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16. Abstract

The rolling wheel deflectometer (RWD) offers the benefit to measure pavement deflection without causing any traffic interruption or compromising safety along tested road segments. This study describes a detailed field evaluation of the RWD system in Louisiana in which 16 different test sites representing a wide array of pavement conditions were tested. Measurements were used to assess the repeatability of RWD measurements, the effect of truck speeds, and to study the relationship between RWD and falling weight deflectometer (FWD) deflection measurements and pavement conditions. Based on the results of the experimental program, it was determined that the repeatability of RWD measurements was acceptable with an average coefficient of variation at all test speeds of 15 percent. In addition, the influence of the testing speed on the measured deflections was minimal. The scattering and uniformity of the FWD and RWD data appears to closely follow the conditions of the roadway. Both test methods appear to properly reflect pavement conditions and structural integrity of the road network by measuring a greater average deflection and scattering for sites in poor conditions. RWD deflection measurements were in general agreement with FWD deflections measurements; however, the mean center deflections from RWD and FWD were statistically different for 15 of the 16 sites.

This study developed and validated a direct and simple model for determining the pavement structural number (SN) using RWD deflection data. To develop this model, the relationship between the average RWD surface deflection and the peak FWD deflection was investigated. The developed model correlates a pavement's SN to two RWD-measured properties (average RWD deflection and RWD index). The developed model was fitted to RWD data collected in 16 road sections (each 1.5 miles), referred to as research sites, in Louisiana. The model was then validated based on FWD and RWD data collected on 52 road sections in Louisiana. Results showed a good agreement between SN calculations obtained from FWD and RWD deflection testing. While the developed model is independent of the pavement thickness and layer properties, it provides promising results as an indicator of structural integrity of the pavement structure at the network level. The fitting statistics support the use of the proposed model as a screening tool for identifying structurally deficient pavements at the network level.

Based on the RWD evaluation conducted in District 05, this study recommends extending the use of RWD to the other districts in Louisiana. The RWD index (RI) is recommended to be adopted on a provisional basis by the Louisiana Department of Transportation and Development (LADOTD) Pavement Management System (PMS) as a network structural analysis index with three categories: thin pavements less than 3 in. thick, medium pavements between 3 to 6 in., and thick pavements greater than 6 in. It should be incorporated into the PMS system and placed on Geographic Information System (GIS) maps.

The structural number equation should be considered valid and used as a tool to evaluate the structural condition of pavements for network purposes with similar categories as the RI. The PMS section will incorporate the SN values in their process using trigger values outlined in the report. If the PMS section considers the new index to be of significant value, then another district will be assessed with the RWD.

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Implementation of Rolling Wheel Deflectometer (RWD) in PMS and Pavement Preservation

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IMPLEMENTATION STATEMENT

Based on the findings and the results of this project, the use of RWD at the network level is beneficial and should be extended to the other districts in Louisiana. In addition, RWD data should be collected regularly in Louisiana at a frequency of once every four years. Validation and possible modification of the developed models should be conducted based on independent data collected in the other districts. The proposed testing and assessment strategies should be implemented by the Department and through the PMS division in LADOTD.

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INTRODUCTION

The current LADOTD pavement management system is based on pavement condition data that are collected biennially using the ARAN system in order to provide a continuous assessment of the road network. Conditions of the pavement are assessed using cracking, rutting, roughness, patching, and faulting data. Collected data are then analyzed to calculate a composite performance index on a scale from zero to 100. A number of threshold values are also used to trigger a specific course of maintenance and rehabilitation (M&R) actions *[1]*.

The LADOTD pavement management system may be substantially improved if the structural conditions of in-service pavements are considered in selecting suitable treatment methods. This may help avoid applying routine preventive treatment techniques on structurally deficient pavements. Surface deflection is a popular method used to determine the structural capacity of an in-service pavement without the need to remove or disturb the existing pavement *[2]*. Deflection measurements have also been widely used in pavement management activities. At the project level, deflection measurements are used to identify distressed pavement locations and for overlay pavement design. At the network level, deflection testing has mainly been used to identify uniform and homogeneous pavement sections. However, due to the cost and slow FWD testing process, the use of deflection testing in network PMS activities has been limited. In addition, delays due to lane closures may compromise the safety of the traveling public and highway workers, which may not justify the value of the collected data *[3]*.

The use of RWD, which measures deflections at highway speeds, offers the potential to cost-effectively characterize the structural capacity of the road network without major delays. LADOTD recently conducted a field evaluation of RWD continuous deflection measurements in District 05. Through the proposed research activities, an evaluation of RWD in Louisiana was conducted and collected data were analyzed to incorporate measured deflection into the existing PMS via GIS. Research activities also assessed the potential use of RWD to characterize the structural conditions of the road network and to identify structurally deficient pavements.

Literature Review

The evaluation of pavement structural capacity and integrity is an important component of PMS to assist in the selection of suitable maintenance and rehabilitation strategies *[4]*. The FWD is a non-destructive deflection testing method that is widely used at the project-level to assess the structural conditions of in-service pavements. In the FWD test, a force pulse is applied to the pavement surface by dropping a weight on the pavement surface and measuring surface deflections via specially designed geophones with a high level of accuracy. This test setup produces an impact load with duration of 25-30 milliseconds, which corresponds to a wheel velocity of 50 mph in the upper layer *[5]*. Due to its stop-and-go operation, the use of FWD at the network level has been limited.

The current pavement management system for LADOTD is based on pavement condition measurements that are collected once every two years using the Automatic Road Analyzer (ARAN®) system that provides a continuous assessment of the road network *[1]*. Collected data are reported every 0.1 mile and are analyzed to calculate a Pavement Condition Index (PCI) on a scale from zero to 100. The PCI varies from 95 to 100, 85 to 94, 65 to 84, 50 to 64, and 49 or less for very good, good, fair, poor, and very poor roads, respectively. The state PMS may be substantially improved if the structural conditions of in-service pavements are considered in pavement management decisions. However, due to the large size of the state road network and the difficulty to conduct a routine testing program for pavement deflection measurements using FWD, the necessity of a device to measure pavement deflections at traffic speeds is evident.

RWD offers the benefit to measure pavement deflection without causing any traffic interruption or compromising safety along the tested road segments. This innovative system, which measures deflections at traffic speeds, offers the potential to cost-effectively characterize the structural integrity of the road network without major delays. In spite of these promising benefits, the repeatability and characteristics of RWD measurements need to be established as well as the relationship between deflection measurements conducted using RWD and FWD. This study presents a detailed field evaluation of the RWD system in Louisiana in which 16 road segments representing a wide array of pavement conditions were tested using RWD and FWD.

Pavement Condition Evaluation

Pavement evaluation is performed to assess the functional and structural conditions of roadways for routine monitoring in order to select proper corrective actions. Functional condition is related to the roughness and ride quality of a highway section. Structural condition deals with a pavement's ability to withstand traffic loads and environmental conditions, which can be measured by determining material properties, layer thicknesses, and surface deflections *[6]*.

At the network level, conventional evaluation approaches may be used to develop performance criteria and determine maintenance and rehabilitation priorities, in terms of spending efforts and funding availability. At the project level, a more focused evaluation is conducted in order to help determine causes of different types of existing distresses and to select the best treatment strategy. A visual condition survey presents a basic method for evaluating functional and structural pavement conditions. Although widely considered as a highly subjective approach for determining the current condition of the roadway section, the visual survey serves generally well for preliminary identification of different distress types and locations, as well as estimating the severity of these distresses.

The process of evaluating roadway segments is divided into two main steps: (a) preliminary pavement analysis and (b) detailed pavement evaluation and design. The first step occurs during the project-scoping phase, while the second step occurs during planning, specifications, and estimating of the development phase (detailed design).

Destructive Testing. The term *destructive-testing* refers to the nature of those testing approaches that cause damage to the pavement structure, which in turn may influence the structural or the functional capacity of the tested highway section. Thus, these types of testing procedures require immediate maintenance remedies in order to avoid problems resulting from a decreased structural or functional capacity of the roadway. Traffic disturbance due to lane closure during testing and limited resources and maintenance actions following the testing become conventional obstacles facing implementation of destructive testing. However, destructive testing provides necessary information not otherwise provided by non-destructive testing, in terms of data type and level of details. Such data include:

- Visual inspection of different pavement layers and distress identification; and
- Mechanical and physical properties by laboratory testing of samples obtained from coring, trenching, and Shelby tube.

Non-Destructive Testing. Non-destructive testing is a general term used to describe the process of evaluating existing pavement structures, while causing no disturbance to the functional or structural condition of the tested section, which subsequently requires no subsequent corrective action *[7]*. Due to the high cost associated with destructive testing in PMS, in terms of time and expenditures for restoration of the tested pavement, nondestructive testing represents a promising alternative. Numerous methods have been introduced and modified for more efficient testing of pavement structures, which may be classified into two major categories *[8]*: (1) seismic-based testing or (2) deflection testing. Seismic-based testing is based on measuring the velocity of propagating stress waves through the pavement, while deflection testing focuses on measuring the pavement response due to applying a relatively large load to the pavement surface. Although these types of testing processes have sufficient operational and economic benefits, they do not directly measure engineering properties of the pavement structure. Therefore, further testing and analysis is always required to provide an assessment of in-situ pavement conditions *[4]*.

Since nondestructive testing provides promising benefits when compared to conventional destructive testing methods, the implementation of non-destructive testing became an integral part of maintenance and rehabilitation strategies around the world *[9]*. Conducting testing programs using FWD and the Ground Penetrating Radar (GPR) benefits state agencies, as these tests supply data about layer properties and thicknesses as well as pavement response. In addition, FWD and GPR information can also aid decision makers to prioritize maintenance activities and allocate available resources adequately. However, issues such as data collection expenses, a lack of simplified procedures with direct results, and scarcity of resources represent major obstacles in the use of FWD and GPR as completely reliable tools for pavement testing at the network level *[10]*.

Deflection Measurement Devices

The Benkelman Beam. The Benkelman beam was introduced in the early 1950s during the Western Association of State Highway Organizations (WASHO) Road Test and consists of a support beam and a probe arm *[11]*. The device frame is provided by an arm that is 8 ft. long and that is extended to a probe point. The probe arm is equipped with a gauge at 4 ft. behind the pivot to measure the relative vertical distance between the pivot arm and the frame, Figure 1. During the testing procedure, the probe is placed between the dual tires of a loaded truck *[4]*. The truck is placed such that one of the rear dual wheels is positioned on the point of measurement. The probe is placed between the two wheels to measure surface deflection, which equals to double the difference between the final and initial readings *[8]*. In order to maintain a high level of accuracy for the collected data, it is preferable to limit the measurements to be within the deflected region of the pavement, which occurs within a radius of 8 ft. around the loading point.

A major limitation of the Benkelman beam is the inability to determine the entire deflection basin and avoid the front support interference with the deflection basin. Furthermore, it was found that the Benkelman beam is unable to measure the deflection resulting from thick rigid pavements *[12]*. As a solution to this problem, two or more beams should be used to conduct the test. Simplicity and low cost are major advantages of this type of deflection testing with a daily production of 50-100 test points using a crew of three technicians *[13]*.

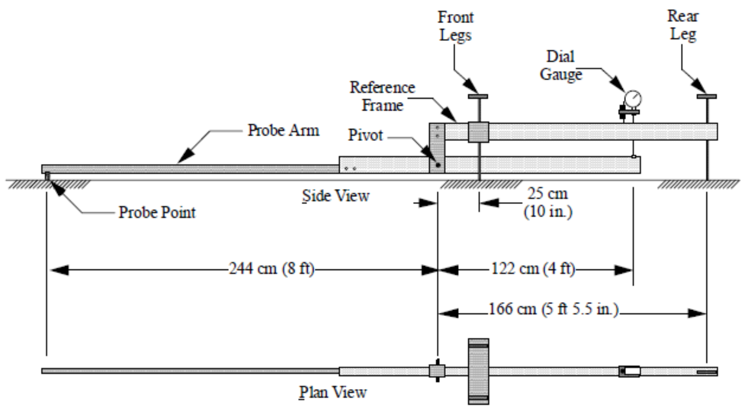


Figure 1
Simplified schematic of Benkelman beam *[14]*

The Falling Weight Deflectometer. FWD was first developed to measure pavement surface deflection in airports, due to aircraft loading while moving at intermediate speeds. Although it was first introduced in Europe, FWD has been in use in the United States since the 1980s. The device is used to measure surface deflection by applying an impulsive force to the pavement surface. Data collected from FWD testing are heavily used for assessing the structural capacity of existing pavements, design and research purposes, as well as pavement management needs. A survey conducted by "NCHRP synthesis 381" indicated that more than 81 FWD devices are used by 45 different state highway agencies in the US *[15]*. The large number of FWD devices in service reflects the reliability and importance of this test procedure in the current state of practice in pavement management. Several types of FWD are available commercially; the most popular ones are KUAB, JILS, Phoenix, Carl Bro, and the Dynatest, which is the most popular type in the US, Figure 2. FWD devices consist of the following four main components:

- An impulsive-force generator that enables application of variable weights to the pavement surface from different heights;
- A loading plate to spread the impulsive-force uniformly through the tested layer surface;
- Three or more sensors (currently available FWD devices have up to 9 sensors for deflection basin determination); and
- Data acquisition, processing, and storage system.

Figure 2
Falling weight deflectometer *[16]*

To simulate different vehicle loads and traveling speeds, the force level and duration may be varied by changing the dropping height, load (1 kip to 35 kip), the stiffness of the plate that is in contact with the surface, and the force pulsing duration (30 to 40 msec). All types of FWD devices use an 11.8-in. diameter circular plate to apply the predesigned force to the pavement surface. Load cells are also utilized to measure the vertical, dynamic force, Figure 3. A set of transducers are mounted in one row to measure the deflection basin, resulting from the applied dynamic force. The Dynatest FWD uses seven to nine geophones to measure the pavement deflection, while the Phoenix FWD uses three *[17, 18]*. FWD testing is conducted by positioning the FWD at the desired testing point. The loading plate and deflection sensors are then lowered to contact the pavement surface. The drop weight is then raised hydraulically *[19]*. When the drop weight is at the selected drop height, an electrical release drops the weight onto the loading plate and impacts the pavement. Temperature is also recorded during testing at the layer surface and is calculated at mid-depth. A data acquisition system measures the load cell and deflection transducer outputs *[20]*.

The test is repeated several times and the results are averaged. Tests may also be performed using different drop heights, hence different force levels, at each testing location. After testing is complete, the loading plate and sensors are raised, and the device is towed to the next test location. Typical daily production for the FWD, operated by a crew of one or two technicians, consists of 100-300 test locations per day *[18, 21]*.

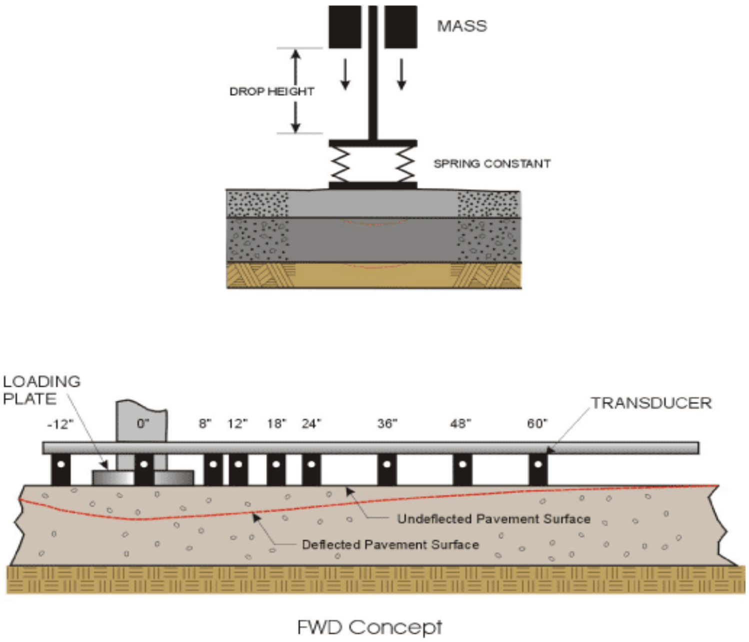


Figure 3
Falling weight deflectometer measurement mechanism *[22]*

The FWD testing pattern depends on two key elements: project layout and project size. Several aspects should be considered during the planing and implemetation phases of the testing program, such as total length of test sections, total number of measurement points, and number of lanes. The project layout is the first element to affect the FWD testing program, since the layout depends on whether testing will be implemented in only one direction or in both directions. This should be decided depending on available data about the roadway section that shows indications of structural deficiencies that need to be verified. For multi-lane roadways, the testing pattern should be designed, based on the outer lane and possibly the inner lanes *[15]*.

The number of testing points is determined depending on the total length of the roadway section. Accordingly, the project size is designed based on the directional length of pavement *[23, 24]*. After field testing is complete, FWD data are prepared for the analysis phase. Several analysis tools and software packages are used by highway agencies for performing both backcalculation and forward calculation, calculating load transfer efficiency and quality control and assurance purposes *[25]*.

Although FWD has become the predominant approach for NDT for many highway agencies and pavement management systems in the United States, a number of concerns were expressed through several testing programs carried out using FWD. These concerns are listed as follows *[15]*:

- Due to the stationary nature of testing, FWD is capable of testing a maximum of 300 points per day, which approximately converts to only 30 miles with 0.1-mi increments. Thus, the entire testing program could last for years to cover a complete network without even performing multiple replicates *[26]*.
- A complete closure of at least one lane of the roadway is needed during testing, which greatly disturbs traffic flow and decreases highway capacity. This could be a subtaintial problem in highways with high traffic volumes and during rush hours.
- During testing, a crew of one to three technicans sets up the device and performs testing on the specified logmiles from the testing program and along a selected lane of the highway. The highway might be an interstate or a primary road, which means that the operating traffic speed might be as high as 70 mph. Consequently, the crew of technicians is exposed to risk during the testing period, in addition to vehicle safety reduction, due to the lane closure *[27]*.
- The effect of the impact of the applied force to the pavement surface may affect the structural capacity of the pavement layer, especially in thin pavements at loads of 12,000 or 16,000 lb. This could damage thin pavements, thus rendering FWD a destructive testing methodology.

The aforementioned points raise concerns about the cost-effectiveness of using FWD as a primary tool of non-destructive measurement of pavement resposes.

Usage and Applications of the Falling Weight Deflectometer. Zaghloul et al. demonstrated the use of FWD to evaluate pavement structural performance. The authors also introduced a new Structural Adequacy Index (SAI) model based on FWD data for the New Jersey Department of Transportation (NJDOT) *[28]*. FWD testing was implemented at the network level for the New Jersey road network by collecting a total of 30,000 deflection basins. Data provided from FWD measurements were used to calculate the following: subgrade resilient modulus (M_R), effective pavement elastic modulus (E_p), and an effective and required structural number for the existing pavement (SN_{eff}, SN_{req}). For rigid pavements, the following properties were calculated:

- Effective Modulus of Subgrade Reaction (K_{static});
- Effective Portland Cement Concrete (PCC);
- Elastic Modulus (E_{PCC});
- Modulus of Rupture (S_c);
- Effective thickness of the existing PCC slab (D_{eff});

- Required PCC thickness based on a future traffic volume (D_{req}); and
- Load Transfer Efficiency (LTE) at the joints.

FWD was first adopted by the Texas Department of Transportation (TxDOT) as a comprehensive approach to assess the structural condition of pavement *[29]*. This posed a potential substitute for applying conventional seal coats, overlays, and other methods of surface treatments, which were applied to provide adequate protection for the pavement surface. TxDOT uses a structural, screening index called the Structural Strength Index (SSI) as a solution for the problem associated with the unavailability of layer thickness information. The lack of thickness data represented an obstacle for a backcalculation process of the layer's moduli. However, TxDOT found that the SSI was not sufficient to show a differentiation between pavements in good condition and those pavements needing rehabilitation. Thus, Zhang et al. (2003) examined the validity of several structural estimates to quantify pavement deterioration, based on changes in the Pavement Management Information System (PMIS) score values. In addition, the study proposed the SN and the Structural Condition Index (SCI) as screening tools for maintenance and rehabilitation decisions. The study concluded that SN may be used as an acceptable indicator of the structural condition of pavements and as a platform for maintenance and rehabilitation decisions *[30]*.

Garg and Marsey (2002) conducted a comparison between FWD and static deflection measurements of flexible pavements *[31]*. The study examined the results of the by means of a rapid and non-destructive technique, utilized to measure pavement surface deflection. The testing applied the FWD and static load tests to flexible pavements at the National Airport Pavement Test Facility (NAPTF). Based on the results from static and FWD tests performed on six flexible pavements, the researchers reported a significant variation between the resulting deflection in each case. This was due to the fact that the pulse loading generated by FWD is equivalent to a single-wheel dynamic loading, while static tests are equivalent to six-wheel gear configuration. Thus, a significant difference between predications based on FWD and static tests is expected *[31]*.

Noureldin and co-authors highlighted the importance of using FWD and GPR for pavement evaluation at the network level, as well as to provide recommendations for future improvements *[32]*. The Indiana Department of Transportation (INDOT) utilizes FWD testing as a solution for drawbacks associated with conventional testing, such as high expenses associated with data collection, limited resources, and the complexity of available analysis approaches. In addition, the adoption of FWD as a non-destructive method of pavement testing addressed the need to collect pavement surface deflection. Further, FWD can provide information regarding pavement layers and properties to be used as an input for the new "2002 AASHTO Pavement Design

Guide" based on a mechanistic-empirical approach for the design of pavement structures. The analysis found that the estimates for combined pavement thicknesses utilizing FWD measurements, matched GRR estimates in some cases, with slight variations in other cases. The authors concluded that utilizing both FWD and GPR for network level testing is beneficial since the resulting data could be used during the design phase or the maintenance and rehabilitation phase *[32]*.

In order to help decision makers set maintenance and rehabilitation strategies on interstates and primary roads, the Virginia Department of Transportation (VDOT) conducted an automated surface distress survey *[33]*. Although highways vary between flexible, composite, and rigid pavements with a total of approximately 27,000 lane-miles, there is no specific approach to quantify the structural capacity of these pavements. Thus, a FWD interstate testing program was designed and implemented in two phases. The first phase was conducted on two different interstates with a total of 124 lane-miles, while the second phase included the rest of Virginia's interstate system. It was concluded that FWD may be used at the network level to determine the structural condition of pavements. In addition, FWD could assist pavement designers and pavement management systems define network needs and manage available funds for maintenance. The study recommended structural evaluation of the intestate system based on FWD, as well as expansion of the testing to include the primary road network *[34]*.

Howard and co-authors (2007) studied the ability of FWD to simulate traffic loads on thin AC roads with low traffic volumes *[35]*. FWD testing normally collects pavement surface deflection, while other pavement responses, such as stresses and strains within pavement layers, remain unknown. A testing program was implemented over a six-month period, including both FWD and traffic loads, operating at approximately 35 ± 5 mph to aid comparison with FWD. A truck with a back axle weight of 20,000 lb. was used to generate around 2,100 passes, while a Dynatest 8000 FWD was used to generate approximately 500 drops with different weights of 6,000, 9,000, and 12,000 lb. Plaxis software was used to create an advanced finite element model to compare pavement strain and stress responses from FWD and traffic loads. According to the results, pavement responses in terms of asphalt stain, base pressure, and subgrade pressure from traffic loads were higher than from the FWD loads for nearly all of the tested sections and the variation increased with depth. The authors had no specific explanation to explain the difference between the two cases. However, it was suggested that a new finite element model could be created to simulate the sudden impulse nature of the FWD loading. Further testing under a wider range of conditions and pavement characteristics was recommended *[35]*.

Zaghloul et al. (2005) synthesized a number of studies discussing the repeatability and reproducibility of FWD testing and are summarized as follows *[36]*:

- Bentsen et al. collected testing results of accuracy and repeatability analysis of seven different nondestructive deflection devices. It was found that reproducibility results using different types of FWD devices needs to be verified. On the other hand, calibrated FWD units showed good repeatability *[37]*.
- Van Gurp used three different types of FWD devices: KUAB, Phoenix, and Dynatest. He reported that approximately all the results were repeatable, though there are significant variations between the results of different types of FWD for the same location *[38]*.
- A study was conducted using one FWD device with three different sets of buffers to examine the reproducibility of FWD results. The study concluded that different shapes and sizes of deflection buffers have an impact on the rise time and the shape of the loading waves, which subsequently affect deflection values *[39]*.

Continuous Deflection Measurement. Several limitations are encountered with discrete pavement testing methods, as demonstrated in the previous sections. One problem is caused by the existence of a wide array of variation in pavements, such as various subbase and subgrade types, layer thicknesses, and various AC types due to design and construction, and existing stiff under-layers. Although discrete testing methods provide relatively accurate measurements for pavement deflection, continuous deflection measurements provide properties for the entire roadway segment. Continuous deflection profiles can locate areas of weakness along the pavement section, as well as exact locations of severe distresses. In addition, continuous profiling provides the benefit of identifying and eliminating irrelevant data, due to the existence of bridges, horizontal or vertical curves. Deflection values along the pavement section show a variability that indicates the overall condition of the highway section, and also increases the reliability of the design and rehabilitation plans.

Since the early 1950s, various approaches were developed to measure pavement deflections, including stationary and continuous measurement methods. The need for a continuous deflection testing method has been widely supported in literature due to time constraints and the necessity to divert traffic from the tested lane when stationary devices are used. Arora et al. identified five devices that have been evaluated for continuous deflection testing *[40]*: (a) Texas Rolling Dynamic Deflectometer (RDD), (b) Airfield Rolling Weight Deflectometer (ARWD), (c) Rolling Wheel Deflectometer (RWD), (d) Road Deflection Tester (RDT), and (e) Traffic Speed Deflectometer (TSD). Although these devices continuously measure pavement surface deflections, they often operate at very slow traveling speeds (e.g., 3.0 mph for RDD).

The Rolling Dynamic Deflectometer. The Center for Transportation Research at the University of Texas at Austin developed a nondestructive tool for pavement response due to traffic loading. Cooperation between the US Air Force, College of Engineering at the University of Texas at Austin, and Teledyne, Inc. funded the development and modification of a Vibroseis truck that was originally used for oil exploration *[8]*. The dynamic loading system of the Vibroseis truck was modified to a servo-hydraulic loading system, in order for the truck to apply dynamic forces while traveling, Figure 4.

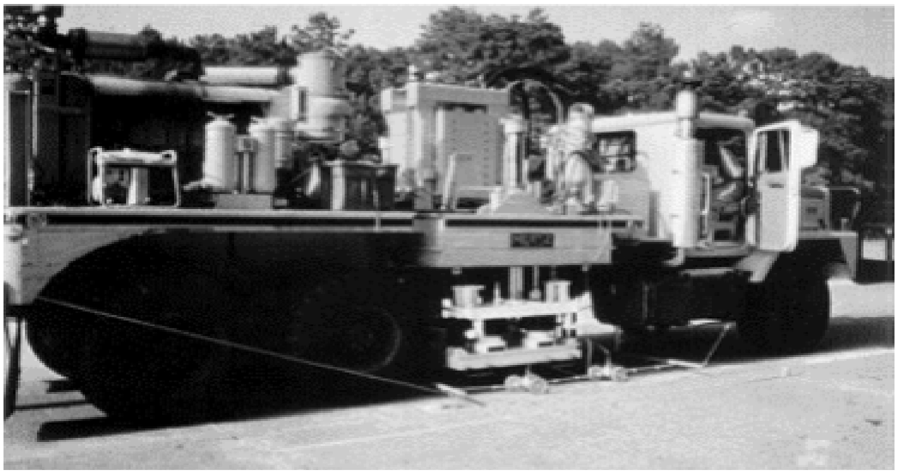


Figure 4
Rolling dynamic deflectometer *[8]*

A powerful diesel engine is used to feed a driving, hydraulic pump. The hydraulic pump generates a combination of both static and dynamic forces to the pavement surface, using two rollers placed parallel to the truck wheel axles. These rollers are used to push a set of sensors forward in the direction of travel as well. Load cells are provided to measure the magnitude of applied forces. A testing criterion for testing and data acquisition is selected before performing the test. The operating sampling frequency is also selected, according to the required data accuracy. The rolling dynamic deflectometer may be used for other testing purposes, such as estimating pavement depth and measuring pavement resistance to fatigue cracking *[8]*. The RDD demonstrated good potential for providing continuous profiles of flexible (and rigid) pavement structures. A comparison between RDD and FWD data showed very good correlation *[41]*. However, the RDD extracts only three deflection values compared with seven for the currently-used FWD. Additionally, the maximum operating speed of the RDD is 3 mph, which makes the testing extremely time-consuming and inappropriate for operating on interstates and primary roads *[40]*.

Airfield Rolling Weight Deflectometer. ARWD was originally developed by Quest Integrated, Inc. to test the deflection of airfield pavements on the basis of the Benkelman beam approach for deflection measurement *[42, 43]*. The ARWD system is composed of a towed trailer that is capable of travelling with speeds up to 20 mph. A loading platform is used to load the pavement through the trailer rear tire with loads of approximately 20,000 lb. As shown in Figure 5, a horizontal 33-ft. long beam is used to carry four equally spaced optical sensors (9 ft. apart), one is positioned near the load and the other three are placed ahead of the load in the same line. The deflection is measured by using a method of laser triangulation and parallax. A laser beam is used, rather than a horizontal beam, in order to avoid any data errors due to thermal and vibration effects. The sensors are responsible for measuring the distance to the pavement surface, and then the deflection is calculated from the difference between the slope of the beam and the slope of the pavement at the initial point and after the device moves to the next point. The trailer is also provided with a data-acquisition system used for data collection, storage, and analysis *[40]*.

Figure 5
Airfield rolling weight deflectometer prototype by Quest Integrated, Inc. *[40]*

A major benefit of using the ARWD is that appropriate counteractions may be used when the manufacturer considered that elements cause errors during testing. Consequently, the ARWD is expected to display a high-quality performance at low speeds. Conversely, considering the fact that the device was designed with a relatively low traveling speed, the problems of traffic mobility confusion and low productivity were revisited. Additionally, there is a significant loss of accuracy on highway curves, due to the required straight path *[40]*.

