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INDIANA DEPARTMENT OF TRANSPORTATION
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Correlation between Resilient Modulus (M_R) of Soil, Light Weight Deflectometer (LWD), and Falling Weight Deflectometer (FWD)

Sung Soo Park, Tommy Nantung, Antonio Bobet

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AUTHORS

Sung Soo Park
Graduate Research Assistant
Lyles School of Civil Engineering
Purdue University

Tommy Nantung, PhD, PE
Section Manager
Office of Research and Development
Indiana Department of Transportation
(765) 463-1521
tnantung@indot.in.gov
Corresponding Author

Antonio Bobet, PhD
Professor of Civil Engineering
Lyles School of Civil Engineering
Purdue University
(765) 494-5033
bobet@purdue.edu

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16. Abstract

INDOT adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) beginning January 1, 2009, which is based on the FHWA Long Term Pavement Performance (LTTP) field study. The resilient modulus of the soil, MR, is required to implement the new design guide, as well as the pavement input parameters. The soil resilient modulus test requires special, expensive, equipment, significant time investment and effort, which has led researchers to develop MR prediction models and alternative methods to estimate the resilient modulus using non-destructive tests such as Falling Weight Deflectometer, FWD, Light Weight Deflectometer, LWD, and Dynamic Cone Penetrometer, DCP. The objectives of the project are geared toward a practical approach for pavement design procedures to effectively determine the soil resilient modulus for rehabilitation projects, targeting specifically untreated subgrade soils type A-6 and A-7-6. A total of four sites in Indiana were selected to conduct FWD, LWD, and DCP tests, as well as resilient modulus tests in the laboratory. In addition to the output from the four sites, additional data were collected from the data repository of INDOT which has geotechnical and pavement information. Extensive analysis and comparisons were done in an attempt at establishing relationships between the field tests and the laboratory results. The study showed the following: (1) high quality FWD tests conducted on top of the pavement can be used to estimate the subgrade MR, as long as site conditions and pavement layers thickness are well known; (2) the results of FWD tests on top of the subgrade are not reliable, as they are affected by the low confinement of the soils; and (3) LWD and DCP tests can be used to provide and assessment of the quality and uniformity of the subgrade, but do not provide reliable estimates of the stiffness of the subgrade.

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EXECUTIVE SUMMARY

CORRELATION BETWEEN RESILIENT MODULUS (M_R) OF SOIL, LIGHT WEIGHT DEFLECTOMETER (LWD), AND FALLING WEIGHT DEFLECTOMETER (FWD)

Introduction

INDOT adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) beginning on January 1, 2009. This is a new design guide based on the FHWA Long Term Pavement Performance (LTPP) field study that has existed for more than 20 years. The new guide requires pavement design input parameters that are more accurate, representing actual conditions in the field, as well as other tests for input parameters that are sensitive to the performance of the pavement during its design life. One such parameter is the resilient modulus, M_R. The resilient modulus is obtained in the laboratory and requires specialized and expensive equipment. The test itself needs significant time and effort. These limitations have led researchers to develop M_R prediction models according to type of soil, as well as alternative methods to estimate the resilient modulus using non-destructive tests such as *FWD*, *LWD*, and *DCP*. The tests are fast and easy and, as a result, are widely used for compaction control in the U.S. In Indiana, *DCP* and *LWD* are recommended for chemically stabilized subgrade soils and aggregates. However, limited work has been conducted to relate M_R and *DCP* for Indiana soils. Clearly, robust and credible correlations between M_R and *FWD*, *LWD*, and *DCP* are needed.

The objectives of this project are geared toward a practical approach for pavement design procedures to effectively determine the soil resilient modulus for rehabilitation projects. The ultimate goal of the research is to create guidelines for selecting values of soil subgrade stiffness, targeting specifically untreated subgrade soils type A-6 and A-7-6. A total of four sites located around in Indiana were selected to conduct the field tests. Laboratory tests were also performed using soil samples obtained from the sites.

The scope of the project was expanded to further investigate relations between field *FWD* and laboratory resilient modulus tests using the data repository of INDOT to obtain additional geotechnical and pavement information.

Findings

The objective of this study is to assess the potential use of *FWD*, *LWD*, and *DCP* tests to estimate the resilient modulus of fine-grained soil subgrades (A-6 and A-7-6). Four sites were selected to conduct the field tests, with subgrade soils from the sites classified as: (1) US 31 (A-4 soils with 58% of passing #200 and 8.5 *PI*), (2) SR 37 (A-7-6 soils with 88% of passing #200 and 23.8 *PI*), (3) SR 641 (A-6 soils with 83% of passing #200 and 18.4 *PI*), and (4) Ramp A (A-6 soils with 72% of passing #200 and 14.0 *PI*). In addition to the outcomes from the four sites, additional data were collected from the data repository of INDOT, which has geotechnical and pavement information. Analysis of the field *FWD* and laboratory M_R tests led to the following conclusions:

- The results obtained from *FWD* tests conducted on top of the pavement can be used to estimate the resilient modulus of the subgrade soils obtained from the laboratory as long as the quality of the tests is high and the pavement layer thicknesses are accurate.
- The results from *FWD* tests conducted on top of the subgrade are not reliable, likely due to the lack of confinement of the soil.
- The stiffness calculated from *LWD* tests performed on top of the subgrade does not compare well with the resilient modulus of the soil obtained in the laboratory.
- The correlation by Salgado and Yoon (2003) does not show a good relation between the soil stiffness obtained from *DCP* and the M_R obtained from the laboratory test.

Implementation

- High-quality *FWD* tests conducted on top of the pavement provide reasonable estimates of the resilient modulus of the subgrade soils.
- The results obtained from *LWD* and *DCP* conducted on top of the subgrade can be used for quality control of the subgrade, but they may not provide reliable estimates of the resilient modulus of the soil obtained in the laboratory.
- To obtain good-quality data from *FWD* tests, the LTPP protocol is recommended for research-level work, with a small modification for production-level testing. There are other very good protocols available that INDOT could explore for use.

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1. INTRODUCTION AND RESEARCH OBJECTIVES

INDOT adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) beginning on January 1, 2009. This is a new design guide based on the FHWA Long Term Pavement Performance (LTTP) field study for more than 20 years. The new pavement design guide requires pavement design input parameters that are more accurate, representing actual conditions in the field, and other tests input parameters that are sensitive to the performance of the pavement during its design life. One such parameter is the resilient modulus, M_R. The resilient modulus is obtained in the laboratory and requires a specialized and expensive equipment. The test itself needs significant time and effort. These limitations have led researches to develop M_R prediction models according to type of soil and alternative methods to estimate the resilient modulus using non-destructive tests such as *FWD*, *LWD*, and *DCP*. The tests are fast and easy and, as a result, are widely used for compaction control in the U.S. In Indiana, *DCP* and *LWD* are recommended for compaction control of subgrade soils and aggregates. However, limited work has been conducted to relate M_R and *DCP* for Indiana soils, (e.g., Salgado & Yoon, 2003). Clearly, robust and credible correlations between M_R and *FWD*, *LWD*, and *DCP* are needed for Indiana.

The objectives of this project are geared toward a practical approach for pavement design procedures to effectively determine the soil resilient modulus for rehabilitation projects. The ultimate goal of the research is to create guidelines for selecting values of soil subgrade stiffness, targeting specifically untreated subgrade soils type A-6 and A-7-6. The following are the milestones to achieving this end goal:

1. Review the state of stress-strain in laboratory resilient modulus testing and field *FWD*, *LWD*, and *DCP* tests.
2. Review conversion factors/models between *FWD*, *LWD*, and *DCP* and laboratory resilient modulus.
3. Provide recommendations for obtaining the subgrade resilient modulus for pavement rehabilitation projects.

To accomplish these milestones, an extensive literature review and a combination of field testing and laboratory testing was undertaken. The objective was to develop approximate methods to estimate M_R using field tests such as *FWD*, *LWD*, and *DCP*. A number of correlations already exist between the tests (Abu-Farsakh, Nazzal, Alshibli, & Seyman, 2005; Fleming, Frost, & Rogers, 2000; Nazzal, Abu-Farsakh, Alshibli, & Mohammed, 2007; Siekmeier et al., 2009; Salgado & Yoon, 2003), but it is not clear how credible or accurate the estimates obtained from those correlations are, and in particular for those soils found in Indiana.

It was decided to identify sites under construction where the subgrade was classified as A-6 and A-7-6 and was not chemically treated. At each of the sites, the field tests were conducted and, at the same location, soil samples were taken to do the resilient modulus test in the laboratory, such that meaningful comparisons could be established between the field and the laboratory tests. The following sections provide the summary of the work done and the major conclusions. Further information, test details, test data and analysis are included in the Appendices.

2. FIELD TESTS

The sites to conduct the field tests were selected based on the following criteria: type of soils, compacted subgrade with natural soils not chemically modified, and testing availability. This project targeted fine-grained soils classified as A-6 and A-7-6. An important consideration was site availability, including access, project schedule but also that the project fit the designed testing plan (details described in this section), which required a testing area 90 m (300 ft) long and flat subgrade to be able to perform *FWD*, *LWD* and *DCP* tests. These requirements, which were essential for the project, made the site selection quite challenging. Additional issues such as equipment availability, weather and coordination with the contractor added difficulty to the task. The authors are very thankful to INDOT personnel for their collaboration, and in particular to Mr. Nayyar Siddiki. Without the support of INDOT, the project would not have been possible.

Four sites were identified and selected for the project. The locations are shown in Figure 2.1. The projects chosen were: (1) US 31 in Kokomo, (2) SR 37 in Paoli, (3) SR 641 in Terre Haute, (4) Ramp line A connecting SR 641 and SR 46 in Terre Haute. At each of the four sites, the soil properties were determined using the laboratory tests summarized in Table 3.1. According to AASHTO M 145-91 (2012), the soil samples from US 31 were classified as A-4, from SR 641 and Ramp A as A-6, and the soil from SR 37 as A-7-6.

FWD, *LWD*, and *DCP* tests were performed to evaluate the stiffness of the subgrade soils. A 90 m (300 ft) length section at each site was selected and 11 test points were marked at 9 m (30 ft) intervals where the tests were performed (see Figure 2.2). All of the eleven points are labeled with numbers 1 to 11, which will be used to identify the soil samples at a site, e.g., US 31_1. All the field tests were conducted at the same location to reduce material variability and obtain meaningful comparisons between the resilient moduli estimated from the tests and obtained in the laboratory. At a given location, all of the three tests were done adjacent to each other to avoid site disturbance; all of the tests were done within a radius of 0.3 m (1 ft). In addition, sand cone tests and nuclear gauge tests were performed to measure unit weight and water content of the in-situ soils. More details are found in Appendix Chapter 3.

ELMOD 5, the Boussinesq's equation, and Salgado and Yoon's (2003) relationship are used to calculate the equivalent modulus from *FWD*, *LWD*, and *DCP* test results, respectively.

2.1 Site on US 31

The site was located in Kokomo, Howard County, Indiana. See Figure 2.3.

On August 23, 2013, *FWD*, *LWD*, *DCP* and nuclear tests were conducted on the subgrade. Figure 2.4 shows the estimated moduli from *FWD*, *LWD*, and *DCP*, and includes the in-situ water content and optimum water content. The *FWD* modulus (E_{FWD}) ranged from 36 MPa to 134 MPa. The estimated modulus from the *LWD* tests ranged from 37 to 90 MPa. The values of the modulus estimated from the *DCP* were around 95 MPa. The stiffness of the subgrade obtained using the *FWD*, E_{FWD}, *and* computed using ELMOD 5 are highly variable, while that obtained using *LWD*, E_{LWD}, and *DCP*, E_{DCP}, are relatively uniform. Also, the stiffness obtained from *DCP* is consistently higher than that obtained from *LWD*.

On May 13, 2014, *FWD* tests were conducted at the same location, but on top of the pavement, when the site was open to traffic. The results are plotted in Figure 2.5 together with the results of the *FWD* tests done on top of the subgrade. The figure shows uniform values and a reduction of the subgrade modulus when using the results of the test completed on top of the pavement. We hypothesize that this is due to two issues: one is the increase in confinement of the soil provided by the pavement, and the other is due to the loading of the subgrade due to traffic and changes of moisture content of the subgrade while in service.

2.2 Site on SR 37

The site was located on SR 37 in Mitchell, Lawrence County, Indiana, and consisted of a road-widening project (see Figure 2.6).

On September 23, 2013, *FWD*, *LWD*, *DCP* and sand cone tests were conducted on the subgrade. Figure 2.7 plots the estimated moduli obtained using the results of *FWD*, *LWD*, and *DCP* tests, as well as the in-situ water content and optimum water content of the subgrade.

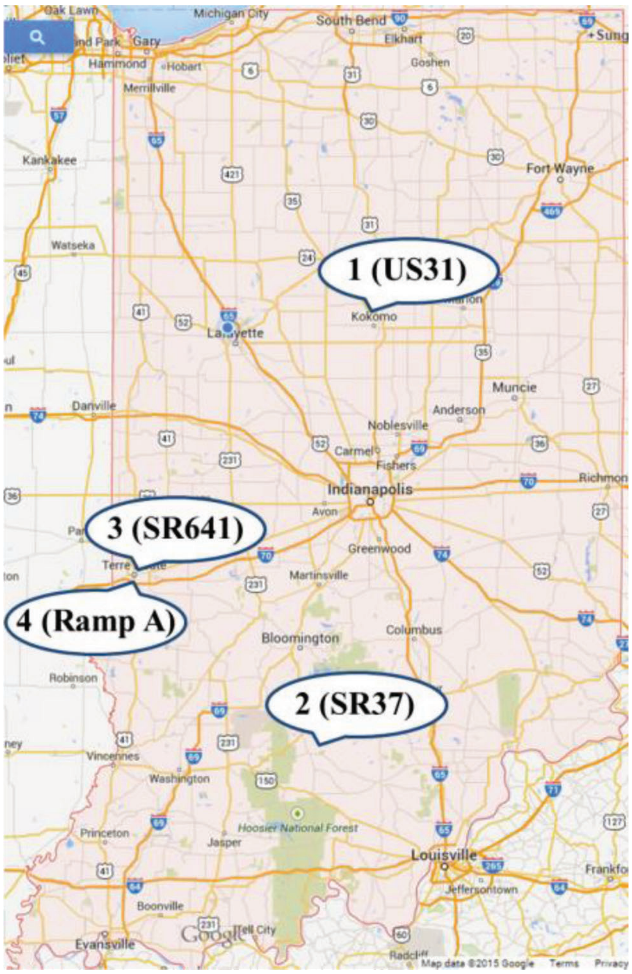


Figure 2.1 Location of the four test sites.

Figure 2.3 Photograph of the US 31 site.

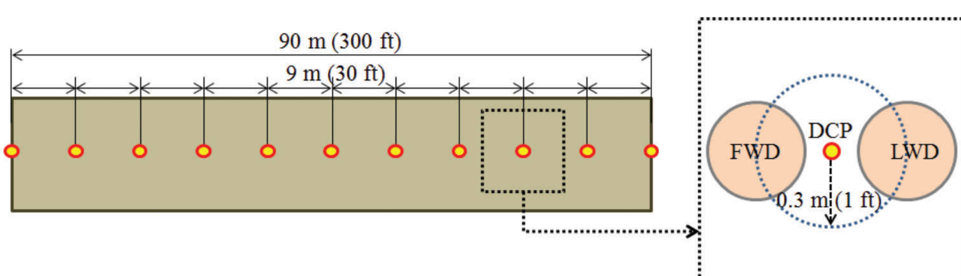


Figure 2.2 Schematic of selected section and test location.

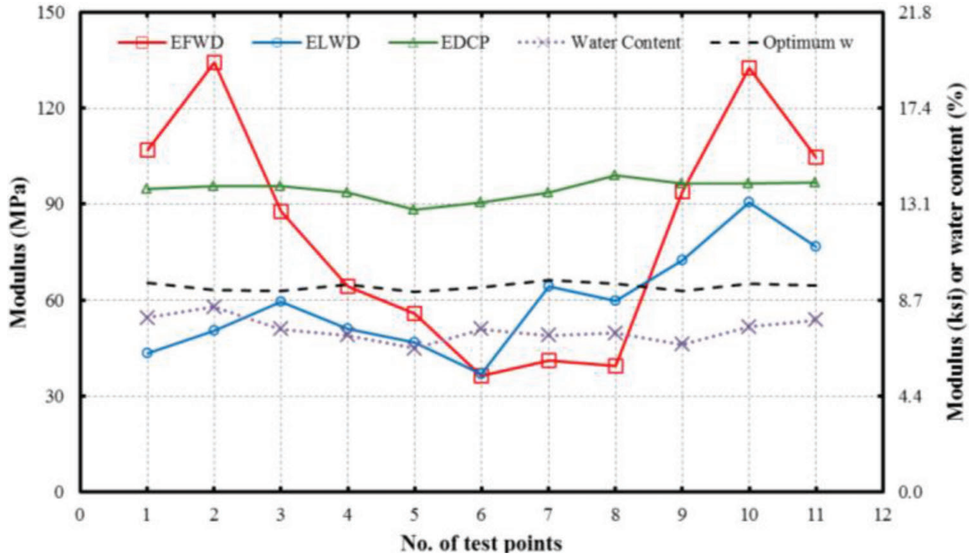


Figure 2.4 E_{FWD}, E_{LWD}, E_{DCP} and water content at US 31.

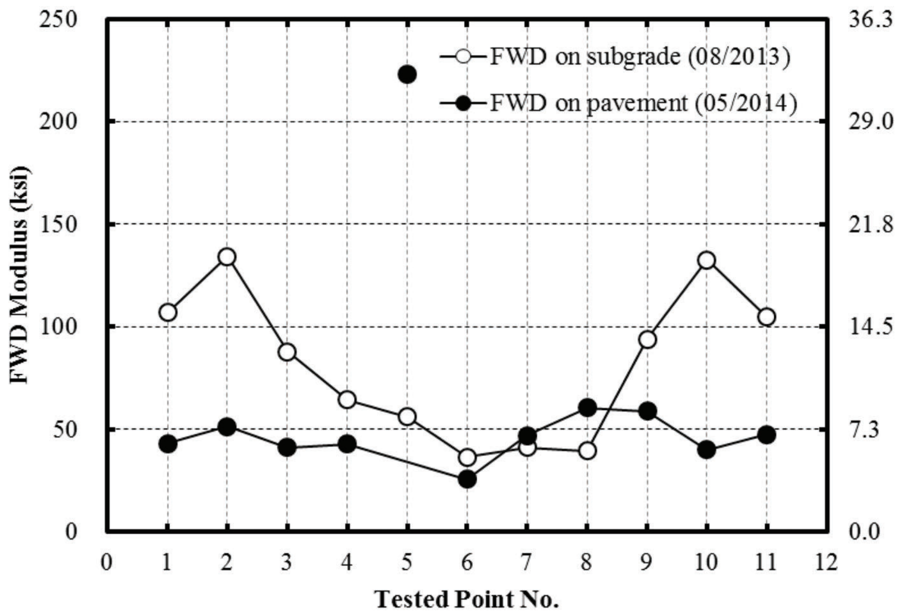


Figure 2.5 Comparison of E_{FWD} of the subgrade, from FWD tests on top of the subgrade and on top of the pavement. US 31.

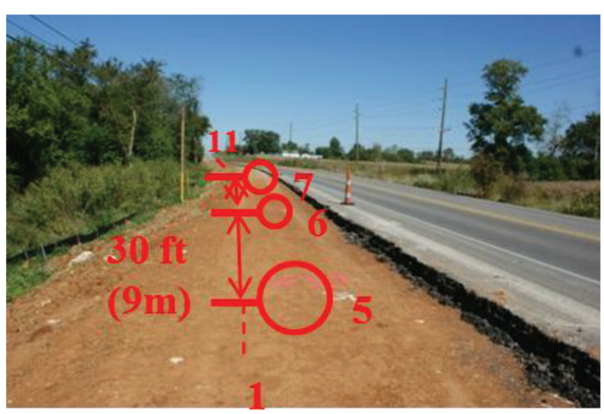


Figure 2.6 View of SR 37 site.

The values of E_{FWD}, as in the previous case, were highly variable and ranged from 31 MPa to 111 MPa, while from *LWD* and *DCP* were more uniform. The modulus values computed from *LWD* oscillated from 15 MPa to 31 MPa and those from *DCP* from 25 MPa to 65 MPa. The water content was close to the optimum, except at No. 5 location. Also, as with the previous site, the moduli obtained from *DCP* is larger than from *LWD*.

FWD tests were conducted on top of the pavement in 2005, in the spring (05/26/2015) and in the summer (08/04/2015), and at exactly the same locations as the tests performed earlier on top of the subgrade. Tests were done at points close to the rail to investigate the effect of lateral confinement. The subgrade moduli, from the *FWD* measurements, are plotted in Figure 2.8 together with the values obtained from the tests performed on top of the subgrade. Consistent with the findings at the US 31 site (Figure 2.5), testing on the pavement results is more uniform results, which supports the notion of increased consistency due to confinement, and perhaps due to effects of traffic (loading) history at the site, as well as changes in moisture content of the subgrade. As average, the moduli obtained from the tests in August are larger than those obtained in May, which is thought to be associated with a smaller water content of the subgrade in the summer than in the spring. Also, it can be noticed that the differences between the results obtained at the center of the site and at the edge, close to the railing, are not large, arguably within soil variability, which seems to indicate that there is no

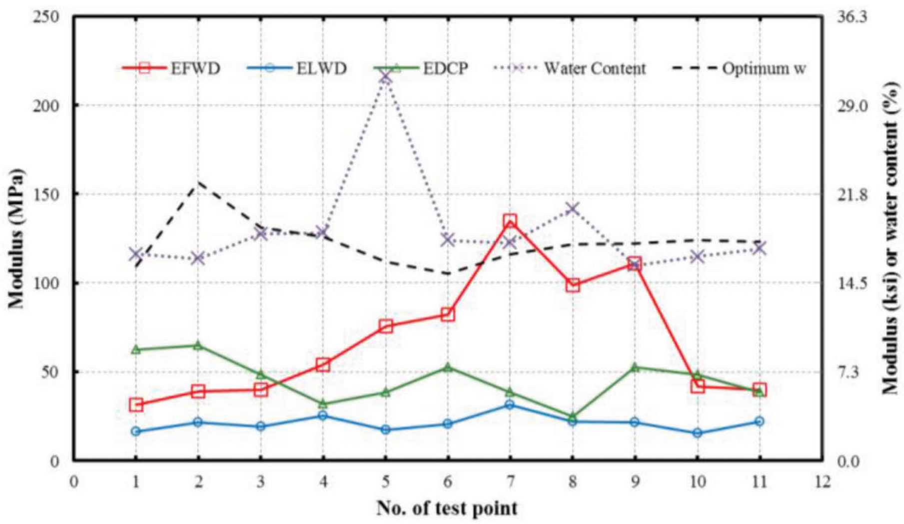


Figure 2.7 E_{FWD}, E_{LWD}, E_{DCP} and water content of the subgrade at the SR 37 site.

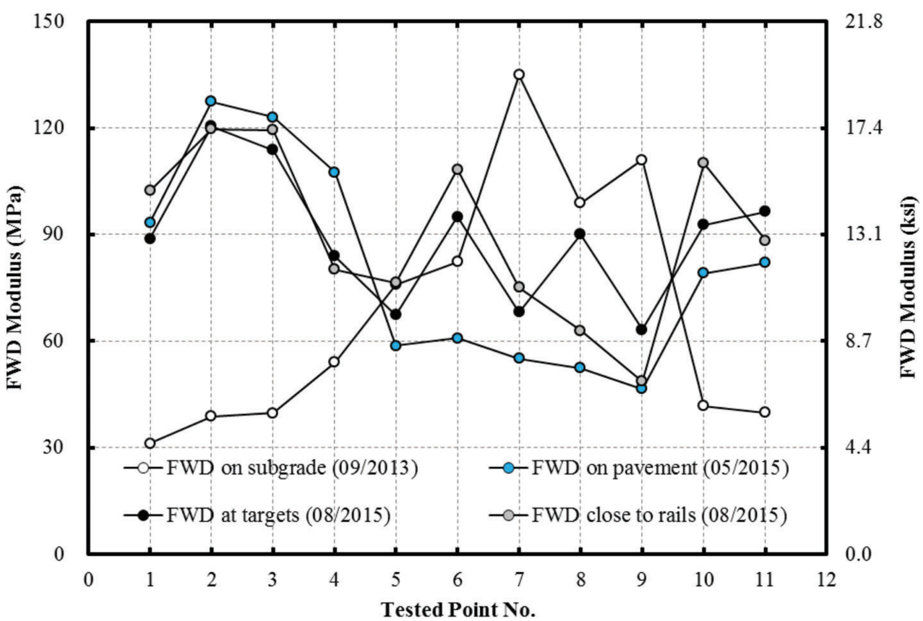


Figure 2.8 Comparison of E_{FWD} of the subgrade from *FWD* tests performed on top of the subgrade and on top of the pavement, SR 37.

substantial difference in confinement between the two locations.

2.3 Site on SR 641

The site was located on SR 641 in Terre Haute, Vigo County, Indiana. Figure 2.9 is a photograph of the site where the tests were performed.

The results of the *FWD*, *LWD*, *DCP*, and nuclear gauge tests, conducted on September 18, 2014, are plotted in Figure 2.10. The E_{FWD} values ranged from 8 MPa to 15 MPa and were lower than at the previous two sites (US 31 and SR 37). One of the reasons for the lower values was the higher water content at the site, which at most locations was about 4% higher than the optimum. The E_{LWD} was quite uniform, with values in the range of 5 MPa to 28 MPa. The modulus computed from *DCP* was very small. As shown in Figure 2.10, the estimated modulus values were below 20 MPa, which is too low. The high water content of the soil due to heavy rain before the tests was thought to be the cause for the low values.

2.4 Site on Ramp A

The site was located on Ramp A in Terre Haute, Vigo County, Indiana. Figure 2.11 provides a view of the site.

The field testing included *FWD*, *LWD*, and *DCP* tests and was conducted on June 2, 2015. Figure 2.12 plots the stiffness moduli obtained from *FWD*, *LWD*, and *DCP* tests, in-situ water content, and optimum water content. The *FWD* modulus ranged from 15 MPa to 25 MPa, except for one outlier (No. 11). The E_{LWD} values were fairly uniform and ranged from 9 MPa to 23 MPa. The E_{DCP} values were between 55 MPa to 70 MPa. They are higher than at SR 641, although the soils at SR 641 and Ramp A are analogous, i.e., they are both A-6; this may be due to the different water content at the two sites, with the one in SR 641 with

Figure 2.9 View of the SR 641 site.

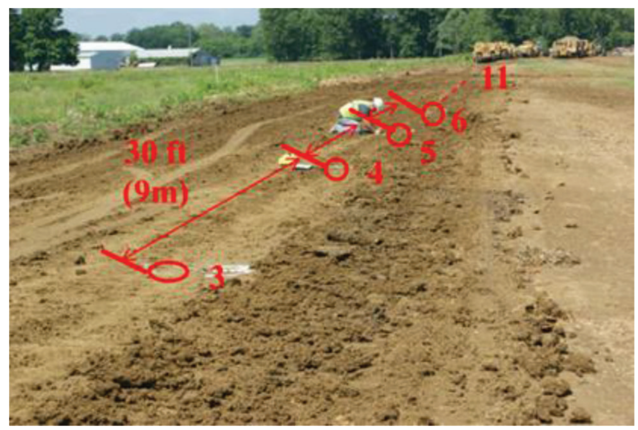


Figure 2.11 View of Ramp A site.

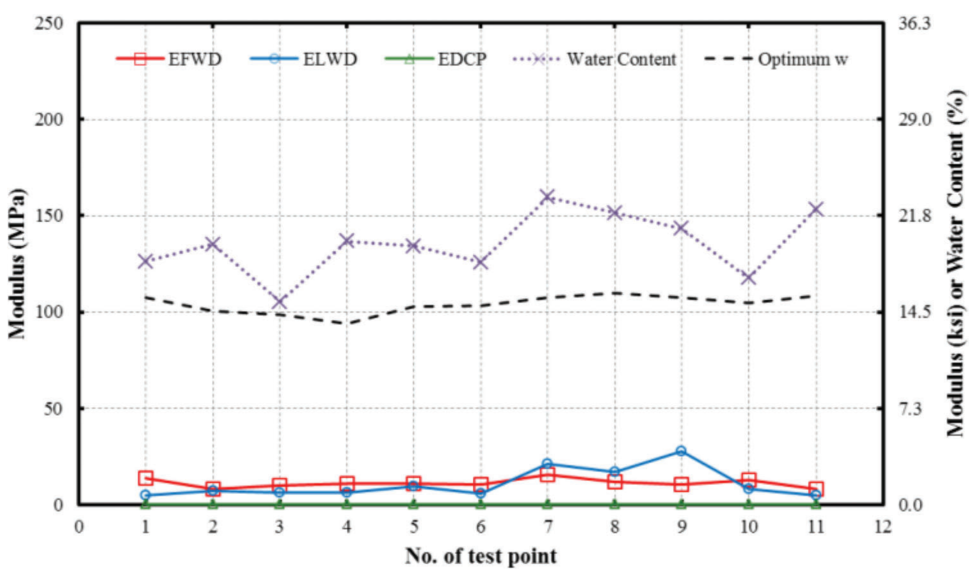


Figure 2.10 E_{FWD}, E_{LWD}, E_{DCP} and water content at the SR 641 site.

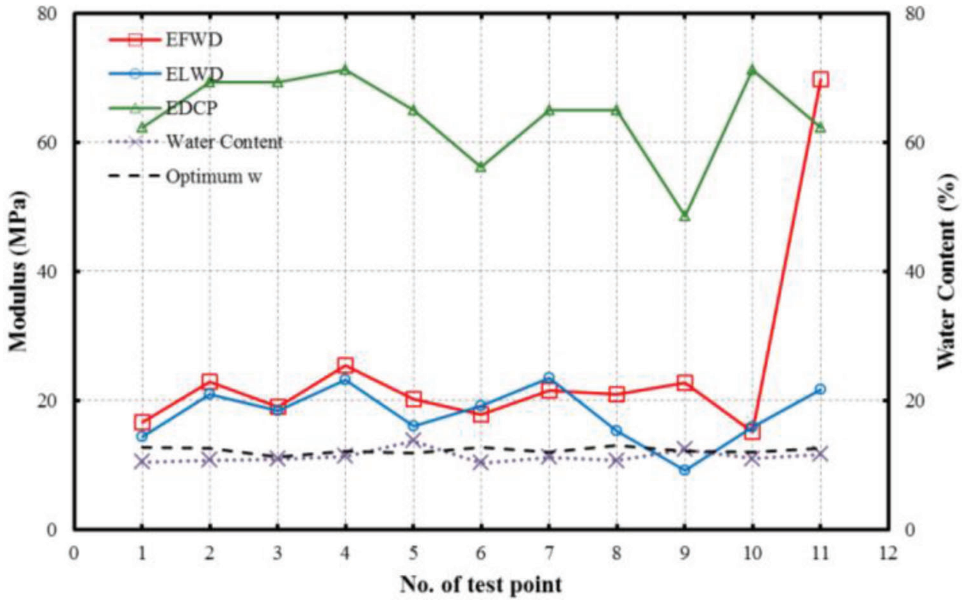


Figure 2.12 E_{FWD}, E_{LWD}, E_{DCP} and water content of the subgrade at Ramp A.

higher water content due to heavy rain in the days preceding testing. The in-situ water content (11.3%) of Ramp A site was similar to the optimum water content (12.3%). The modulus obtained from *FWD* and from *LWD* tests were similar, which was not the case in previous sites. Estimates from *DCP* yielded higher modulus than from the other two tests, which is also the case at all other sites.

3. LABORATORY TESTS

Laboratory tests were performed on the soil samples collected at each of the 11 locations at the four sites (US 31, SR 37, SR 641, and Ramp A). The following tests were conducted: Soil Characterization (sieving, hydrometer, compaction and Atterberg limit tests) and resilient modulus tests.

Table 3.1 provides a summary of the properties of the soils such as percentage of fines, Atterberg limits, in-situ water content, maximum dry unit weight, and soil classification. Based on AASHTO M 145-91 (2012), the subgrade at US 31 is classified as A-4, at SR 37 as A-7-6, and at SR 641 and Ramp A, as A-6. The details of the tests and the results can be found in Appendix Chapter 4.

The resilient modulus tests were performed to determine the stiffness of the soils in accordance to AASHTO T 307-99 (2007). Eleven tests were done at each site, each test on the soil samples taken at each one of the eleven stations. This was done to assess the variability of the results and provide a direct comparison between the laboratory results and the results from the field tests, since each would correspond to the exact same location.

Table 3.2 lists the average, minimum, and maximum values of the resilient modulus tests. The average M_R is: 45.1 MPa or 6.5 ksi at US 31; 92.1 MPa or 13.4 ksi at SR 37; 81 MPa or 11.8 ksi at SR 641; and 64.3 MPa or 9.3 ksi at Ramp A. There are some differences along each site, although soil properties (passing #200, *LL*, *PL*, *PI*, max. dry unit weight, and optimum water content) do not show variability. The differences between max. M_R and min. M_R were 55.9 MPa (8.1 ksi) for US 31 soils, 43.9 MPa (6.4 ksi) for SR 37 soils, 54.4 MPa (7.9 ksi) for SR 641 soils, and 26.6 MPa (3.8 ksi) for Ramp A soils.

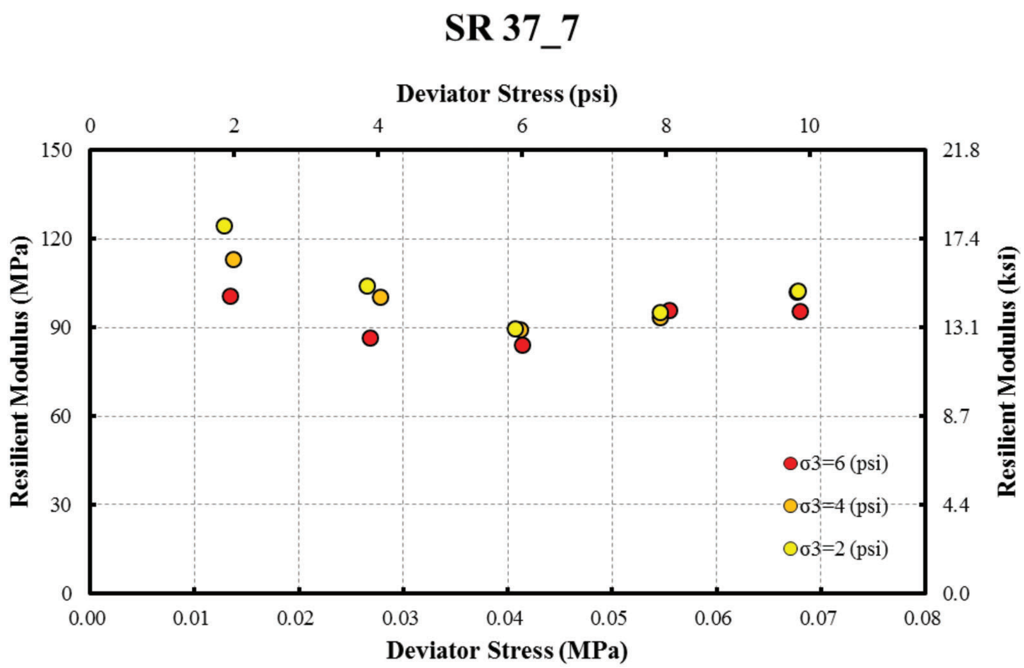
Representative results of the resilient modulus tests are plotted in Figure 3.1. The trend for all subgrade soils (US 31, SR 37, SR 641 and Ramp A sites) is similar: there is no clear effect of deviator stress, nor of confining stress. There are a few exceptions that show a somewhat deviator stress dependency, but most of the results are not affected by deviator stress. All the resilient modulus test results are included in Appendix Chapter 4.

TABLE 3.1
Characterization and classification of the subgrade soils

Site	#200 Passing (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index	Optimum Water Content (%)	Max. Dry Unit Weight (kN/m^3)	AASHTO Classification
US 31	58	18.6	10	8.5	9	21.0	A-4
SR 37	88	41.4	17.7	23.8	16~20	17.0	A-7-6
SR 641	83	31.2	13.8	19.4	14~16	18.0	A-6
Ramp A	72	29.0	15.0	14.0	12~13	18.7	A-6

TABLE 3.2
Average, max. and min. values of the resilient modulus at US 31, SR 37, SR 641, and Ramp A

	US 31	SR 37	SR 641	Ramp A
Average (MPa)	45.1	92.1	81.2	64.3
Min. (MPa)	23.5	71.2	51.1	54.4
Max. (MPa)	79.4	115.1	105.5	81.0



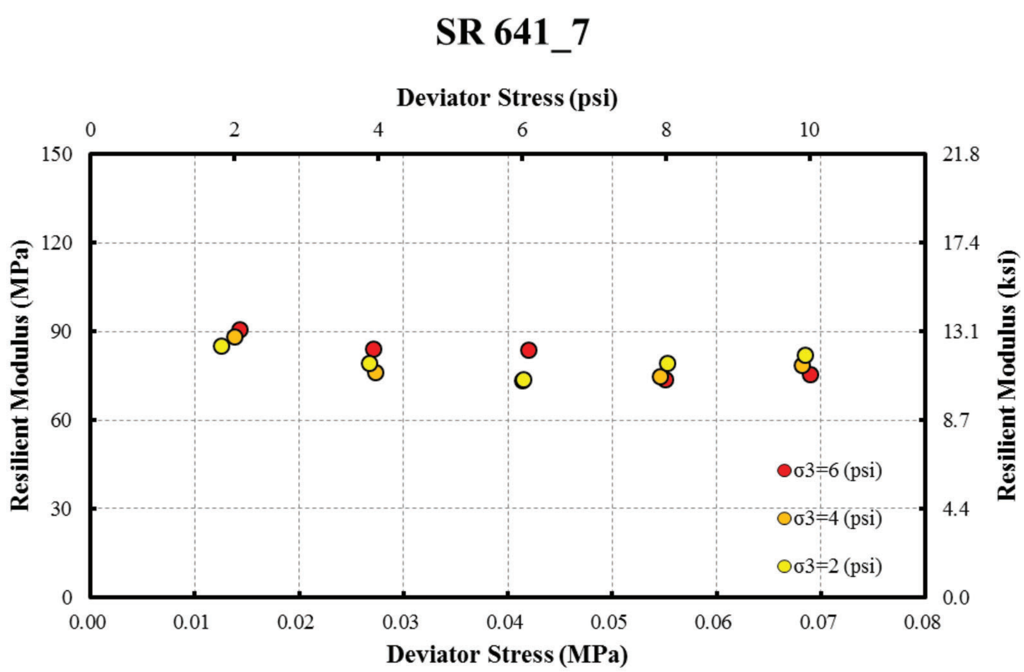
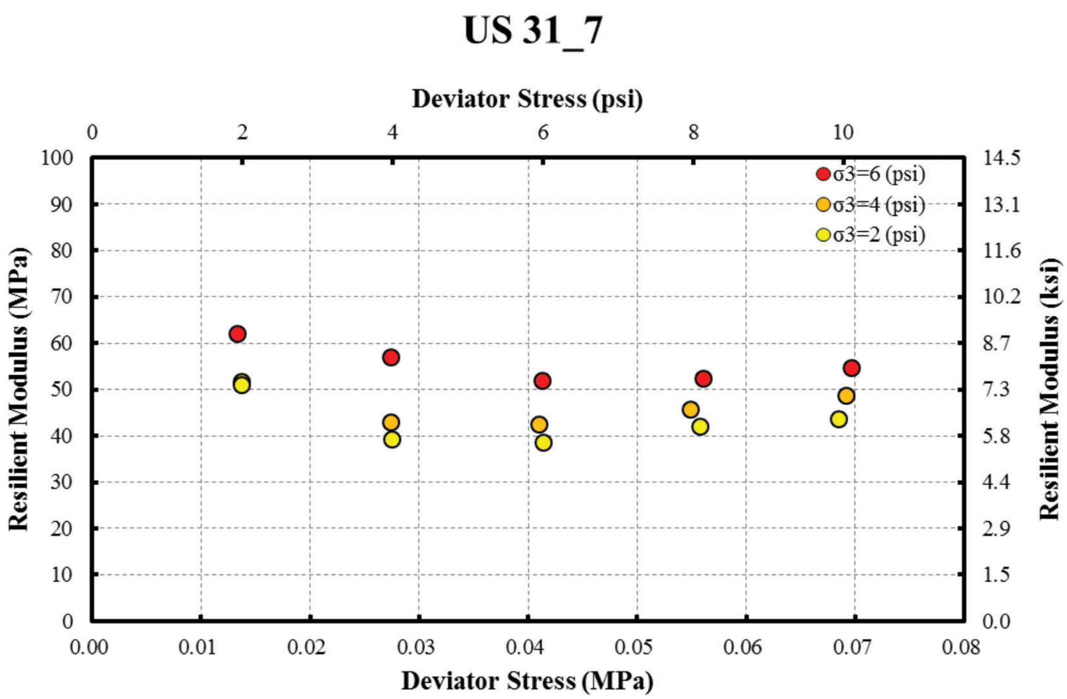


Figure 3.1 Select resilient modulus test results at US 31, SR 37, SR 641, and Ramp A. (Figure continued next page.)



