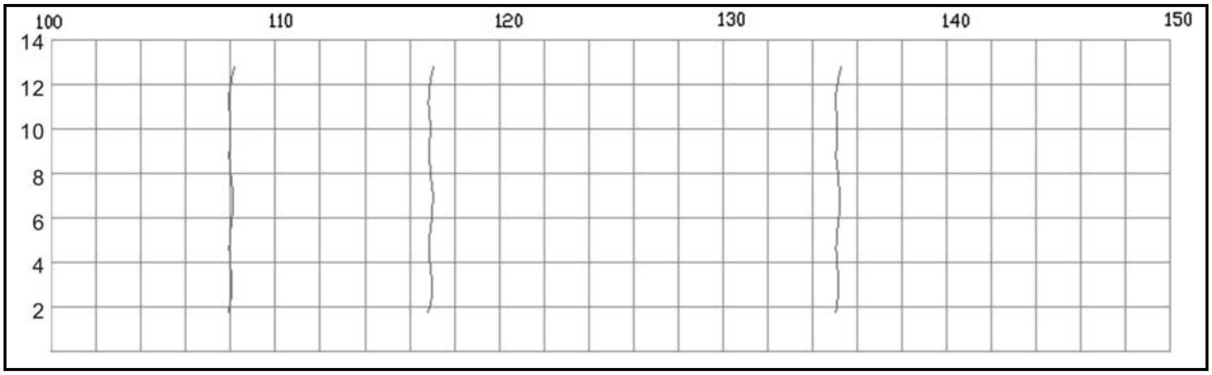


Figure C3. Visual Survey Chart for I-35 R1 Lane (100 to 150 ft, 07/20/2007 survey).

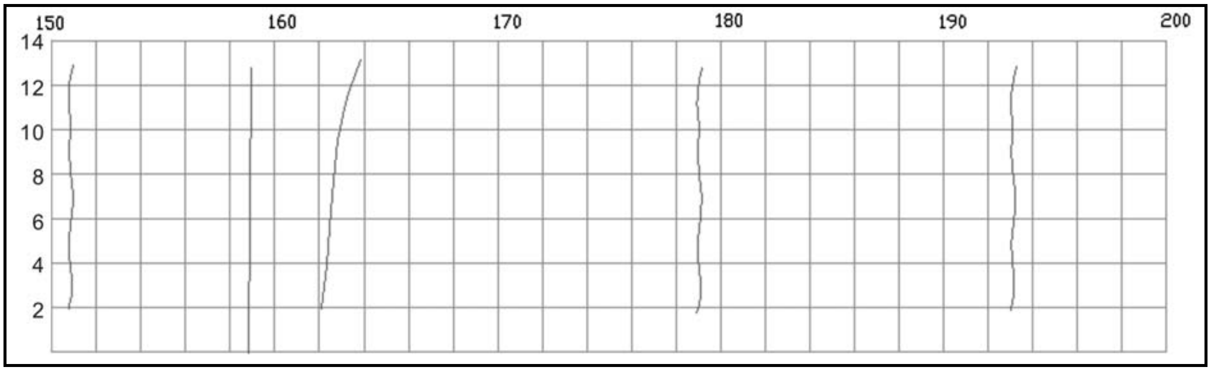


Figure C4. Visual Survey Chart for I-35 R1 Lane (150 to 200 ft, 07/20/2007 survey).

Figure C5. Visual Survey Chart for I-35 R1 Lane (200 to 250 ft, 07/20/2007 survey).

Figure C6. Visual Survey Chart for I-35 R1 Lane (250 to 300 ft, 07/20/2007 survey).

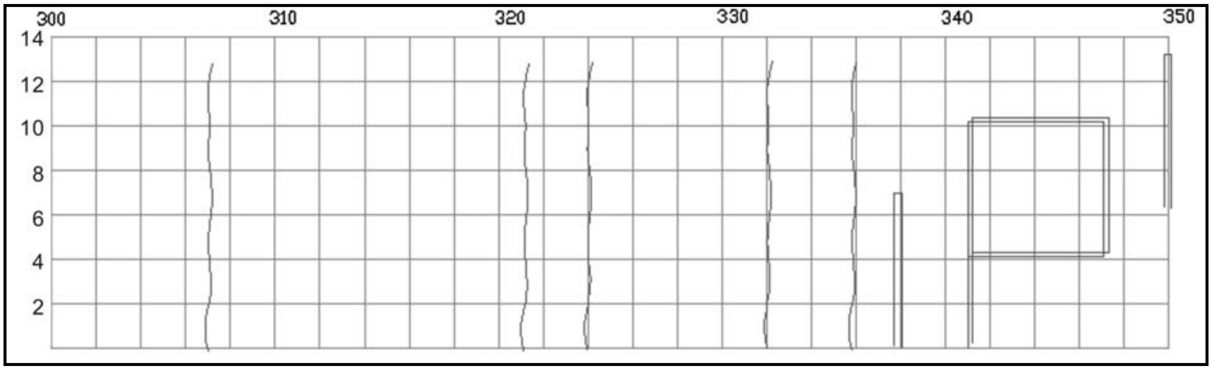


Figure C7. Visual Survey Chart for I-35 R1 Lane (300 to 350 ft, 07/20/2007 survey).

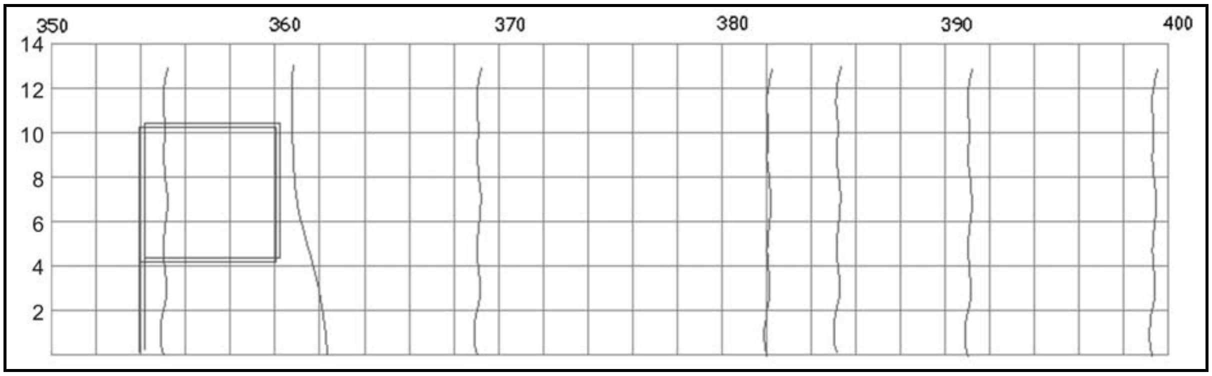


Figure C8. Visual Survey Chart for I-35 R1 Lane (350 to 400 ft, 07/20/2007 survey).

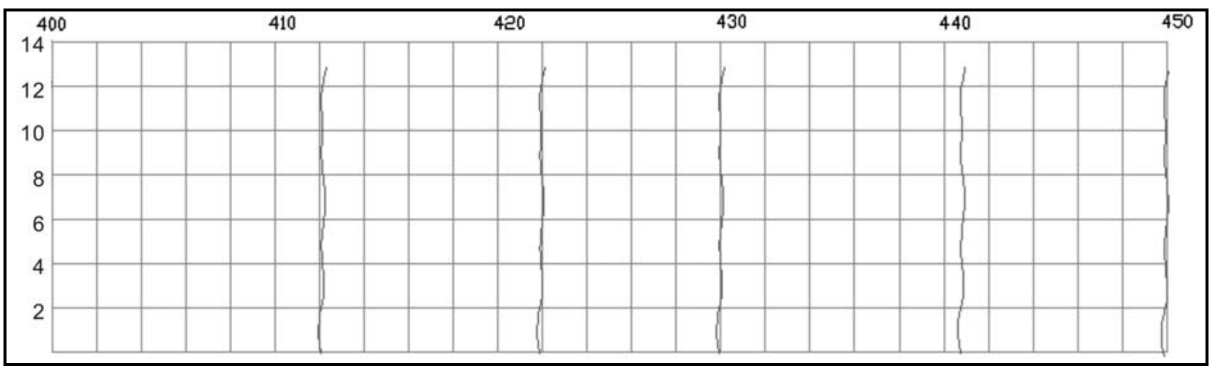


Figure C9. Visual Survey Chart for I-35 R1 Lane (400 to 450 ft, 07/20/2007 survey).

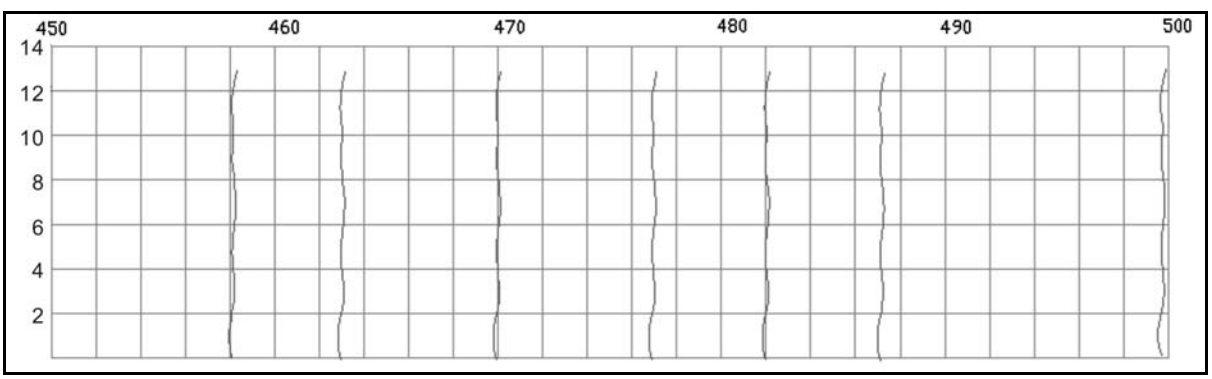


Figure C10. Visual Survey Chart for I-35 R1 Lane (450 to 500 ft, 07/20/2007 survey).

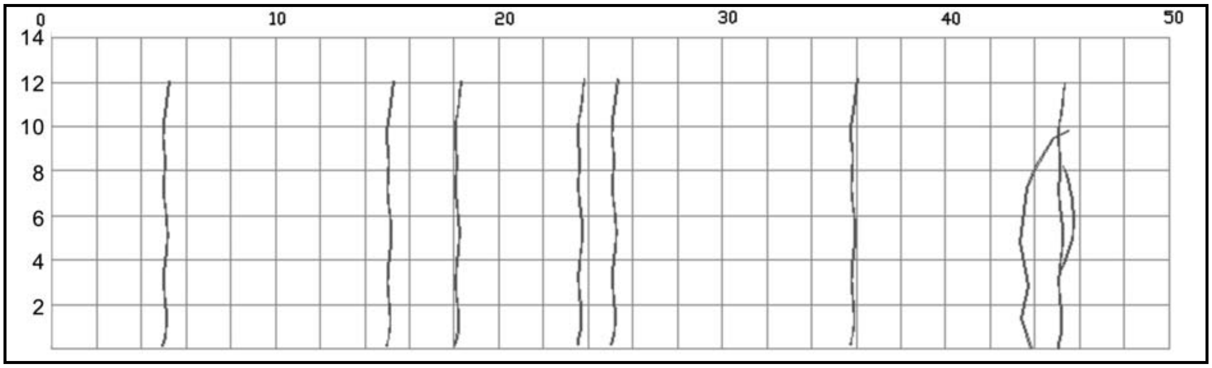


Figure C11. Visual Survey Chart for I-35 R2 Lane (0 to 50 ft, 07/20/2007 survey).

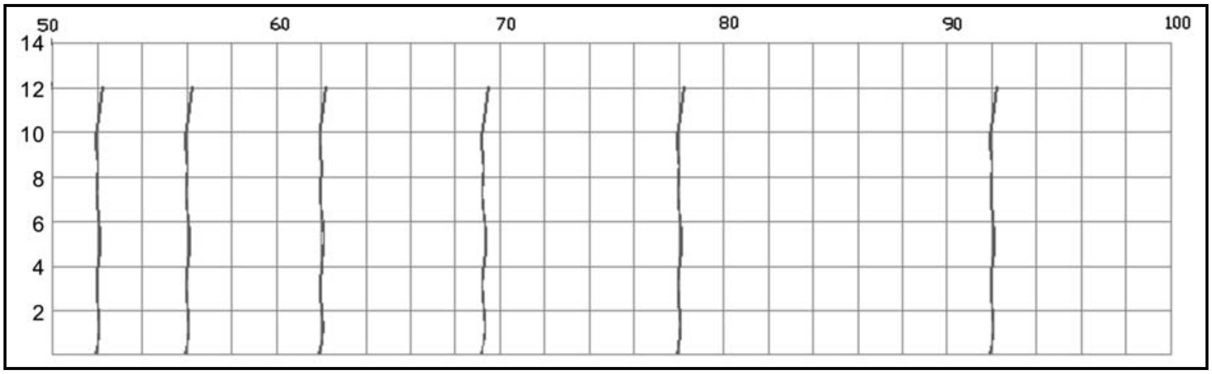


Figure C12. Visual Survey Chart for I-35 R2 Lane (50 to 100 ft, 07/20/2007 survey).

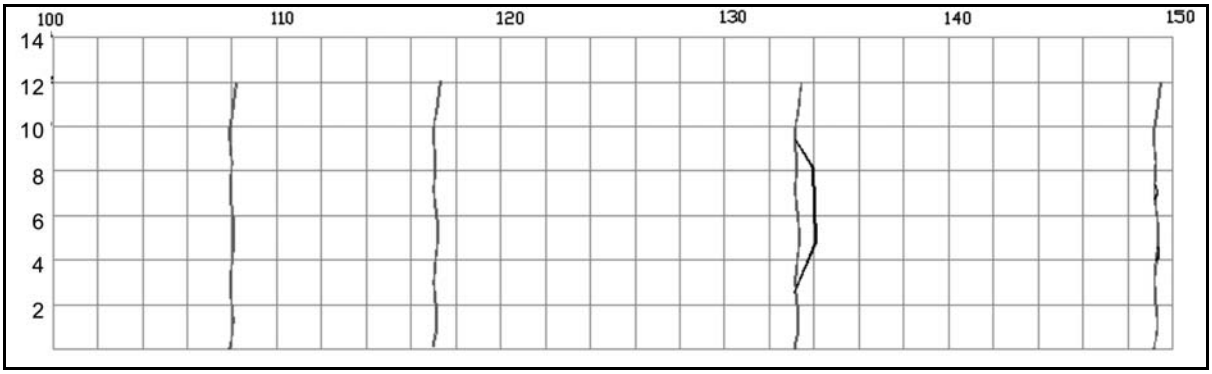


Figure C13. Visual Survey Chart for I-35 R2 Lane (100 to 150 ft, 07/20/2007 survey).

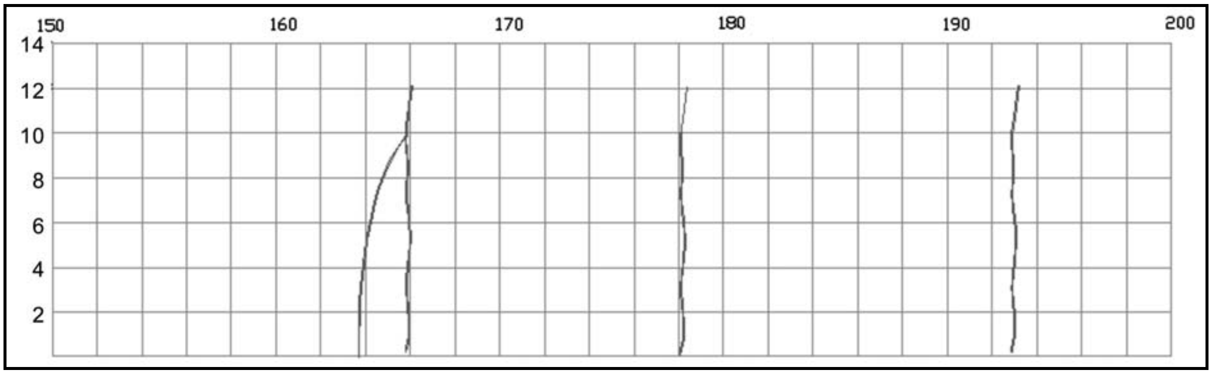


Figure C14. Visual Survey Chart for I-35 R2 Lane (150 to 200 ft, 07/20/2007 survey).

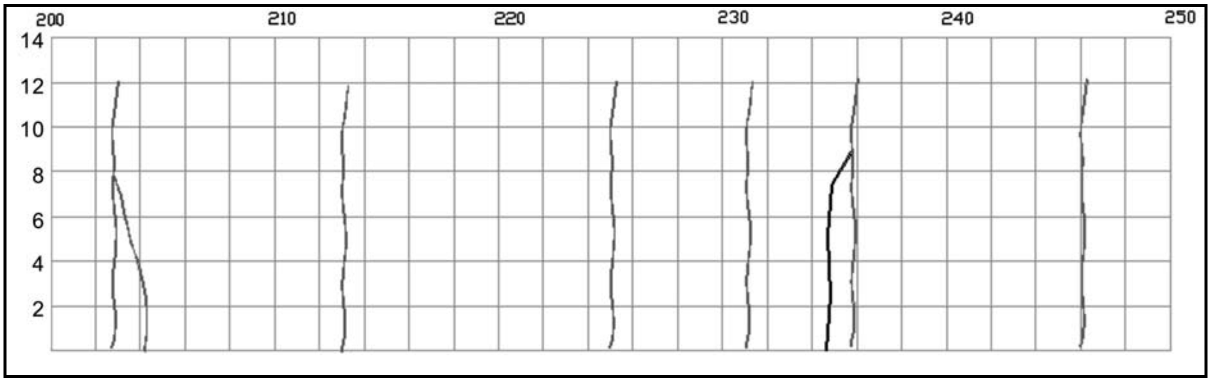


Figure C15. Visual Survey Chart for I-35 R2 Lane (200 to 250 ft, 07/20/2007 survey).

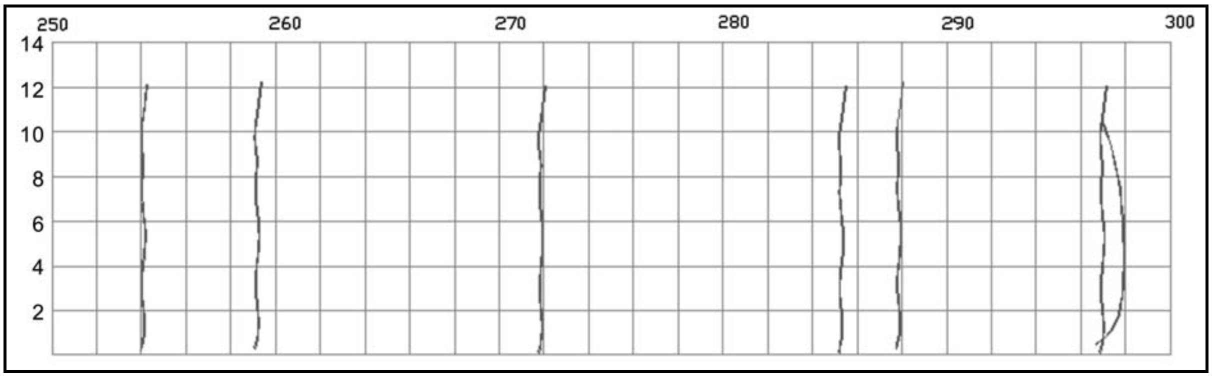


Figure C16. Visual Survey Chart for I-35 R2 Lane (250 to 300 ft, 07/20/2007 survey).

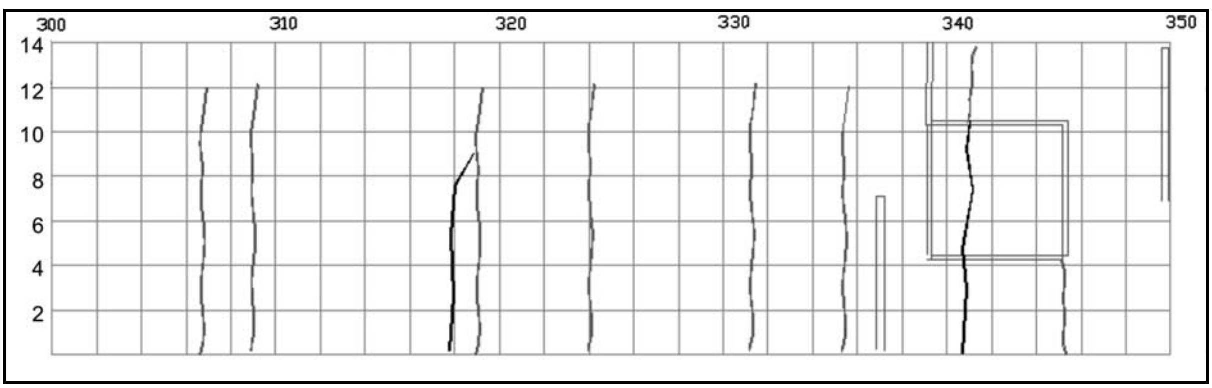


Figure C17. Visual Survey Chart for I-35 R2 Lane (300 to 350 ft, 07/20/2007 survey).

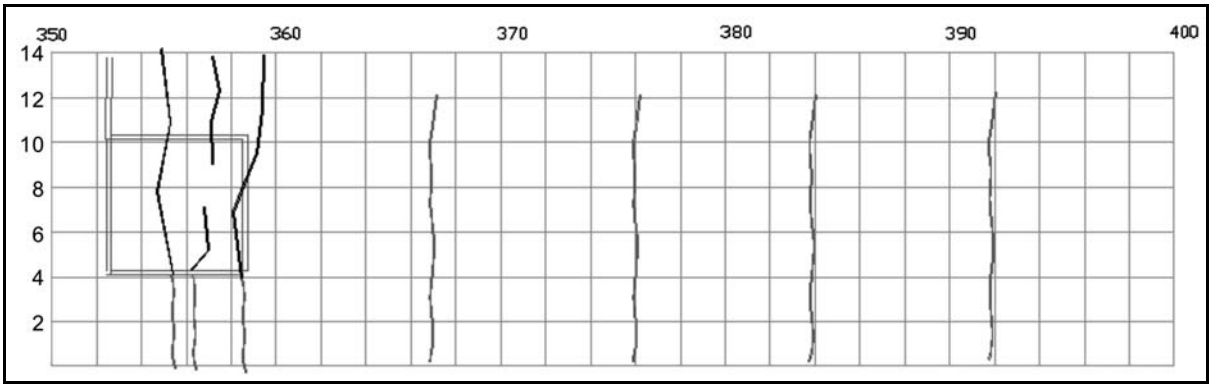


Figure C18. Visual Survey Chart for I-35 R2 Lane (350 to 400 ft, 07/20/2007 survey).

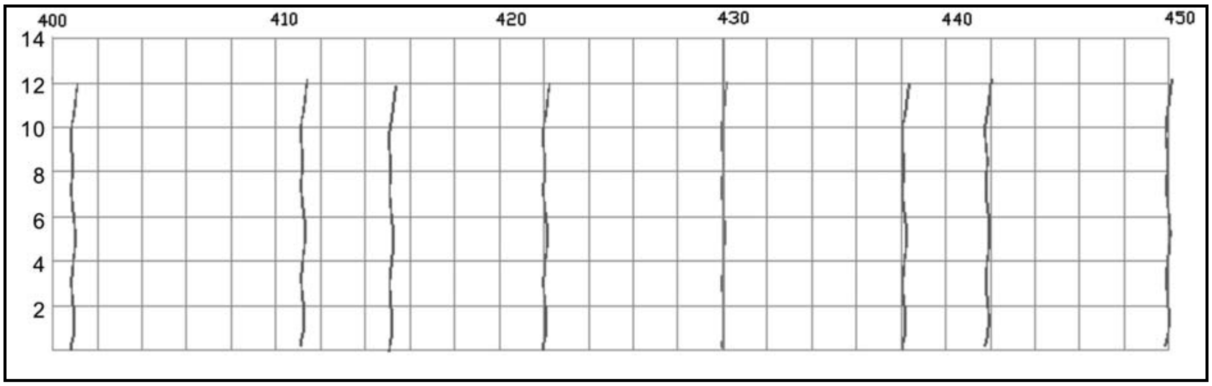


Figure C19. Visual Survey Chart for I-35 R2 Lane (400 to 450 ft, 07/20/2007 survey).

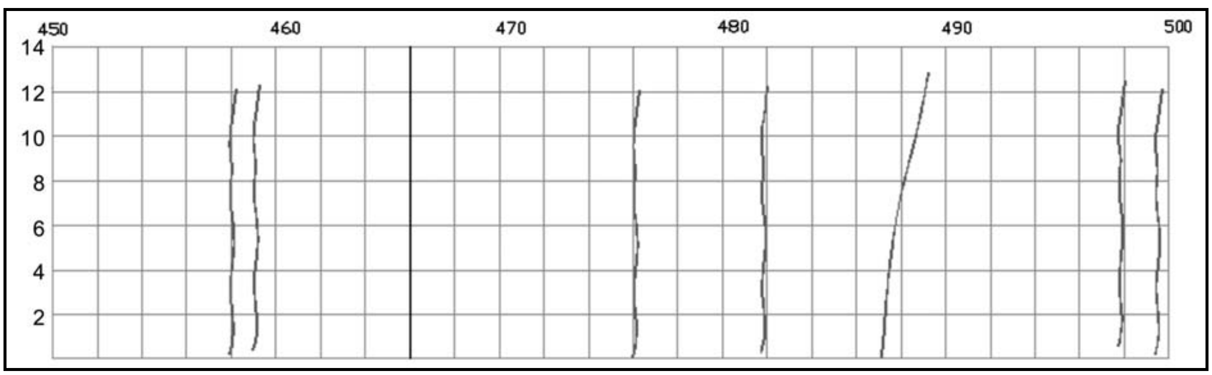


Figure C20. Visual Survey Chart for I-35 R2 Lane (450 to 500 ft, 07/20/2007 survey).

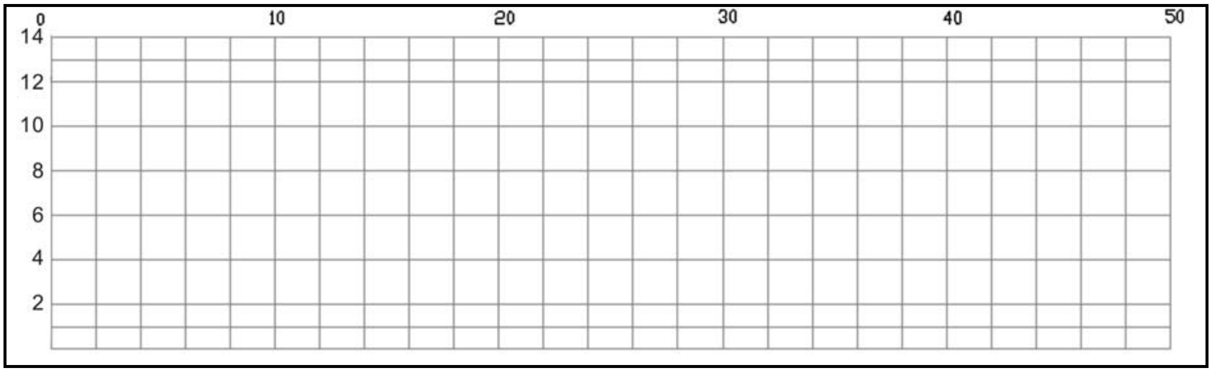


Figure C21. Visual Survey Chart for I-35 L1 Lane (0 to 50 ft, 07/20/2007 survey).

Note: On the survey charts for the HMAC sections, the edge stripe is at 1 ft on the vertical axis, and the centerline at 13 ft.

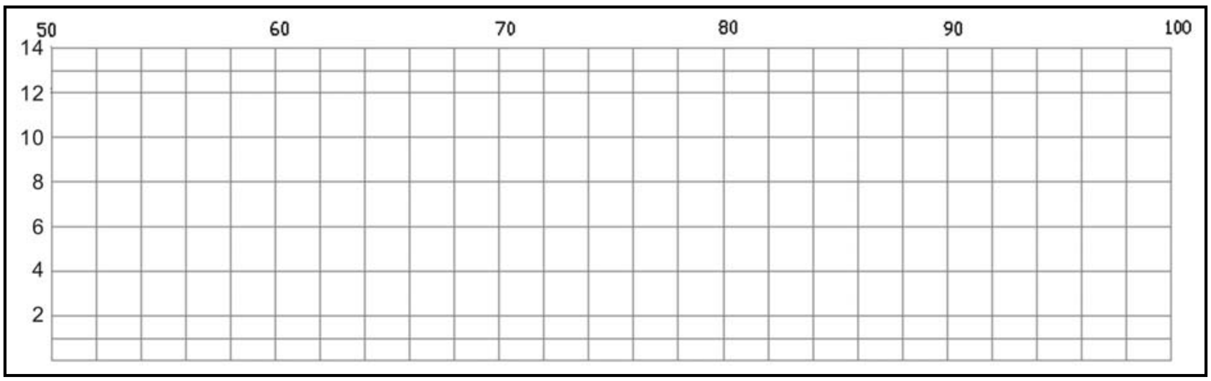


Figure C22. Visual Survey Chart for I-35 L1 Lane (50 to 100 ft, 07/20/2007 survey).