The Road Deflection Tester. In collaboration with the Swedish National Road and Transport Research Institute, the Swedish National Road Administration created the RDT in 1991, devising one of the earliest continuous deflection measurement devices, Figure 6. Since its introduction in 1991, the device went through several modifications concerning the truck size, loading, and data acquisition system to improve the accuracy of results *[43]*.

Figure 6
General overview of the RDT *[43]*

The current RDT is a Scania R143 truck and was developed in 1997, very similar to the present RWD. Two arrays of lasers with 10 non-contact laser sensors are mounted on the truck; the first is located 8.2 ft. behind the front axle; whereas, the second is located 13.1 ft. behind the rear axle. This enables the device to collect sufficient measurements outside and near the center of the deflection basin, Figure 7. The loading system of the RDT consists of the engine of the truck, as well as additional weight, to create a force of 8,000 to 14,000 lb. and numerous sensors are positioned in different places of the truck, which are expected to monitor its behavior during operation *[44]*.

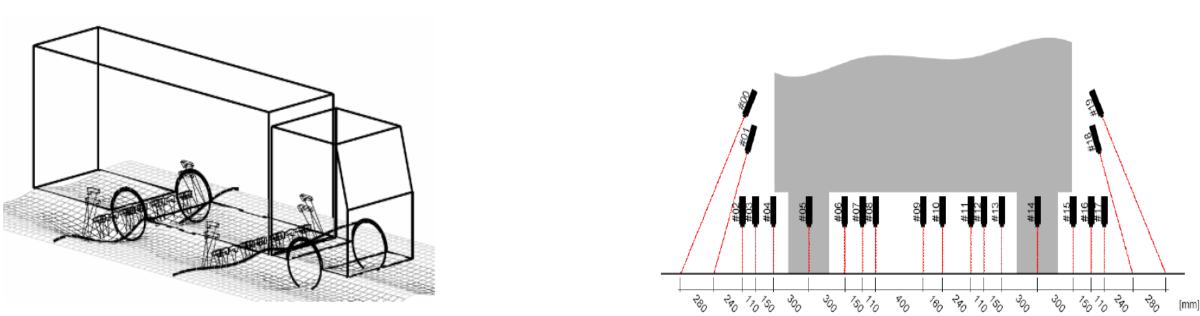


Figure 7
Schematic of sensor and wheel arrangements in the RDT *[44]*

The RDT is one of very few devices that provide a complete transverse deflection profile, rather than single deflection values; this profile could be useful in conducting a backcalculation of pavement layer moduli. In addition, the RDT takes into consideration testing conditions; thus, it is assumed that the travelling speed of the device is larger than the propagating speed of the loading waves through the pavement. Accordingly, the RDT, as a maximum deflection, is assumed to occur approximately 1.5 ft. behind the rear axle, where the load exists. However, this assumption should be investigated, due to the fact that the location of maximum deflection varies depending on the travelling speed, as well as the pavement stiffness. Results should be corrected due to this assumption. Furthermore, the system depends mainly on averaging numerous data points to minimize the error resulting from the vertical vibration of the truck *[40]*.

The Traffic Speed Deflectometer. To achieve traffic mobility and safety, Greenwood Engineering developed deflectographs with high traveling speeds in order to overcome problems in traditional, stationary-pavement deflection measurement devices *[22]*. The High Speed Deflectograph (HSD) is based on the Doppler technology, coupled with two laser sensors to measure the deflection velocity of the pavement surface, Figure 8. The deflection is equal to the difference between the deflected shape and the undeflected pavement as the truck travels. The trailer is capable of measuring pavement deflections at speeds up to 50 mph. The pavement is loaded through a wheel load of around 11,000 lb., with sensors to maintain a consistency in loading *[22]*.

Figure 8
Traffic speed deflectometer *[45]*

In 2006, Greenwood changed the name of HSD to TSD, which became commercially available. The light weight and intermediate length of the TSD trailer are excellent features that add more flexibility to the test. Additionally, the current version of the TSD has the potential to estimate the deflection velocity bowl, which enhances the accuracy of results. On the other hand, the device does not directly measure pavement responses or engineering properties as it measures the deflection velocity instead and calculates the deflection based on these measurements; this process might introduce an additional source of error during measurements or calculation.

Like other devices for continuous deflection measurement, the TSD is composed of a towing truck with a horizontal beam placed between the trailer axles, parallel to the direction of travel. A number of equally spaced, measuring laser sensors are fixed on the beam. A secondary measuring system is used for adjusting positions of the sensors and focus. The truck load is then transferred to the road surface through the tires, while traveling at a highway speed. The laser sensors direct laser rays to the pavement surface, and thus measure the velocity in the direction of the rays. The Danish highway M30 was tested in 2001 by the high speed deflectograph, at a 70-80 km/h traveling speed. The resulting data was compared with existing FWD data to determine testing reliability. Results showed that there was a significant deviation in the velocity measurements, which may be adjusted when compared to existing data from previous testing *[45]*. Table 1 presents a comparison between the different continuous testing devices.

Table 1
Comparison of Continuous Deflection Testing Devices *[40]*

Device	**Rolling Dynamic Deflectometer (RDD)**	**Airfield Rolling Weight Deflectometer (ARWD)**	**Road Deflection Tester (RDT)**	**Traffic Speed Deflectometer (TSD)**	**Rolling Wheel Deflectometer (RWD)**
Manufacturer	UT Austin	Dynatest Consulting and Quest Integrated	Swedish National Road Administration and VTI	Greenwood Engineering, Denmark	Applied Research Associates
Estimated Cost	N/A	N/A	N/A	$2,400,000	N/A
Operational Speed	3 mph	20 mph	60 mph	50 mph	20 to 60 mph
Applied Load	10 kips static + 5 kips dynamic	9 kips	8-14 kips	11 kips	18 kips (fixed)
Sampling Frequency	2-3 ft.	9 ft.	0.5 in.	0.8 in.	0.6 in.
Deflection Accuracy	0.05 mils	N/A	±10 mils	±4 mils/s	±2.5 mils
Number of Measurement Points	Up to 4	1	Up to 3	Up to 7	Up to 4
Comments	Very slow for network level testing	Unavailability of previous data	Unavailability of previous data	N/A	Appropriate for network level deflection testing

The Rolling Wheel Deflectometer

The RWD was developed by Applied Research Associates (ARA), Inc. to measure pavement surface deflections at traffic speeds and to characterize the load carrying capacity of in-service pavements. The first prototype was introduced in the late 1990s and was designed to perform measurements on airfield pavements at a maximum speed of 6 mph *[46]*. The latest version of the RWD was introduced in 2003 and can collect deflections at traffic speeds, see Figure 9. It consists of a 53-ft. long semitrailer applying a standard 18,000-lb. load on the pavement structure by means of a regular dual-tire assembly over the rear single axle *[47]*.

Figure 9
General overview of the rolling wheel deflectometer

The RWD measures wheel deflections at the pavement surface by means of a spatially coincident method, which compares the profiles of the surface in both undeflected and deflected states *[48]*. As the RWD travels on top of the pavement, triangulation lasers mounted, on a 25.5-ft. aluminum beam and placed at 8-ft. intervals, are used to measure surface deflections. The beam is mounted on the right side of the semitrailer to follow the right wheel path on the right lane, usually the weakest location on the pavement structure. Three spot lasers are placed in front of the loaded wheel to define the unloaded surface, and one spot laser is placed directly on top of the loaded dual-tire assembly in order to measure the deflected surface, Figure 10. The laser sensors are set to collect a reading at a fixed interval of 0.6 in. at all truck speeds. Prior to the field testing program described in this study, a more accurate and stable deflection measurement system, customized for pavement applications, was installed. The upgraded system has a 4-in. measurement deflection range and has an accuracy of ± 0.001 in. This study was the first testing program conducted with the new and improved laser deflection system.

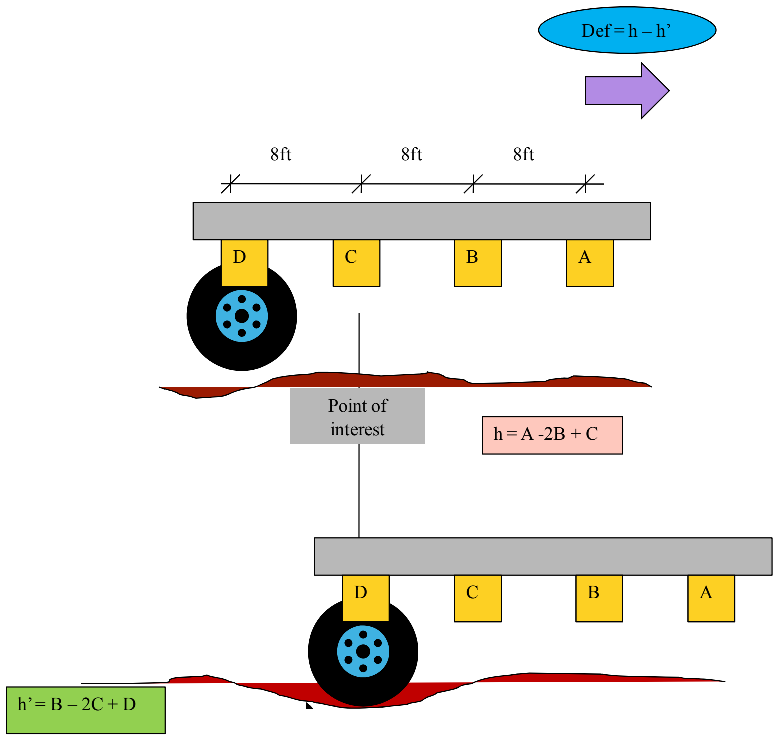


Figure 10
Illustration of the spatially-coincident method

States' Evaluation of Rolling Wheel Deflectometer. Over the past few years, the Federal Highway Administration (FHWA) conducted several field demonstrations in 13 different states to evaluate roadway pavements. In collaboration with ARA and state Departments of Transportation, testing programs were designed and implemented mainly on asphalt concrete (AC) pavements with a number of PCC pavements in some states. These testing programs were designed and implemented to introduce the RWD to transportation entities in each state. Although most of the testing programs have some similarities (i.e., testing and analysis procedures), numerous roadway conditions were tested in the different states. In addition, state agencies had different objectives for utilizing the RWD; for instance, New Jersey Department of Transportation (NJDOT) considered the inclusion of RWD data into its PMS database, while the Minnesota Department of Transportation (MnDOT) used RWD to determine seasonal load restrictions on its pavements. In addition, results were compared with available data from FWD testing to validate the reliability of using the RWD. A brief description of each testing program is provided.

California. In May 2006, ARA performed field testing over a four-day period using RWD in Sacramento, California. Eleven roadways from different categories were selected by the California Department of Transportation (Caltrans) and tested, inclusive of the interstate, U.S., and state routes. This process totaled 685 lane-miles of both AC and PCC pavements, eliminating irrelevant data (less than 5.0 percent), due to truck bouncing and pavement factors. Deflection values were averaged over 0.1-mile intervals and plotted to provide a deflection profile for each roadway. The results indicated the magnitude and variability of pavement response and stiffness.

RWD testing was conducted on multiple runs on selected sections to be compared with FWD data. Results demonstrated that lower-value deflections occur on interstate pavements, while higher deflections occur on the state highways. According to pavement conditions, results showed that pavements with higher distress amounts produced a high deflection, while pavements with lower distress amounts produced lower deflection magnitudes. RWD made multiple test runs at various speeds at specific locations to be compared with FWD data. The provided statistical summary from the report includes mean deflection values for all 24 pavement sections, which ranged from 5 to 24 mils, with the lowest deflections generally occurring on the thicker, higher volume, interstate pavements *[49]*.

Connecticut. A complete testing program was implemented by ARA in Connecticut on 26 different roadways, including interstate, U.S., and state highways. The whole testing program was performed in September 2007, over an eight-hour testing period. The total length of tested pavements was 212 lane-miles from the 26 roadways selected by the Connecticut Department of Transportation (ConnDOT). The RWD device was traveling with an operating speed ranging from 40 to 65 mph. The tested sites consisted of two-lane and multi-lane roads, mostly AC pavements with some composite pavements. Non-representative data, due to truck-bouncing and other factors, were removed (less than 1.0 percent). Deflection profiles for tested sections were provided to show the deflection magnitude and variability over each section length.

The mean deflection values ranged from 5 to 13 mils, suggesting that lower deflection values occur on thicker pavement sections, while higher deflection values occur on thin or structurally deteriorated roads. A structural rating was assigned by the ARA, based on representative deflections (which equal to mean deflection plus two standard deviations to account for structural variability within the pavement section). This rating resulted in the following classification for the 26 tested roads: (a) Fair: 23.1 percent, (b) Good: 61.5 percent, and (c) Very

Good: 15.4 percent. The study recommended enhancing RWD to provide longitudinal profiles, in order to help calculating the International Roughness Index (IRI) *[50]*.

Indiana. A study funded by the Indiana Department of Transportation (INDOT) and FHWA included an RWD testing program at three sites in Indiana. The test was performed in September 2004, over about 688 lane-miles, and with 18 test passes. The testing speed was around 55 mph, with a decrease up to 10 mph, due to the road geometry and the prevailing speed of trucks. Several parameters were collected along with deflection values, such as truck speed, pavement surface temperature, and GPS coordinates. During the analysis process, detailed layer characteristics, including layer thicknesses and types, were not available. Thus, RWD deflections were not corrected to a standard temperature. Based on the results, repetitive passes for the same location showed very good matching, with a slight deviation in deflection magnitudes due to temperature variations. The study recommended that the results could be included in current pavement management data, to benefit INDOT at the network level. In addition, development of a structural index, based on RWD results, was recommended *[51]*.

Iowa. ARA tested approximately 278 lane-miles in Iowa over a three-day period in July 2006. Thirty pavement sections in nine different roadways including AC, PCC, and continuously-reinforced concrete pavement (CRCP) pavements were tested. The Iowa Department of Transportation utilized pavement structure data, including layer thicknesses and types, in order to normalize deflections to a standard temperature of 68˚F. A negligible amount of data of around 5 percent was excluded due to truck and pavement factors. Representative deflection values ranged from 5 to 23 mils, which correspond to pavements with a high to medium structural capacity. RWD also collected continuous digital images of each tested road, while traveling at 55 to 65 mph.

The statistical analysis for the tested sections showed that sections with thicker pavements on the interstate had lower deflections and good uniformity. Additional testing using FWD was instantly performed at 0.5-mi intervals after RWD testing was conducted, and the FWD data then was normalized to the same standard temperature. A comparison between RWD and FWD data, provided by Iowa DOT, showed that the two devices have a very good correlation. The study recommended that enhancements could be applied to the current RWD to collect additional data, such as condition rating and IRI for different PMS activities *[52]*.

Kansas. The Kansas Department of Transportation (KDOT), in cooperation with ARA, designed a RWD testing program to be implemented in Kansas in 2006. The test included 17 roadways, with a total length of 506 lane-miles over a 3-day period. Different pavement sections produced mean deflections between 5 and 14 mils. When operated at an average speed ranging from 50 to 65 mph, RWD made 15 passes over certain sections, which were compared with FWD data collected by KDOT. The data showed very good agreement between FWD and RWD deflections *[53]*.

Minnesota. ARA tested a total of 21 AC pavement roads in District 3, Wright County, and McLeod County in Minnesota in September 2005. The 21 tested roads were subdivided into 33 individual pavement sections, according to distinct changes along the roadway length. Mean deflection values ranged from 7 to 29 mils, so sections were classified into three categories of (a) low deflection (< 10 mils), (b) medium deflection (10 to 20 mils), and (c) high deflection (20 to 30 mils). The Minnesota Department of Transportation provided pavement structural data, including FWD data, for the tested roads to be compared with RWD data. However, FWD data were collected in different years and under different climatic conditions. In addition, the likelihood of maintenance or rehabilitation applications on these roads was expected to affect the comparison between FWD and RWD data. Taking all these factors into consideration, the comparison showed that RWD and FWD data for all roads exhibited very similar trends *[54]*.

New Hampshire. During a three-day testing period in July 2008, thirteen asphalt concrete pavement roadways were tested in New Hampshire using RWD. ARA completed the testing program over approximately 648 lane-miles, which were selected by the New Hampshire Department of Transportation (NHDOT). RWD data, averaged over 0.1-mile intervals and plotted for each roadway, provided a deflection profile showing both the magnitude and the variability of deflection values. Mean deflections were found to range from 7 to 15 mils, and were used to determine the representative deflection for each section by adding 2 standard deviations. The representative deflection was used in a conceptual rating criterion, developed by ARA to classify the 25 road sections into fair, good, and very good pavements. Additional FWD data were collected on Interstate I-93 to be compared with RWD data collected for the same location. FWD data was normalized to a standard temperature and was found to be well-correlated with RWD deflections *[55]*.

New Jersey. The New Jersey Department of Transportation (NJDOT) selected 18 different roads, including multi-lane, rural, and urban undivided highways, to be tested using the RWD. ARA performed the testing program on the selected roadways in October 2005, including flexible and composite pavements for a total of approximately 800 lane-miles. The calibration process was applied to the RWD laser at the FAA Tech Center in Atlantic City before using the testing as a quality assurance procedure. Although a repeatability analysis was not performed in this project, all 18 roads were tested in both directions; the resulting mean deflections fell between 4 and 19 mils. Structural layer information was available through coring logs that were collected for the tested roads. Over the two years prior to RWD testing, NJDOT implemented

FWD testing on 14 of the 18 roads. A sample analysis between FWD and RWD showed a good agreement between both devices *[56]*.

New Mexico. RWD was utilized for the first time in New Mexico to perform a testing program on two four-lane highways of AC pavement: US 550 and US 70. In September 2008, the testing program, over 447 lane-miles, was conducted by ARA during a three-day period. As expected, lower deflection values corresponded to thicker, stronger pavements, while higher deflection values represented thinner or weaker pavements. Results showed a majority of deflection values ranging from 3 to 31 mils for the two locations. In collaboration with Dynatest, the New Mexico Department of Transportation (NMDOT) provided FWD results for the two tested roadways to be compared with RWD data from the testing program. RWD deflections were found to be slightly higher than FWD deflections, which may be due to different dates of collection and accordingly different weather and distress conditions *[57]*.

Kentucky-West Virginia- Ohio. ARA, in cooperation with the Kentucky Department of Transportation (KYDOT), the West Virginia Department of Transportation (WVDOT), and the Ohio Department of Transportation (ODOT) performed RWD testing on 11 highways in these three states. A total of 437 lane-miles were tested over a two-day period in September 2005. The resulting deflection values were normalized to a standard temperature of 68˚F, then averaged over 0.1-mile intervals and plotted to create a deflection profile for each highway. The mean deflections for homogeneous sections fell between 4 and 21 mils. ODOT provided FWD data for several pavement sections in Ohio to be compared with RWD results. This comparison showed similar deflection trends for the same road sections *[58]*.

Oregon. In May 2006, a testing program using the RWD was conducted on 14 roadways in Oregon. The tested roadways consisted of interstate, U.S., and state highways with AC pavement sections, with the exceptions of one roadway with PCC and another with a composite AC/PCC pavement. RWD measured continuous deflection profiles in the outer wheel path of the outer traffic lane for a total of 579 lane-miles. Less than 5 percent of the data was considered irrelevant, due to factors related to the testing process. Therefore, these data were removed before the analysis. Mean deflection values fell between 4 and 21 mils; lower deflections representing thicker pavements or pavements were in good condition, while higher deflection values represented thinner pavements or highly deteriorated pavements. In addition to the deflection profiles provided for each roadway, RWD was able to collect digital images for each roadway during testing. Provided FWD data for several test sections showed excellent correlation with RWD data *[59]*.

Texas. Sponsored by FHWA and the Texas Department of Transportation (TxDOT), an RWD research and testing program was conducted by ARA in College Station, Texas. The testing included six pavements representing a variety of pavement conditions, with a total of 38 individual test sections. The study evaluated the modifications applied to the RWD system after the limited field tests implemented in 2002, as well as measured the accuracy of RWD data by comparing the results with other devices, such as the FWD and the RDD. The TxDOT provided further data gathering for the testing location in order to obtain IRI, rutting, and texture, utilizing the TxDOT Modular Vehicle (TMV). A 100-ft. sample unit was selected for data averaging to reduce the random error for better data representation; the sample was found to be satisfactory for pavement management practices. A number of sections were selected specially for intensive RWD testing, due to their unique structural configuration. For example, a number of these sections include cement-treated bases. The MDD was used in these sections to provide reference deflection measurements for comparison with RWD deflections. Results showed that the first and second runs for each test session were often higher than the rest of the runs, indicating that there was a warming-up effect. The study concluded that RWD is a very useful tool at network level, yet considering that several aspects (such as thermal effects) should be investigated *[60]*.

Natchez Trace. In conjunction with the Eastern Federal Lands Highway Division (EFLHD) and FHWA Office of Asset Management, ARA implemented a deflection-based testing program in 2004, using the RWD at the Natchez Trace Parkway. Deflection data were collected in 30- to 40-mi segments. A total of 26 test passes were conducted over a five-day period to cover the entire length of both directions of the parkway. At the beginning of daily testing, a warm-up procedure was followed to ensure obtaining accurate results. The test design speed was 55 mph, with approximately a 5 to 10 mph variation, due to operating traffic or the existence of horizontal or vertical curves. During testing, automated digital images were incorporated in order to create a video of field description. Notes about the pavement surface were also made periodically. RWD was connected to a GPS device, such that the corresponding coordinates for each RWD reading were collected for geo-referencing of the test data. RWD propriety software was also used for data processing and calculating parameters, such as mean RWD deflection, mean surface temperature, and temperature variations.

Due to the unavailability of pavement layer information such as layer thicknesses and types, data were not normalized to a single temperature. As a result, due to high temperature fluctuations during the tests (from 40 to 93˚F), surface deflections followed a temperature-related trend. It was noted that most of the areas unaffected by temperature fluctuations were chip seal pavements, while areas where significant temperature fluctuations occurred were primarily HMA. Results showed that the mean value for the entire roadway is 25 mils, which are typical for thin and medium-thick AC layers over different subgrades. It was recommended, based on the testing results, that the RWD data could be used to develop tools such as a structural index, based on RWD deflection and other pavement information, which could aid pavement managers *[61]*. Table 2 presents a summary of the testing of RWD in the different states.

Table 2
Comparison between conditions and results of testing programs for different states

	State Reports	Date	Testing Period	Roadways	Tested Sections	Operating Speed	Speed Range	Total Lane-Miles
1	California	2006	4 days	11	24	55 -65 mph	< 5%	685
2	Connecticut	2007	8 hours	26	26	40 -65 mph	< 1%	212
3	Indiana	2004	—	9	3	45 -65 mph	30 -70 mph	—
4	Iowa	2006	3 days	9	30	30 -70 mph	30 -70 mph	278
5	Kansas	2006	3 days	17	—	50 -65 mph	30 -70 mph	506
6	Minnesota	2005	3 days	21	—	30 -70 mph	30 -70 mph	—
7	New Hampshire	2008	3 days	13	—	30 -70 mph	30 -70 mph	—
8	New Jersey	2005	7 days	18	—	—	—	—
9	New Mexico	2008	3 days	2	—	45 -65 mph	45 -65 mph	447
10	Kentucky-W Virginia-Ohio	2005	2 days	11	—	—	—	—
11	Oregon	2006	4 days	14	—	50 -65 mph	50 -65 mph	579
12	Texas	2003	7 days	6	—	5-65 mph	5-65 mph	264
13	Natchez Trace	2004	—	—	—	55 mph	55 mph	—

	State Reports	Deflections Range (mils)	Speed Range (mph)	Removed Data	Normalization Temperature	FWD Comparison	Repeatability
1	California	5 - 24	30 -70	< 5%	—	—	√
2	Connecticut	5 - 13	30 -70	< 1%	68˚F	√	—
3	Indiana	15 - 37	30 -70	—	—	—	√
4	Iowa	3 - 18	30 -70	< 5%	68˚F	√	—
5	Kansas	5 - 14	30 -70	< 5%	68˚F	√	—
6	Minnesota	7 - 29	30 -70	< 5%	68˚F	√	—
7	New Hampshire	7 - 15	30 -70	< 1%	68˚F	√	—
8	New Jersey	4 - 19	30 -70	< 5%	68˚F	√	—
9	New Mexico	3 - 31	45 -65	< 1%	68˚F	√	—
10	Kentucky- W. Virginia-Ohio	4-21	—	< 5%	68˚F	√	—
11	Oregon	4-21	50 -65	< 5%	68˚F	√	—
12	Texas	—	5-65	—	—	√	√
13	Natchez Trace	7-48	55	< 5%	—	—	—

"—" Information not available or analysis not conducted.

"√" Analysis was conducted.

Research Evaluation of the Rolling Wheel Deflectometer. Since its introduction in 1996, RWD has gradually transitioned from conceptual testing to real-world applications by incorporating its measurements into PMS activities at the network level *[3]*. Accordingly, a number of studies were conducted to evaluate RWD technology in the assessment of in-service pavements at the network level. Gedafa and co-workers evaluated the use of RWD for network deflection measurements in Kansas and compared these data to FWD deflection measurements *[2]*. The authors highlighted that remaining life models adopted in Kansas make use of a single input from pavement surface deflection, thereby requiring time-consuming measurements with FWD. The use of the RWD would allow for deflection testing at regular speeds without traffic protection or lane closure. Results of this study showed no significant difference between center deflections measured using the RWD and the FWD. Since the structural conditions of the pavement network did not change significantly over a four-year period, the authors recommended collecting RWD deflection data at four-year intervals.

For better distribution of limited funds, a study was conducted by Vavrik et al. to develop a methodology that incorporates RWD measurements into PMS activities *[62]*. In this study, the authors developed a PMS for Champaign County (Illinois) that incorporated deflection measurements obtained with the RWD to replace the dependency on the historically used engineering judgment approach. A treatment matrix, used as a decision-making tool, determines a recommended treatment method based on the Pavement Condition Index, traffic volume, and RWD deflection data. Based on this study, the authors concluded that RWD is an important component of the proposed PMS, as well as other pavement preservation programs, in order to avoid applying preventive maintenance treatments on pavements that are not structurally sound.

A study was conducted in Virginia to evaluate the effectiveness of RWD as a pavement structural assessment tool *[63]*. Deflection testing was conducted at three sites in order to evaluate the influence of pavement types, temperature, and surface texture on the repeatability of the measurements and their correlation to FWD. Results of this study indicated that while the range of RWD and FWD deflection measurements was similar, the results of both testing methods did not correlate well. In addition, RWD deflection measurements were not repeatable on all test sections. However, it is noted that FWD and RWD testing measurements were not conducted concurrently and were phased out by a five-month testing gap. As part of the Strategic Highway Research Program [SHRP 2 – RO6 (F)], a national effort is also underway to evaluate continuous deflection devices, including RWD, as a necessary non-destructive tool to achieve the motto to “get in, get out, and stay out.”

Herr and co-authors provided an overview of deflection measurement technologies, inclusive of the Benkelman beam, FWD, the deflectograph, and MDD *[64]*. The authors reported advantages and disadvantages for each device and listed a number of requirements for a new deflection measurement technology, such as a continuous measurement of deflection values, increased safety during testing, and a decreased operational cost. A detailed description of RWD elements and a deflection measurement procedure were also provided. It was concluded from the study that the aforementioned requirements for a new deflection measurement approach applies to the RWD since the device would maximize safety and provide cost-effective measurements. In addition, it was found that the RWD loading system has the potential to overcome limitations, associated with the fast transient load of the FWD and the assumptions in the FWD backcalculation procedure *[64]*.

Structural Assessment of Pavements

A roadway condition is described by a combination of two principal elements: the surface layer quality, which specifies the capability of the pavement structure to facilitate vehicle mobility, and the structural strength, which indicates the capacity of the pavement structure to withstand different traffic loads and various environmental conditions *[65]*. Thus, structural assessment of existing pavements provides vital information about current and future conditions of these pavements, which can help decision makers set maintenance strategies and allocate available funds *[66]*.

Structural Assessment Based on Visual Survey. Structural assessment, when based on a visual survey, consists of evaluating current pavement conditions in accordance with the surveyor's visual inspection. The assessment process depends on the type, extent, and severity of distress to decide which type of repair should be applied and to prioritize corrective actions. Occurrences are recorded in area or linear units, or number per station or slab, based on the distress type. Two types of a visual survey are conducted:

- At the network level. Recordings are usually captured for each 0.1-mi section, then summarized by a 0.5-mi section to be included in the PMS database.
- At the project level. Recordings are usually summarized over the entire length of the project. Project level surveys commonly require the survey to be conducted on foot to minimize subjectivity and, therefore, guarantee a higher level of accuracy, while specifying the type and extent of each distress type.

A decrease in subjectivity and variability during the inspection process of different distresses is recommended in order to obtain consistent and reliable results.

Structural Assessment Based on Destructive Testing. Trenching and coring are conventional methods that have been used for extracting undisturbed layer samples, inclusive of surface, base, subbase, and subgrade layers. Several characteristics may be obtained from extracted samples, including structural layer thicknesses and layer properties from laboratory testing conducted on the samples. Trenching and coring is also used to identify causes of severe distresses, such as rutting and cracking.

Structural Assessment Based on Non-Destructive Testing

Ground Penetrating Radar. GPR is a geophysical technology, used in the pavement practices as a method of pavement testing and evaluation *[67]*. The GPR technique is based on transmitting electromagnetic waves by means of short wave lengths to the pavement, Figure 11. The device antenna receives the reflected waves with a certain amplitude and travelling speed, depending on pavement layer thicknesses and material properties. Tests using GPR have no negative effects on the pavement structure and can be conducted at highway speed by mounting the device on a van. Thickness determination of existing pavement layers, employing the GPR, was recently standardized as an ASTM D – 4748.