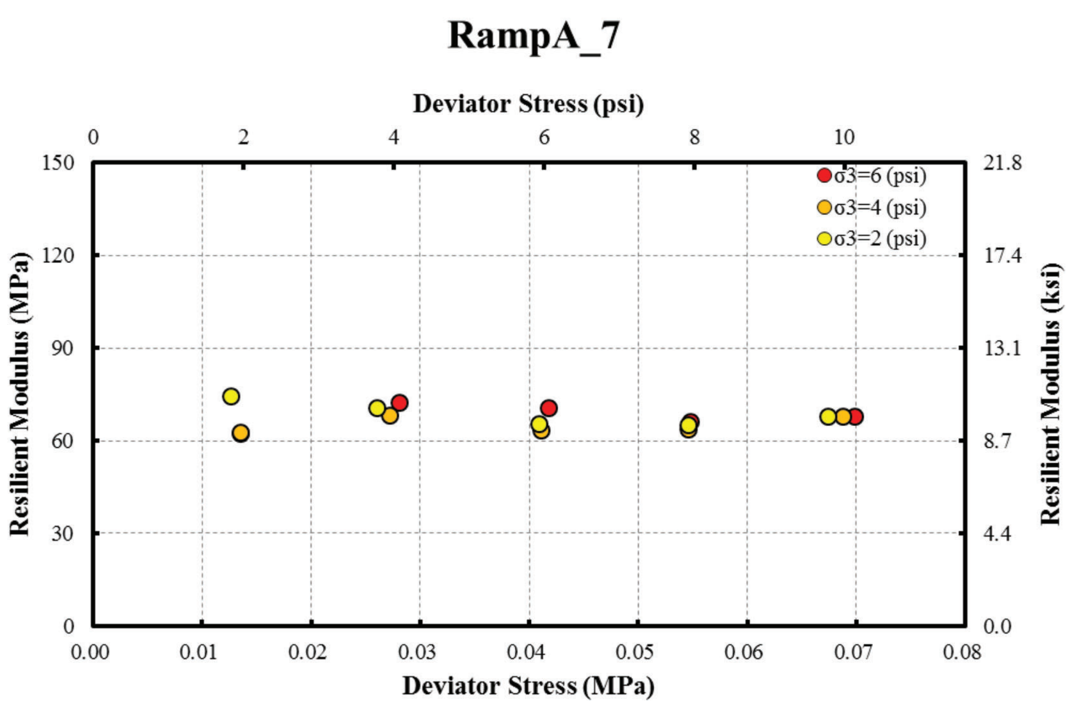


Figure 3.1 (Continued)

4. ANALYSIS OF TEST RESULTS

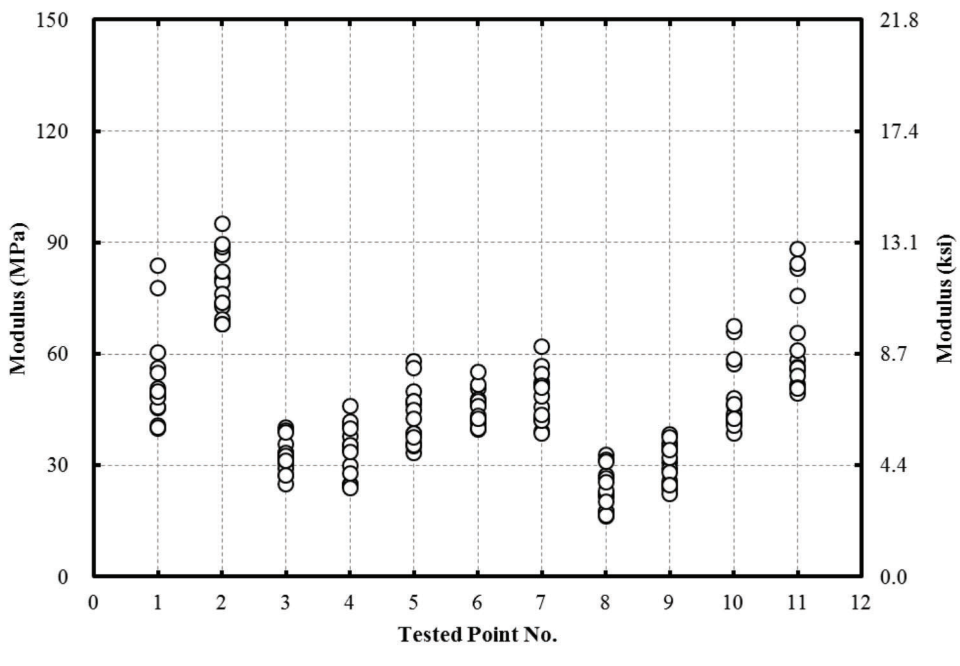
There are a number of proposals in the literature for M_R models for cohesive soils. Most models depend on the bulk stress, the deviator stress, and the confining stress. The universal model (Uzan, 1985) and the Octahedral stress model (NCHRP, 2004) are well known. However, the resilient modulus test results from the four selected sites (see Section 3) do not show stress-dependent behavior, not only on confining stress, but also on deviator stress. For this study, a stress-independent model seems to apply. That is,

$$M_R = k_1 p_a$$

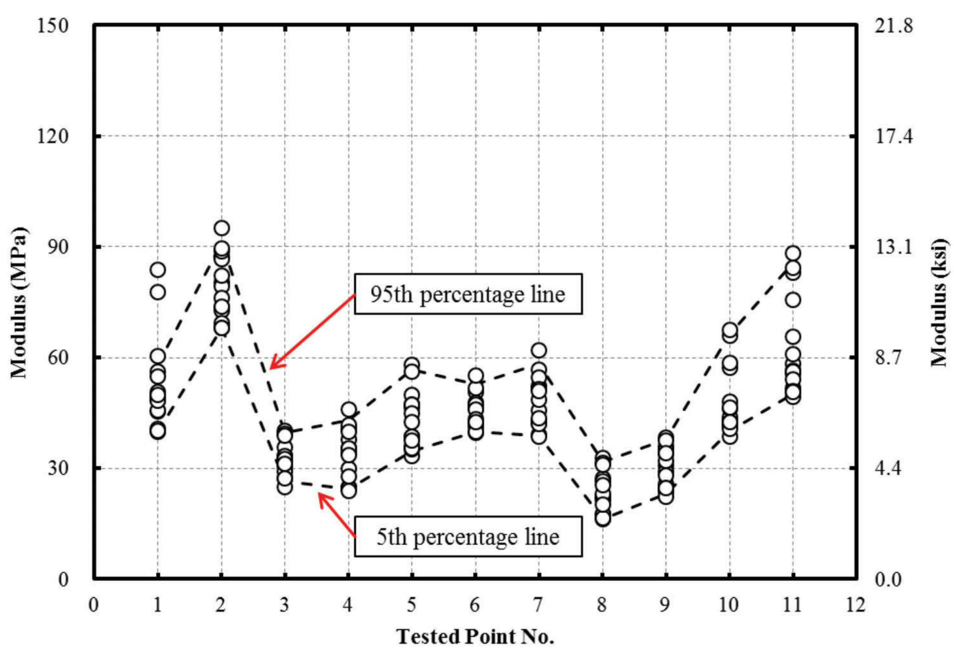
where, k_1 = regression coefficient; p_a = atmospheric pressure.

An attempt has been made to obtain insight into the regression coefficient as a function of a number of soil parameters such as optimum water content, maximum dry unit weight, percentage soil passing #200 sieve, and Atterberg limits, which are classified as important input variables (Drumm, Boateng-Poku, & Pierce, 1990; George, Bajracharya, & Stubstad, 2004). In general, there is a poor correlation between soil properties and resilient modulus, with values of the R^2 index as low as 0.14.

The M_R values from the laboratory are compared with those estimated from *FWD*, *LWD*, and *DCP* tests.

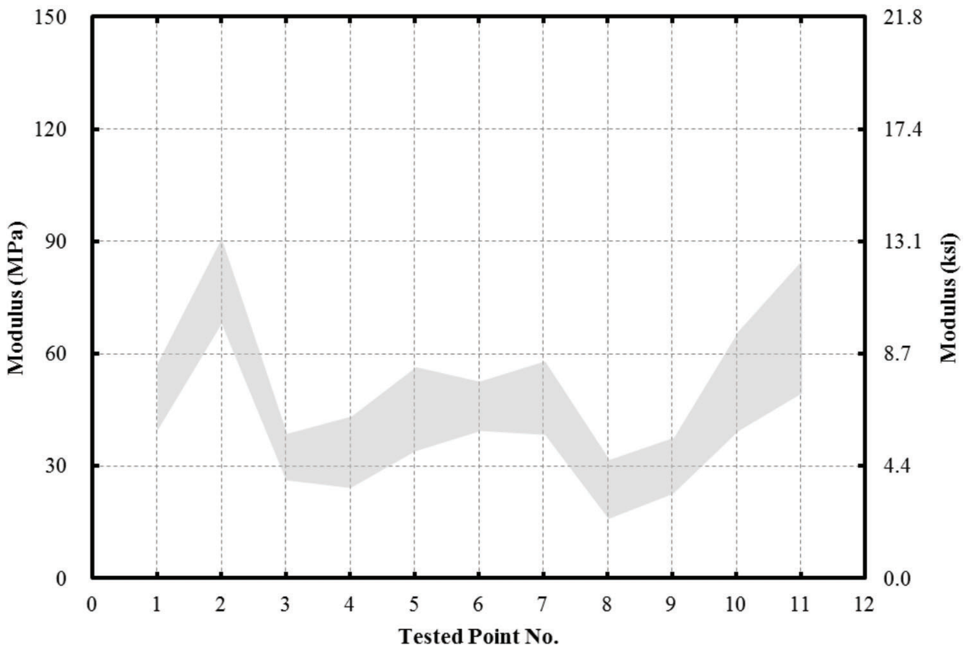


(a) M_R of soil samples



(b) 95th and 5th percentile line

Figure 4.1 Range of resilient modulus values (US 31 site). (Figure continued next page.)



(c) Range of *MR* values

Figure 4.1 (Continued)

The code ELMOD 5 is used to obtain the soil stiffness from *FWD* tests, the Boussinesq's equation is utilized to calculate the soil stiffness from the *LWD* tests, and the Salgado and Yoon's (2003) relationship to interpret the *DCP* results. The values of the resilient modulus obtained in the laboratory show variations based on location, i.e., from test point to test point, and on confinement and deviator stress, albeit there was no strong relation, as discussed. Figure 4.1a includes all the laboratory tests on US 31. To facilitate comparisons, the laboratory tests are grouped into a band that includes all the results that fall into the 5% and 95% percentile, as depicted in Figure 4.1b. This is plotted, for the rest of the discussion, as the band of results where the M_R falls; see Figure 4.1c.

Figure 4.2 shows a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on A-4 soils (US 31). The horizontal axis represents the test location at the site while the vertical axis provides the values of the resilient modulus in MPa (left) or ksi (right). As one can see, estimates from *DCP* results overestimate the stiffness of the soil. This is also the case with the values from *FWD* when obtained from tests conducted on top of the subgrade. Results from *LWD* and from *FWD* conducted on top of the pavement provide acceptable estimates, with somewhat better results from the *FWD*.

Figure 4.3 shows a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on A-7-6 soils (SR 37). None of the estimates using measurements from *LWD*, *DCP* or *FWD* tested directly on the subgrade provide acceptable values. Results from *FWD* tests performed on the pavement give a better approximation of the resilient modulus obtained in the laboratory, in particular from the test conducted in the summer. The *FWD* measurements made in the spring also provide acceptable results, albeit on the low side. As mentioned earlier, the smaller values of the *FWD* modulus in May are likely associated with a higher water content of the subgrade due to rain.

Figure 4.4 shows a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on A-6 soils (SR 641 site). Similar to the observations made for the A-7-6 soils in Figure 4.3, none of the estimates from the field tests, including the *FWD* performed directly on the subgrade provide good estimates. They are all too low.

Figure 4.5 is analogous to the previous figures. It presents a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on Ramp A, where the soils are classified as A-6 (similar to the SR 641 site with resilient modulus values shown in Figure 4.4). The *LWD* and the *FWD* results are similar to each other, but they are too low. The *DCP* modulus coincides well with the modulus obtained in the laboratory. Note that this is quite different from what was found in SR 641, which has a similar soil.

Figure 4.6 provides a comparison of the resilient modulus obtained in the laboratory following AASHTO T 307-99 (2007) and from *FWD* tests. For the comparison, the average values of the laboratory test results are used (see discussion in Section 4). In the figure, hollow symbols are used for the *FWD* modulus obtained from

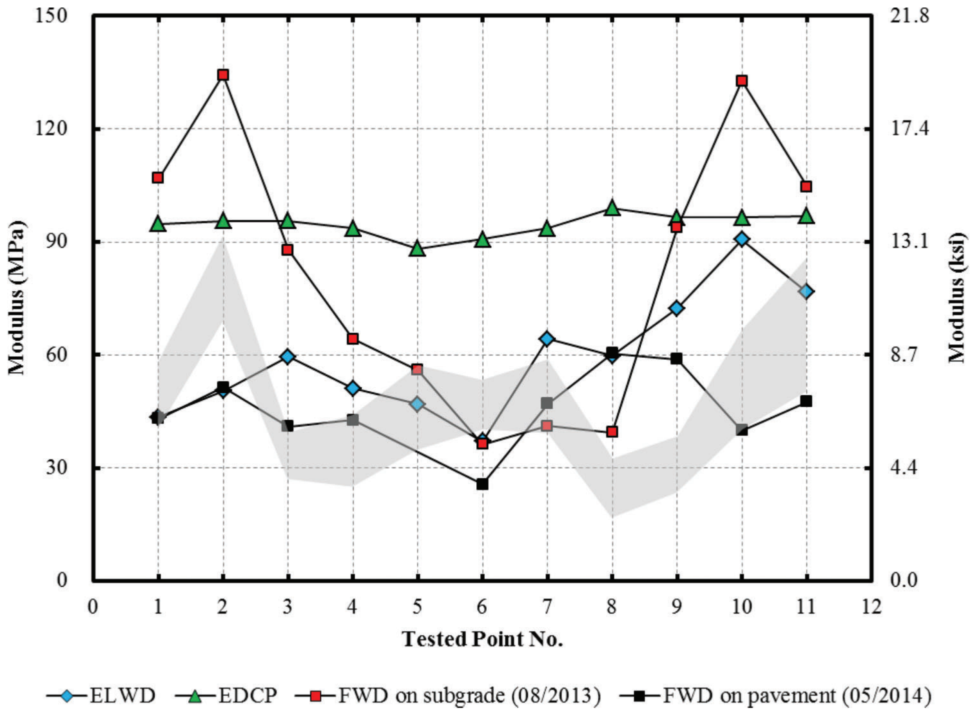


Figure 4.2 Comparison between the resilient modulus measured in the laboratory and obtained from *FWD*, *LWD*, and *DCP* tests on A-4 soils (US 31 site).

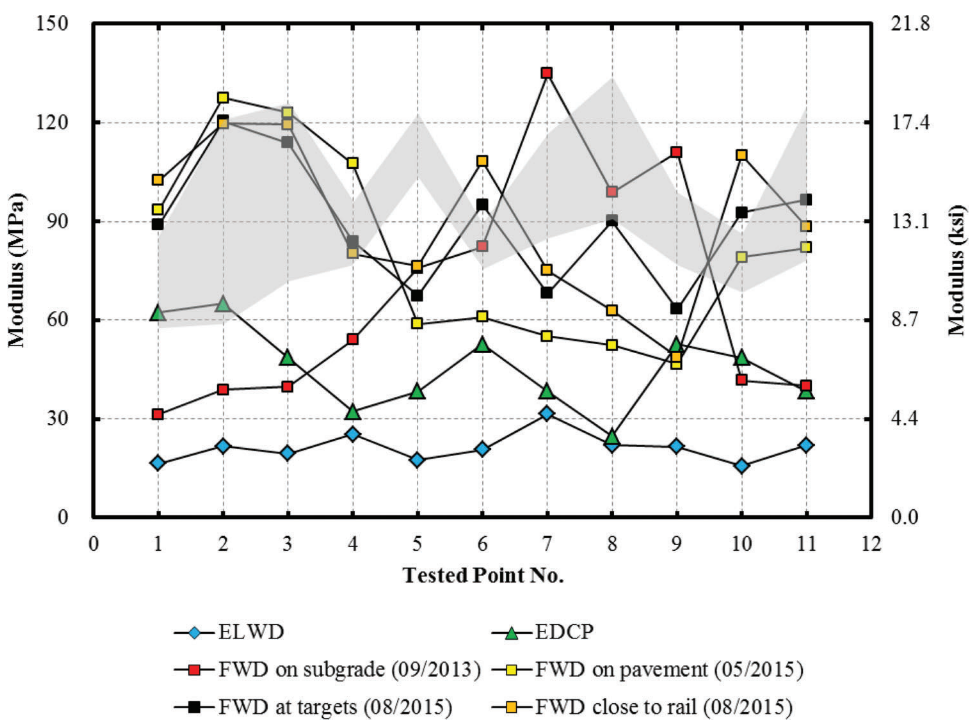


Figure 4.3 Comparison between the resilient modulus measured in the laboratory and obtained from *FWD*, *LWD*, and *DCP* tests on A-7-6 soils (SR 37 site).

tests done directly on top of the subgrade and solid symbols from tests on top of the pavement.

The *FWD* results on the subgrade soils (hollow symbols) seem to be randomly scattered and do not show a clear trend, nor there seems to be a relation between these values and the resilient modulus from the laboratory tests. In contrast, the modulus obtained from *FWD* tests performed on top of

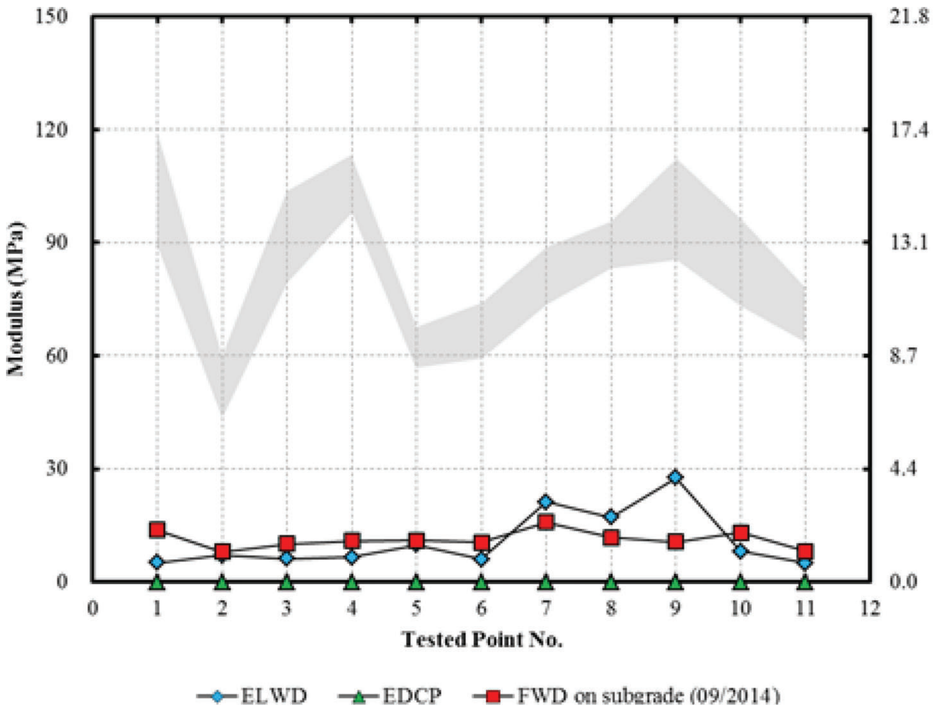


Figure 4.4 Comparison between the resilient modulus measured in the laboratory and obtained from *FWD*, *LWD*, and *DCP* tests on A-6 soils (SR 641 site).

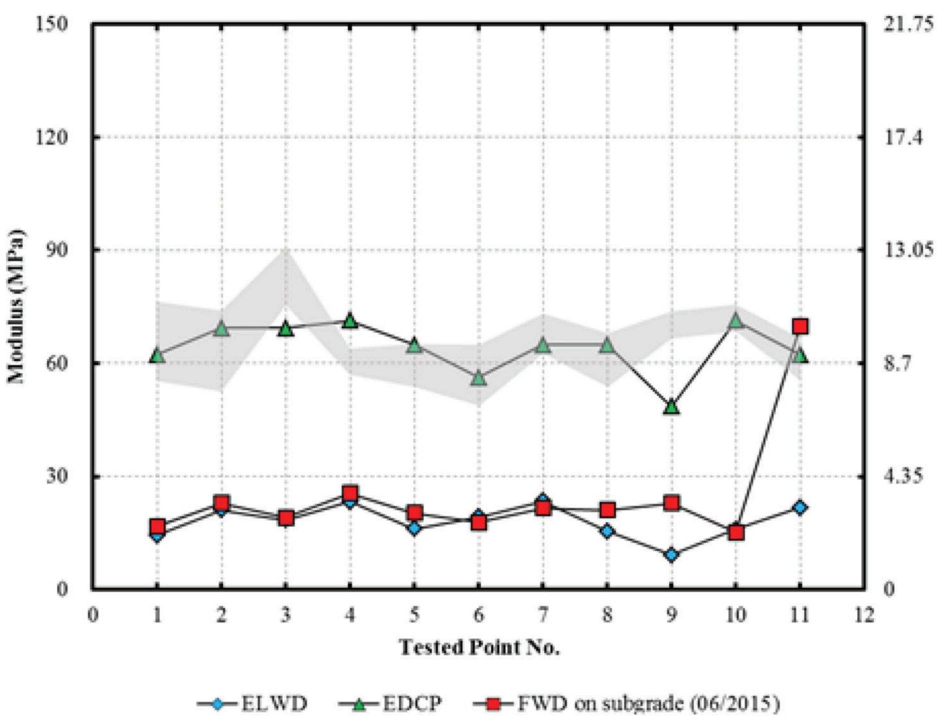


Figure 4.5 Comparison between the resilient modulus measured in the laboratory and from *FWD*, *LWD*, and *DCP* tests on A-6 soils (Ramp A site).

the pavement do compare well with the laboratory measurements. Note that this is the case for all eleven points at each of the two sites where the test could be completed.

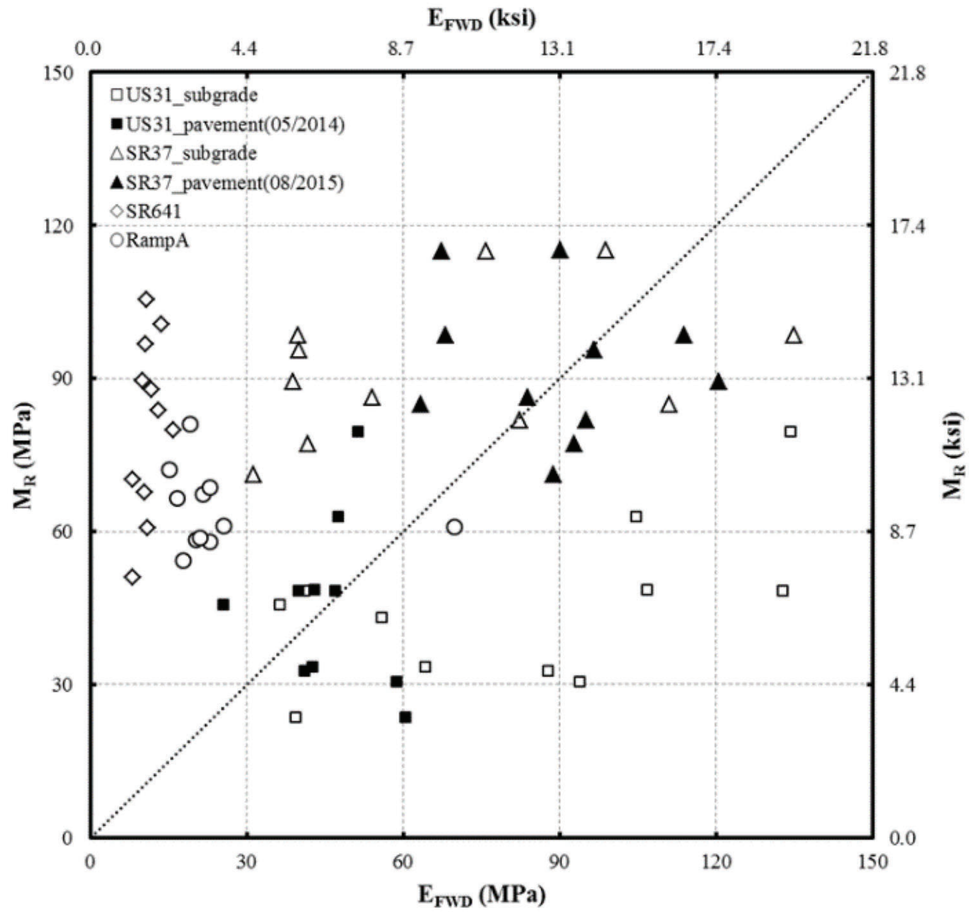


Figure 4.6 Resilient modulus. Comparison between results from laboratory and *FWD* tests.

5. DATA COLLECTION FROM INDOT

Given the need to collect and analyze additional data to support the observations in Figure 4.6, it was decided to expand the scope of the project and mine the data repository of INDOT to obtain additional geotechnical and pavement information that could be used to further investigate relations between field *FWD* and laboratory resilient modulus tests.

Mr. Nayyar Siddiki provided resilient modulus data, obtained from laboratory tests, that included: resilient modulus at 6 psi deviator stress and 2 psi confining stress at optimum moisture content; location based on road number and county; soil classification; and Proctor tests results. Dr. Yigong Ji provided *FWD* data since 2008 that included: location based on road number and RP number; deflection data; and *FWD* modulus calculated using ELMOD 5.

The resilient modulus and the *FWD* data were obtained independently of each other; that is, the test location and time do not match. The first step consisted of pairing the data, as best as possible, based primarily on location.

To facilitate pairing the two sets of data, the commercial software ArcMap, a GIS based code, was used. The locations of the resilient modulus and *FWD* tests were digitized and included in the software, as shown in Figure 5.1.

The locations of the soil samples used to obtain the resilient modulus in the laboratory are approximate. They are given in terms of the road number on a County. Therefore, M_R data are assumed representative of the entire road, which may carry a significant uncertainty, as the values are compared with the stiffness obtained from *FWD*, where the location is well specified.

Using ArcMap has distinctive advantages for the project, as the software allows users to classify the data into "layers": three layers for the resilient modulus, each for a particular type of soil, namely A-7-6, A-6 and A-4, and one for the *FWD*. The layers are color-coded and can be activated or deactivated, thus facilitating the spatial visualization of the data into different categories.

The data, as displayed into ArcMap, from the field, *FWD*, and laboratory, M_R, is paired by location. Clearly, given the uncertainties discussed, mostly from the M_R location, it is not possible to unequivocally establish one-to-one relations. Instead, the data is classified into three tiers. If the M_R data has only one type of soil at a given road and only one *FWD* data point exists, the data is classified as tier one. If the M_R data has only one type of soil but multiple *FWD* data

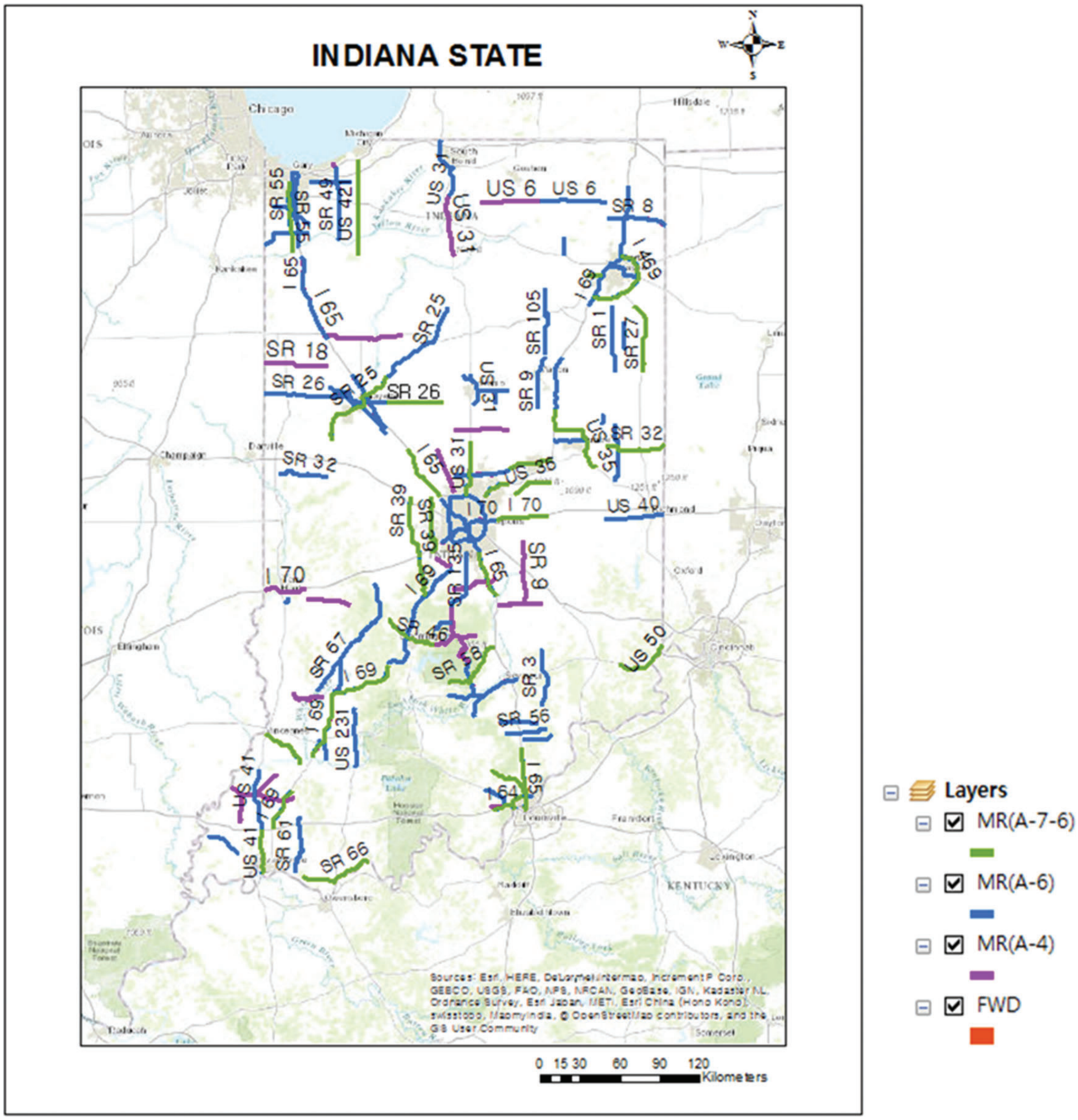
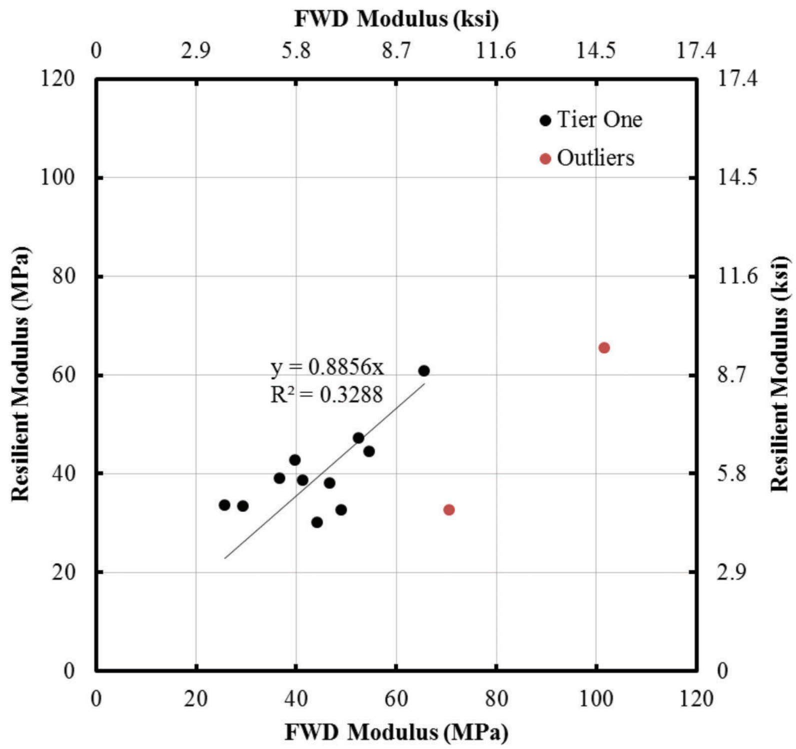


Figure 5.1 Location of collected resilient modulus and *FWD* data in ArcMap.

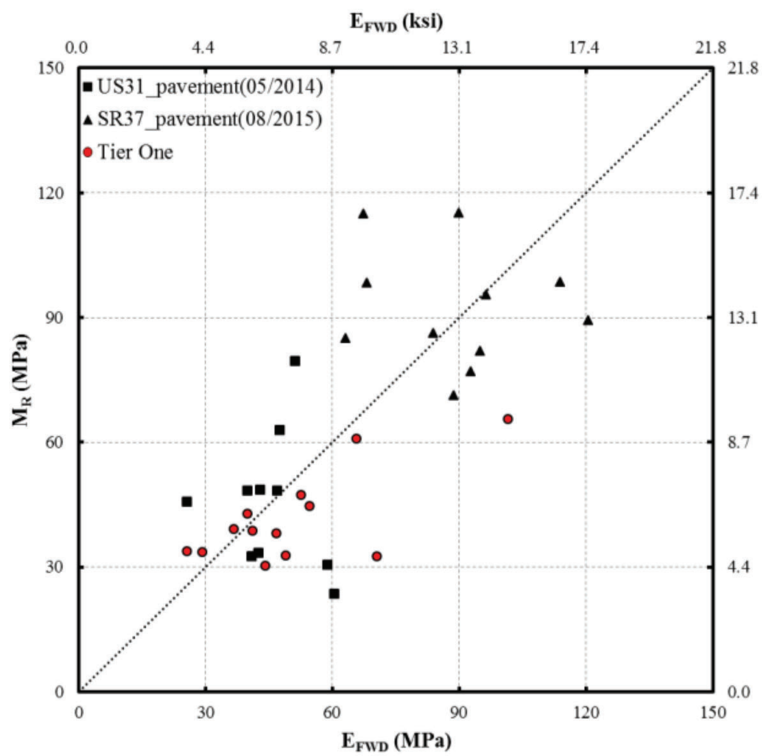
exist, at the same location, the data falls into tier two. If the M_R data includes multiple types of soils, the data are categorized as tier three regardless of the number of *FWD* data.

Thirteen data points fall into tier one, which is the tier thought to be the most reliable for comparison purposes. The tier one data is plotted in Figure 5.2a, which shows a good correlation between the resilient and the *FWD* modulus, and similar to the observations made during phase one of the project (see Figure 4.6, reproduced here as Figure 5.2b, which also includes the data from Figure 5.2a).

Tier two includes 37 data points and tier three 118 data points. The comparison between M_R and *FWD* data, classified as tier two and three, is shown in Figure 5.3a and Figure 5.3b, respectively. Tier two and tier three do not show a strong correlation between M_R and *FWD*. This is somewhat expected due to the higher uncertainty associated with how the values of M_R and *FWD* have been matched.