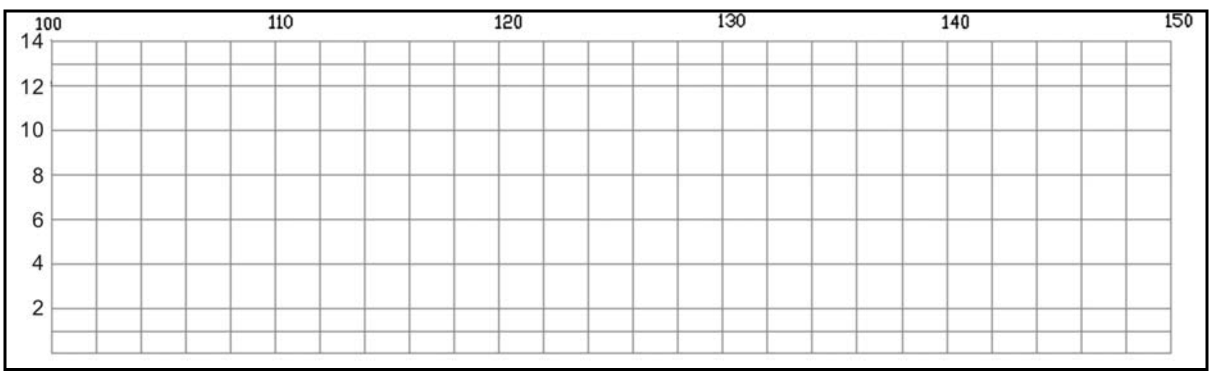


Figure C23. Visual Survey Chart for I-35 L1 Lane (100 to 150 ft, 07/20/2007 survey).

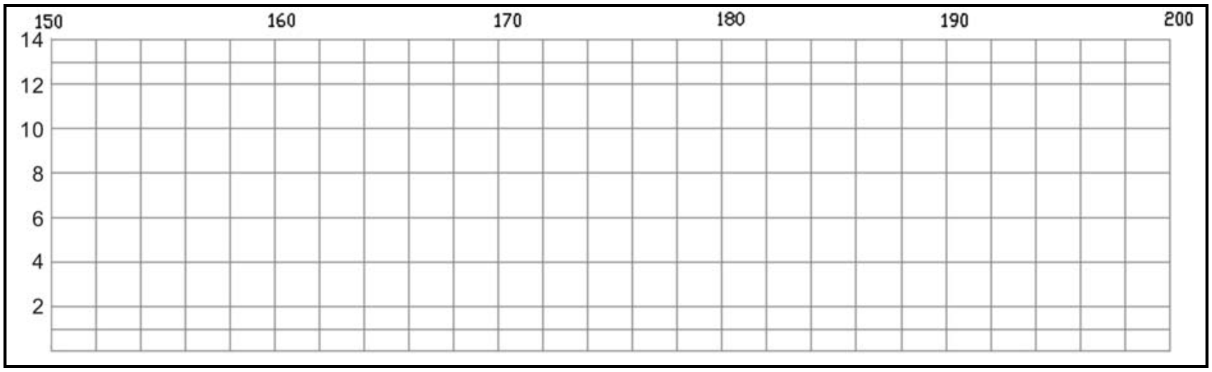


Figure C24. Visual Survey Chart for I-35 L1 Lane (150 to 200 ft, 07/20/2007 survey).

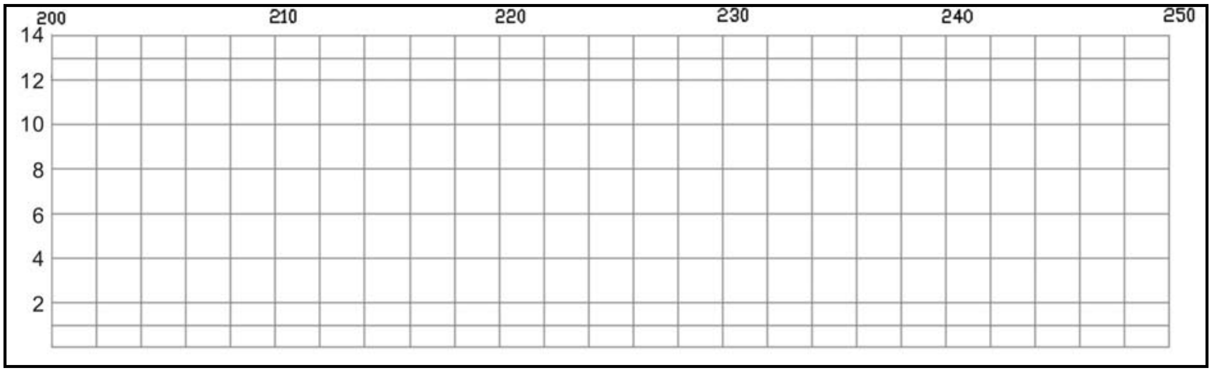


Figure C25. Visual Survey Chart for I-35 L1 Lane (200 to 250 ft, 07/20/2007 survey).

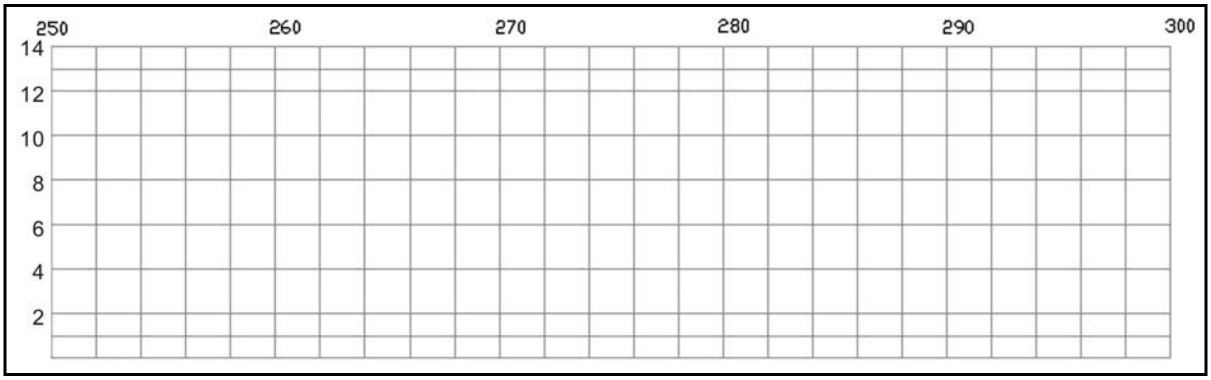


Figure C26. Visual Survey Chart for I-35 L1 Lane (250 to 300 ft, 07/20/2007 survey).

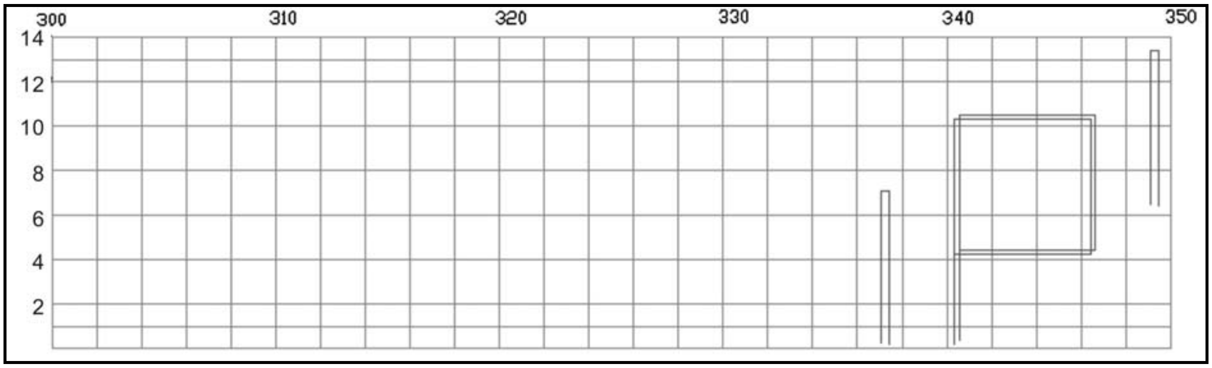


Figure C27. Visual Survey Chart for I-35 L1 Lane (300 to 350 ft, 07/20/2007 survey).

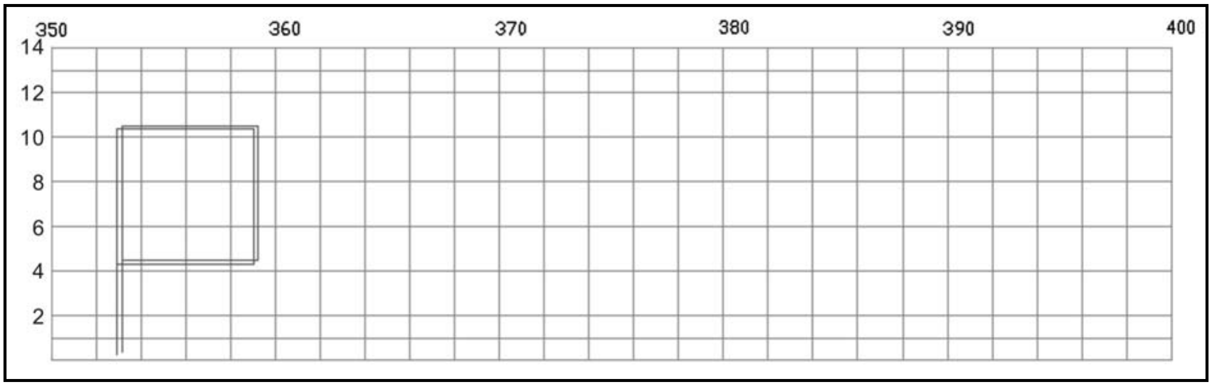


Figure C28. Visual Survey Chart for I-35 L1 Lane (350 to 400 ft, 07/20/2007 survey).

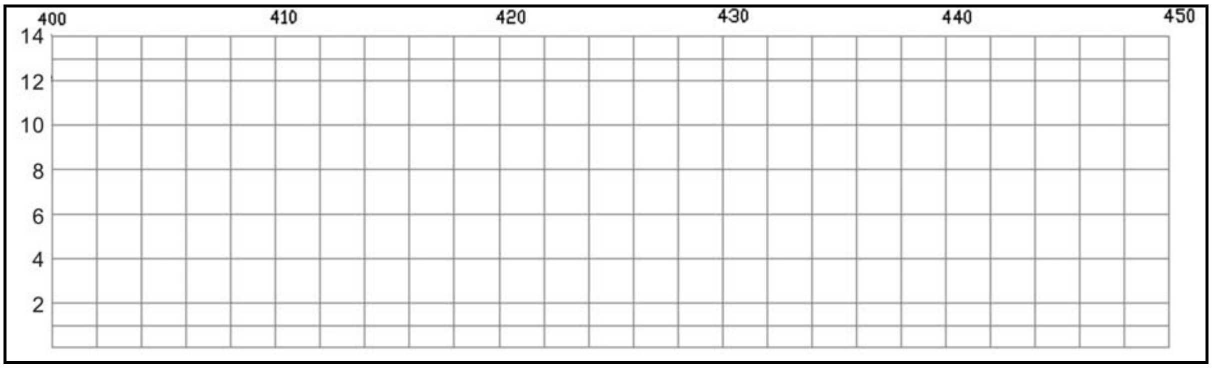


Figure C29. Visual Survey Chart for I-35 L1 Lane (400 to 450 ft, 07/20/2007 survey).

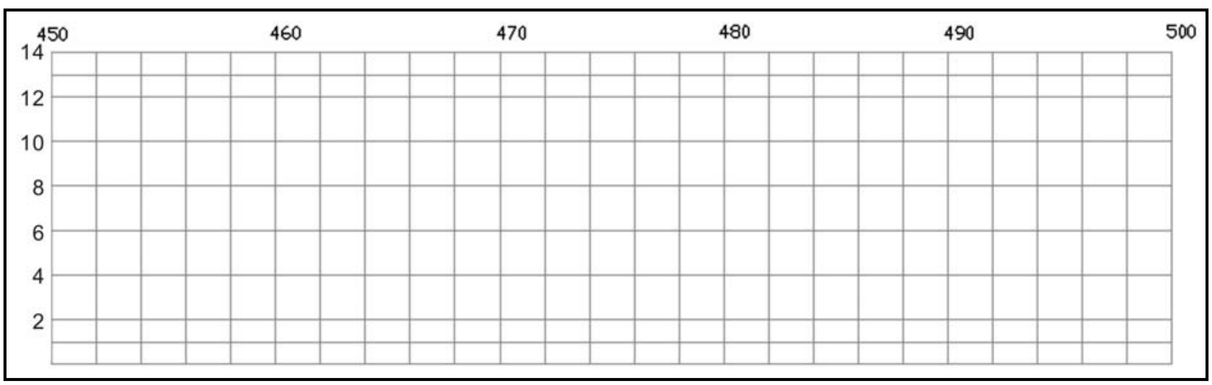


Figure C30. Visual Survey Chart for I-35 L1 Lane (450 to 500 ft, 07/20/2007 survey).

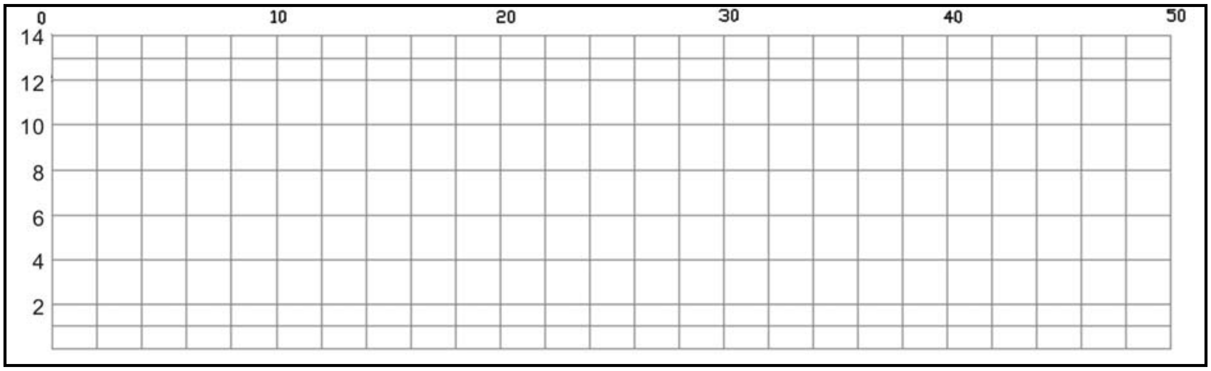


Figure C31. Visual Survey Chart for I-35 L2 Lane (0 to 50 ft, 07/20/2007 survey).

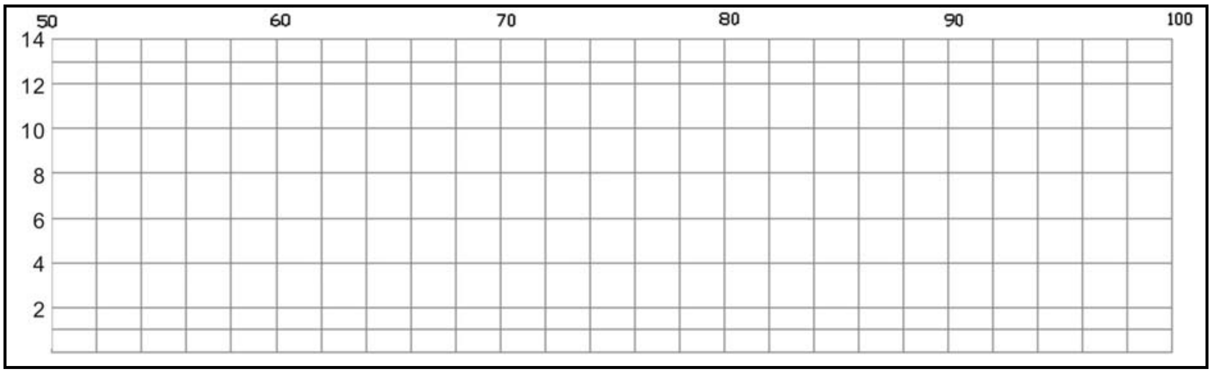


Figure C32. Visual Survey Chart for I-35 L2 Lane (50 to 100 ft, 07/20/2007 survey).

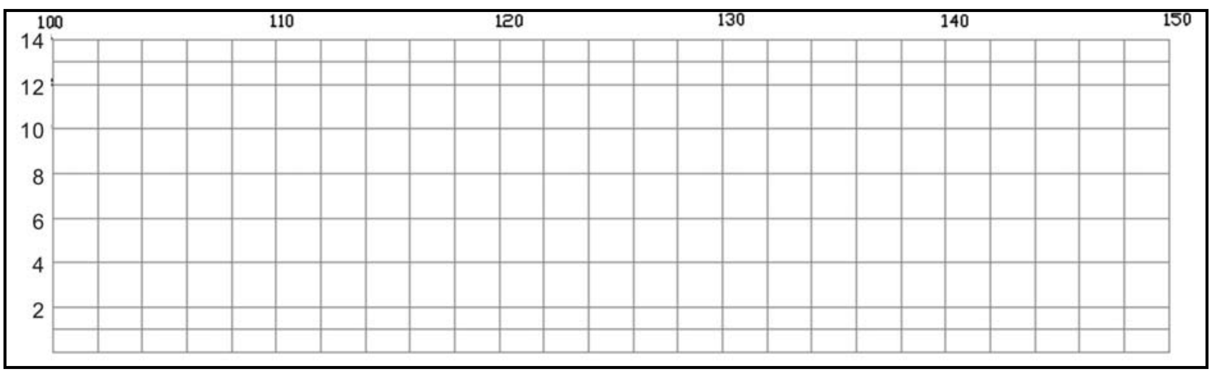


Figure C33. Visual Survey Chart for I-35 L2 Lane (100 to 150 ft, 07/20/2007 survey).

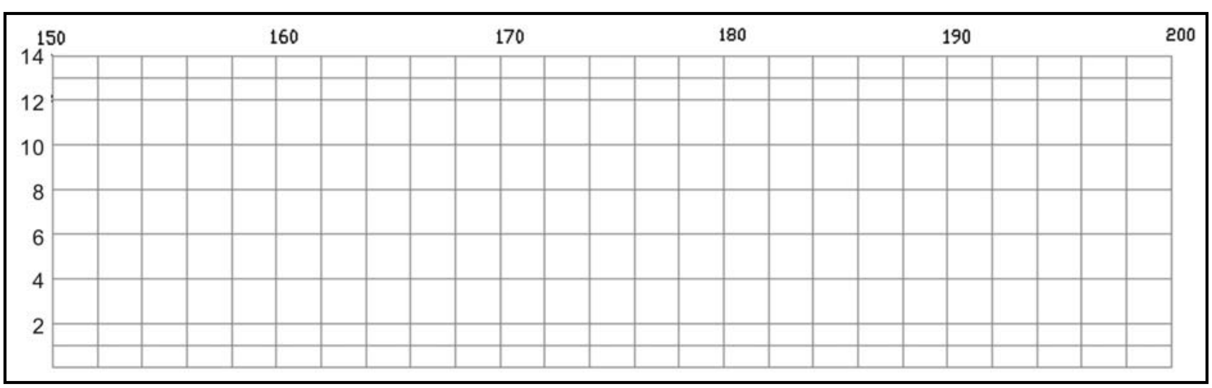


Figure C34. Visual Survey Chart for I-35 L2 Lane (150 to 200 ft, 07/20/2007 survey).

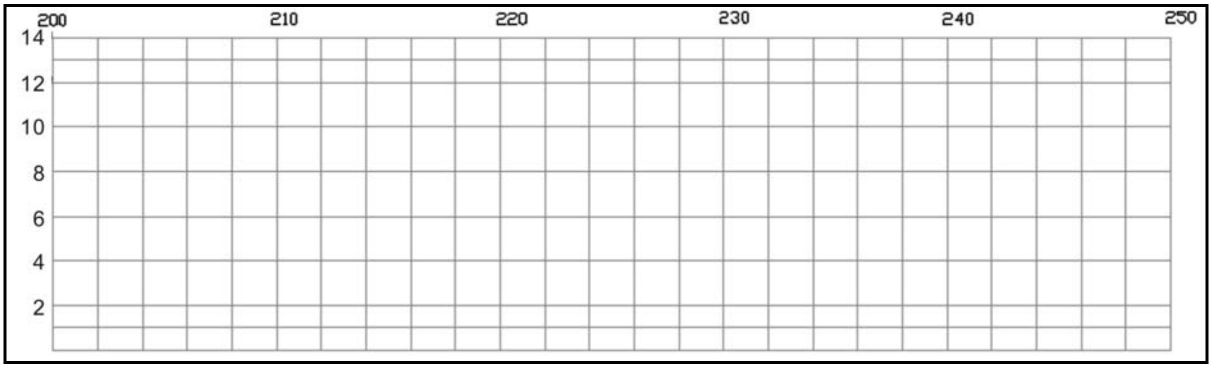


Figure C35. Visual Survey Chart for I-35 L2 Lane (200 to 250 ft, 07/20/2007 survey).

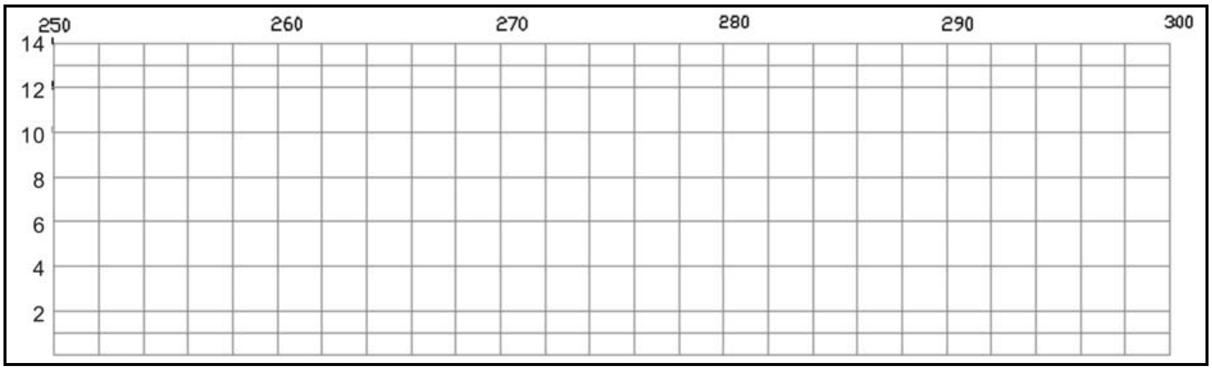


Figure C36. Visual Survey Chart for I-35 L2 Lane (250 to 300 ft, 07/20/2007 survey).

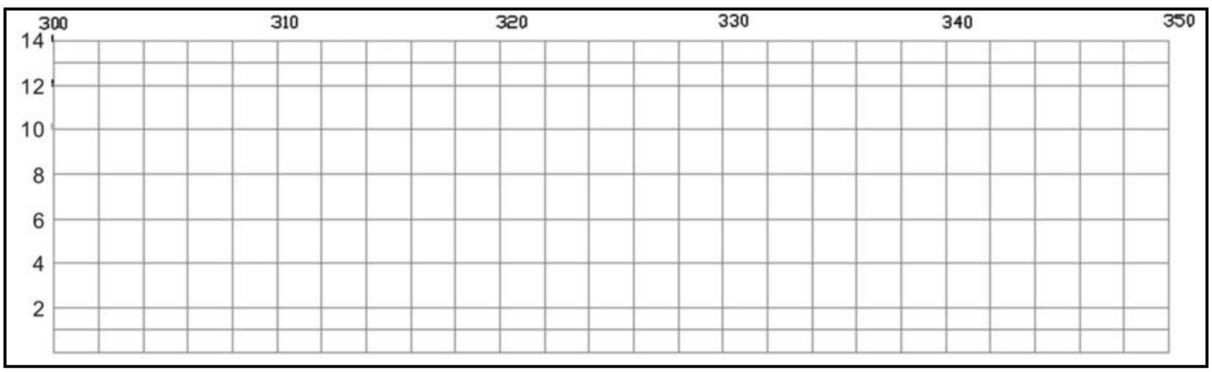


Figure C37. Visual Survey Chart for I-35 L2 Lane (300 to 350 ft, 07/20/2007 survey).

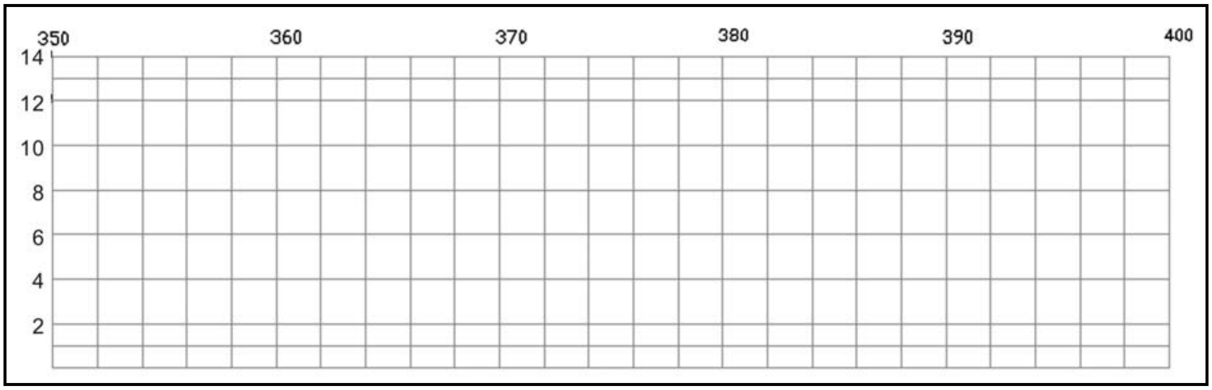


Figure C38. Visual Survey Chart for I-35 L2 Lane (350 to 400 ft, 07/20/2007 survey).

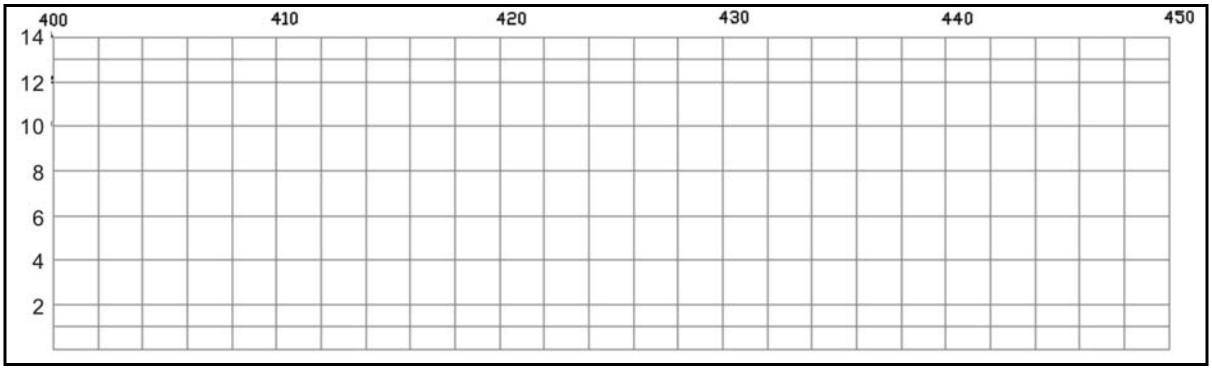


Figure C39. Visual Survey Chart for I-35 L2 Lane (400 to 450 ft, 07/20/2007 survey).