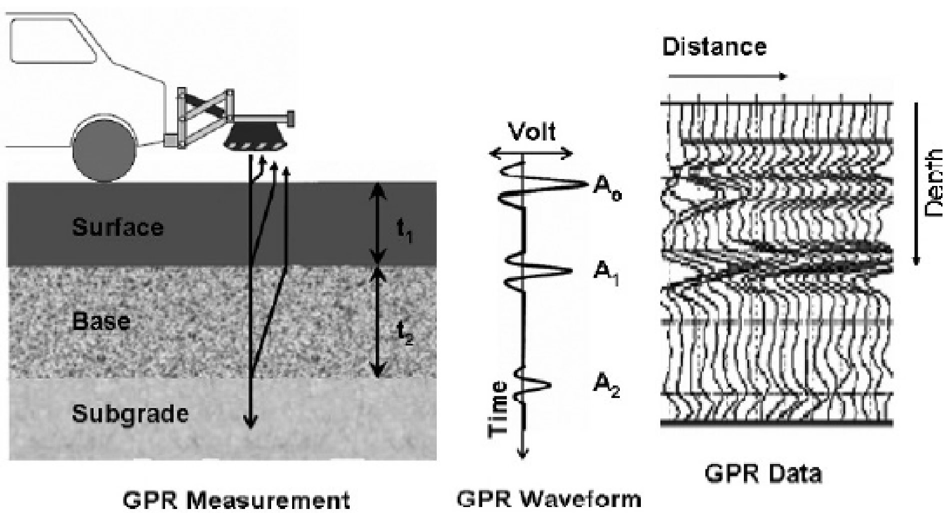


Figure 11
Ground penetrating radar mechanism *[67]*

Analysis Methods Based on Deflection Testing. A pavement SN has been widely utilized as a parameter for describing the structural capacity and quantifying pavement strength. Although a number of research studies have challenged the level of accuracy provided by the structural number approach, it has been adopted by many organizations and state agencies as the main criterion for design and evaluation of pavement structures *[68, 69]*. The structural number is obtained using layer thicknesses and material properties, determined from laboratory or field testing. The AASHO road test introduced the following expression for structural number calculation *[23]*:

$$SN = \Sigma a_i h_i \quad (1)$$

where,
SN = structural number;
a = material and layer coefficient; and
h= layer thickness (in.).

The Transport and Road Research Laboratory evaluated the structural number as an indicator of pavement strength *[70]*. A modified structural number SNC was proposed to include the variation in subgrade strength:

$$SNC = \Sigma\, a_i h_i + SN_{sg} \quad (2)$$

where,
SNC = modified structural number;
$SN_{sg} = 3.51(\log CBR) - 0.85\,(\log CBR)^2 - 1.43$; and
CBR = in situ California bearing ratio (percentage).

The 1986 AASHTO design guide for pavement structures introduced two approaches for structural number determination from deflection values obtained by using FWD. The first approach depends on performing a backcalculation of different layer moduli, and then layer coefficients are determined through the AASHTO procedure *[23]*. Several problems are associated with this approach although it is widely used by many highway agencies. For instance, one of these problems is a necessity to provide the exact measurements of layer thicknesses, which usually requires performing coring. In addition, this approach requires in-depth experience with the backcalculation procedure. The second approach, provided by AASHTO, is based on the deflection value from the outer sensor of the FWD, to be used to determine subgrade modulus. Then, using the maximum deflection and subgrade modulus, the structural number could be determined from the following equation:

$$D_0 = 1.5P/\pi a\ \{\ [(0.0045h^3)/SN^3] * [1\text{-}1/(1+(h/a)^2)^{1/2}] + 1/E_s(1+40000SN^2/a^2E_s^{2/3})^{1/2}\ \} \quad (3)$$

where,
D_0 = peak FWD deflection;
P = FWD load (lb.);
h= pavement layer thickness (in.);
a = load radius;
E_s = subgrade modulus; and
SN = structural number.

Several approaches were developed to determine structural capacity indicators. The analytical methods covered in this review are categorized as follows: (a) using the shape of deflection bowl, (b) backcalculation of moduli, and (c) impulse methods for near-field measurements.

Using Shape of Deflection Bowl. Findings indicate that the calculated structural number based on equation (1) lacks accuracy, due to the fact that the equation is based on Burmister's two layer theory. The theory assumes an infinite, linearly elastic subgrade, which in actual conditions usually lays over stiff layers or bedrock *[70]*. The magnitude of deformation that occurs within the pavement structure may be described by the following equation:

$$SIP = D_0 - D_{1.5Hp} \tag{4}$$

where,

SIP = structural index of pavement;

D_0 = peak deflection measured at offset of 1.5 times Hp under standard 40-kN (9,000-lb) FWD load; and

H_p = total pavement thickness.

To relate the structural index to the structural number, the following expression was developed:

$$SN = k_1 SIP^{k2} H_p^{\,k3} \tag{5}$$

where,

SN = structural number (in.);

SIP = structural index of pavement (µm);

H_p = total pavement thickness (mm); and

k_1, k_2, k_3 = coefficients as listed in Table 3.

Table 3

Coefficients of SN versus SIP relationship

Surface Type	k_1	k_2	k_3	r^{2*}	n**
Surface Seals	0.1165	-0.3248	0.8241	0.984	1944
Asphalt Concrete	0.4728	0.481	0.7581	0.957	5832

* Coefficient of Determination
** Sample Size

The best way to relay different conditions for the roadway segments is to plot surface deflection data collected from FWD testing. The plot demonstrates variations of pavement deflection along the roadway section under testing, and subsequently shows weak zones that correspond to relatively higher deflection values *[70]*. The inner sensor reading represents the entire pavement depth, while the outer sensor reading represents the deep layer response.

The *1993 AASHTO Guide for Design of Pavement Structures* provides three different methods for obtaining the pavement SN. One of these methods is called the non-destructive testing method (NDT) and depends on deflection values provided from conducting non-destructive testing *[71]*. It is hypothesized in this method that the structural capacity of the pavement structure relies greatly on both the total thickness and the overall stiffness. The relationship between SN_{eff}, thickness, and stiffness in the AASHTO guide is:

$$SN_{eff} = 0.0045 * h_p * (E_p)^{1/3} \tag{6}$$

where,
h_p = total thickness of all pavement layers above the subgrade, in.; and
E_p = effective modulus of pavement layers above the subgrade, psi.

The approach of using deflection data to determine the pavement structural number is necessary for evaluating existing pavements because the data convey structural adequacy or deficiency. However, a pavement structural number is not the only parameter to address the structural condition of the pavement; deflection data should be introduced with subgrade support and operating traffic loads in order to obtain a complete evaluation.

Backcalculation of Moduli. The second approach of deflection analysis methods is to perform a backcalculation of deflection values, which is a mechanistic evaluation. Backcalculation procedures depend on performing consecutive attempts to match deflection magnitudes, resulting from FWD testing, together with a calculated surface deflection response from an identical pavement with assumed layer moduli *[72]*.

Multiple iterations are usually performed in order to obtain an acceptable match between the assumed layer moduli in the backcalculation model and the measured moduli. The most widely used types for backcalculation are the following *[49]*:

- The first type is a traditional backcalculation technique that matches measured deflections against those calculated from theory. Some of the software programs that make use of this technique are EVERCALC, MODCOMP, and MODULUS.
- The second type is based on the equivalent layer method. BOUSDEF and ELMOD are examples of software programs using this approach.

The conventional backcalculation approach depends on utilizing conditions accompanied with deflection testing procedures, such as type of loading, plate geometry (in case of using FWD), and other conditions in order to create theoretical deflection basins. These created deflections are to be compared with the actual deflection values measured by the device and the error is calculated to ensure that it falls within the allowable zone. This process is repeated iteratively until the difference between the theoretical deflection and the measured one is insufficient, or until one of the layer moduli reaches a desired limit. Figure 12 shows a typical flowchart that represents the main elements in backcalculation programs *[73]*.

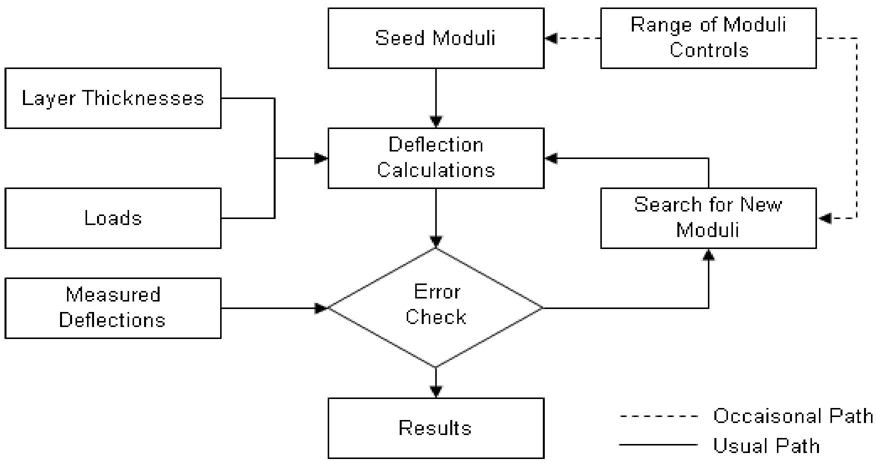


Figure 12
Backcalculation flowchart *[73]*

The following expression represents the root mean square error (RMSE), which is the main measure of convergence or goodness-of-fit between the theoretical deflection basin and the measured deflection:

$$RMSE = \sqrt{\sum_{i=1}^{n_d}\left[\frac{1}{n_d}\left(\frac{d_{ci} - d_{mi}}{d_{mi}}\right)^2\right]} \tag{7}$$

where,
n_d = total number of deflection sensors used;
d_{ci}= calculated pavement surface deflection at sensor i; and
d_{mi}= measured pavement surface deflection at sensor i.

Structural Assessment Based on Surface Curvature Index. TxDOT currently uses the statistical structural strength index (SSIF), originally developed by the Texas Transportation Institute (TTI). By using the FWD data stored in the Pavement Management Information System (PMIS), the SSIF is obtained from the following equation:

$$SSIF = 100*(SSI)^{1/(RF*TF)} \quad (8)$$

where,
SSI = structural strength index;
RF = rainfall factor; and
TF = traffic factor.

The structural strength index may be determined as the difference between deflections from the first and second sensors (W_1 and W_2) of the FWD, as follows:

$$SSI = W_1 - W_2 \quad (9)$$

It was found that the SSI does not offer enough accuracy to distinguish between pavement structures that have significant variation in terms of structural capacity. In addition, the SSI does not provide a direct relation between FWD data and the SN that is widely used for pavement design and evaluation *[30]*.

Interpretation of FWD Deflection Basins Using Mechanistic Approaches. Hoffman introduced a direct and simple method for structural evaluation of flexible pavements, based on FWD deflection measurements. Due to the difficulties in determining layer thicknesses, this approach focuses on determining the effective structural number (SN_{eff}) and the equivalent subgrade modulus without knowing pavement layer thicknesses *[74]*. By incorporating Hogg model parameters for a thin slab resting on an elastic foundation into the AASHTO equation, the following expression was introduced:

$$SN_{eff} = 0.0182*l_0*E_{sg} \quad (10)$$

where,
l_0 = characteristic length, in cm; and
E_{sg} = Subgrade modulus of elasticity, in MPa.

As depicted in equation (10), the effective structural number is related to l_0 and Esg. The characteristic length l_0 is determined from the following equation using FWD test results:

$$l_0 = A*e^{B*AREA} \quad (11)$$

where,
l_0 = characteristic length in cm;
AREA = deflection basin area, in in.; and
A, B = curve fitting coefficients.

Knowing the pressure on the FWD testing plate and the center deflection, the subgrade modulus may be determined from the following expression:

$$E_{sg} = m * \frac{p}{D_0} * l_0^n \tag{12}$$

where,

E_{sg} = Subgrade modulus of elasticity, in MPa;

p = pressure on FWD testing plate, in kPa;

D_0 = FWD deflection under loading plate, in μm; and

m, n = curve fitting coefficients.

OBJECTIVES

The ultimate goal of this study was to conduct a detailed field evaluation of the RWD system in Louisiana. Through this evaluation, the following objectives were achieved:

- Quantify the repeatability and the effects of testing speeds on RWD measurements;
- Study the relationship between RWD and FWD deflection measurements and pavement conditions; and
- Develop a simple model to estimate pavement SN from RWD deflection measurements.

SCOPE

To achieve the aforementioned objectives, the proposed research activities were divided in two phases. In the first phase, RWD and FWD measurements were collected, and measurements were used to assess the repeatability and characteristics of RWD measurements and the effect of the test speed on the measured deflection. In the first phase, a relationship between FWD deflection data and RWD measurements was also established. In the second phase, an analysis scheme was developed for using RWD measurements as a tool to identify structurally deficient pavements. Based on RWD measurements, SN and RI-based GIS maps were developed to identify structurally deficient pavements and to assess the overall condition of the pavement structure. The cost-effectiveness of the technology was investigated.

METHODOLOGY

Develop and Coordinate a Field Test Plan

A comprehensive field testing plan was conducted for collecting RWD and FWD measurements on selected flexible and surface treatments (i.e., chip seal) pavement test sites in District 05. Testing was coordinated through the PMS division of LADOTD. Selected test sites were representative of the pavement network in Louisiana in terms of pavement classification, design, and conditions. All sections were asphalt-surfaced, since the use of the RWD on concrete pavement surfaces has not been validated. However, composite sections with a concrete layer underneath the asphalt surface were included to evaluate RWD use for such pavement types. To assess the effects of vehicle speed on the measured deflection, RWD testing was conducted on the test sections at different speeds (e.g., 20, 30, 40, 50 and 60 mph). FWD tests were conducted at the same time of RWD testing on the outer wheel path. Temperature was recorded in conjunction with each test. To assist in the analysis, pavement designs of the selected sites were obtained using cores and by reviewing construction documents. In addition, the test plan included supportive measurements, such as roughness, pavement temperature, and distress survey for the selected sites.

RWD Description

As mentioned in the literature review section, RWD is a pioneer device for cost-effective measurements of pavement deflection and surface properties. The most recent version of the RWD was developed by ARA in collaboration with FHWA Office of Asset Management. It consists of a 53-ft. long semitrailer applying a standard 18,000-lb. load on the pavement structure by means of a regular dual-tire assembly over the rear single axle *[47]*. A general view of the 53-ft. custom designed RWD trailer is shown in Figure 13. The trailer is specifically designed to be long enough to separate the deflection basin, due to the 18-kip rear axle load from the effect of the front axle load. In addition, the trailer can accommodate the aluminum beam so that the laser range needed to tolerate any bouncing of the trailer during operation could be minimized.

Beam deflection system Cooling and loading system

Figure 13
General overview of the rolling wheel deflection system

The latest version of the RWD, which was introduced in 2003, can collect deflections at traffic speeds. Several modifications and upgrades were introduced to the RWD with respect to the laser sensors, data acquisition system, and software. The laser collection system was moved between the tires, and a new procedure was introduced for laser calibration. The laser sensors are set to collect a reading at a fixed interval of 0.6 in. at all truck speeds. Prior to the field testing program described in this study, a more accurate and stable deflection measurement system customized for pavement applications was installed. The upgraded system has a 4-in. measurement deflection range and has an accuracy of ± 0.001 in. This study was the first testing program conducted with the new and improved laser deflection system. In the new system, four

Selcom Model SLS 6000 laser triangulation sensors are mounted at approximately 3.6 ft. above the roadway surface with a 4-in. measurement range. The laser sensors work simultaneously to determine pavement deflections under the wheel load, with one sensor placed between the dual tires to determine the maximum deflection (Figure 14). Two additional sensors are placed in front of the wheels to measure a secondary pavement deflection.

Figure 14
Laser sensors placed between the dual tires

The whole system of the beam and laser sensors is isolated in a thermal-chamber to prevent external factors, such as wind and temperature fluctuation, from affecting the measurements during testing. The rear axle and wheels were designed and placed to prevent any conflict with laser paths. A two-person crew, driver, and operator is sufficient to perform the entire test as the RWD enables the operator to control the sensors, as well as to collect and store the data, through the use of a computer in the tractor. RWD is also equipped with a GPS for geo-referencing as well as an infrared thermometer for measuring surface temperature of the pavement.

RWD and FWD Comparison. Table 4 compares the operating conditions of RWD and FWD in structural evaluation of in-service pavements. As shown in this table, major differences exist between RWD and FWD-deflection determination methods, especially with respect to the load setup. While RWD applies a three-dimensional transient wheel load on the pavement surface, resulting in both vertical and horizontal stresses, FWD applies a vertical load pulse over a circular plate *[47]*. Although pavement deflection data differs in magnitude and shape, both methods are expected to exhibit similar trends, thereby providing a comparable assessment of pavement structural integrity. This is due to the fact that both methods are based on the same concept: thin, distressed, and soft pavements exhibit greater deflections than thick, stiff pavements.

Table 4
Comparison of Operating Conditions for the RWD and the FWD

Factor	**Rolling Wheel Deflectometer**	**Falling Weight Deflectometer**
Operational Speed	Traffic Speed	Stationary
Deflection Sensor Accuracy	0.246 mils	0.01 mils
Number of Operators	2	1
Productivity [mi/day]	100 to 200	2.5 to 25
Number of Sensors	1 to 2	3 to 9
Applied Load (lbs.)	9,000[1]	5,850 to 18,000
Load Type	Transient wheel load	Impact circular plate

[1] per dual wheel assembly

Field Testing Program

RWD Testing. The complete field testing program requested by LADOTD consisted of two phases. In the first phase, the complete asphalt road network (about 1,250 miles) in District 05, referred to as network sites, was tested using the RWD deflection system based on ARA standard testing protocol. LTRC also selected 58 sections to be tested using FWD. In the second phase, 16 road-sections (1.5 miles each), referred to as research sites, were selected by LADOTD and LTRC and used for a detailed evaluation of the RWD technology as shown in Figure 15.

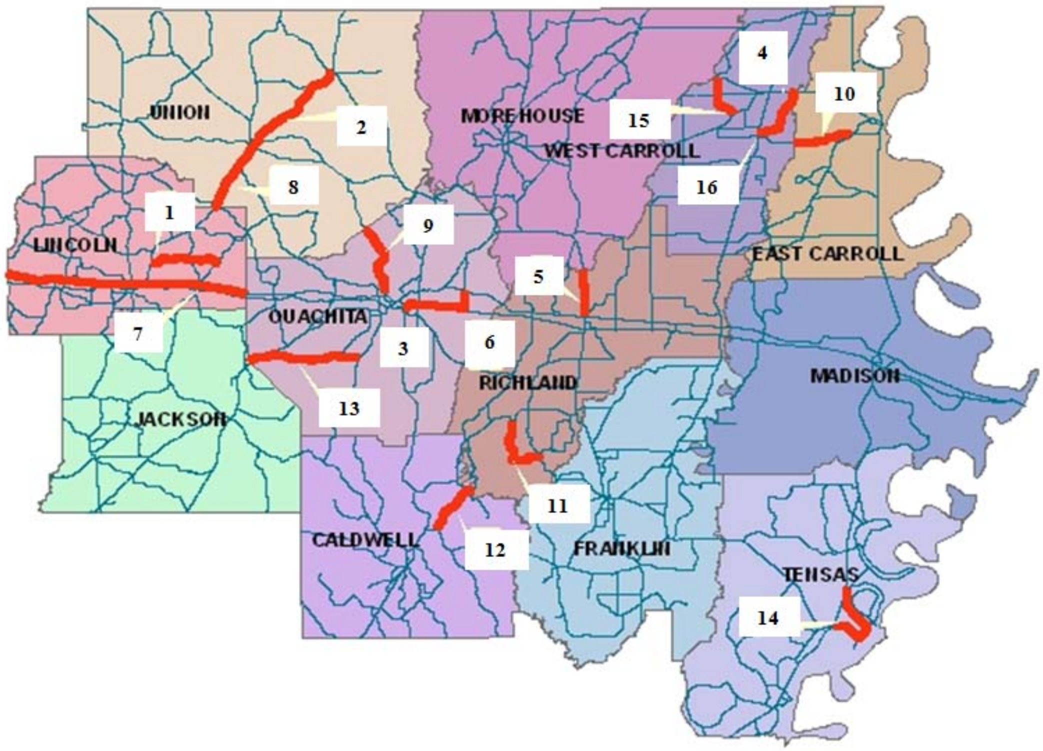


Figure 15
Locations of the 16 research sites in District 05

In addition to RWD testing, the test plan in Phase II included conducting FWD testing on selected flexible and surface treatment pavement test sites. The testing plan specified that FWD testing should be conducted within 24 hours following completion of RWD testing on the selected sites in order to maintain the same testing conditions. The field testing program for RWD and FWD was conducted successfully in December 2009 with no major problems during the course of the experiment. Table 5 provides a summary of the pavement sections selected for testing in the second phase, i.e., the research sites.

Table 5
General description of the 16 research sites

ID	Control Section	Route	Parish	Pavement Type	Type of Treatment	Last Rehabilitation Date	State Project Logmile Limits	RWD Test Site Logmiles	Traffic
1	831-05	LA 821	Lincoln	Asphalt	A3- Asphalt Overlay	2/2/2009	0.0 - 4.5	2.0 - 3.5	1,185
2	069-03	LA 33	Union	Asphalt	A7- AC Surface Treatment	9/15/2006	0.0 - 6.8	3.0 - 4.5	2,585
3	326-01	LA 594	Ouachita	Asphalt	A3- Asphalt Overlay	5/1/2006	0.0 - 3.6	2.0 - 3.5	6,398
4	862-14	LA 589	W. Carroll	Asphalt	A5- AC Overlay	11/18/2009	3.5 - 7.5	4.0 - 5.5	244
5	071-02	US 425	Richland	Asphalt	A5- AC Overlay	7/11/2008	0.6 - 5.5	1.0 - 2.5	3,490
6	326-01	LA 594	Ouachita	Asphalt	Z1- RCND AGGR SURF	8/4/2003	3.8 - 9.3	5.0 - 6.5	5,623
7	451-05	I-20	Lincoln	Composite	A3- Asphalt Overlay	6/7/2005	21.3 - 27.3	23.5 - 25.0	29,357
8	069-02	LA 33	Union	Asphalt	M3- Micro-Surfacing	12/6/1996	0.0 - 5.7	2.0 - 3.5	6,409
9	315-02	LA 143	Ouachita	Asphalt	A3- Asphalt Overlay	8/3/2004	3.7 - 9.0	6.0 - 7.5	3,859
10	333-03	LA 582	E. Carroll	Asphalt	ZA- Asphalt Pavement Rehabilitation	5/15/2003	0.0 - 6.9	3.0 - 4.5	650
11	341-01	LA 576	Richland	Asphalt	A1- New AC	5/1/1966	4.5 - 8.1	4.9 - 6.4	800
12	166-01	LA 133	Caldwell	Asphalt	A1- New AC	2/4/1986	3.9 - 6.1	4.0 - 5.5	1,898
13	067-08	LA 34	Ouachita	Asphalt	A3- Asphalt Overlay	—	4.3 - 13.3	5.5 - 7.0	7,017
14	020-30	LA 128	Tensas	Composite	A3- Asphalt Overlay	—	0.0 - 3.6	1.5 - 3.0	2,963
15	332-01	LA 878	W. Carroll	Surface Treatment	A3- Asphalt Overlay	1/15/2009	0.0 - 3.3	.5 - 2.0	424
16	862-14	LA 589	W. Carroll	Surface Treatment	A5- AC Overlay	9/10/2008	0.0 - 3.5	1.0 - 2.5	244

"—" Not Applicable

Continued

ID	Surface Type	Base Type	Layer Thicknesses (in.)			Surface Layer Group	Distresses	Comments	IRI [2009]	PCI [2009]	Condition (PCI)
			Layer 1 (Surface)	Layer 2 (Base)	Layer 3 (Subbase)						
1	Asphalt	Granular	5	8	0	Medium	Stripping	HMA thickness is greater than criteria	145	81	Fair
2	Asphalt	Granular	7	3	8	Thick	—	—	89	92	Good
3	Asphalt	St-Sand	4	7.5	0	Medium	—	Base is stabilized	55	99	Very Good
4	Asphalt	St-Sand	1.75	15.75	0	Thin	—	—	N/A	N/A	Very Good
5	Asphalt	St-Sand	8.5	8.5	0	Thick	—	HMA thickness is greater than criteria	64	98	Very Good
6	Asphalt	St-Sand	8.5	8	19.5	Thick	Fabric	HMA thickness is less than criteria	61	99	Very Good
7	Asphalt	PCC	4	13.5	2	Medium	Fabric	HMA thickness is greater than criteria	71	97	Very Good
8	Asphalt	Granular	7.25	16.75	0	Thick	—	—	149	70	Fair
9	Asphalt	St-Sand	9.5	7.5	2	Thick	Separation	Base is stabilized	57	99	Very Good
10	Asphalt	Granular	9.5	8.5	0	Thick	Stripping	—	179	64	Poor
11	Asphalt	St-Sand	4	7	0	Medium	Stripping	—	309	57	Poor
12	Asphalt	St-Sand	6	8	0	Medium	—	—	192	77	Fair
13	Asphalt	St-Sand	8.5	9	0	Thick	—	—	99	87	Good
14	Asphalt	PCC	7.25	8	0	Thick	Stripping	HMA thickness is greater than criteria	202	63	Poor
15	Surface treatment	St-Sand	1.25	11.75	0	Thin	—	Base is stabilized	202	82	Fair
16	Surface treatment	St-Sand	1.5	7	0	Thin	—	Base is stabilized	257	60	Poor

"—" Not Applicable

To assess repeatability and the effects of truck speed, triplicate runs were performed at different speeds of 20, 30, 40, 50, and 60 mph. However, the test speed was restricted by the posted speed limits on a number of sites. Only Site 7 was selected on the interstate highway system I-20, which permitted testing at 60 mph. However, testing at 50 mph was conducted on 8 of the 16 sites. Road segments were also selected to represent different pavement conditions as described by the PCI, with varying HMA thicknesses and base types. Traffic volume widely varied in the selected sections from an annual average daily traffic (AADT) of 244 to 29,357; these traffic volumes range from low to heavy. Table 6 presents a summary of RWD testing on the research sites as reported by ARA, Inc. Table 7 summarizes the locations that had data removed, as reported by ARA, Inc.

Pavement temperature was recorded in conjunction with each test. The pavement surface temperature ranged from 29.3 to 69.8°F with an average temperature of 48.2°F during the entire testing process. To assist in the analysis, pavement design of the selected sites was obtained using cores and construction documents. Figure 16 shows the coring location for research site 12, while Figure 17 shows the core sample for the same location, which provided accurate information about layer types and thicknesses. In addition, the test plan included supportive measurements, such as roughness, pavement temperature, and distress survey for the selected sites.

Figure 16
Coring test location – Site 12

Figure 17
Core sample – Site 12

Table 6
Summary of RWD testing at different speeds

Site No.	Speed, mph					Comments
	20	30	40	50	60	
1	✓	✓	✓	✓		55 mph speed limit.
2	✓	✓	✓			55 mph speed limit. Top speed limited by 40 mph curve at start.
3	✓	✓	✓			Posted speed limit is 45 mph.
4	✓	✓	✓			Top speed limited by road conditions (curves and rolling terrain).
5	✓	✓	✓	✓		55 mph speed limit.
6	✓	✓	✓	✓		55 mph speed limit.
7	✓	✓	✓	✓	✓	65 mph speed limit.
8	✓	✓	✓	✓		55 mph speed limit.
9	✓	✓	✓	✓		55 mph speed limit.
10	✓	✓	✓			Top speed limited by road conditions (horizontal curves).
11	✓	✓				Top speed limited by road condition (roughness).
12	✓	✓	✓			Top speed limited by road condition (roughness).
13	✓	✓	✓	✓		55 mph speed limit.
14	✓	✓				Top speed limited by road condition (roughness).
15	✓	✓	✓	✓		55 mph speed limit.
16	✓	✓				Top speed limited by road conditions (curves and roughness).

Table 7
Non-representative data removed from the files

Site No.	Logmile From	Logmile To	Event
1	2.25	2.3	Bridge
	3.4	3.5	Vertical curve (lasers out of range)
3	2.55	2.725	Traffic lights (speed reduced)
	3.15	3.5	Wet pavement
5	2.15	2.25	Bridge
7	24.25	24.625	Wet pavement (60 mph runs only)
9	6.85	6.975	Bridge
11	5.1	5.15	Bridge
	6.075	6.125	Bridge
13	5.9	6	Horizontal curve
	6	6.225	Horizontal curve
	6.725	6.775	Bridge
14	2.175	2.4	Bridge
15	0.925	1	Bridge
	1.575	1.675	Bridge and wet pavement
16	1.8	1.925	Horizontal curve
	2.025	2.15	Horizontal curve

FWD Testing. Nondestructive FWD deflection testing was conducted to measure the load response characteristics of the pavement layers and subgrade. Deflection testing was performed in accordance with ASTM D 4694, "Standard Test Method for Deflections with a Falling Weight-Type Impulse Load Device" and D 4695, "Standard Guide for General Pavement Deflection Measurements." The FWD device shown in Figure 18 was configured to have a 9-sensor array, with sensors spaced at 0, 8, 12, 18, 24, 36, 48, 60, and 72 in. from the load plate. Three load levels of 9,000, 12,000, and 15,000 lb. were used in the FWD deflection-testing program. The FWD testing was conducted at a frequency of 0.1 mile with the testing locations selected in the middle of the average interval used in RWD testing. FWD tests were conducted at the same time of RWD testing on the outer wheel path.