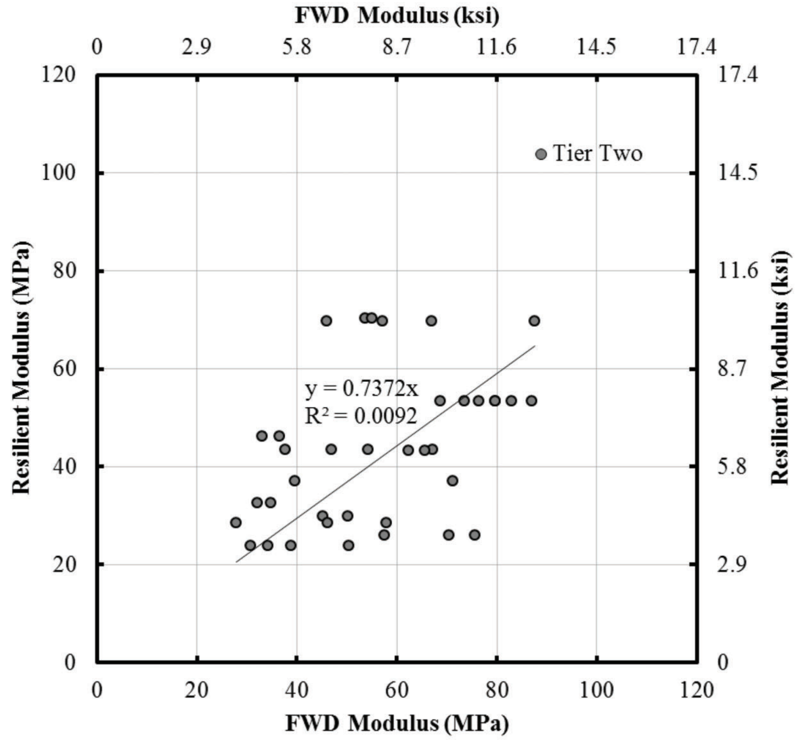


(a) Tier one

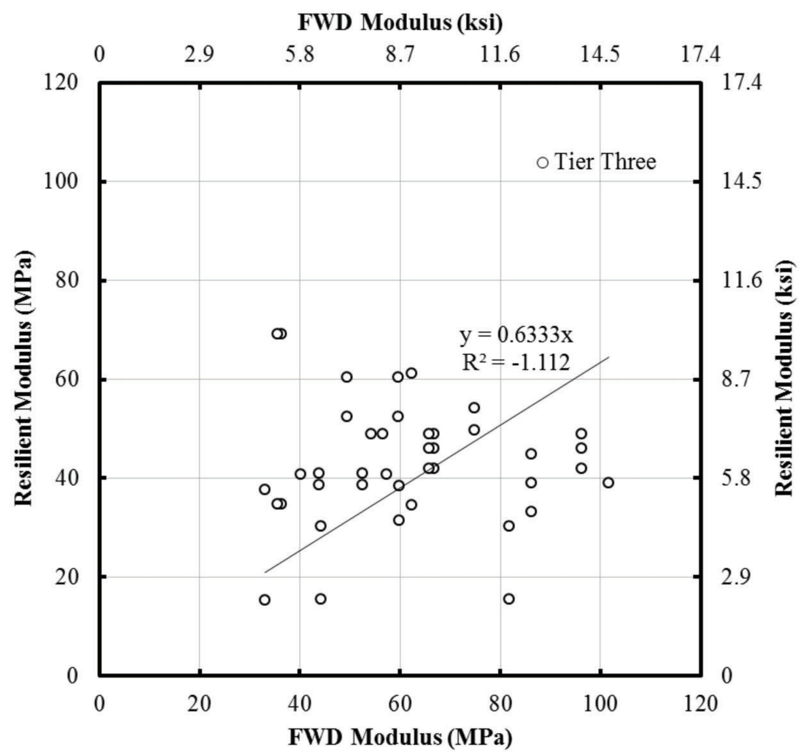


(b) Tier one vs. phase one results

Figure 5.2 Comparison between M_R and FWD for tier one data.



(a) Tier two



(b) Tier three

Figure 5.3 Comparison between M_R and FWD for tier two and tier three data.

6. ANALYSIS OF COLLECTED DATA

Resilient modulus data and *FWD* data have been collected from INDOT databases (see preceding Section and Appendix Chapter 6 for more details). Two sets of data were collected: resilient modulus from the Geotechnical Engineering Division and *FWD* from the Pavement Division. The two independent data sets were paired based on location and classified into three tiers.

As shown in Figure 5.2b, the data categorized into tier one shows a meaningful correlation between M_R and *FWD*, which supports the results from phase one of the project. Tier two and tier three data do not show strong correlations due to the uncertainty associated with location and type of soil. As a consequence, it was decided to further investigate tier one data only.

ELMOD5, the code used by INDOT to interpret the results from *FWD* tests, uses a three-layer pavement model. This is thought to be a limitation, since pavements are built with several layers, and so more precise interpretation of the *FWD* results could possibly be attained using models that allow for a larger number of pavement layers. In discussions with Dr. Orr, it was decided to use the code MODTAG (Borter & Irwin, 2006), in conjunction with ELMOD. MODTAG is a back-calculation program developed by VDOT (Virginia Department of Transportation) and Cornell University. MODTAG uses an iterative deflection basin fit method that adjusts the moduli of pavement layers until the calculated deflection matches the measured deflection basin. Further, ELMOD5 assumes that the stiffness of the pavement layers decreases with depth, which may or may not be always the case. This limitation does not exist with MODTAG.

First, outcomes from US 31 and SR 37 were analyzed because of the high quality of the data and pavement layer information. Results of a three-layer analysis using MODTAG were compared with the results using ELMOD5, also using a three-layer model. This was done to identify any differences between the two codes when all input parameters and problem definition are identical. The results are shown in Figure 6.1a. Data points obtained with ELMOD5 are plotted using black symbols and data points using MODTAG are plotted with white symbols. The results from MODTAG are similar to the results from ELMOD5, since both results are distributed around the 1:1 reference line. Figure 6.1b shows similar outcomes, but using a three-layer model with MODTAG for US 31 (US 31 has three layers: asphalt surface, base and subgrade) and a four-layer model for SR 37 (SR 37 has four layers: asphalt surface, base, subbase and subgrade). Included in the figure are the results from ELMOD5 to compare with the results from MODTAG. The *FWD* moduli calculated with MODTAG are relatively lower than those calculated with ELMOD5. The M_R or *FWD* modulus using ELMOD5 is roughly 1.6 times higher than the *FWD* results obtained using MODTAG. Additional calculations were done increasing the number of layers. Figure 6.1c shows the results using MODTAG with a four-layer pavement for US 31, by adding a natural subgrade layer, and a five-layer pavement for SR 37, also by adding a natural subgrade layer.

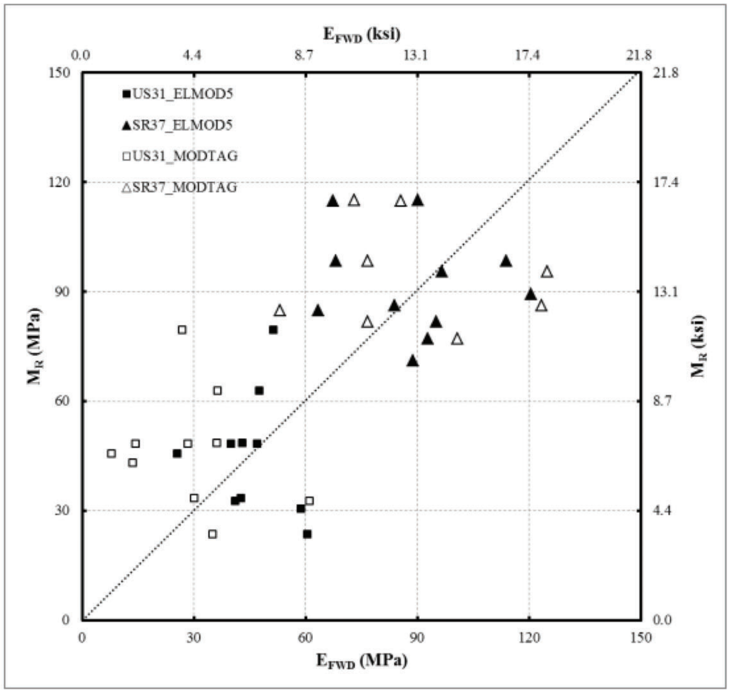
Figure 6.1 indicates that by just adding layers to the model, the variability/scatter of the results increases. This is somehow an unexpected result, as a better, more precise description of the pavement, should improve the accuracy and quality of the interpretation. To investigate the issue, manual backcalculation was performed, taking advantage of the expertise of Dr. Orr. The details of the analyses can be found in Appendix Chapter 8.

For the MODTAG analysis, five-layer models were adopted, i.e., 4″ AC, 6″ granular base, 14″ upper subgrade, 48″ middle subgrade and infinite subgrade for US 31; and 4″ AC, 6″ AC base, 14″ lime treated, 12″ upper subgrade and infinite subgrade for SR 37. The code CHEVLAY (Irwin, 1994), a multi layered elastic program, was used to further investigate the results. The details of CHEVLAY analysis can be found in Appendix Chapter 8.

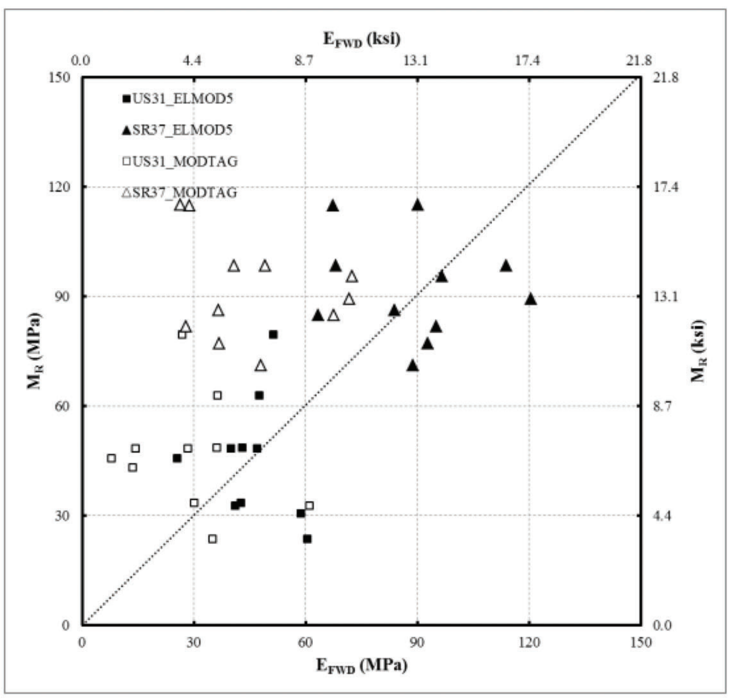
Results from MODTAG and ELMOD5 are compared in Figure 6.2. Figure 6.2a is a comparison between MODTAG and ELMOD5, for US 31, and Figure 6.2b for SR 37. Data points obtained with ELMOD5 are plotted using black symbols and data points using MODTAG are plotted with white symbols (Figure 6.2). Utilizing the analysis resources provided by CHEVLAY, US 31 results from MODTAG are in good agreement with results from ELMOD5, except for few outliers. However, there are disparities between the two results for SR 37, as shown in Figure 6.2b. ELMOD5 results are, generally, 2.2 times higher than MODTAG. The disparity is likely due to errors in pavement thickness or data input; more specifically, the analysis seemed to indicate that the actual thicknesses of the pavement layers might be different that those of design (used by ELMOD5).

In addition to data collected from US 31 and SR 37, additional information was gathered, as discussed, from the INDOT database. With help from Dr. Jusang Lee, the pavement information for tier one data was obtained from ProjectWise. ProjectWise is a program used to store data from INDOT projects. Unfortunately, not all desired information from the thirteen sites in tier one was available. The pavement layer thickness was found for only two sites, namely SR 60 and SR 46; see Table 6.1 and Table 6.2. *FWD* deflection files (.F25) for the two sites were found with the help of Dr. Seonghwan Cho. Backcalculation of *FWD* results for SR 60 was not successful. The *FWD* raw data file (.F25) for the site might have been corrupted and thus produced errors during backcalculation.

The analysis of SR 46 was conducted with MODTAG, with a four-layer pavement, i.e., 4.5″ AC, 5.5″ granular base, 14″ upper subgrade, and infinite subgrade. Note that a 4.5″ surface layer was used because the upper two layers were too thin to be differentiated in the calculations. The results from MODTAG are compared with the

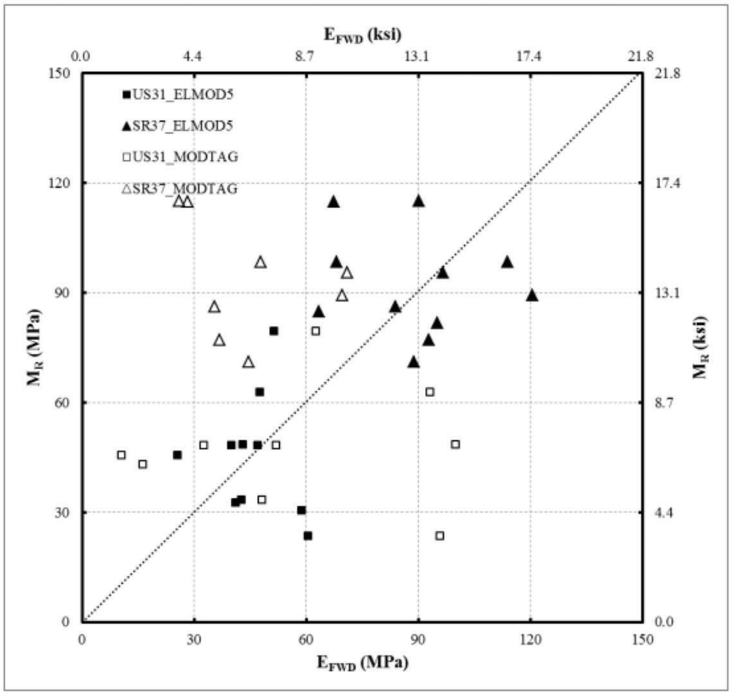


(a) Analysis of US 31 and SR 37 using ELMOD and MODTAG with a three-layer pavement



(b) Analysis of US 31 and SR 37 using MODTAG with a three-layer pavement for US 31 and four-layer for SR 37

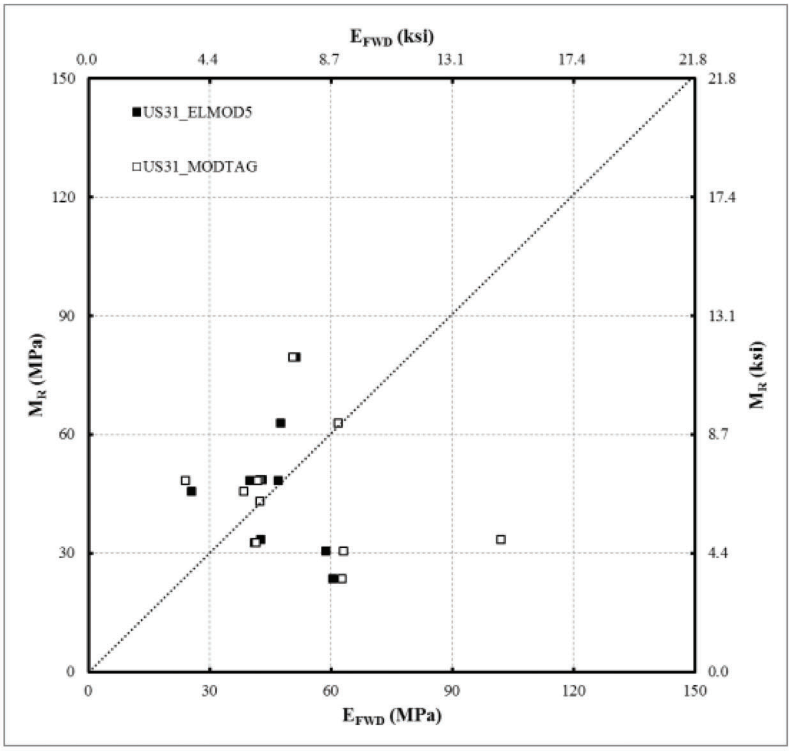
Figure 6.1 MODTAG vs. ELMOD5. Comparison between results from M_R and FWD tests. (Figure continued next page.)



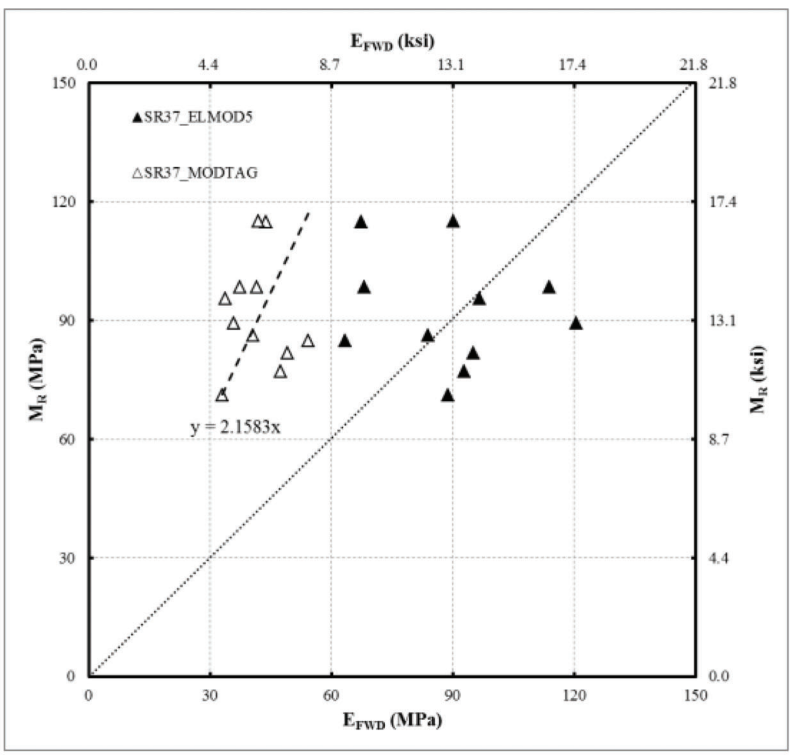
(c) Analysis of US 31 and SR 37 using MODTAG with a four-layer pavement for US 31 and five-layer for SR 37

Figure 6.1 (Continued)

results from ELMOD in Figure 6.3. The figure shows that the *FWD* modulus obtained from MODTAG is 3.7 times larger than from ELMOD and that the *FWD* modulus from MODTAG is 5.7 times higher than the M_R obtained in the laboratory. The discrepancies are thought to be associated with inaccurate pavement information such as pavement condition and/or layer thicknesses.



(a) US 31



(b) SR 37

Figure 6.2 Comparison between results from MODTAG and ELMOD5.

TABLE 6.1
Pavement layer information from ProjectWise for SR 60

Road	Layer	Material	Thickness (inches)
SR 60	Surface	HMA	3.5
	Base	HMA	6.5
	Subgrade	Type IB	14

TABLE 6.2
Pavement layer information from ProjectWise for SR 46

Road	Layer	Material	Thickness (inches)
SR 46	Surface	HMA	1.5
	Interface	HMA	3
	Base	HMA	5.5
	Subgrade	Type IB	14

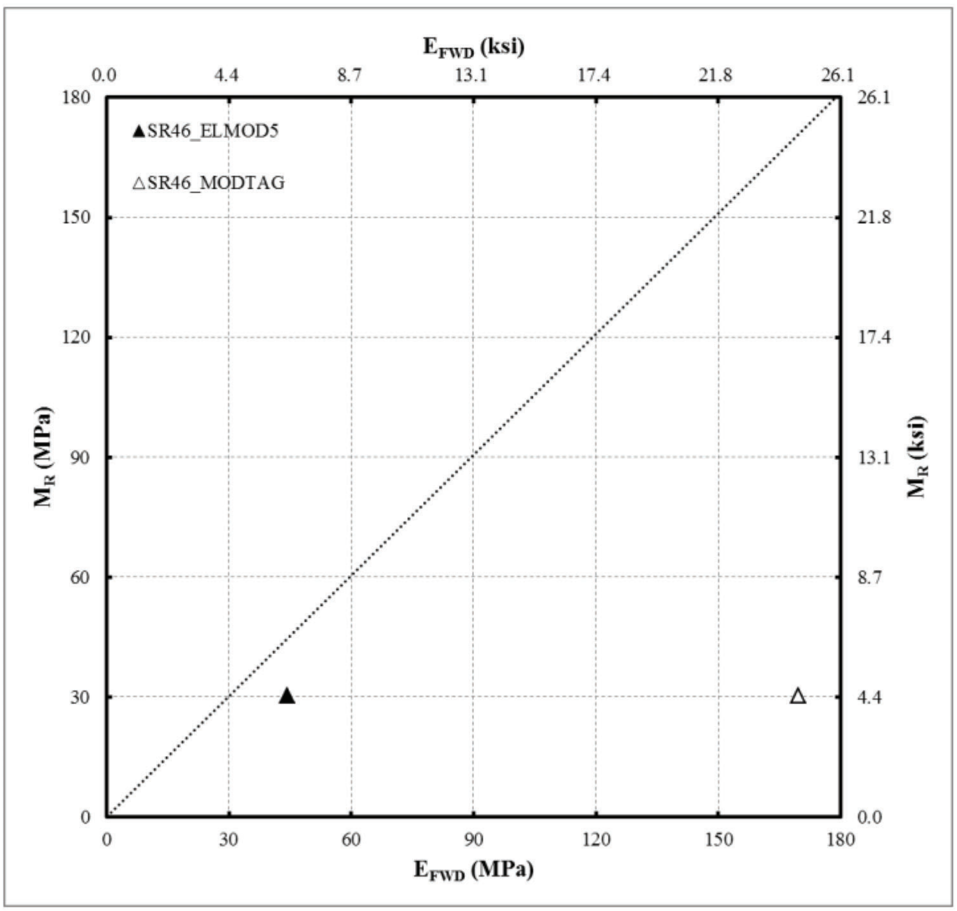


Figure 6.3 MODTAG vs. ELMOD5. Comparison between results from MODTAG and ELMOD5 for SR 46.

7. CONCLUSIONS AND IMPLEMENTATION

Since INDOT adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) at the beginning of 2009, obtaining accurate and representative values of the resilient modulus needed for the design has proven to be difficult. This is particularly the case when designing the reconstruction of the pavement of existing roads. The reason is the need to direct sampling of the subgrade soil and access to the equipment required to perform the resilient modulus tests in the laboratory following the standard AASHTO T-307-99 (2007).

The problem is compounded by the length of the project, as it requires a large number of representative soil samples. An alternative, which would be efficient and cost effective, is to obtain the resilient modulus from indirect, non-destructive tests. The goal of the project is to assess the potential of the following tests to estimate the resilient modulus of the subgrade: Falling Weight Deflectometer, *FWD*, Light Weight Deflectometer, *LWD*, and Dynamic Cone Penetrometer, *DCP*.

The following types of subgrade were specifically targeted for the project: untreated soils type A-6 and A-7-6. This objective has proven challenging; first, because these soils are usually chemically treated to improve their stability and engineering properties and so it has not been easy to identify the right project; and second, because the actual type of soil placed in-situ may not fit into these categories. In addition, coordination with the job contractor, subcontractor, and technical personnel from INDOT and others to access the site and perform all the tests at the same time has been challenging. In other occasions, the weather or equipment availability or equipment trouble have delayed the work. Fortunately, four sites had been available for testing, thanks to the work and help of INDOT personnel. The first site was on US 31 around Kokomo, Indiana. The soil is classified as A-4, according to AASHTO, with 58% passing No. 200 sieve and PI, Plastic Index, 8.5%. The second site was on SR 37 around Paoli, Indiana. The soil is defined as A-7-6, with 88% of soil passing the No. 200 sieve and PI = 23.8%. The third site was on SR 641 at Terre Haute. The soil is classified as A-6 according to AASHTO with 89% passing #200 sieve and PI = 20.2%. The last site was Ramp line A connecting SR 641 and SR 46 at Terre Haute. The soil has 72% of passing #200, and 30.6% PI, so it is classified as A-6.

FWD, *LWD*, and *DCP* tests were performed on the four selected sites. A representative 90 m long section at each site was chosen. In each section, eleven points at 9 m intervals were identified to run the three tests. After pavement construction, *FWD* tests were conducted on US 31 and SR 37. In addition, at each of the eleven points on each site, in-situ water content, optimum moisture content, maximum dry unit weight, granulometry, Atterberg limits and resilient modulus tests were performed.

The scope of the project was expanded to further investigate relations between field *FWD* and laboratory resilient modulus tests using the data repository of INDOT to obtain additional geotechnical and pavement information. The ARC GIS program was used to visualize M_R and *FWD* data so that the two independent data sets were paired. The data collected was classified into three tier categories, based on the degree of uncertainty associated with the data, which originated mostly from difficulties in determining whether the data paired originated at the same location. Tier one data, having the highest confidence in how the data was paired, showed good agreement between *FWD* and M_R, similar to the results from the first phase of the project. Tier two and tier three data did not show a strong correlation due to the higher uncertainty associated with how the data points were paired.

Based on the results from all the field and laboratory tests, the following conclusions can be reached:

1. The subgrade modulus obtained from *FWD* tests conducted on top of the pavement compares very well with the resilient modulus of the subgrade, i.e., $M_R = E_{FWD}$.
2. Results from *FWD* tests conducted directly on top of the subgrade are not reliable, likely due to the lack of confinement of the soil.
3. The stiffness obtained from *LWD* tests performed on top of the subgrade does not compare well with the resilient modulus of the soil obtained in the laboratory. The values obtained from *LWD* are too low.
4. There is not a good relation between the soil stiffness obtained from *DCP* and from the laboratory using the correlation by Salgado and Yoon (2003), which was deemed appropriate in this study.
5. While *LWD* and *DCP* have not provided acceptable estimates of soil stiffness, they can be used to estimate quality consistency of the subgrade. The research has shown that the field measurements using either method are sensitive to the quality of the construction and can be used to identify those areas with lower quality than others.
6. When good quality *FWD* data is obtained, its results, in terms of stiffness of the subgrade, can be used to estimate the resilient modulus, M_R, of the subgrade. If the data were collected using good quality assurance and quality control (QA/QC), then the backcalculation results would be able to be used to determine the modulus for design.
7. Good quality *FWD* data requires a strong complete *FWD* testing protocol. The LTPP protocol is recommended for research level work, with a small modification for production level testing. There are also other very good protocols available that INDOT could explore for their use.
8. Good quality *FWD* data can only be achieved when pavement layer thickness is accurate. Using construction plans and overall specifications may not be sufficient. If good as-built drawings showing thickness are available, then they may be used. If not, ground penetrating radar (GPR) or other non-destructive tests may be performed in conjunction with the *FWD* tests to determine the geometry of the pavement.
9. The correlations proposed between *FWD* and M_R are based on limited, yet highly reliable, data from the tests on top of pavement. It would be desirable to extend the database used in the project to further confirm such an important conclusion. This could be done by identifying sites under construction where a test campaign similar to that completed under phase one of the project could be conducted.

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APPENDIX

A.1. INTRODUCTION AND PROBLEM STATEMENT

1.1 Introduction

Since the AASHTO (1986) *Guide for Design of Pavement Structures* recommended highway agencies to use a resilient modulus (M_R) obtained from a repeated triaxial test for the design of subgrades, many researchers have made a significant effort to obtain more accurate, straightforward, and reasonable M_R values that are representative of the field conditions.

INDOT adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) beginning on January 1, 2009. This is a new design guide based on the FHWA Long Term Pavement Performance (LTPP) field study for more than 20 years. The new pavement design procedures and pavement design input parameters included in the guide differ from those in the AASHTO (1993) Pavement Design Guide, which was based on the AASHTO Road Test conducted in the 1950s. The new input parameters are based on a more realistic method that determines how the designed pavement will perform, month-by-month, during its design life. As a result, the new pavement design guide requires pavement design input parameters that are more accurate, representing actual conditions in the field, and other tests input parameters that are sensitive to the performance of the pavement during its design life.

This new pavement design procedure requires specific input parameters that will determine the outcomes of the designs; that is, it will judge whether performance criteria set by INDOT "pass" or "fail." There are a few parameters in the design procedure that are highly sensitive to the performance prediction of pavement during the design process. One such parameter is the soil resilient modulus. In the new pavement design, there is no issue related to the resilient modulus because soil information can be easily obtained from the soil exploration report. In addition, soil tests of a new pavement or pavement reconstruction project that have been conducted in a laboratory offer more appropriate results to be used as input parameters for a pavement design. However, for a pavement rehabilitation project, soil information is rather difficult to obtain. Soil coring is needed to determine the soil laboratory testing parameters. In addition, the number of pavement rehabilitation projects has outpaced the ability of INDOT to provide soil information.

In most pavement rehabilitation projects, the project length is typically large. It is extremely expensive to explore the complete soil subgrade strength characteristics along the mainline pavement. Another disadvantage of field soil sampling is the fact that, in a pavement rehabilitation project, the subgrade soil has been "naturally" compacted by passing traffic for a long time. Therefore, soil sampling (disturbed samples) from the field that is remolded in the lab does not offer a true representation of the strength characteristics of the soil.

The soil strength characteristics, such as the resilient modulus, may be measured using non-destructive testing equipment. Equipment such as the Light Weight Deflectometer (*LWD*), or Falling Weight Deflectometer (*FWD*) may be utilized to determine the in-situ resilient modulus for pavement design. The resilient modulus values from this non-destructive testing may be used as an input parameter in the MEPDG for designing pavement rehabilitation projects. MEPDG users could potentially select whether to input the soil resilient modulus from the lab or directly from the results of the *LWD* or *FWD*. Another major advantage of such equipment is that it can test hundreds of locations that cover the whole pavement rehabilitation project in a short period of time.

1.2 Problem Statement

The resilient modulus is thought to represent realistically the behavior of the soil when subjected to repeated traffic loadings, and so it is one of the most critical parameters for the design of the subgrade.

Since the AASHTO (1986) *Guide for Design of Pavement Structures* recommended the use of M_R for pavement design, several research projects on the resilient modulus of subgrade soils have been completed under the Joint Transportation Research Program (JTRP), such as Lee, Bohra, Altschaeffl, and White (1993), FHWA/INDOT/JTRP-92/23, and Kim and Siddiki (2006). From those JTRP projects, Lee et al. (1993) suggested correlations between the resilient modulus and the unconfined compressive strength of Indiana soils. Kim and Siddiki (2006) suggested predictive models of the resilient modulus based on soil properties and unconfined compression tests. The models require twelve different soil parameters, namely the optimum moisture content, natural moisture content, MCR (Moisture Content Ratio = Moisture Content / Optimum Moisture Content), MDD (Maximum Dry Density), DD (Dry Density), SATU (Degree of saturation), %Compaction, %Sand (percentage of sand in the soil obtained from the Particle size distribution curve), %Silt (percentage of silt in the soil, from the Particle size distribution curve), %Clay (percentage of clay, also from the Particle size distribution curve), liquid limit, and plasticity index. It has to be realized, that significant time and effort is required to obtain all the parameters. Therefore, there is a need to have models with a minimum number of parameters that can estimate the resilient modulus.

The resilient modulus test requires a specialized and expensive equipment, and the test itself needs a lot of time and effort. These limitations have led researches to develop alternative methods to estimate the resilient modulus using non-destructive tests such as *FWD*, *LWD*, and *DCP*. The tests are fast and easy and, as a result, are widely used for compaction control in the US. In Indiana, *DCP* and *LWD* are recommended for compaction control of subgrade soils and aggregates.

However, limited work has been conducted to relate M_R and *DCP* for Indiana soils (for example, Salgado & Yoon, 2003). Clearly, robust and credible correlations between M_R *and* FWD, LWD, *and* DCP *are needed for Indiana soils.*

A.2. RESEARCH OBJECTIVES

The objectives of this research project are geared toward determining a practical solution for pavement design procedures to effectively determine the soil resilient modulus for rehabilitation projects. The ultimate goal of the research is to create guidelines for selecting values of soil subgrade stiffness, targeting specifically untreated subgrade soils type A-6 and A-7-6. The following are the milestones to achieving this end goal:

1. Review the state of stress-strain in laboratory resilient modulus testing and field *FWD*, *LWD*, and *DCP* tests.
2. Review conversion factors/models between *FWD*, *LWD*, and *DCP* and laboratory resilient modulus.
3. Provide recommendations for obtaining the subgrade resilient modulus for pavement rehabilitation projects.

A.3. FIELD TESTS ON INDIANA SOILS

3.1 Introduction

A key parameter for a pavement foundation is the resilient modulus (M_R). The resilient modulus can be determined directly through a laboratory test. However, the test is sophisticated and requires significant time and resources such as testing equipment and a number of representative soil samples collected from the field. Considering time and cost, it is desirable to develop approximate methods to estimate M_R using field tests such as *FWD*, *LWD*, and *DCP*. A number of correlations already exist between the tests (Abu-Farakh, Nazzal, Alshibli, & Seyman, 2005; Fleming, Frost, & Rogers, 2000; Nazzal, Abu-Farsakh, Alshibli, & Mohammad, 2007; Siekmeier et al., 2009; Yoon et al., 2003), but it is not clear how credible or accurate the estimates obtained from those correlations are, and in particular for those soils found in Indiana.

In this chapter, the results from the field tests conducted at four sites in Indiana are presented and discussed.

3.2 Site selection

The site location was selected based on following criteria: types of soils, compacted subgrade with natural soils that are not chemically modified, and testing availability. This project targets fine-grained soils such as A-6 and A-7-6 type soils, as well as untreated subgrade. Available site area based on schedules of projects is required to fit the designed testing plan (details described in Appendix Section 3.3); the site needs 90 m (300 ft) long for 11 stations of the field testing, and demands flat subgrade to obtain the results of FWD and LWD. In addition, availability of FWD testing equipment and weather condition are considered for site selection.

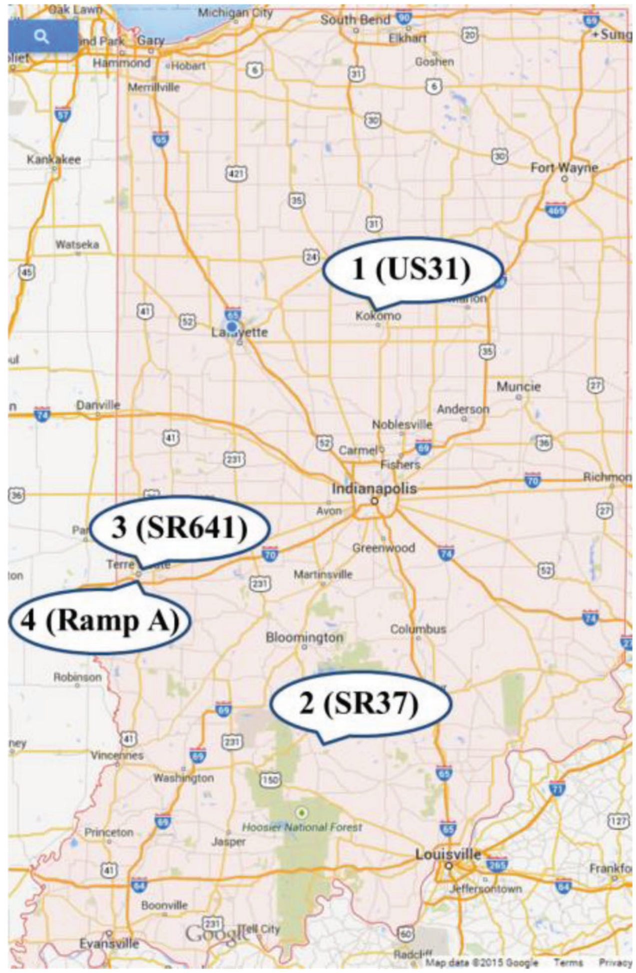


Figure A.3.1 Location of the four test sites.

The location of the four field testing sites selected is shown in Figure A.3.1. The projects chosen were: (1) US 31 in Kokomo, (2) SR 37 in Paoli, (3) SR 641 in Terre Haute, (4) Ramp line A connecting SR 641 and SR 46 in Terre Haute. At each of the four sites, the soil properties were determined using the laboratory tests described in Appendix Chapter 4. Based on the laboratory test results (summarized in Table A.4.5), the soil samples collected at each site were classified according to AASHTO M 145-91 (2012). The soil samples from US 31 were classified as A-4, from SR 641 and Ramp A as A-6, and the soil samples from SR 37 were classified as A-7-6.

3.3 Test Methods

FWD, *LWD*, and *DCP* tests were performed to evaluate the stiffness of in-situ soils directly. A 90 m (300 ft) length section at each site was selected and 11 test points were marked at 9 m (30 ft) intervals where the tests were performed (see Figure A.3.2). All of the eleven points are labeled with numbers 1 to 11, which will be used to identify the soil samples at a site, e.g. US31_1.

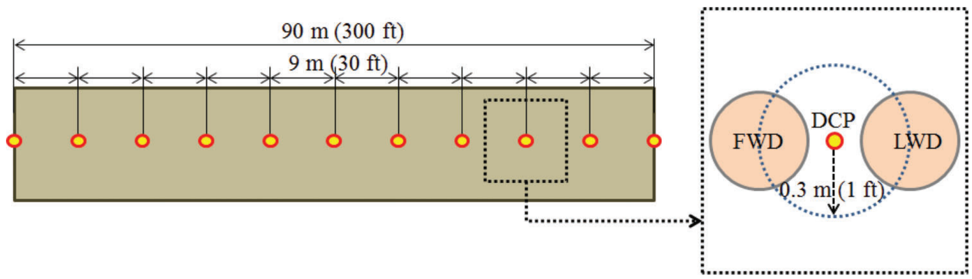


Figure A.3.2 Schematic of selected section and a test location.

All the field tests were conducted at the same location to reduce material variability and obtain meaningful comparisons between the resilient moduli estimated from the tests and that obtained in the laboratory. At a given location, all of the three tests were done adjacent to each other to avoid site disturbance; all of the tests were done within a radius of 0.3 m (1 ft). In addition, sand cone tests and nuclear gauge tests were performed to measure unit weight and water content of the in-situ soils.

*3.3.1 Falling Weight Deflectometer (*FWD*) Test*

The Falling Weight Deflectometer (*FWD*) is one of the most commonly used tools for non-destructive evaluation of pavement layers. Typically, *FWD* is comprised of a loading system, several sensors, and a towing equipment.

Eleven *FWD* tests were performed at 9 m intervals at each site. The Dynast 8000 *FWD* Test System was used for *FWD* tests. The radius of the loading plate was 15 cm, and nine sensors aligned with the center of the loading plate measured the deflection of the soil. The forces generated by the *FWD* were 25 to 40 kN (2500 to 9000 lbs).

There has been a difficulty to find a correction factor between the laboratory resilient modulus and the back-calculated modulus from *FWD*. The correction factor is introduced to adjust the difference between laboratory M_R and the back-calculated stiffness from *FWD*. Previous studies have shown a variety of the ratio of M_R/E_{FWD}; the ratio of M_R/E_{FWD} can be regarded as a correction factor for *FWD*, because a correction factor "C" for *FWD* can be expressed in a generalized form as:

$$C = \frac{M_R}{E_{FWD}}$$

It has been reported that the ratio of M_R/E_{FWD} varied from 0.1 to 3.5 (Von Quintus & Killingsworth, 1998) based on the LTPP program database. Ping, Yang, and Gao (2002) reported the ratio is 0.6, and Rahim and George (2003) reported the ratio of M_R/E_{FWD} ranged from 0.82 to 2.0 with an average value of 1.4. Dawson et al. (2009) showed the ratio of M_R/E_{FWD} is 1.0.