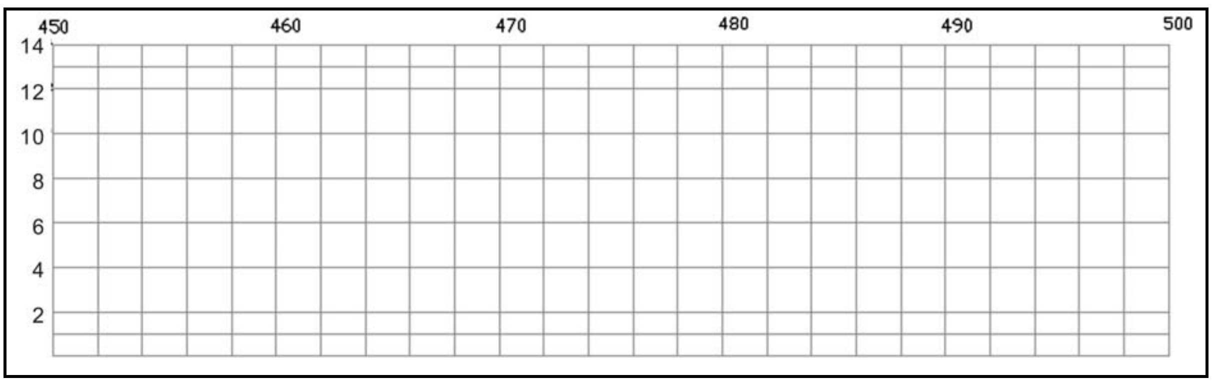


Figure C40. Visual Survey Chart for I-35 L2 Lane (450 to 500 ft, 07/20/2007 survey).

Table C2. Summary of Distress Survey on WIM Site 531 along I-35 (July 15, 2008).

Lane	PMIS Distress Score	LTPP Distress Score
Northbound outside, R1 (CRCP)	100	62 (low severity transverse cracks). 8-ft average crack spacing
Northbound inside, R2 (CRCP)	100	67 (low severity transverse cracks). 7-ft average crack spacing
Southbound outside, L1 (HMAC)	100	
Southbound inside, L2 (HMAC)	100	

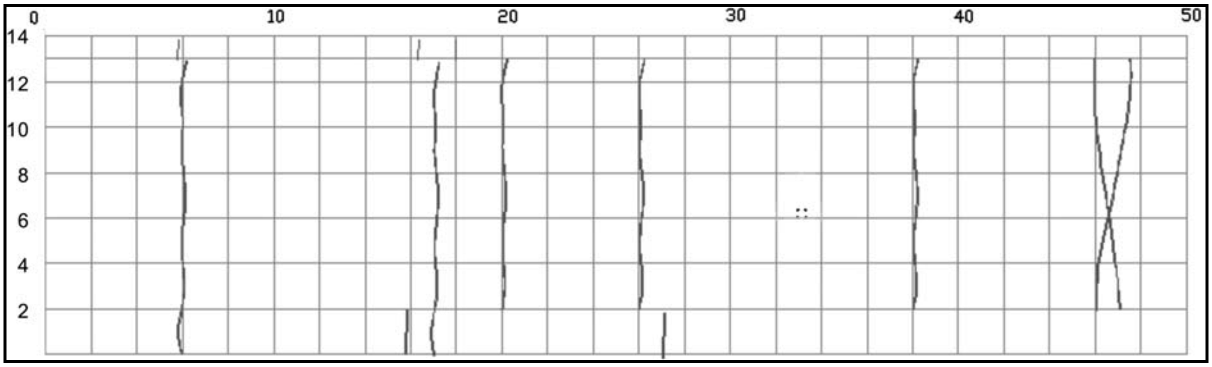


Figure C41. Visual Survey Chart for I-35 R1 Lane (0 to 50 ft, 07/15/2008 survey).

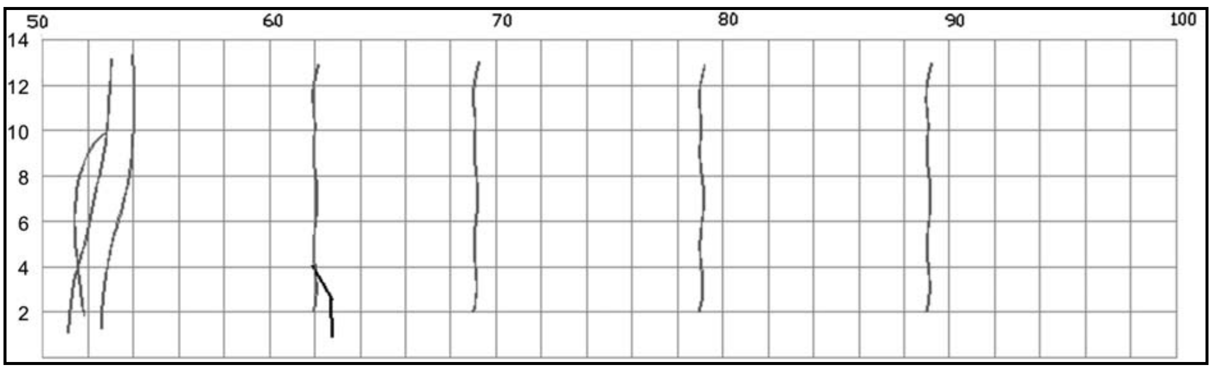


Figure C42. Visual Survey Chart for I-35 R1 Lane (50 to 100 ft, 07/15/2008 survey).

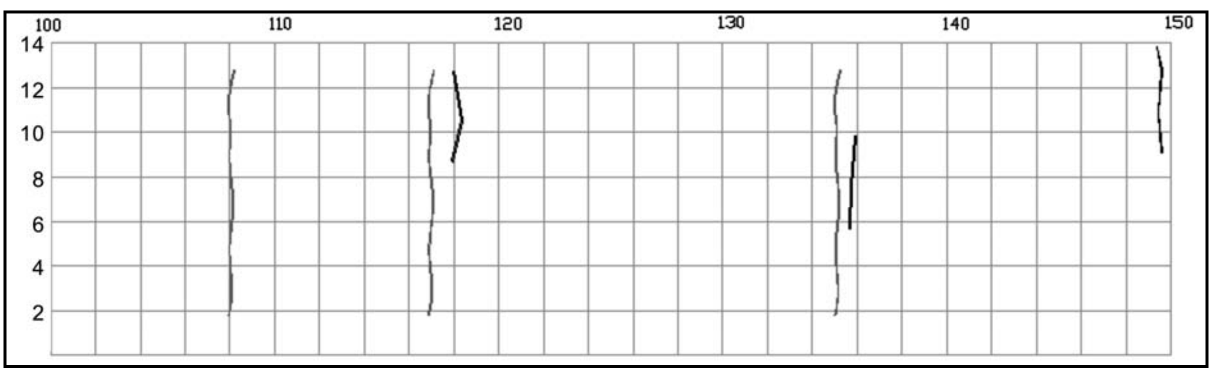


Figure C43. Visual Survey Chart for I-35 R1 Lane (100 to 150 ft, 07/15/2008 survey).

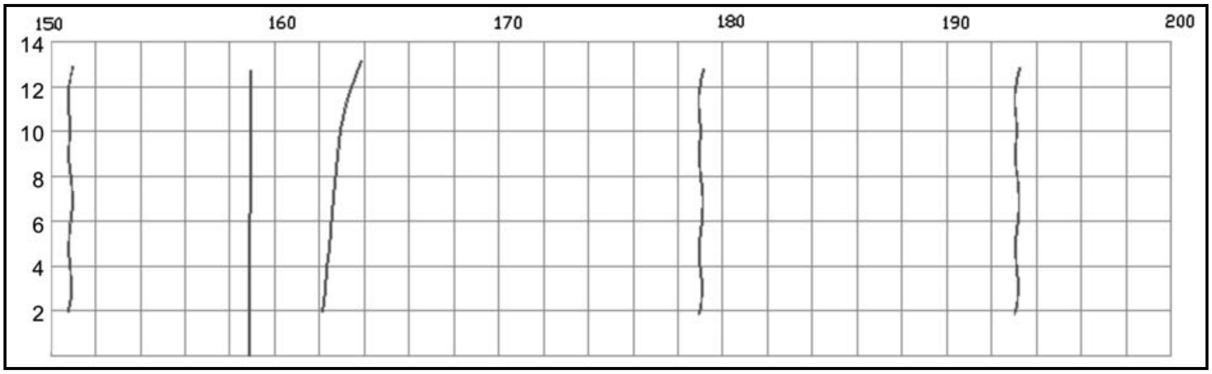


Figure C44. Visual Survey Chart for I-35 R1 Lane (150 to 200 ft, 07/15/2008 survey).

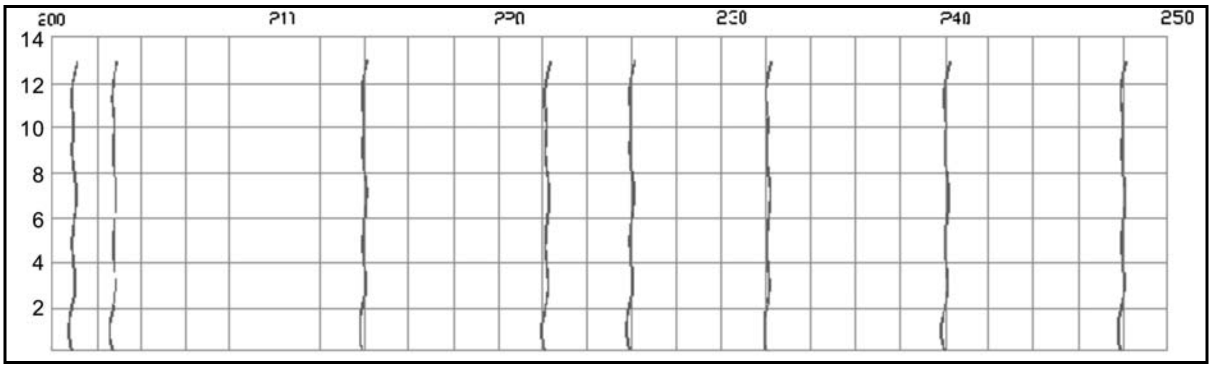


Figure C45. Visual Survey Chart for I-35 R1 Lane (200 to 250 ft, 07/15/2008 survey).

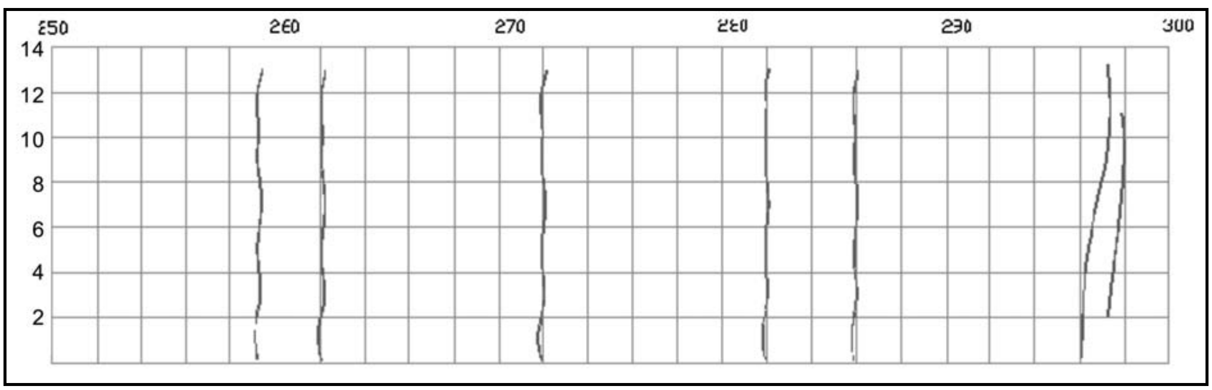


Figure C46. Visual Survey Chart for I-35 R1 Lane (250 to 300 ft, 07/15/2008 survey).

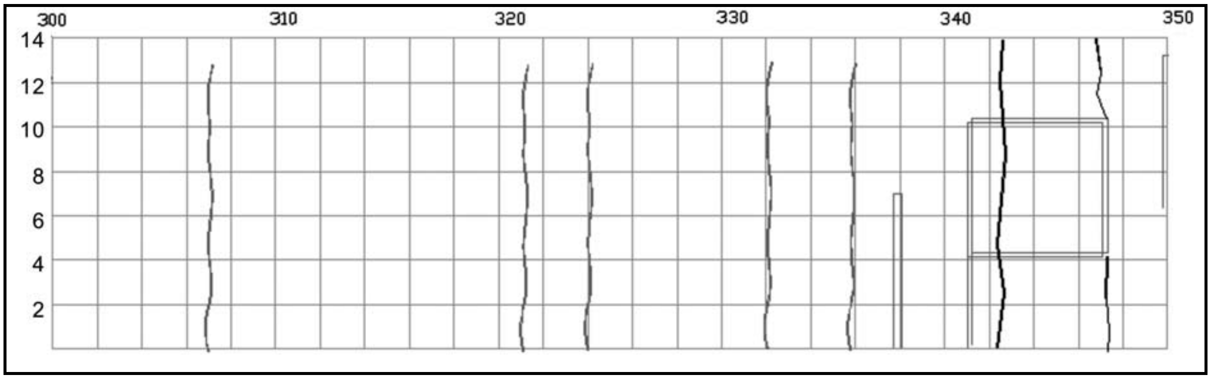


Figure C47. Visual Survey Chart for I-35 R1 Lane (300 to 350 ft, 07/15/2008 survey).

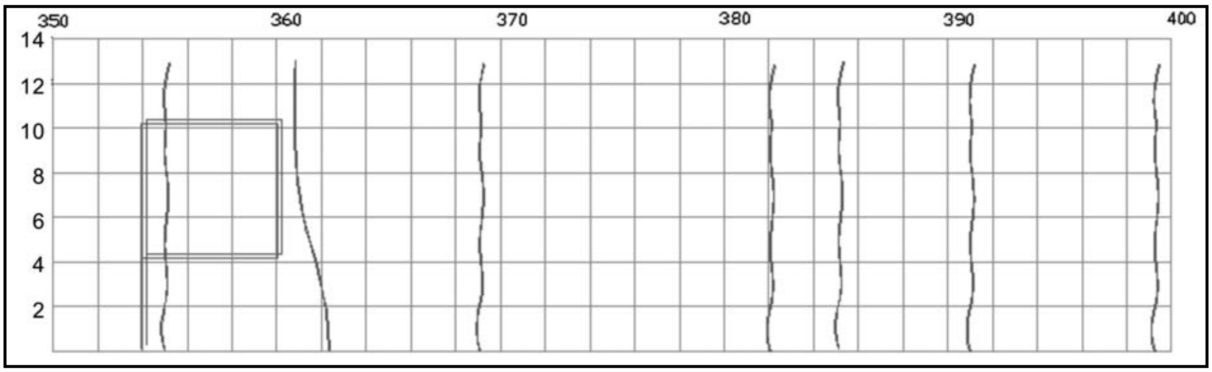


Figure C48. Visual Survey Chart for I-35 R1 Lane (350 to 400 ft, 07/15/2008 survey).

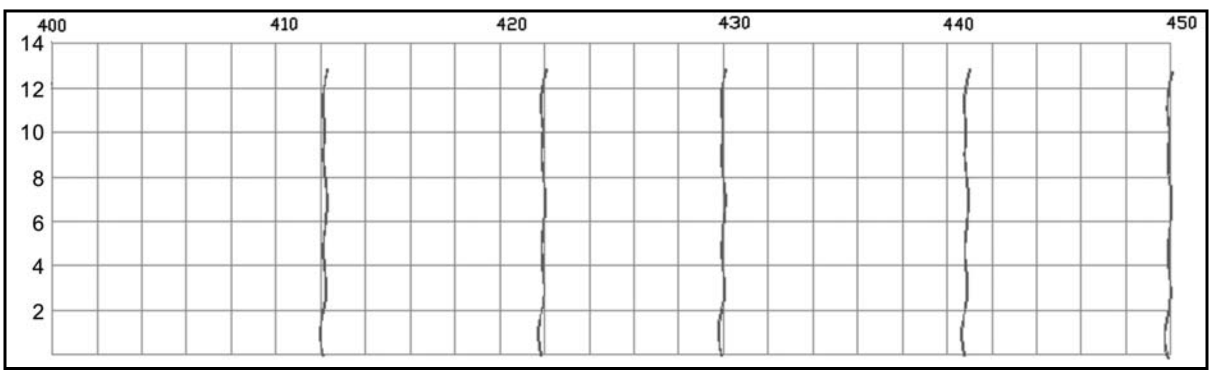


Figure C49. Visual Survey Chart for I-35 R1 Lane (400 to 450 ft, 07/15/2008 survey).

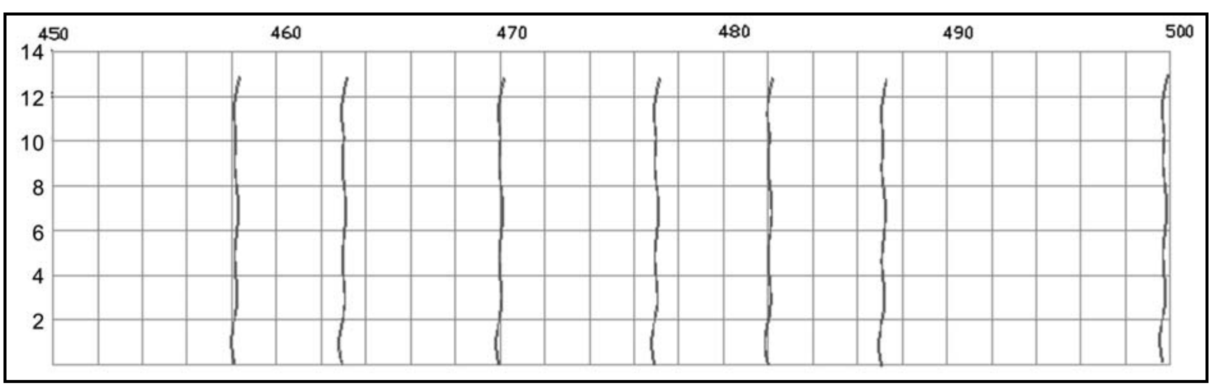


Figure C50. Visual Survey Chart for I-35 R1 Lane (450 to 500 ft, 07/15/2008 survey).

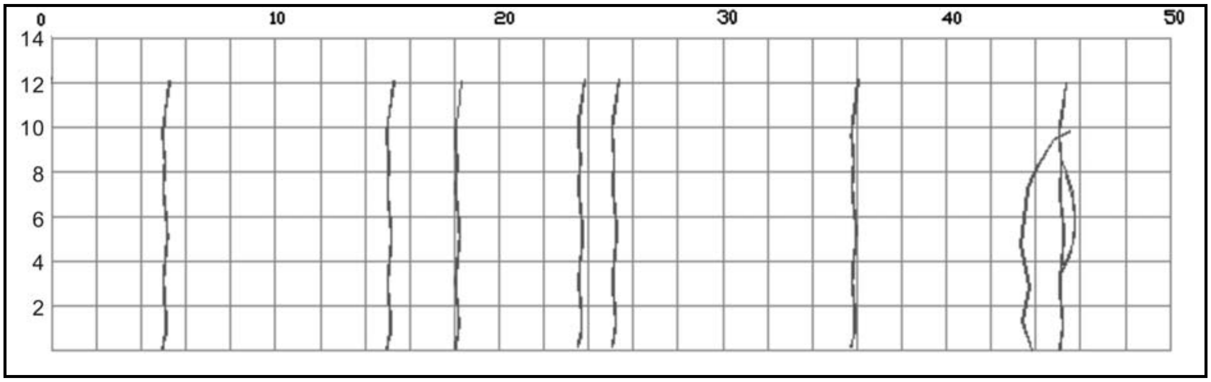


Figure C51. Visual Survey Chart for I-35 R2 Lane (0 to 50 ft, 07/15/2008 survey).

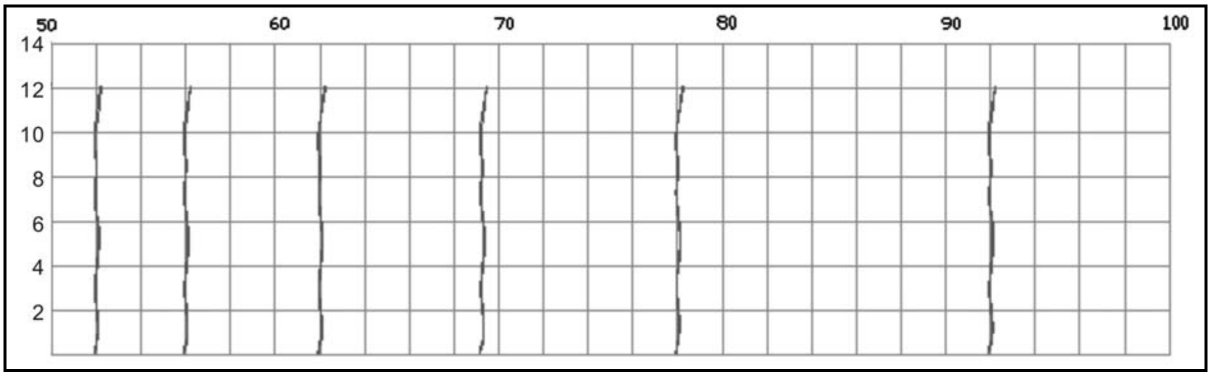


Figure C52. Visual Survey Chart for I-35 R2 Lane (50 to 100 ft, 07/15/2008 survey).

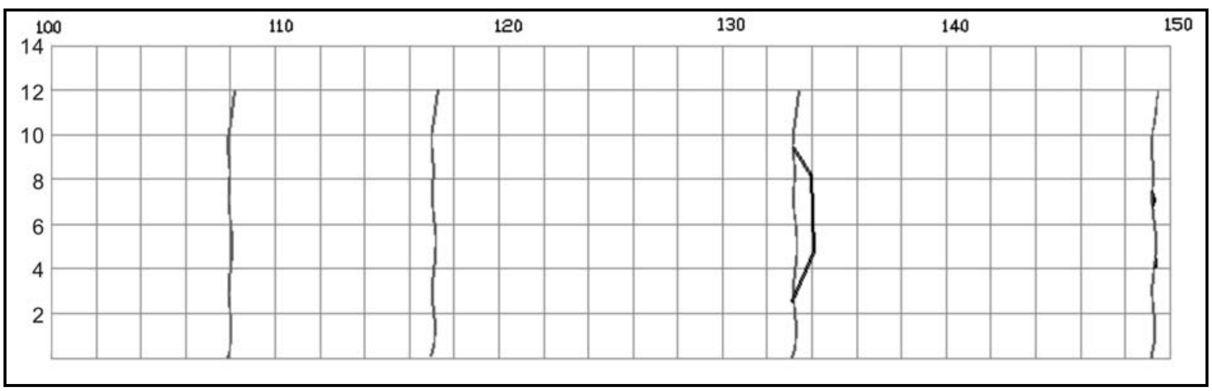


Figure C53. Visual Survey Chart for I-35 R2 Lane (100 to 150 ft, 07/15/2008 survey).

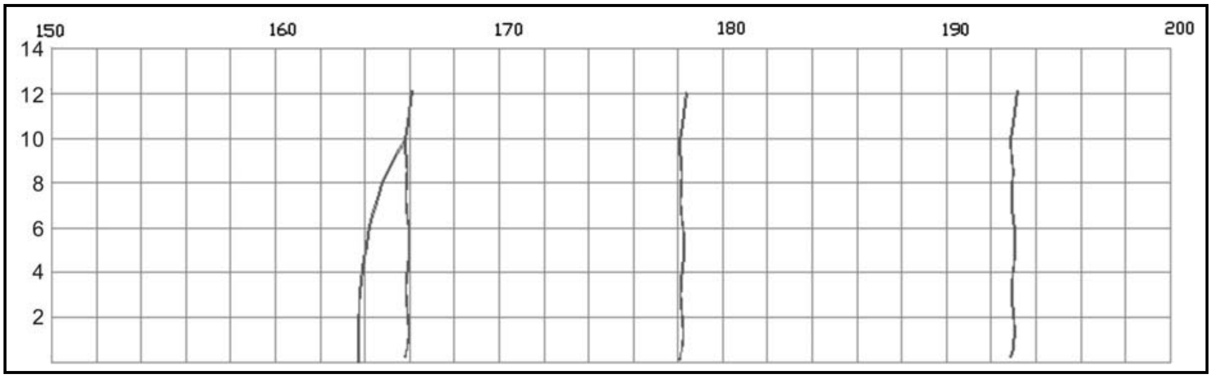


Figure C54. Visual Survey Chart for I-35 R2 Lane (150 to 200 ft, 07/15/2008 survey).

Figure C55. Visual Survey Chart for I-35 R2 Lane (200 to 250 ft, 07/15/2008 survey).

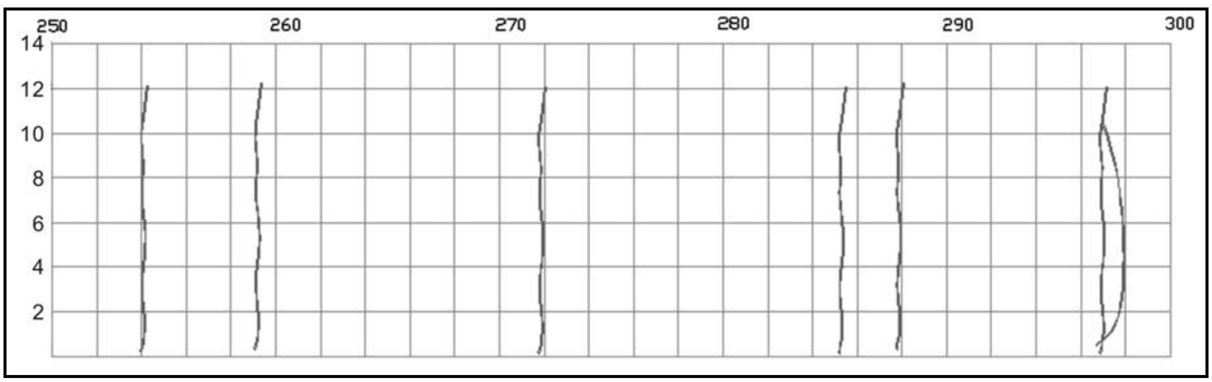


Figure C56. Visual Survey Chart for I-35 R2 Lane (250 to 300 ft, 07/15/2008 survey).

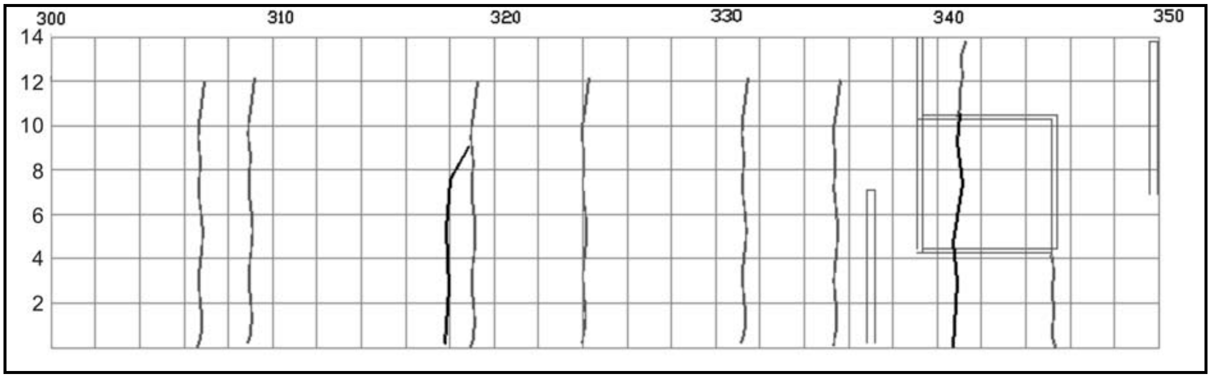


Figure C57. Visual Survey Chart for I-35 R2 Lane (300 to 350 ft, 07/15/2008 survey).

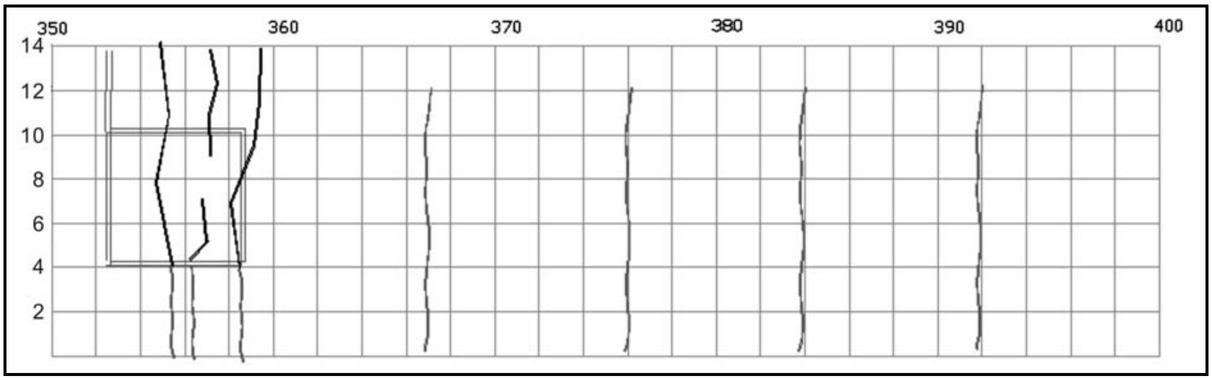


Figure C58. Visual Survey Chart for I-35 R2 Lane (350 to 400 ft, 07/15/2008 survey).