Figure 18

Illustration of the FWD test device used in the testing program

Data Processing and Filtering. During RWD testing, laser deflection readings are measured at 0.6-in. intervals. Irrelevant data such as measurements collected on top of a bridge, sharp horizontal and vertical curves, and at traffic signals were removed. Erroneous data may also be obtained if the pavement surface is wet or in areas with severe cracking at the pavement surface. Valid deflection measurements were then averaged for two primary reasons: (a) minimizing the truck bouncing and vibration effects on the measured deflections and (b) decreasing the data to a manageable file size. After the averaging process is complete, deflections are normalized to a standard temperature of 68˚F for sound comparison between data collected at different times of the day. Figure 19 presents the raw data collected on Site 9 by the four laser sensors.

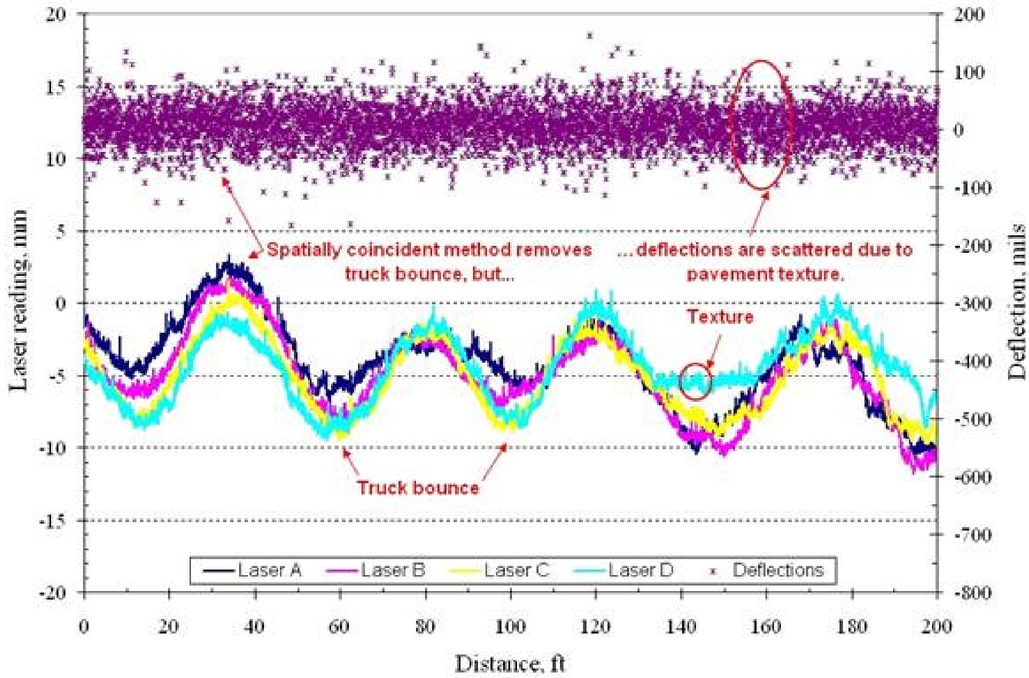


Figure 19
Example of individual laser readings and deflections for Site 9 (315-02), LA 143 north of West Monroe (after ARA, Inc.)

DISCUSSION OF RESULTS

Quantification of Repeatability and Effects of Test Speed

RWD measurements were analyzed to develop graphs illustrating the variation of deflection data within the different test sections. These graphs were used to identify homogeneous sections and areas of distresses on the selected sites and to illustrate the benefits of the technology at the network level. In addition, collected RWD measurements were compared to FWD deflection data and a statistical analysis of variance (ANOVA) was conducted to determine whether the two sets of measurements were equivalent or statistically different. Selected test sites were compared individually (FWD vs. RWD) and based on a global comparison of all test sites for the two measurement methods. Collected data were correlated to roughness measurements, and the dependencies of RWD data on speed, temperature, and design characteristics were evaluated.

Data Processing and Filtering

As previously mentioned, the spatially coincident method was used to process raw data, which were collected at approximately 0.6-in. intervals. Typically, an averaging length of 0.1 mi. (528 ft.) is used for data averaging, which corresponds to 10,728 readings per average. This interval is selected to reduce the standard error of the mean to be within ± 1 percent of the global mean. The research team also evaluated an averaging interval of 0.025 mi for the 16 research sites in Louisiana, representing 2,682 readings in each interval. However, the resulting standard deviation of mean was around ± 2 percent, almost double the single result from the 0.1-mi interval. Therefore, the 0.1-mi interval was used in this study. To illustrate the effects of averaging, the data for one run of the RWD on Site 9 was averaged at three different intervals of 33, 132, and 528 ft. Figure 20 illustrates the effect of the averaging interval on reducing data scattering. As shown in this figure, data scattering due to truck bouncing and vibrations may be controlled by increasing the averaging interval.

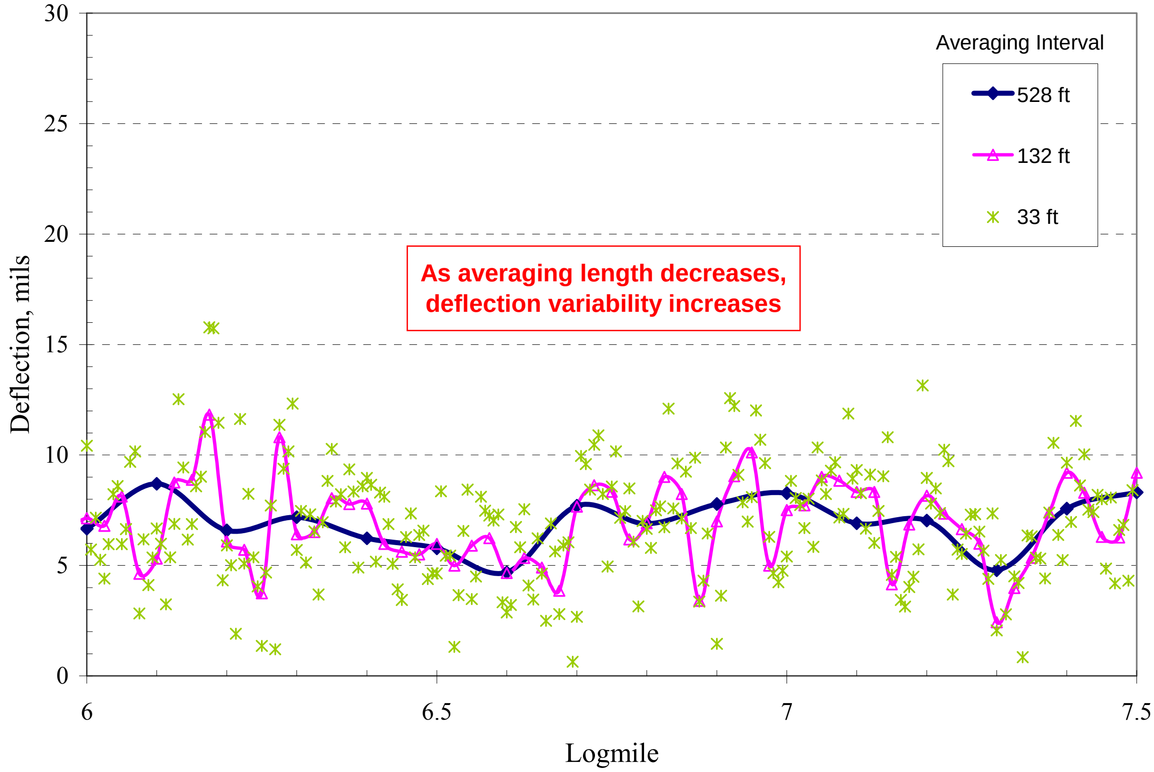


Figure 20
Effect of averaging interval on deflection variability – Site 9

Figure 21 presents the variation of the mean standard deviation with the averaging interval length. As shown in this figure, variability in the measurements decreases rapidly with an increase in the averaging interval length, until it reaches a near-asymptotic level. Based on the results shown in this figure and in order to minimize the effects of truck vibrations on the measured deflections, individual readings were averaged every 0.1 mi. This corresponds to the average of 10,728 individual readings for each 0.1-mi test interval. The averaging process reduces the error in the individual measurements, caused by bouncing and random vibrations of the truck, to within ± 1 mils of the interval mean *[75]*. Based on this approach, Figures 22 and 23 present the processed data for the sensor right on top of the wheel (D_0) and the sensor 18-in. from the load (D_{18}) for Sites 2 and 12, respectively.

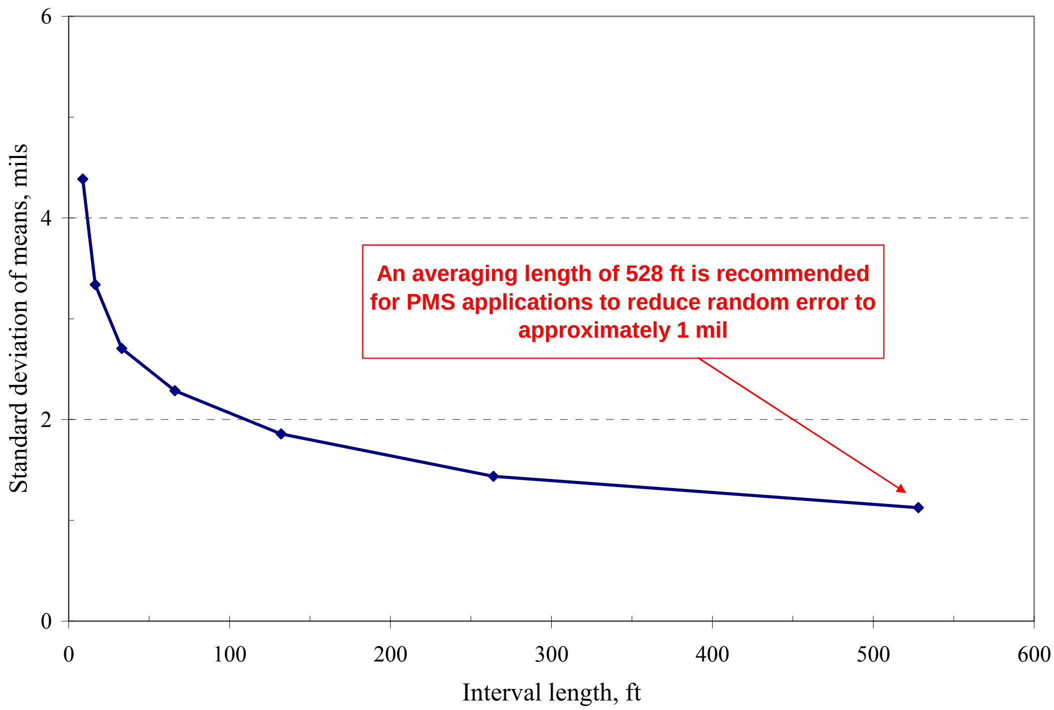


Figure 21
Random error in average deflections with the averaging interval – Site 9

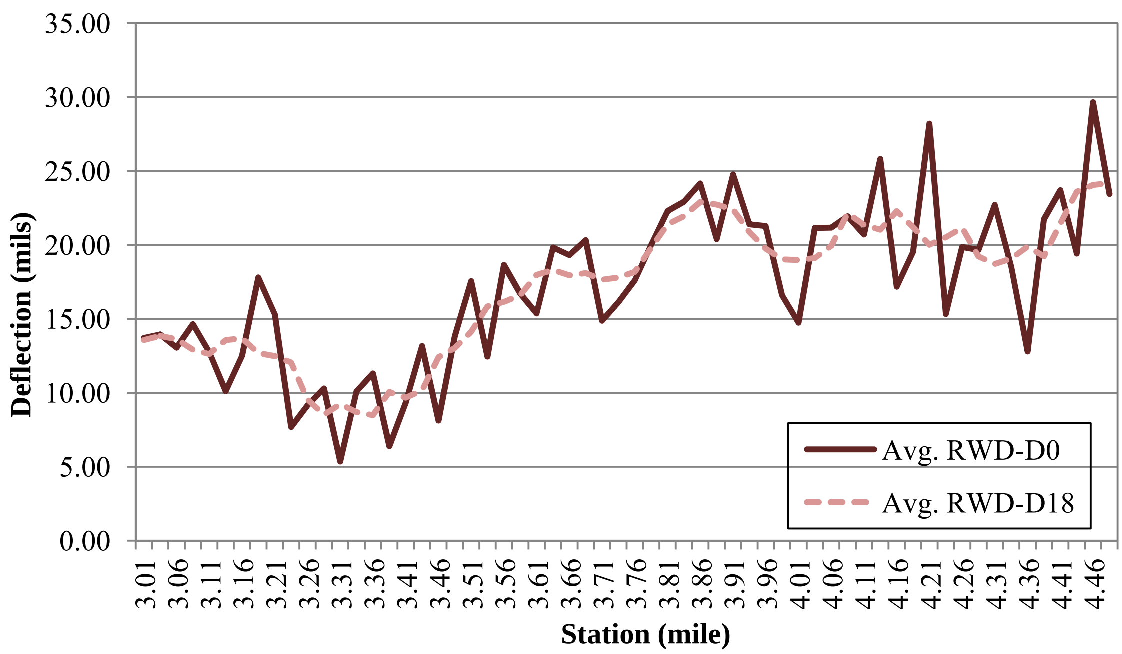


Figure 22
Typical RWD deflection profile - Site 2

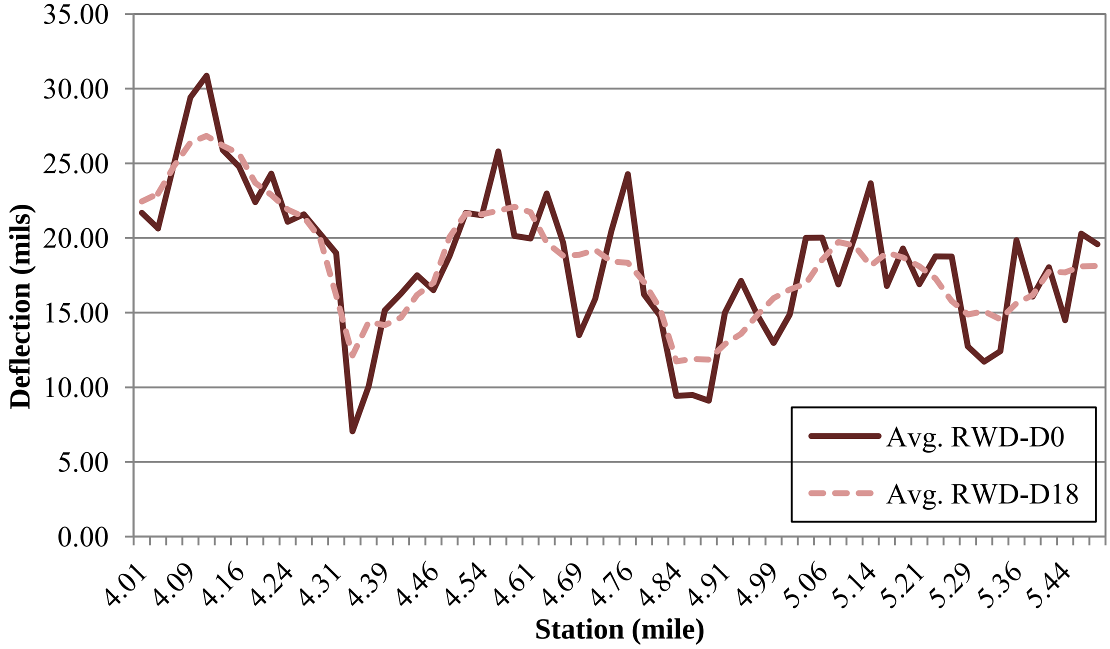


Figure 23
Typical RWD deflection profile - Site 12

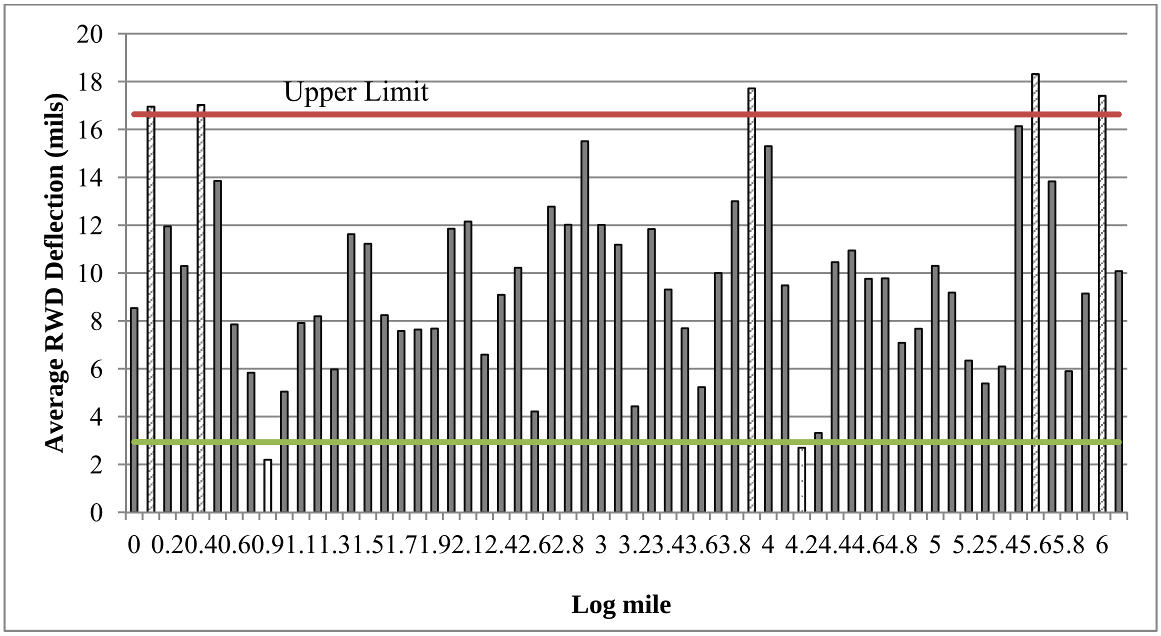
Further evaluation of RWD data collected on the network sites revealed that RWD deflection data would occasionally exhibit a sudden drop in the magnitude of deflection. These drops in deflection were not indicative of a change in design or pavement structural conditions. The research team contacted ARA, Inc. to get their feedback on this trend. According to the consultant, these sudden drops in deflection may be caused by an equipment factor such as the lasers penetrating cracks, which may affect deflection calculations. Examples of this trend are presented in Figure 24 (a to e) for five test sites. To address this trend, deflection measurements that did not fall between an upper limit of 170 percent of the average and a lower limit of 30 percent of the average were assumed to be outliers and were removed as described by the following equations, see Figure 24:

$$\text{Upper limit} = \text{average} + 0.7 * \text{average} \tag{13}$$

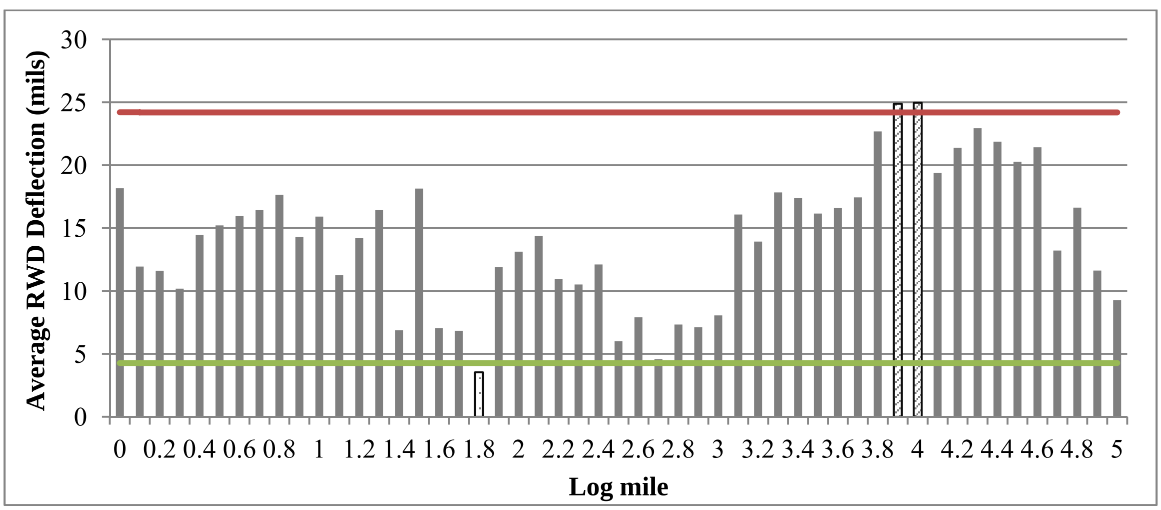
$$\text{Lower limit} = \text{average} – 0.7 * \text{average} \tag{14}$$

where,
average = average deflections computed over the control section.

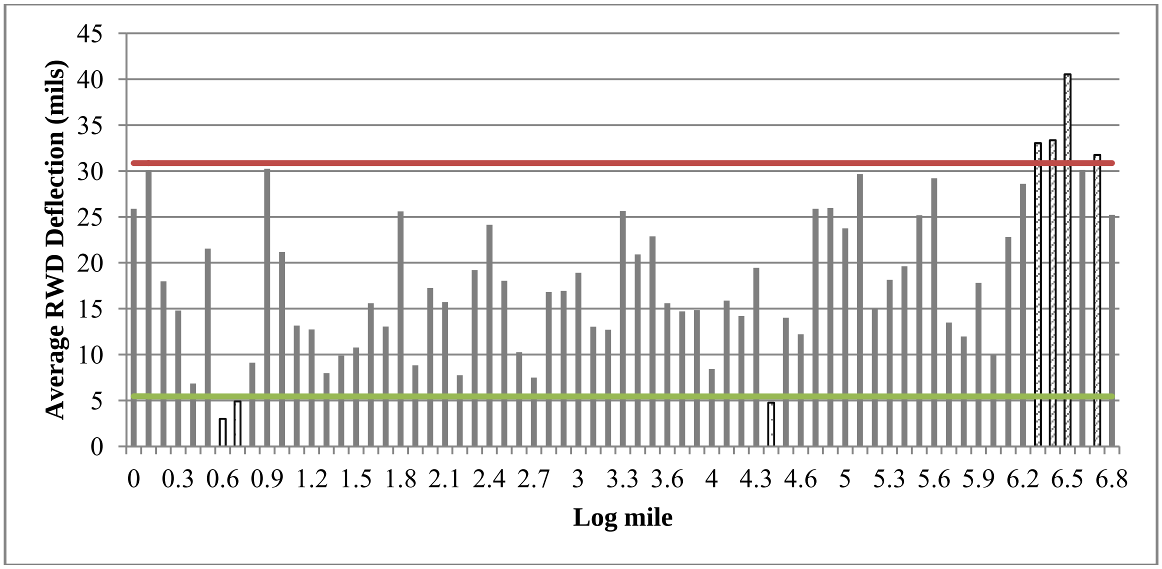
The research team recommends this practice for routine RWD testing conducted by LADOTD to ensure that deflection measurements are representative of actual pavement conditions. Table 8 presents the number of sites that required correction according to equations (13) and (14).



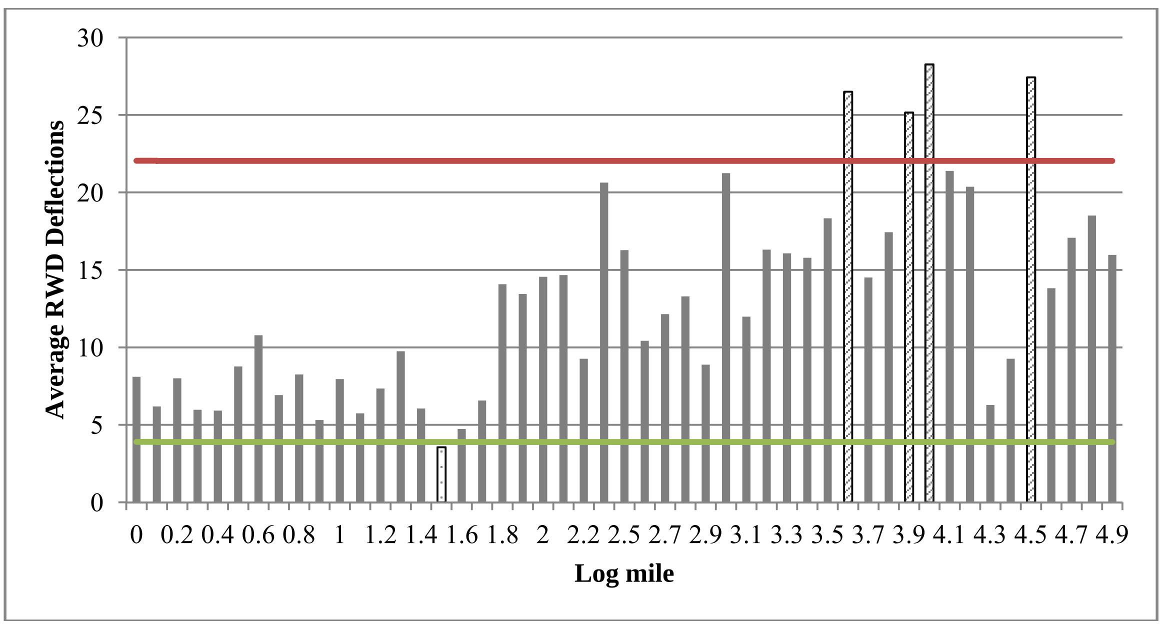
(a) Section 089-06



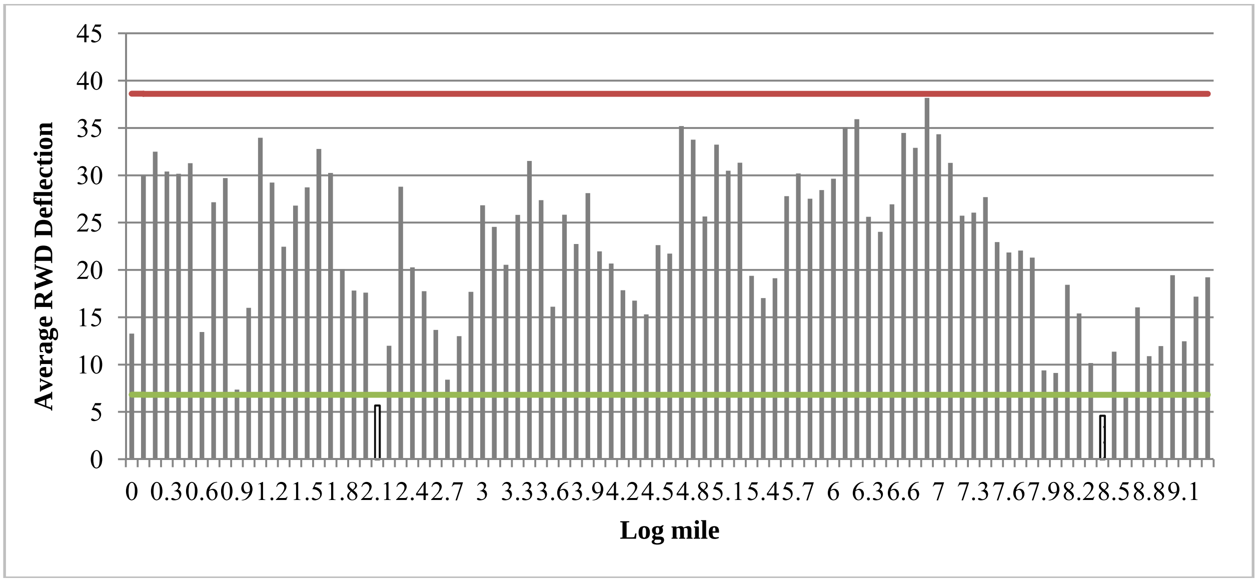
(b) Section 166-03



(c) Section 333-03



(d) Section 308-07



(e) 167-04

Figure 24
Processing of RWD deflection data to identify outliers

Table 8
Number of sites with necessary correction

Description	**Number of sites**	**Number of sites with correction**
Research sites	16	1
Validation sites	58	6
Network sites	220	42

Effects of Temperature Variation on RWD Deflection Measurements

The RWD has the capability of measuring pavement surface temperatures during operation utilizing an infrared thermometer. Testing runs were conducted over a wide range of pavement temperatures from 35 to 69°F, see Table 9. Hence, temperature correction was necessary to control the effects of temperature on the measured deflections. Surface deflections were corrected for variation in pavement temperature by shifting the measurements to a standard temperature of 68°F, using the BELLS and the AASHTO 1993 methods. This process was conducted using additional information, such as the previous day's mean air temperature and surface layer thicknesses. The same procedure was also used to correct FWD-deflection data. A comparison between uncorrected and corrected average RWD deflections for Sites 2 and 12 is presented in Figures 25 and 26, respectively. For all sites, corrected deflection profiles were shifted upwards by a magnitude varying between 0 and 15 mils.