*3.3.2 Light Weight Deflectometer (*LWD*) Test*

The Light Weight Deflectometer (*LWD*) test is designed to estimate M_R of the in-situ soils. *LWD* is also known as Portable Falling Weight Deflectometer (*PFWD*) or Light Falling Weight Deflectometer (*LFWD*). *LWD* consists of a 10 kg loading device and measuring sensor (accelerometer or geophone) attached to a loading plate. With the deflection from the *LWD* test, E_{LWD} is calculated using the Boussinesq's elastic equation.

$$E_{LWD} = \frac{A\sigma(1-\nu^2)r}{\delta} = \frac{AF_{peak}(1-\nu^2)}{\pi\delta r}$$

where, σ is the applied stress; ν is the Poisson's ratio; F_{peak} is the applied peak force; δ is the peak deflection (mm); and A is the contact stress distribution parameter (π/2 for rigid plate and 2 for flexible plate).

Eleven *LWD* tests were performed at 9 m intervals at each site to obtain the in-situ M_R of the subgrade, in accordance to ASTM E2583-07 (2011). The Zorn Instruments *ZFG* 2000 was used for the *LWD* tests. The device has a 10 kg hammer and 30 cm diameter loading plate and generates 7.07 kN of peak force.

Fleming et al. (2000) and Nazzal et al. (2007) reported that the *LWD* correlates well with the *FWD*.

$E_{FWD} = 1.031\ E_{LWD}$ (Fleming et al., 2000)

$E_{FWD} = 0.964\ E_{LWD}$ (Nazzal et al., 2007)

*3.3.3 Dynamic Cone Penetration (*DCP*) Test*

The Dynamic Cone Penetration (*DCP*) test provides the penetration resistance of in-situ materials. The *DCP* is composed of a 8 kg drop hammer with 575 mm of drop height and a 16 mm diameter steel drive rod, with either a replaceable point tip or a disposable cone tip. Results of the *DCP* test are expressed, as a function of depth, in the form of penetration per blow (mm/blow), which is the Dynamic Cone Penetration Index (*DCPI*).

Eleven *DCP* tests were performed at 9 m intervals at each site to estimate the M_R of the in-situ subgrade materials. The test procedure followed ASTM D6951-03 (2015). *DCP* blows were recorded for a 300 mm penetration into the soil, and the *DCPI* was obtained. The relationship provided by Salgado and Yoon (2003) between E_{DCP} and *DCPI* is adopted for this study, since the relationship was obtained from Indiana soils.

$E_{DCP} = -3279\ DCPI + 114100$ (kPa)

Many correlations between the M_R and *DCP* index have been developed (George & Uddin, 2000; Hassan, 1996; Herath, 2005; Mohammad, 2007).

E_{DCP} (psi) = 7013 – 2040 ln($DCPI$) Hassan et al. (1996)

E_{DCP} (MPa) = 532.1 ($DCPI$)-0.492 George and Uddin (2000)

E_{DCP} (MPa) = 16.28 + 928.24/($DCPI$) Herath et al. (2005)

3.3.4 Sand Cone Test

The sand cone test determines the density and unit weight of in-situ soils using a sand cone apparatus. The sand cone density apparatus consists of a sand container, sand cone and base plate. Typically, standard Ottawa sand is used to calculate the volume of the hole.

The test was used on SR 37 site, where five sand cone tests were completed to measure the dry unit weight and water content of the in-situ soils, in accordance with ASTM D1556-07 (2007).

3.3.5 Nuclear Gauge Test

The nuclear gauge test measures density and moisture of in-situ soils using nuclear equipment, with low-level radiation. The density of the material can be measured by direct transmission, backscatter, or backscatter/air-gap ratio methods. The water content can be determined by backscatter mode irrespective of the mode being used for density.

Eleven nuclear gauge tests were performed at 9 m intervals on US 31, SR 641 and Ramp A site to estimate the dry unit weight and water content of the in-situ soils. The procedure followed ASTM D6938-10 (2010).

3.4 Field Tests on Fine-Grained Materials (A-4, A-6, and A-7-6) and Discussion

A total of four sites are selected; US 31 site for A-4 soils, SR 641 and Ramp line A sites for A-6 soils, and SR 37 site for A-7-6 soils.

3.4.1 Site on US 31

The site was selected at US 31 in Kokomo, Howard County, Indiana. See Figure A.3.3. The subgrade soils were silty soils with 58% passing the No. 200 sieve. The Plasticity Index (PI) of the soil was 8.6% with 18.9% Liquid Limit (LL) and 10.2% Plastic Limit (PL). According to AASHTO classification, the subgrade materials at this site were classified as A-4, based on the soil grain distribution and the soil index properties. The dry unit weight and the water content of the in-situ soils were 20.6 kN/m^3 and 7.4%, respectively, measured by nuclear gauge. The maximum dry unit weight was 20.6 kN/m^3 and the optimum water content was around 9.3% (see Figure A.4.2), based on the Proctor test (ASTM D698-12, 2012). In-situ water contents at the US 31 site and optimum water content obtained from Proctor tests are shown in the Table A.3.1. The water contents at the site were a little lower than the optimum water content (9.3%).

Figure A.3.3 Photograph of the US 31 site.

On August 23, 2013, field tests, FWD, LWD, DCP and nuclear tests were conducted on the subgrade of the US 31 site. Results from the FWD tests are shown in Table A.3.2, which includes the applied load, the deflection measured and the estimated modulus. The modulus from the FWD test (E_{FWD}) is computed using the software ELMOD 5. The values of E_{FWD} vary from 36 MPa to 134 MPa.

The deflection and estimated modulus from the LWD tests on subgrades soils are listed in Table A.3.3. E_{LWD} is calculated using the Boussinesq's equation. The values of E_{LWD} range from 37 to 90 MPa.

The results of the DCP tests are listed in Table A.3.4. The values of the $DCPI$ obtained range from 4.6 mm/blow to 7.9 mm/blow.

Figure A.3.4 shows the estimated moduli from FWD, LWD, and DCP, and includes in-situ water content and optimum water content. Some of similarities and differences are observed in the figure. E_{FWD} computed by ELMOD 5 is highly variable, while E_{LWD}s and E_{DCP}s are relatively consistent. Also, the stiffness obtained from DCP is consistently higher than that obtained from LWD.

On May 13, 2014, FWD tests were conducted at the same location, but on top of the pavement, and after the site was open to public. The results are listed in Table A.3.5. The FWD modulus of the subgrade computed from tests done on top of the subgrade (Table A.3.2) and computed from tests on top of the pavement (Table A.3.5) are plotted in Figure A.3.5. The figure shows a reduction of the subgrade modulus when using the results of the test on top of the pavement and uniform results. We hypothesize that this is due to two issues: one is the increased in confinement to the soil provided by the pavement, which provides more consistent results; the other is due to the loading of the subgrade due to traffic and the possible changes of moisture content of the subgrade while in service.

3.4.2 Site on SR 37

The site was located on SR 37 in Mitchell, Lawrence County, Indiana, and consisted of a road-widening project (see Figure A.3.6).

TABLE A.3.1
In-situ water content and optimum water content at US 31

	1	2	3	4	5	6	7	8	9	10	11
$w_{in\text{-}situ}$ (%)	7.9	8.4	7.4	7.1	6.5	7.4	7.1	7.2	6.7	7.5	7.8
w_{opt} (%)	9.5	9.2	9.1	9.4	9.1	9.3	9.6	9.4	9.1	9.4	9.4

TABLE A.3.2
Summary of *FWD* tests on the subgrade at the US 31 site

	1	2	3	4	5	6	7	8	9	10	11
Load applied (kN)	38.33	37.46	34.44	34.59	32.30	30.16	30.31	30.60	36.78	38.14	37.41
Deflection (mm)	1.46	1.17	1.69	1.94	2.34	3.25	3.03	3.25	1.65	1.22	1.51
Modulus (MPa)	106.90	134.33	87.81	64.20	55.96	36.31	41.15	39.42	93.82	132.76	104.66

TABLE A.3.3
Summary of *LWD* tests on the subgrade at the US 31 site

	1	2	3	4	5	6	7	8	9	10	11
Deflection (mm)	0.518	0.446	0.378	0.441	0.480	0.608	0.350	0.376	0.311	0.248	0.293
Modulus (MPa)	43.42	50.42	59.54	51.03	46.92	37.03	64.24	59.80	72.36	90.62	76.72

TABLE A.3.4
Blows of *DCP* tests and the *DCPI* on the subgrade at the US 31 site

	1	2	3	4	5	6	7	8	9	10	11
No. of blows (300 mm penetration)	51	53	53	48	38	42	48	65	56	56	57
***DCPI* (mm/blow)**	5.88	5.66	5.66	6.25	7.89	7.14	6.25	4.62	5.36	5.36	5.26

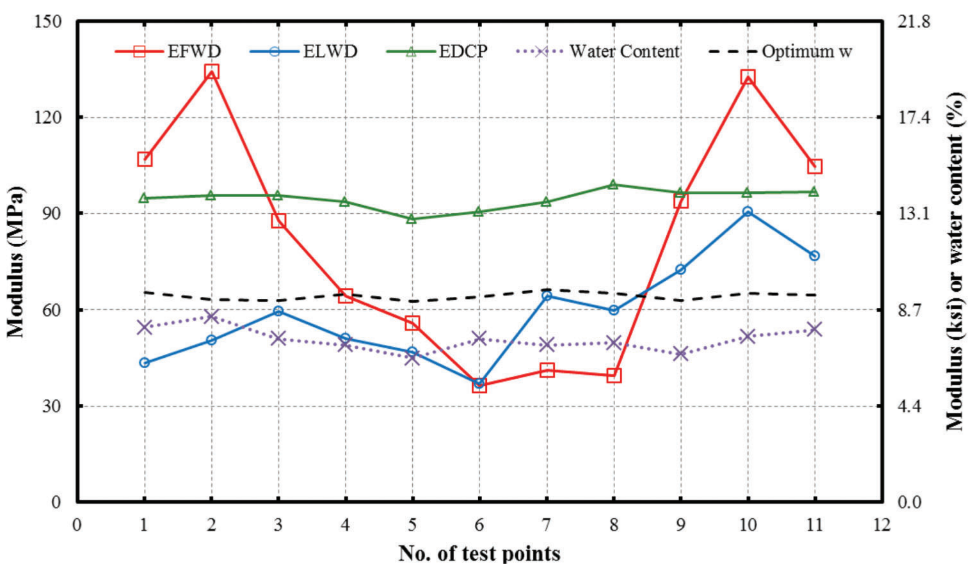


Figure A.3.4 E_{FWD}, E_{LWD}, E_{DCP} and water content on US 31.

The subgrade soils were clayey soils with 88% passing the No. 200 sieve. The Plasticity Index (*PI*) of the subgrade soil was 23.7% with 41.3% Liquid Limit (*LL*) and 17.6% Plastic Limit (*PL*). According to AASHTO, the subgrade materials were classified as A-7-6. The dry unit weight and the water content of the in-situ soils were 16.4 kN/m^3 and 19.2%, respectively, measured with the sand cone test. The maximum dry unit weight was 17.2 kN/m^3 and the optimum water content was around 17.8%, based on the Proctor test (ASTM D698-12, 2012).

TABLE A.3.5
Summary of *FWD* tests on pavement structure of US 31 site

	1	2	3	4	5	6	7	8	9	10	11
Modulus (MPa)	43.07	51.32	41.02	42.64	223.19	25.61	47.04	60.47	58.86	39.98	47.58

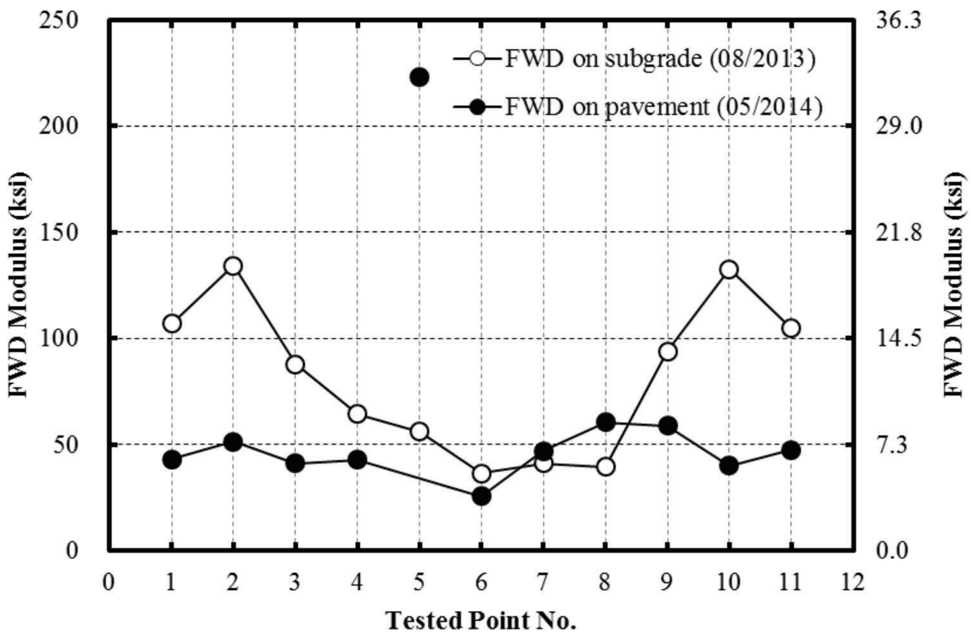


Figure A.3.5 Comparison of E_{FWD} of the subgrade, from *FWD* tests on top of the subgrade and on top of the pavement, US 31.

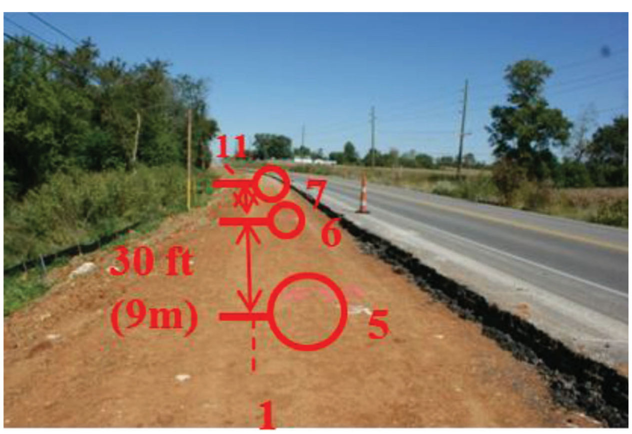


Figure A.3.6 View of SR 37 site.

On September 23, 2013, field tests including *FWD*, *LWD*, *DCP* and sand cone tests were conducted on the subgrade of the SR 37 site.

The applied load, deflection, and the estimated modulus obtained from the *FWD* tests are listed in Table A.3.6. The values of E_{FWD} vary from 31 MPa to 111 MPa. The *LWD* test results are summarized in Table A.3.7. E_{LWD}s are uniform and range from 15 MPa to 31 MPa. The results of the *DCP* tests are listed in Table A.3.8, with *DCPI* values ranging from 15 mm/blow to 27 mm/blow. In-situ water content and optimum water content are summarized in Table A.3.9. The water content was close to the optimum, except at No. 5 location.

Figure A.3.7 plots the estimated moduli from *FWD*, *LWD*, and *DCP* tests, in-situ water content and optimum water content of the subgrade. The soil is classified as A-7-6 according to AASHTO. The values of E_{FWD} have a significant variation, while E_{LWD} and E_{DCP} results are relatively uniform, which is similar to what was observed at US 31 (Figure A.3.4). Also, as with the previous site, the moduli obtained from DCP is larger than from LWD.

FWD tests were conducted on top of the pavement in the spring of 2005 (05/26/2015) and summer (08/04/2015) at exactly the same locations. Additional tests were performed on points close to the rail to investigate the effect of the lateral confinement on the results. The subgrade moduli, from the *FWD* measurements, are listed in Table A.3.10 and are displayed in Figure A.3.8, together with the values obtained from the tests performed on top of the subgrade. Consistent with the findings at the US 31 site (Figure A.3.5), testing on the pavement results is more uniform results, which supports the notion of the effects of confinement on the soil resulting in increased consistency of the results, and changes in modulus values associated with the traffic (loading) history of the site, as well as changes in moisture content of the subgrade. For example, as average,

TABLE A.3.6
Summary of *FWD* tests on subgrade of SR 37 site

	1	2	3	4	5	6	7	8	9	10	11
Load applied (kN)	23.79	29.58	29.43	27.73	32.01	31.18	34.88	33.28	34.20	27.53	28.75
Deflection (mm)	3.26	3.25	2.57	2.19	1.87	1.82	1.20	1.56	1.40	2.55	2.51
Modulus (MPa)	31.25	38.81	39.70	54.00	75.84	82.29	134.89	98.79	110.98	41.74	39.94

TABLE A.3.7
Summary of *LWD* tests on subgrade of SR 37 site

	1	2	3	4	5	6	7	8	9	10	11
Deflection (mm)	1.370	1.037	1.160	0.890	1.290	1.093	0.717	1.027	1.047	1.443	1.027
Modulus (MPa)	16.43	21.71	19.40	25.29	17.45	20.58	31.40	21.92	21.50	15.59	21.92

TABLE A.3.8
Blows of *DCP* tests and the *DCPI* on subgrade of SR 37 site

	1	2	3	4	5	6	7	8	9	10	11
No. of blows (300 mm penetration)	19	20	15	12	13	16	13	11	16	15	13
***DCPI* (mm/Blow)**	15.8	15.0	20.0	25.0	23.1	18.8	23.1	27.3	18.8	20.0	23.1

TABLE A.3.9
In-situ water contents and the optimum water contents on SR 37

	1	2	3	4	5	6	7	8	9	10	11
win-situ (%)	16.8	16.5	18.5	18.6	31.3	18.0	17.8	20.6	15.9	16.7	17.3
wopt (%)	15.9	22.7	19.0	18.3	16.2	15.3	16.8	17.6	17.7	18.0	17.8

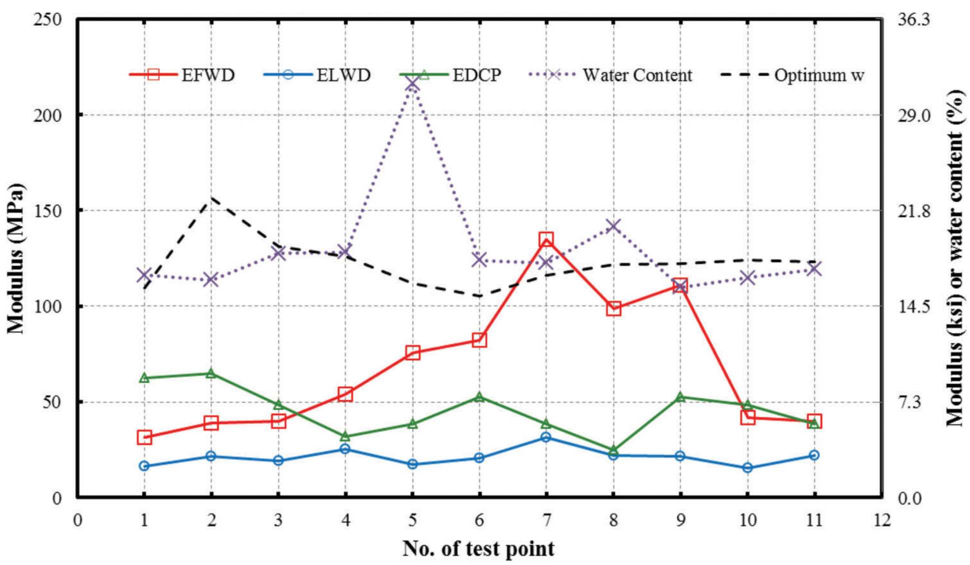


Figure A.3.7 E_{FWD}, E_{LWD}, E_{DCP} and water content of the subgrade at the SR 37 site.

the moduli obtained from the tests in August are larger than those obtained in May, which is thought to be associated with a smaller water content of the subgrade in the summer than in the spring. Also, it can be noticed, that the differences between the results obtained at the center of the site and at the edge, close to the railing, are not very different, arguably within soil variability, which seems to indicate that there is no substantial difference in confinement between the two locations.

TABLE A.3.10
Summary of *FWD* tests performed on top of the pavement at the SR 37 site

	1	2	3	4	5	6	7	8	9	10	11
Modulus (MPa) (05/2015)	93.39	127.45	122.97	107.57	58.74	60.87	55.10	52.39	46.59	79.18	81.98
Modulus (MPa) (08/2015)	88.83	120.50	113.86	83.87	67.38	94.98	68.13	90.04	63.24	92.71	96.47
Modulus close to rails (MPa) (08/2015)	102.34	119.67	119.36	80.19	76.43	108.33	75.07	62.94	48.70	110.04	88.30

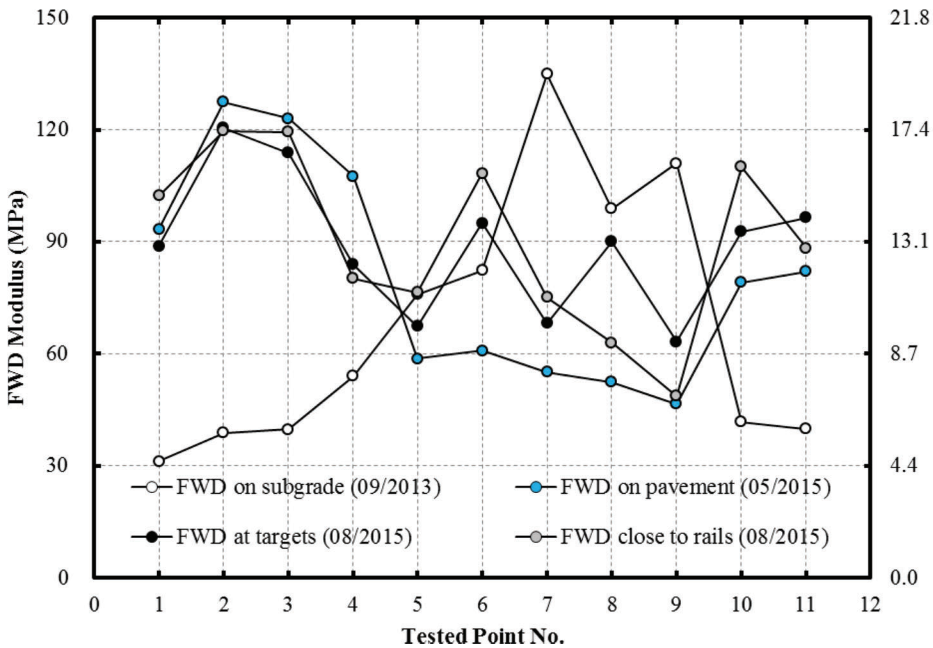


Figure A.3.8 Comparison of E_{FWD} of the subgrade from *FWD* tests performed on top of the subgrade and on top of the pavement. SR 37.

3.4.3 Site on SR 641

The site was located on SR 641 in Terre Haute, Vigo County, Indiana. Figure A.3.9 is a photograph of the site where the tests were performed. The subgrade soils were clayey soils with 88% passing the No. 200 sieve. The Plasticity Index (*PI*) was 18.4%, with 34.8% Liquid Limit (*LL*) and 16.4% Plastic Limit (*PL*). According to AASHTO classification, the subgrade materials were classified as A-6, given the soil size distribution and index properties. The dry unit weight and the water content of the soils were 17.6 kN/m^3 and 19.6%, respectively, measured with the nuclear gauge. The maximum dry unit weight was 17.9 kN/m^3 and the optimum water content was around 15.1%, both obtained from the Proctor test (ASTM D698-12, 2012).

The results of the *FWD* tests, conducted on September 18, 2014, are listed in Table A.3.11. ELMOD 5 was used to calculate the *FWD* modulus (E_{FWD}). The E_{FWD} values range from 8 MPa to 15 MPa and are lower than at the previous two sites (US 31 and SR 37). One of the reasons for the lower values is the higher water content of the site, which at most locations was about 4% higher than the optimum (Table A.3.12). The results of *LWD* tests are listed in Table A.3.13.

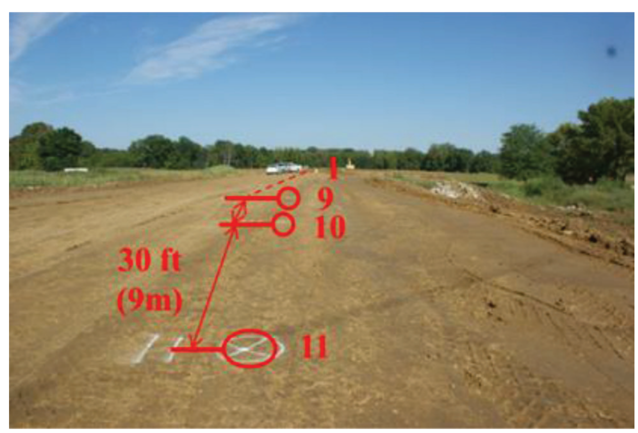


Figure A.3.9 View of the SR 641 site.

The *LWD* modulus (E_{LWD}) was calculated using the Boussinesq's equation. The E_{LWD} was quite uniform, with values in the range of 5 MPa to 28 MPa. The results of *DCP* tests are listed in Table A.3.14. The values are in a range of 27 mm/blow to 60 mm/blow. This range is also higher than at the other two sites (US 31 and SR 37), which may be caused, as explained, by the high water content.

TABLE A.3.11
Summary of *FWD* tests on subgrade of SR 641 site

	1	2	3	4	5	6	7	8	9	10	11
Load applied (kN)	8.27	6.03	7.39	7.10	8.71	7.54	12.06	9.15	8.46	8.51	6.42
Deflection (mm)	2.31	2.19	2.22	2.20	2.37	2.24	2.31	2.35	2.26	2.27	2.23
Modulus (MPa)	13.67	7.99	9.97	10.82	10.99	10.40	15.76	11.77	10.51	12.98	8.00

TABLE A.3.12
In-situ water contents and the optimum water contents on SR 641

	1	2	3	4	5	6	7	8	9	10	11
$w_{in\text{-}situ}$ (%)	18.3	19.6	15.3	19.8	19.4	18.2	23.1	21.9	20.8	17.1	22.2
w_{opt} (%)	15.6	14.6	14.3	13.6	14.9	15.0	15.6	15.9	15.6	15.2	15.7

TABLE A.3.13
Summary of *LWD* tests on subgrade of SR 641 site

	1	2	3	4	5	6	7	8	9	10	11
Deflection (mm)	4.43	3.19	3.68	3.46	2.32	3.80	1.06	1.32	0.82	2.81	4.62
Modulus (MPa)	5.08	7.05	6.11	6.50	9.70	5.93	21.16	17.09	27.56	8.01	4.87

TABLE A.3.14
Blows of *DCP* tests and the *DCPI* on subgrade of SR 641 site

	1	2	3	4	5	6	7	8	9	10	11
No. of blows (300 mm penetration)	7	8	6	6	8	8	10	11	9	6	5
***DCPI* (mm/Blow)**	42.9	37.5	50.0	50.0	37.5	37.5	30	27.3	33.3	50.0	60.0

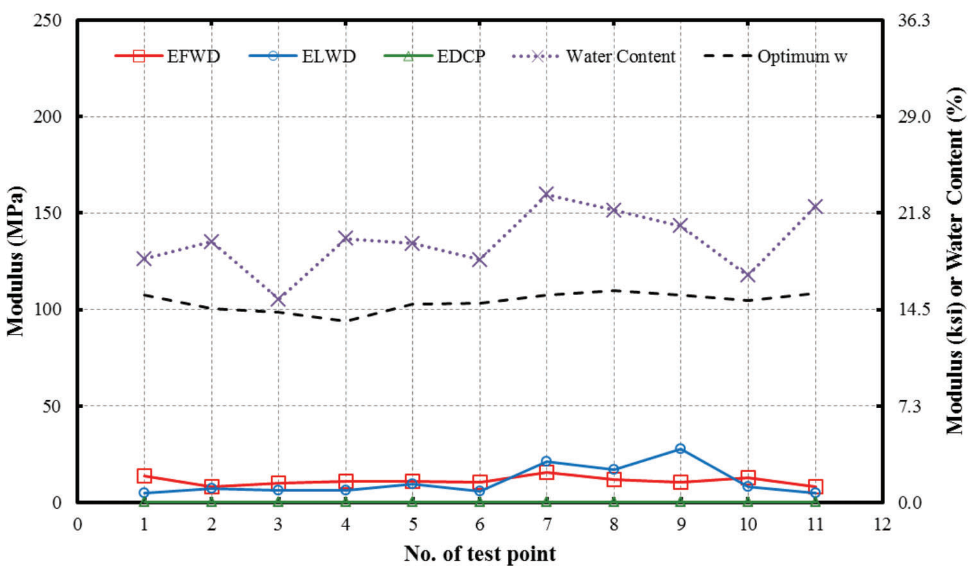


Figure A.3.10 E_{FWD}, E_{LWD}, E_{DCP} and water content at the SR 641 site.

Figure A.3.10 plots the moduli obtained from *FWD*, *LWD*, and *DCP* tests, the in-situ water content, and the optimum water contents. The soils at SR 641 are classified as A-6 according to AASHTO. Most estimated modulus values were below 20 MPa, which is too low. As mentioned, the high water content due to heavy rain before the tests may be the cause for the low values.

3.4.4 Site on Ramp A

The site was located on Ramp A in Terre Haute, Vigo County, Indiana. Figure A.3.11 provides a view of the site. The subgrade soils were clayey soils with 72% passing the No. 200 sieve. The Plasticity Index (*PI*) was 14%, the Liquid Limit (*LL*) 29% and the Plastic Limit (*PL*) 15%. The subgrade soils are classified as A-6, based on the soil grain distribution and index properties. The dry unit weight and the water content of the in-situ soils were 19.6 kN/m^3 and 11.3%, respectively, measured with the nuclear gauge. The maximum dry unit weight was 18.7 kN/m^3 and the optimum water content 12.3%, from the Proctor test (ASTM D698-12, 2012).

The field testing was conducted on June 2, 2015. The results of the *FWD* tests are listed in Table A.3.15. The modulus from the *FWD* tests is calculated using ELMOD 5. The values range from 15 MPa to 25 MPa, except for one outlier (No. 11). Deflections and estimated modulus from *LWD* tests are shown in Table A.3.16. As before, the Boussinesq's solution is used to calculate the modulus (E_{LWD}). The E_{LWD} values are fairly uniform and range from 9 MPa to 23 MPa. The *DCPI* and blow counts from *DCP* tests are listed in Table A.3.17. The values range from 13 mm/blow to 20 mm/blow. They are higher than at SR 641, although the soils at SR 641 and Ramp A are analogous, i.e. they are both A-6; this may be due to the different water

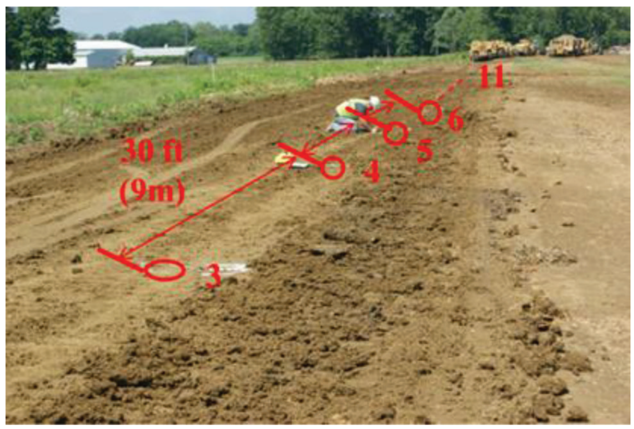


Figure A.3.11 View of Ramp A site.

TABLE A.3.15
Summary of *FWD* tests on subgrade of Ramp A site

	1	2	3	4	5	6	7	8	9	10	11
Load applied (kN)	21.70	24.86	22.77	25.10	22.48	23.98	26.61	25.49	21.36	23.79	32.55
Deflection (mm)	3.28	3.28	3.28	2.39	3.28	3.28	3.28	3.28	3.15	3.11	1.55
Modulus (MPa)	16.66	22.92	19.07	25.50	20.20	17.77	21.59	20.99	22.80	15.08	69.70

TABLE A.3.16
Summary of *LWD* tests on subgrade of Ramp A site

	1	2	3	4	5	6	7	8	9	10	11
Deflection (mm)	1.56	1.07	1.22	0.97	1.40	1.17	0.96	1.47	2.46	1.42	1.04
Modulus (MPa)	14.40	21.03	18.40	23.20	16.11	19.18	23.44	15.34	9.15	15.89	21.71

TABLE A.3.17
Blows of *DCP* tests and the *DCPI* on subgrade of Ramp A site

	1	2	3	4	5	6	7	8	9	10	11
No. of blows (300 mm penetration)	19	22	22	23	20	17	20	20	15	23	19
***DCPI* (mm/Blow)**	15.8	13.6	13.6	13.0	15.0	17.6	15.0	15.0	20.0	13.0	15.8

TABLE A.3.18
In-situ water contents and the optimum water contents on Ramp A

	1	2	3	4	5	6	7	8	9	10	11
$w_{in\text{-}situ}$ (%)	10.5	10.8	10.9	11.4	13.8	10.3	11.2	10.7	12.4	11.0	11.7
w_{opt} (%)	12.7	12.6	11.3	12.2	11.9	12.7	12.0	13.0	12.1	12.0	12.6

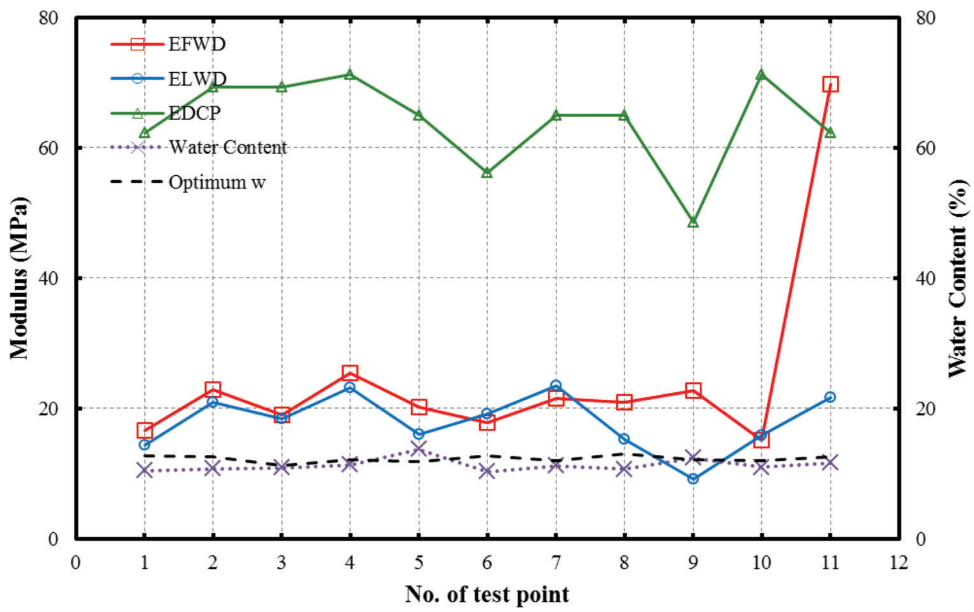


Figure A.3.12 E_{FWD}, E_{LWD}, E_{DCP} and water content of the subgrade at Ramp A.

content of the two sites, with the one in SR 641 with higher water content due to heavy rain in the days preceding testing. The in-situ water contents and the optimum water content from Proctor tests are summarized in Table A.3.18. The in-situ water content (11.3%) was similar to the optimum water content (12.3%). It needs to be mentioned that roughly the two top centimeters of the subgrade were wet; the in-situ water content, however, was obtained at roughly 30 centimeters below the surface.

Figure A.3.12 plots the stiffness moduli obtained from *FWD*, *LWD*, and *DCP* tests, in-situ water contents, and optimum water contents, at Ramp A. The modulus obtained from *FWD* and from *LWD* tests are similar, which is not the case in previous sites. Estimates from *DCP* yield higher modulus than from the other two tests, which is also the case in all other sites.

A.4. LABORATORY TESTING

Laboratory tests have been performed on the soil samples collected at the four sites (US 31, SR 37, SR 641, and Ramp A). The following tests are conducted: Soil Characterization (sieving, hydrometer, compaction and Atterberg limit tests) and resilient modulus tests.

4.1 Soil Characterization

4.1.1 Grain Size Distribution

All the collected soils were subjected to wet sieving according to the ASTM C136-14 (2015) standard test procedure. After sieving, representative samples were chosen for the hydrometer test (ASTM D422-63, 2007). Figure A.4.1 shows the grain size distribution of representative soils at each site. As results, 42% of sand, 42% of silt, and 16% of clay comprise US 31 subgrade. SR 37 subgrade consists of 11% of sand, 42% of silt, and 47% of clay. SR 641 subgrade is composed of 66% of silt, and 22% of clay. Ramp A subgrade has 28% of sand, 51% of silt, and 21% of clay. US 31 subgrade has relatively higher ratio of sand than the other three sites; therefore, percent finer at #200 is lower than the other subgrades. SR 641 subgrade has higher ratio of silt and the US 31 subgrade and SR 37 subgrade have the same percentage of silt. SR 37 subgrade includes higher ratio of clay while US 31 and SR 37 subgrades are similar.

4.1.2 Compaction Tests

Proctor tests were performed to determine the optimum moisture content at which a given soil reaches its maximum dry density, for a given compaction effort.

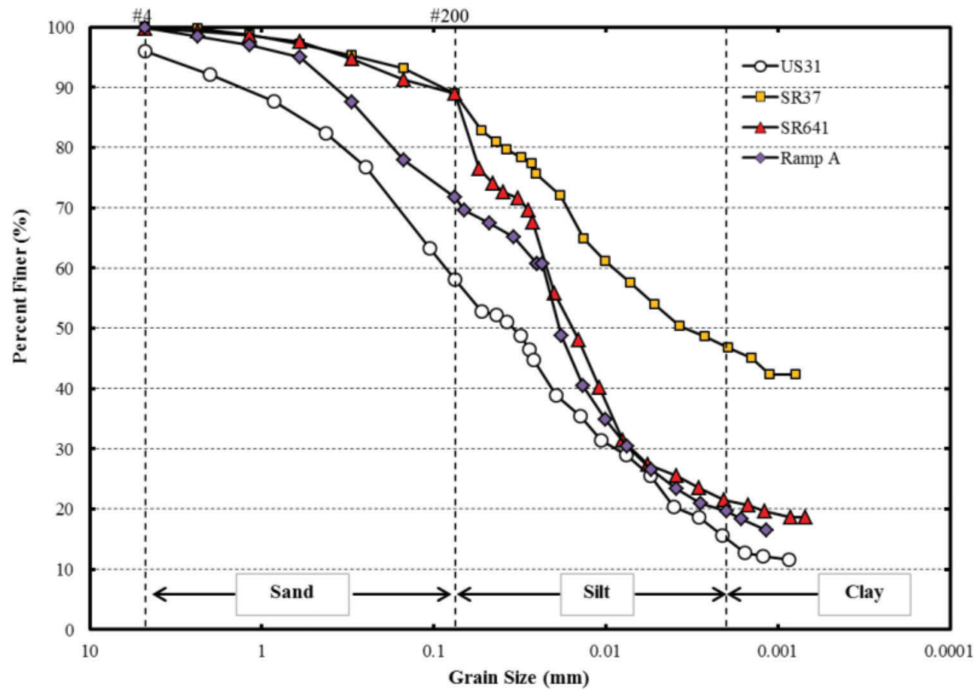


Figure A.4.1 Grain size distribution of soil samples from US 31, SR 37, SR 641, and Ramp A sites.