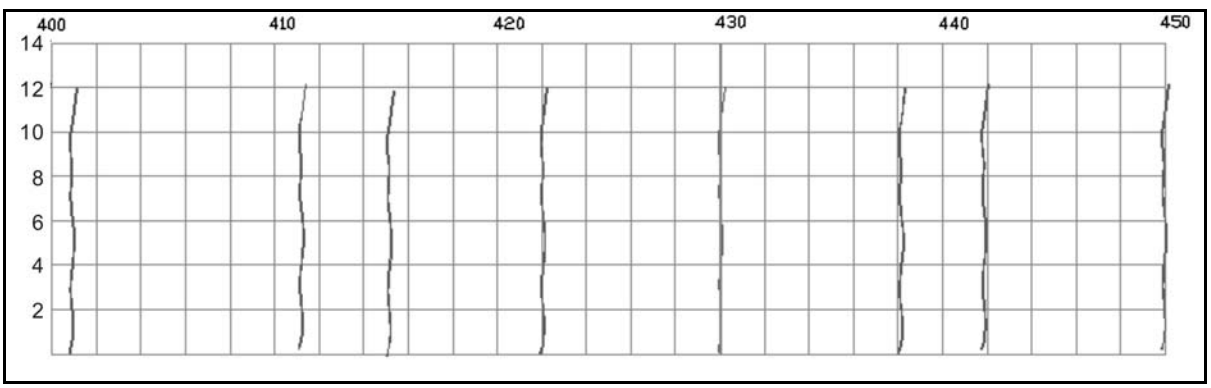


Figure C59. Visual Survey Chart for I-35 R2 Lane (400 to 450 ft, 07/15/2008 survey).

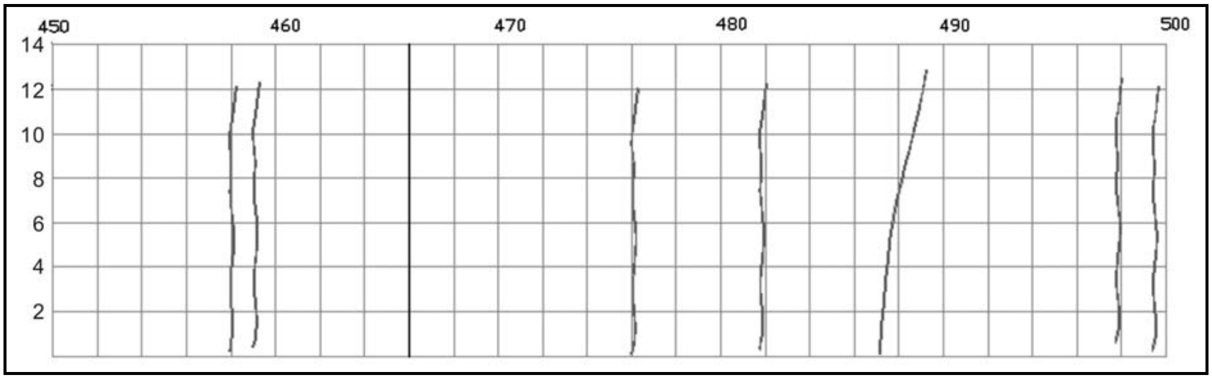


Figure C60. Visual Survey Chart for I-35 R2 Lane (450 to 500 ft, 07/15/2008 survey).

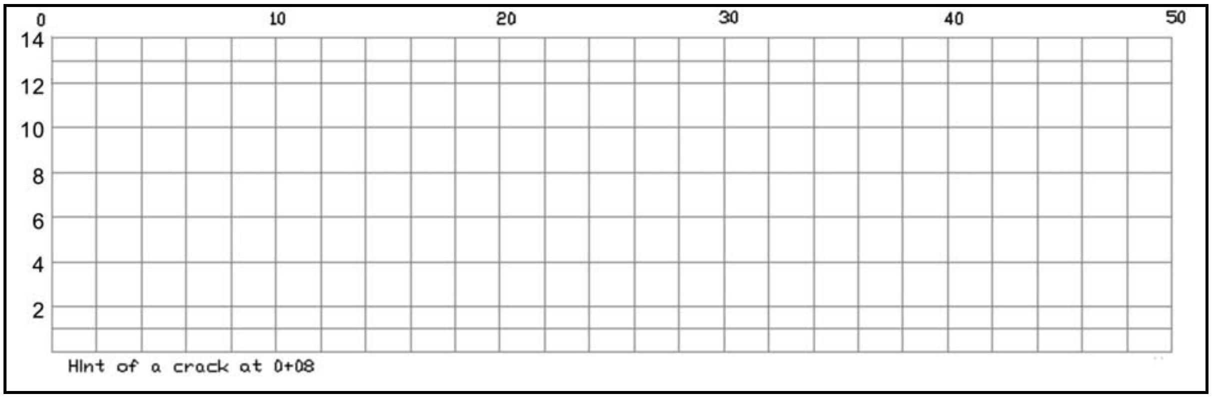


Figure C61. Visual Survey Chart for I-35 L1 Lane (0 to 50 ft, 07/15/2008 survey).

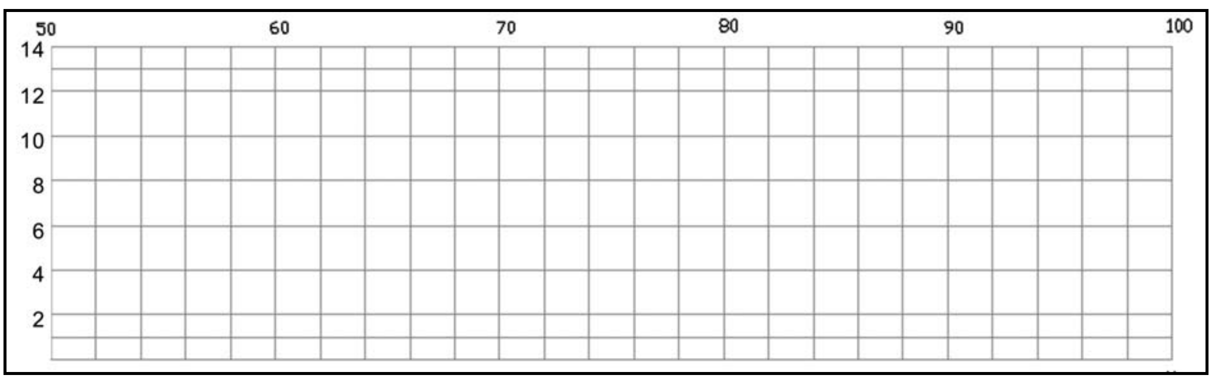


Figure C62. Visual Survey Chart for I-35 L1 Lane (50 to 100 ft, 07/15/2008 survey).

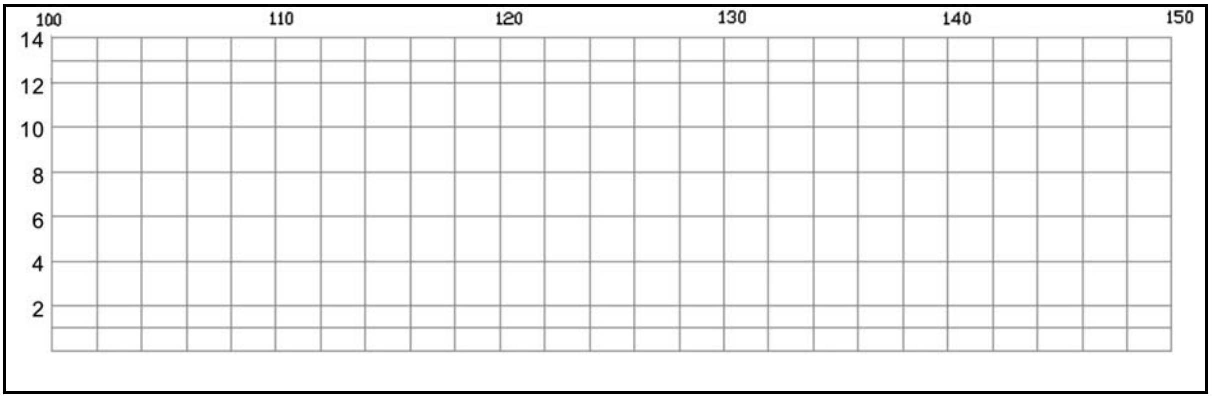


Figure C63. Visual Survey Chart for I-35 L1 Lane (100 to 150 ft, 07/15/2008 survey).

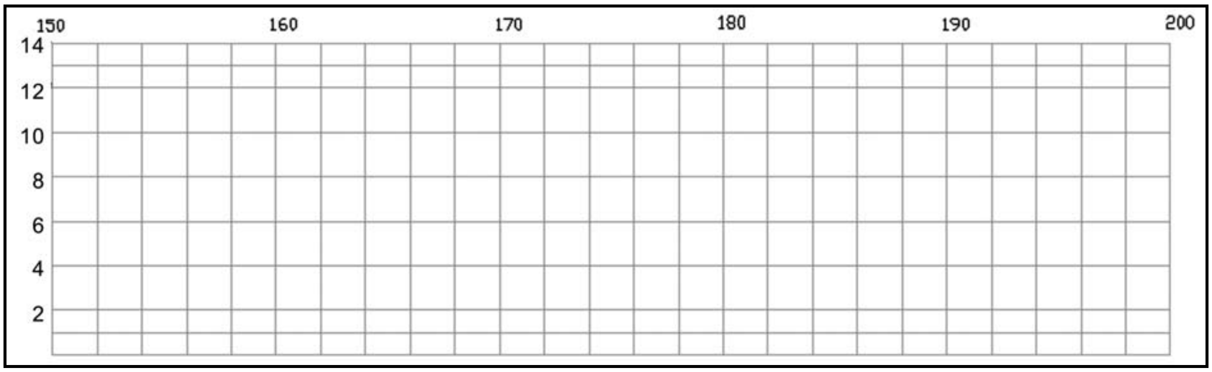


Figure C64. Visual Survey Chart for I-35 L1 Lane (150 to 200 ft, 07/15/2008 survey).

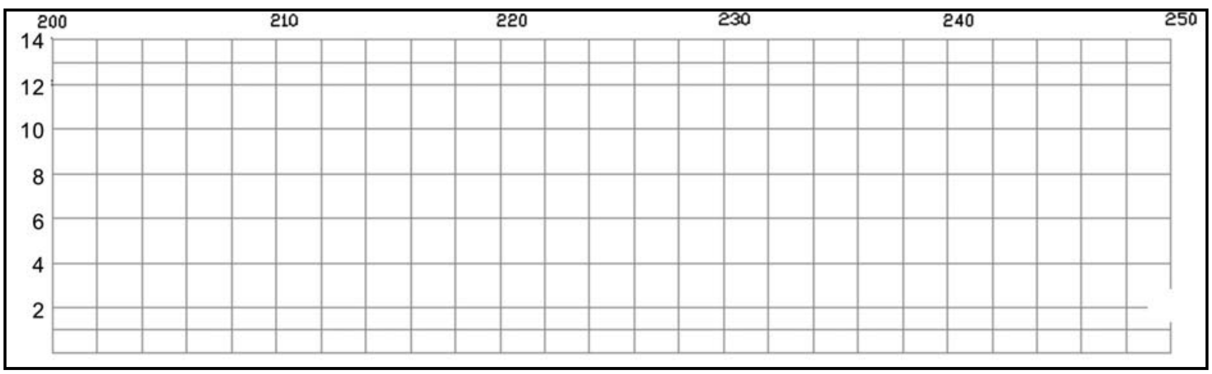


Figure C65. Visual Survey Chart for I-35 L1 Lane (200 to 250 ft, 07/15/2008 survey).

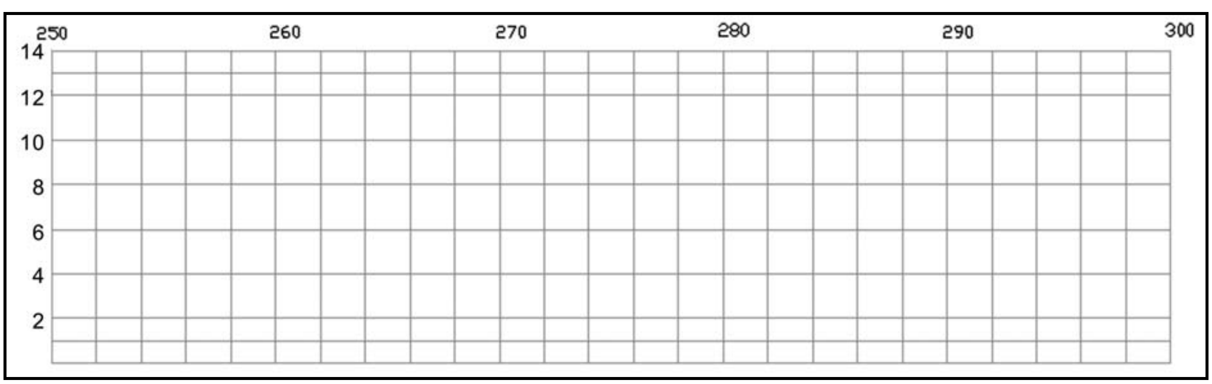


Figure C66. Visual Survey Chart for I-35 L1 Lane (250 to 300 ft, 07/15/2008 survey).

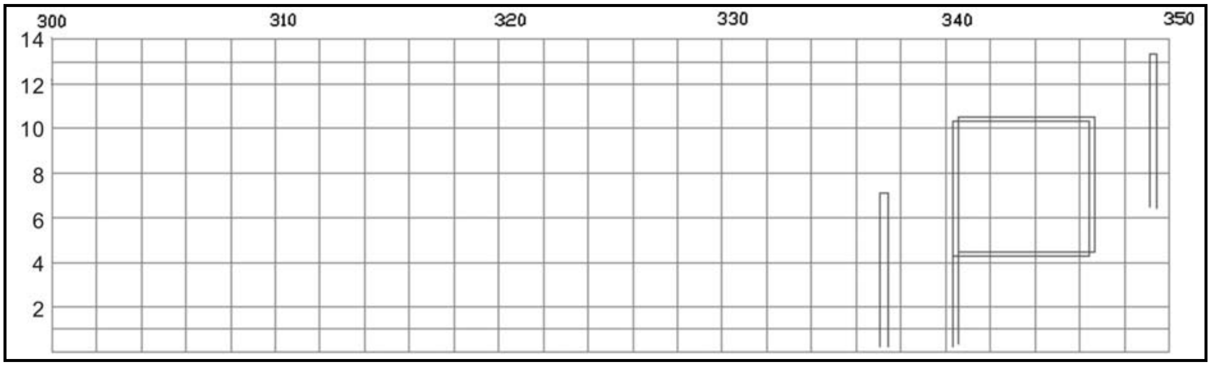


Figure C67. Visual Survey Chart for I-35 L1 Lane (300 to 350 ft, 07/15/2008 survey).

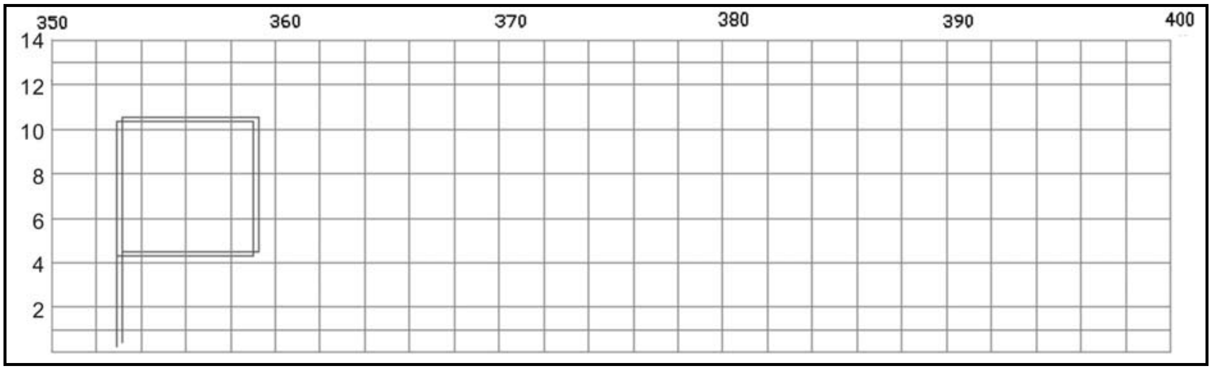


Figure C68. Visual Survey Chart for I-35 L1 Lane (350 to 400 ft, 07/15/2008 survey).

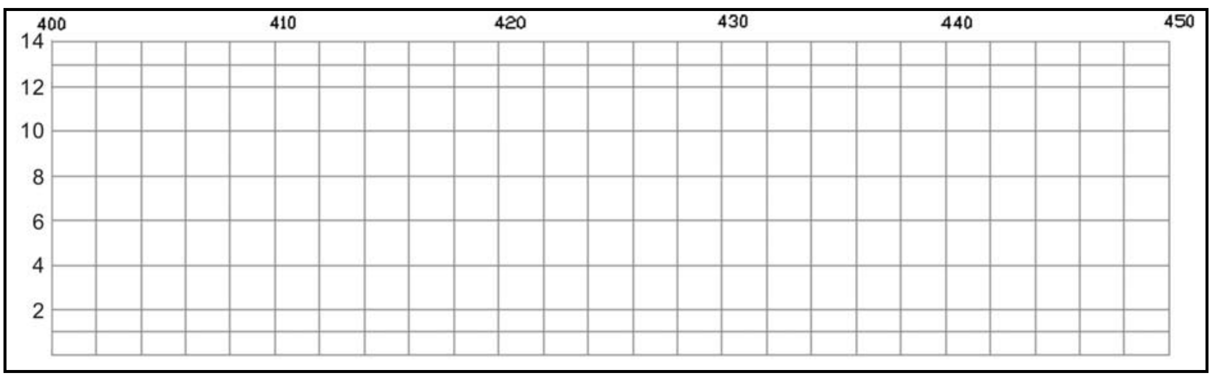


Figure C69. Visual Survey Chart for I-35 L1 Lane (400 to 450 ft, 07/15/2008 survey).

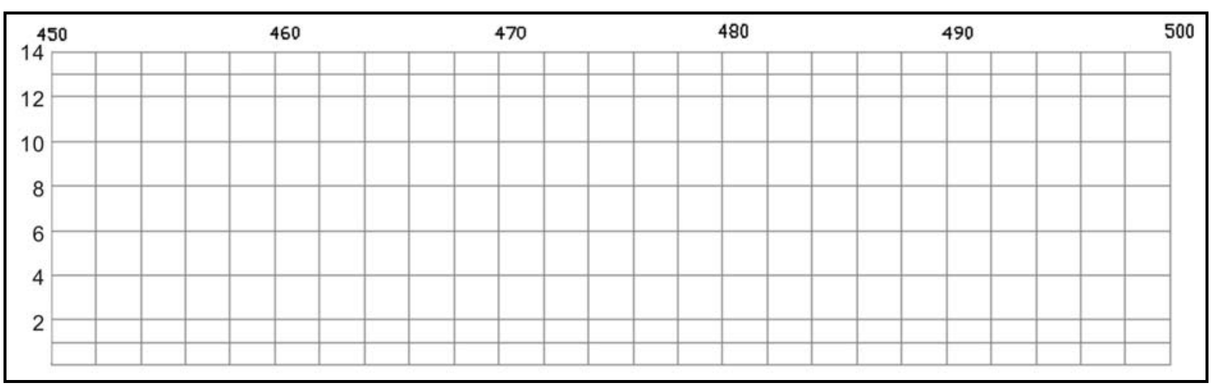


Figure C70. Visual Survey Chart for I-35 L1 Lane (450 to 500 ft, 07/15/2008 survey).

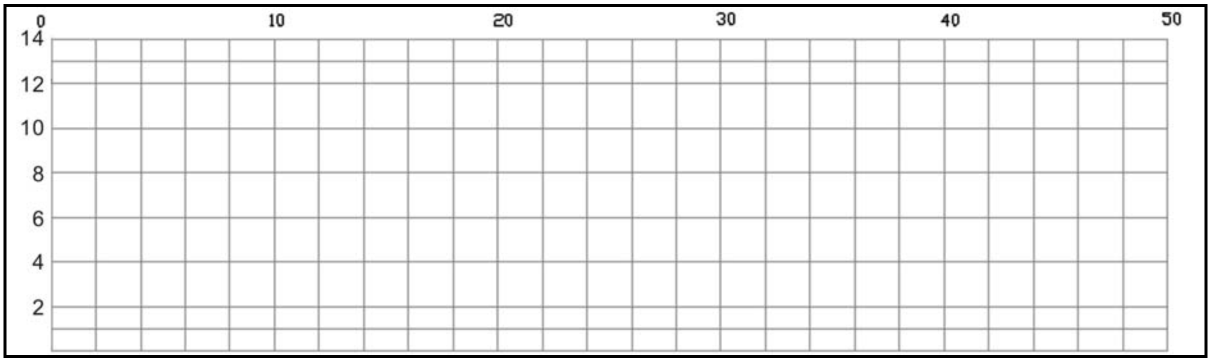


Figure C71. Visual Survey Chart for I-35 L2 Lane (0 to 50 ft, 07/15/2008 survey).

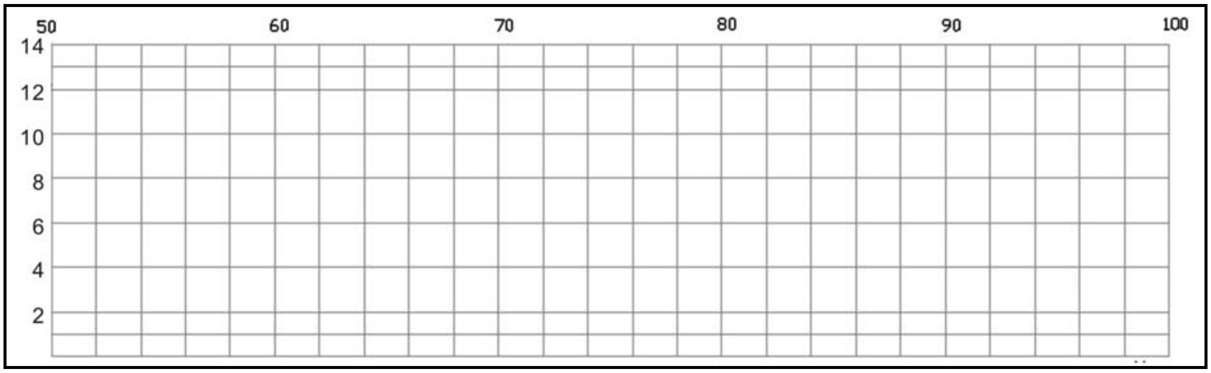


Figure C72. Visual Survey Chart for I-35 L2 Lane (50 to 100 ft, 07/15/2008 survey).

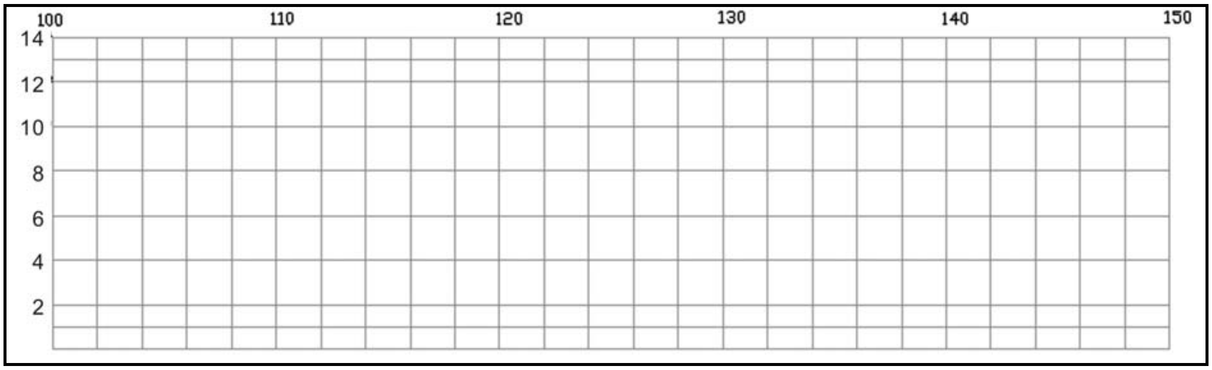


Figure C73. Visual Survey Chart for I-35 L2 Lane (100 to 150 ft, 07/15/2008 survey).

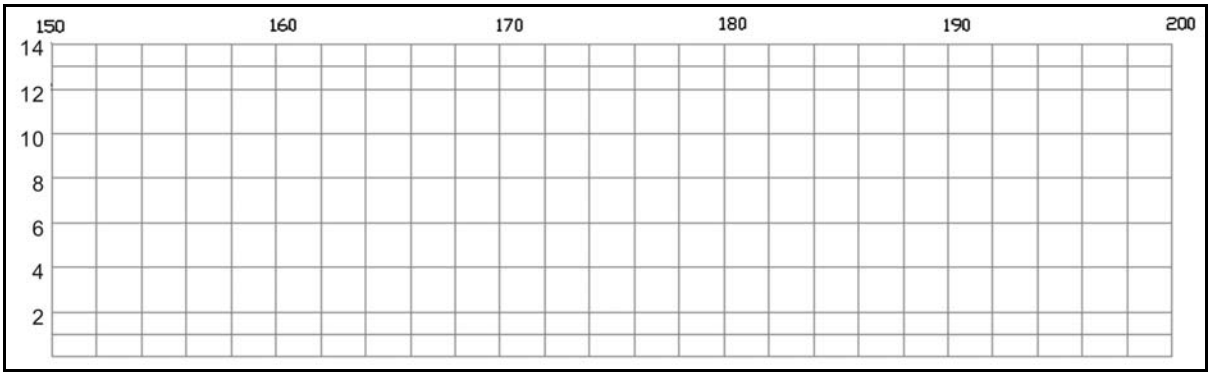


Figure C74. Visual Survey Chart for I-35 L2 Lane (150 to 200 ft, 07/15/2008 survey).

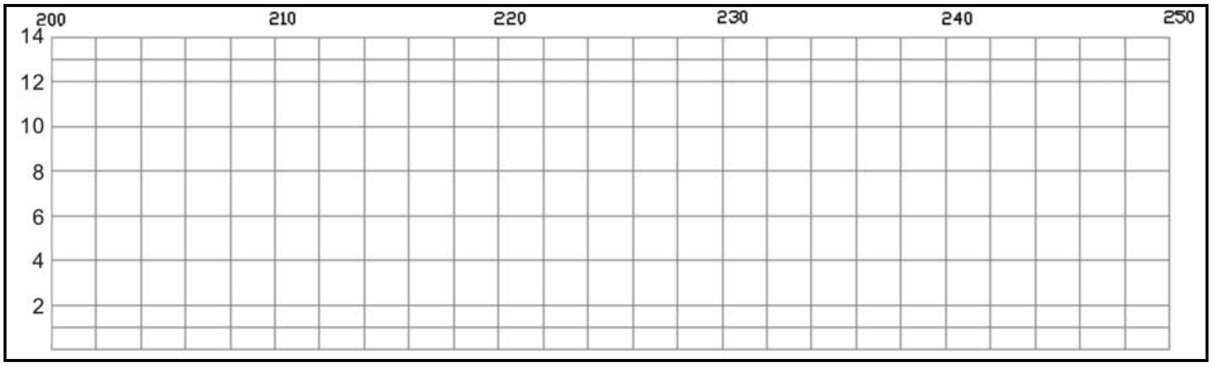


Figure C75. Visual Survey Chart for I-35 L2 Lane (200 to 250 ft, 07/15/2008 survey).