Table 9
Avg. air temperature and avg. pavement surface temperature during testing

Test Date	Avg. Air Temperature	Avg. Pavement Temperature
12/2/2009	58	51
12/3/2009	51	49
12/4/2009	45	46
12/5/2009	41	59
12/6/2009	48	53
12/7/2009	61	52
12/9/2009	61	57
12/10/2009	48	51

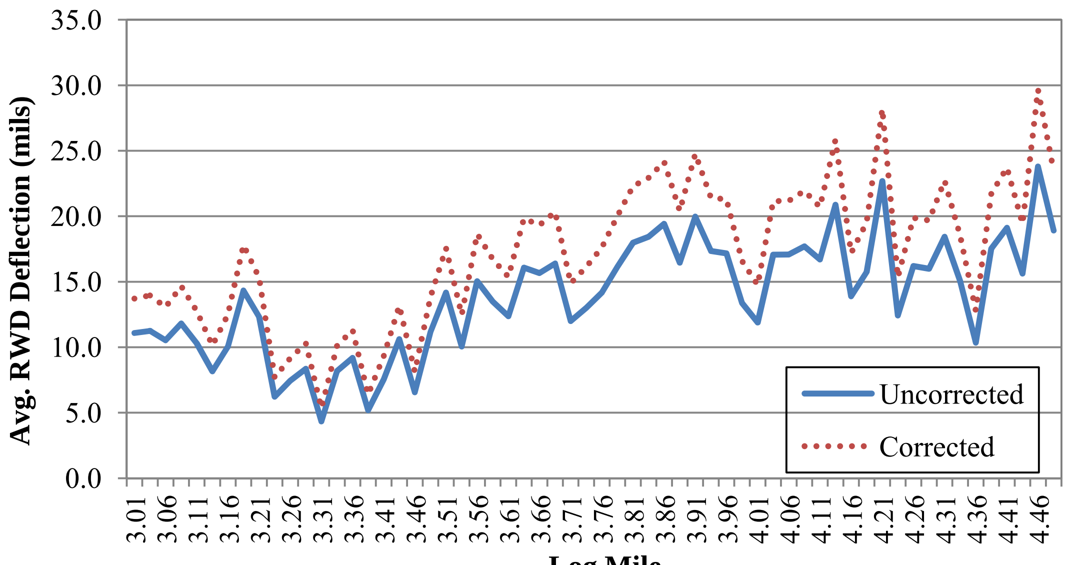


Figure 25
Comparison between uncorrected and corrected deflection values for Site 2

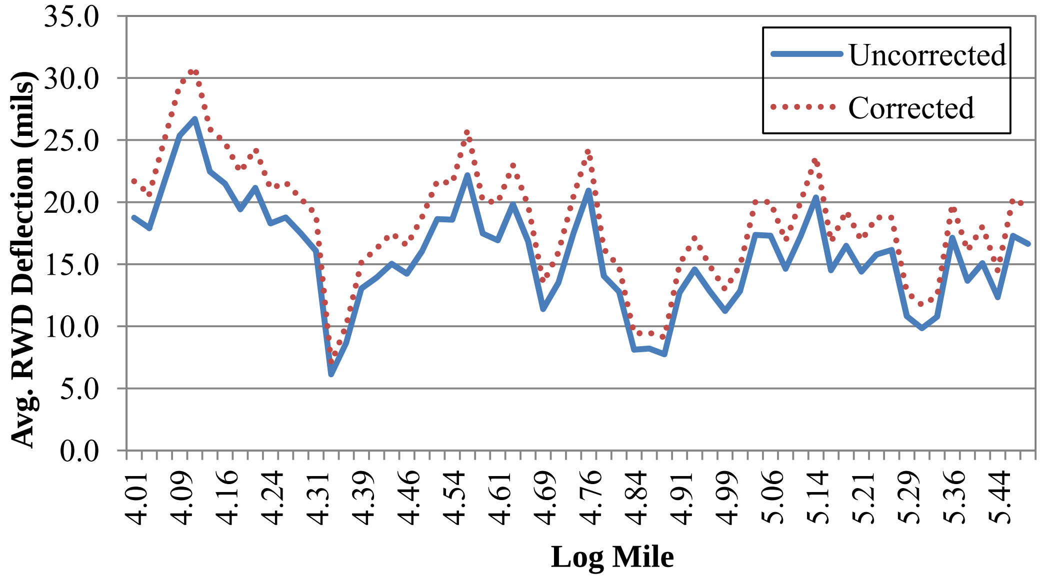
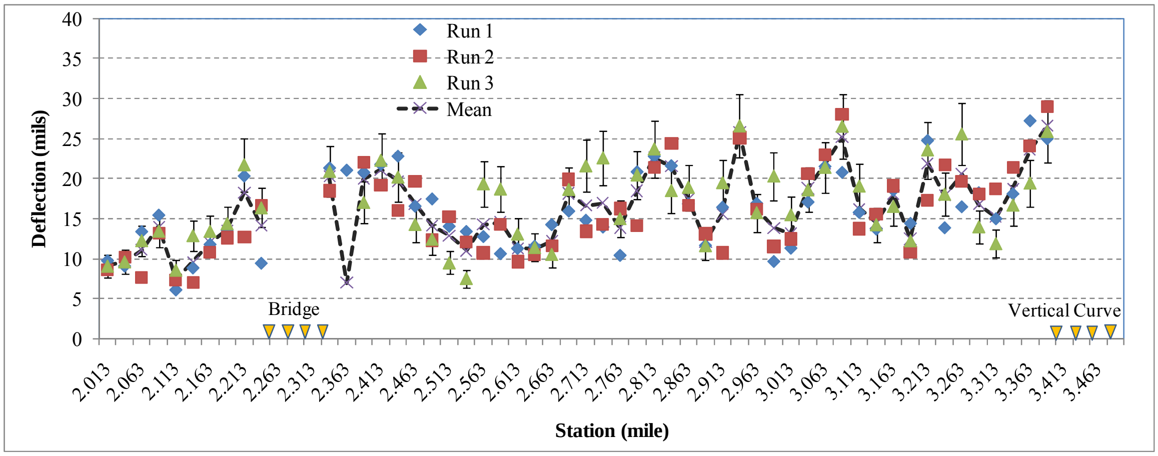


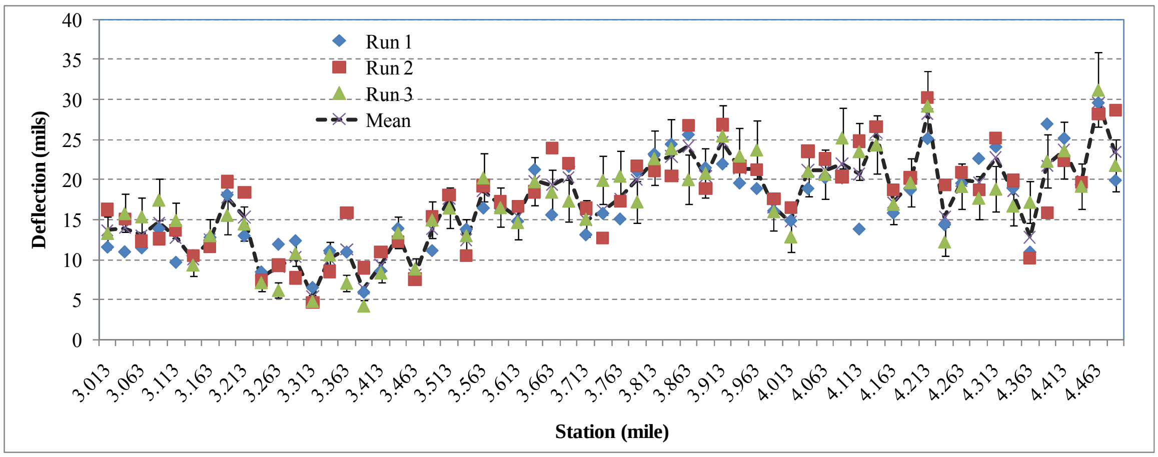
Figure 26
Comparison between uncorrected and corrected deflection values for Site 12

Repeatability Analysis

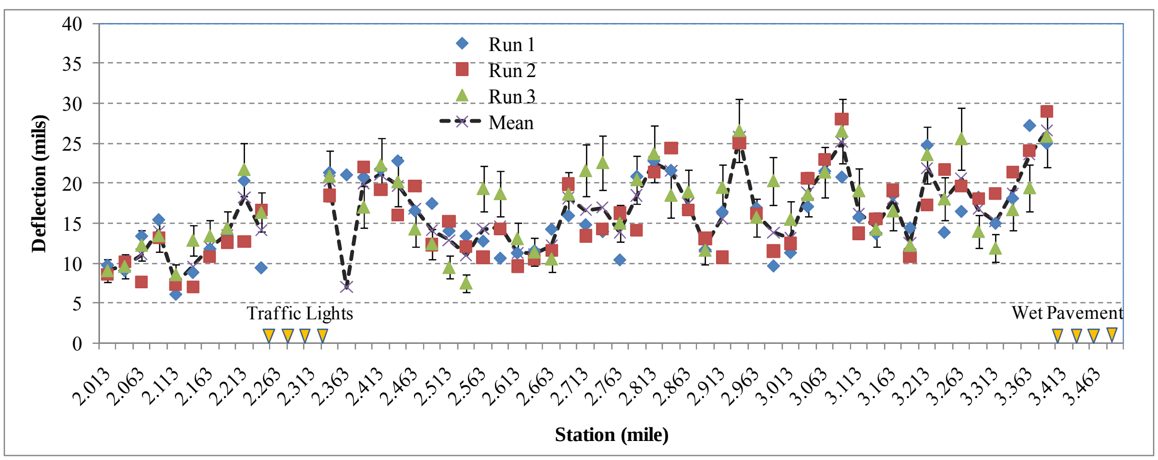
Figure 27 (a through p) illustrates the measured pavement surface deflections for the 16 research sites. In these figures, the individual runs are presented, together with the average of the measurements. Error bars representing the estimated variations from the mean values are also shown in this figure. As shown in Figure 27, measurements are more scattered in sites with poor conditions than in sites in relatively good condition at the time of the survey. The uniformity of the measurements through the length of the test section is also evident in sites in good pavement condition.



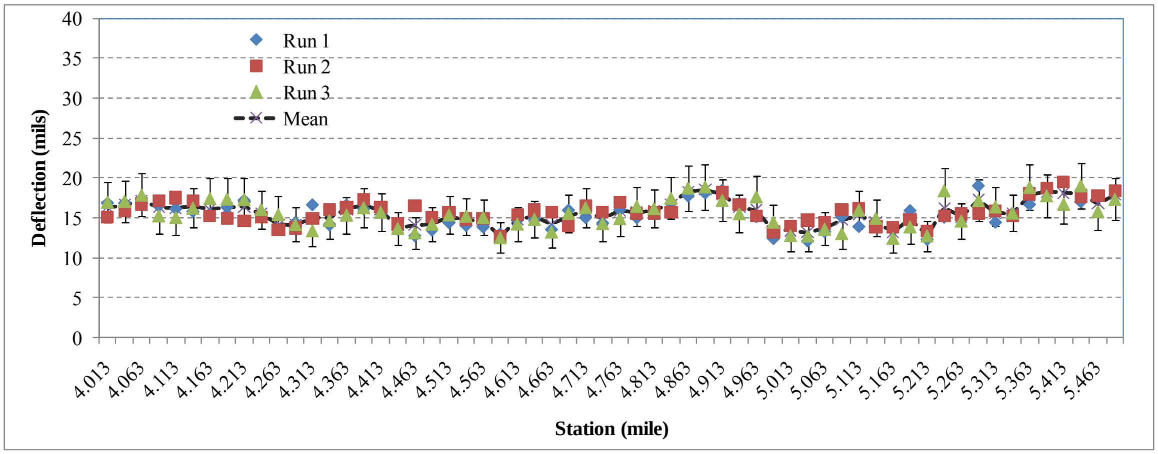
(a) Site 1 (PCI = 81)



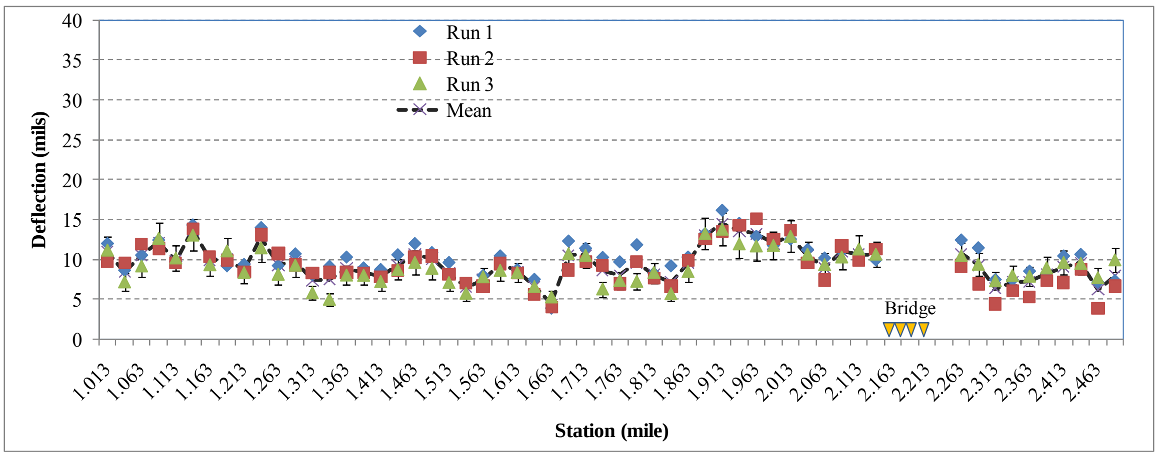
(b) Site 2 (PCI = 92)



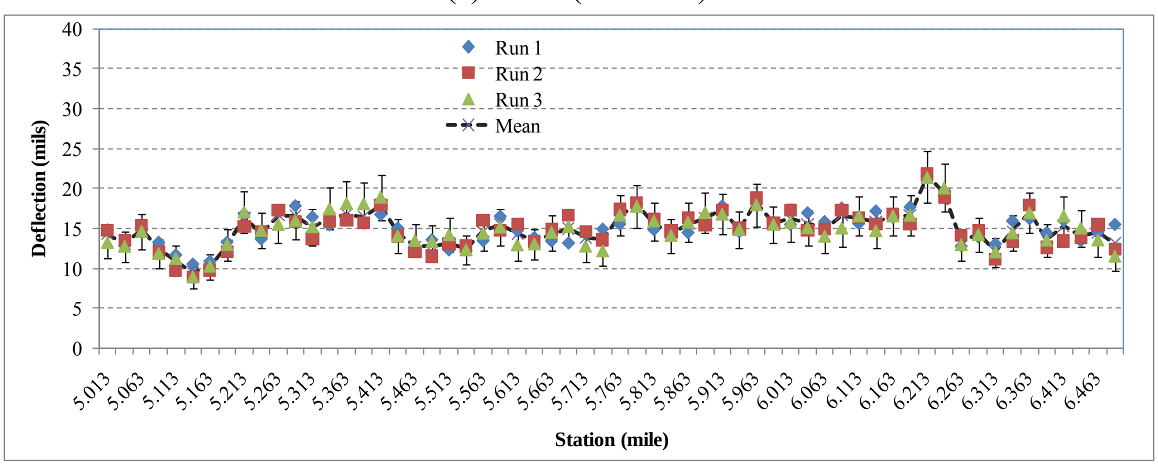
(c) Site 3 (PCI = 99)



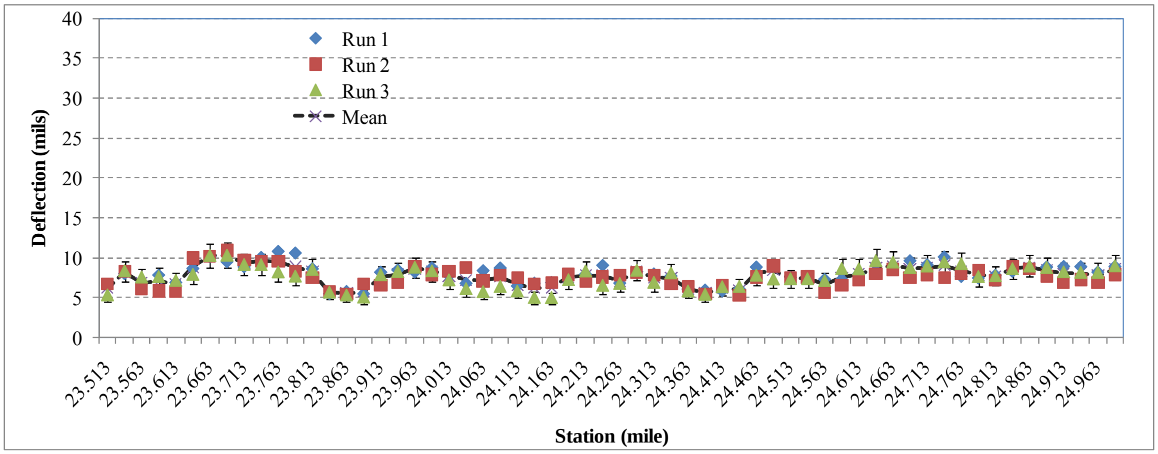
(d) Site 4 (Newly Constructed)



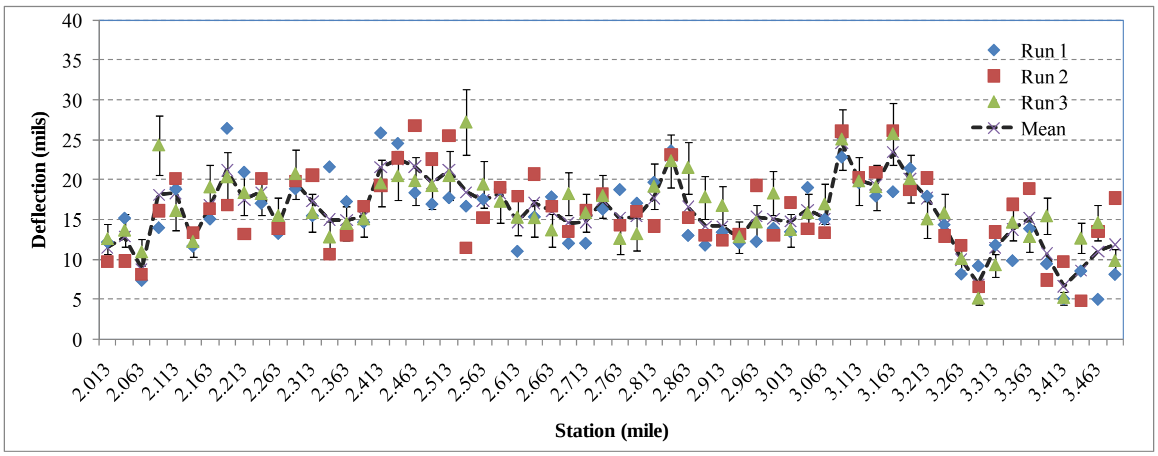
(e) Site 5 (PCI = 98)



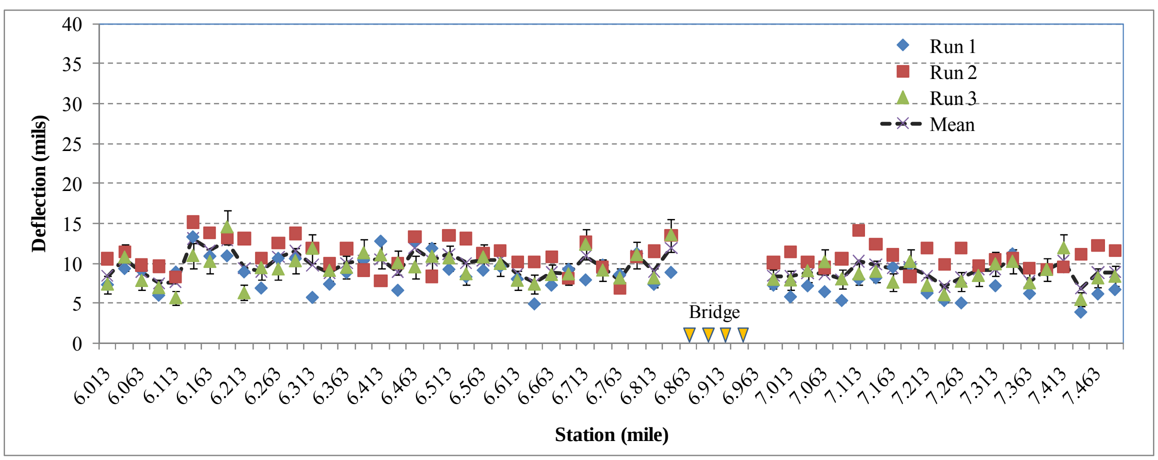
(f) Site 6 (PCI = 99)



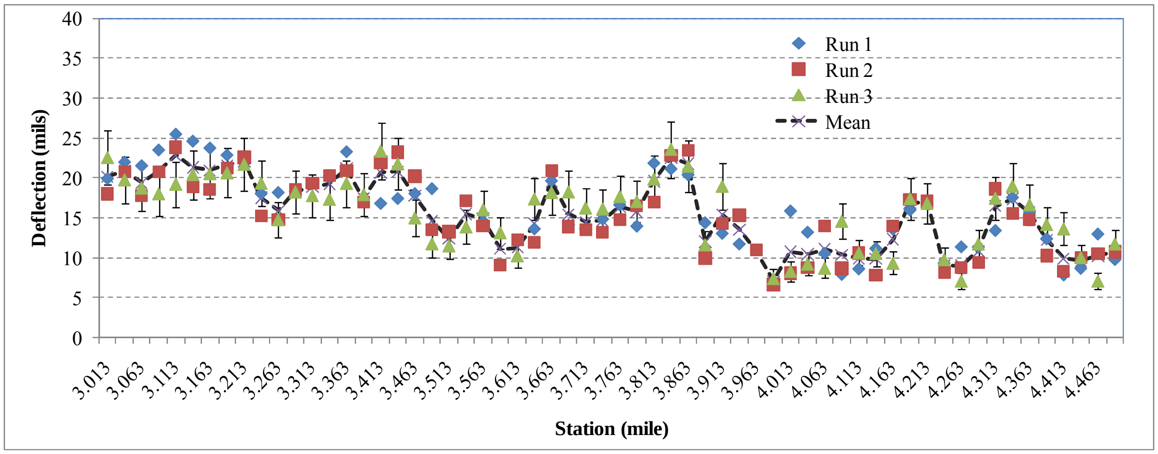
(g) Site 7 (PCI = 97)



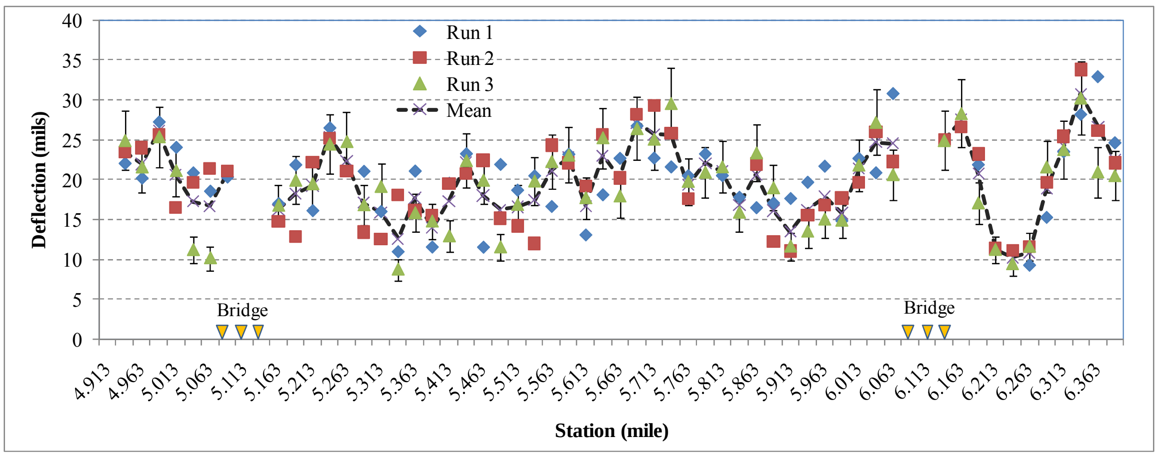
(h) Site 8 (PCI = 70)



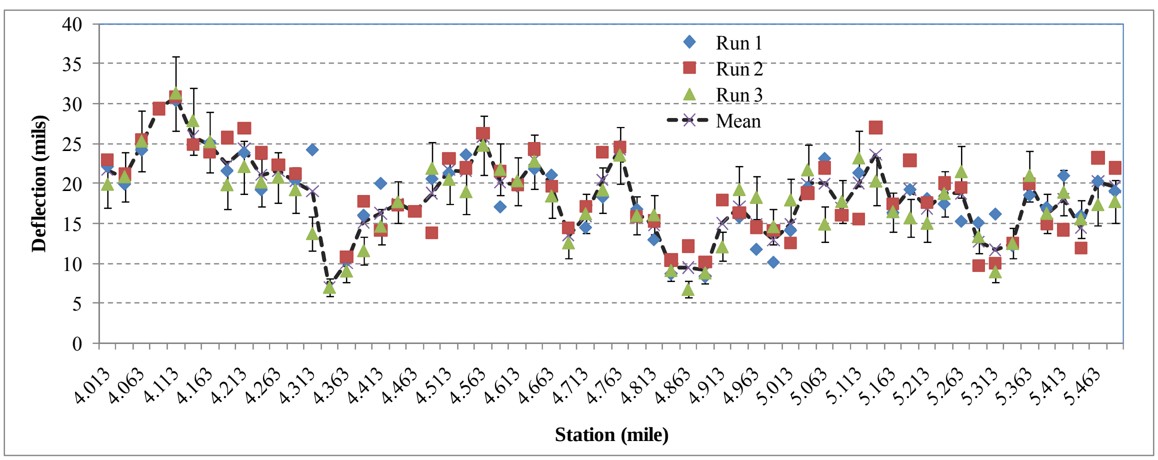
(i) Site 9 (PCI = 99)



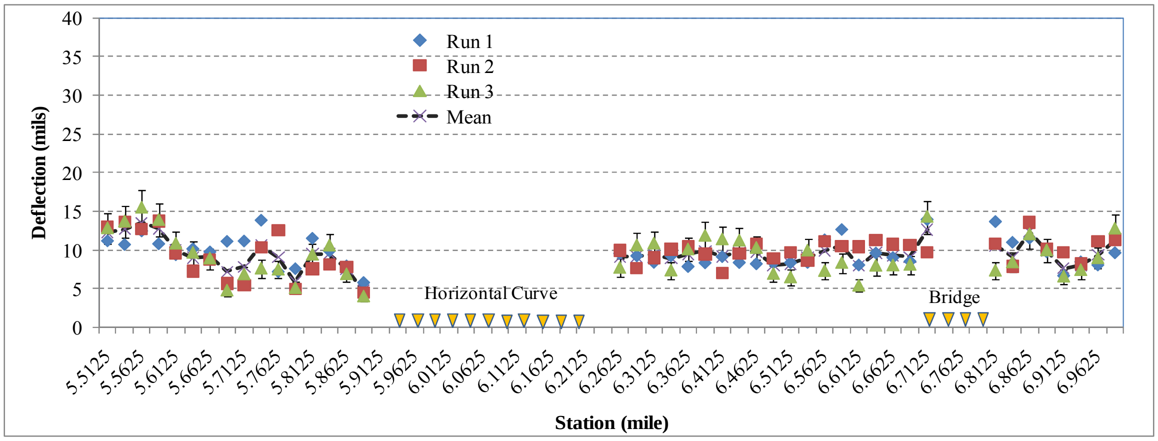
(j) Site 10 (PCI = 64)



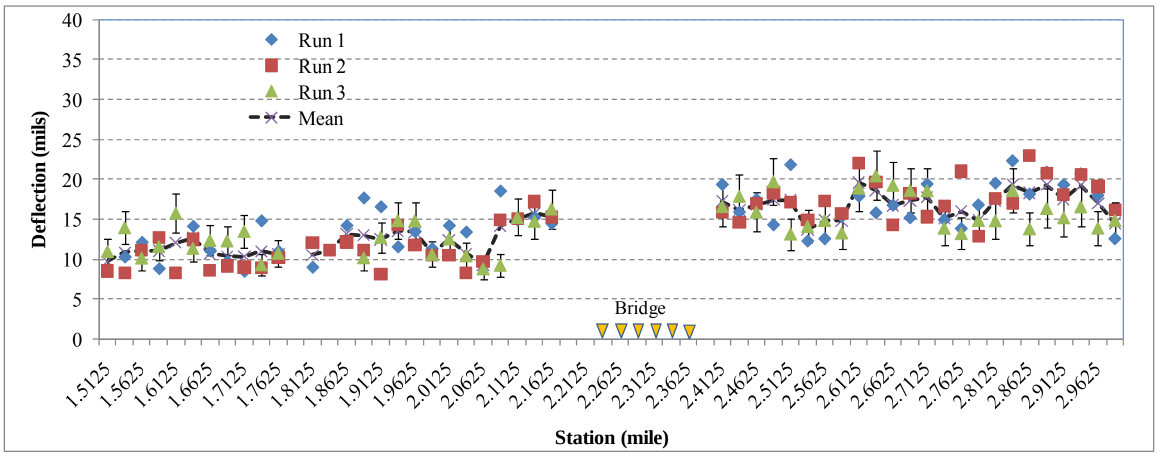
(k) Site 11 (PCI = 57)



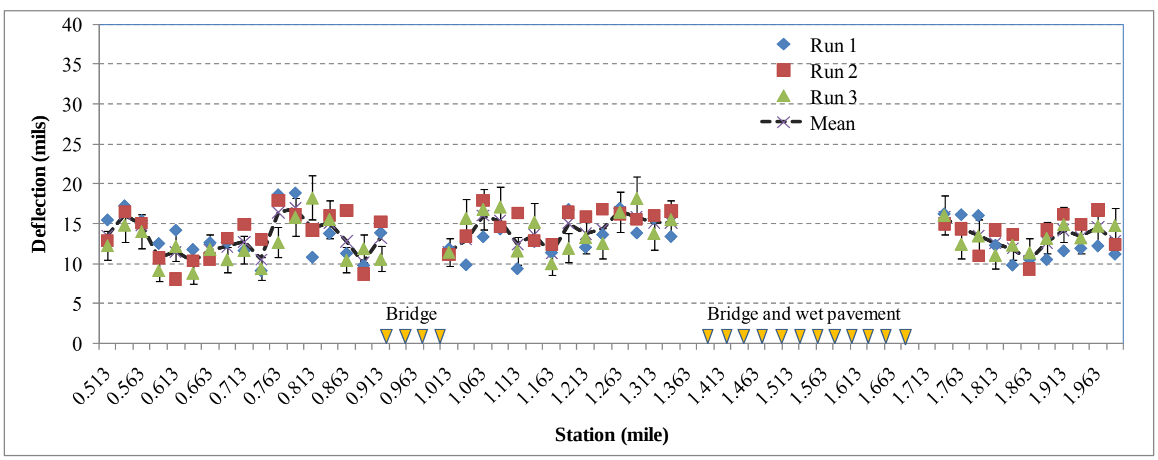
(l) Site 12 (PCI = 77)



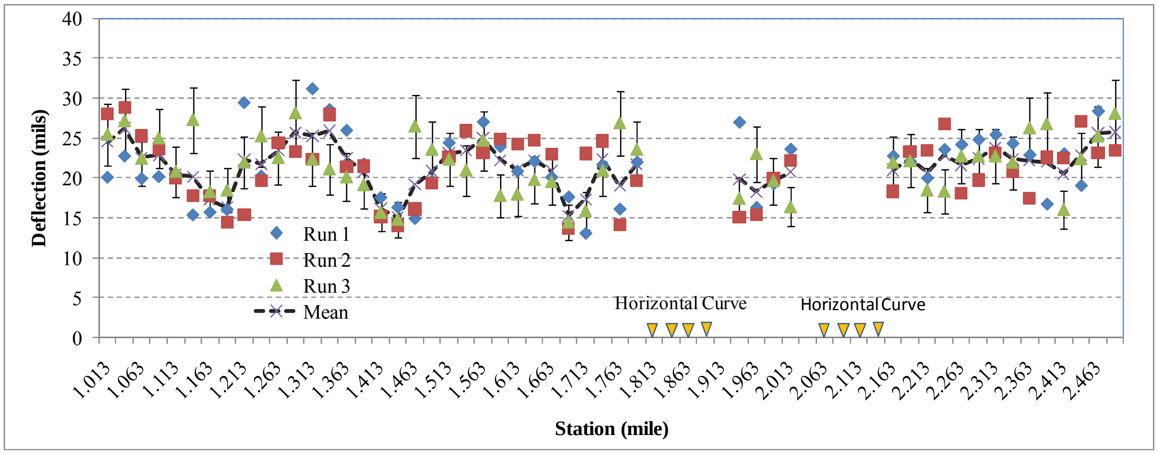
(m) Site 13 (PCI = 87)



(n) Site 14 (PCI = 63)



(o) Site 15 (PCI = 82)



(p) Site 16 (PCI = 60)

Figure 27
Variation of pavement deflection for all project sites (Sites 1 to 16)

The repeatability of RWD measurements is presented in Table 10 as described by the coefficient of variation [COV (%) = standard deviation x 100/ average] for the 16 research sites at the various testing speeds. As shown in this table, the repeatability of the measurements was acceptable with a COV ranging from 7 to 20 percent with an average of 15 percent. One may observe by comparing the test site conditions presented in Table 5 to the deflection variability reported in Table 10, that the majority of the sites in good condition presented better uniformity and less variability than sites in poor condition. One anomaly for this trend is Site 9, which had a high COV while being in a good condition. As shown in Figure 27, measurements were fairly uniform on Site 9; however, the high COV may be due to the low deflection mean on this road, given its thick design (9.5-in. HMA on top of 7.5-in. of cement-treated base). It is also noted from the results presented in Table 10 that the smallest average deflection was recorded for Site 7, which was a thick, composite pavement on the interstate highway system with a 4-in. asphalt overlay on top of a 13.5-in. concrete layer. These results indicate that RWD deflection measurements successfully reflect pavement conditions and structural integrity of the road network by providing for a greater average in deflection and scattering for sites in poor conditions.