They were done according to ASTM D698-12 (2012) and AASHTO T 99 (2011).

The Proctor test results of the US 31 soils are plotted in Figure A.4.2. As one can see, all eleven tests are similar. The maximum dry unit weight was 20.6 kN/m^3 (131.3 lb/ft^3) and the optimum moisture content was 9.3%. Figure A.4.3 shows the results of the compaction test of SR 37 soils; the maximum dry unit weight was 17.2 kN/m^3 (109.2 lb/ft^3) and the optimum moisture content 17.8%. SR 37 soils showed differences in the results depending on the location of the soil samples. Clearly, the soil at station 2 is quite different from the rest and shows smaller maximum dry density at much higher optimum water content. Compaction tests of SR 641 soils are shown in Figure A.4.4. The maximum dry unit weight of SR 641 soils was 17.9 kN/m^3 (113.7 lb/ft^3) and the optimum moisture content was 15.1%. The similarity of the results points to similar soils along the site. For Ramp A soils, the maximum dry unit weight was 17.2 kN/m^3 (109.2 lb/ft^3) and the optimum moisture content 17.8%. Figure A.4.5 plots the Proctor test results of Ramp A soils. The maximum dry unit weight was 18.7 kN/m^3 (119.2 lb/ft^3) and the optimum moisture content was 12.3%. The results show very little variability.

4.1.3 Atterberg Limit Tests

Atterberg limit tests were performed on all the soil samples. Both the liquid limit and the plastic limit tests were conducted according to the AASHTO T 89-10 (2011) standard test procedure. The test results at each station, for all four sites, are given in Tables A.4.1 to 4.4.

Average values for US 31 soils are 18.7% for liquid limit, LL, 10.1% for plastic limit, PL, and 8.6% for plasticity index, PI. SR 37 soils have 40.6% liquid limit, 18.4% plastic limit and 22.2% plasticity index (see Table A.4.2). SR 641 soil samples had 34.8%, 16.4%, and 18.4% LL, PL and PI respectively. Ramp A subgrade soils had average values of *LL*, *PL*, and *PI* 29.1%, 15.0%, and 14.0%, respectively.

4.1.4 Soil Classification According to AASHTO M 145-91 (2012)

Table A.4.5 provides a summary of selected properties of the soils collected such as percentage of fines, Atterberg limits, in-situ water content, maximum dry unit weight and soil classification. Based on AASHTO M 145-91 (2012), the subgrade at US 31 is classified as A-4, at SR 37 as A-7-6, and at SR 641 and Ramp A, as A-6.

4.2 Resilient Modulus Tests

The 1986 AASHTO guide for the design of flexible pavements requires the input value of the resilient modulus (M_R) for the subgrade soils. M_R has been used to capture the behavior of subgrade soils, which is nonlinear and time-dependent under traffic loading. For the design of both flexible and rigid pavements, having accurate and representative values of M_R is essential.

M_R is obtained by repeated loading in triaxial tests, and is obtained from the following equation:

$$M_R = \frac{\sigma_d}{\varepsilon_r}$$

where σ_d is the cyclic deviator stress; and ε_r is the recoverable axial strain.

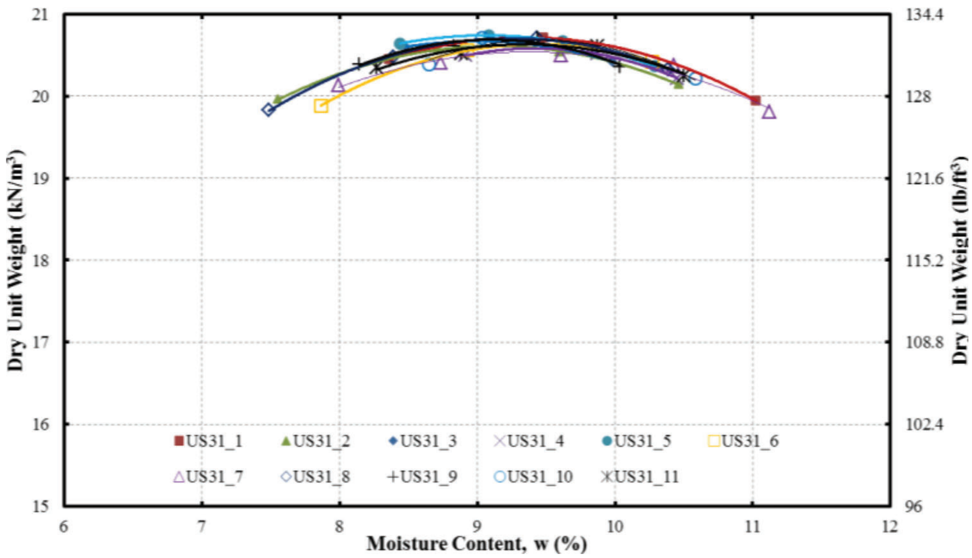


Figure A.4.2 Proctor test results of US 31 subgrade soils.

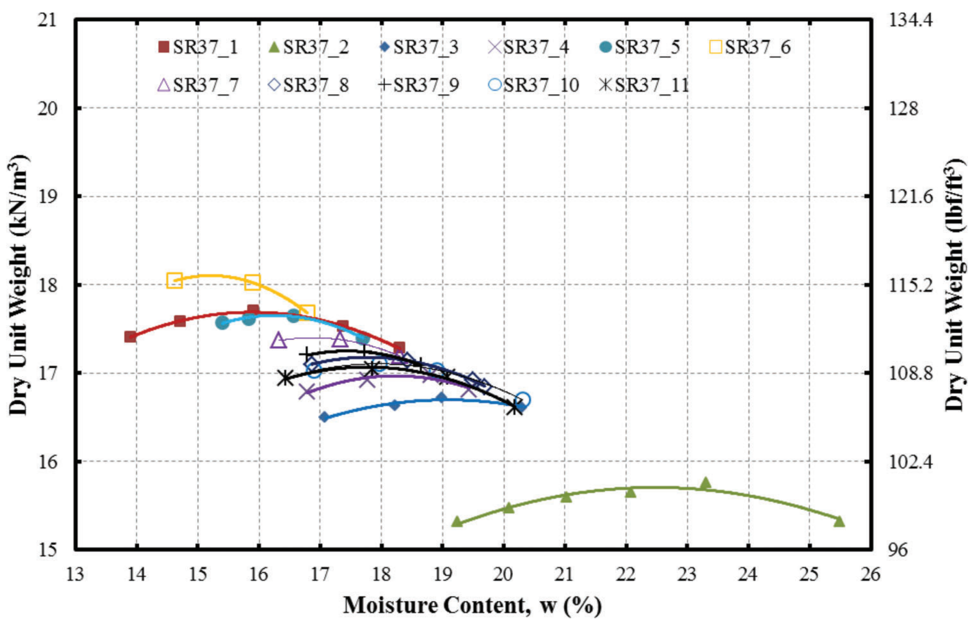


Figure A.4.3 Proctor test results of SR 37 subgrade soils.

AASHTO T 307-99 (2007) describes the current standard test method to determine the resilient modulus. In AASHTO T 307-99 (2007), a series of repeated deviator stresses (2, 4, 6, 8, 10 psi) and confining stresses (6, 4, 2 psi) simulate traffic conditions. It requires 500 to 1000 load applications with 6 psi of confining stress and 4 psi of deviator stress. Table A.4.6 provides the loading sequence that must be followed for each test.

The resilient modulus tests have been performed to determine the stiffness of the soils collected at US 31, SR 37, SR 641 and Ramp A sites, in accordance to AASHTO T 307-99 (2007). Eleven tests are done at each site, each test on the soil samples taken at each one of the eleven stations. This is done to assess the variability of the results and to provide a direct comparison between the laboratory results and the results from the field tests, since each would correspond to the exact same location.

Figure A.4.6 plots the resilient modulus of the US 31 subgrade soils. The eleven soil samples were taken from the site at 9 m intervals, as explained in Appendix Chapter 3. There are some differences of the M_R values along the site, although soil properties (passing #200, *LL*, *PL*, *PI*, Max. dry unit weight, and optimum water content) do not show much variability. The M_R of No.3, No.4, No.8, No.9 soils is roughly 40 MPa, while the M_R of No.2 soil is around 80 MPa. Most of the results show

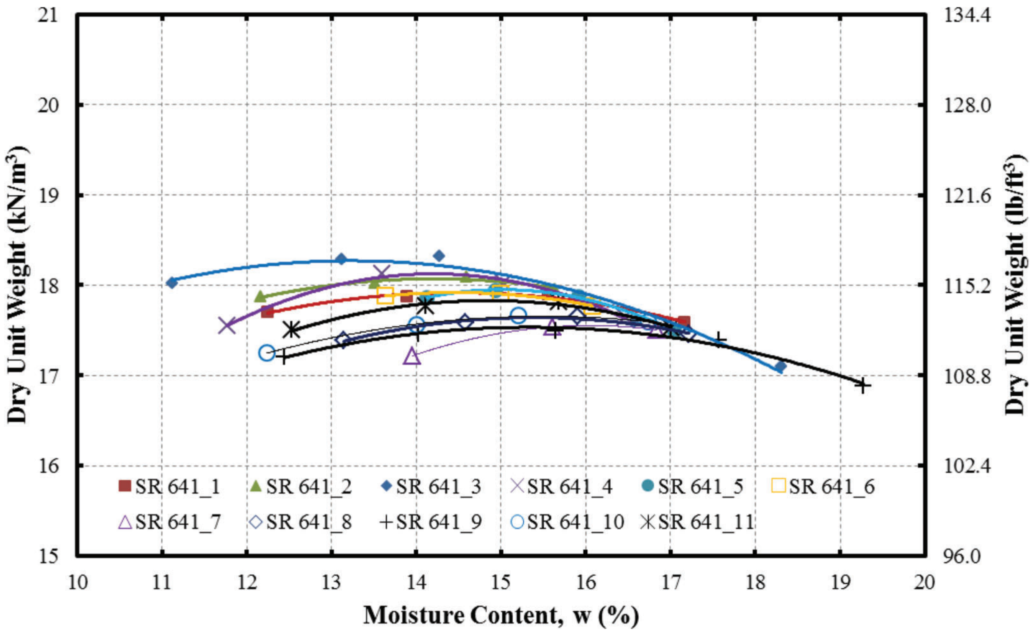


Figure A.4.4 Proctor test results of SR 641 subgrade soils.

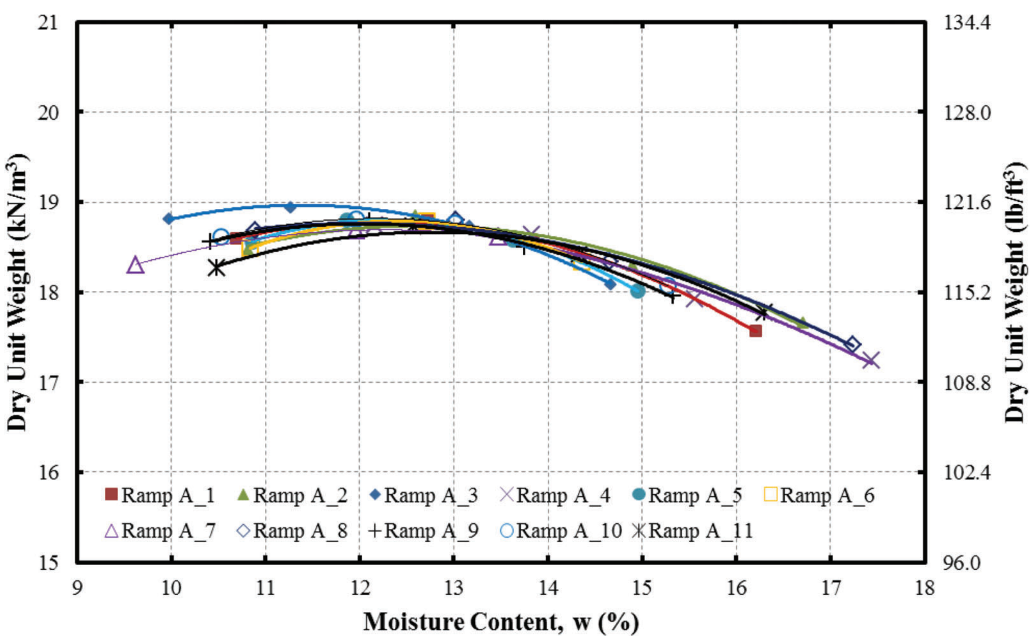


Figure A.4.5 Proctor test results of Ramp A subgrade soils.

TABLE A.4.1
Atterberg limits: US 31 subgrade

Soil Sample	US 31_1	US 31_2	US 31_3	US 31_4	US 31_5	US 31_6	US 31_7	US 31_8	US 31_9	US 31_10	US 31_11
LL	18.9	20.1	18.2	17.1	18.1	19.9	19.4	18.9	18.4	19.1	18.7
PL	10.7	10.8	9.3	9.3	10.2	10.3	9.4	11.1	10.4	10.6	9.8
PI	8.2	9.3	8.9	7.8	7.9	9.6	10.0	7.8	8.0	8.5	8.9

TABLE A.4.2
Atterberg limits: SR 37 subgrade

Soil Sample	SR 37_1	SR 37_2	SR 37_3	SR 37_4	SR 37_5	SR 37_6	SR 37_7	SR 37_8	SR 37_9	SR 37_10	SR 37_11
LL	39.5	40.1	45.9	51.5	34.9	33.5	39.0	41.8	38.1	39.4	39.6
PL	15.0	15.1	20.5	27.7	15.4	16.0	18.2	17.5	16.7	16.5	16.4
PI	24.5	25.0	25.4	23.7	19.5	17.5	20.8	24.3	21.4	22.9	23.2

TABLE A.4.3
Atterberg limits: SR 641 subgrade

Soil Sample	SR 641_1	SR 641_2	SR 641_3	SR 641_4	SR 641_5	SR 641_6	SR 641_7	SR 641_8	SR 641_9	SR 641_10	SR 641_11
LL	33.1	32.8	33.2	33.1	33.7	34.9	36.4	36.2	37.6	36.5	35.8
PL	15.3	16.8	13.8	17.6	16.2	15.8	16.1	15.2	18.2	17.7	18.1
PI	17.8	16.0	19.4	15.5	17.5	19.1	20.3	21.0	19.4	18.8	17.7

TABLE A.4.4
Atterberg limits: Ramp A subgrade

Soil Sample	RampA_1	RampA_2	RampA_3	RampA_4	RampA_5	RampA_6	RampA_7	RampA_8	RampA_9	RampA_10	RampA_11
LL	30.1	28.8	28.3	28.7	29.6	29.2	29.1	29.3	29.1	28.0	29.5
PL	15.0	15.1	14.6	14.6	15.3	15.6	15.3	15.0	14.8	14.4	15.4
PI	15.1	13.8	13.7	14.0	14.3	13.6	13.9	14.3	14.3	13.5	14.1

TABLE A.4.5
Characterization test results and classification of the subgrade soils

Site	#200 Passing (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index	Water Content$_{opt}$ (%)	Max. Dry Unit Weight (kN/m^3)	AASHTO Classification
US 31	58	18.6	10	8.5	9	21.0	A-4
SR 37	88	41.4	17.7	23.8	16~20	17.0	A-7-6
SR 641	83	31.2	13.8	19.4	14~16	18.0	A-6
Ramp A	72	29.0	15.0	14.0	12~13	18.7	A-6

TABLE A.4.6
The resilient modulus test procedure (AASHTO T 307-99, 2007)

	Confining Pressure, σ_3		Max. Axial Stress, σ_{max}		Cyclic Stress, σ_{cyclic}		Constant Stress, 0.1 σ_{max}		
Sequence No.	kPa	psi	kPa	psi	kPa	psi	kPa	psi	No. of Load Applications
0	41.4	6	27.6	4	24.8	3.6	2.8	0.4	500-1000
1	41.4	6	13.8	2	12.4	1.8	1.4	0.2	100
2	41.4	6	27.6	4	24.8	3.6	2.8	0.4	100
3	41.4	6	41.4	6	37.3	5.4	4.1	0.6	100
4	41.4	6	55.2	8	49.7	7.2	5.5	0.8	100
5	41.4	6	68.9	10	62.0	9.0	6.9	1.0	100
6	27.6	4	13.8	2	12.4	1.8	1.4	0.2	100
7	27.6	4	27.6	4	24.8	3.6	2.8	0.4	100
8	27.6	4	41.4	6	37.3	5.4	4.1	0.6	100
9	27.6	4	55.2	8	49.7	7.2	5.5	0.8	100
10	27.6	4	68.9	10	62.0	9.0	6.9	1.0	100
11	13.8	2	13.8	2	12.4	1.8	1.4	0.2	100
12	13.8	2	27.6	4	24.8	3.6	2.8	0.4	100
13	13.8	2	41.4	6	37.3	5.4	4.1	0.6	100
14	13.8	2	55.2	8	49.7	7.2	5.5	0.8	100
15	13.8	2	68.9	10	62.0	9.0	6.9	1.0	100

that the confining stress and the deviator stress have a minor influence on M_R, except perhaps at locations No. 2 and No.11.

The M_R test results of SR 37 soils are shown in Figure A.4.7. Similar to US 31 soils, the differences in M_R among the soil samples are small. The resilient modulus ranges from 60 MPa (8.7ksi) to 135 MPa (19.6ksi). Most of the results show that confining stress and deviator stress have a minor influence on M_R. There are exceptions: No.2 and No.8 soils show an increase of M_R with the increase of deviator stress, while it decreases with the increase of deviator stress for No. 3 and No. 11 soils.

The resilient modulus for SR 641 soils are plotted in Figure A.4.8. The values are between 45 MPa (6.5 ksi) and 125 MPa (18.1 ksi). As with the other cases, there are differences; for example the resilient modulus of No.2, No.5, and No.6 soil samples are 50~70 MPa, which are lower than the other soils. These differences are associated with soil/site variability even though the soil properties (passing #200, LL, PL, PI, the Max. dry unit weight, and the optimum water content) are fairly uniform. Consistent with the trends observed at the other sites, the resilient modulus does not show a strong dependence on confinement or deviator stress.

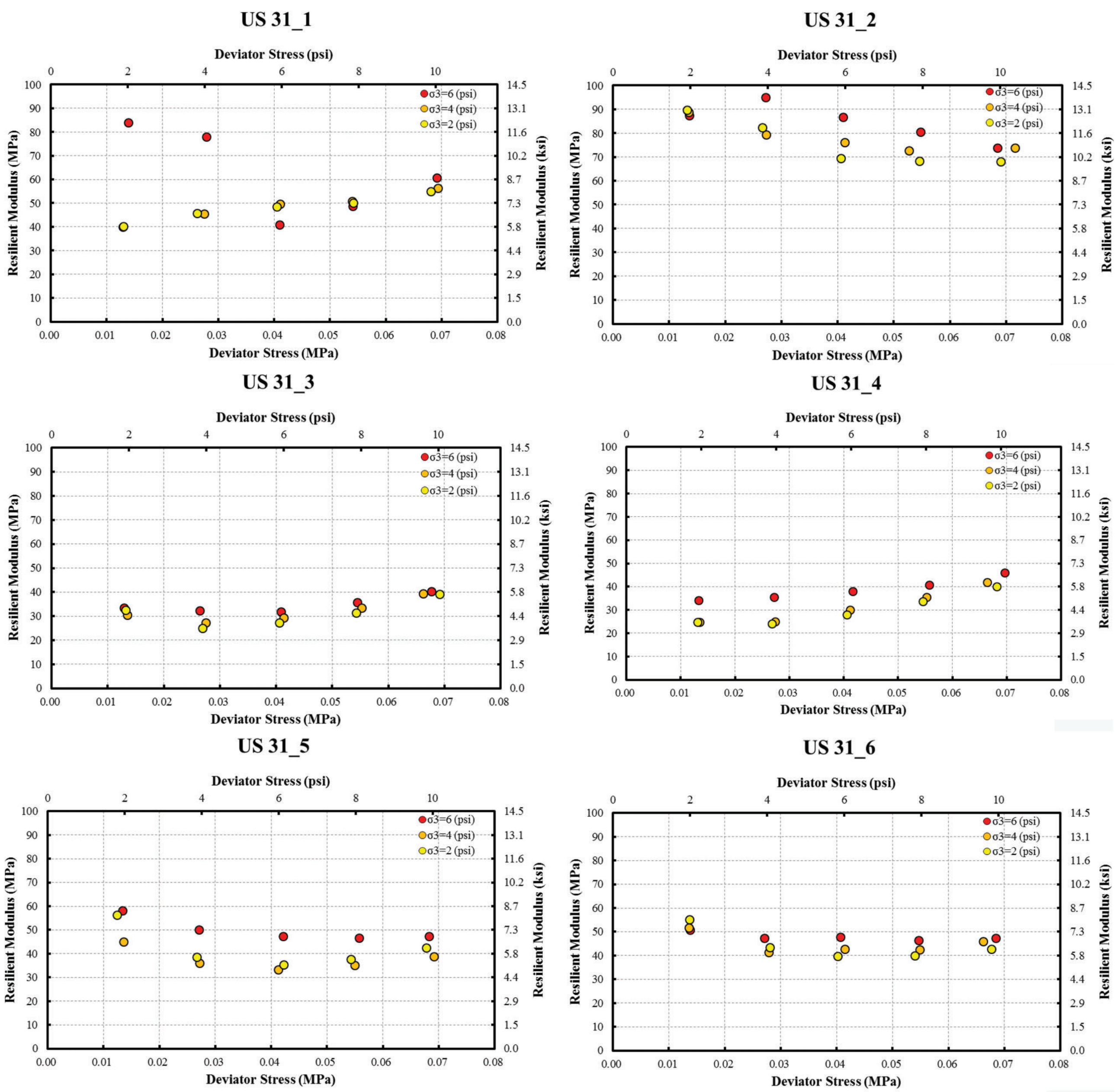


Figure A.4.6 Resilient modulus test results of US 31 soils. (Figure continued next page.)

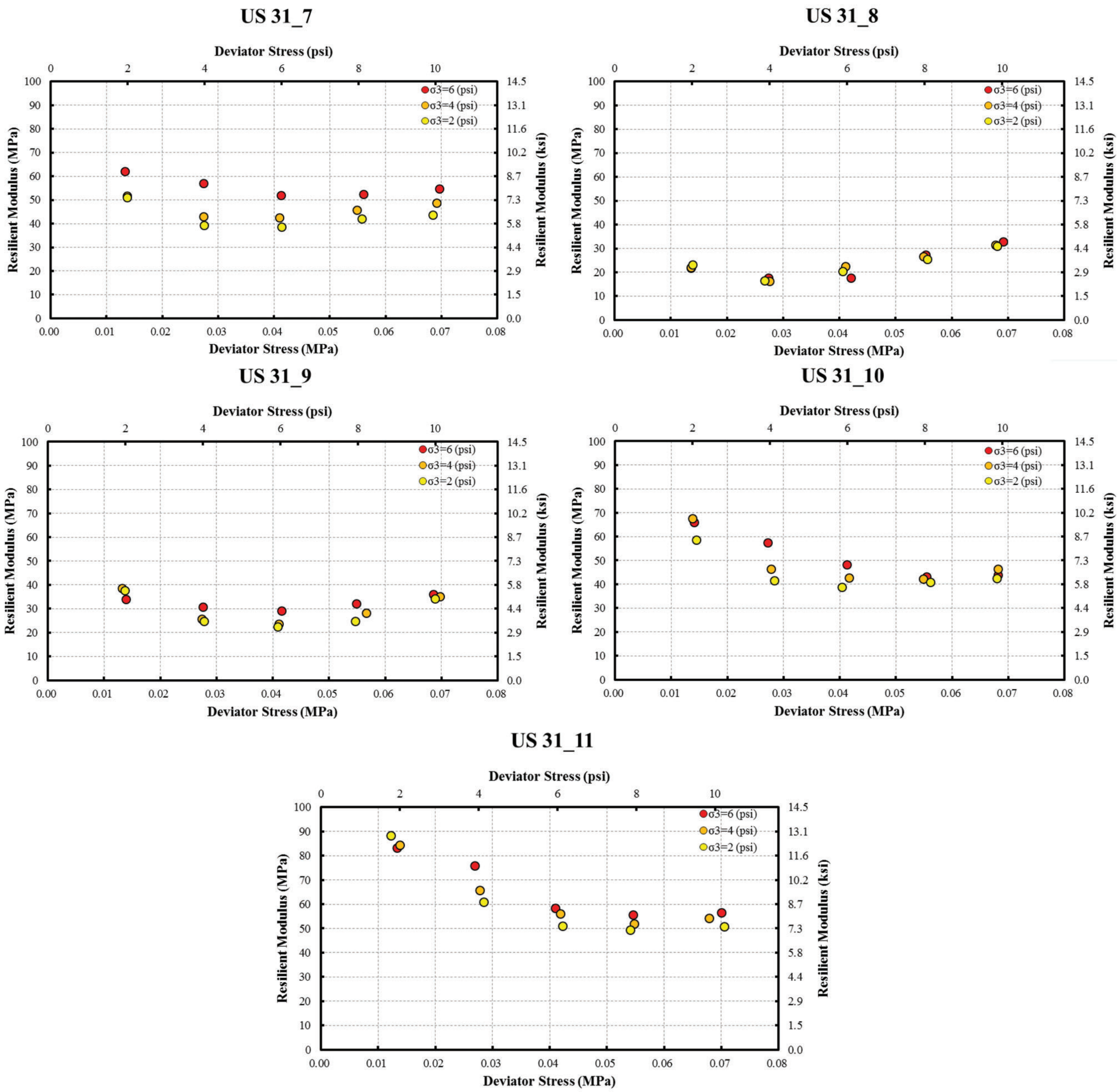


Figure A.4.6 (Continued)

The results of the resilient modulus test for Ramp A soils are shown in Figure A.4.9. The M_R values are within a range of 45 MPa (6.5 ksi) and 90 MPa (13.0 ksi). M_R of No. 3 soil is about 80 MPa, which is somewhat higher than the rest, but the difference is small. Most of the M_R values of Ramp A soils are around 60 MPa (8.7 ksi). Similar to the other soils from the other sites (US 31, SR 37, and SR 641 sites), there is no clear effect of deviator stress, nor of confining stress.

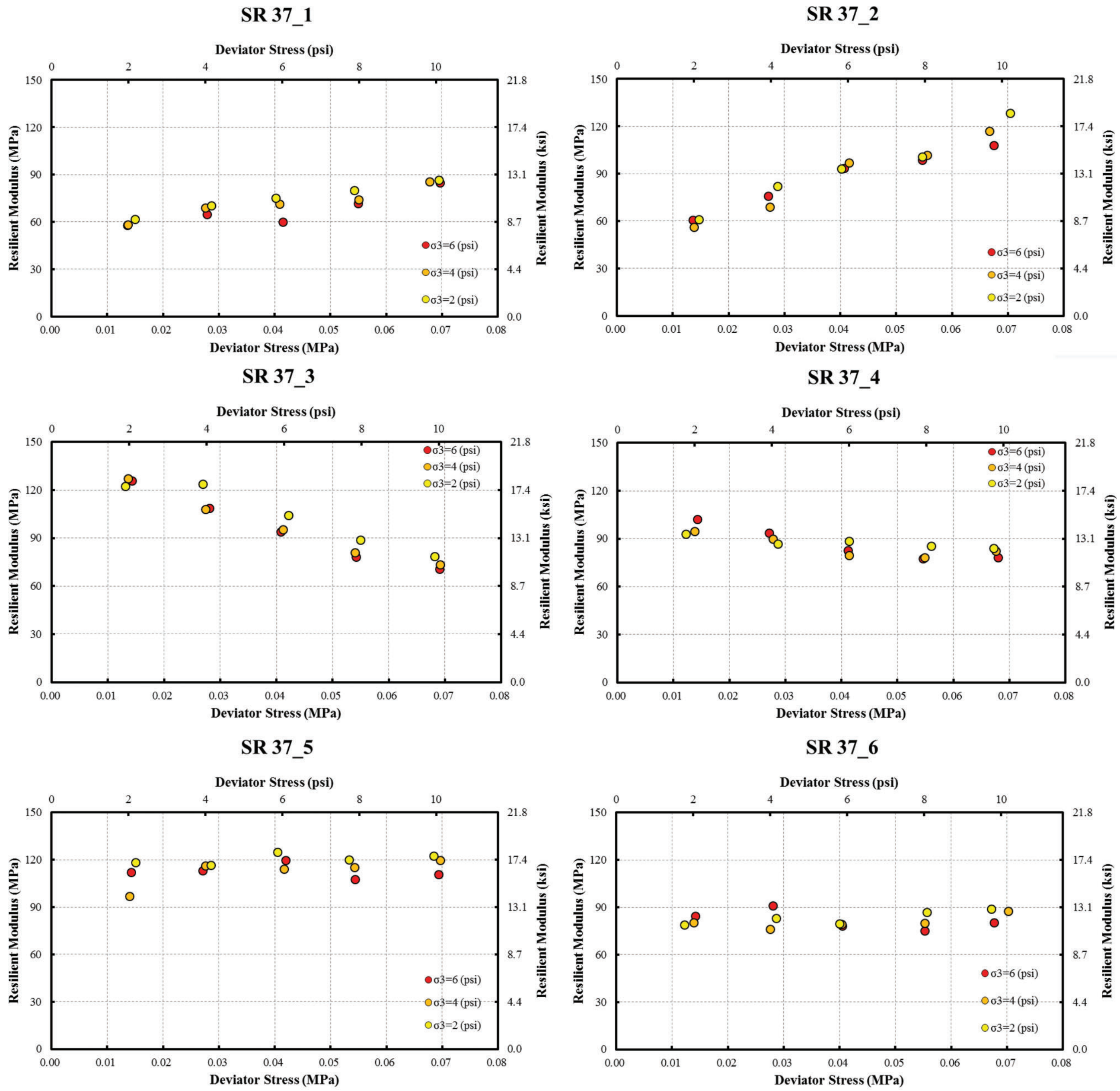


Figure A.4.7 The resilient modulus test results of SR 37 soils. (Figure continued next page.)

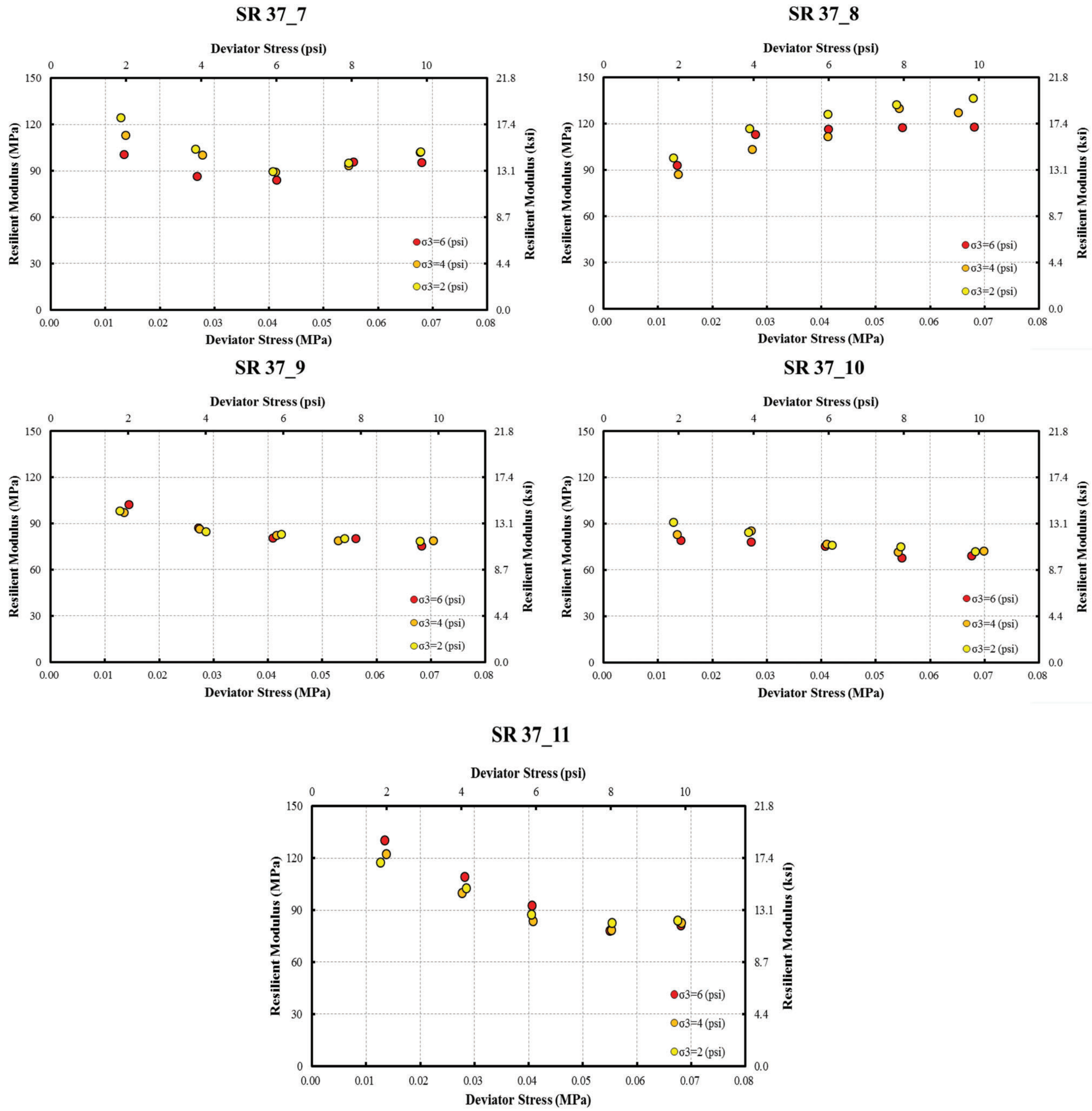


Figure A.4.7 (Continued)

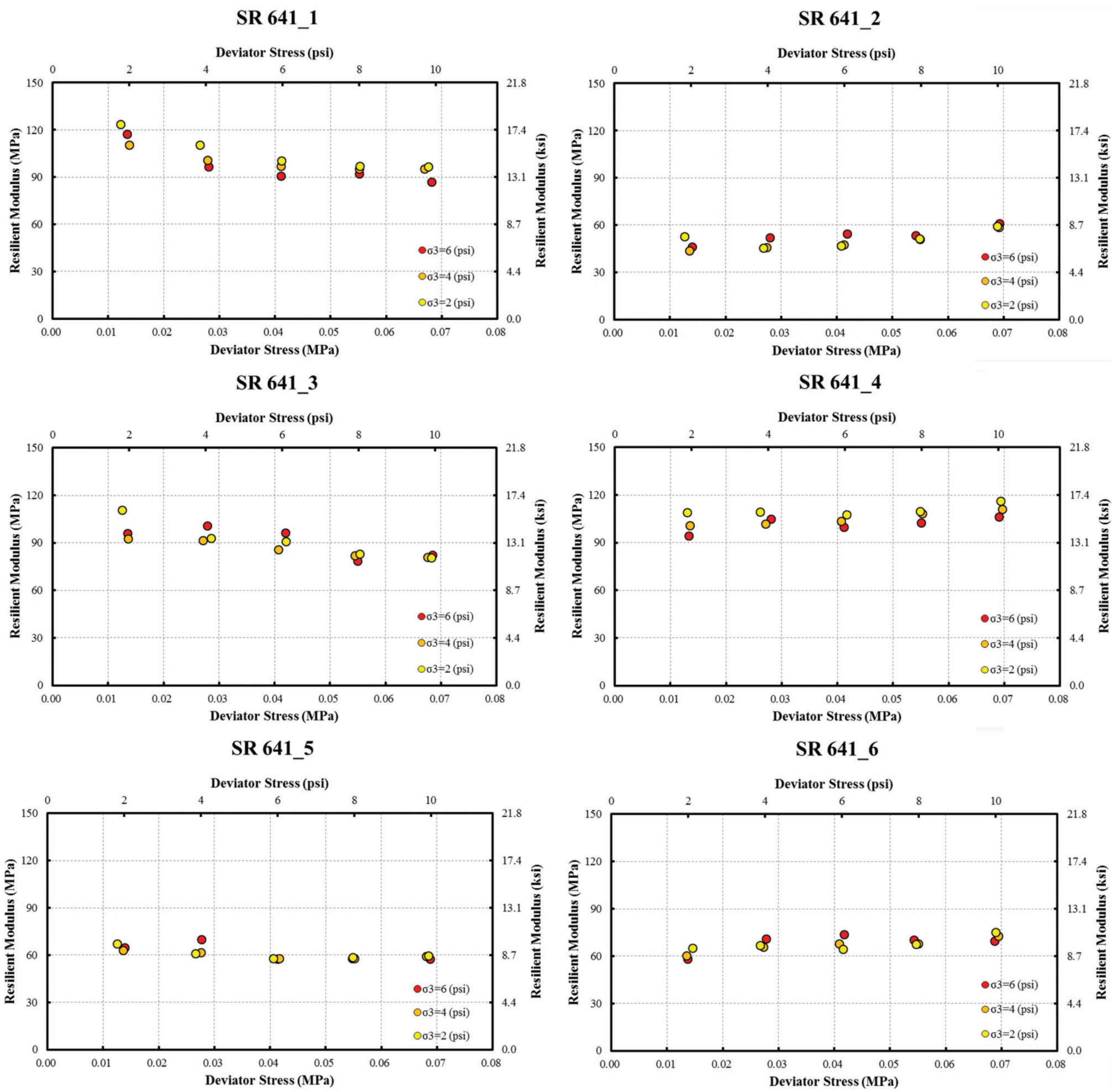


Figure A.4.8 The resilient modulus test results of SR 641 soils. (Figure continued next page.)