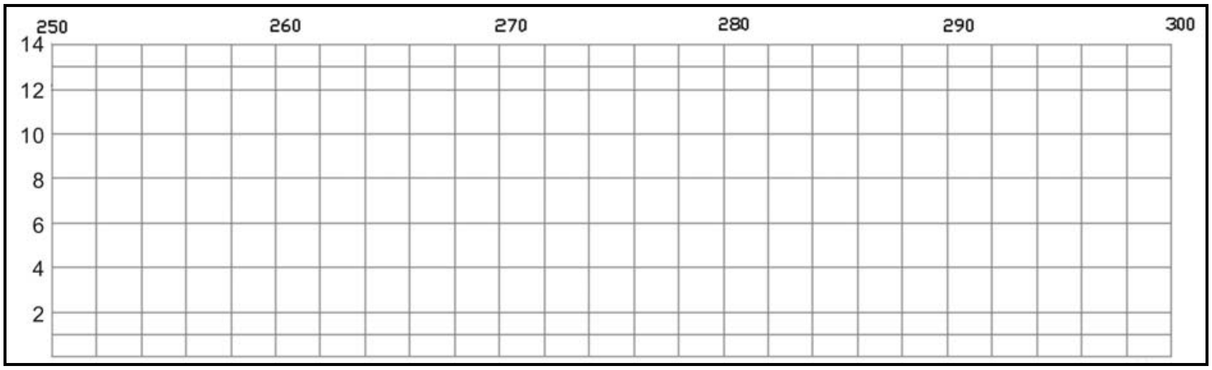


Figure C76. Visual Survey Chart for I-35 L2 Lane (250 to 300 ft, 07/15/2008 survey).

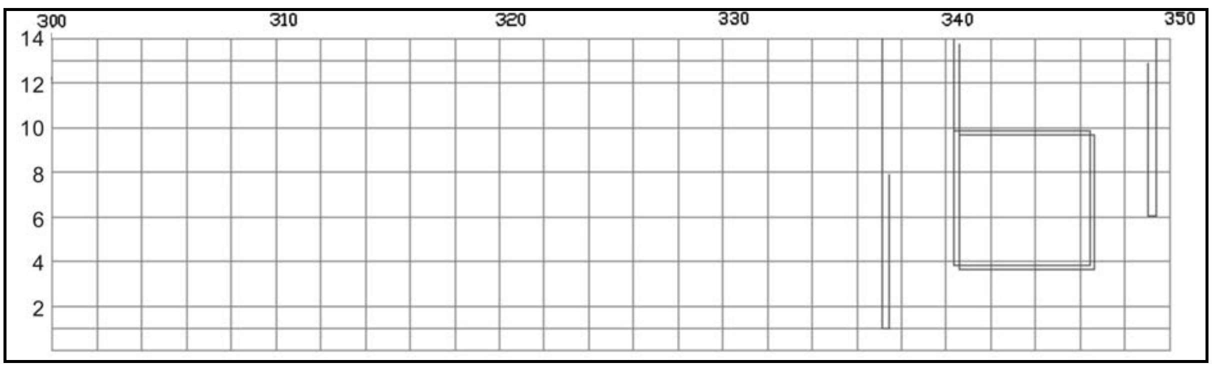


Figure C77. Visual Survey Chart for I-35 L2 Lane (300 to 350 ft, 07/15/2008 survey).

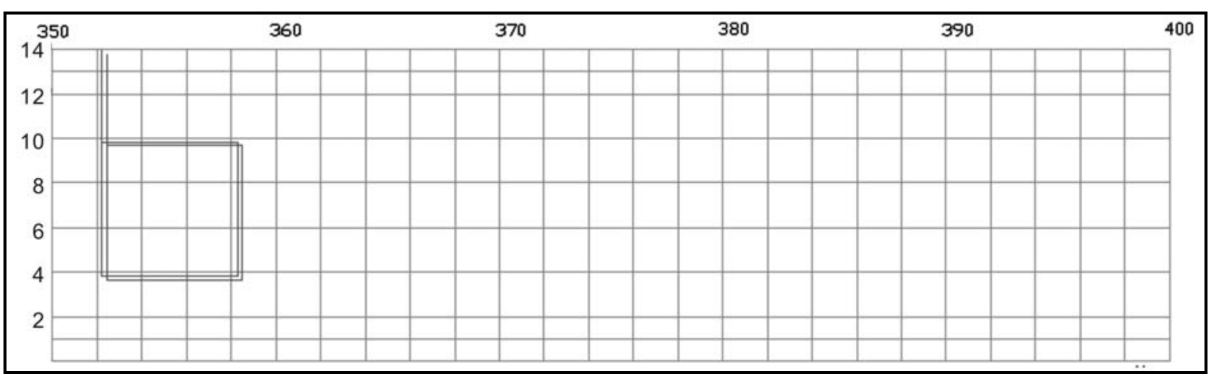


Figure C78. Visual Survey Chart for I-35 L2 Lane (350 to 400 ft, 07/15/2008 survey).

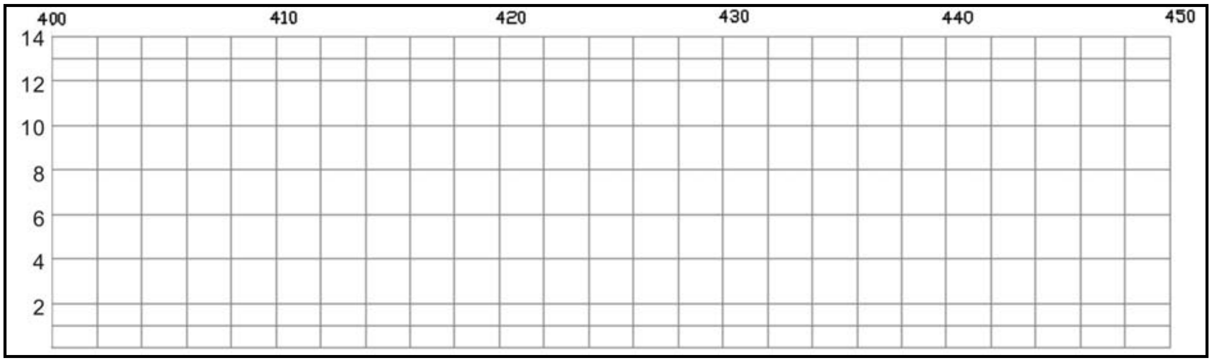


Figure C79. Visual Survey Chart for I-35 L2 Lane (400 to 450 ft, 07/15/2008 survey).

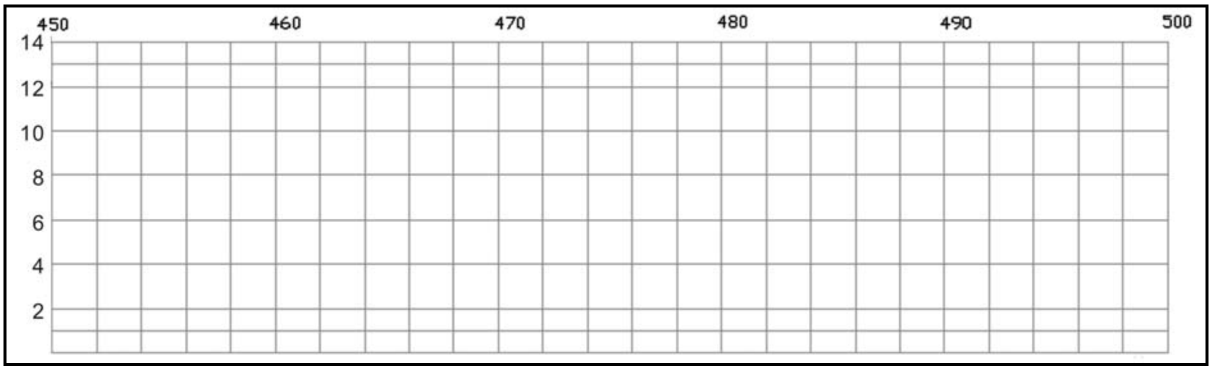


Figure C80. Visual Survey Chart for I-35 L2 Lane (450 to 500 ft, 07/15/2008 survey).

APPENDIX D

FWD DEFLECTIONS AND BACKCALCULATED LAYER MODULI FROM EVALUATION OF CANDIDATE WIM SITES

Table D1. FWD Deflections on L1 Lane of US77 WIM Site in Robstown.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9995	6.75	5.41	4.01	3.03	2.40	1.94	1.54	60
10	9983	6.70	5.42	4.06	2.95	2.37	1.85	1.50	59
20	9963	6.89	5.57	4.21	3.12	2.29	1.91	1.56	59
30	9955	6.44	5.11	3.97	2.96	2.45	1.98	1.61	59
40	9923	7.55	6.12	4.63	3.50	2.62	2.07	1.78	60
50	9844	8.05	6.56	4.83	3.70	2.78	2.26	1.75	59
60	9884	8.26	6.75	5.12	3.74	2.82	2.30	1.76	56
70	9860	7.69	6.44	4.93	3.76	2.83	2.37	1.82	57
80	9836	8.50	6.93	5.22	3.86	2.98	2.37	1.86	57
90	9880	8.36	6.85	5.18	3.72	3.01	2.27	1.93	58
100	9828	8.32	6.83	5.15	3.88	2.94	2.33	1.91	56
110	9832	9.52	7.59	5.64	4.03	2.99	2.47	1.89	55
120	9840	8.46	6.68	4.89	3.62	2.79	2.11	1.61	55
130	9900	7.33	5.81	4.30	3.09	2.50	2.06	1.70	55
140	9908	7.25	5.65	4.26	3.14	2.38	2.03	1.56	55
150	9844	6.88	5.43	4.19	3.17	2.40	1.94	1.50	55
160	9836	7.07	5.70	4.20	3.23	2.46	1.98	1.57	56
170	9844	8.02	6.56	4.98	3.69	2.87	2.30	1.83	56
180	9880	7.61	6.01	4.54	3.36	2.54	2.09	1.61	55
190	9812	7.27	5.93	4.68	3.49	2.82	2.20	1.87	55
200	9792	7.55	6.04	4.65	3.58	2.74	2.22	1.80	56
210	9836	8.16	6.81	5.39	4.01	3.13	2.56	2.06	58
220	9780	8.79	7.17	5.33	3.95	2.95	2.30	1.87	57
230	9840	8.06	6.39	4.66	3.35	2.54	2.11	1.65	57
240	9844	8.13	6.42	4.53	3.21	2.49	1.94	1.63	57
250	9888	7.92	6.24	4.43	3.23	2.48	1.90	1.61	58
260	9900	7.17	5.66	4.03	2.92	2.18	1.73	1.43	59
270	9840	6.28	5.05	3.77	2.75	2.21	1.85	1.53	59
280	9824	6.72	5.28	3.89	2.81	2.29	1.86	1.55	59
290	9816	7.04	5.61	4.22	3.18	2.35	1.90	1.54	58
300	9780	7.46	5.78	4.16	2.99	2.20	1.72	1.36	59
310	9776	7.42	5.83	4.17	3.06	2.24	1.81	1.51	55
320	9737	7.86	6.13	4.39	3.14	2.29	1.83	1.39	56
330	9745	7.33	5.72	4.12	3.04	2.27	1.77	1.43	57
340	9840	7.13	5.75	4.23	3.03	2.32	1.82	1.44	58
350	9808	7.86	6.19	4.53	3.20	2.41	1.78	1.41	54
360	9808	7.88	6.21	4.50	3.28	2.43	1.84	1.40	58
370	9741	6.48	5.07	3.63	2.60	2.04	1.50	1.17	57
380	9780	6.48	5.08	3.67	2.63	1.90	1.46	1.11	56
390	9868	5.88	4.59	3.24	2.37	1.74	1.38	1.11	55
400	9808	5.78	4.54	3.35	2.50	1.91	1.44	1.15	60
410	9824	5.68	4.43	3.30	2.51	1.91	1.54	1.24	58
420	9848	6.79	5.43	4.05	2.86	2.09	1.66	1.18	59
430	9832	5.97	4.70	3.39	2.43	1.76	1.53	1.13	59

Table D1. FWD Deflections on L1 Lane of US77 WIM Site in Robstown (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9844	6.06	4.90	3.59	2.64	2.00	1.49	1.22	59
450	9876	5.88	4.56	3.33	2.44	1.86	1.45	1.25	58
460	9864	6.35	5.05	3.81	2.85	2.10	1.67	1.31	58
470	9900	5.69	4.37	3.21	2.39	1.90	1.59	1.31	58
480	9935	5.11	3.98	3.02	2.36	1.83	1.67	1.30	58
490	9908	5.06	4.01	2.96	2.25	1.85	1.41	1.24	58
500	9852	5.35	4.09	2.93	2.17	1.65	1.34	1.19	58

Table D2. FWD Deflections on L2 Lane of US77 WIM Site in Robstown.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9927	6.64	5.21	3.96	3.08	2.60	2.17	1.87	73
10	9931	6.46	4.90	3.77	3.04	2.47	2.14	1.80	68
20	9951	5.99	4.61	3.63	2.92	2.42	2.03	1.78	70
30	9923	5.87	4.69	3.65	2.99	2.48	2.09	1.77	71
40	9915	5.80	4.56	3.55	2.80	2.39	2.06	1.60	72
50	9943	5.74	4.62	3.55	2.77	2.28	1.99	1.69	67
60	9939	5.30	4.28	3.36	2.74	2.27	2.01	1.69	72
70	9935	5.34	4.30	3.42	2.81	2.32	2.02	1.65	71
80	9908	5.40	4.35	3.47	2.78	2.31	1.97	1.62	70
90	9876	5.50	4.42	3.43	2.66	2.09	1.87	1.60	68
100	9939	5.37	4.36	3.34	2.68	2.10	1.81	1.61	73
110	9931	5.89	4.85	3.66	2.85	2.35	1.97	1.64	71
120	10003	5.51	4.61	3.62	2.86	2.30	1.98	1.57	72
130	9959	6.18	5.00	3.76	3.01	2.40	2.15	1.98	72
140	9995	6.30	4.93	3.78	2.94	2.42	2.07	1.88	73
150	9951	6.11	4.73	3.53	2.77	2.11	1.87	1.75	71
160	9900	6.53	5.13	3.89	2.96	2.31	1.91	1.48	73
170	9884	7.40	5.87	4.43	3.36	2.74	2.17	1.83	72
180	9892	7.07	5.74	4.41	3.28	2.55	1.96	1.60	72
190	9892	6.84	5.57	4.23	3.21	2.47	2.01	1.70	71
200	9868	6.98	5.76	4.44	3.39	2.62	2.18	1.82	73
210	9880	6.71	5.55	4.36	3.40	2.59	2.01	1.52	71
220	9884	6.52	5.22	4.07	3.19	2.53	2.00	1.63	72
230	9884	5.45	4.52	3.57	2.88	2.28	1.83	1.53	72
240	9935	5.17	4.19	3.28	2.64	2.17	1.81	1.45	73
250	9908	5.32	4.09	3.15	2.55	2.12	1.73	1.36	71
260	9915	5.67	4.57	3.46	2.72	2.13	1.78	1.49	72
270	9880	5.81	4.55	3.35	2.54	2.18	1.84	1.57	72
280	9904	6.26	4.96	3.70	2.87	2.21	1.85	1.47	71
290	9892	5.79	4.29	3.29	2.64	2.09	1.77	1.67	71
300	9896	5.75	4.59	3.49	2.71	2.24	1.86	1.63	73
310	9999	5.14	3.97	3.10	2.42	2.05	1.74	1.58	73
320	9931	5.00	3.87	3.05	2.40	2.04	1.88	1.63	73
330	9919	5.14	3.99	3.18	2.54	2.02	1.70	1.35	73
340	9880	5.43	4.03	3.11	2.49	1.99	1.80	1.62	73
350	9915	5.18	4.03	3.17	2.52	2.12	1.87	1.53	73
360	10003	5.28	4.13	3.27	2.54	2.06	1.94	1.53	75
370	9955	5.49	4.45	3.48	2.67	2.18	1.78	1.51	75
380	9904	5.59	4.35	3.37	2.63	2.05	1.76	1.44	74
390	9884	5.53	4.40	3.46	2.74	2.15	1.80	1.56	74
400	9951	5.67	4.46	3.46	2.72	2.22	1.84	1.56	76
410	9884	6.50	4.98	3.83	2.94	2.35	1.93	1.61	78
420	9880	6.09	4.80	3.69	2.83	2.26	1.78	1.50	77
430	9892	6.08	4.67	3.65	2.72	2.28	1.83	1.56	77

Table D2. FWD Deflections on L2 Lane of US77 WIM Site in Robstown (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9935	6.07	4.81	3.67	2.87	2.33	1.96	1.58	76
450	9995	6.11	4.91	3.71	2.87	2.26	1.85	1.54	78
460	9900	6.00	4.74	3.60	2.81	2.22	1.91	1.59	74
470	9939	5.77	4.48	3.42	2.67	2.19	1.81	1.64	75
480	9915	5.38	4.37	3.43	2.63	2.17	1.78	1.46	76
490	9908	5.71	4.65	3.59	2.74	2.17	1.78	1.52	77
500	9915	5.40	4.34	3.36	2.63	2.09	1.76	1.45	73

Table D3. FWD Deflections on R1 Lane of US77 WIM Site in Robstown.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	10043	6.18	4.93	3.87	2.95	2.41	1.94	1.64	63
10	10027	6.36	5.14	3.93	3.00	2.43	1.98	1.65	61
20	10031	6.38	5.17	4.00	3.05	2.44	2.02	1.66	62
30	9963	6.29	5.14	4.02	3.13	2.53	2.12	1.79	62
40	9955	6.46	5.32	4.16	3.19	2.48	2.10	1.76	62
50	9955	6.56	5.36	4.19	3.20	2.53	2.12	1.76	61
60	9995	6.26	5.17	3.97	3.14	2.44	2.07	1.65	63
70	9935	6.18	5.11	3.91	3.05	2.37	1.96	1.58	62
80	9919	6.19	5.01	3.87	2.94	2.40	2.02	1.69	62
90	9931	6.37	5.23	4.06	3.20	2.45	2.10	1.67	61
100	9951	6.86	5.61	4.30	3.32	2.60	2.24	1.85	63
110	9959	6.89	5.62	4.32	3.30	2.60	2.18	1.73	65
120	9975	6.63	5.52	4.28	3.28	2.62	2.15	1.80	64
130	9911	6.39	5.43	4.20	3.22	2.53	2.17	1.86	64
140	9888	6.66	5.47	4.33	3.30	2.61	2.22	1.68	64
150	9931	6.73	5.54	4.27	3.22	2.54	2.07	1.75	65
160	9919	7.04	5.66	4.37	3.33	2.65	2.20	1.83	65
170	9931	7.10	5.81	4.35	3.32	2.63	2.14	1.70	65
180	9892	7.22	5.83	4.38	3.23	2.45	1.94	1.62	65
190	9864	6.57	5.14	3.87	2.81	2.18	1.76	1.56	65
200	9975	6.79	5.43	4.06	3.07	2.35	1.87	1.63	65
210	9975	6.48	5.20	3.86	2.76	2.15	1.58	1.35	65
220	9975	6.24	4.98	3.59	2.72	2.15	1.86	1.56	65
230	9908	6.28	4.94	3.74	2.75	2.20	1.76	1.33	65
240	9951	6.16	4.86	3.62	2.66	2.13	1.80	1.43	65
250	9852	6.13	4.76	3.52	2.65	2.09	1.62	1.35	66
260	9904	6.87	5.54	4.20	3.04	2.28	1.77	1.39	65
270	9840	7.41	5.29	3.43	2.49	1.97	1.71	1.40	65
280	9820	6.57	5.07	3.65	2.72	1.99	1.66	1.46	65
290	9816	6.47	5.06	3.68	2.76	2.11	1.78	1.48	65
300	9860	5.87	4.61	3.54	2.70	2.26	1.80	1.47	65
310	9908	6.02	4.91	3.75	2.87	2.36	1.87	1.63	63
320	9872	5.99	4.77	3.57	2.74	2.16	1.75	1.50	63
330	9904	6.79	5.10	3.69	2.81	2.25	2.01	1.60	64
340	9836	7.33	5.37	3.77	2.90	2.31	1.86	1.65	65
350	9868	7.96	6.16	4.49	3.23	2.43	1.93	1.56	62
360	9832	6.26	4.98	3.60	2.79	2.24	1.79	1.56	66
370	9927	6.37	5.04	3.86	3.06	2.34	1.99	1.66	65
380	9880	6.83	5.52	4.25	3.35	2.74	2.15	1.81	64
390	9812	7.22	5.60	3.93	3.10	2.42	1.96	1.62	63
400	9828	6.06	4.81	3.62	2.76	2.28	1.82	1.57	67
410	9959	6.04	4.86	3.71	2.81	2.27	1.76	1.44	69
420	9856	5.90	4.70	3.46	2.74	2.11	1.73	1.44	68
430	9836	6.61	5.34	4.03	2.96	2.24	1.83	1.37	68

Table D3. FWD Deflections on R1 Lane of US77 WIM Site in Robstown (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9848	5.82	4.59	3.48	2.72	2.06	1.73	1.44	67
450	9868	6.04	4.69	3.45	2.71	2.03	1.74	1.44	70
460	9943	5.95	4.72	3.41	2.69	1.97	1.74	1.51	68
470	9904	6.37	4.93	3.66	2.63	2.13	1.83	1.38	68
480	9923	6.51	4.88	3.57	2.75	2.24	1.67	1.45	69
490	9947	6.90	5.12	3.61	2.74	2.22	1.77	1.52	69
500	9880	8.17	5.90	3.92	2.94	2.31	1.88	1.57	68

Table D4. FWD Deflections on R2 Lane of US77 WIM Site in Robstown.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	10023	5.65	4.34	3.29	2.62	2.11	1.80	1.43	69
10	10015	4.82	3.68	2.86	2.31	1.96	1.74	1.51	67
20	9955	5.25	4.03	3.07	2.47	2.04	1.82	1.43	68
30	9971	5.07	3.98	3.04	2.41	2.06	1.83	1.51	68
40	9991	5.07	4.06	3.29	2.59	2.09	1.65	1.37	69
50	9979	5.63	4.57	3.55	2.76	2.09	1.76	1.41	67
60	9935	5.50	4.35	3.45	2.69	2.33	1.97	1.59	68
70	9931	6.24	4.96	3.67	2.84	2.19	1.91	1.51	68
80	9951	6.17	5.00	3.77	2.90	2.24	1.78	1.52	67
90	9908	5.91	4.78	3.57	2.76	2.22	1.96	1.63	67
100	9852	6.69	5.17	3.89	2.98	2.32	2.06	1.76	68
110	9955	6.32	4.91	3.74	2.86	2.17	1.96	1.60	70
120	9939	6.29	4.91	3.75	2.87	2.31	1.89	1.69	70
130	9876	5.85	4.76	3.70	2.93	2.40	2.03	1.67	69
140	9955	5.67	4.71	3.70	2.83	2.35	1.88	1.54	69
150	9904	5.74	4.62	3.67	2.87	2.33	2.05	1.49	71
160	9923	5.91	4.82	3.70	2.95	2.43	1.95	1.69	68
170	9892	6.17	4.87	3.70	2.74	2.12	1.71	1.53	69
180	9908	6.08	4.79	3.59	2.67	2.20	1.89	1.65	69
190	9868	6.04	4.72	3.51	2.70	2.10	1.83	1.57	70
200	9908	5.67	4.62	3.47	2.57	2.11	1.84	1.54	67
210	9935	5.82	4.61	3.50	2.75	2.21	1.90	1.54	67
220	9880	5.68	4.53	3.47	2.65	2.13	1.78	1.54	67
230	9892	6.24	4.79	3.59	2.62	1.98	1.67	1.32	67
240	9927	6.11	4.71	3.54	2.72	2.11	1.72	1.42	67
250	9923	5.61	4.45	3.33	2.61	2.06	1.84	1.55	67
260	9892	6.78	5.46	4.18	3.17	2.45	1.94	1.65	66
270	9884	6.40	5.13	3.93	2.93	2.37	1.97	1.56	67
280	9876	6.49	5.03	3.76	2.99	2.20	1.98	1.63	67
290	9876	6.69	5.08	3.78	2.89	2.29	1.90	1.67	67
300	9979	6.31	4.96	3.65	2.73	2.28	1.85	1.62	66
310	9888	6.32	4.97	3.67	2.84	2.22	1.82	1.59	68
320	9884	6.55	5.26	3.91	2.98	2.37	1.92	1.65	67
330	9884	6.48	5.09	3.87	2.91	2.28	2.08	1.66	67
340	9856	6.60	5.29	3.95	3.04	2.44	1.99	1.62	66
350	9852	7.10	5.51	4.00	2.98	2.39	1.87	1.44	68
360	9892	6.52	5.08	3.83	2.93	2.33	1.81	1.57	69
370	9943	6.30	5.00	3.89	2.93	2.36	1.99	1.76	69
380	9911	6.30	5.06	3.83	2.94	2.30	1.80	1.53	69
390	9904	6.36	5.06	3.63	2.89	2.24	1.83	1.50	69
400	9816	6.17	4.93	3.63	2.75	2.21	1.77	1.44	69
410	9856	6.10	4.80	3.55	2.68	2.14	1.78	1.46	69
420	9908	5.90	4.71	3.58	2.76	2.21	1.81	1.47	69
430	9848	6.28	5.13	3.87	2.89	2.23	1.78	1.52	69

Table D4. FWD Deflections on R2 Lane of US77 WIM Site in Robstown (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9872	6.72	5.33	4.04	3.02	2.36	1.93	1.54	69
450	9856	7.20	5.66	4.07	3.07	2.30	1.87	1.61	69
460	9820	7.25	5.69	4.13	2.95	2.27	1.86	1.62	70
470	9784	7.30	5.50	3.99	2.94	2.30	1.88	1.47	70
480	9828	6.79	5.24	3.64	2.58	2.09	1.70	1.38	69
490	9860	6.46	5.07	3.68	2.72	2.17	1.80	1.66	69
500	9888	6.61	5.24	3.87	2.97	2.37	1.91	1.62	70

Table D5. Backcalculated Layer Moduli for L1 Lane of US77 in Robstown.

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
0	1250.1	62.6	14.8	1.10
10	1410.9	53.9	15.7	1.36
20	1482.9	47.2	15.8	1.66
30	1284.9	74.3	14.0	1.20
40	1410.3	42.2	14.1	0.52
50	1190.4	43.5	12.8	1.23
60	1266.4	38.2	12.9	1.65
70	1542.4	41.4	12.4	1.53
80	1188.6	39.0	12.2	1.02
90	1254.0	37.9	12.7	1.43
100	1290.5	37.9	12.4	0.67
110	937.1	34.4	12.1	2.13
120	1018.3	41.0	13.4	0.53
130	1050.6	57.5	14.2	2.15
140	1077.6	57.6	14.5	1.99
150	1361.2	54.4	14.7	0.99
160	1273.9	53.0	14.4	0.93
170	1284.1	42.1	12.6	1.13
180	1148.9	49.1	14.0	1.32
190	1521.7	49.0	12.6	0.92
200	1247.4	51.7	12.6	0.68
210	1555.1	38.1	11.4	1.40
220	1198.0	32.5	12.7	0.70
230	1009.4	44.5	14.1	2.18
240	942.1	42.5	15.1	1.60
250	1003.1	44.0	15.3	1.07
260	1121.0	47.4	17.1	1.41
270	1297.1	67.0	15.7	2.24
280	1080.2	64.5	15.5	1.84
290	1351.6	48.1	15.2	0.94
300	1058.4	43.6	17.0	0.94
310	1068.5	45.9	16.2	1.43
320	1009.4	40.4	16.2	1.48
330	1077.8	47.2	16.1	0.64
340	1303.4	44.1	16.2	1.36
350	1159.7	35.8	16.4	0.63
360	1118.0	38.3	15.8	0.30
370	1227.8	51.2	18.8	1.18
380	1355.8	43.6	20.4	0.77
390	1322.5	57.6	21.4	1.30
400	1515.2	59.3	19.5	0.36
410	1379.7	72.5	18.3	0.57
420	1458.9	40.2	18.4	1.67
430	1293.1	59.6	19.9	2.99

Table D5. Backcalculated Layer Moduli for L1 Lane of US77 in Robstown (continued).