Table 10
Variability of RWD measurements in the test sites

Site ID	Test Speed (mph)						Average COV (%)
	20		30	40	50	60	
	Average Deflection (mils[1])	COV (%)	COV (%)				
1	16.43	16	17	14	13	—[2]	15
2	17.15	14	17	18	—	—	16
3	12.58	13	12	13	—	—	13
4	15.62	6	8	9	—	—	8
5	9.50	13	13	16	15	—	14
6	14.99	6	7	8	9	—	7
7	7.75	9	11	17	13	16	13
8	15.98	18	22	19	20	—	20
9	9.53	20	18	16	13	—	17
10	15.51	14	17	16	—	—	16
11	19.89	15	23	—	—	—	19
12	18.41	12	39	15	—	—	22
13	9.51	18	18	16	20	—	18
14	14.37	16	21	—	—	—	19
15	13.54	14	14	16	15	—	15
16	21.55	15	17	—	—	—	16

[1] 1 mil = 0.025 mm; — Not Available - test speed was restricted by the posted speed limits on a number of sites.

Effects of Speed on RWD Deflection Measurements

To assess the effects of truck speed on the measured deflection, RWD testing was conducted on the research sites at different speeds (i.e., 20, 30, 40, 50, and 60 mph). Figure 28 illustrates the variation of the measured deflections with truck speed. As shown in this figure, the influence of the testing speed on the measured deflection was minimal. A statistical analysis of variance was conducted between the different speeds (except vehicle speed of 60 mph, since only one site was tested at this speed) presented in Figure 28 and revealed that the data groups are not statistically different at a level of significance of 0.05, see Table 11.

Table 11
Analysis of variance (ANOVA) of the effect of test speeds

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	55.27	3	18.42	1.10	0.35	2.79

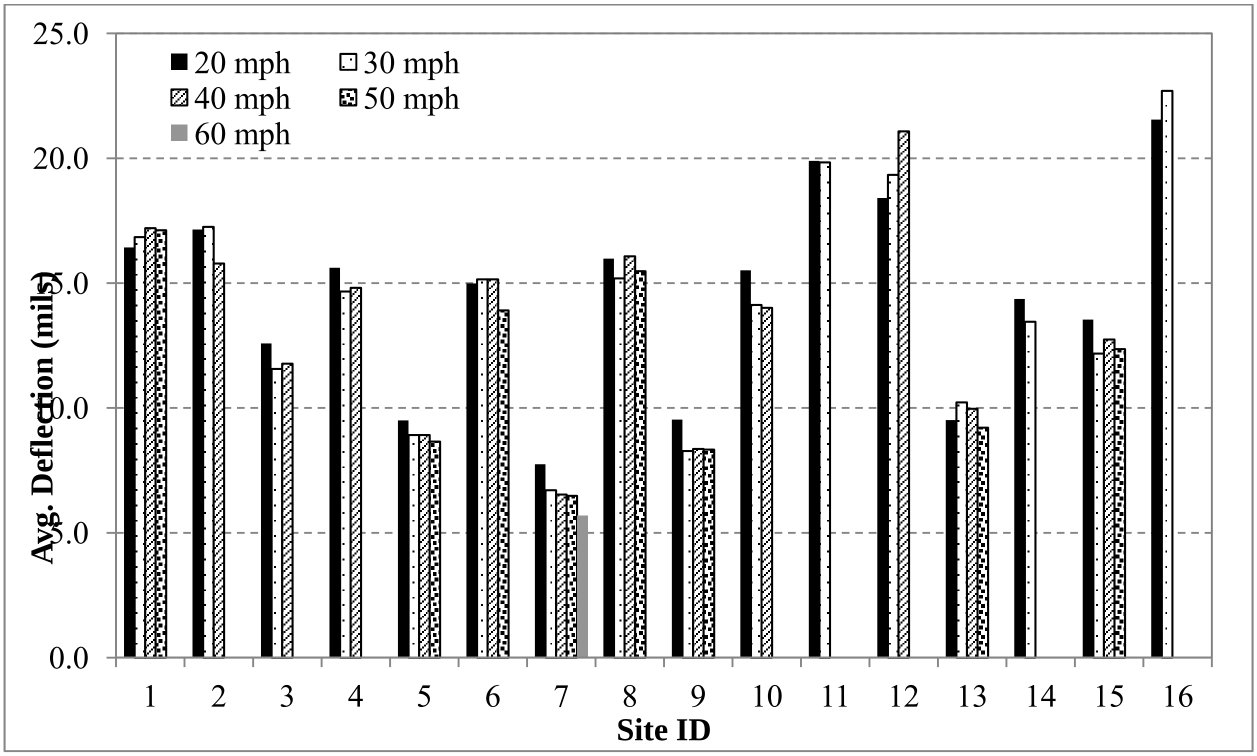


Figure 28
Average RWD deflections measured at different speeds

Since the test speed is restricted by the posted speed limit and the safe operation of a tractor and a semitrailer, this would allow comparing pavement surface deflections measured at different speeds and road conditions. However, past research has indicated that RWD measurements may be affected by significant acceleration or deceleration to maintain constant speed, as it may cause excessive bouncing and vibration of the trailer during testing *[57]*. These results were unexpected given that longer loading time should result in higher deflections. The reason for this trend may be due to two factors. First, testing was conducted in December with an average air temperature of 25˚F. At this low temperature, AC pavement layer tends to be stiff and elastic with small viscoelastic response. Second, the laser sensor accuracy of 1 mils may not detect variations between different testing speeds especially at low temperatures. Therefore, the effect of the testing speed should be considered in future studies at different times of the year especially in the summer months.

Comparison between RWD and FWD Deflection Results

FWD Results

LADOTD used a Dynatest FWD model to test the 16 research sites in District 05. Discrete deflection measurements were collected and deflection profiles were plotted to demonstrate surface condition and stiffness variation along the entire length of each test section. Figures 29 and 30 show two typical FWD deflection profiles for Sites 2 and 12, respectively. Although three load levels of 9,000, 12,000, and 15,000 lb. were used in FWD testing, results used for a comparison with RWD data were the measurements collected at 9,000 lb., to correspond to the tire load used in the RWD.

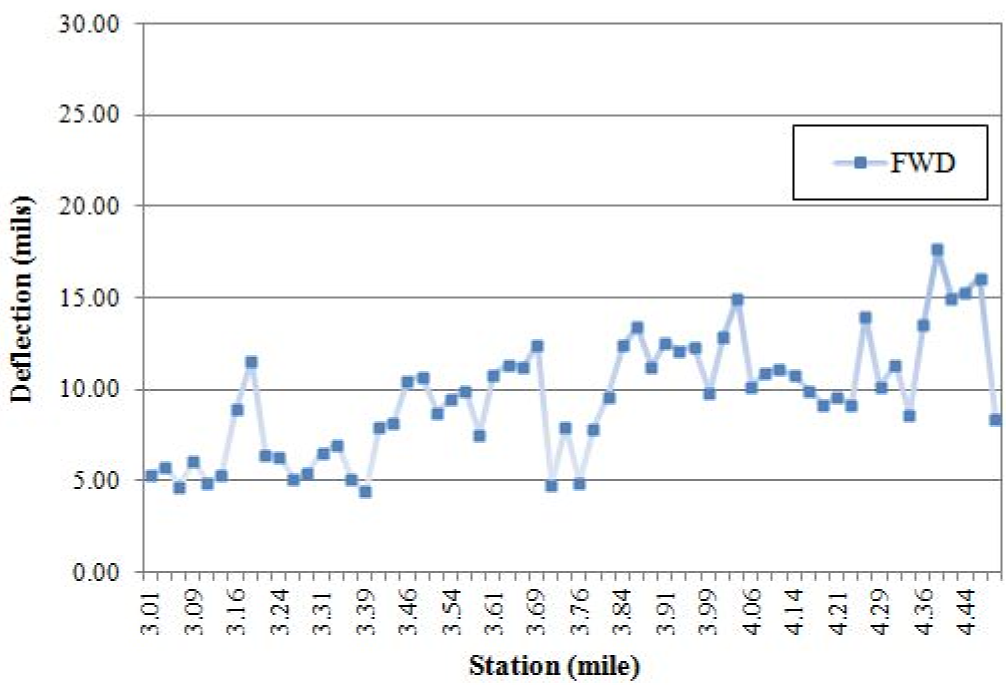


Figure 29
Typical FWD deflection profile for Site 2

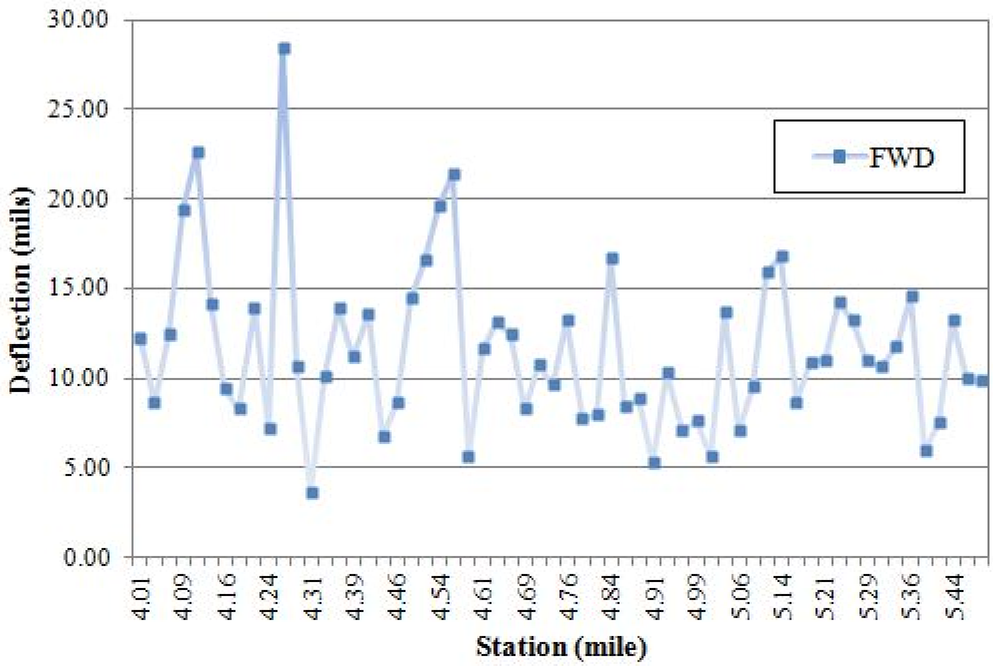
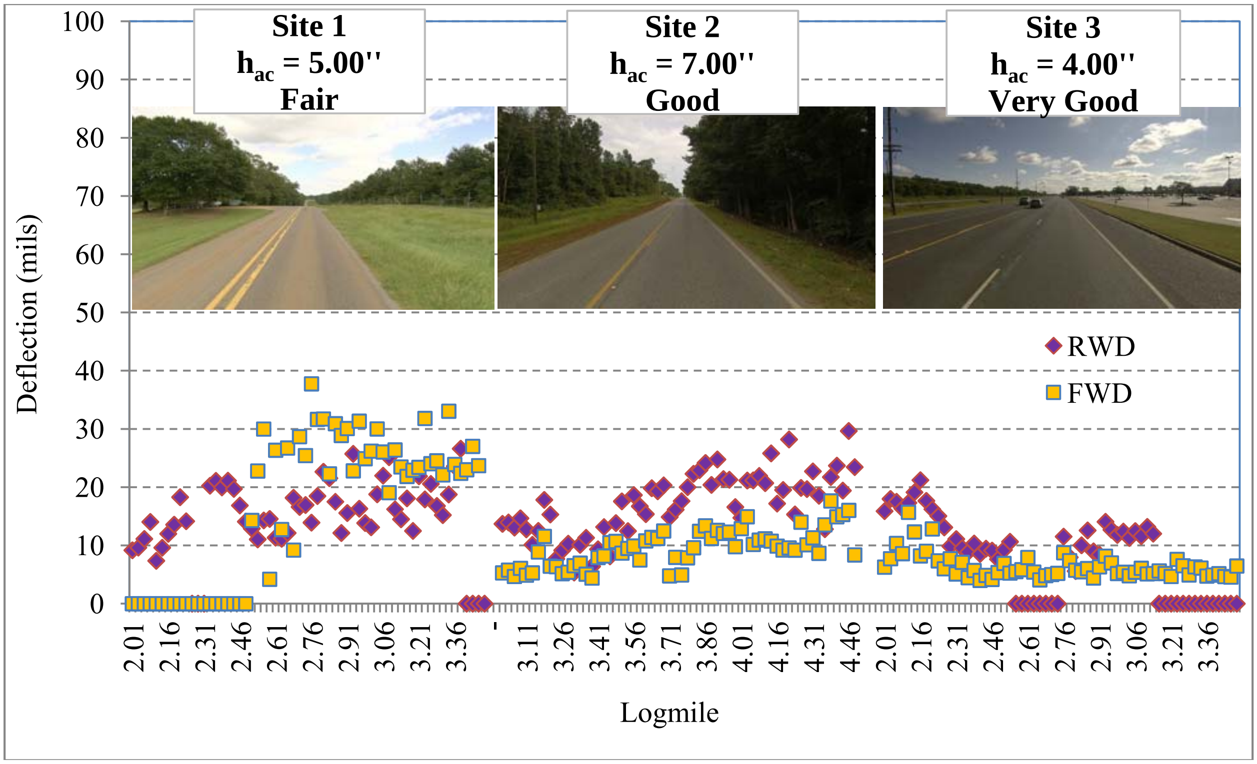


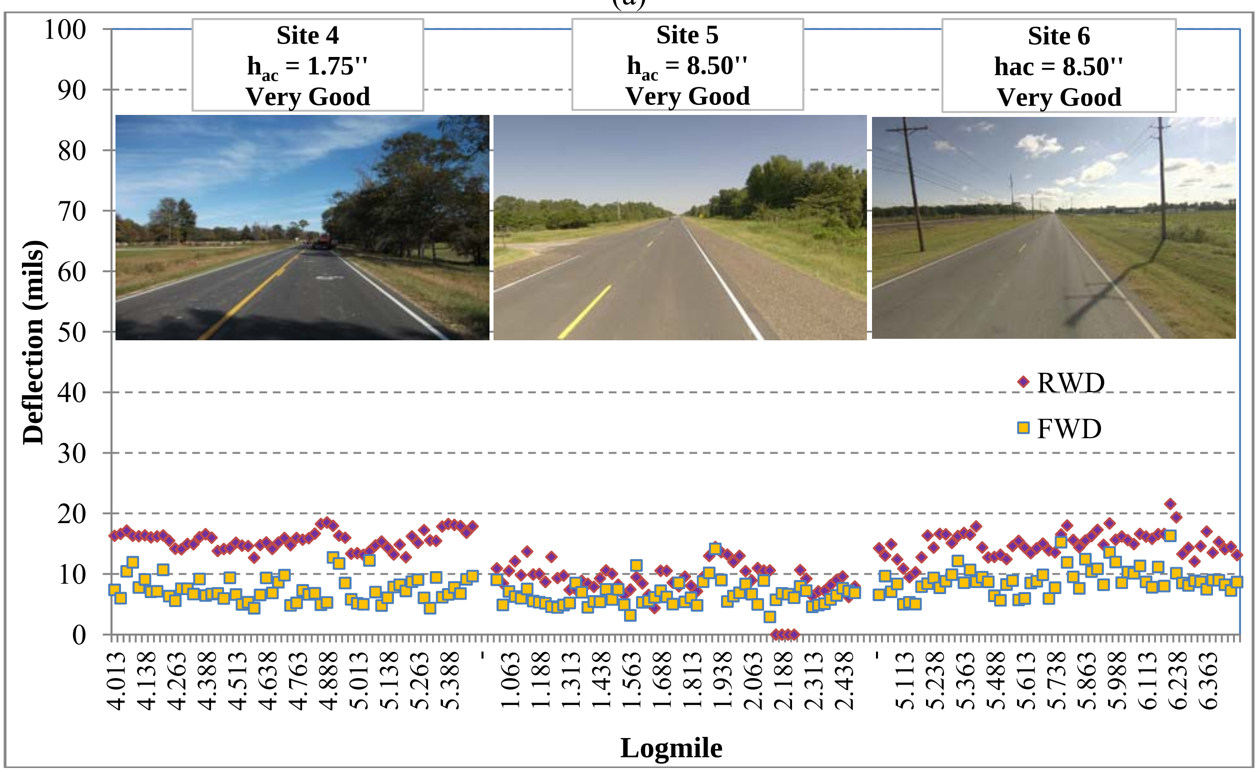
Figure 30
Typical FWD deflection profile for Site 12

Comparison between FWD and RWD Measurements

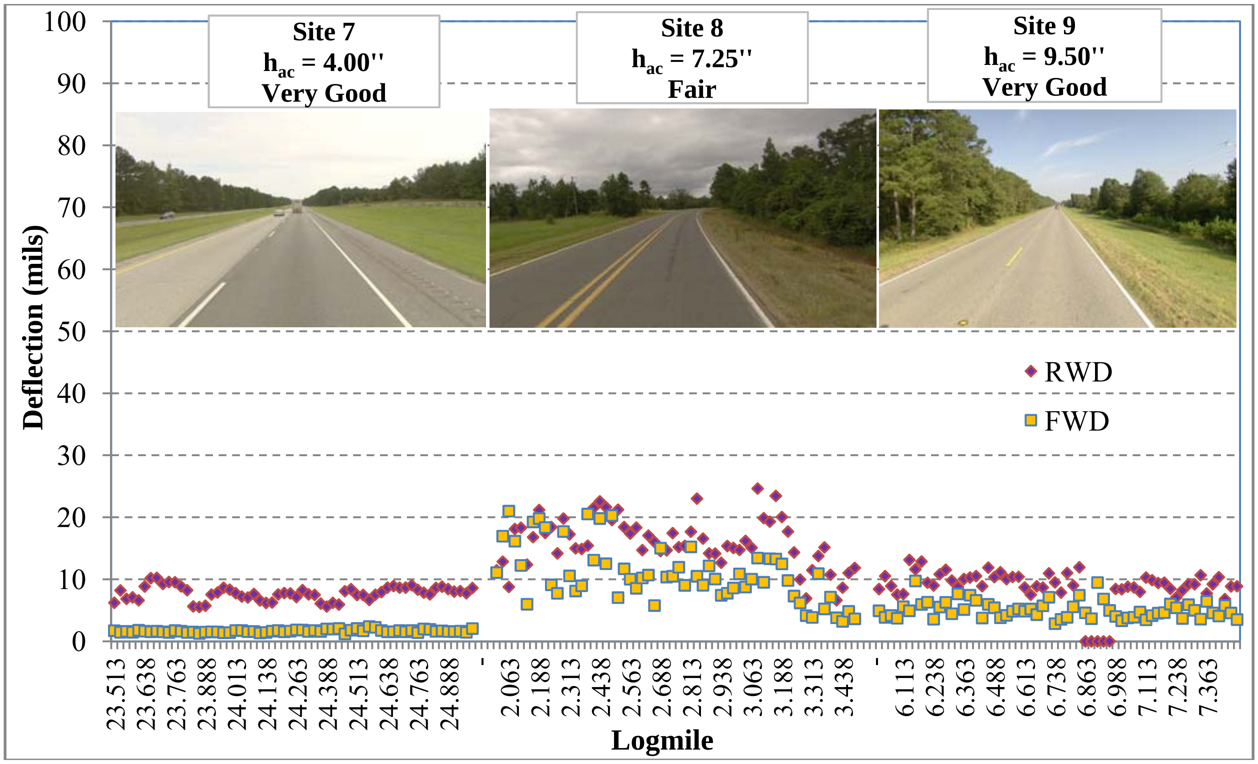
RWD measurements were compared to FWD center deflection data and measured at a load level of 9,000 lb. for each test site as well as its variation with pavement conditions. Figure 31 (a through f) illustrates the variation of the average RWD and FWD deflections for the 16 research sites. As shown in this figure, the scattering and uniformity of FWD and RWD data appear to closely follow the conditions of the roadway. For example, uniform data were measured for Sites 3 and 9, which were in very good condition. In contrast, highly scattered data were measured for Sites 1 and 10, which were in fair and poor condition, respectively. It is also noted from the results presented in Figure 35 that a better agreement between the two test methods was observed for roads with good conditions. It is possible that, for roads with poor conditions, the increase in road roughness may have caused greater truck bouncing and vibrations during RWD deflection measurements, which may explain the differences in FWD and RWD measurements for these sites.


Site 1
h_{ac} = 5.00''
Fair
Site 2
h_{ac} = 7.00''
Good
Site 3
h_{ac} = 4.00''
Very Good
RWD
FWD
Deflection (mils)
Logmile

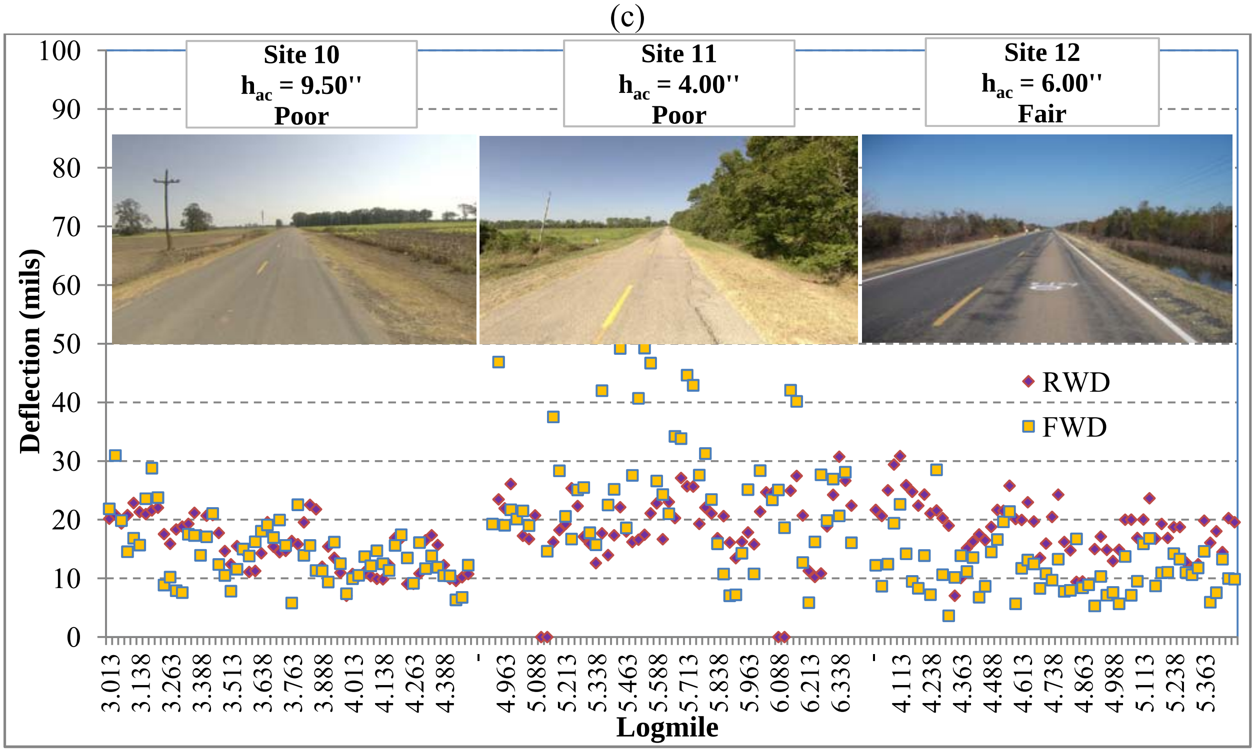
(a)


Site 4
h_{ac} = 1.75''
Very Good
Site 5
h_{ac} = 8.50''
Very Good
Site 6
hac = 8.50''
Very Good
RWD
FWD
Deflection (mils)
Logmile

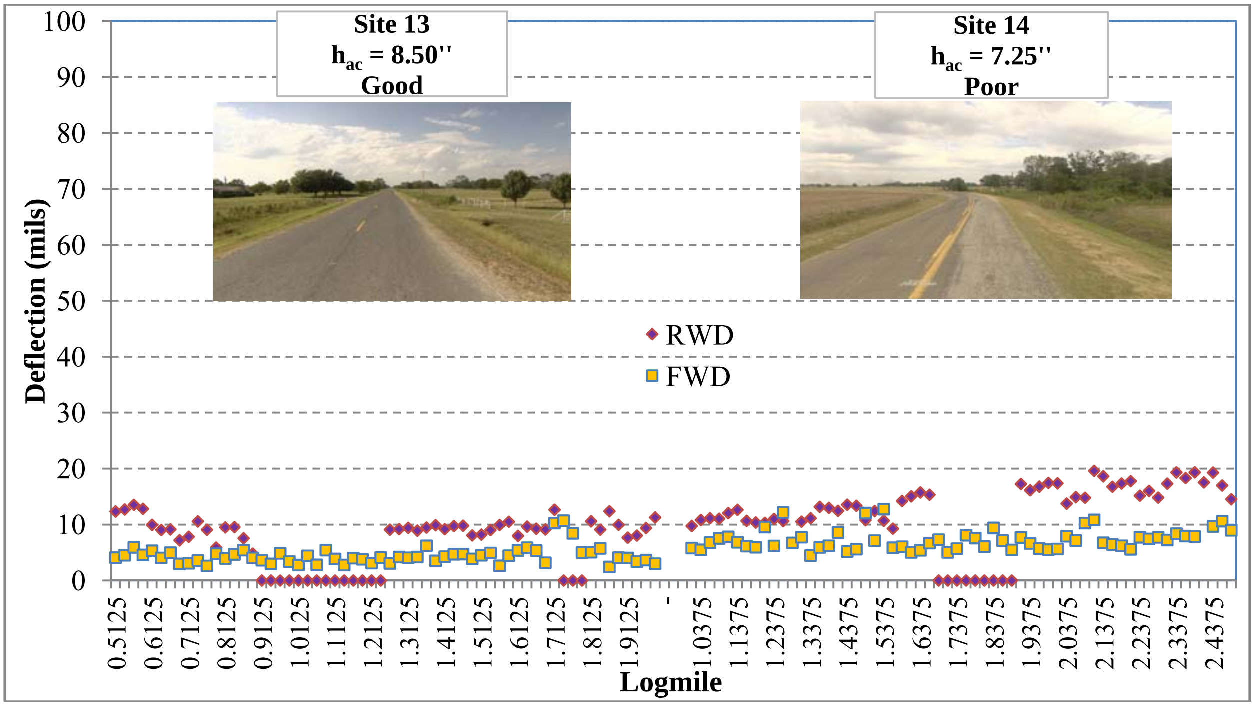
(b)


Site 7
h_ac = 4.00''
Very Good
Site 8
h_ac = 7.25''
Fair
Site 9
h_ac = 9.50''
Very Good
RWD
FWD
Deflection (mils)
Logmile

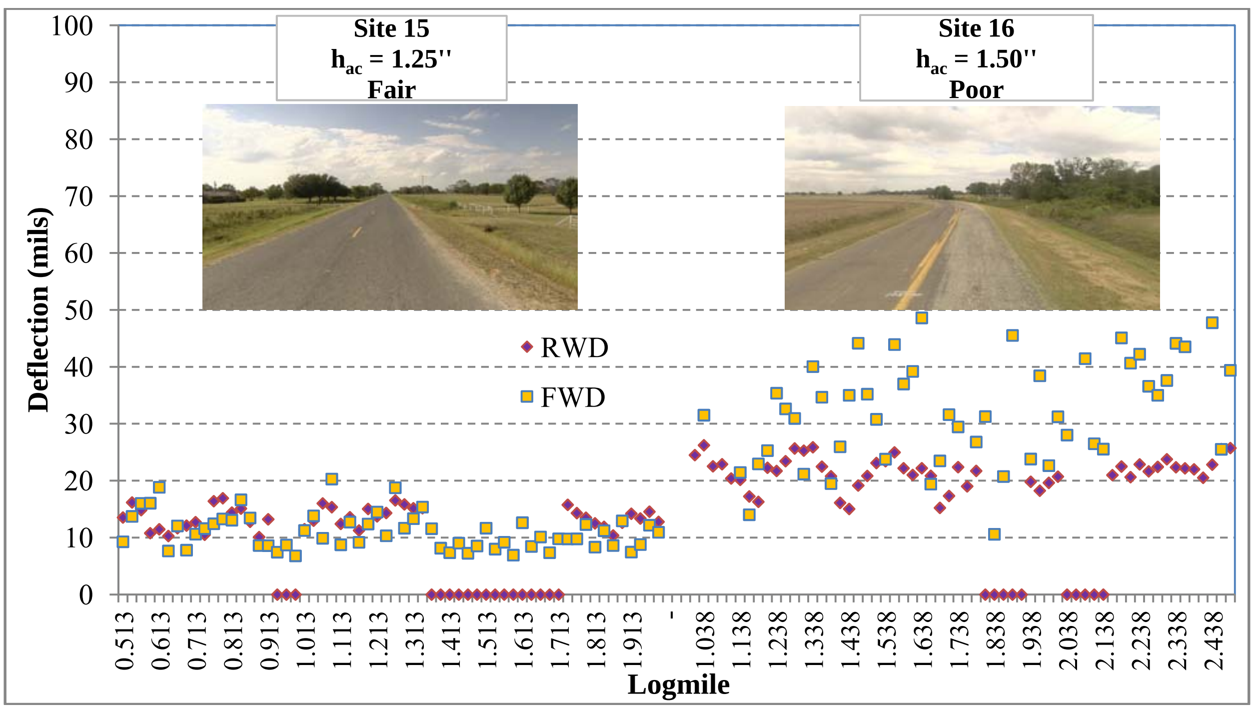
(c)


Site 10
h_ac = 9.50''
Poor
Site 11
h_ac = 4.00''
Poor
Site 12
h_ac = 6.00''
Fair
RWD
FWD
Deflection (mils)
Logmile

(d)



(e)



(f)

Figure 31
Relationships between average RWD and FWD deflection measurements

Based on these results, it is determined that, although the two test methods are based on different loading configurations, both appear to report similar trends in their measurements. This was anticipated, since both are based on the same concept that thin, distressed, and soft pavements exhibit greater deflections than thick, stiff pavements in good conditions. These observations support the validity of RWD measurements as compared to FWD measurements at the network level. Table 12 presents the statistical comparison of the magnitudes of the center deflections, measured from both conducted test methods, using a paired t-test for means at a significance level of 0.05. As shown in this table, the mean center deflections from the RWD and the FWD were statistically different in 15 of the 16 sites. With the exception of Sites 1 and 16, the RWD resulted in a greater surface deflection than the FWD. A difference in deflections was expected, since the loading configurations adopted in both test setups were distinctively different: a vertical pulse applied over a circular plate for the FWD and a three-dimensional transient stress field applied over a dual-tire assembly for the RWD.