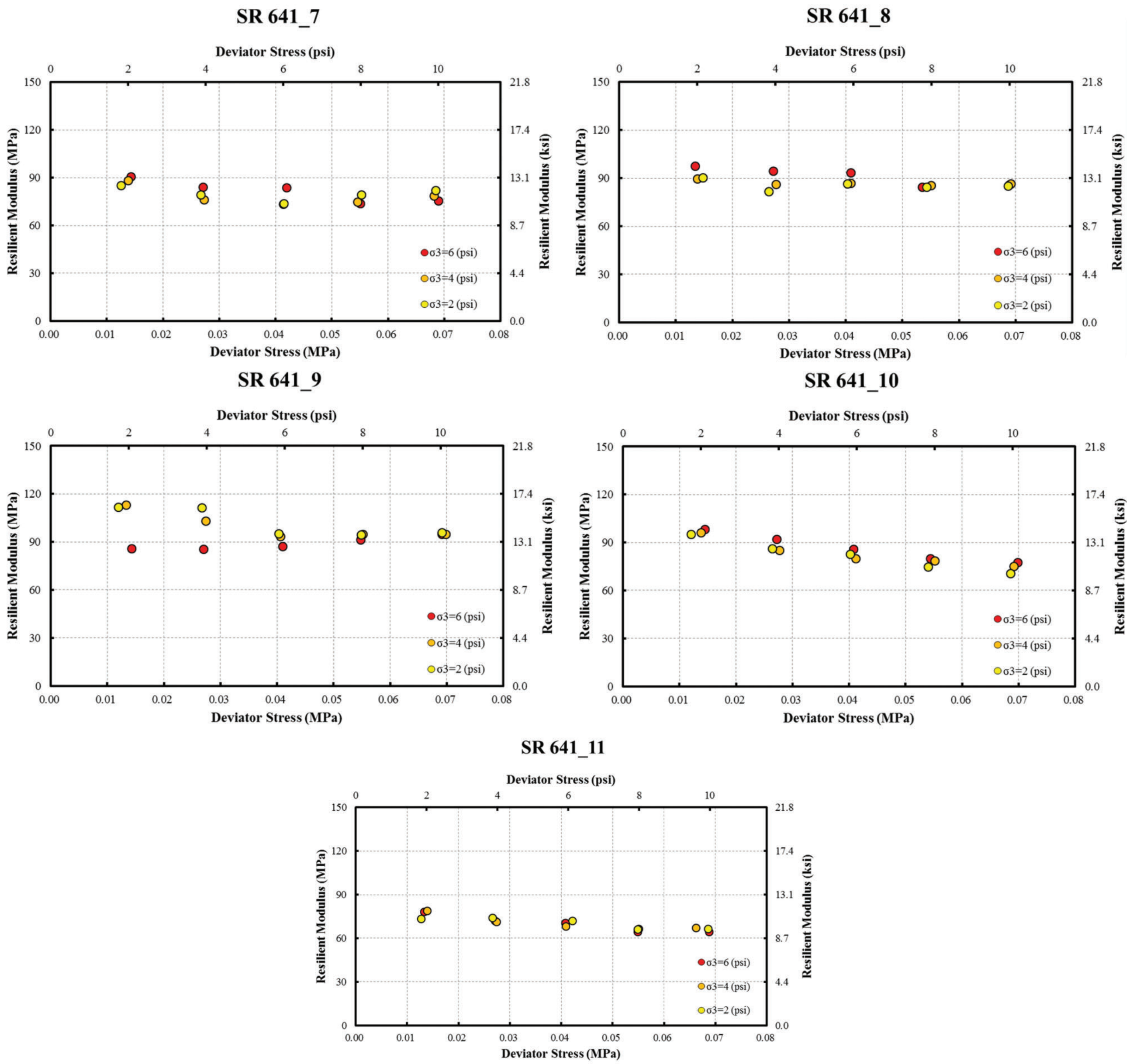


Figure A.4.8 (Continued)

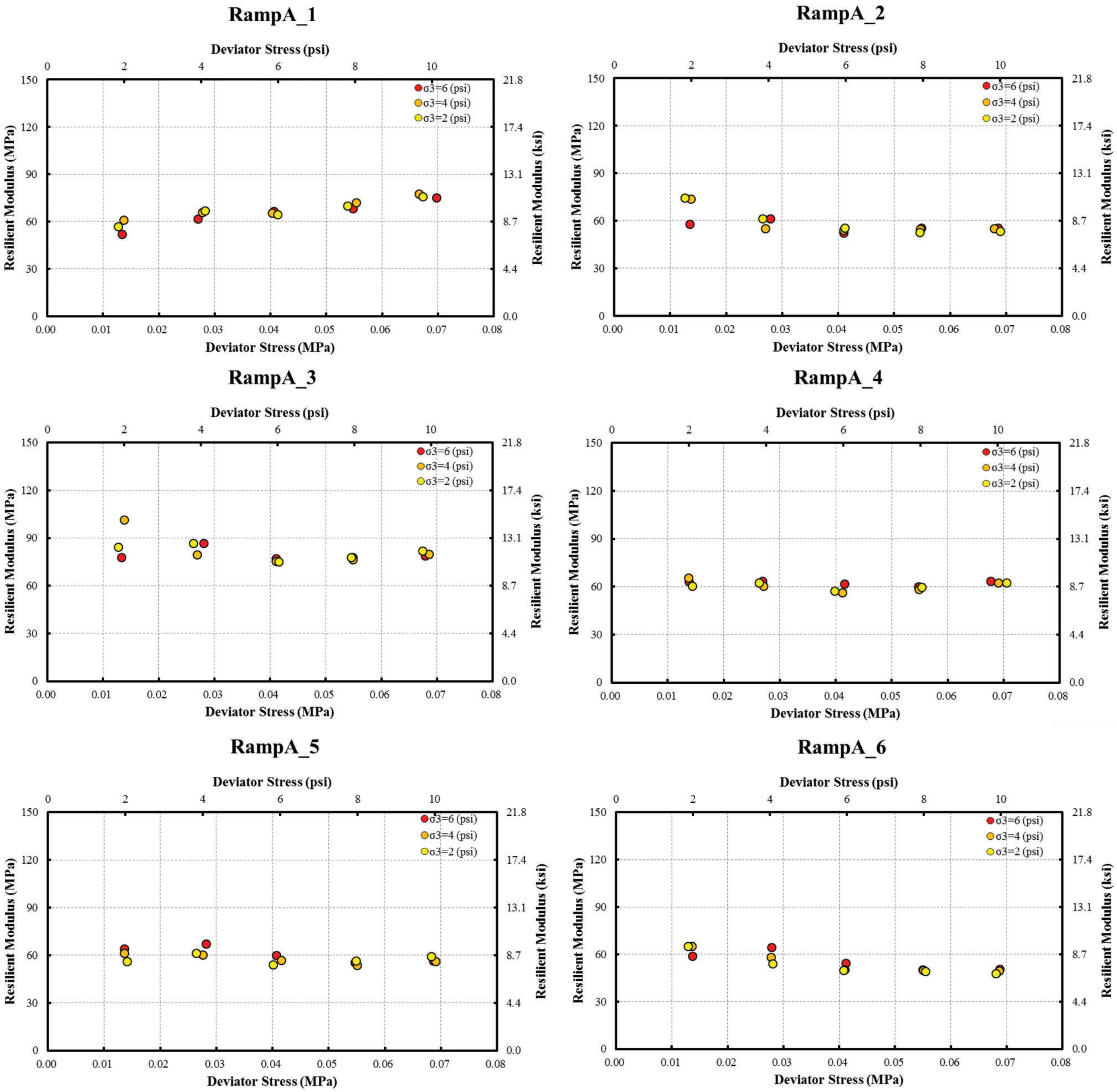


Figure A.4.9 The resilient modulus test results of Ramp A soils. (Figure continued next page.)

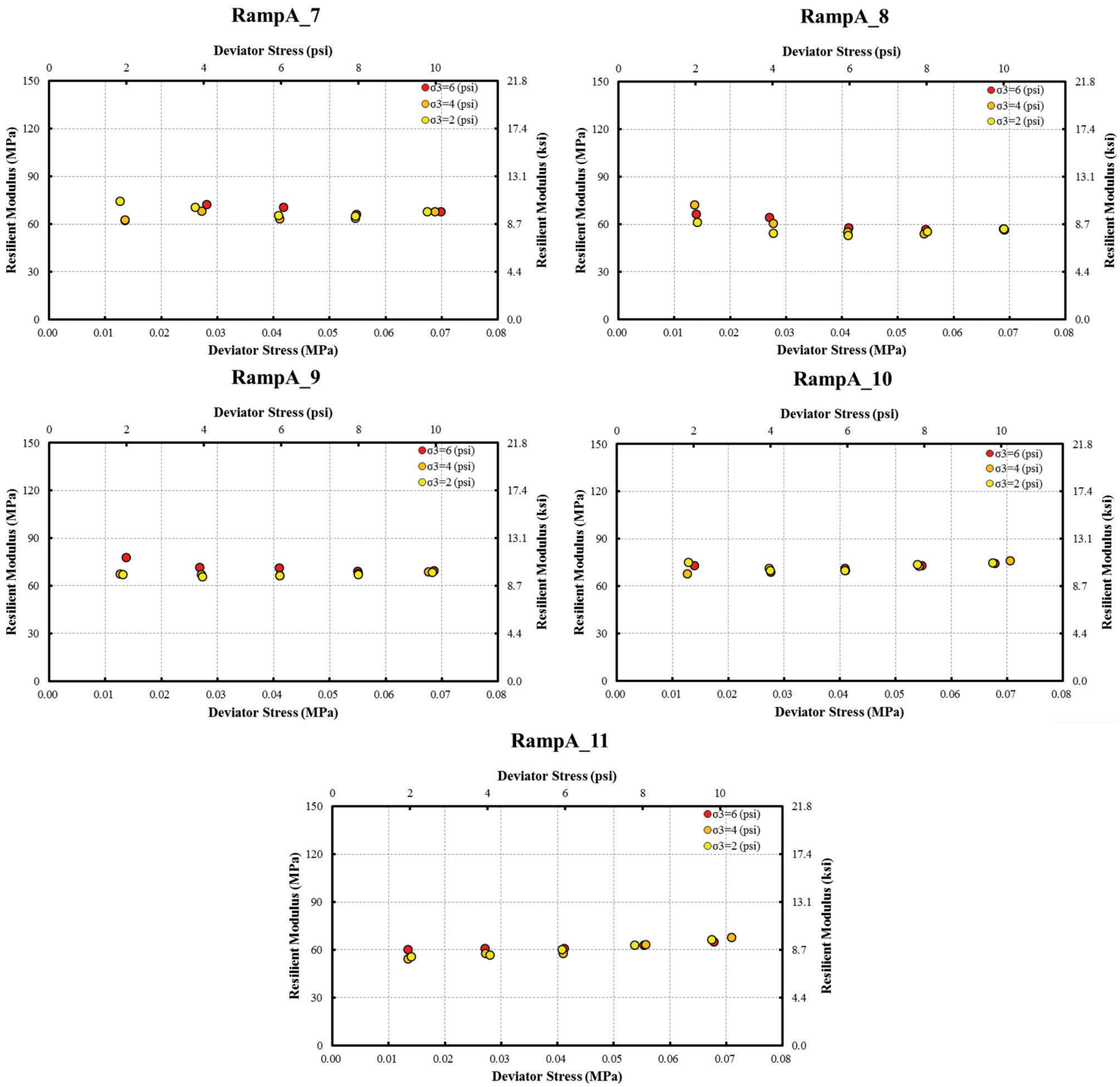


Figure A.4.9 (Continued)

A.5. ANALYSIS OF TEST RESULTS

Four sites (US 31, SR 37, SR 641, and Ramp A) were selected to run field tests and collect soils samples for laboratory testing. Field testing and lab testing have been conducted to find physical and mechanical properties of the soil samples.

In this chapter, a correlation model between M_R and soil properties is attempted such that appropriate M_R values can be predicted using statistical processes using regression analysis. Existing correlations between M_R and *FWD*, *LWD*, and *DCP* tests are discussed by comparing the M_R in the laboratory with estimates from correlations with field tests.

5.1 Regression Analysis

There are a number of proposals for M_R models for cohesive soils. Uzan (1985) proposed the following model, which is known as the universal model:

$$M_R = k_1 p_a \left(\frac{\sigma_b}{p_a}\right)^{k2} \left(\frac{\sigma_d}{p_a}\right)^{k3}$$

where, k_1, k_2, k_3 = regression coefficients; σ_b = the sum of the principal stresses = bulk stress; σ_d = deviator stress; p_a = atmospheric pressure.

In 1987, a constitutive model was presented by Lade and Nelson (1987) that supported Uzan's model. The model was based on the assumption that the modulus was a function of the first and the second invariants of the deviatoric stress tensor, for granular materials. Uzan and Scullion (1990) suggested that the model could be used for all types of soils.

Thompson and Robnett (1979) proposed the arithmetic model, which was used in the ILLI-PAVE program. The model states that:

$$M_R = k_2 + k_3(k_1 - \sigma_d), \text{ if } \mathrm{k}_1 > \sigma_d$$

$$M_R = k_2 + k_4(k_1 - \sigma_d), \text{ if } \mathrm{k}_1 < \sigma_d$$

where, k_1, k_2, k_3, k_4 = material and physical property parameters; σ_d = deviatoric stress.

In the MEPDG, the Octahedral stress model (NCHRP, 2004) is recommended:

$$M_R = k_1 p_a \left(\frac{\sigma_b}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3}$$

where, k_1, k_2, k_3, = material and physical property parameters; τ_{oct} = octahedral stress; p_a = atmospheric pressure.

Most proposed models depend on the bulk stress, the deviator stress, and the confining stress. However, the resilient modulus test results from the four selected sites (see Chapter 4) do not show stress-dependent behavior, not only on confining stress, but also on deviator stress. For this study, a stress-independent model seem to apply. That is,

$$M_R = k_1 p_a$$

where, k_1 = regression coefficient; p_a = atmospheric pressure.

The regression coefficient k_1 is found using linear regression for the following parameters: optimum water content, maximum dry unit weight, percentage soil passing #200 sieve, and Atterberg limits. These physical soil properties have been mentioned as important input variables (Drumm, Boateng-Poku, & Pierce, 1990; George, Bajracharya, & Stubstad, 2004).The measured M_R and the predicted M_R are plotted in Figure A.5.1, Figure A.5.2, and Figure A.5.3 for the different soil types. For the measured M_R (obtained in the laboratory following AASHTO T 307-99, 2007), the average value is taken as representative of each soil sample. For the regression analysis, the SAS program is used. Table A.5.1 lists the relations obtained.

Figures A.5.1, A.5.2, and A.5.3 show a comparison between the measured and the predicted M_R values using the regression model in Table A.5.1. In general, there is a poor correlation between soil properties and resilient modulus, as indicated by the low values of the R^2 index that ranges from 0.14 to 0.74.

5.2 Correlation between Soil Moduli Obtained from Laboratory Resilient Modulus Tests and from *FWD*, *LWD* and *DCP* Field Tests

M_R values from the laboratory are compared with those estimated from *FWD*, *LWD*, and *DCP* tests performed in the field. ELMOD 5 is used to obtain the soil stiffness from *FWD* tests, the Boussinesq's equation for the *LWD* tests, and Salgado and Yoon's (2003) relationship to interpret the *DCP* results.

The values of the resilient modulus from the laboratory tests show some variation based on location, i.e. from test point to test point, and based on confinement and deviator stress, albeit there was no strong relation, as discussed. Figure A.5.4a includes all the laboratory tests on US 31. To facilitate comparisons, the laboratory tests are group into a band that includes all the results that fall into the 5% and 95% range, as depicted in Figure A.5.4b. This is plotted, for the rest of the discussion, as a band of results; see Figure A.5.4c.

Figure A.5.5 shows a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on A-4 soils (US 31). The horizontal axis represents the test location at the site while the vertical axis provides the values of the resilient modulus in MPa (left) or ksi (right). As one can see, estimates from *DCP* results overestimate the stiffness of the soil. This is also the case with the values from *FWD* when obtained from tests conducted on top of the subgrade. Results from *LWD* and from *FWD*

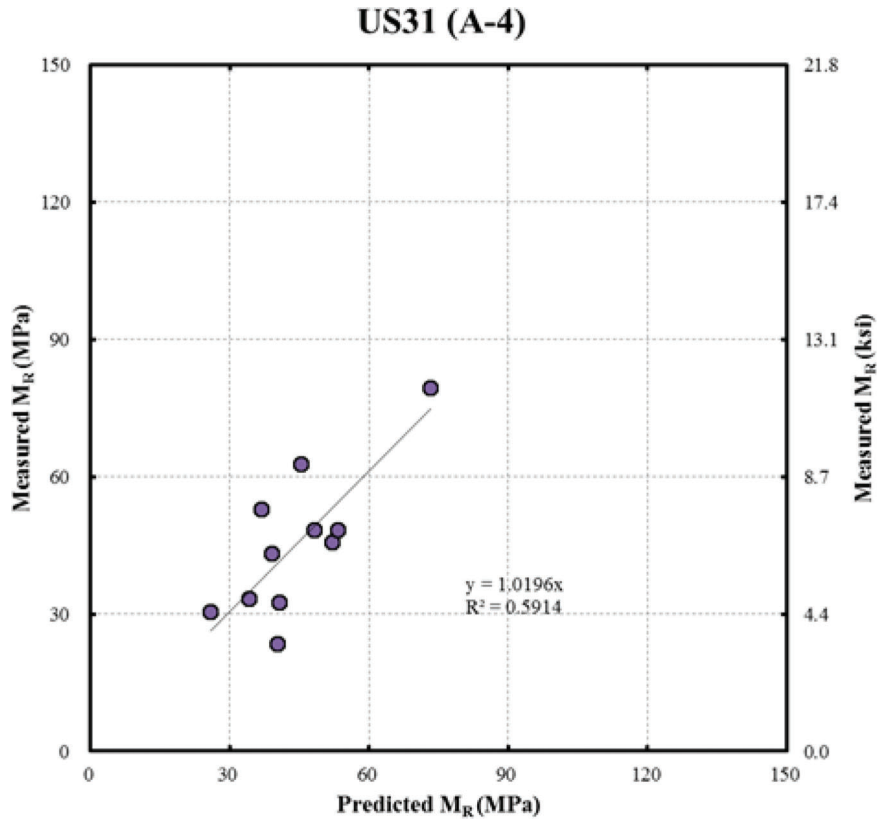


Figure A.5.1 Measured M_R vs Predicted M_R for A-4 soils (US 31).

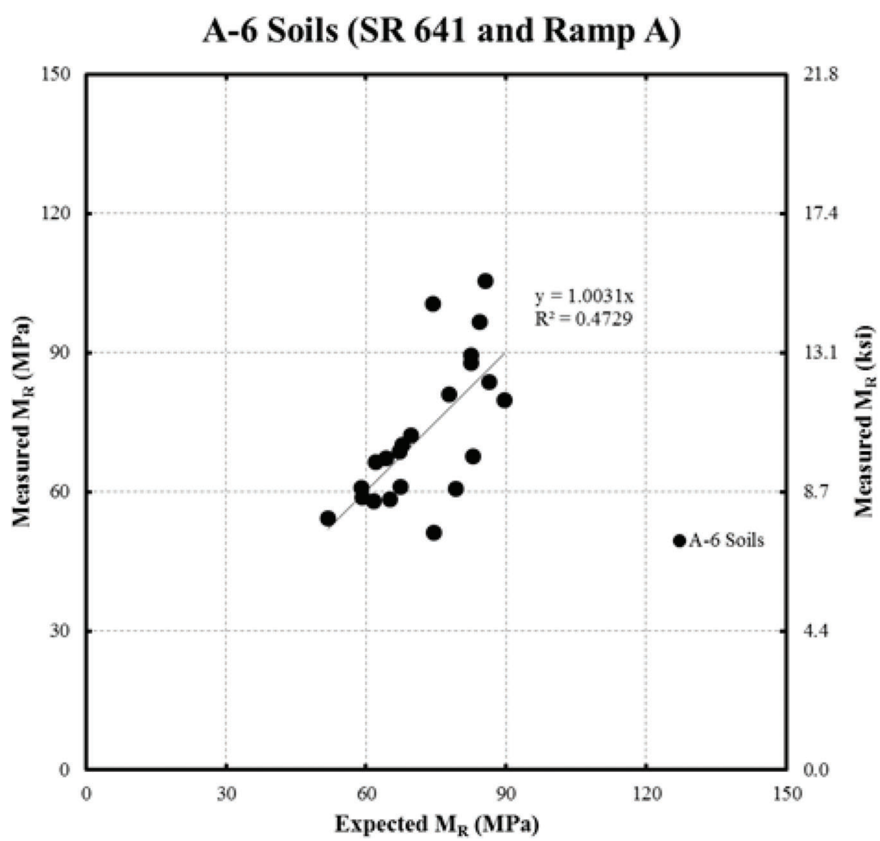


Figure A.5.2 Measured M_R vs Predicted M_R for A-6 soils (SR 641 and Ramp A).

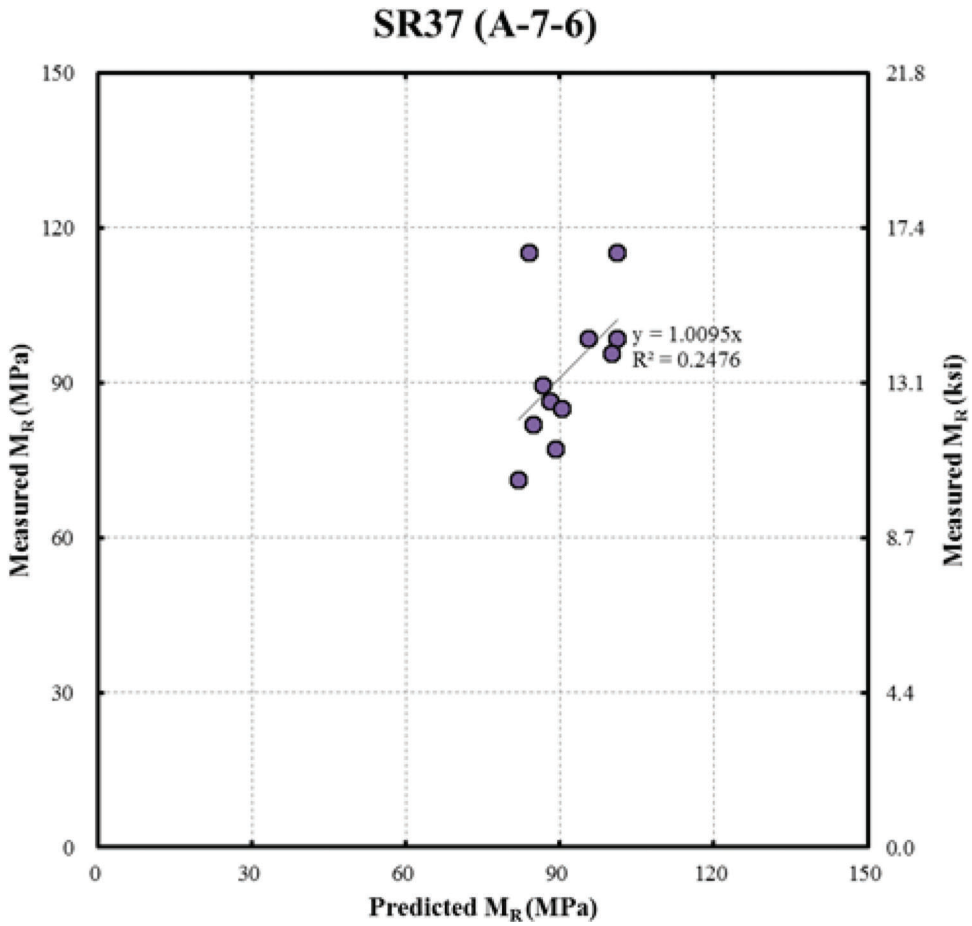


Figure A.5.3 Measured M_R vs Predicted M_R for A-7-6 soils (SR 37).

TABLE A.5.1
Regression coefficient, k_1, for each site.

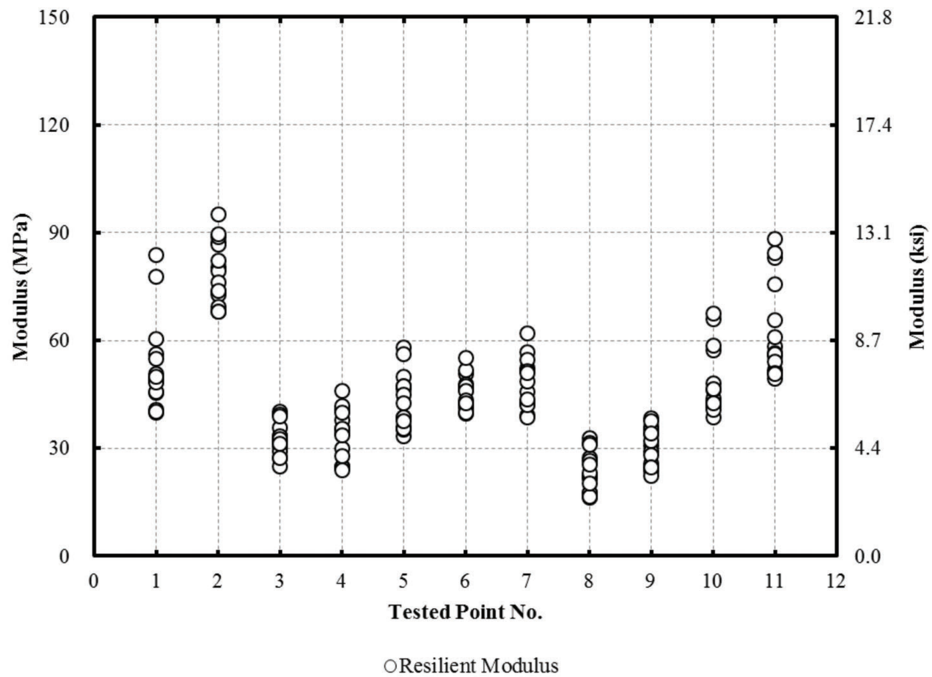
A-4 soils (US 31)	$k_1 = 7.15315 + 0.12450 \times OMC - 0.45279 \times Max.\gamma_d + 0.22516 \times LL + 0.01575 \times PL - 0.05063 \times \%200 \; R^2 = 0.59$
A-6 soils (SR 641)	$k_1 = 11.30669 - 0.15388 \times OMC - 0.44156 \times Max.\gamma_d + 0.01526 \times LL - 0.03706 \times PL - 0.00271 \times \%200 \; R^2 = 0.14$
A-6 soils (Ramp A)	$k_1 = 1.69335 - 0.07003 \times OMC + 0.05124 \times Max.\gamma_d - 0.00442 \times LL - 0.13179 \times PL - 0.01313 \times \%200 \; R^2 = 0.74$
A-7-6 soils (SR 37)	$k_1 = 27.33899 - 0.33287 \times OMC - 1.15747 \times Max.\gamma_d - 0.04394 \times LL + 0.03868 \times PL - 0.00482 \times \%200 \; R^2 = 0.24$

conducted on top of the pavement provide acceptable estimates, with somewhat better results from the *FWD*.

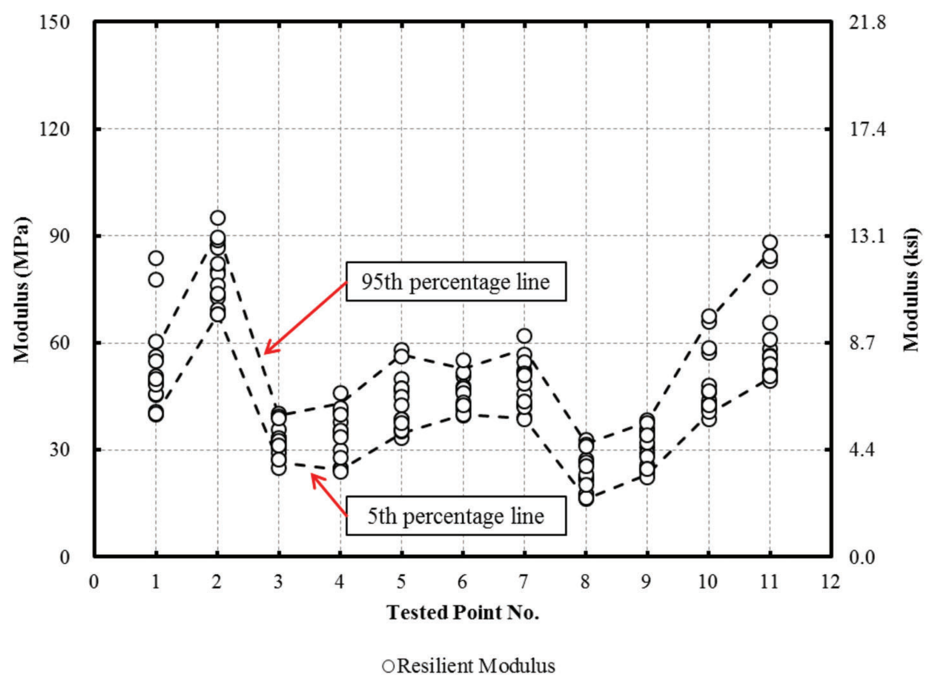
Figure A.5.6 shows a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on A-7-6 soils (SR 37). None of the estimates using measurements from *LWD*, *DCP* or *FWD* tested directly on the subgrade provide acceptable values. Results from *FWD* tests performed on the pavement give a better approximation of the resilient modulus obtained in the laboratory, in particular from the test conducted in the summer. The *FWD* measurements made in the spring also provide acceptable results, albeit on the low side. As mentioned earlier, the smaller values of the *FWD* modulus in May are likely associated with a higher water content of the subgrade due to rain.

Figure A.5.7 shows a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on A-6 soils (SR 641 site). Similar to the observations made for the A-7-6 soils in Figure A.5.6, none of the estimates from the field tests, including the *FWD* performed directly on the subgrade provide good estimates. They are all too low.

Figure A.5.8 is analogous to the previous figures. It presents a comparison between the M_R measured in the laboratory and estimated from *FWD*, *LWD*, and *DCP* tests on Ramp A site, where the soils are classified as A-6 (similar to SR 641 site with resilient modulus values shown in Figure A.5.7). The *LWD* results and the FWD results are similar to each other, but they are too low. The *DCP* modulus coincides well with the modulus obtained in the laboratory. Note that this is

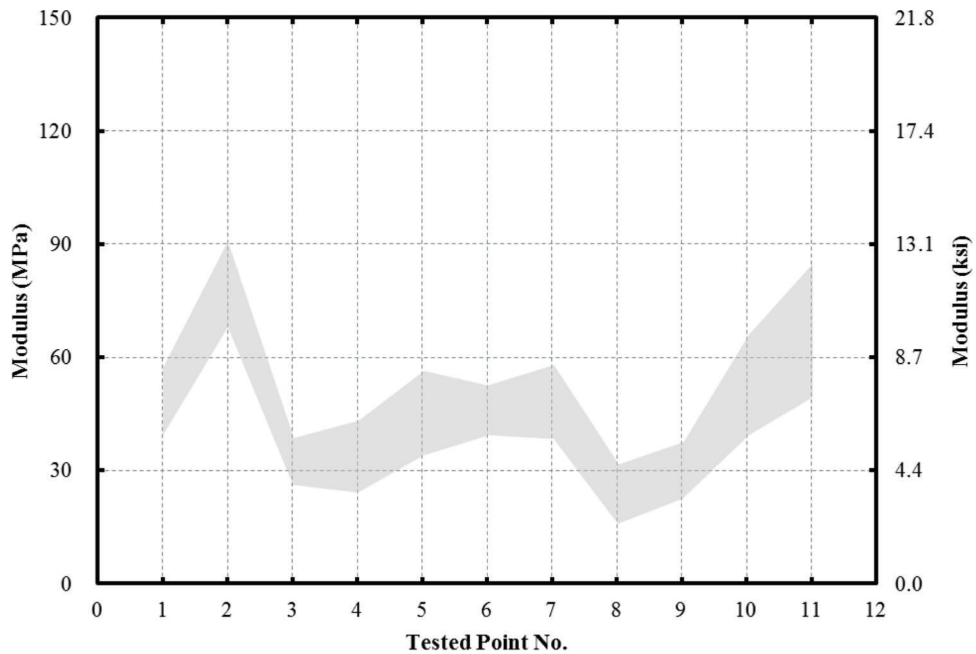


(a) Resilient modulus test results from all of the soil samples (US 31 site).



(b) Results within the 95th and 5th percentage lines (US 31 site).

Figure A.5.4 Range of resilient modulus values. (Figure continued next page.)



(c) Band of resilient modulus values (US 31 site).

Figure A.5.4 (Continued)

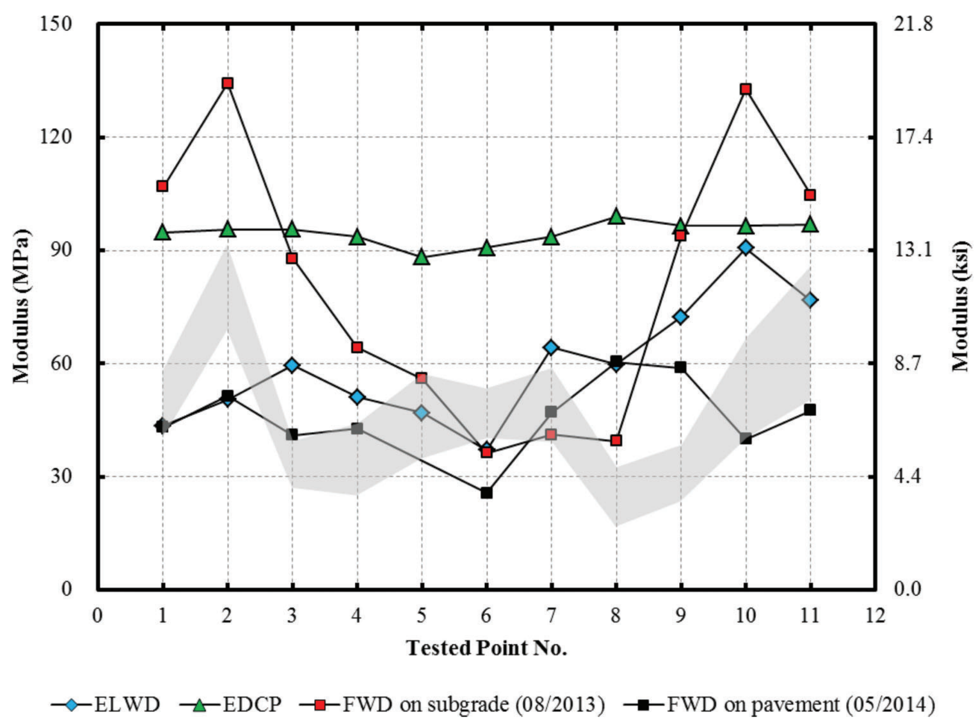


Figure A.5.5 Comparison between the resilient modulus measured in the laboratory and obtained from *FWD*, *LWD*, and *DCP* tests on A-4 soils (US 31 site).

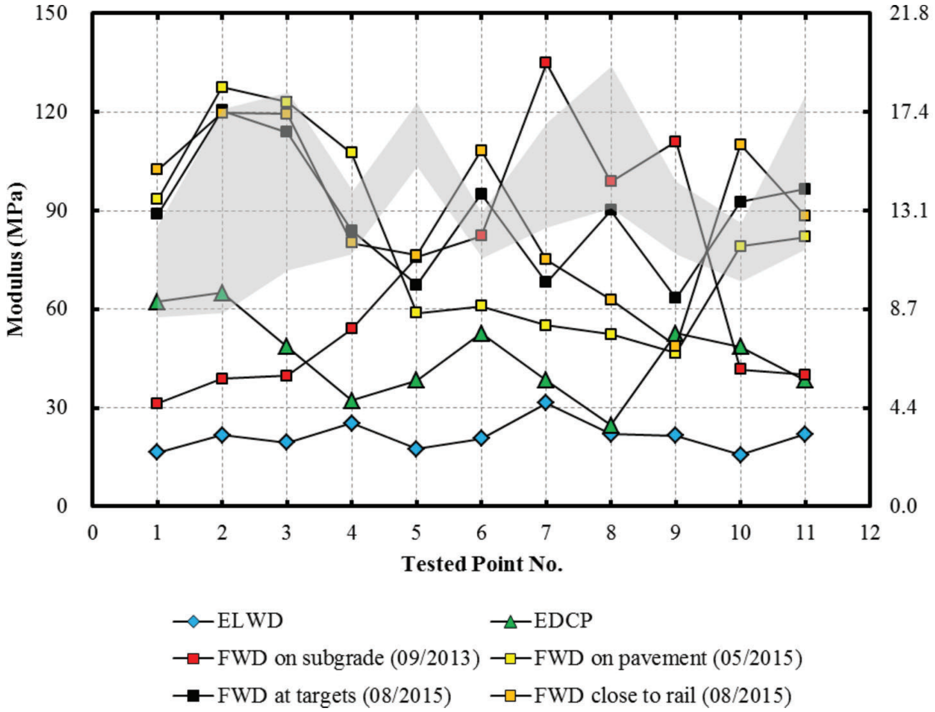


Figure A.5.6 Comparison between the resilient modulus measured in the laboratory and obtained from *FWD*, *LWD*, and *DCP* tests on A-7-6 soils (SR 37 site).

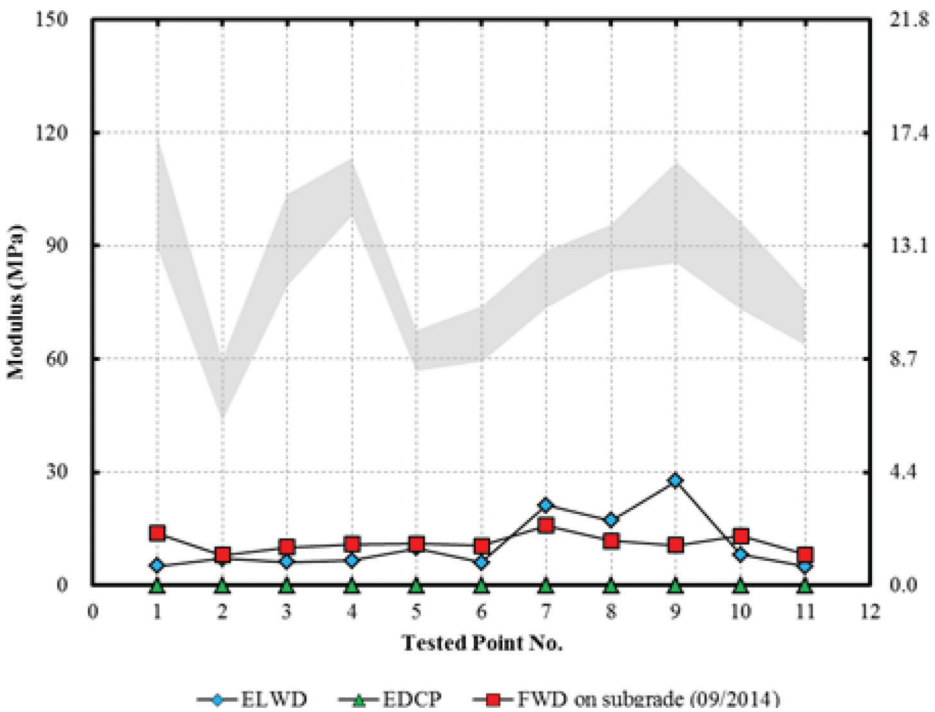


Figure A.5.7 Comparison between the resilient modulus measured in the laboratory and obtained from *FWD*, *LWD*, and *DCP* tests on A-6 soils (SR 641 site).

quite different from what was found in SR 641, which is similar soil.