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
440	1666.0	46.9	19.6	0.56
450	1338.2	62.5	19.9	0.54
460	1536.4	51.1	17.4	0.88
470	1134.8	85.9	18.2	1.70
480	1274.4	114.5	17.1	2.44
490	1563.9	86.9	19.6	1.32
500	1257.6	77.7	21.7	1.28

Table D6. **Backcalculated Layer Moduli for L2 Lane of US77 in Robstown.**

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
0	927.7	86.1	14.0	1.29
10	817.0	102.6	14.1	1.01
20	975.5	109.6	14.4	0.35
30	1040.0	113.2	13.7	0.61
40	1032.1	113.5	14.4	1.39
50	1040.0	93.5	16.1	2.21
60	1040.0	141.2	14.3	1.83
70	1040.0	142.5	14.0	1.37
80	1040.0	127.9	14.5	1.42
90	1040.0	95.4	17.2	2.91
100	1040.0	88.0	18.5	3.86
110	1040.0	80.2	16.5	2.77
120	1040.0	118.0	14.7	2.49
130	1040.0	90.0	14.6	2.38
140	1012.8	89.0	15.0	1.36
150	1023.1	81.1	17.2	1.87
160	1040.0	65.0	16.7	1.11
170	1021.8	56.1	14.3	0.82
180	1040.0	54.8	15.3	1.82
190	1040.0	59.7	15.4	1.39
200	1040.0	61.1	14.2	1.90
210	1040.0	56.9	15.5	2.92
220	1040.0	67.1	15.4	1.76
230	1040.0	83.9	17.6	5.22
240	1040.0	136.7	15.5	1.93
250	1040.0	117.0	17.1	0.59
260	1040.0	84.9	17.8	2.19
270	986.6	95.0	17.0	2.41
280	1040.0	67.7	17.5	1.53
290	872.1	106.7	17.1	0.73
300	1040.0	92.5	16.5	1.48
310	1040.0	128.7	17.4	1.42
320	1040.0	154.5	16.0	2.61
330	1040.0	128.0	17.0	1.71
340	858.1	129.0	17.1	1.76
350	1040.0	139.9	15.8	1.72
360	1040.0	131.2	16.0	3.26
370	1040.0	95.7	17.3	2.16
380	1040.0	87.3	18.4	2.25
390	1040.0	94.9	17.1	2.09
400	1040.0	92.6	17.1	1.79
410	988.8	75.0	16.0	0.65
420	1040.0	77.5	16.9	1.20
430	1040.0	82.5	16.7	1.33

Table D6. Backcalculated Layer Moduli for L2 Lane of US77 in Robstown (continued).

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
440	1040.0	85.3	15.9	1.20
450	1040.0	76.1	16.8	1.19
460	1040.0	83.2	16.6	1.59
470	1040.0	96.4	16.8	0.75
480	1040.0	87.4	18.4	3.99
490	1040.0	83.5	17.5	2.27
500	1040.0	88.1	18.5	3.23

Table D7. Backcalculated Layer Moduli for R1 Lane of US77 in Robstown.

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
0	1540.0	80.5	14.8	0.75
10	1540.0	73.7	14.8	0.83
20	1540.0	73.4	14.6	1.14
30	1540.0	82.4	13.4	1.10
40	1540.0	72.5	13.8	1.63
50	1540.0	70.8	13.7	1.37
60	1540.0	76.6	14.1	1.57
70	1540.0	60.5	16.4	3.81
80	1540.0	79.3	14.4	1.48
90	1540.0	75.5	13.7	1.67
100	1535.1	66.0	13.3	1.61
110	1540.0	63.2	13.6	1.29
120	1540.0	70.1	13.3	1.25
130	1540.0	76.5	13.1	2.34
140	1540.0	69.4	13.0	1.68
150	1540.0	62.8	14.0	1.22
160	1420.7	64.4	13.3	1.03
170	1540.0	46.3	15.3	3.07
180	1535.2	46.1	15.6	0.75
190	1391.4	59.7	17.0	1.20
200	1521.7	55.6	16.0	0.38
210	1540.0	51.5	18.7	1.52
220	1273.6	77.6	16.5	2.54
230	1445.1	67.8	16.8	0.90
240	1347.0	75.4	16.9	2.04
250	1359.0	70.1	17.9	0.43
260	1540.0	47.5	16.9	0.82
270	607.1	69.5	18.8	3.63
280	1223.2	60.1	18.3	1.43
290	1223.2	67.7	16.9	1.66
300	1396.4	90.2	15.6	0.86
310	1540.0	76.7	15.3	1.15
320	1526.8	74.0	16.7	0.59
330	831.5	89.1	15.2	2.73
340	735.6	75.0	15.8	1.04
350	1094.8	45.4	15.7	0.98
360	1283.6	75.4	16.1	1.14
370	1420.7	77.6	14.6	1.14
380	1474.2	69.9	12.9	0.62
390	956.1	68.2	14.9	1.26
400	1386.1	81.4	15.6	0.90
410	1540.0	74.7	16.2	1.13
420	1508.2	76.9	16.8	1.07
430	1540.0	54.6	16.4	1.46

Table D7. Backcalculated Layer Moduli for R1 Lane of US77 in Robstown (continued).

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
440	1540.0	77.8	16.9	1.17
450	1280.9	80.2	17.1	1.68
460	1387.0	77.3	17.6	2.62
470	1141.1	77.2	16.7	2.67
480	1041.8	78.3	16.9	1.21
490	832.7	78.4	16.8	1.33
500	610.6	62.4	16.3	1.93

Table D8. Backcalculated Layer Moduli for R2 Lane of US77 in Robstown.

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
0	1186.9	113.4	16.3	0.93
10	1150.7	175.7	16.4	1.61
20	1119.5	143.8	15.9	1.86
30	1265.6	148.3	15.7	2.14
40	2000.0	100.8	16.8	1.47
50	2000.0	58.2	19.2	3.56
60	1359.0	127.0	14.1	1.31
70	1320.8	78.5	16.0	1.92
80	1799.8	61.0	16.9	0.28
90	1407.7	90.6	15.4	2.35
100	1067.3	81.2	14.7	2.10
110	1253.9	81.1	15.8	2.29
120	1324.3	80.1	15.6	0.62
130	1617.9	93.9	14.0	0.90
140	2000.0	79.4	15.3	1.02
150	1605.5	100.9	14.0	1.63
160	1694.2	85.9	14.5	0.62
170	1603.3	62.5	17.8	0.94
180	1250.0	87.1	16.1	2.22
190	1255.1	84.9	16.6	1.78
200	1578.2	85.5	16.7	2.73
210	1365.7	97.5	15.5	1.31
220	1598.6	85.3	16.6	1.13
230	1310.3	67.3	18.6	1.80
240	1328.8	77.2	17.3	0.53
250	1361.3	100.8	16.4	2.20
260	1680.9	53.4	15.5	0.34
270	1488.1	70.3	15.2	1.45
280	1166.8	79.8	15.4	2.31
290	1023.9	79.1	15.7	0.86
300	1221.4	80.4	16.3	1.63
310	1318.7	73.7	16.4	0.79
320	1432.6	65.7	15.6	0.86
330	1203.1	81.1	14.9	2.91
340	1365.8	69.9	14.8	0.85
350	1108.7	61.4	16.0	0.93
360	1344.1	68.4	16.1	0.53
370	1449.2	77.8	15.0	1.44
380	1711.0	61.2	16.5	0.20
390	1281.4	74.2	16.3	1.32
400	1442.5	70.0	16.8	1.02
410	1309.2	78.0	16.9	1.32
420	1566.9	79.8	16.3	0.63
430	1848.5	53.8	17.1	0.75

Table D8. Backcalculated Layer Moduli for R2 Lane of US77 in Robstown (continued).

Station (ft)	Backcalculated Modulus (ksi)			% Error per Sensor
	Surface	Base	Subgrade	
440	1416.9	61.5	15.6	0.98
450	1209.1	53.6	16.4	1.10
460	1191.9	51.3	16.8	1.81
470	933.4	63.0	16.1	1.13
480	1015.8	62.9	18.4	2.40
490	1212.9	69.6	17.0	1.70
500	1293.3	69.6	15.5	0.71

Table D9. FWD Deflections on L1 Lane of SH19 Project at Trinity.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9113	6.70	3.45	2.07	1.72	1.43	1.39	1.30	100
55	9435	9.06	4.19	2.35	1.83	1.65	1.56	1.24	100
100	9304	9.50	4.91	3.10	2.37	2.04	1.77	1.35	100
157	9232	10.25	5.20	3.13	2.39	2.09	1.87	1.40	100
200	10221	10.60	5.54	3.35	2.56	2.28	2.07	1.51	100
250	9240	9.83	5.05	2.98	2.33	2.09	1.87	1.44	96
350	9546	8.20	4.31	2.88	2.29	2.06	1.87	1.45	96
400	9351	7.82	3.75	2.41	1.96	1.75	1.61	1.23	96
450	9951	8.36	3.89	2.54	2.01	1.78	1.62	1.25	96
502	9312	6.61	2.69	1.69	1.46	1.32	1.24	1.02	95
601	9252	7.13	2.90	2.04	1.74	1.56	1.47	1.20	95
650	9089	8.19	3.51	2.30	1.90	1.69	1.54	1.24	95
700	9137	8.00	3.66	2.06	1.62	1.41	1.30	1.07	95
750	9081	8.61	4.00	2.37	1.81	1.57	1.48	1.17	106
800	9121	9.41	4.13	2.37	1.84	1.65	1.48	1.14	106
850	8994	8.02	3.85	2.41	1.94	1.73	1.55	1.20	106
900	8700	7.78	3.55	2.26	1.83	1.67	0.05	1.29	106
950	9061	7.90	3.67	2.31	1.93	1.69	1.59	1.33	106
1000	9041	7.04	3.22	2.00	1.72	1.59	1.50	1.25	100
1050	8978	7.56	3.23	2.04	1.73	1.58	1.50	1.22	100
1100	9105	7.83	3.54	2.46	2.00	1.78	1.62	1.31	100
1150	8930	7.59	3.29	2.37	2.11	1.94	1.80	1.49	100
1200	9034	6.22	2.87	2.09	1.87	1.76	1.65	1.38	100
1250	9010	6.88	3.19	2.15	1.92	1.72	1.57	1.31	101
1300	9010	6.81	2.86	1.99	1.73	1.58	1.50	1.17	101
1350	8918	7.90	3.54	2.40	1.98	1.81	1.70	1.34	101
1400	8938	7.65	3.56	2.27	1.91	1.73	1.61	1.28	101
1450	8994	7.24	3.40	2.26	1.97	1.76	1.63	1.28	101
1500	8994	5.91	2.98	2.17	1.85	1.69	1.56	1.24	106
1550	9284	5.64	2.66	2.00	1.72	1.56	1.45	1.13	106
1600	9204	5.82	2.84	2.08	1.78	1.61	1.49	1.19	106
1650	8902	7.85	3.59	2.26	1.84	1.56	1.44	1.13	106
1700	8946	6.85	3.03	2.00	1.72	1.52	1.42	1.17	106
1750	8891	7.88	3.76	2.22	1.78	1.53	1.34	1.09	106
1803	8902	6.53	2.93	2.04	1.72	1.52	1.38	1.13	106
1850	8696	9.78	4.83	2.96	2.19	1.85	1.71	1.35	106
1900	8827	9.26	4.57	2.86	2.18	1.87	1.67	1.33	106
2000	8767	8.69	4.91	3.20	2.25	1.86	1.63	1.31	108

Table D9. FWD Deflections on L1 Lane of SH19 Project at Trinity (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
2054	8775	11.09	5.98	3.68	2.59	2.09	1.88	1.48	108
2100	8787	10.74	5.88	3.37	2.22	1.75	1.58	1.17	108
2150	8771	10.31	5.89	3.60	2.48	2.01	1.76	1.39	108
2202	8867	9.78	5.23	3.07	2.09	1.70	1.50	1.17	108
2255	8823	12.35	6.78	3.92	2.55	2.01	1.76	1.34	110
2303	9137	9.27	5.27	3.43	2.55	2.08	1.85	1.42	110
2350	9320	7.31	4.04	2.92	2.33	2.04	1.89	1.46	110
2400	9407	9.56	5.37	3.54	2.67	2.22	1.96	1.47	110

Table D10. FWD Deflections on L2 Lane of SH19 Project at Trinity.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9133	6.57	3.02	1.85	1.50	1.36	1.29	1.05	111
50	9006	7.12	3.37	2.12	1.75	1.61	1.48	1.19	111
103	9010	7.89	3.97	2.65	2.11	1.85	1.67	1.26	111
150	8962	9.17	4.43	2.83	2.24	1.89	1.70	1.25	111
203	9049	8.17	4.28	2.83	2.20	1.92	1.78	1.36	111
250	9057	8.40	4.30	2.72	2.14	1.88	1.69	1.29	111
302	9125	7.32	3.76	2.46	1.92	1.71	1.57	1.17	111
350	9041	6.35	3.21	2.28	1.86	1.68	1.54	1.21	111
401	9057	6.65	3.51	2.37	1.92	1.70	1.59	1.23	111
450	9053	7.01	3.54	2.57	2.06	1.80	1.63	1.22	111
500	8982	6.53	2.98	2.10	1.70	1.48	1.37	1.09	109
550	9133	6.11	2.85	1.92	1.57	1.39	1.28	1.05	109
600	9034	6.22	2.83	1.89	1.53	1.37	1.29	1.02	109
650	9010	6.50	3.08	2.16	1.76	1.50	1.44	1.15	109
700	9034	7.45	3.29	2.03	1.61	1.46	1.37	1.08	109
750	9030	7.79	3.62	2.34	1.88	1.70	1.58	1.29	115
800	9010	7.76	3.46	2.17	1.74	1.58	1.47	1.17	115
850	9049	6.37	2.94	2.02	1.66	1.50	1.42	1.15	115
900	8946	6.40	3.00	2.20	1.83	1.69	1.57	1.24	115
950	8986	6.58	3.05	2.07	1.74	1.59	1.52	1.21	115
1000	9065	6.36	2.89	1.89	1.62	1.43	1.36	1.11	112
1050	9026	6.75	3.13	2.11	1.70	1.50	1.43	1.16	112
1106	9077	6.47	3.10	2.12	1.74	1.54	1.42	1.08	112
1150	9010	6.74	3.28	2.35	1.91	1.69	1.55	1.24	112
1200	9030	6.70	3.20	2.23	1.87	1.67	1.56	1.25	112
1257	8990	6.93	3.52	2.39	1.91	1.72	1.57	1.22	114
1300	8934	7.35	3.60	2.50	2.05	1.85	1.70	1.36	114
1350	9022	5.62	3.00	2.26	1.91	1.72	1.60	1.28	114
1400	8998	6.36	3.04	1.96	1.64	1.50	1.39	1.09	114
1450	8966	6.12	3.11	2.23	1.87	1.67	1.54	1.21	114
1500	9034	5.51	2.77	2.11	1.77	1.62	1.50	1.19	116
1550	9057	5.47	2.73	2.09	1.76	1.59	1.49	1.15	116
1600	8990	6.07	2.98	2.05	1.70	1.55	1.45	1.11	116
1650	8938	5.83	3.04	2.23	1.83	1.63	1.48	1.15	116
1700	8950	6.95	3.39	2.26	1.85	1.63	1.51	1.18	116
1750	8986	6.41	3.09	2.08	1.69	1.46	1.37	1.06	118
1800	8994	6.11	2.72	1.89	1.61	1.46	1.37	1.08	118
1850	8970	6.39	3.36	2.48	1.99	1.75	1.61	1.26	118
1900	8851	7.02	3.68	2.63	2.11	1.82	1.65	1.30	118
1950	8906	7.43	3.79	2.69	2.11	1.82	1.67	1.31	118
2000	8942	8.66	4.44	2.92	2.21	1.89	1.73	1.33	119

Table D10. FWD Deflections on L2 Lane of SH19 Project at Trinity (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
2050	8815	8.71	4.61	3.03	2.26	1.91	1.71	1.35	119
2100	8835	8.84	4.63	2.98	2.19	1.79	1.57	1.20	119
2150	8779	9.91	5.21	3.27	2.35	1.94	1.74	1.33	119
2200	8926	9.62	5.09	3.28	2.48	2.10	1.87	1.46	119
2250	8855	9.00	4.80	3.13	2.44	2.11	1.91	1.47	124
2300	8875	8.24	4.47	2.96	2.27	1.97	1.83	1.46	124
2350	8938	7.63	3.95	2.90	2.36	2.05	1.87	1.46	124
2400	8946	7.38	3.50	2.66	2.24	1.96	1.80	1.39	124

Table D11. FWD Deflections on R1 Lane of SH19 Project at Trinity.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9049	9.12	4.84	3.11	2.27	1.89	1.70	1.24	133
50	9014	9.28	4.83	3.16	2.38	1.98	1.72	1.30	133
100	8966	10.93	5.22	3.28	2.46	2.02	1.77	1.31	133
150	9125	7.92	3.89	2.69	2.07	1.71	1.54	1.16	133
200	9026	8.15	3.93	2.67	2.07	1.71	1.52	1.15	133
250	9093	6.16	2.73	2.00	1.61	1.38	1.26	1.00	136
300	9232	6.29	2.69	1.90	1.56	1.41	1.29	1.01	136
352	9097	8.80	3.82	2.09	1.63	1.42	1.32	0.98	136
400	9085	7.94	3.28	1.94	1.62	1.44	1.36	1.07	136
450	8875	10.08	4.45	2.37	1.80	1.56	1.43	1.11	136
500	8990	7.49	3.45	2.34	1.85	1.59	1.46	1.12	134
550	8660	15.22	7.07	3.32	2.36	1.95	1.69	1.31	134
600	9022	11.13	4.94	2.44	1.86	1.59	1.43	1.10	134
650	8918	9.46	4.02	2.53	2.02	1.76	1.61	1.26	134
701	9026	7.98	3.51	2.29	1.84	1.59	1.46	1.11	134
750	9018	8.63	4.00	2.55	2.04	1.80	1.65	1.28	130
800	9069	7.36	3.49	2.37	1.85	1.60	1.46	1.16	130
850	9121	7.63	3.08	1.80	1.52	1.37	1.29	1.02	130
900	9093	6.83	2.74	1.66	1.44	1.35	1.31	1.07	130
950	8815	9.71	3.72	1.82	1.39	1.30	1.24	1.01	130
1000	8930	6.24	2.54	1.81	1.48	1.32	1.24	0.96	127
1052	8994	7.17	3.08	1.95	1.61	1.47	1.40	1.12	127
1100	9053	7.13	2.84	1.65	1.28	1.19	1.15	0.94	127
1150	8986	6.87	3.03	1.64	1.35	1.27	1.22	1.02	127
1200	8895	7.37	3.02	2.05	1.72	1.56	1.45	1.11	127
1251	8740	9.44	4.18	2.57	2.13	1.87	1.74	1.33	126
1301	8823	8.39	4.24	3.13	2.44	2.09	1.90	1.40	126
1350	8771	11.24	5.08	3.09	2.35	1.96	1.75	1.30	126
1400	9113	5.54	2.31	1.66	1.33	1.17	1.08	0.84	126
1450	8783	9.63	4.99	3.49	2.71	2.28	2.04	1.51	126
1500	8712	10.00	5.01	3.59	2.72	2.28	2.00	1.51	123
1550	8978	6.59	3.38	2.72	2.18	1.87	1.65	1.26	123
1600	8891	8.27	3.52	2.59	2.03	1.72	1.51	1.15	123
1650	8859	7.31	3.10	2.05	1.66	1.45	1.37	1.09	123
1700	8847	8.46	3.60	2.15	1.71	1.51	1.36	1.09	123
1750	8970	8.05	3.30	2.17	1.78	1.57	1.48	1.17	118
1800	8946	7.42	3.28	2.22	1.76	1.49	1.39	1.10	118
1850	8990	7.64	3.61	2.43	1.92	1.66	1.53	1.23	118
1900	8914	9.26	3.72	2.07	1.59	1.44	1.31	1.05	118
1950	8914	5.39	2.31	1.69	1.49	1.36	1.32	1.05	118
2000	9121	5.62	2.41	1.66	1.44	1.31	1.28	1.02	114

Table D11. FWD Deflections on R1 Lane of SH19 Project at Trinity (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
2050	8946	6.35	2.65	1.63	1.39	1.32	1.26	1.03	114
2100	9041	5.71	2.31	1.85	1.65	1.51	1.48	1.21	114
2150	8863	7.06	3.89	2.81	2.21	1.87	1.66	1.26	114
2200	8751	10.22	5.13	3.13	2.24	1.82	1.56	1.23	114
2250	8799	8.27	3.62	2.37	1.84	1.55	1.41	1.07	N/A (rain)
2300	8652	7.93	3.61	2.58	2.00	1.74	1.56	1.17	N/A (rain)
2350	8676	6.64	3.06	2.28	1.87	1.64	1.47	1.19	N/A (rain)
2400	8692	7.37	2.94	2.15	1.75	1.56	1.43	1.09	N/A (rain)
2500	8616	7.97	4.14	2.36	1.62	1.44	1.35	1.04	N/A (rain)
2550	8751	6.02	2.90	2.09	1.68	1.47	1.36	1.07	N/A (rain)

Table D12. FWD Deflections on R2 Lane of SH19 Project at Trinity.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9006	8.00	4.23	2.81	2.09	1.75	1.59	1.17	100
50	9014	7.67	3.98	2.96	2.25	1.92	1.72	1.27	100
154	9145	6.22	3.41	2.45	1.91	1.65	1.48	1.13	100
200	9200	6.07	3.40	2.43	1.86	1.59	1.42	1.09	100
250	9129	5.59	2.53	1.89	1.58	1.37	1.27	1.00	104
300	9157	5.00	2.24	1.70	1.40	1.26	1.19	0.96	104
354	9097	6.15	2.60	1.81	1.53	1.34	1.22	1.02	104
400	9153	5.73	2.44	1.81	1.51	1.37	1.31	1.05	104
450	9149	5.66	2.56	2.00	1.67	1.50	1.40	1.10	104
501	8950	5.95	2.81	2.02	1.72	1.56	1.47	1.15	104
550	8994	5.95	2.94	2.02	1.62	1.43	1.33	0.99	104
600	8970	6.83	3.39	2.27	1.82	1.59	1.48	1.14	104
650	9053	7.83	3.79	2.46	1.94	1.65	1.50	1.20	104
702	9121	5.62	2.60	1.92	1.63	1.48	1.39	1.10	104
750	9049	5.46	2.84	2.07	1.64	1.44	1.34	1.09	110
800	9041	5.60	2.57	1.87	1.57	1.38	1.11	0.85	110
851	9081	5.51	2.36	1.72	1.50	1.37	1.30	1.06	110
900	9125	6.15	2.65	1.85	1.54	1.38	1.30	1.01	110
951	9109	5.43	2.31	1.81	1.52	1.35	1.26	0.98	110
1000	9125	5.19	2.66	2.01	1.68	1.43	1.31	1.11	104
1050	8982	5.04	2.69	1.99	1.59	1.41	1.31	1.02	104
1100	9002	5.85	3.16	2.19	1.71	1.44	1.33	1.02	104
1151	9069	7.01	3.70	2.57	2.03	1.76	1.61	1.25	104
1200	8986	7.45	4.24	3.12	2.41	2.05	1.86	1.41	104
1261	8966	5.31	2.95	2.38	1.95	1.73	1.61	1.24	105
1300	9022	6.50	3.92	2.83	2.24	1.91	1.69	1.33	105
1350	8966	6.50	3.98	2.95	2.21	1.86	1.69	1.28	105
1402	8962	6.52	3.83	2.75	2.13	1.85	1.67	1.30	105
1451	8970	6.12	3.37	2.58	1.98	1.74	1.59	1.24	105
1500	8910	6.37	3.34	2.41	1.86	1.59	1.43	1.12	102
1550	9049	5.28	2.67	1.93	1.54	1.37	1.26	1.01	102
1600	8906	5.28	2.76	2.02	1.63	1.41	1.29	1.02	102
1652	8990	5.04	2.50	1.83	1.50	1.36	1.26	1.01	102
1709	8910	5.74	2.93	2.13	1.72	1.48	1.37	1.11	102
1750	8867	6.76	3.31	2.42	1.91	1.62	1.47	1.15	108
1802	9034	5.31	2.82	2.13	1.69	1.48	1.34	1.07	108
1854	9065	5.87	3.02	2.31	1.83	1.61	1.48	1.13	108
1901	9105	4.40	2.22	1.70	1.38	1.26	1.18	0.94	108
1952	9053	5.16	2.30	1.68	1.44	1.36	1.28	1.04	108
2000	9022	6.02	2.78	1.86	1.53	1.40	1.30	1.06	107
2050	8998	5.24	2.69	1.93	1.63	1.44	1.38	1.15	107

Table D12. FWD Deflections on R2 Lane of SH19 Project at Trinity (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
2100	9121	4.03	2.06	1.76	1.58	1.44	1.38	1.12	101
2160	9077	5.21	2.50	1.93	1.62	1.48	1.39	1.09	107
2200	8990	5.99	2.95	2.09	1.65	1.44	1.32	1.07	107
2254	8938	6.36	3.39	2.18	1.62	1.30	1.18	0.93	109
2300	9022	6.46	2.87	1.92	1.54	1.34	1.26	0.99	109
2351	9049	5.56	2.75	1.89	1.59	1.34	1.27	1.04	109
2400	8978	5.98	2.99	2.06	1.70	1.48	1.35	1.12	109
2450	8910	7.07	3.81	2.54	1.94	1.63	1.44	1.12	109
2505	8898	6.76	3.48	2.48	1.91	1.67	1.47	1.15	107
2550	9002	7.83	3.91	2.39	1.79	1.55	1.40	1.13	107
2600	8819	8.18	4.53	2.83	2.04	1.74	1.59	1.21	107

Table D13. Backcalculated Moduli on L1 Lane of SH19 Project at Trinity.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	327.9	143.3	120.0	38.6	6.01
55	250.0	82.9	120.0	36.3	7.35
100	289.3	77.2	120.0	27.2	3.16
157	278.5	62.7	120.0	26.7	4.32
200	309.1	70.6	120.0	27.1	5.11
250	288.7	68.6	120.0	27.1	5.57
350	252.8	149.2	120.0	28.3	5.89
400	250.0	140.3	120.0	33.8	7.40
450	250.0	135.1	120.0	35.0	6.99
502	250.0	189.3	120.0	47.8	12.02
601	250.0	177.5	120.0	39.8	11.55
650	250.0	113.2	120.0	35.0	8.89
700	257.2	96.4	120.0	41.2	6.35
750	250.0	87.3	120.0	35.7	5.69
800	250.0	70.4	120.0	35.5	6.99
850	250.0	115.7	120.0	32.8	6.56
950	250.0	124.3	120.0	33.8	7.93
1000	250.0	164.5	120.0	38.0	10.66
1050	250.0	130.2	120.0	38.1	10.87
1100	250.0	140.6	120.0	32.5	8.15
1150	250.0	168.1	120.0	30.7	11.98
1200	353.4	200.0	120.0	35.5	13.20
1250	250.0	199.6	120.0	35.0	10.21
1300	250.0	189.7	120.0	39.2	12.21
1350	250.0	131.3	120.0	31.7	9.77
1400	250.0	132.6	120.0	33.8	8.91
1450	250.0	170.6	120.0	33.3	9.27
1500	444.7	200.0	120.0	34.9	10.09
1550	514.6	200.0	120.0	39.9	11.97
1600	451.5	200.0	120.0	38.0	10.67
1650	250.0	113.4	120.0	35.5	6.26
1700	250.0	171.6	120.0	39.2	9.80
1750	255.3	102.9	120.0	36.5	5.01
1803	250.0	200.0	120.0	38.6	8.95
1850	265.6	60.8	120.0	27.6	3.28
1900	250.0	76.8	120.0	28.4	3.62
2000	474.2	63.3	107.2	26.7	2.39
2054	382.2	39.1	111.5	23.4	2.68

Table D13. Backcalculated Moduli on L1 Lane of SH19 Project at Trinity (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
2100	460.2	36.4	76.6	27.9	2.94
2150	494.5	41.5	88.5	24.6	2.23
2202	450.5	42.8	105.4	29.5	2.47
2255	380.3	36.7	51.6	24.8	2.53
2303	449.1	63.4	120.0	24.7	1.87
2350	290.9	200.0	120.0	27.4	5.38
2400	386.3	72.3	120.0	24.1	1.82

Table D14. Backcalculated Moduli on L2 Lane of SH19 Project at Trinity.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	204.0	220.0	120.0	43.6	8.69
50	190.0	207.8	120.0	36.4	8.38
103	207.2	155.7	120.0	29.8	5.07
150	197.9	97.4	120.0	28.5	3.72
203	251.6	119.8	120.0	28.0	4.66
250	251.1	104.7	120.0	29.3	4.91
302	256.1	154.2	120.0	32.7	5.79
350	296.9	220.0	120.0	34.0	8.08
401	262.0	218.1	120.0	32.8	6.57
450	235.0	220.0	120.0	31.0	5.94
500	221.8	220.0	120.0	39.1	7.44
550	269.6	220.0	120.0	42.5	8.38
600	240.8	220.0	120.0	43.0	8.87
650	243.6	220.0	120.0	37.0	7.27
700	172.8	174.5	120.0	39.7	7.98
750	170.4	183.2	120.0	33.6	7.36
800	166.8	168.1	120.0	36.8	7.76
850	247.6	220.0	120.0	39.4	9.42
900	267.1	220.0	120.0	35.3	10.04
950	227.7	220.0	120.0	37.6	9.86
1000	233.7	220.0	120.0	41.8	9.40
1050	207.0	220.0	120.0	38.2	7.79
1106	246.2	220.0	120.0	37.8	7.77
1150	238.5	220.0	120.0	33.8	7.28
1200	235.6	220.0	120.0	35.0	8.54
1257	218.1	220.0	120.0	33.1	6.52
1300	193.7	220.0	120.0	30.8	7.39
1350	499.1	220.0	120.0	33.0	9.14
1400	238.1	220.0	120.0	39.9	9.02
1450	320.0	220.0	120.0	34.5	8.26
1500	501.9	220.0	120.0	35.6	10.60
1550	453.1	220.0	120.0	37.9	10.57
1600	295.2	220.0	120.0	38.1	9.19
1650	376.0	220.0	120.0	34.7	7.64
1700	204.0	220.0	120.0	34.6	6.86
1750	239.3	220.0	120.0	38.6	6.91
1800	263.0	220.0	120.0	41.8	10.62
1850	305.0	220.0	120.0	31.6	6.44
1900	224.1	220.0	120.0	29.7	4.80
1950	199.6	208.8	120.0	29.4	4.66
2000	246.4	96.6	120.0	27.8	3.50
2050	290.8	80.2	120.0	26.7	2.46