Table 12
Statistical analysis of the RWD and FWD center deflections

Site ID	Average FWD	Average RWD	Pearson Correlation	P-value	Decision
1	24.81	16.43	0.13	< 0.0001	Not Equal
2	9.58	17.15	0.65	< 0.0001	Not Equal
3	6.38	12.58	0.78	< 0.0001	Not Equal
4	7.44	15.62	0.22	< 0.0001	Not Equal
5	6.51	9.50	0.41	< 0.0001	Not Equal
6	8.97	14.99	0.66	< 0.0001	Not Equal
7	1.66	7.75	0.15	< 0.0001	Not Equal
8	10.88	15.98	0.59	< 0.0001	Not Equal
9	5.07	9.53	0.20	< 0.0001	Not Equal
10	14.35	15.51	0.44	0.19	Equal
11	26.86	19.89	0.38	< 0.0001	Not Equal
12	11.58	18.41	0.44	< 0.0001	Not Equal
13	4.43	9.51	0.22	< 0.0001	Not Equal
14	8.09	14.37	0.14	< 0.0001	Not Equal
15	11.07	13.54	0.35	0.003	Not Equal
16	37.11	21.55	0.06	< 0.0001	Not Equal

Although the mean deflections from the RWD and the FWD were not statistically equal, the average Pearson correlation coefficient, which varies from 0 for independent variables to 1 for perfectly linearly-correlated variables, was 0.36. This indicates that a level of correlation may exist between FWD and RWD data, yet not necessarily, linear in nature. Figure 32 compares the average FWD center deflections to the average RWD deflections measured at different truck speeds for the 16 test sites. The comparisons presented in this figure are based on the average deflection measurements for RWD and FWD for each individual site. As shown in this figure, an exponential model appears to adequately describe the relationship between the two data sets. However, a linear model did not successfully describe the relationship between FWD and RWD deflection measurements. While the correlation between the two test methods was acceptable at all test speeds, the best fitting between the two data sets was obtained at a speed of 50 mph, the speed that the FWD is assumed to simulate. These measurements indicate that RWD deflection measurements reported similar trends to FWD deflection measurements. Nevertheless, it is important to note that while both test methods report similar trends in deflection measurements, the applications of each test method remain different. While RWD is recommended as a screening tool at the network level to identify structurally deficient sections, the FWD may be applied as a more accurate structural evaluation tool, by assessing the structural capacity of the pavement and by conducting a complete backcalculation of layer moduli.

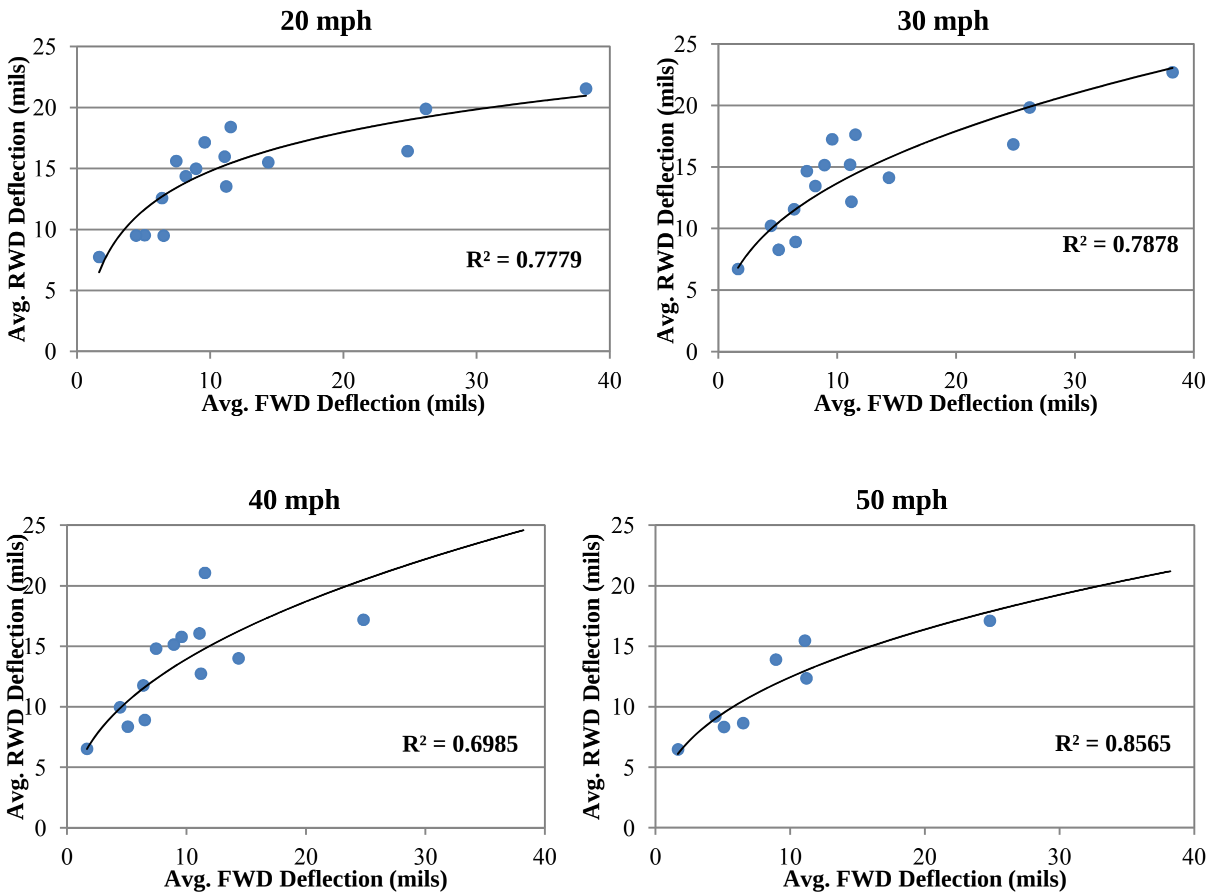


Figure 32
Correlation between average RWD and FWD center deflections

A Model to Estimate Pavement Structural Number at the Network Level Using RWD Data

Based on the data collected and analysis presented in this study, a methodology was developed to predict the pavement SN from RWD measurements. While the AASHTO procedure allows calculating the structural number from the effective pavement modulus (E_p), RWD deflection measurements only provide the center maximum deflection, thus making the calculation of E_p more difficult. Therefore, the AASHTO procedure is not directly applicable to RWD measurements. In this study, a regression model was developed to directly estimate the pavement structural number from RWD deflection measurements at the network level. The objective of this model is to serve as a quick and simple screening tool at the network level to identify structurally deficient pavements. The model was developed into three phases: (1) Phase I: Model development and calibration based on RWD and FWD deflection data collected on the research sites; (2) Phase II: Model validation based on RWD and FWD deflection testing on the validation sites; and (3) Phase III: Model demonstration based on RWD deflection testing on the network sites.

Phase I: Model Development

As a reference for model development and validation, the AASHTO procedure was used to determine SN_{eff} values, based on FWD deflection data. This approach assumes that the subgrade resilient modulus can be obtained from a backcalculation procedure by relating it to the surface deflection at a large distance from the load as shown in equation (14):

$$M_R = \frac{0.24 * P}{d_r * r} \qquad (14)$$

where,

M_R = backcalculated subgrade-resilient modulus (psi);

P = applied load (lb.); and

d_r = deflection at a distance r (72 in.) from the center of the load (in.).

The effective modulus, which describes the strength of all pavement layers above the subgrade, can be computed from FWD deflection measured at the center of the load plate given the subgrade resilient modulus obtained from equation (14) and the total thickness of the pavement. These properties may be used to compute the effective modulus (E_p) using equation (15):

$$\frac{M_R d_0}{1.5qa} = \frac{1}{\sqrt{1+\left(\frac{D}{a} * \sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + \frac{\left[1-\frac{1}{\sqrt{1+\left(D/a\right)^2}}\right]}{\left(\frac{E_p}{M_R}\right)} \qquad (15)$$

where,

E_p = effective modulus of all pavement layers above the subgrade (psi);

d_0 = deflection measured at the center of the load plate and adjusted to a standard temperature of 68^{o}F (in.);

q = load plate pressure (psi);

a = load plate radius (in.);

D = total thickness of pavement layers above the subgrade (in.); and

M_R = subgrade-resilient modulus (psi).

Using the total thickness of the pavement layers and the effective pavement modulus calculated from equation (15), the effective structural number (SN_{eff}) may be calculated using the following expression:

$$SN_{eff} = 0.0045 * D * (E_P)^{1/3} \quad (16)$$

where,
D = total thickness of the pavement layers (in.); and
E_p = effective pavement modulus of all layers above the subgrade (psi).

Table 13 shows the calculated effective SN values for the 16 research sites. As shown in this table, SN_{eff} values ranged between 1.97 to 8.05, which is indicative of pavement sections in different structural conditions. One should note that the SN_{eff} for Site 12 was unexpectedly high given the center deflection measured by FWD on this section. This anomaly is apparent if this site is compared to Sites 8, 13, and 14. Therefore, this site was not considered in the model development phase in order to avoid negatively influencing the fitting process.

Table 13
SN_{eff} calculations for the 16 research sites

Site ID	CSec	d_0	d_r	M_R	D	E_p	SN_{eff}
1	831-05	24.81	1.25	23,941	13.00	32,148.59	1.97
2	069-03	9.58	1.28	23,368	30.00	100,341.87	4.62
3	326-01	6.38	1.58	19,003	11.50	764,506.50	5.83
4	862-14	7.44	1.51	19,932	17.50	247,565.62	5.76
5	071-02	6.50	1.64	18,300	17.00	370,510.84	5.00
6	326-01	8.97	2.60	11,561	36.00	141,401.68	5.59
7	451-05	1.66	0.73	40,987	27.00	1,361,985.37	8.05
8	069-02	10.88	1.39	21,610	24.00	96,942.10	3.69
9	315-02	5.07	1.98	15,146	23.00	497,064.73	7.21
10	333-03	14.35	2.00	15,001	18.00	91,493.20	3.61
11	341-01	26.86	2.06	14,592	11.00	43,226.66	2.65
12	166-01	11.58	2.72	11,019	14.00	262,404.08	5.13
13	067-08	4.43	1.23	24,339	17.50	591,948.97	6.02
14	020-30	8.09	2.29	13,098	15.25	412,118.44	5.33
15	332-01	11.07	1.96	15,301	24.00	113,400.57	3.84
16	862-14	37.11	2.25	13,313	8.50	28,036.06	2.29

The correlation between the effective structural number (SN_{eff}) and the two main inputs of the AASHTO procedure is illustrated in Figure 33 (a and b). As shown in this figure, there is a good correlation between SN_{eff} and the average maximum FWD deflection, while the correlation between SN_{eff} and the total thickness of the pavement layers is much weaker. Figure 33 (c and d) also presents the correlation between SN_{eff} and IRI and PCI for the calibration and validation sites. As shown in this figure, there is a good correlation between SN_{eff} and the surface roughness as described by the IRI.

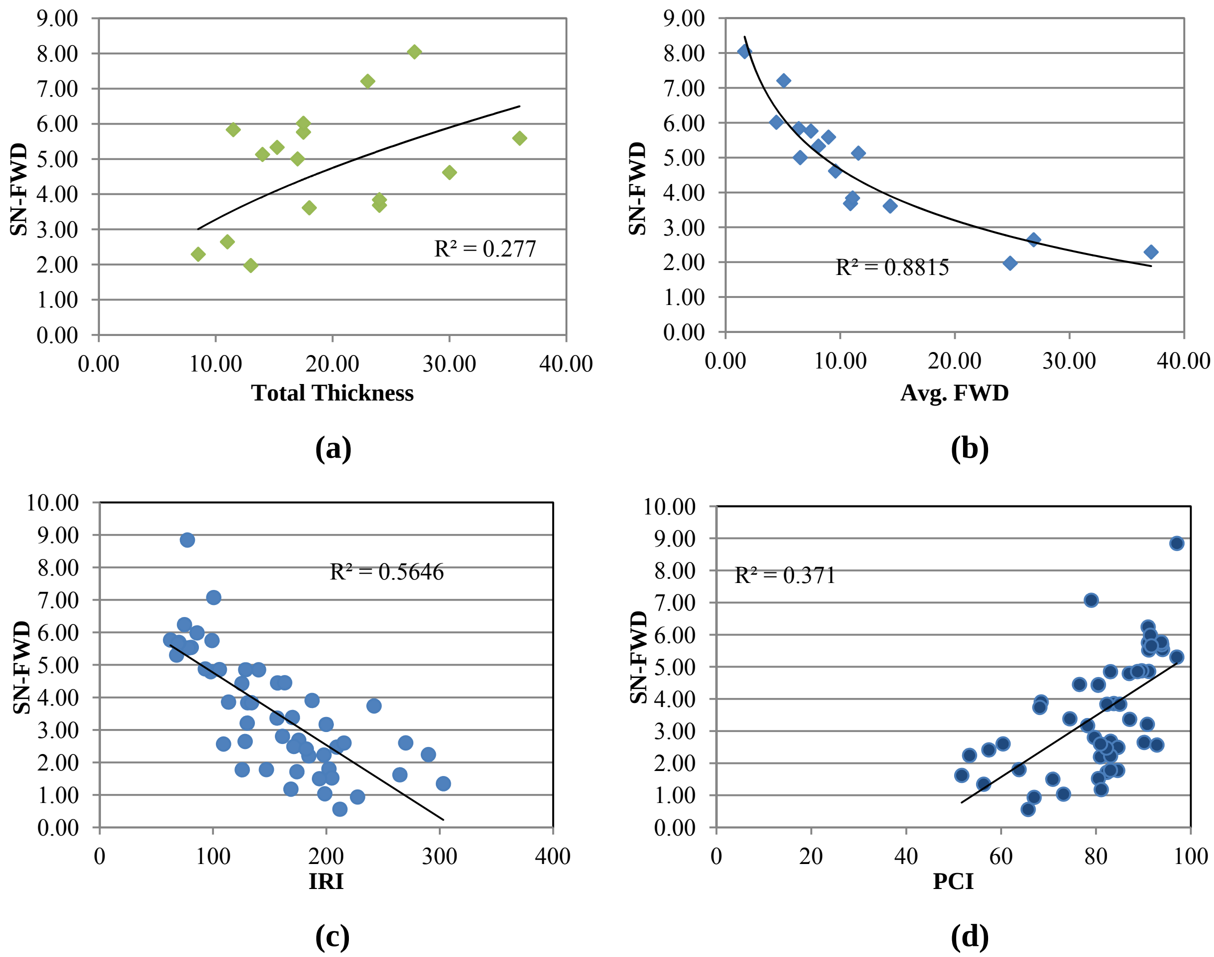


Figure 33
Relationship between SN_{eff} and average FWD deflection and total pavement thickness

Model Description and Calibration. One limitation of the RWD data is that only the center deflection is measured. Therefore, it is not possible to estimate the subgrade resilient modulus based on the deflection away from the load. However, as shown in Figure 34, the average RWD-deflection correlates well with the FWD center deflection.

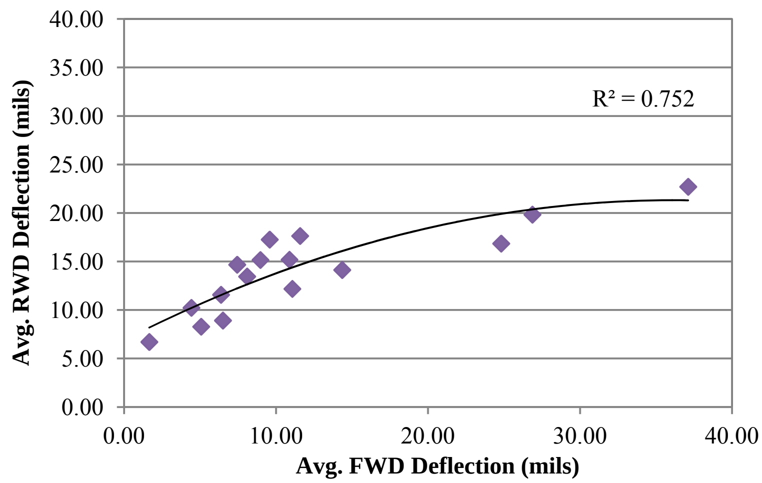


Figure 34
Relationships between avg. FWD deflection and avg. RWD deflection

Given that RWD only measures the center deflection, the developed model introduced a new parameter known as the RI defined as follows:

$$RI = \text{Avg. RWD deflection} * \text{SD of RWD deflection} \qquad (17)$$

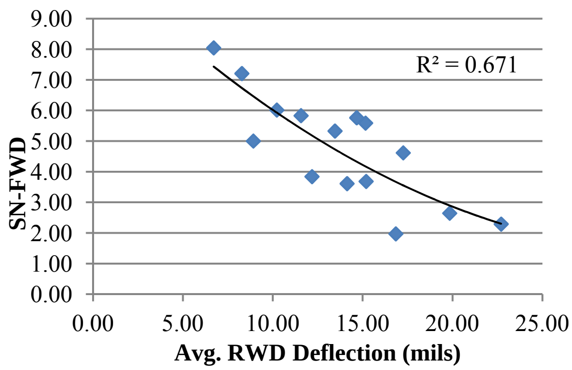
where,

RI = RWD Index (mils^2);

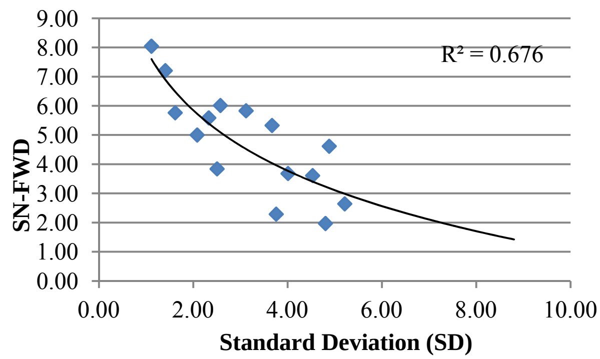
Avg. RWD deflection = Average deflection measured on a road segment (mils); and

SD of RWD deflection = standard deviation of RWD deflection on a road segment (mils).

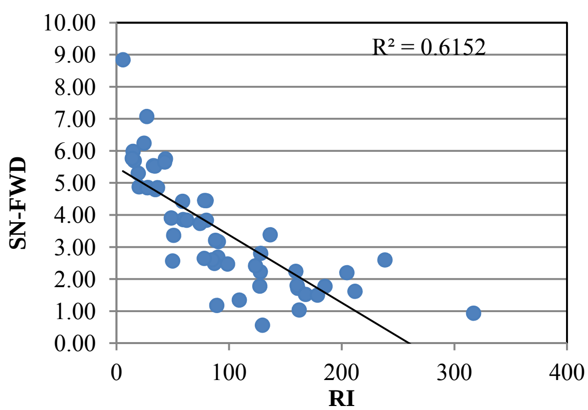
The reason this parameter was used in the developed model was that RWD deflections were observed to reflect the deterioration of the pavement structure, through both an increase in the magnitude of the deflection and an increase in the scattering and variability of the deflection measurements; see previous sections. Therefore, an increase in RI is indicative of the increase in deflection and scattering through a road segment. To validate this observation, the effective pavement structural numbers calculated for the research sites based on the AASHTO procedure were correlated to the average RWD deflections, the standard deviation of RWD deflections, and RI. Figure 35 (a to c) presents the relationships between pavement SN calculated from FWD and the average RWD deflections, the standard deviation of RWD deflections, and RI for the development and validation sites. As shown in these figures, the three RWD-measured parameters correlated well with the pavement SN determined from FWD. Therefore, these properties were used in the developed model to predict pavement SN based on RWD measurements. Table 14 presents the variation of the model variables for the research sites. Figure 35(d) presents the relationship between RI and PCI for the calibration and validation sites. As shown in this figure, the correlation between these two quantities is not apparent.



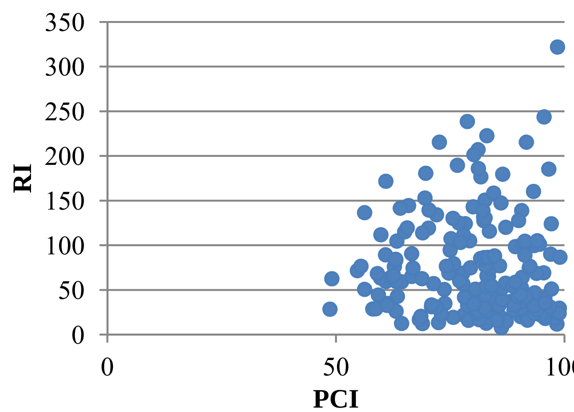
(a)



(b)



(c)



(d)

Figure 35
Relationship between SN_{eff} and model variables

Table 14
Model variables for the 16 research projects

Site ID	CSec	RWD	SD	RI	SN_{eff}
1	831-05	16.84	4.80	80.89	3.51
2	069-03	17.25	4.88	84.24	3.42
3	326-01	11.57	3.12	36.08	5.12
4	862-14	14.67	1.62	23.70	5.20
5	071-02	8.92	2.08	18.56	6.29
6	326-01	15.16	2.33	35.28	4.71
7	451-05	6.71	1.11	7.47	7.39
8	069-02	15.19	4.01	60.89	4.04
9	315-02	8.28	1.41	11.64	6.79
10	333-03	14.13	4.53	64.02	4.08
11	341-01	19.84	5.21	103.39	2.96
12[1]	166-01	19.34	8.80	155.17	2.31
13	067-08	10.22	2.57	26.32	5.69
14	020-30	13.46	3.67	49.37	4.49
15	332-01	12.18	2.50	30.49	5.23
16	862-14	22.70	3.76	85.30	3.04

[1]: not considered in the fitting process (see previous section).

During the model development phase, multiple linear and nonlinear regression models were evaluated to relate RWD deflection to the dependent variable, pavement SN. Goodness of fit parameters, such as R^2 and Root-Mean Square Error (RMSE), were used to assess the accuracy of the model. While linear regression failed to accurately describe the data, nonlinear regression was more successful. Based on various expressions evaluated during the course of this study, the following relationship between SN- and RWD-measured parameters was the most accurate:

$$SN_{RWD} = -6.37 - \frac{150.69 * RI^{-0.81}}{RI + 19.04} + 23.52 * RWD^{-0.24} - 1.39 * \ln(SD) \tag{18}$$

where,

RI = RWD Index ($mils^2$) = Avg. RWD deflection * SD of RWD deflection;

SD = standard deviation of RWD deflection on a road segment (mils);

RWD = Avg. RWD deflection measured on a road segment (mils); and

SN_{RWD} = Pavement Structural Number based on RWD measurements.

Based on the developed model, Figure 36 shows the correlation between the FWD-calculated SN and the RWD-calculated SN for the developed sites, based on equation (18). As shown in this figure, the coefficient of determination for the fitting process was acceptable at 0.75.

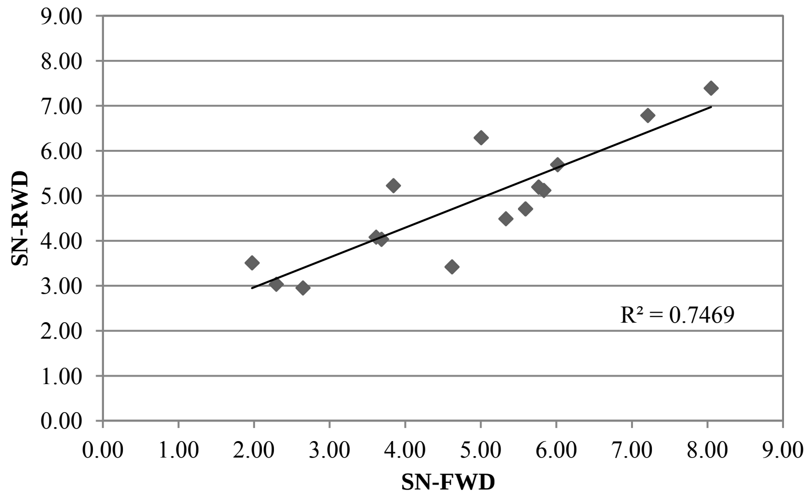


Figure 36
Relationships between SN based on FWD and SN based on RWD

Phase II: Model Validation

An additional 58 in-service pavement sections were tested in Louisiana, using RWD and FWD as part of the first phase of the testing program (Network Sites). These sections were not used in the model development phase. These pavements cover a wide array of pavements with various ages, structural configurations, subgrade properties, and traffic loads. Although control sections usually represent segments with uniform rehabilitation histories and traffic loads, 6 out of the 58 sections were removed from the validation process. FWD and RWD data collected on these sites did not appear to reflect the same pavement conditions. This may be due to measurement errors or inaccurate RWD measurements, which were noted to be more frequent in ultra-thin sections (HMA ≤ 0.5 in.). RWD deflection, standard deviation, and RWD index were calculated for each section and substituted into equation (18) to predict the SN values for the pavement sections.

Figure 37 illustrates the correlation between FWD and RWD deflection data on the network sites.