Figure A.5.9 provides a comparison of the resilient modulus obtained in the laboratory following AASHTO T 307-99 (2007) and from FWD tests. For the comparison, the average values of the laboratory test results are used (see discussion in Appendix Chapter 5). In the figure, hollow symbols are used for the

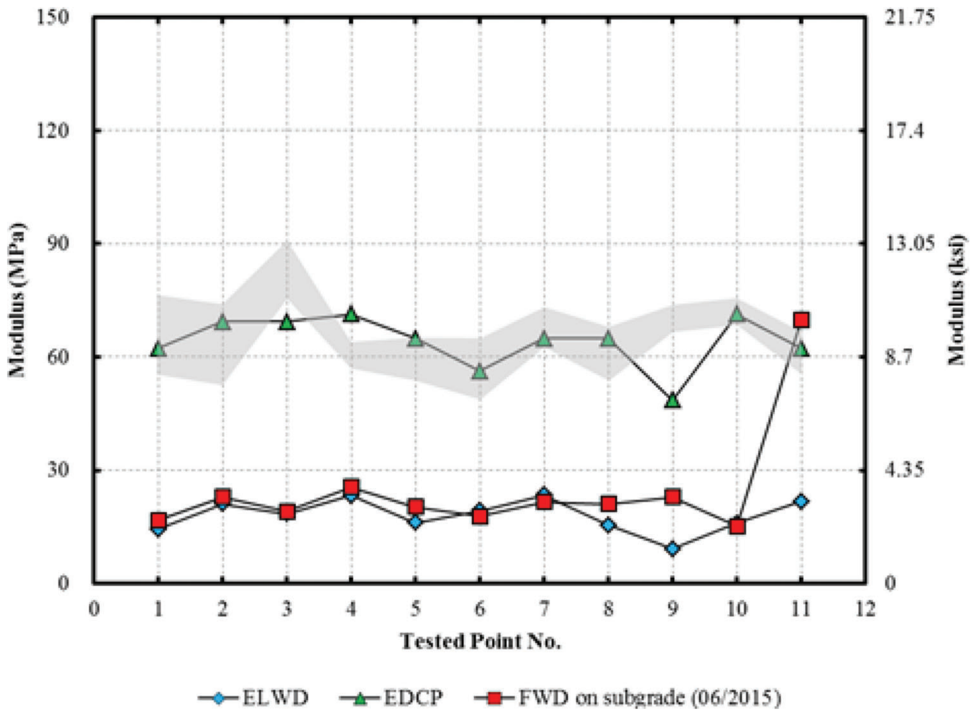


Figure A.5.8 Comparison between the resilient modulus measured in the laboratory and from *FWD*, *LWD*, and *DCP* tests on A-6 soils (Ramp A site).

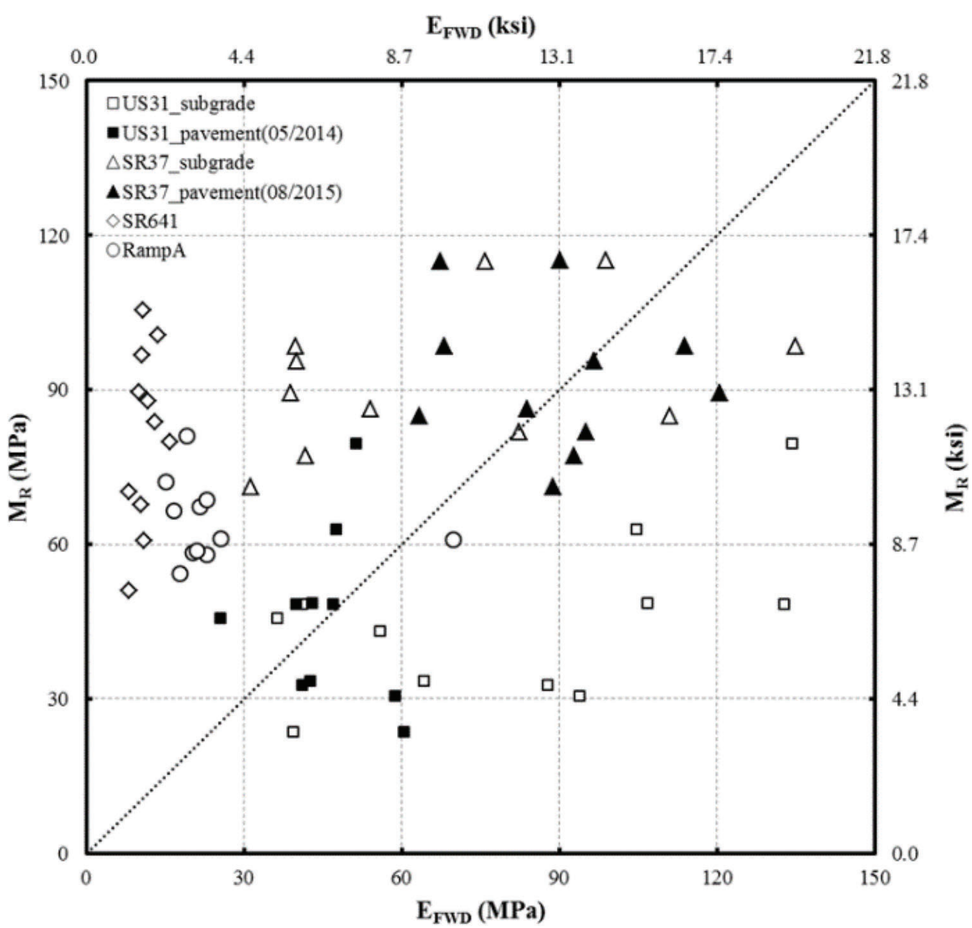


Figure A.5.9 Resilient modulus. Comparison between results from laboratory and *FWD* tests.

FWD modulus obtained from tests done directly on top of the subgrade and solid symbols from tests on top of the pavement.

The *FWD* results from subgrade soils (hollow symbols) seem to be randomly scattered and do not show a clear trend, nor there seems to be a relation between these values and the resilient modulus from laboratory tests. In contrast, the modulus obtained from *FWD* tests performed on top of the pavement do compare well with the laboratory measurements. Note that this is the case for all eleven points at each of the two sites where the test could be completed.

A.6. CONCLUSIONS AND IMPLEMENTATION

There is the need to collect and analyze additional data to support the conclusions regarding possible correlations between *FWD* and resilient modulus data collected (US 31 and SR 37). It was decided to expand the scope of the project and mine the data repository of INDOT to obtain additional geotechnical and pavement information that could use to further investigate relations between field *FWD* and laboratory resilient modulus tests. This appendix chapter describes the activities undertaken for data collection and analysis.

6.1 Collected Data from INDOT

Mr. Nayyar Siddiki provided resilient modulus data, obtained from laboratory tests, that include: resilient modulus at 6 psi deviator stress and 2 psi confining stress at optimum moisture content; location based on road number and county; soil classification; and Proctor tests results. Dr. Yigong Ji provided *FWD* data since 2008 that includes: location based on road number and RP number; deflection data; and *FWD* modulus calculated using ELMOD5.

6.2 Data Visualization

The resilient modulus and the *FWD* data were obtained independently of each other; that is, the test location and time do not match. The first step consisted of pairing the data, as best as possible, based primarily on location.

There were two issues regarding the comparison of the M_R data with the *FWD* data collected from the INDOT database. The first issue was the location. The M_R data included broad location information, i.e., road number and County, while the *FWD* data had specific location information, given by road and RP numbers. The second issue was the soil type. The *FWD* data did not include soil type, while the M_R data did.

To facilitate pairing the two sets of data, the commercial software ArcMap, a GIS based code, was used. The locations of the resilient modulus and *FWD* tests were digitized and included in the software, as shown in Figure A.6.1.

The locations of the soil samples used to obtain the resilient modulus in the laboratory are approximate. They are given in terms of the road number on a County. Therefore, M_R data are assumed representative of the entire road, which may carry a significant uncertainty, as the values are compared with the stiffness obtained from *FWD*, where the location is well specified. Figure A.6.2 shows an example of a digitized road associated with a particular M_R data on ArcMap. The soil sample was collected from an unspecified location in I-65, in Marion County.

Similar plots are produced for the *FWD* data, as shown in Figure A.6.3. As mentioned, the *FWD* data contains relatively specific information of its location, which is given by road number and RP number, e.g., I-65 from RP-100+00 to RP-107+60.

Using ArcMap has distinctive advantages for the project, as the software allows to classify the data into "layers"; that is into groups of data with similar characteristics. It also allows the display of data associated with a particular feature by clicking on the feature. For example, Figure A.6.1 shows the different layers defined: three layers for the resilient modulus, each for a particular type of soil, namely A-7-6, A-6 and A-4, and one for the *FWD*. The layers are color-coded and can be activated or deactivated, thus facilitating the spatial visualization of the data into different categories. By clicking on a particular road segment, which comprises the start and end of the RPs, a pop-up window displays a table that contains the modulus, in psi, from the *FWD* database, road number and county. See Figure A.6.4. Analogous information is displayed for the road segments associated with the laboratory resilient modulus, where the pop-up window contains the M_R value, in psi, road number, County, and soil type.

6.3 Data Classification

The data, as displayed into ArcMap, from the field, *FWD*, and laboratory, M_R, is paired by location. Clearly, given the uncertainties discussed, mostly from the M_R location, it is not possible to unequivocally establish one-to-one relations. Instead, the data is classified into three tiers. If the M_R data has only one type of soil at a given road and only one *FWD* data point exists, the data is classified as tier one. If the M_R data has only one type of soil but multiple *FWD* data exist, at the same location, the data falls into tier two. If the M_R data includes multiple types of soils, the data are categorized as tier three regardless of the number of *FWD* data.

Thirteen data points fall into tier one, which is the tier thought to be the most reliable for comparison purposes. The tier one data is plotted in Figure A.6.5a, which shows a good correlation between the resilient and the *FWD* modulus, and similar to the observations made during phase one of the project (see Figure A.5.9, reproduced here as Figure A.6.5b, which also includes the data from Figure A.6.5a).

Tier two includes 37 data points and tier three 118 data points. The comparison between M_R and *FWD*

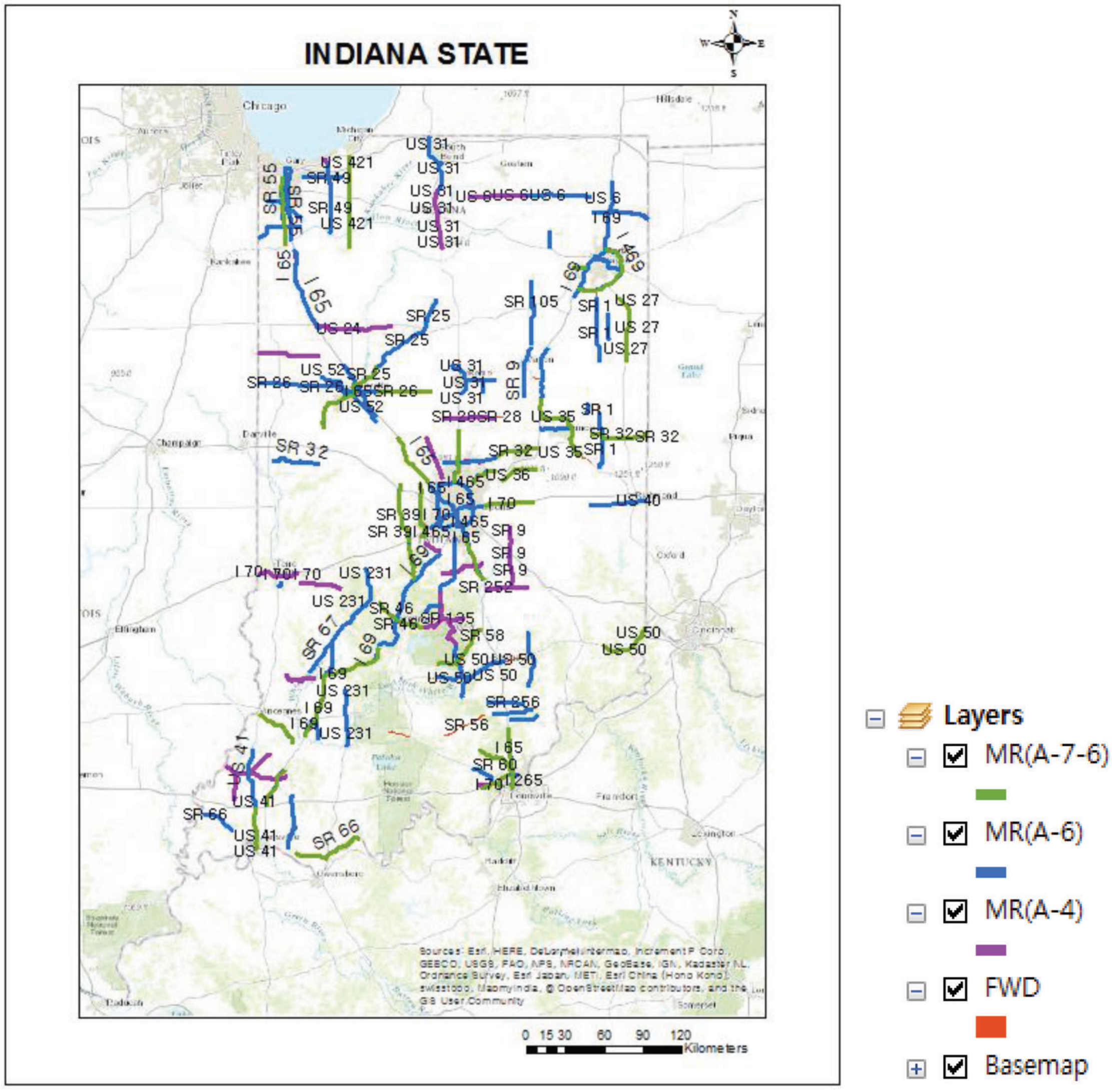


Figure A.6.1 Location of collected resilient modulus and *FWD* data in ArcMap.

data, classified as tier two and three, is shown in Figure A.6.6 and Figure A.6.7, respectively. Tier two and tier three do not show a strong correlation between M_R and *FWD*. This is somewhat expected due to the higher uncertainty associated with how the values of M_R and *FWD* have been matched.

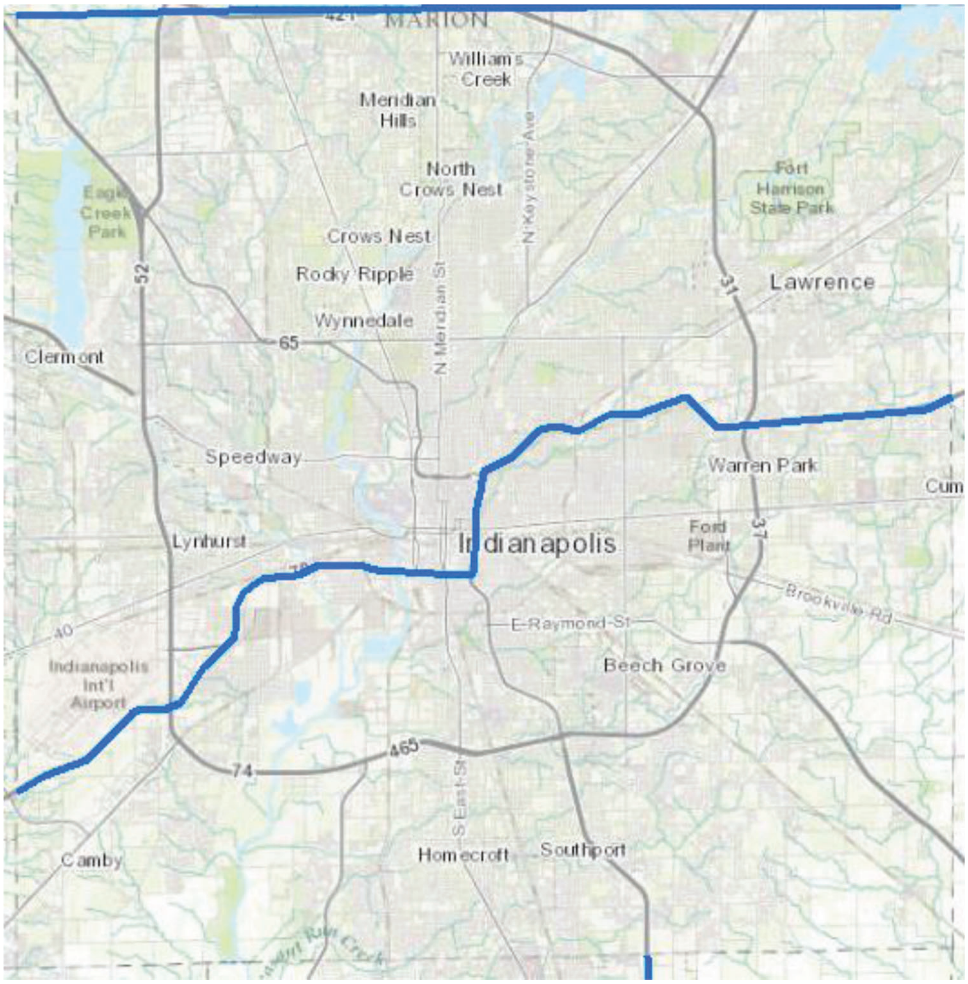


Figure A.6.2 An example of digitized M_R data(A-4) in Marion County.

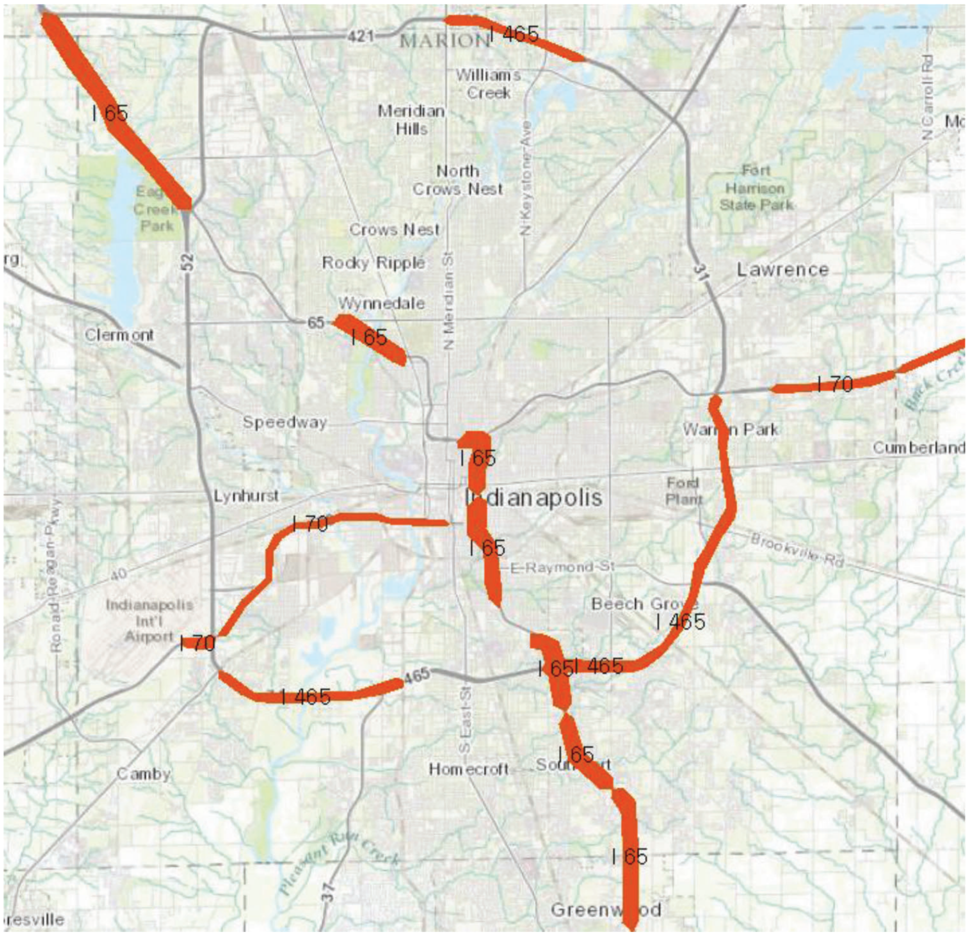


Figure A.6.3 An example of digitized *FWD* data in Marion County.

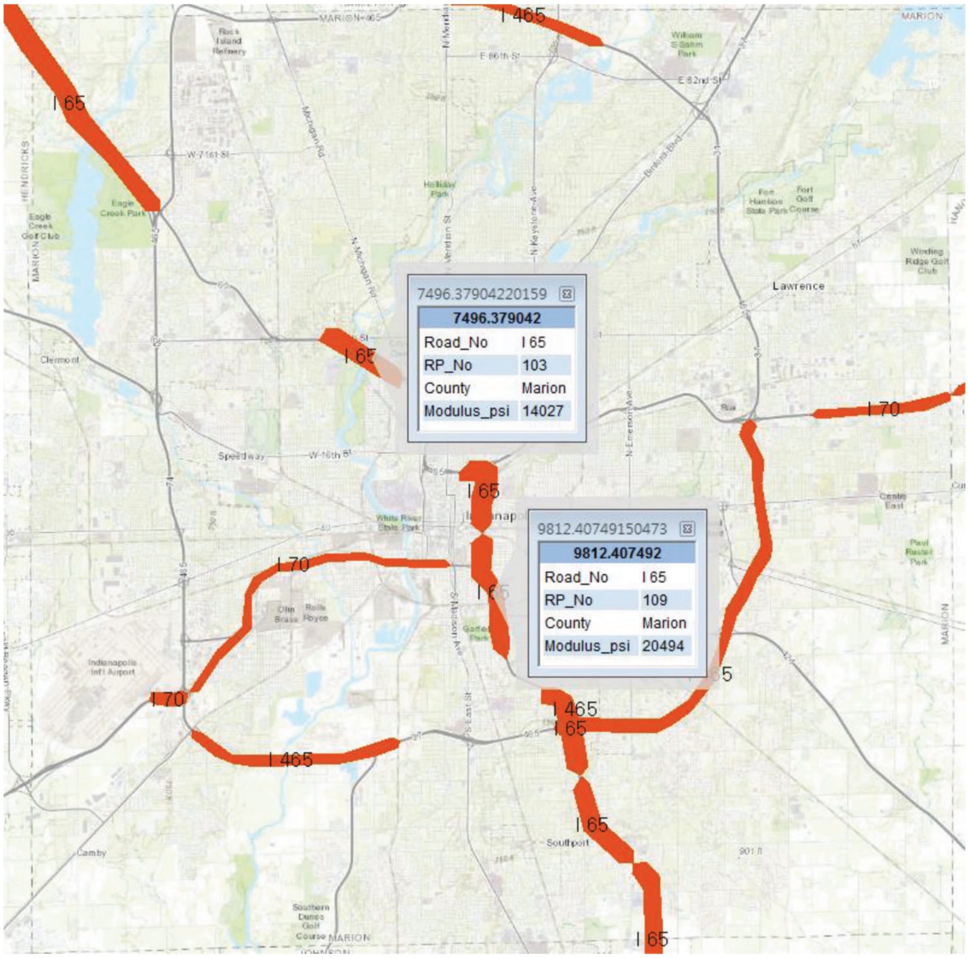
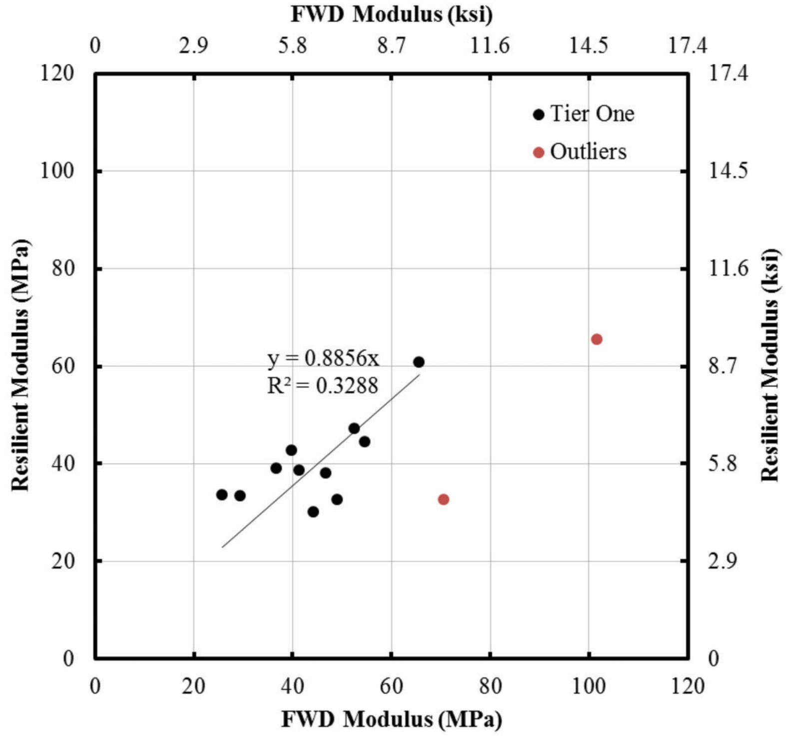
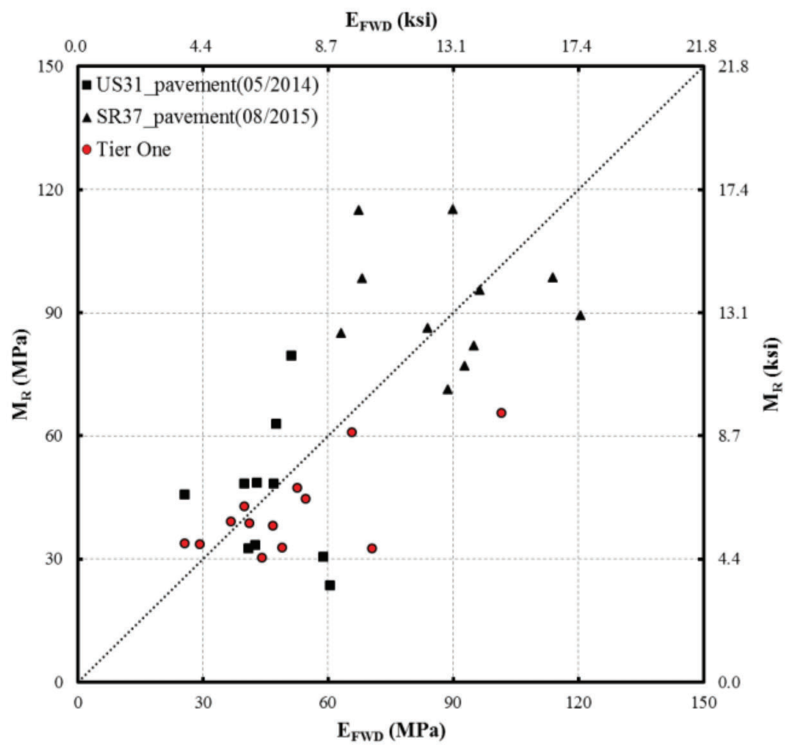


Figure A.6.4 An example of pop-up windows for *FWD* data in Marion County.



(a) Tier one



(b) Tier one vs. phase one results

Figure A.6.5 Comparison between M_R and FWD for tier one data.

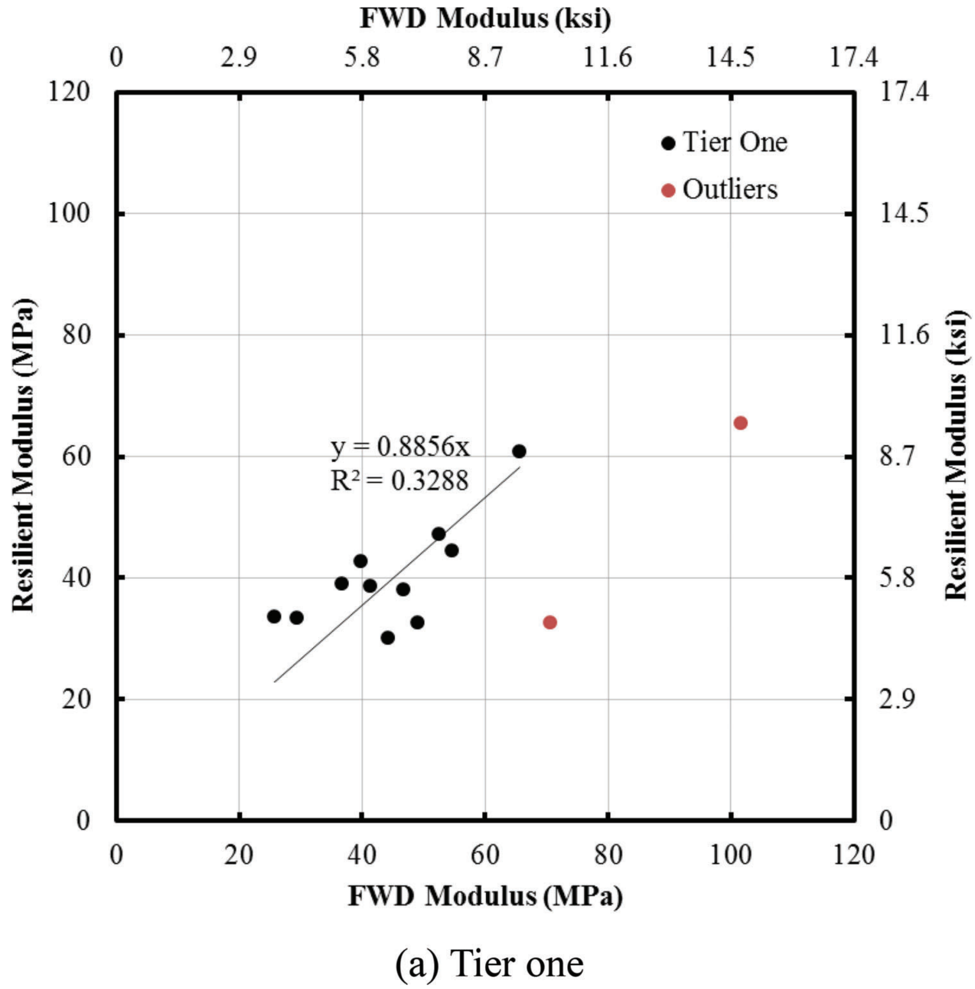


Figure A.6.6 Comparison between M_R and FWD for tier two data.

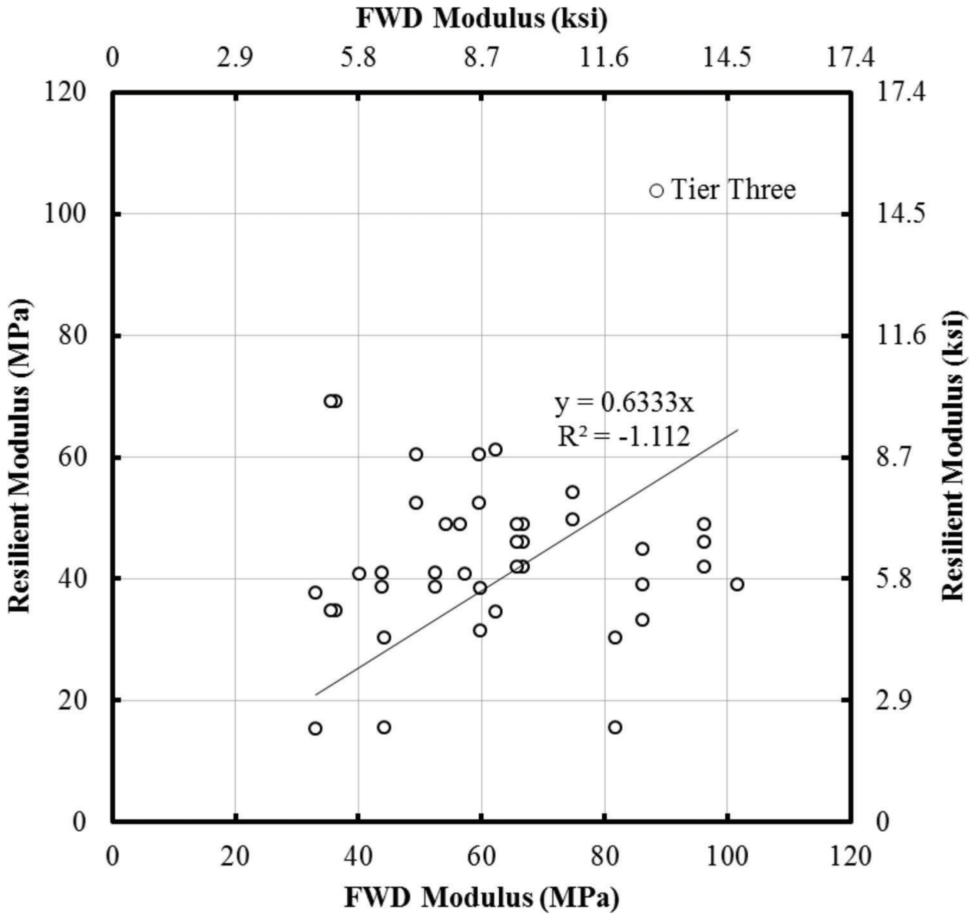


Figure A.6.7 Comparison between M_R and *FWD* for tier three data.

A.7. ANALYSIS OF COLLECTED DATA

Resilient modulus data and *FWD* data have been collected from INDOT databases. Two sets of data were collected: resilient modulus from the Geotechnical Engineering Division and *FWD* from the Pavement Division. The two independent data sets were paired based on location and classified into three tiers.

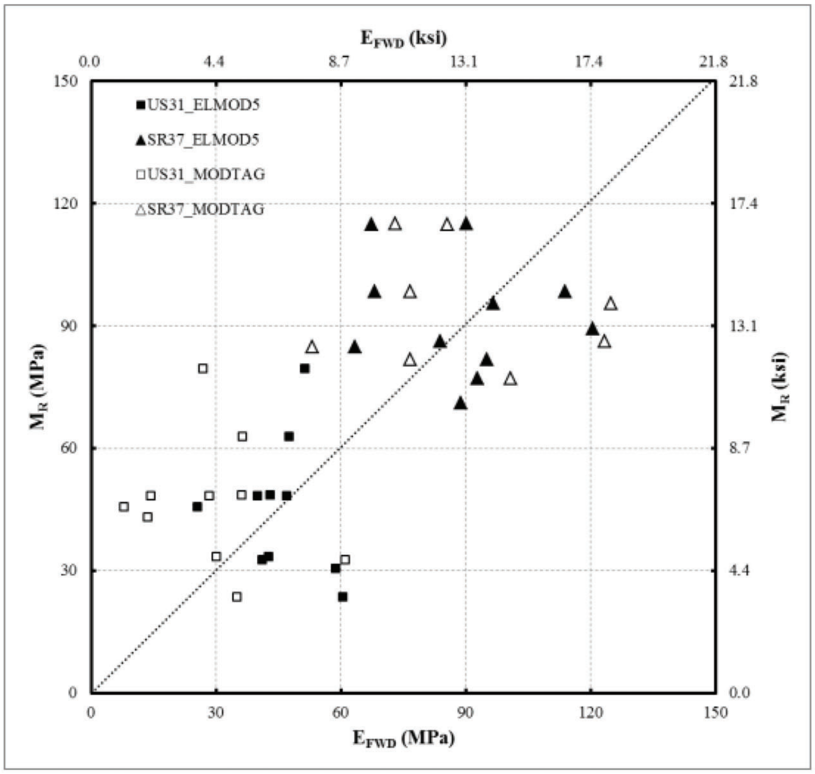
As shown in this section, the data categorized into tier one shows a meaningful correlation between M_R and *FWD*, which supports the results from phase one of the project. Tier two and tier three data do not show strong correlations due to the uncertainty associated with location and type of soil. As a consequence, it was decided to further investigate tier one data only.

In this appendix chapter, correlations between *FWD* and M_R data are discussed.

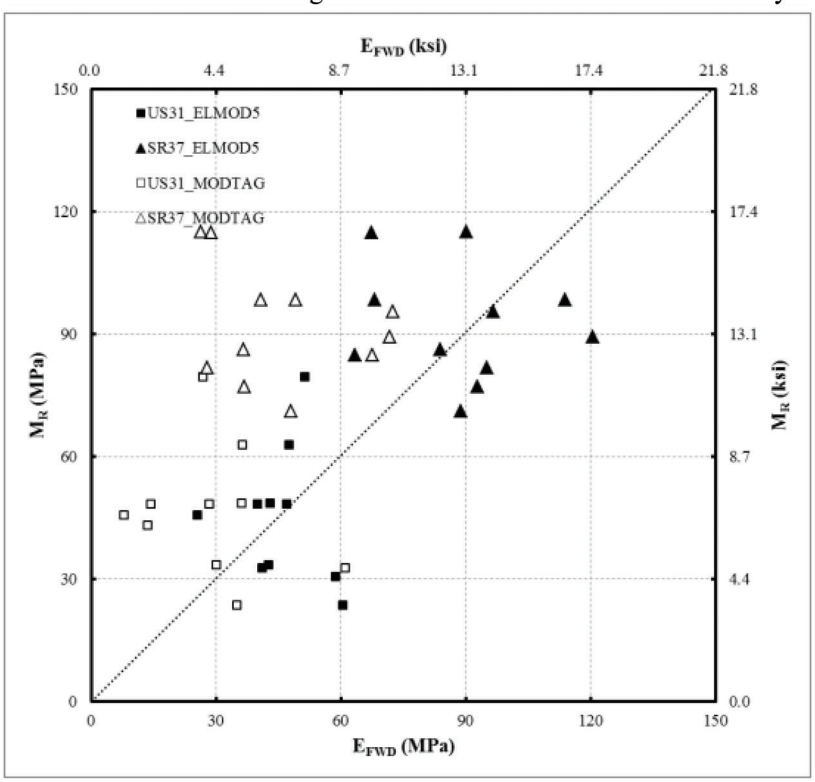
7.1 Analysis of *FWD* Results

ELMOD5, the code used by INDOT to interpret the results from *FWD* tests, uses a three-layer model. This is thought to be a limitation, since pavements are built with several layers, and so more precise interpretation of the *FWD* results could possibly be attained using models that allow for a larger number of pavement layers. In discussions with Dr. Orr, it was decided to use the code MODTAG (Borter & Irwin, 2006), in conjunction with ELMOD. MODTAG is a back-calculation program developed by VDOT (Virginia Department of Transportation) and Cornell University. MODTAG uses an iterative deflection basin fit method that adjusts the moduli of pavement layers until the calculated deflection matches the measured deflection basin. Further, ELMOD5 assumes that the stiffness of the pavement layers decreases with depth, which may or may not be always the case. This limitation does not exist with MODTAG.