Table D14. Backcalculated Moduli on L2 Lane of SH19 Project at Trinity (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
2100	316.9	67.9	120.0	28.3	1.81
2150	321.7	50.2	120.0	25.7	2.50
2200	279.3	67.6	120.0	24.7	2.58
2250	261.8	87.5	120.0	24.6	3.84
2300	291.3	100.9	120.0	26.2	4.12
2350	199.6	220.0	120.0	26.9	5.28
2400	207.3	220.0	120.0	29.2	7.65

Table D15. Backcalculated Moduli on R1 Lane of SH19 Project at Trinity.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	300.7	88.1	84.3	28.1	2.87
50	250.0	100.0	81.6	27.2	2.83
100	250.0	62.9	79.8	26.6	3.54
150	250.0	135.9	100.0	33.3	4.28
200	250.0	126.4	96.4	32.1	4.26
250	299.8	200.0	100.0	44.5	9.97
300	293.6	200.0	100.0	45.3	11.54
352	250.0	81.1	100.0	40.7	7.60
400	250.0	111.5	100.0	42.8	10.69
450	250.0	56.0	100.0	35.4	6.31
500	250.0	154.0	100.0	35.4	7.21
550	250.0	50.0	25.0	27.0	9.38
600	250.0	50.0	84.9	34.8	6.29
650	250.0	80.1	97.5	32.5	8.38
701	250.0	126.8	100.0	35.8	8.16
750	250.0	105.9	96.8	32.3	7.68
800	250.0	167.7	100.0	35.2	6.78
850	250.0	122.9	100.0	45.6	11.70
900	250.0	177.5	100.0	46.7	14.59
950	250.0	53.0	100.0	45.1	11.04
1000	270.6	200.0	100.0	45.3	11.90
1052	250.0	158.5	100.0	41.0	11.37
1100	250.0	133.8	100.0	50.8	12.09
1150	250.0	144.8	100.0	48.6	11.67
1200	250.0	156.4	100.0	38.5	11.91
1251	250.0	83.1	90.2	30.1	9.21
1301	250.0	154.3	78.3	26.1	6.25
1350	250.0	54.1	81.9	27.3	5.28
1400	402.8	200.0	100.0	51.6	11.84
1450	250.0	109.1	70.1	23.4	4.92
1500	250.0	96.0	69.5	23.2	4.54
1550	408.6	200.0	91.7	30.6	7.45
1600	250.0	118.2	100.0	33.3	7.39
1650	250.0	147.2	100.0	39.6	9.71
1700	250.0	91.8	100.0	38.3	8.38
1750	250.0	120.6	100.0	37.5	10.51
1800	250.0	148.6	100.0	37.7	7.58
1850	250.0	149.7	100.0	33.7	7.02
1900	250.0	67.7	100.0	41.3	9.14
1950	424.0	200.0	100.0	49.4	16.81
2000	394.5	200.0	100.0	49.7	15.06
2050	250.0	200.0	100.0	47.3	13.96

Table D15. Backcalculated Moduli on R1 Lane of SH19 Project at Trinity (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
2100	379.7	200.0	100.0	47.2	18.86
2150	305.5	200.0	86.6	28.9	4.71
2200	274.3	59.2	84.6	28.2	1.44
2250	250.0	104.3	100.0	35.3	6.43
2300	250.0	129.6	95.0	31.7	7.07
2350	257.7	200.0	100.0	35.0	8.86
2400	250.0	151.7	100.0	37.5	11.21
2500	403.8	68.0	100.0	35.8	5.46
2550	341.6	200.0	100.0	38.5	8.61

Table D16. Backcalculated Moduli on R2 Lane of SH19 Project at Trinity.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	302.6	100.4	120.0	30.2	2.79
50	194.2	220.0	120.0	28.7	3.32
154	336.6	220.0	120.0	34.3	4.83
200	363.5	220.0	120.0	35.2	3.99
250	362.1	220.0	120.0	45.8	10.87
300	490.9	220.0	120.0	51.2	12.74
354	246.0	220.0	120.0	47.5	10.57
400	354.2	220.0	120.0	45.5	12.84
450	397.7	220.0	120.0	42.0	11.87
501	329.4	220.0	120.0	39.1	10.83
550	304.4	220.0	120.0	41.3	7.83
600	219.0	220.0	120.0	36.3	6.49
650	206.8	147.4	120.0	34.3	4.73
702	386.4	220.0	120.0	43.1	11.93
750	439.2	220.0	120.0	40.4	7.68
800	346.9	220.0	120.0	45.8	8.99
851	362.5	220.0	120.0	48.1	14.22
900	250.5	220.0	120.0	47.0	10.92
951	407.9	220.0	120.0	46.8	13.00
1000	606.8	220.0	120.0	40.6	8.73
1050	632.2	220.0	120.0	40.6	8.60
1100	346.7	220.0	120.0	38.3	4.81
1151	235.3	220.0	120.0	32.2	5.07
1200	241.3	209.2	120.0	26.0	2.80
1261	835.7	220.0	120.0	31.9	8.69
1300	366.5	219.2	120.0	28.1	2.93
1350	425.7	185.9	120.0	27.9	2.61
1402	334.7	216.3	120.0	29.1	3.77
1451	372.8	220.0	120.0	32.2	5.50
1500	277.1	220.0	120.0	35.0	4.54
1550	442.2	220.0	120.0	43.7	8.50
1600	479.7	220.0	120.0	40.4	7.42
1652	508.8	220.0	120.0	45.3	10.30
1709	360.7	220.0	120.0	38.7	7.05
1750	232.3	220.0	120.0	34.4	5.39
1802	546.4	220.0	120.0	38.8	7.42
1854	386.8	220.0	120.0	36.3	7.39
1901	900.0	220.0	96.4	51.1	12.97
1952	442.7	220.0	120.0	49.0	14.35
2000	279.3	220.0	120.0	44.1	10.15
2050	508.7	220.0	120.0	41.3	9.94

Table D16. Backcalculated Moduli on R2 Lane of SH19 Project at Trinity (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
2100	900.0	220.0	120.0	51.9	19.24
2160	512.6	220.0	120.0	43.0	12.31
2200	308.2	220.0	120.0	40.2	7.17
2254	406.4	121.0	120.0	39.7	2.48
2300	213.7	220.0	120.0	44.9	7.92
2351	391.7	220.0	120.0	42.6	8.30
2400	320.2	220.0	120.0	39.1	7.65
2450	318.8	131.5	120.0	32.8	2.94
2505	239.9	220.0	120.0	33.5	4.82
2550	290.2	94.9	120.0	35.4	4.50
2600	412.2	69.8	120.0	29.4	3.23

Table D17. FWD Deflections on L1 Lane of I-10 WIM Section.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	10058	14.75	8.69	3.67	2.04	1.44	1.12	0.92	79
11	9900	15.96	8.97	3.79	2.07	1.45	1.13	0.99	79
20	9796	16.68	9.18	3.64	1.92	1.36	1.07	0.89	79
30	9864	11.85	6.80	2.99	1.67	1.23	0.98	0.86	79
40	9931	9.52	5.73	2.87	1.74	1.31	1.02	0.86	79
51	9979	9.01	5.79	2.78	1.68	1.24	0.99	0.87	88
61	9788	12.89	8.20	3.61	2.01	1.45	1.13	1.00	88
70	9900	10.52	7.27	4.05	2.60	1.85	1.44	1.22	88
81	9911	11.43	8.20	4.62	2.83	2.07	1.59	1.23	88
92	9908	10.43	7.79	4.54	2.83	1.97	1.52	1.24	88
100	9935	8.04	6.54	3.94	2.67	1.89	1.44	1.15	86
110	9975	7.39	5.69	3.52	2.38	1.81	1.40	1.14	86
120	9923	8.33	6.35	4.04	2.78	2.10	1.61	1.33	86
130	9975	7.96	6.41	4.09	2.87	2.12	1.57	1.31	86
141	9872	10.20	7.23	4.22	2.74	2.04	1.57	1.25	86
151	9911	9.72	7.30	4.28	2.90	2.16	1.67	1.37	87
161	9864	11.63	8.15	4.71	3.10	2.30	1.74	1.40	87
172	9852	11.83	8.17	5.13	3.42	2.44	1.78	1.40	87
180	9959	9.67	7.18	4.66	3.19	2.30	1.69	1.30	87
190	9915	9.92	7.76	4.97	3.28	2.25	1.64	1.35	87
200	9892	9.02	7.40	4.74	3.19	2.36	1.77	1.35	87
211	9971	9.02	7.20	4.83	3.28	2.48	1.86	1.49	87
221	9876	9.74	7.97	5.27	3.57	2.55	1.87	1.54	87
231	9840	8.77	7.40	4.93	3.42	2.46	1.84	1.46	87
243	9908	8.18	6.53	4.42	3.06	2.29	1.75	1.38	87
250	9804	6.99	5.33	3.69	2.65	2.04	1.60	1.32	90
261	9975	6.15	4.70	3.30	2.43	1.87	1.46	1.20	90
269	9967	6.50	5.87	3.57	2.48	1.91	1.50	1.22	90
280	9860	9.59	6.56	4.04	2.71	2.08	1.61	1.33	90
290	9721	12.05	8.42	4.71	3.07	2.28	1.77	1.43	90
300	9717	12.05	9.15	5.71	3.86	2.84	2.23	1.85	93
311	9753	11.26	8.16	5.13	3.50	2.74	2.17	1.69	93
320	9681	12.75	9.28	5.40	3.60	2.71	2.06	1.62	93
329	9598	15.12	11.25	5.98	3.75	2.75	2.08	1.69	93
341	9701	12.13	9.64	5.73	3.85	2.82	2.14	1.71	93
350	9637	16.66	11.93	6.80	4.36	3.16	2.41	1.91	91
360	9649	16.39	11.85	6.80	4.37	3.20	2.45	1.98	91
370	9729	13.51	10.11	6.04	3.98	2.98	2.28	1.78	91
380	9673	12.45	9.76	6.00	4.05	2.93	2.19	1.70	91
391	9725	12.31	9.47	6.02	4.12	3.03	2.30	1.81	91
400	9776	12.71	9.97	6.46	4.30	3.07	2.28	1.81	92
410	9729	11.78	9.32	5.89	3.94	2.86	2.15	1.72	92
420	9701	13.42	9.57	5.62	3.71	2.69	2.06	1.65	92
429	9753	11.32	8.69	5.25	3.59	2.65	2.01	1.65	92

Table D17. FWD Deflections on L1 Lane of I-10 WIM Section (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
441	9753	12.41	9.11	5.55	3.74	2.79	2.14	1.70	92
450	9717	11.62	8.97	5.64	3.89	2.91	2.28	1.83	97
460	9737	12.78	9.57	6.02	4.21	3.14	2.38	1.90	97
470	9681	14.51	10.76	6.30	4.18	3.06	2.34	1.91	97
481	9721	15.19	11.26	6.42	4.22	3.00	2.22	1.73	97
490	9693	13.72	9.35	5.37	3.47	2.51	1.87	1.52	97
500	9764	11.59	8.46	5.11	3.44	2.50	1.93	1.54	95

Table D18. FWD Deflections on L2 Lane of I-10 WIM Section.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	10221	5.65	3.72	2.34	1.71	1.27	1.00	0.85	102
10	10142	5.21	3.35	2.17	1.57	1.22	0.97	0.81	102
21	10058	5.36	3.38	2.02	1.41	1.11	0.90	0.75	102
30	10015	5.92	3.66	2.07	1.43	1.09	0.87	0.75	102
40	9983	5.11	3.20	2.07	1.51	1.20	0.93	0.74	102
51	10019	5.23	3.31	2.08	1.53	1.20	0.94	0.77	103
61	9911	5.81	3.78	2.29	1.62	1.24	0.96	0.78	103
70	9911	6.39	4.09	2.37	1.74	1.28	0.99	0.81	103
80	9888	6.56	4.35	2.67	1.87	1.42	1.07	0.85	103
90	9880	6.22	4.19	2.56	1.80	1.36	1.04	0.82	103
100	9911	5.41	3.81	2.48	1.80	1.35	1.04	0.81	104
110	9915	5.57	3.52	2.31	1.71	1.31	1.05	0.89	104
120	9915	5.22	3.69	2.43	1.81	1.41	1.12	0.92	104
130	9840	6.24	4.31	2.85	2.00	1.47	1.11	0.90	104
141	9900	4.98	3.29	2.16	1.60	1.20	0.95	0.73	104
150	9892	4.31	3.09	2.07	1.57	1.22	0.95	0.76	111
160	9896	4.76	3.09	2.04	1.52	1.19	0.97	0.78	111
169	9931	5.31	3.40	2.26	1.69	1.32	1.04	0.83	111
182	9856	4.93	3.61	2.28	1.69	1.31	1.03	0.87	111
190	9852	5.94	3.98	2.50	1.83	1.39	1.07	0.84	111
201	9836	5.62	3.94	2.60	1.85	1.39	1.07	0.85	102
210	9828	5.42	3.94	2.60	1.83	1.39	1.06	0.85	102
221	9820	4.94	3.55	2.40	1.78	1.39	1.11	0.91	102
231	9832	5.26	3.99	2.63	1.87	1.45	1.14	0.89	102
240	9856	5.59	3.76	2.39	1.74	1.38	1.10	0.93	102
251	9911	5.63	3.94	2.53	1.89	1.44	1.15	0.92	107
261	9911	5.26	3.65	2.32	1.68	1.31	1.01	0.80	107
271	9911	5.39	3.37	2.15	1.58	1.17	0.94	0.80	107
281	9908	5.00	3.28	2.03	1.48	1.17	0.91	0.74	107
290	9915	5.22	3.37	2.15	1.56	1.24	1.00	0.81	107
300	9900	5.46	3.52	2.30	1.74	1.38	1.11	0.92	113
311	9852	5.18	3.58	2.34	1.74	1.37	1.10	0.89	113
321	9856	5.31	3.80	2.52	1.87	1.45	1.14	0.92	113
330	9824	5.51	3.74	2.41	1.80	1.39	1.09	0.89	113
340	9828	5.90	4.19	2.60	1.96	1.55	1.25	1.02	113
350	9796	6.44	4.61	2.89	2.15	1.71	1.37	1.13	106
360	9844	5.81	4.06	2.63	1.94	1.54	1.26	1.10	106
370	9852	5.15	3.67	2.53	1.93	1.59	1.34	1.04	106
380	9824	5.94	4.35	3.00	2.30	1.87	1.50	1.23	106
391	9856	6.41	4.60	3.13	2.39	1.90	1.54	1.25	106
400	9868	6.19	4.43	2.89	2.13	1.71	1.38	1.11	102
411	9876	5.73	4.07	2.66	2.01	1.64	1.34	1.06	102
421	9828	6.03	4.14	2.67	2.07	1.61	1.29	1.06	102
431	9800	5.89	4.09	2.72	2.06	1.68	1.33	1.09	102

Table D18. FWD Deflections on L2 Lane of I-10 WIM Section (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9796	5.97	4.26	2.91	2.19	1.73	1.38	1.06	102
451	9828	5.73	4.10	2.89	2.26	1.83	1.46	1.21	106
460	9800	5.78	3.96	2.78	2.13	1.72	1.39	1.10	106
470	9816	5.34	3.81	2.72	2.11	1.74	1.43	1.22	106
481	9832	5.96	4.10	2.89	2.29	1.84	1.50	1.28	106
491	9868	5.76	4.09	3.01	2.33	1.88	1.50	1.28	106
501	9856	5.79	4.26	3.02	2.35	1.91	1.54	1.26	104

Table D19. FWD Deflections on R1 Lane of I-10 WIM Section.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	10337	7.86	5.15	3.11	2.21	1.70	1.32	1.02	89
11	10190	7.15	5.12	3.02	2.06	1.54	1.19	0.93	89
20	10146	7.27	4.86	2.67	1.81	1.39	1.07	0.86	89
30	10074	7.34	5.01	2.70	1.83	1.43	1.11	0.92	89
41	9963	8.89	6.16	3.44	2.20	1.64	1.25	1.01	89
49	9911	10.54	6.83	3.63	2.30	1.67	1.28	1.01	94
61	9796	9.13	7.38	3.86	2.38	1.81	1.35	1.10	94
70	9884	10.80	7.59	4.18	2.78	2.10	1.66	1.35	94
80	9741	13.06	10.05	5.71	3.56	2.54	1.89	1.54	94
90	9788	12.48	9.05	5.12	3.20	2.33	1.78	1.42	94
100	9884	9.39	7.07	4.31	3.05	2.31	1.80	1.46	88
110	9848	10.12	7.85	4.81	3.32	2.56	1.99	1.65	88
120	9792	9.80	7.53	4.85	3.40	2.58	1.99	1.53	88
130	9804	10.51	7.67	4.88	3.45	2.61	1.98	1.57	88
140	9757	10.93	8.26	5.05	3.42	2.53	1.93	1.56	88
151	9780	11.83	9.11	5.38	3.51	2.48	1.81	1.46	94
160	9828	9.47	8.02	5.19	3.61	2.63	1.94	1.49	94
171	9729	12.91	9.84	5.61	3.71	2.70	2.02	1.56	94
180	9792	10.56	7.81	4.96	3.48	2.57	1.96	1.55	94
190	9884	6.30	5.81	4.02	3.01	2.26	1.74	1.42	94
203	9844	7.02	5.58	3.98	2.98	2.32	1.83	1.50	92
210	9880	6.71	5.70	4.13	3.09	2.43	1.93	1.59	92
220	9872	6.95	5.86	4.01	2.92	2.34	1.79	1.46	92
231	9860	7.43	6.10	4.23	3.05	2.31	1.78	1.41	92
241	9705	10.70	8.00	4.84	3.31	2.47	1.83	1.53	92
250	9677	11.65	8.62	5.24	3.41	2.47	1.87	1.46	99
260	9800	10.49	7.91	5.01	3.42	2.49	1.87	1.50	99
270	9788	8.35	6.71	4.33	3.10	2.29	1.73	1.40	99
280	9844	7.37	5.83	3.89	2.81	2.16	1.66	1.39	99
290	9844	8.59	6.61	4.20	2.99	2.23	1.70	1.38	99
300	9884	7.56	5.93	3.97	2.89	2.23	1.71	1.32	94
310	9900	7.04	5.42	3.52	2.59	2.01	1.57	1.30	94
320	9939	7.70	5.39	3.63	2.73	2.15	1.73	1.44	94
330	9808	10.28	7.25	4.28	2.96	2.38	1.91	1.57	94
340	9757	10.39	8.66	5.14	3.72	2.88	2.33	1.96	94
350	9618	15.85	12.36	7.53	4.32	2.91	2.13	1.61	93
360	9757	11.87	8.67	5.23	3.67	2.82	2.21	1.78	93
370	9673	12.99	9.52	5.71	3.94	2.94	2.24	1.80	93
380	9653	14.94	10.80	6.35	4.16	2.97	2.20	1.72	93
390	9598	14.26	10.41	5.94	3.78	2.72	2.08	1.57	93
400	9649	13.60	10.00	5.70	3.72	2.75	2.10	1.60	92
411	9693	13.39	9.38	5.43	3.63	2.67	2.05	1.63	92
420	9665	12.29	8.48	5.07	3.46	2.60	1.97	1.65	92
431	9657	11.47	8.19	4.83	3.29	2.48	1.90	1.57	92

Table D19. FWD Deflections on R1 Lane of I-10 WIM Section (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9713	11.19	8.02	4.68	3.22	2.37	1.85	1.56	92
450	9669	10.82	8.35	5.12	3.52	2.55	1.94	1.68	98
461	9637	11.47	8.14	4.78	3.26	2.46	1.88	1.55	98
471	9645	11.38	8.15	4.69	3.15	2.35	1.83	1.53	98
481	9693	10.84	7.59	4.36	2.88	2.17	1.68	1.35	98
490	9764	9.61	6.97	4.33	3.04	2.30	1.82	1.36	98
500	9741	10.04	7.64	4.82	3.39	2.51	1.94	1.59	103

Table D20. FWD Deflections on R2 Lane of I-10 WIM Section.

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
0	9927	5.70	3.94	2.71	1.96	1.51	1.19	0.96	110
10	9784	6.61	4.64	3.04	2.19	1.68	1.31	1.08	110
21	9796	7.98	4.86	3.07	2.20	1.70	1.36	1.10	110
29	9800	6.18	4.74	3.22	2.33	1.81	1.42	1.17	110
41	9800	5.51	4.70	3.34	2.49	1.87	1.45	1.17	110
51	9804	5.82	4.86	3.47	2.49	1.95	1.49	1.20	105
60	9757	5.69	4.94	3.71	2.79	2.15	1.66	1.33	105
70	9713	6.31	5.57	4.15	3.15	2.44	1.89	1.52	105
81	9820	7.04	6.11	4.46	3.27	2.48	1.91	1.52	105
90	9685	8.44	7.11	4.81	3.41	2.53	1.92	1.51	105
110	9776	8.69	7.30	5.17	3.75	2.86	2.20	1.74	105
120	9673	8.44	7.76	5.13	3.63	2.74	2.09	1.69	105
130	9594	12.11	9.14	5.93	4.11	3.04	2.29	1.82	105
141	9594	13.08	9.35	5.96	4.09	3.07	2.33	1.83	105
151	9685	9.83	7.63	5.10	3.64	2.74	2.10	1.69	105
161	9657	11.81	8.68	5.69	4.00	2.97	2.24	1.77	107
170	9629	11.33	8.83	5.49	3.70	2.73	2.06	1.63	107
181	9633	9.35	7.26	4.55	3.10	2.28	1.76	1.43	107
190	9697	7.53	5.97	4.07	2.88	2.28	1.72	1.38	107
199	9757	6.82	5.94	4.16	3.07	2.37	1.86	1.53	107
210	9657	8.24	6.91	4.76	3.33	2.50	1.94	1.61	106
220	9705	9.19	7.38	5.23	3.75	2.84	2.19	1.78	106
231	9629	9.88	7.33	5.12	3.68	2.79	2.17	1.74	106
241	9776	6.89	6.38	4.58	3.36	2.57	1.99	1.61	106
250	9594	8.89	6.75	4.61	3.28	2.50	1.96	1.59	106
260	9689	8.27	6.41	4.41	3.21	2.46	1.89	1.52	111
270	9657	8.28	6.70	4.62	3.28	2.44	1.84	1.48	111
280	9713	8.44	6.66	4.70	3.42	2.55	1.94	1.57	111
289	9669	7.79	7.13	4.84	3.42	2.60	2.01	1.59	111
301	9673	8.79	7.39	4.88	3.47	2.61	1.98	1.59	111
310	9685	8.66	6.94	4.75	3.31	2.52	1.87	1.49	107
320	9717	7.26	5.86	4.07	2.92	2.24	1.74	1.43	107
330	9673	7.74	5.93	4.17	3.05	2.34	1.82	1.47	107
339	9669	8.19	6.60	4.56	3.34	2.51	1.94	1.60	107
350	9594	9.88	7.82	5.17	3.59	2.73	2.09	1.71	107
360	9602	10.46	7.93	5.17	3.64	2.68	2.06	1.69	110
370	9562	9.27	7.34	4.54	3.23	2.53	2.02	1.69	110
380	9510	10.12	7.56	4.66	3.21	2.50	1.94	1.60	110
390	9522	11.16	8.13	5.01	3.40	2.55	1.94	1.58	110
400	9582	10.26	7.81	5.04	3.44	2.57	2.00	1.64	110
410	9594	9.64	7.59	5.04	3.56	2.67	2.03	1.64	111
420	9618	9.89	7.72	5.21	3.69	2.77	2.11	1.70	111
430	9653	9.78	7.67	5.13	3.64	2.74	2.14	1.73	111

Table D20. FWD Deflections on R2 Lane of I-10 WIM Section (continued).