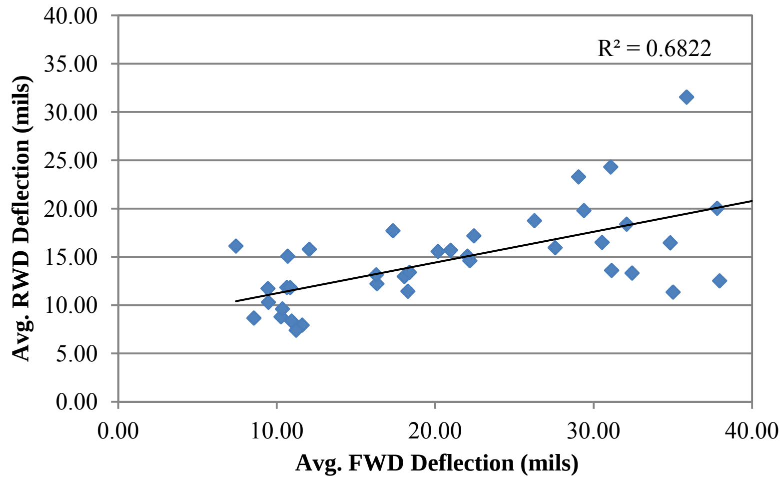
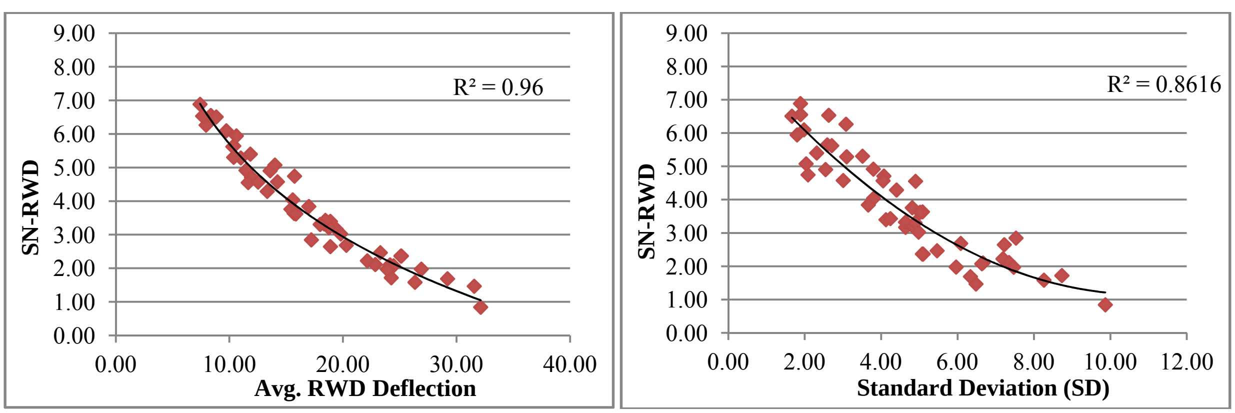
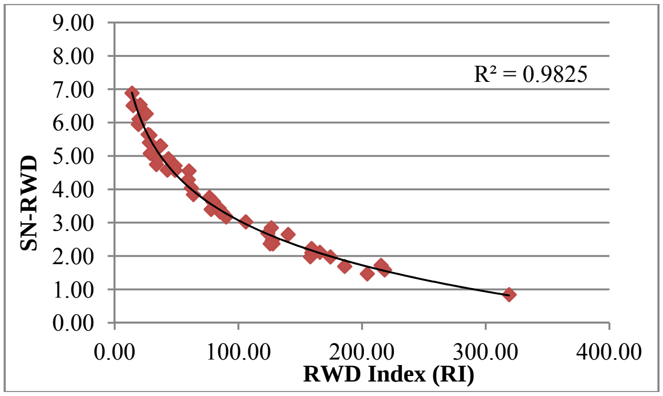


Figure 37
Relationships between avg. FWD deflection and avg. RWD Deflection

Figure 38 illustrates the relationships between pavement SN calculated from RWD and the average RWD deflections, the standard deviation of RWD deflections, and RI for the network sites. Figure 38(c) presents the correlation between SN_{RWD} and RI. As shown in this figure, there is an excellent correlation between these two quantities, and as proposed in this study, the structural assessment of the pavement may be directly based on the RI.



(a) **(b)**



(c)
Figure 38
Relationship between SN-RWD and model variables

Figure 39 presents the relationship between SN based on FWD deflections and SN based on RWD deflection data. As shown in this figure, there was an acceptable agreement between SN calculations based on FWD and RWD deflection testing. The RMSE, which is widely used as a measure of precision, indicated that the average deviation between the model and the FWD-calculated SN values was 0.63. The coefficient of determination (R^2) was 0.77 indicating an acceptable fit. Therefore, the model statistics support the use of the proposed model as a screening tool for identifying structurally deficient pavements at the network level.

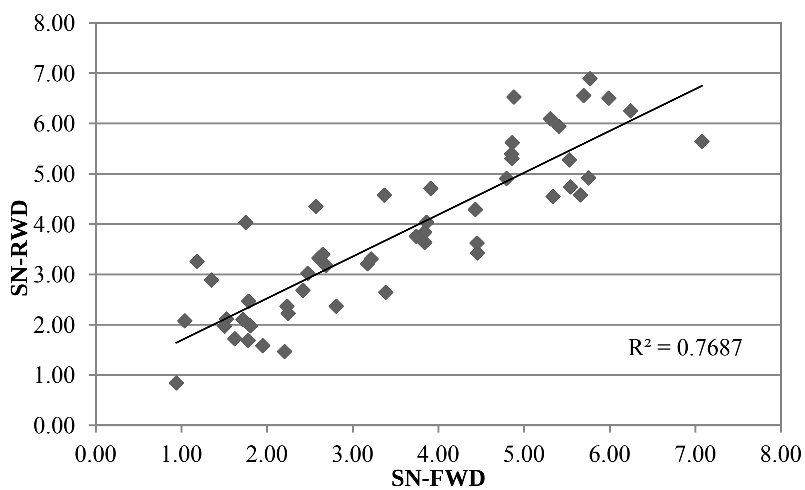


Figure 39
Relationships between SN based on FWD and SN based on RWD for the independent network sites

Phase III: Applications of the Structural Model at the Network Level

The structural capacity prediction model was applied to 220 sections tested in Louisiana using RWD. For each section, the average RWD deflection and the RWD Index were calculated. The structural number was then calculated based on equation (18), and the results were incorporated into PMS via a GIS map, Figure 40. In addition to the structural capacity prediction model, a GIS map was developed based on the RI. The benefit of the RWD Index is that it is a measured property that does not depend on subsequent calculations. The GIS map developed based on the RI is presented in Figure 41. The thresholds of the RWD Index and the SN to define good, fair, and poor pavement structural conditions are presented in Table 15. These values were developed based on inputs from experts' opinions and existing Louisiana thresholds. As shown in Table 15, pavement sites were grouped into three categories for analysis:

- Thin pavements – less than 3 in. of AC
- Medium pavements – 3 to 6 in. of AC
- Thick pavements – more than 6 in. of AC

The aforementioned pavement categories and the thresholds presented in Table 15 are based on the results of this project and may be updated and modified based on the results of future studies. GIS maps may be used to identify homogeneous sections, distressed pavements, as well as to display the response of the RWD to different pavement conditions. Based on the results presented in Figure 40, District 05 may identify the sites in poor structural conditions. The model developed in this study should be used in coordination with other surface distress indices, such as rutting, cracking, etc., to evaluate and rate a pavement section for maintenance and rehabilitation purposes. However, it is noted that the results of the assessment based on SN and RI are different from an assessment based solely on PCI (Figure 42) or IRI (Figure 43).

Table 15
Threshold values for SN and RI for different pavement conditions

Pavement Condition	**Structural Number Range**			**RI Range**			**PCI**	**IRI**
	Thin[1]	**Medium[2]**	**Thick[3]**	**Thin**	**Medium**	**Thick**		
Poor	< 2	< 3	< 4	< 40	< 39	< 25	< 64	> 200
Fair	2 to 3	3 to 5	4 to 7	40-110	39-109	25-79	64 - 84	120-200
Good	> 3	> 5	> 7	> 110	> 109	> 79	> 85	< 120

[1]: AC thickness < 3 in.; [2] AC thickness: 3 to 6 in.; [3] AC thickness > 6 in.

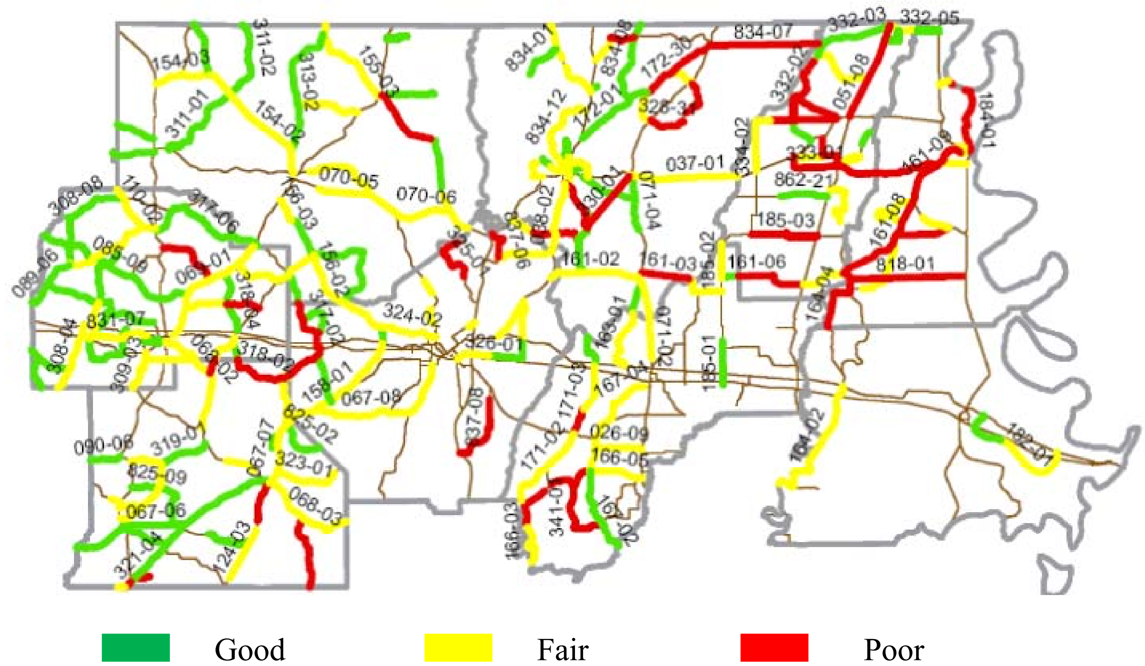


Figure 40
Applications of the SN model to identify structurally deficient pavements

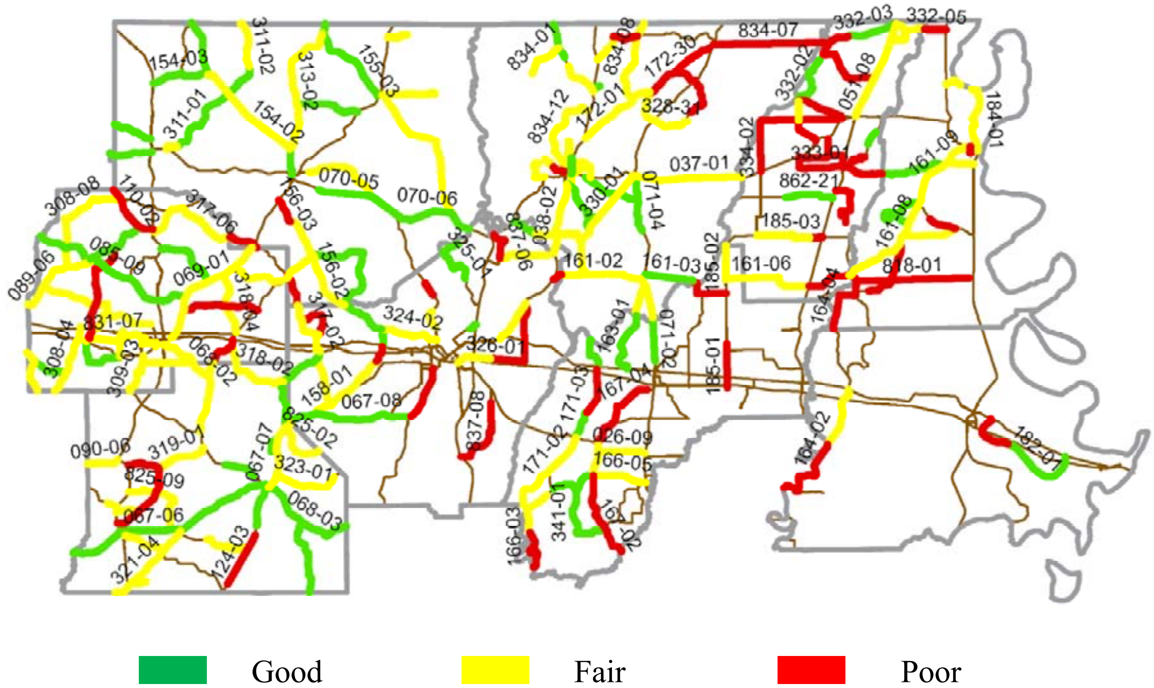


Figure 41
Road conditions in District 05 of Louisiana using RI

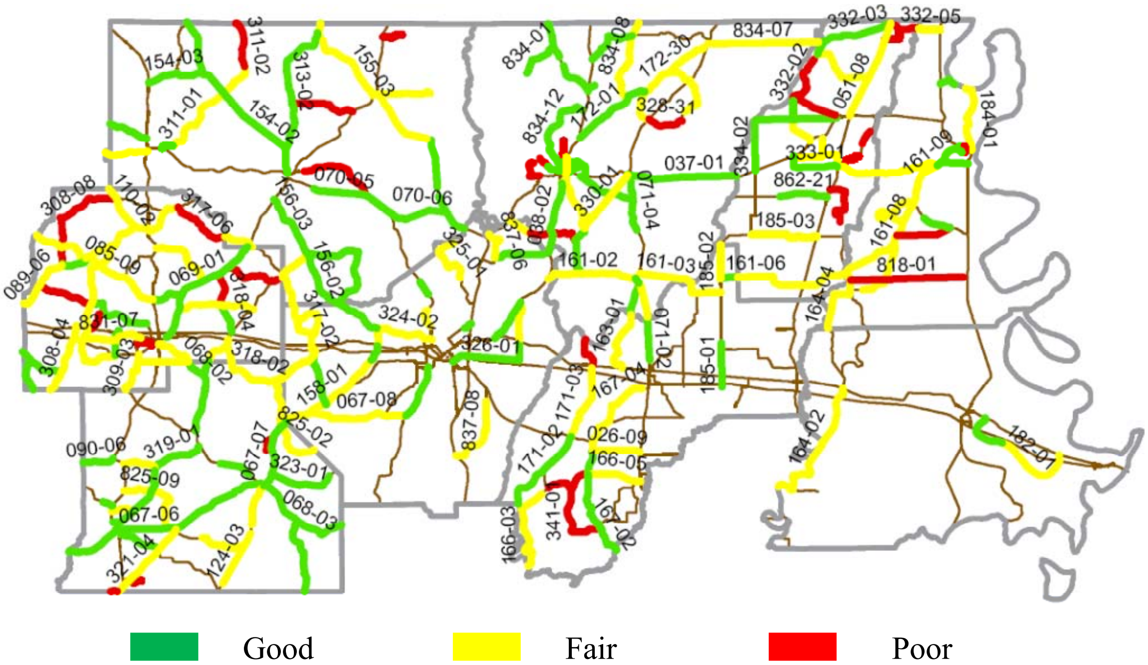


Figure 42
Road conditions in District 05 of Louisiana using PCI

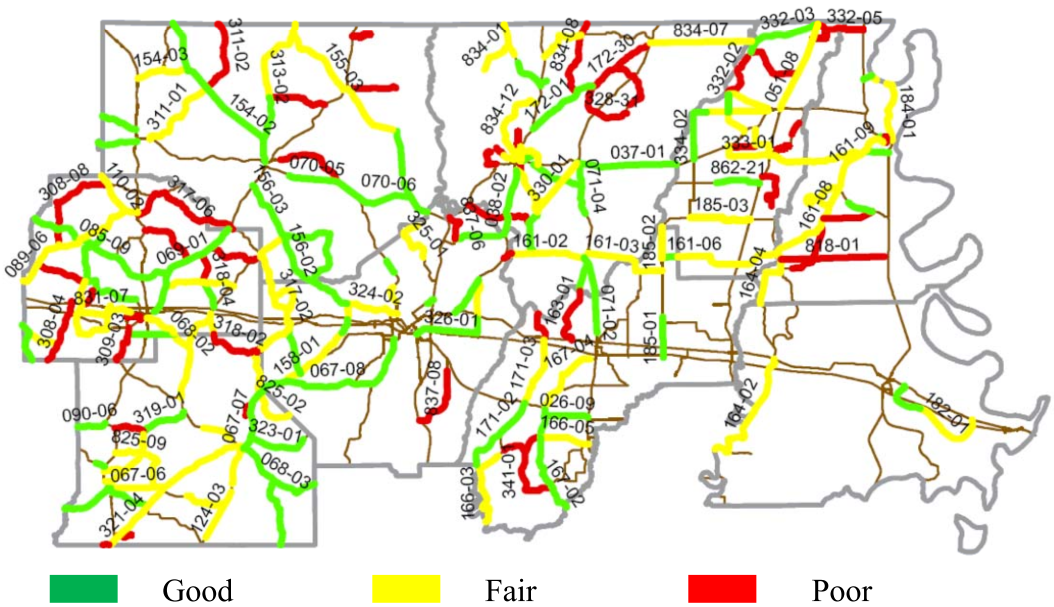


Figure 43
Road conditions in District 05 of Louisiana using IRI

It is worth noting that the developed SN model was calibrated and validated based on data obtained with the FWD system. The correlation chart presented in Figure 44 may be used to relate the thresholds presented in Table 15 to the SN calculated from the Dynaflect deflection system.

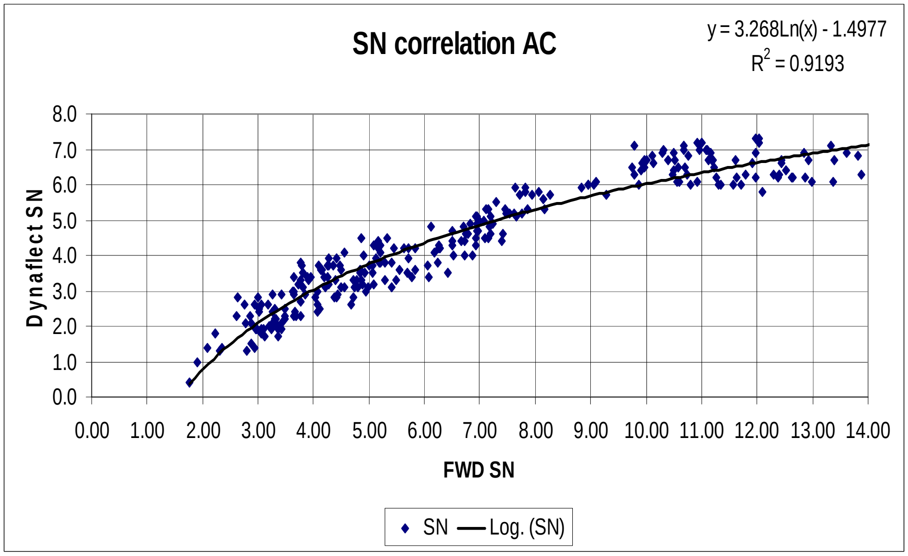


Figure 44
Correlation between SN calculated from FWD and SN calculated from Dynaflect *[76]*

Cost of RWD Testing

This study collected data from the consultant on the cost of testing the road network in District 05 in 2009. This information would be useful for future testing plans in other districts in Louisiana. The consultant indicated that the productivity of RWD strongly depends on the type of roads to be tested. For example, the cost per mile is much lower for interstate roads than for local and secondary roads. Overall, the productivity in District 05 was about 80 to 100 lane-mi per 8-hr testing day because the entire road network, including secondary and local roads, was tested. Table 16 presents the cost of RWD testing for three scenarios: (1) high productivity for primary and Interstate roads; (2) medium productivity for secondary roads; and (3) low productivity for local roads. The cost of RWD testing is compared to typical FWD testing as a baseline reference, also obtained from the consultant.

Table 16
Productivity and cost details of RWD and FWD

Item	Units	FWD	RWD
FWD and RWD Daily Cost	$ per day	$4,200	$10,500
FWD and RWD Daily Productivity - Interstate	ln-mi per day	30	250
FWD and RWD Daily Productivity - Secondary	ln-mi per day	20	150
FWD and RWD Daily Productivity - Local Roads	ln-mi per day	15	100
Per Mi Cost - Interstate and Multilane Primary Routes	**$ per ln-mi**	**$140**	**$42**
Per Mi Cost - Primary and Secondary Two-Lane Routes	**$ per ln-mi**	**$210**	**$70**
Per Mi Cost - Local Rural Routes	**$ per ln-mi**	**$280**	**$105**
Traffic Control - Interstate and Multilane Primary Routes	$ per day	$3,200	n/a
Traffic Control - Primary and Secondary Two-Lane Routes	$ per day	$1,600	n/a
Traffic Control - Local Rural Routes	$ per day	$800	n/a
Traffic Control - Interstate and Multilane Primary Routes	$ per ln-mi	$107	n/a
Traffic Control - Primary and Secondary Two-Lane Routes	$ per ln-mi	$80	n/a
Traffic Control - Local Rural Routes	$ per ln-mi	$53	n/a
Combined FWD and Traffic Control - Interstate and Primary Routes	**$ per ln-mi**	**$247**	**$42**
Combined FWD and Traffic Control - Primary and Secondary Routes	**$ per ln-mi**	**$290**	**$70**
Combined FWD and Traffic Control - Local Rural Routes	**$ per ln-mi**	**$333**	**$105**

From the presented productivity and cost data, one may notice that while the daily cost of RWD is greater than that for FWD, RWD has a much higher daily productivity than FWD and it does not require traffic control. However, as previously mentioned, it is important to note that while both test methods report similar trends in deflection measurements, the applications of each test method remain different. While RWD is recommended as a screening tool at the network level to identify structurally deficient sections, the FWD may be applied as a more accurate structural evaluation tool, by assessing the structural capacity of the pavement and by conducting a complete backcalculation of layer moduli to assist in overlay design.

Recommended Testing Procedure

Based on the analysis and results presented in this study, the researchers recommend that RWD testing be implemented in Louisiana as a screening tool at the network level to identify structurally deficient sections. The flowchart presented in Figure 45 highlights the implementation strategy for RWD into Louisiana PMS. As shown in this figure, it is recommended to conduct RWD testing every four years as the rate of structural deterioration of pavements would allow for this testing frequency. The selection of cost-effective treatments is currently being evaluated in Project 10-4P.

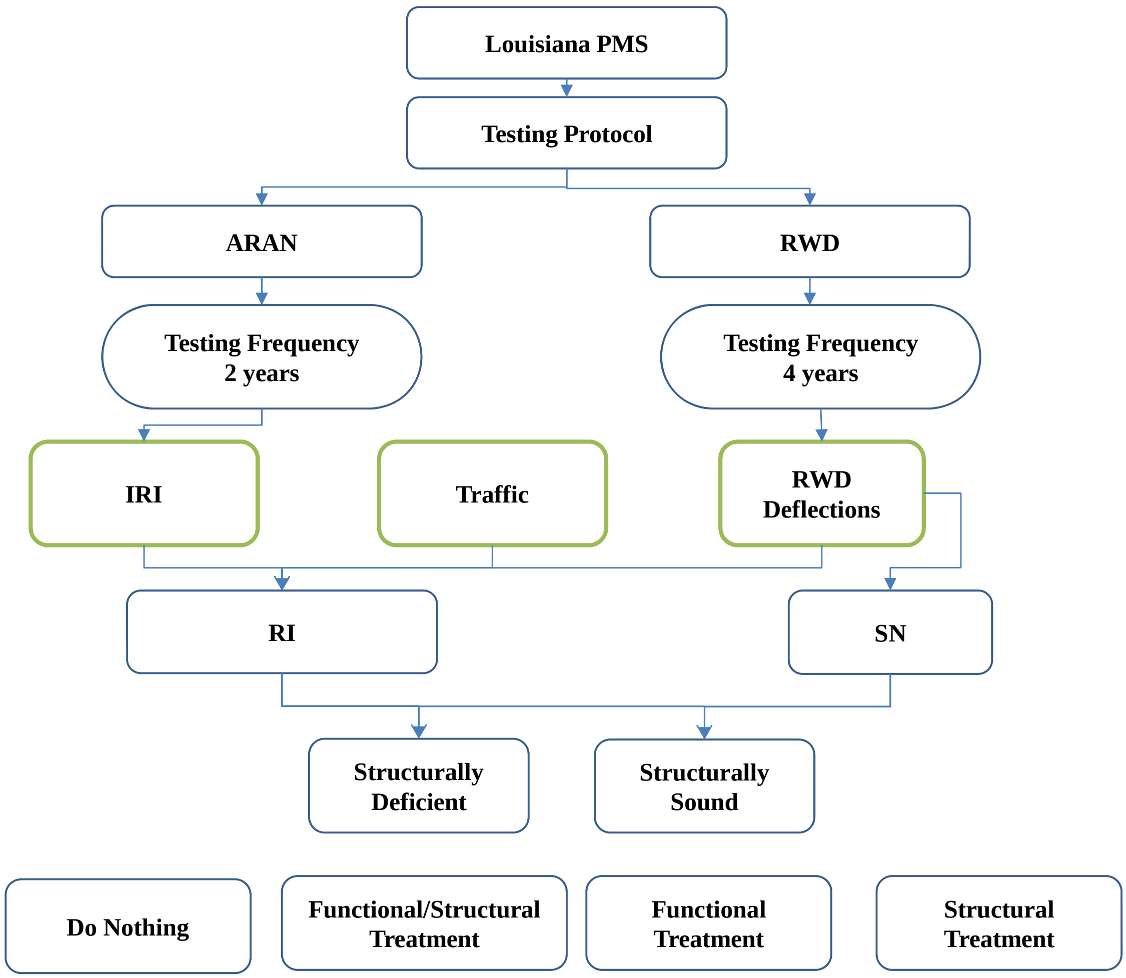


Figure 45
Proposed implementation plan for RWD in Louisiana PMS

CONCLUSIONS

Based on results of the analysis conducted in this study, the following findings and conclusions may be drawn:

- Repeatability of RWD measurements was acceptable with an average coefficient of variation at all test speeds of 15 percent.
- The influence of the testing speed on the measured deflections was minimal. Since the test speed is restricted by the posted speed limit, testing can be conducted at different speeds while allowing for direct comparison of the measured deflections.
- The scattering and uniformity of FWD and RWD data appears to closely follow the conditions of the roadway. Both test methods appear to properly reflect pavement conditions and the structural integrity of the road network by providing for a greater average deflection and scattering for sites in poor conditions.
- RWD deflection measurements were in general agreement with FWD deflections measurements; however, the mean center deflections from the RWD and the FWD were statistically different for 15 of the 16 sites.
- A model was developed to estimate pavement SN based on RWD deflection data. Although the SN expression developed is independent of the pavement thickness and layer properties, it provides promising results as an indicator of structural integrity of pavement structure at the network level. However, further evaluation of the proposed model is needed prior to its use at the project level.

RECOMMENDATIONS

Based on the evaluation detailed in this report, the research team recommends extending the use of RWD to the other districts in Louisiana. The RWD Index is recommended to be adopted on a provisional basis by LADOTD PMS as a network structural analysis index with three categories thin pavement less than 3 in. thick, medium-pavements between 3 to 6 in., and thick-pavements greater than 6in. It should be incorporated into the PMS system and placed on GIS maps. The structural number from the RWD equation should also be considered valid and used as a tool to evaluate the structural condition of pavements for network purposes with similar categories as the RI. The PMS section will incorporate the SN values in their process using trigger values outlined in the report. If the PMS section considers the new index to be of significant value, then another district will be assessed with the RWD. In addition, the following issues should be addressed in future research to enhance the use of RWD in Louisiana:

- Data processing software should be modified to provide the capability of multiple-interval averaging. In addition, a procedure of filtering insufficient measurements, due to wet pavements, bridges, sharp curves, traffic signals, and unreasonable readings, should be included as well.
- Validation and possible modification of the developed models should be conducted based on independent data collected in another district.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ADT	Average Daily Traffic
ARA	Applied Research Associates
ARAN®	Automatic Road Analyzer
ARWD	Airfield Rolling Weight Deflectometer
ASTM	American Society for Testing Materials
cm	centimeter(s)
ConnDOT	Connecticut Department of Transportation
COV	Coefficient of Variation
CRCP	Continuously Reinforced Concrete Pavement
EFLHD	Eastern Federal Lands Highway Division
ESAL	Equivalent Single Axle Load
FHWA	Federal Highway Administration
ft.	foot (feet)
FWD	Falling Weight Deflectometer
GIS	Geographic Information System
GPR	Ground Penetrating Radar
HMA	Hot Mix Asphalt
HSD	High Speed Deflectograph
IRI	International Roughness Index
in.	inch(es)
INDOT	Indiana Department of Transportation
KDOT	Kansas Department of Transportation
KYDOT	Kentucky Department of Transportation
LADOTD	Louisiana Department of Transportation and Development
lb.	pound(s)
LTRC	Louisiana Transportation Research Center
m	meter(s)
MDD	Multi-Depth Deflectometer
mils	one thousandth of an inch
MnDOT	Minnesota Department of Transportation

M&R	Maintenance and Rehabilitation
NAPTF	National Airport Pavement Test Facility
NDT	Non-Destructive Testing
NHDOT	New Hampshire Department of Transportation
NJDOT	New Jersey Department of Transportation
NMDOT	New Mexico Department of Transportation
ODOT	Ohio Department of Transportation
PCC	Portland Cement Concrete
PCI	Pavement Condition Index
PMS	Pavement Management System
PMIS	Pavement Management Information System
psi	Pounds per square inch
RI	RWD Index
RDD	Rolling Dynamic Deflectometer
RDT	Road Deflection Tester
RMSE	Root Mean Square Error
RWD	Rolling Wheel Deflectometer
SAI	Structural Adequacy Index
SCI	Structural Condition Index
SN	Structural Number
SSI	Structural Strength Index
SSIF	Structural Strength Index
TMV	TxDOT Modular Vehicle
TSD	Traffic Speed Deflectometer
TTI	Texas Transportation Institute
TxDOT	Texas Department of Transportation
VDOT	Virginia Department of Transportation
USDOT	US Department of Transportation
WASHO	Western Association of State Highway Organizations
WVDOT	West Virginia Department of Transportation

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