Station (ft)	Load (lb)	FWD Sensor Deflection (mils)							Pavement Temp. (°F)
		W_1	W_2	W_3	W_4	W_5	W_6	W_7	
440	9637	9.41	7.41	4.93	3.43	2.59	2.00	1.61	111
450	9633	8.76	6.78	4.49	3.18	2.48	1.93	1.58	111
460	9653	9.22	7.35	4.89	3.49	2.61	1.99	1.61	112
471	9614	10.46	8.01	5.05	3.44	2.54	1.93	1.50	112
480	9586	10.09	7.45	4.79	3.36	2.52	1.90	1.53	112
490	9701	9.11	6.80	4.47	3.24	2.43	1.88	1.50	112
500	9697	8.50	6.51	4.48	3.22	2.46	1.85	1.49	112

Table D21. Backcalculated Moduli on L1 Lane of I-10 WIM Section.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	151.1	100.0	15.0	31.1	7.54
11	110.0	100.0	15.0	29.8	7.56
20	110.0	100.0	15.0	29.9	12.03
30	248.6	112.1	15.0	37.4	7.46
40	593.4	108.5	25.9	36.2	5.70
51	766.3	114.8	21.1	38.2	6.94
61	248.8	100.0	15.0	30.6	7.53
70	973.4	100.0	20.0	25.1	3.83
81	915.9	100.0	15.0	22.8	4.49
92	1000.0	112.3	15.0	23.8	3.92
100	1000.0	246.2	16.9	25.4	4.34
110	1000.0	237.9	38.3	26.4	4.29
120	1000.0	214.8	32.6	22.7	3.48
130	1000.0	303.2	17.9	23.5	3.19
141	1000.0	102.0	28.2	23.0	4.01
151	1000.0	144.3	21.2	22.3	4.86
161	842.6	100.0	20.6	20.6	3.96
172	927.0	100.0	20.9	19.2	1.17
180	1000.0	193.9	15.2	21.4	2.05
190	1000.0	166.7	15.3	20.8	2.89
200	1000.0	237.2	15.0	20.7	3.51
211	1000.0	239.0	22.3	19.5	2.61
221	1000.0	210.2	15.0	18.7	2.92
231	1000.0	270.1	15.2	19.5	2.95
243	1000.0	297.5	23.0	21.0	2.78
250	1000.0	311.3	70.3	22.9	2.50
261	1000.0	390.6	105.8	25.4	2.19
269	1000.0	365.4	36.5	25.4	6.07
280	1000.0	100.0	62.8	22.4	3.09
290	698.9	100.0	18.3	20.5	4.91
300	1000.0	118.2	16.1	16.3	3.61
311	990.6	100.0	44.4	16.9	3.91
320	773.7	100.0	17.0	17.3	4.54
329	329.4	100.0	15.0	16.2	5.63
341	1000.0	106.7	15.0	16.5	4.16
350	277.4	100.0	15.0	14.2	4.44
360	312.9	100.0	15.0	14.1	4.50
370	785.4	100.0	15.0	15.7	4.12
380	1000.0	107.3	15.0	15.7	2.96
391	1000.0	121.6	15.0	15.5	2.97
400	1000.0	114.3	15.0	14.9	2.36

Table D21. Backcalculated Moduli on L1 Lane of I-10 WIM Section (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
410	1000.0	126.6	15.0	16.2	2.95
420	651.2	100.0	15.0	17.2	3.60
429	1000.0	126.9	16.6	17.8	3.87
441	973.8	100.0	18.9	16.8	3.64
450	1000.0	135.5	16.9	16.1	3.90
460	1000.0	106.7	18.3	15.0	3.23
470	558.6	100.0	15.0	15.0	3.96
481	420.2	100.0	15.0	15.1	3.66
490	484.2	100.0	15.0	18.6	3.40
500	1000.0	106.7	18.1	18.6	3.42

Table D22. Backcalculated Moduli on L2 Lane of I-10 WIM Section.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	950.0	174.5	122.0	47.4	1.34
10	950.0	200.3	150.0	50.1	1.21
21	950.0	156.6	121.1	54.3	2.61
30	950.0	123.7	78.7	54.7	2.21
40	915.8	204.1	150.0	51.4	1.51
51	950.0	181.9	150.0	51.0	1.65
61	950.0	176.1	61.0	48.6	1.84
70	950.0	128.0	66.4	46.2	1.87
80	950.0	165.1	40.0	42.6	1.30
90	950.0	184.6	42.3	44.2	1.56
100	950.0	212.3	114.2	43.4	1.95
110	852.6	185.3	150.0	45.6	0.84
120	950.0	257.2	132.0	42.4	1.71
130	950.0	177.0	70.2	39.4	2.25
141	950.0	242.9	125.0	49.3	0.78
150	950.0	517.2	81.0	50.3	2.06
160	950.0	264.5	150.0	50.8	1.94
169	847.9	225.3	150.0	46.1	1.21
182	950.0	290.9	102.2	45.6	2.52
190	950.0	160.6	104.9	42.5	1.43
201	950.0	214.0	80.9	42.3	1.44
210	950.0	392.9	24.7	44.7	1.84
221	950.0	409.0	72.4	43.2	2.00
231	950.0	510.2	19.4	44.0	3.24
240	950.0	187.5	123.8	43.4	2.25
251	950.0	227.1	93.8	41.4	1.69
261	950.0	214.0	124.3	45.7	2.02
271	950.0	166.4	139.3	49.8	0.77
281	950.0	195.8	137.3	51.8	2.33
290	950.0	191.2	150.0	48.7	2.21
300	899.2	205.4	150.0	44.3	2.16
311	950.0	267.6	113.9	43.8	2.23
321	950.0	352.9	54.7	42.1	2.11
330	950.0	211.3	105.7	43.0	1.58
340	950.0	168.5	150.0	38.4	2.86
350	950.0	189.8	76.0	35.3	3.17
360	950.0	189.2	145.0	38.5	2.38
370	950.0	341.0	150.0	38.2	3.36
380	950.0	265.2	138.1	32.5	2.27
391	950.0	209.4	130.0	31.7	1.83
400	950.0	174.7	142.5	35.0	2.47

Table D22. Backcalculated Moduli on L2 Lane of I-10 WIM Section (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
411	950.0	223.7	148.4	37.0	3.20
421	950.0	173.6	150.0	37.2	1.94
431	950.0	205.7	150.0	36.2	2.35
440	950.0	257.0	94.3	34.7	1.72
451	950.0	296.4	150.0	33.3	1.76
460	902.3	257.7	150.0	35.2	1.95
470	950.0	359.7	150.0	35.2	2.60
481	856.5	278.3	150.0	33.1	2.33
491	950.0	324.0	150.0	32.4	1.19
501	950.0	318.9	146.3	31.9	1.97

Table D23. Backcalculated Moduli on R1 Lane of I-10 WIM Section.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	1000.0	123.7	124.0	29.1	3.35
11	1000.0	211.1	41.8	32.0	4.50
20	1000.0	142.9	66.3	35.2	5.20
30	1000.0	139.5	64.8	34.2	6.15
41	1000.0	100.0	47.6	28.2	3.79
49	717.0	100.0	22.3	28.1	4.41
61	1000.0	135.5	15.0	27.0	7.44
70	898.5	100.0	25.2	22.6	5.62
80	668.1	100.0	15.0	17.6	4.19
90	713.4	100.0	15.0	19.9	4.29
100	1000.0	175.0	24.8	21.0	4.72
110	1000.0	163.5	21.3	18.8	5.01
120	1000.0	186.7	24.0	18.4	3.52
130	1000.0	133.2	31.9	18.2	3.07
140	1000.0	131.0	16.6	18.8	3.96
151	1000.0	100.0	15.0	18.7	2.85
160	1000.0	222.4	15.0	18.4	2.91
171	789.8	100.0	15.0	17.1	3.99
180	1000.0	141.6	24.4	18.3	2.83
190	1000.0	717.0	25.5	21.1	3.48
203	1000.0	349.7	101.2	20.0	2.33
210	1000.0	661.0	37.8	19.6	2.97
220	1000.0	483.2	38.3	20.8	3.94
231	1000.0	324.0	58.2	20.1	2.61
241	1000.0	114.3	22.4	19.3	3.86
250	1000.0	104.9	15.0	18.9	3.28
260	1000.0	147.6	19.4	18.9	2.49
270	1000.0	259.6	24.5	20.7	3.04
280	1000.0	268.4	69.1	21.6	2.91
290	1000.0	233.9	23.2	21.6	3.50
300	1000.0	267.7	66.6	21.2	2.86
310	1000.0	289.3	66.1	23.8	3.92
320	1000.0	167.7	150.0	22.1	3.05
330	900.2	100.0	62.9	19.9	5.51
340	1000.0	141.2	32.0	16.4	6.36
350	327.5	100.0	15.0	14.1	9.13
360	891.7	100.0	35.0	16.6	4.33
370	821.9	100.0	18.9	15.9	3.87
380	443.1	100.0	15.0	15.2	3.03
390	461.1	100.0	15.0	16.4	4.32
400	604.6	100.0	15.0	16.8	4.46

Table D23. Backcalculated Moduli on R1 Lane of I-10 WIM Section (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
411	582.5	100.0	17.4	17.4	4.05
420	654.6	100.0	28.0	17.9	3.56
431	886.1	100.0	26.2	18.7	4.13
440	934.0	100.0	27.2	19.4	3.94
450	1000.0	126.6	20.8	18.0	3.15
461	849.0	100.0	26.7	18.9	4.18
471	901.9	100.0	21.8	19.7	4.62
481	895.1	100.0	26.0	21.4	4.53
490	1000.0	137.1	39.5	20.3	3.88
500	1000.0	182.1	16.4	19.1	3.69

Table D24. Backcalculated Moduli on R2 Lane of I-10 WIM Section.

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
0	1000.0	291.2	150.0	31.4	2.21
10	1000.0	227.3	94.4	28.0	3.00
21	764.6	120.6	150.0	27.7	3.10
29	1000.0	350.3	94.9	25.8	2.99
41	1000.0	729.5	58.3	24.8	2.65
51	1000.0	593.1	62.4	24.3	2.57
60	1000.0	1190.4	15.2	23.2	2.18
70	1000.0	945.8	20.3	19.7	2.29
81	1000.0	649.6	16.6	19.6	2.44
90	1000.0	294.7	22.8	18.4	2.91
110	1000.0	378.5	15.0	17.1	2.62
120	1000.0	316.8	25.7	16.7	4.18
130	1000.0	115.6	20.7	15.1	2.33
141	750.0	100.0	25.5	15.0	2.61
151	1000.0	207.2	21.7	17.3	2.79
161	1000.0	111.7	28.0	15.6	2.01
170	1000.0	126.6	15.0	17.3	3.71
181	1000.0	187.4	17.7	20.5	3.62
190	1000.0	256.5	73.9	20.3	2.77
199	1000.0	662.0	16.6	20.6	3.59
210	1000.0	254.5	44.2	18.1	3.46
220	1000.0	239.7	38.9	16.2	2.08
231	1000.0	145.4	63.8	16.1	1.46
241	1000.0	737.6	15.0	18.8	3.43
250	1000.0	215.1	37.5	18.4	2.62
260	1000.0	239.0	54.6	18.8	2.29
270	1000.0	332.2	16.0	19.8	2.30
280	1000.0	274.9	39.5	18.1	1.57
289	1000.0	416.4	17.0	18.1	4.18
301	1000.0	288.5	15.6	18.5	3.42
310	1000.0	290.6	17.3	19.2	2.51
320	1000.0	320.1	58.6	20.6	2.78
330	1000.0	278.0	61.4	19.8	2.06
339	1000.0	268.6	49.1	18.2	2.39
350	1000.0	210.8	15.6	17.5	3.49
360	1000.0	168.2	18.0	17.4	2.73
370	1000.0	150.1	55.3	17.8	4.76
380	1000.0	126.6	33.2	18.5	4.58
390	1000.0	100.0	29.9	17.7	3.08
400	1000.0	152.7	23.5	17.8	3.12

Table D24. Backcalculated Moduli on R2 Lane of I-10 WIM Section (continued).

Station (ft)	Backcalculated Modulus (ksi)				% Error per Sensor
	Surface	Base	Subbase	Subgrade	
410	1000.0	207.5	22.1	17.4	2.64
420	1000.0	205.0	20.9	16.9	2.49
430	1000.0	218.2	19.5	17.2	3.06
440	1000.0	227.6	18.2	18.2	3.12
450	1000.0	234.9	27.5	19.2	4.03
460	1000.0	239.0	19.1	18.2	2.92
471	1000.0	158.0	15.3	18.5	3.39
480	1000.0	125.1	41.6	18.1	2.35
490	1000.0	180.1	41.2	19.1	2.68
500	1000.0	199.0	64.3	18.6	1.66

APPENDIX E.

MARKERS PLACED ALONG PROPOSED 500-FT WIM SECTIONS ON I-10 IN BALMORHEA

Figure E1 shows the start of the proposed 500-ft WIM sections on the westbound lanes of I-10. Researchers hammered a cotton gin spike with washer on the shoulder to mark the beginning of the sections. The orange arrow painted on the shoulder is in the direction of traffic to indicate the beginning of the sections. The GPS coordinates at this location are:

N 31° 00.615’ and W 103° 48.766’

Also shown in Figure E1 are the two reflective tapes placed at the beginning of the proposed WIM section on each travel lane to trigger the inertial profiler measurements made on the 500-ft WIM sections.

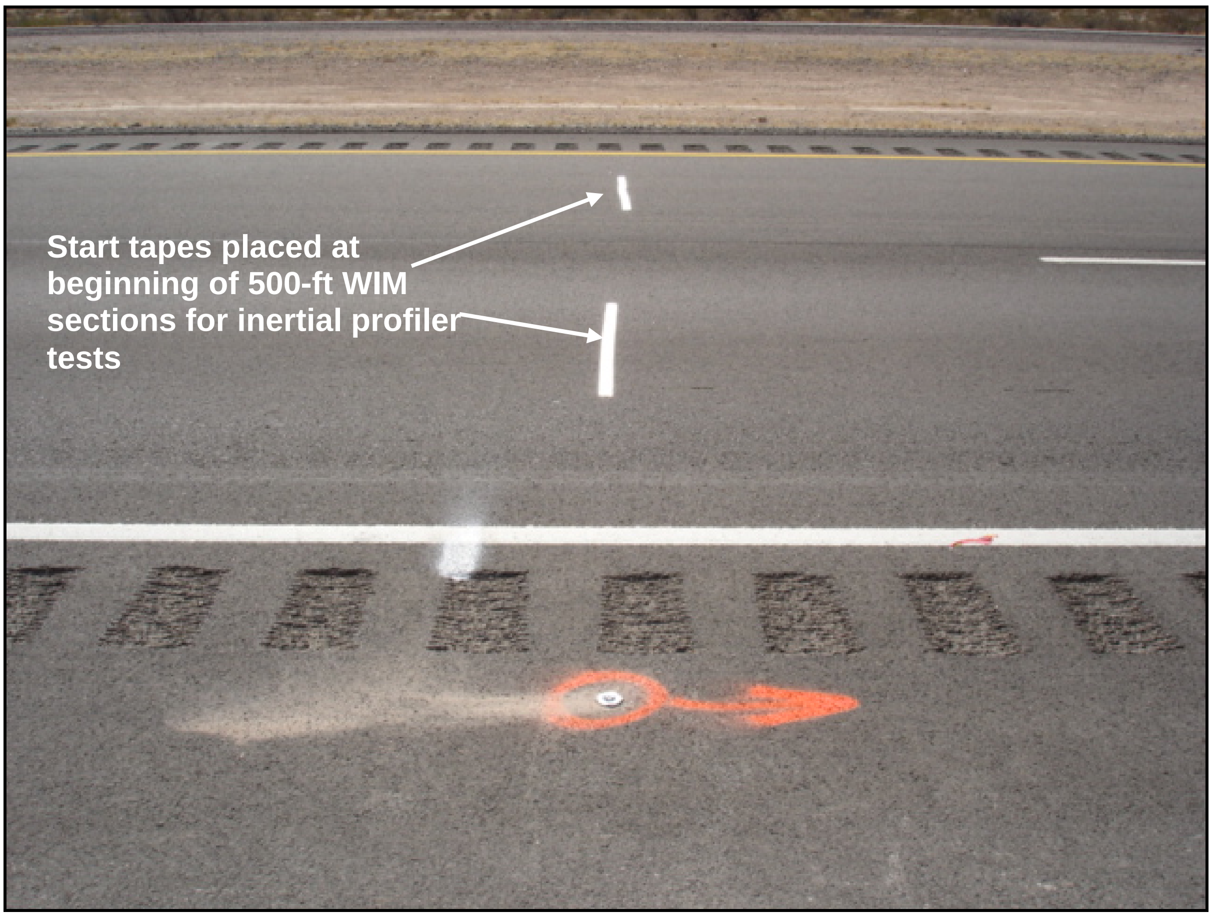


Figure E1. Marker Placed at Start of 500-ft WIM Sections on Westbound Lanes.

The beginning marker shown in Figure E1 is about 230 ft east of RM203 on the westbound lanes as shown in Figure E2.

Figure E2. Reference Marker (RM) 203 at Vicinity of Beginning Section Location on Westbound Lanes.

Researchers also placed a cotton gin spike with washer on the shoulder to mark the proposed location of the WIM sensors on the westbound travel lanes of I-10 in Balmorhea (see Figure E3). In addition, researchers painted the acronym *WIM* on the shoulder to show that the marker is for the proposed location of the WIM sensors on the westbound travel lanes. This location is 350 ft west of the start marker of the proposed WIM section on the westbound lanes. The GPS coordinates at the proposed WIM sensor location are:

N 31° 00.607' and W 103° 48.833'

Figure E3. Marker Placed at Proposed Location of WIM Sensors on Westbound Lanes.

Figure E4 shows the end of the proposed 500-ft WIM sections on the westbound lanes of I-10. Researchers hammered a cotton gin spike with washer on the shoulder to mark the end of the sections. The orange arrow painted on the shoulder is in the opposite direction of traffic along the westbound lanes to indicate the end of the proposed WIM sections, i.e., that the sections go in the opposite direction of traffic from the ending marker shown in Figure E4. The GPS coordinates at this location are:

N 31° 00.605' and W 103° 48.860'

The ending marker shown in Figure E4 is 150 ft west of the WIM marker for the westbound lanes shown in Figure E3.

Figure E4. Marker Placed at End of Proposed WIM Sections on Westbound Lanes.

Figure E5 shows the start of the proposed 500-ft WIM sections on the eastbound lanes of I-10. Researchers also hammered a cotton gin spike with washer on the shoulder to mark the beginning of the sections as they did on the westbound lanes. The orange arrow painted on the shoulder is in the direction of traffic to indicate the beginning of the sections. The GPS coordinates at this location are:

N 31° 00.573' and W 103° 48.896'

Also shown in Figure E5 are the two reflective tapes placed at the beginning of the proposed WIM section on each travel lane to trigger the inertial profiler measurements made on the 500-ft WIM sections.

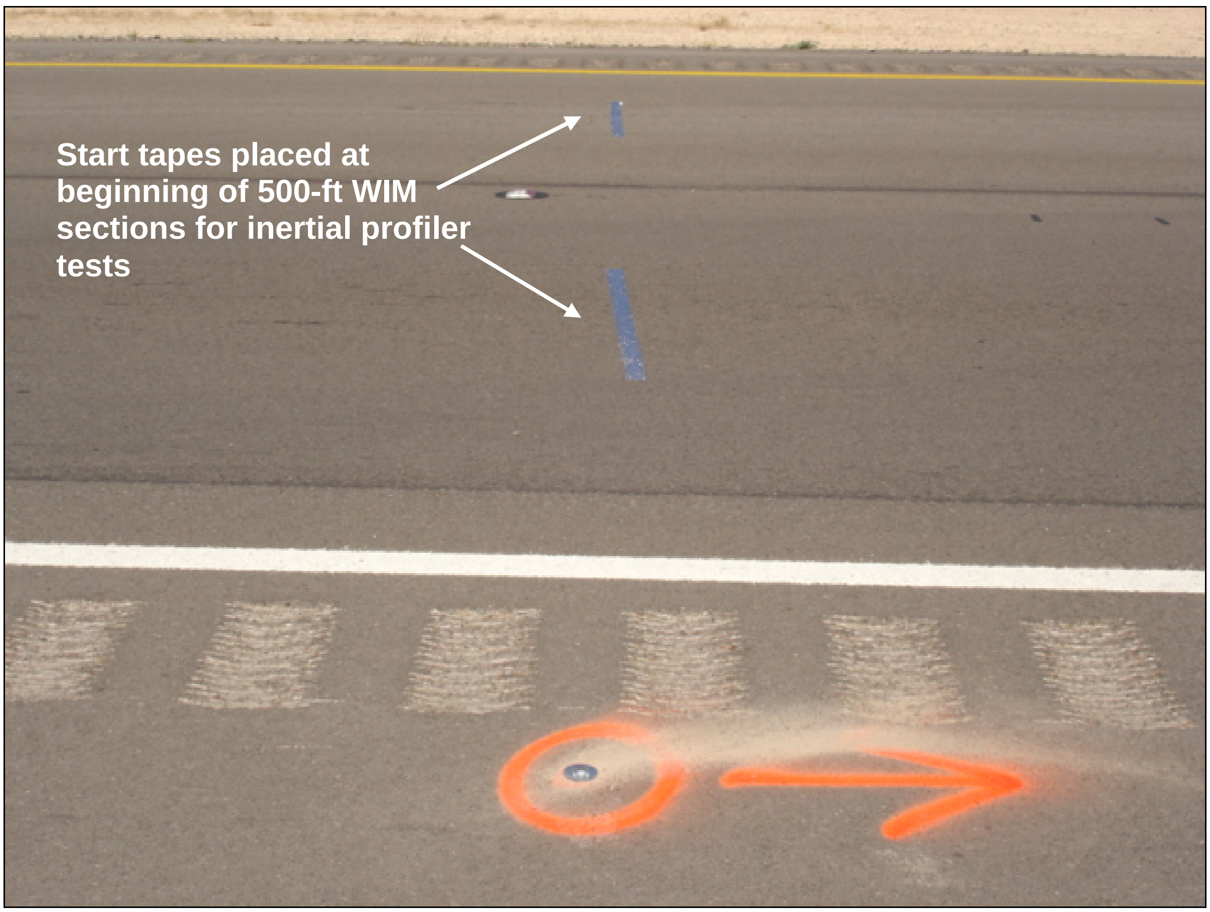


Figure E5. Marker at Start of 500-ft WIM Sections on Eastbound Lanes.

Similar to what was done on the westbound lanes, researchers also placed a cotton gin spike with washer on the shoulder to mark the proposed location of the WIM sensors on the eastbound travel lanes of I-10 in Balmorhea (see Figure E6). In addition, researchers painted the acronym *WIM* on the shoulder to show that the marker is for the proposed location of the WIM sensors on the eastbound travel lanes. This location is 350 ft east of the start marker of the proposed WIM section on the eastbound lanes. The GPS coordinates at the proposed WIM sensor location are:

N 31° 00.508' and W 103° 48.830'

Note that the proposed locations of the WIM sensors on the westbound and eastbound travel lanes align with each other. This alignment is reflected in the longitude coordinates of the proposed WIM locations, which are W 103° 48.833' on the westbound lanes, and W 103° 48.830' on the eastbound lanes. The two longitude coordinates are almost identical. The proposed location of the WIM sensors on the eastbound lanes is about 115 ft west of the sign post for RM203 on the adjacent shoulder.

Figure E6. Marker Placed at Proposed Location of WIM Sensors on Eastbound Lanes.

Figure E7 shows the end of the proposed 500-ft WIM sections on the eastbound lanes of I-10. Just like what was done on the westbound lanes, researchers hammered a cotton gin spike with washer on the shoulder to mark the end of the sections. The orange arrow painted on the shoulder is in the opposite direction of traffic along the eastbound lanes to indicate the end of the proposed WIM sections, i.e., that the sections go in the opposite direction of traffic from the ending marker shown in Figure E7. The GPS coordinates at this location are:

N 31° 00.583' and W 103° 48.802'

The ending marker shown in Figure E7 is 150 ft east of the WIM marker for the eastbound lanes shown in Figure E6.

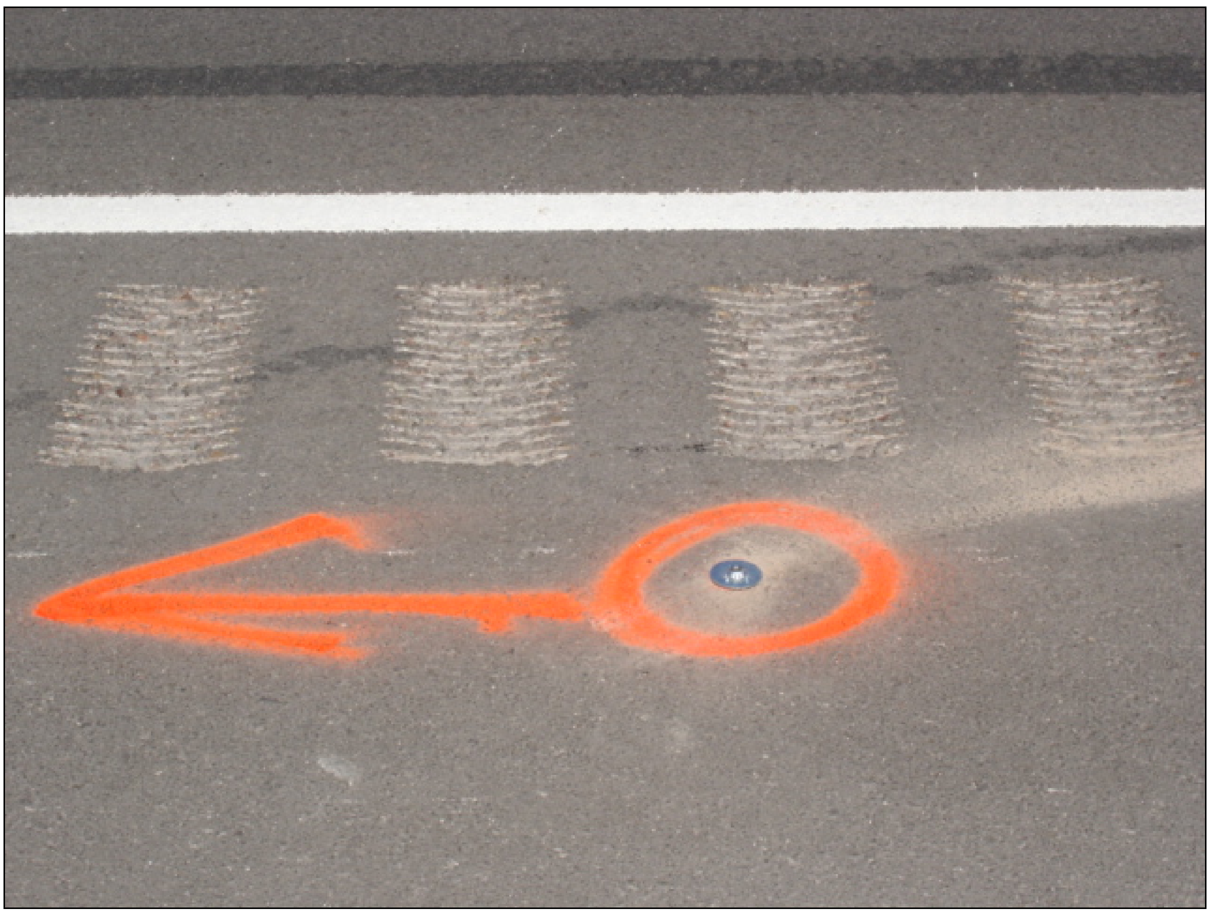


Figure E7. Marker Placed at End of Proposed WIM Sections on Eastbound Lanes.

APPENDIX F.

TRAFFIC ANALYSIS SECTION WIM SCREENING CRITERIA

SYNOPSIS OF PROCESS

Shop personnel send data to the System Support Specialist (data received by lane). The data are then loaded into the WIM analysis tool. Records are screened. From the screened data, seven consecutive days after a calibration (or as close to the most recent calibration) are extracted for each quarter from each station and are loaded into VTRIS. A report is created for DPS through VTRIS and further processing is completed using an Excel spreadsheet. Data is loaded into VTRIS on a quarterly basis and used for the annual Card 7 submission. Quarterly VTRIS files are uploaded into the mainframe processes to build the weight distribution table and Card 7 format file. Corridor analysis uses the weight distribution tables in RDTEST68. Data are fed into the central database, Traffic Log (TLog).

The DPS Weight Report is used as a monitoring type function for catching overweight trucks by DPS. Machine errors can cause the data to be off by ±30 percent. The reports are the Weight 6 and Weight 7 reports. The process creates readable output (reports) summarized in Excel format.

DETAILED DATA PROCESS

Import monthly data – bring 31 days per month into work table.

Queries processed on work table.

WT = Axle weight

SP = Spacings

GH = Ghost axle. Record indicates there is an axle, but there does not exist a configuration with the axle given the spacings.

WT (1) Any axle < 1100 lb (cannot check continuation records)
kicks out as weight violation (non-class 9)

SP (2) Continuation records < 1100 lb

SP (2) Spacings < 3.3 ft rejected (from TxDOT journals depicting tandem spacings)
Texas 6 Class Schemes (non-class 9)
Usually where there is a tandem wheel, analysts have seen 2.3, 2.6, 2.9 ft spacing.

SP (3) 2S2 check on for split tandem at 100 records per station
Yes split – reject
> 6 ft, these are not tandem therefore a “spacing violation”

SP (4) Class 10 3S3 000 < 6 ft spacing between each 76 ft tridem

(000 indicates a tridem)

SP (5) Class 10 long trailer > 40 ft trailer then rejection

~ 12 records per station (every other site)

GH (6) Steer – rear < 6 ft Ghost class 2-Ds (sometimes Class 9)

< 6 ft between rear axle & steering axle (usually 2.6, 2.7 feet spacing)

VTRIS converts body codes (raw) to FHWA Scheme F

Mainframe doesn't have a place for split tandems

(Mainframe – tandem, tridem, quad)

SP (7) Class 12 > 18 ft between trailer axles span between end of trailer and back of 2nd

~ 6 per 100,000 for vehicle codes 531200 or 63200

(8) Serial numbers that are null

SP (9) Class 6 (not common) A-B < 6 feet; B-C < > 6; C-D < > 6

A = First axle

B = Second axle

C = Third axle, and so on

UC (10) Unclassifieds, mark as UC vehicle code 0000

Double Check Class Code

CC (11) 2A Bus A-B ≥ 21 ft rejected

19 ft is bus in class field

190200 – school bus – OK

22000 – six tires/duallies (single unit, 2-axle) – rejected

2SU (12) 3A Bus A-B > 21 ft (Vehicle code 190300)

B-C > 2, < 6

3ST (13) 2D No 3rd axle A-B > 6, < 21

220 dually

4Hvy (14) Class 423 WT ≥ 11,000 lb A-B > 10, < 13

Dmp trks D-E > 3.3, < 6

5 MT (lots of axles are < 3.3 ft)

Almost all with this code fail to pass

(15) Class 422 A-B > 13, < 22

B-C > 6

Not screening for spacing on last two axles, counter (14)

(16) Class 421 A-B > 10, < 22

B-C >10, < 45

45 ft source is permits base / seldom rejections

(17) Dump & SA Trailer 43100 WT >/= 8.5

A-B > 13, < 22

B-C > 3.3, < 6

C-D > 6

431 high – high Class 9 – likely construction area nearby

(18) 3A SU Class 6 2300 A-B > 6, < 22

B-C > 3.3 ft, < 6

(19) 3320 A-B > 6, < 17

B-C > 16

C-D > 3.3 ft, < 6

5% of Class 10s are Quad Trailers

T14 gets recoded if meets

(21) 3340 / 533100 A-B 0 or ≤ 25

B-C 0 or ≤ 6

C-D 0 or < 50

D-E 0 or < 6

Observation: Only bending plates give a vehicle code.

(22) Continuation Records (Codes assigned by bending plate program)

3330 A-B 0 or < 25

B-C 0 or < 6

C-D

(23) 3430 (Class 10)

(24) 5312 Recode

(25) 5222 Recode

(26) 6321

(27) 5322

Problems: Data Collection – piezoelectric vs. bending plate video comparison

CC codes applied at end to blank class fields

- T16 Class 10 4At & 3A Trail
- T19 Body Code is off – recorded to 5312
- T20 Class 13 5322
- T21 blanks get CC

#WT – broken sensor

#SP – Calibration

Wheel path weights are not available to analysts

522-523 volume diff. 1 mile apart

5212 has “default” spacings between C-D

5% have continuation records