First, outcomes from US 31 and SR 37 were analyzed because of the high quality of the data and pavement layer information. Results of a three-layer analysis using MODTAG were compared with the results using ELMOD5, also using a three-layer model. This was done to identify any differences between the two codes when all input parameters and problem definition are identical. The results are shown in Figure A.7.1a. Data points obtained with ELMOD5 are plotted using black symbols and data points using MODTAG are plotted with white symbols. The results from MODTAG are similar to the results from ELMOD5, since both

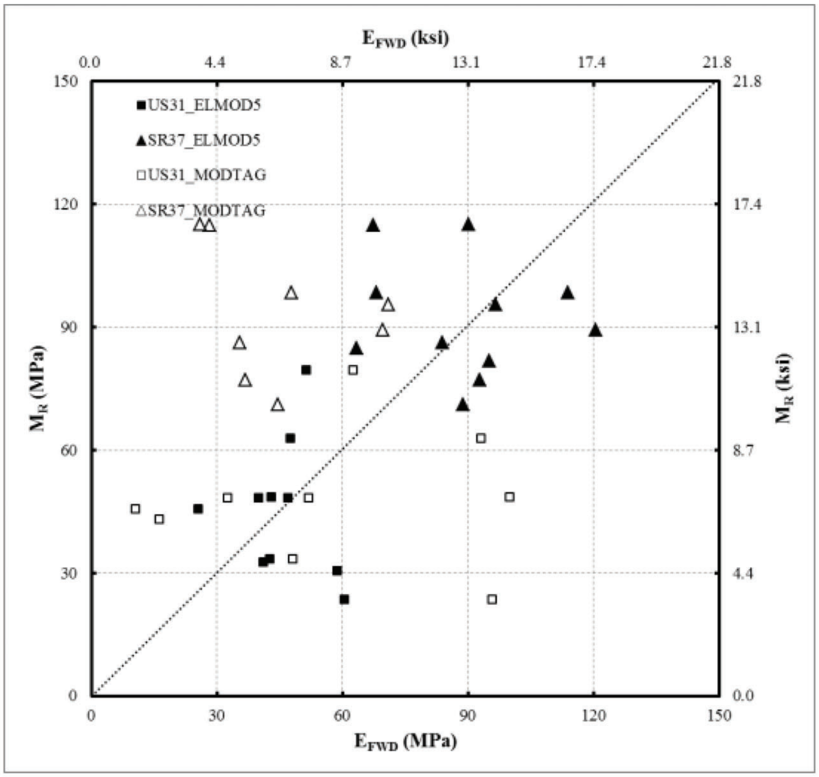


(a) Analysis of US 31 and SR 37 using ELMOD and MODTAG with a three-layer pavement



(b) Analysis of US 31 and SR 37 using MODTAG with a three-layer pavement for US 31 and four-layer for SR 37

Figure A.7.1 MODTAG vs. ELMOD5. Comparison between results from M_R and FWD tests. (Figure continued next page.)



(c) Analysis of US 31 and SR 37 using MODTAG with a four-layer pavement for US 31 and five-layer for SR 37

Figure A.7.1 (Continued)

results are distributed around the 1:1 reference line. Figure A.7.1b shows similar outcomes, but using a three-layer model with MODTAG for US 31 (US 31 has three layers: asphalt surface, base and subgrade) and a four-layer model for SR 37 (SR 37 has four layers: asphalt surface, base, subbase and subgrade). Included in the figure are the results from ELMOD5 to compare with the results from MODTAG. The *FWD* moduli calculated with MODTAG are relatively lower than those calculated with ELMOD5. The M_R or *FWD* modulus using ELMOD5 is roughly 1.6 times higher than the *FWD* results obtained using MODTAG. Additional calculations were done increasing the number of layers. Figure A.7.1c shows the results using MODTAG with a four-layer pavement for US 31, by adding a natural subgrade layer, and a five-layer pavement for SR 37, also by adding a natural subgrade layer.

Figure A.7.1 indicates that by just adding layers to the model, the variability/scatter of the results increase. This is somehow an unexpected result, as a better, more precise description of the pavement, should improve the accuracy and quality of the interpretation. To investigate the issue, manual backcalculation was performed, taking advantage of the expertise of Dr. Orr. The details of the following analysis can be found in Appendix Chapter 9.

For the MODTAG analysis, five-layer models were adopted, i.e., 4″ AC, 6″ granular base, 14″ upper subgrade, 48″ middle subgrade and infinite subgrade for US 31; and 4″ AC, 6″ AC base, 14″ lime treated, 12″ upper subgrade and infinite subgrade for SR 37. The code CHEVLAY (Irwin, 1994a), a multi layered elastic program was used to further investigate the results. CHEVLAY calculates deflections, for the information given such as loads and pavement thickness, based on elasticity theory. The calculated deflections were compared with the actual deflections measured with the *FWD* sensors. Using CHEVLAY, the errors associated with sensors or associated with pavement thickness can be identified. An example of the results obtained from CHEVLAY is shown in Figure A.7.2. The figure shows that the differences between measured and calculated deflections are large for the four sensors close to the center of the test. The differences may be due to errors in pavement thickness or assigned sensors. Identification of the source of the errors was attempted through a parametric analysis where input was systematically changed until the errors were sufficiently small. The thicknesses of the middle subgrade for US 31 and the upper subgrade for SR 37 were decided using CHEVLAY. Note that a sensor at -12 inches from the loading plate was not used for *FWD* backcalculation using MODTAG, since readings from sensors at equal distances from the center can skew the results.

Results from MODTAG and ELMOD5 are compared in Figure A.7.3. Figure A.7.3a is a comparison between MODTAG and ELMOD5, for US 31, and

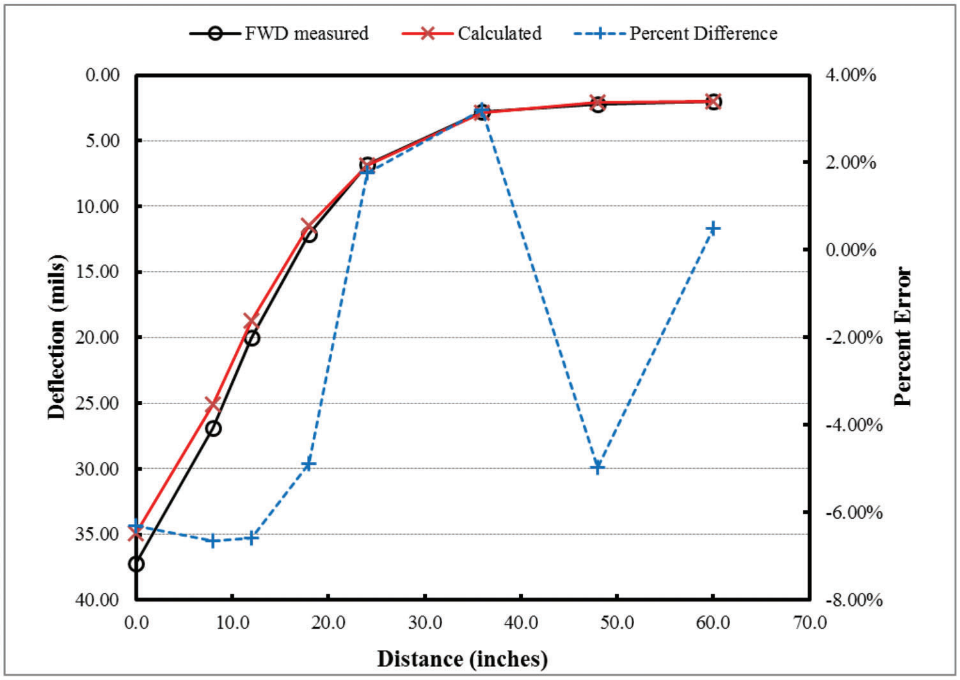


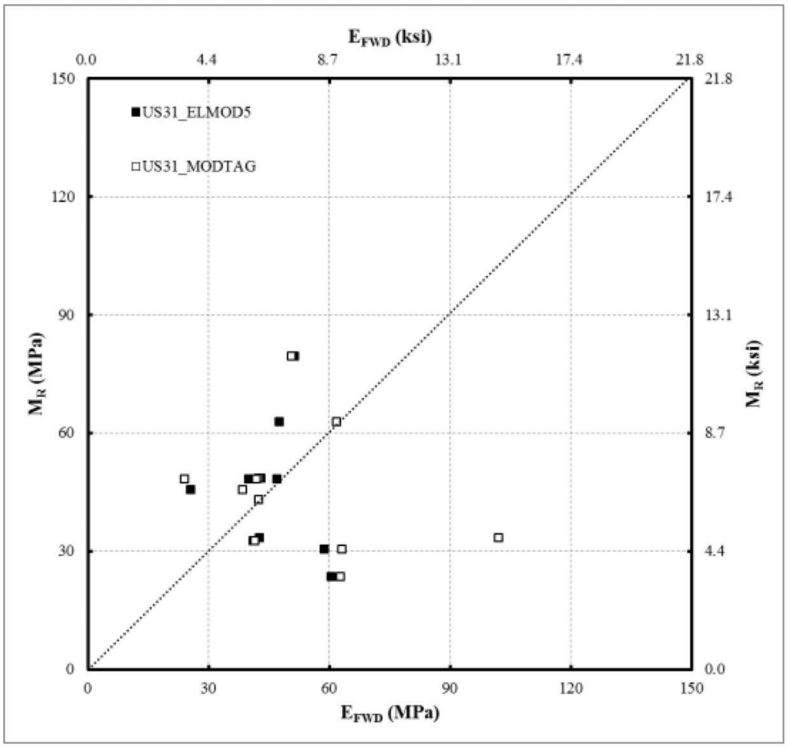
Figure A.7.2 Example of CHEVLAY calculation for US 31.

Figure A.7.3b for SR 37. Data points obtained with ELMOD5 are plotted using black symbols and data points using MODTAG are plotted with white symbols (Figure A.7.3). Utilizing the analysis resources provided by CHEVLAY, US 31 results from MODTAG are in good agreement with results from ELMOD5, except for few outliers, However, there are disparities between the two results for SR 37, as shown in Figure A.7.3b. ELMOD5 results are, generally, 2.2 times higher than MODTAG. The disparity is likely due to errors in pavement thickness or data input; more specifically, the analysis seemed to indicate that the actual thicknesses of the pavement layers might be different that those of design (used by ELMOD5).

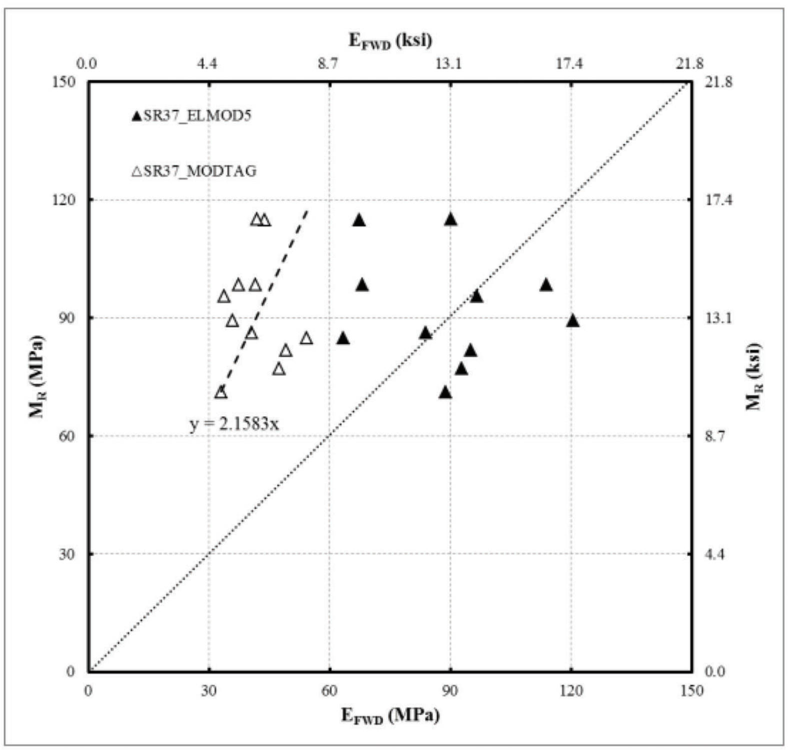
In addition to data collected from US 31 and SR 37, for this project, additional information was gathered, as discussed, from INDOT database. With help from Dr. Jusang Lee, the pavement information for tier one data was obtained from ProjectWise. ProjectWise is a program used to store data from INDOT projects. Unfortunately, not all desired information from the thirteen sites in tier one was available. The pavement layer thickness was found for only two sites, namely SR 60 and SR 46; see Table A.7.1 and Table A.7.2. *FWD* deflection files (.F25) for the two sites were found with the help of Dr. Seonghwan Cho.

Backcalculation of *FWD* results for SR 60 was not successful. The *FWD* raw data file (.F25) for the site might be have been corrupted and thus produced errors during backcalculation.

The analysis of SR 46 was conducted with MODTAG, with a four-layer pavement, i.e., 4.5″ AC, 5.5″ granular base, 14″ upper subgrade, and infinite subgrade. Note that a 4.5″ surface layer was used because the upper two layers were too thin to be differentiated in the calculations. The results from MODTAG are compared with the results from ELMOD in Figure A.7.4. The figure shows that the *FWD* modulus obtained from MODTAG is 3.7 times larger than from ELMOD and that the *FWD* modulus from MODTAG is 5.7 times higher than the M_R obtained in the laboratory. The discrepancies are thought to be associated with inaccurate pavement information such as pavement condition and/or layer thicknesses.



(a) US 31



(b) SR 37

Figure A.7.3 Comparison between results from MODTAG and ELMOD5.

TABLE A.7.1
Pavement layer information from ProjectWise for SR 60

Road	Layer	Material	Thickness (inches)
SR 60	Surface	HMA	3.5
	Base	HMA	6.5
	Subgrade	Type IB	14

TABLE A.7.2
Pavement layer information from ProjectWise for SR 46

Road	Layer	Material	Thickness (inches)
SR 46	Surface	HMA	1.5
	Interface	HMA	3
	Base	HMA	5.5
	Subgrade	Type IB	14

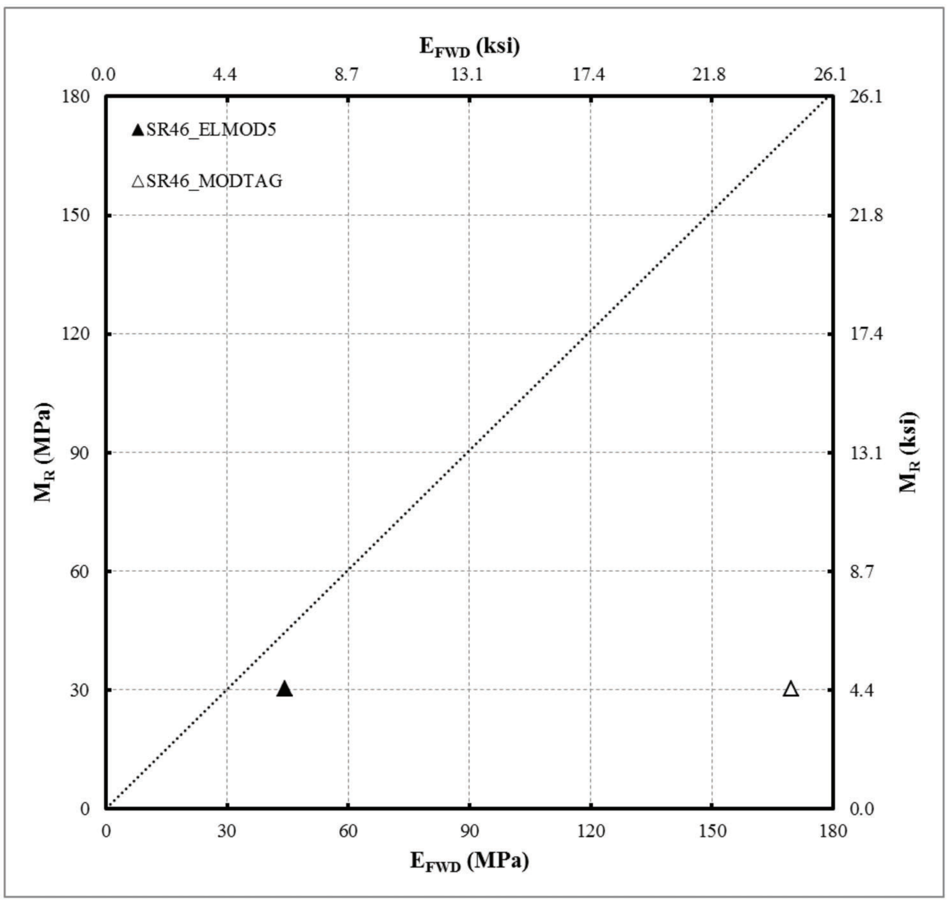


Figure A.7.4 MODTAG vs. ELMOD5. Comparison between results from MODTAG and ELMOD5 for SR 46.

A.8. REVIEW OF DATA FROM INDOT BY DR. DAVID ORR

8.1 Project Scope

The objective of the project is to find a practical solution for INDOT pavement design procedures to effectively determine the soil resilient modulus and propose guidelines for selecting values of soil subgrade stiffness. The proposed work for this study is comprised of four main tasks:

1. Selection of sites to conduct field tests;
2. Field tests at selected sites that include Falling Weight Deflectometer (*FWD*), Lightweight Deflectometer (LWD), and Dynamic Cone Penetration (DCP);
3. Laboratory Resilient Modulus tests of soil samples obtained at the locations where field tests are conducted; and
4. Analysis of data, documentation and final report.

8.2 Outline of Work

This report outlines the work done by David Orr, PE, PhD, Cornell University Local Roads Program (Cornell) to analyze the data provided to Purdue University (Purdue) by the Indiana Department of Transportation (INDOT). During this project, regular conference calls were held between Dr. Antonio Bobet and Sung Soo Park, Purdue, to discuss the project and share results.

Cornell reviewed the data collected by INDOT and Purdue for data quality. The goal was to determine if the relationships between the laboratory and field data collections were valid and the conclusions were supported by the data. In addition, Cornell reviewed the *FWD* testing protocol and the actual data for possible improvements in field testing procedures.

Cornell and Purdue reviewed the reports already provided to INDOT and it was decided to concentrate first on work done on US 31 and SR 37. Cornell reviewed the data as shown below.

There were many issues trying to backcalculate the data, but some correlation could be found.

Towards the end of the project, Purdue was able to obtain data from two additional points. Purdue did backcalculation using ELMOD (REF) and Cornell used MODCOMP (Irwin, 2001). Both programs are widely used and considered very effective if used with proper data and pavement model set up (Koon Meng, 1988; Tam & Brown, 1988).

8.3 US 31

The site was located in Kokomo, Howard County, Indiana. See Figure A.8.1. A total of 11 points were tested using Dynatest *FWD* 8002-222.

The *FWD* testing protocol, according to a review of the data file from the *FWD*, was as shown in Table A.8.1. The actual protocol and target weight is not known.

The *FWD* sensor spacings are 0, -12, 8, 12, 18, 24, 36, 48, and 60 inches.

There was no information on seating drops available.

Figure A.8.1 Photograph of the US 31 site.

TABLE A.8.1
***FWD* set up for testing at US 31**

Drop Height	Target Weight	Number of Drops
1	~6,000	1
2	~9,000	1
3	~12,000	1

TABLE A.8.2
US 31 pavement thickness as provided by Indiana DOT (INDOT)

Pavement Layer	Material	Thickness (inches)
Surface	Asphalt Concrete	4
Base	Unbound Aggregate	6
Subgrade		n.a.

The Long-Term Pavement Performance Study protocol recommends at least 3 seating drops and 3 replicates at each height to reduce the errors due to seating and random error inherent in *FWD* testing (FHWA, 2000). Also, the *FWD* needs to be calibrated on a regular basis using the most current AASHTO Protocol (Irwin, Orr, & Atkins, 2010). INDOT has a proven record of keeping their *FWD*s in good condition and calibrating them on a regular basis.

The pavement structure used was provided to Purdue by INDOT and is shown in Table A.8.2.

Analysis Steps

1. Initial analysis. For this entire project, the analyses were done using MODTAG (Borter & Irwin, 2006) for batch work and MODCOMP (Irwin, 2001) for individual points. Some manual backcalculation was done using CHEVLAY or BISAR as applicable (Irwin, 1994a; Van Cauwelaert, Alexander, White, & Barker, 1988).

For the analysis, sensor 2 was removed (at -12 inches according to file) from *FWD* dataset so there were 8 sensors in the evaluation. Sensor 2 was remove since the sensor was behind the load plate at -12 inches. Another sensor at 12 inches was used in the analysis. The behind sensors may actually induce errors, especially if there is large spatial variability. Any repeated distances typically skew the results. It is recommended that, for asphalt concrete testing, a sensor be at 72 inches rather than -12 inches. The sensor would better be able to detect deep subgrade issues.

The pavement model split the subgrade unto an upper and lower portion. The upper portion is the critical one for comparison as the materials tested in the lab come from this layer, but the pavement engineer needs to review both layers to be sure there is not a compensating layer effect. The upper subgrade is the portion that changes seasonally due to moisture, frost or other climatic effects (Orr & Irwin, 2005). If nothing else is known, the upper subgrade can be set to at least twice the thickness of the layer above or the thickness that would account for the average depth of frost.

2. Set up model with. MODTAG uses an algorithm developed by Irwin to determine the approximate depth to bedrock (Irwin, 2002). If the depth to bedrock is at least 300 inches, then it will have almost no effect on the backcalculation results. For US 31, the depth to bedrock was 300 inches or more and was ignored in the calculations. (See Table A.8.3.)

MODTAG was used to analyze the highest drop height data using a four-layer model. There were several concerns with the four-layer model. A review of the data showed large variation in the surface moduli so BISAR was used to look at possible changes in slip in between the layers in the pavement. BISAR allows the user to set the slip level between the layers. The moduli were manually backcalculated. In the end, the differences in the critical strain due to change slip between the asphalt and base layers only made differences of less than 10% in all but the extreme cases and the most common difference was less than 5%.

A fifth layer was added making the subgrade a 3-layer system. A 5-layer model worked, but had sensitivity issues with some of the layers. MODCOMP assigns the sensors manually, but if the layers are too thin, the assigned sensor may lead to an insensitive layer. Assigning sensors may not resolve the issue, but in this case, the final model manually assigned the sensors to the layers in the model as shown in Table A.8.4.

The final results from MODCOMP backcalculation are shown in Figure A.8.2.

These results, other than the point at station 61, were very similar to the ELMOD data.

8.4 SR 37

A review of SR 37 was not as successful. The SR 37 site is shown in Figure A.8.3. The initial model used the layers as given to Purdue by INDOT and shown in Table A.8.5. The backcalculation results were not adequate with root-mean-square (RMS) errors of over 30% at every station, even using a 6-layer model with 2 layers for the asphalt and 2 for the subgrade. The RMS error for backcalculation should be less than 5% for production data and less than 2% for research quality work (FHWA, 1995, 2000; Irwin, 1994b).

The SLIC transform is an analysis of the *FWD* data that can be quickly used to determine if there are issues with the *FWD* data (Stubstad, Irwin, Lukanen, & Clevenson, 2000). The *FWD* data are plotted in a sigmoid transformation and should plot as a smooth curve in the transformed space. An anomaly can occur due to a bad sensor, a sensor not at the position recorded in the *FWD* data, or due to cracks and defects in the pavement.

TABLE A.8.3
US 31 pavement thickness used in initial model

Pavement Layer	Material	Thickness (inches)	Assumed Poisson's Ratio (μ)
Surface	Asphalt Concrete	4	0.35
Base	Unbound Aggregate	6	0.40
Upper Subgrade		14	0.45
Lower Subgrade		∞	0.45

TABLE A.8.4
US 31 pavement thickness used in 5-layer model

Pavement Layer	Material	Thickness (inches)	Assumed Poisson's Ratio (μ)	Assigned Sensor
Surface	Asphalt Concrete	4	0.35	1
Base	Unbound Aggregate	6	0.40	2
Upper Subgrade		14	0.45	4
Middle Subgrade		24	0.45	6
Lower Subgrade		∞	0.45	8

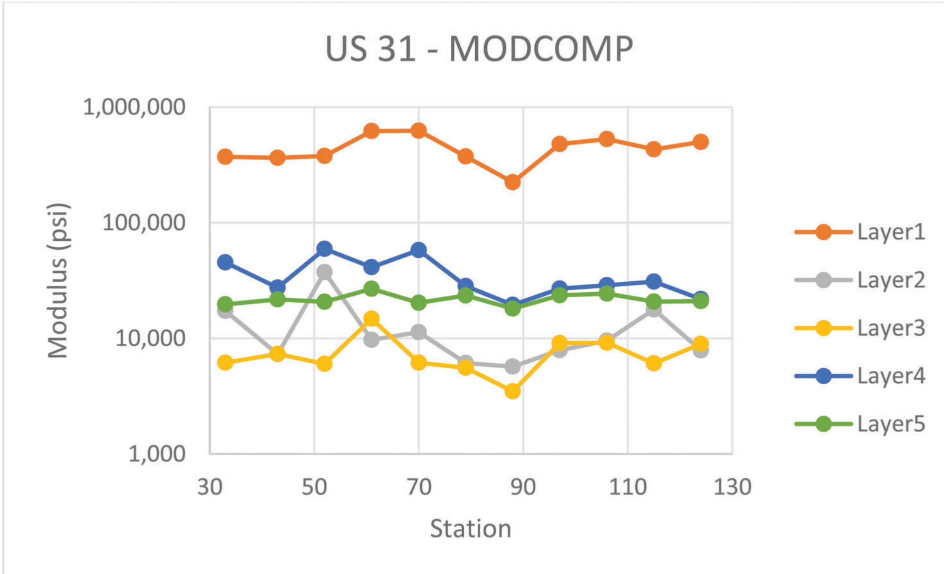


Figure A.8.2 US 31 MODCOMP 5-layer model.

TABLE A.8.5
SR 37 pavement thickness as provided by Indiana DOT (INDOT)

Pavement Layer	Material	Thickness (inches)
Surface	Asphalt Concrete	10
Base	Lime stabilized	14
Subgrade		n.a.

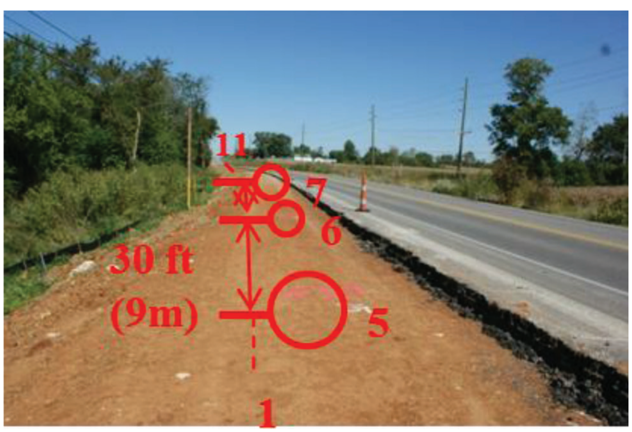


Figure A.8.3 View of SR 37 site.

Figure A.8.4 shows the SLIC transform for the raw data from MODTAG. Note that the software does not supply units, but they are not easily interpretable. The x-axis would be log(radial distance) and the y-axis would be log(-log(deflection of center sensor/sensor deflection)) and have no units.

On initial review, the issue could be sensor 2 or 3 not falling on a nice smooth curve. A more detailed review of the data shows an anomaly that might indicate sensor 2 was out of position by an inch or more. Figure A.8.5 shows the SLIC plot if sensor 2 is at 7 inches rather than at the 8 inches listed in the raw data.

This smother curve illustrates both the power of the SLIC transform and the need to have good QA/QC procedures for testing. It cannot be known what the actual spacing of the sensors is in this case. If the *FWD* operator noted an anomaly in the data while testing, a quick measurement of the sensor spacing could have been done to confirm if the problem was sensor spacing, a possible bad sensor, or just an odd site that due to cracks or subsurface issues does not follow the typical expectation of deflections versus distance.

The model used is shown in Table A.8.6. The pavement analysis was redone assuming sensor 2 was at 7 inches rather than 8 inches. This improved the results some, but still did not provide completely satisfactory answers. The RMS errors were still well above 10%.

Since there were only 11 points, the data was manually backcalculated to see if the values could be improved. Manual backcalculation uses the data from MODCOMP as the starting seed and the same sensor assignments. An INDOT engineer told Purdue, “We do soil treatment if road length is at least 800 ft and width is 8 ft and above. Chemical mod thickness may be either 14 in or 8 [in]. Sub grade Type 1A is not in spec book and we use Type 1B.” The section was reviewed with

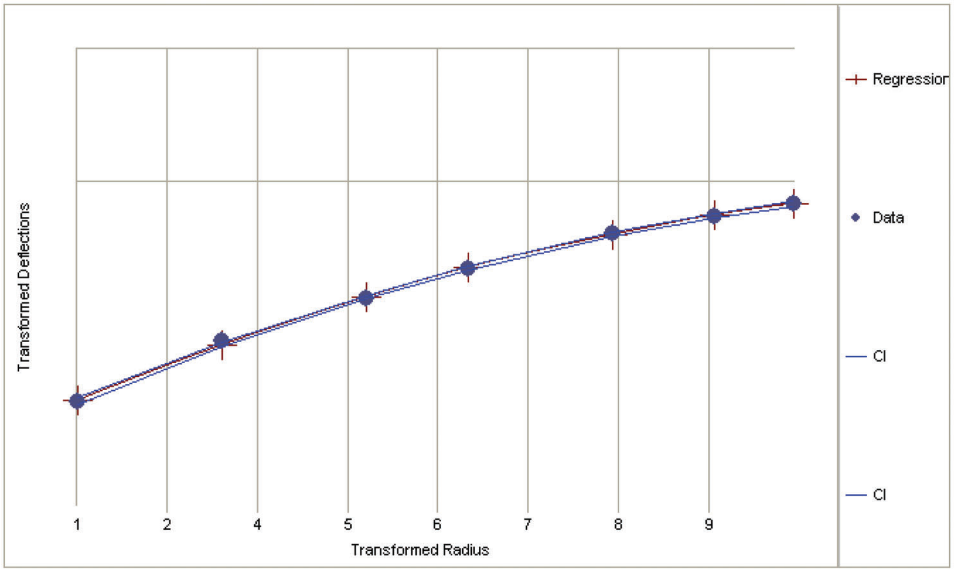


Figure A.8.4 SR 37 SLIC transform plot from MOGTAG.

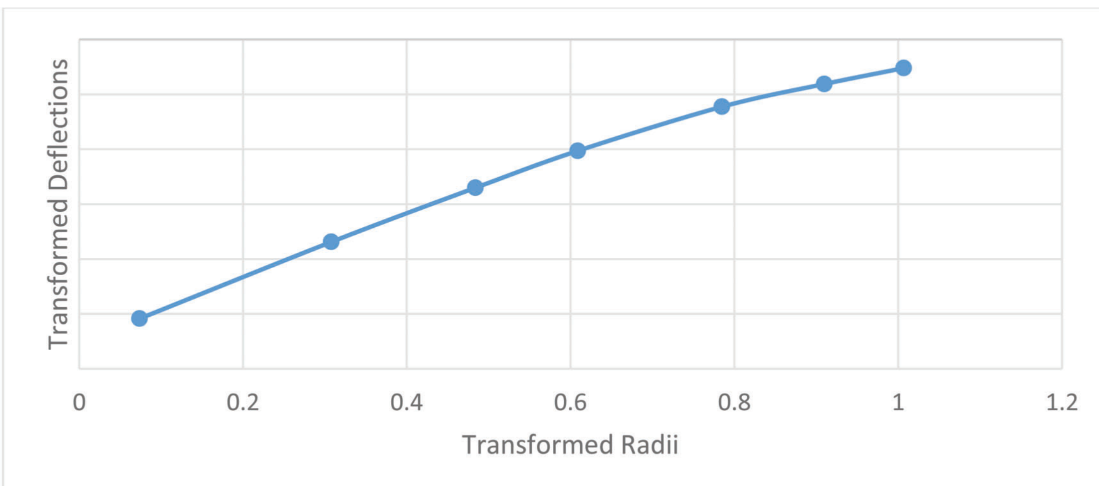


Figure A.8.5 SR 37 SLIC transform with Sensor 2 at 7 inches.

TABLE A.8.6
SR 37 pavement thickness model used in backcalculation

Pavement Layer	Material	Thickness (inches)	Assumed Poisson's Ratio (μ)
1	Asphalt Concrete	4	0.35
2	Asphalt Concrete	6	0.35
3	Lime stabilized	14 or 8	0.35
4	Upper subgrade	12	0.42
5	Lower subgrade	∞	0.45

both a 14 inch and an 8 inch thickness to see the overall effects on the upper subgrade; the layer being compared to the laboratory tests.

Manual backcalculation was more successful with a relatively low RMS error and fairly consistent results as shown in Table A.8.7. Manual backcalculation used

TABLE A.8.7
Manual backcalculation of SR 37 using CHEVLAY

Station	RMSE (%)	Moduli in psi				
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5
				8″ Lime Layer		
0	1.90	285,000	235,000	24,000	39,000	23,000
10	1.78	375,000	140,000	14,000	40,000	25,800
19	1.98	425,000	100,000	8,500	42,000	23,500
28	1.96	485,000	110,000	11,000	47,000	25,000
37	2.58	420,000	110,000	10,000	100,000	25,000
47	2.57	350,000	90,000	11,000	95,000	25,700
55	2.66	370,000	95,000	12,500	67,000	24,000
64	2.07	310,000	102,000	18,000	60,000	24,700
74	3.69	365,000	120,000	25,000	82,000	28,200
83	3.44	308,000	210,000	38,000	72,000	29,000
92	1.62	275,000	110,000	33,500	55,000	21,600
Average	2.4	360,727	129,273	18,682	63,545	25,045
Std. Dev	0.7	64,767	48,299	10,093	21,732	2,151
				14″ Lime Layer		
0	1.59	285,000	235,000	23,000	39,000	22,700
10	1.83	375,000	170,000	16,000	48,000	26,000
19	2.34	350,000	110,000	12,500	35,000	23,700
28	1.85	490,000	105,000	14,500	47,000	25,200
37	4.21	350,000	110,000	15,000	90,000	23,500
47	3.22	350,000	105,000	14,000	95,000	25,250
55	2.45	370,000	95,000	16,500	67,000	23,200
64	1.74	315,000	102,000	21,000	65,000	24,500
74	3.30	355,000	120,000	30,000	70,000	28,100
83	1.80	300,000	180,000	42,000	72,000	28,900
92	2.37	275,000	131,000	35,500	42,000	21,400
Average	2.4	346,818	133,000	21,818	60,909	24,768
Std. Dev	0.8	58,407	43,802	9,867	20,339	2,259

TABLE A.8.8
Critical strains and ESAL lifespan average backcalculated values SR 37

Base Layer Thickness	Strains		Life (million-ESALs)
	Surface	Subgrade	
8″	150.05	-134.47	6.88
14″	144.38	-101.54	8.64

CHEVLAY as the engine and is essentially a non-automated analysis similar to MODCOMP. It is not possible to know why the data would not backcalculate automatically, but the pavement thickness or model inputs are likely to be a culprit.

Also, the average values were put back into the pavement model and the critical strains at the bottom of the asphalt and top of the subgrade were determined using CHEVLAY (Irwin, 1994a). The results are shown in Table A.8.8. The critical strain is in the surface and changes by -3.78% depending upon the thickness of the lime-stabilized base layer. The lifespan in millions of ESALs changes by over 25% (25.6%). Getting the correct thickness in the pavement model is critical.

Layer 4 is the subgrade layer, which is the upper subgrade. The correlation with ELMOD is not great, but this could be a compensating layer effect. Figure A.8.6 shows the moduli of the unbound layers. Note that the lime layer has a much lower value in the manual backcalculation versus the upper subgrade. It is possible the two values are actually going up and down in sympathetic values, making any correlation difficult.

8.5 SR 46

Purdue obtained project level testing from SR 46. Table A.8.9 shows the thickness of the pavement according to INDOT. The *FWD* data was imported into MODTAG and analyzed. Table A.8.10 shows the model used in the backcalculation. Note that the upper two layers of asphalt were combined into a single layer.

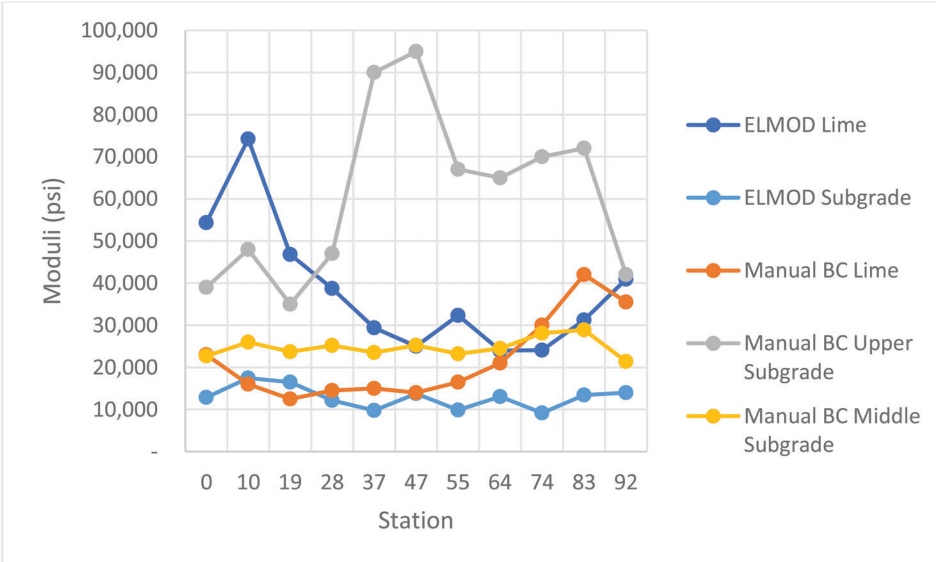


Figure A.8.6 Moduli of unbound layers, SR 37.

TABLE A.8.9
SR 46 pavement thickness as provided by Indiana DOT (INDOT)

Pavement Layer	Material	Thickness (inches)
Surface	Asphalt Concrete	1.5
Surface	Asphalt Concrete	3
Surface	Asphalt Concrete	5.5
Base	Type IB	14
Subgrade		n.a.

TABLE A.8.10
SR 46 pavement thickness model used in backcalculation

Pavement Layer	Material	Thickness (inches)	Assumed Poisson's Ratio (μ)
1	Asphalt Concrete	4.5	0.35
2	Asphalt Concrete	5.5	0.35
3	Lime stabilized	14	0.35
4	Upper subgrade	24	0.42
5	Lower subgrade	Computed to hard bottom	0.45
6	Hard Bottom		0.2

Such thin layers are hard to differentiate in back-calculation.

All of the data were backcalculated even though an anomaly was noted at station 43+05 for the west-bound data set used. The comments in the *FWD* file were reviewed, but the note said no cracks. This kind of anomaly is usually associated with a crack when it only occurs in an isolated point or two. As expected, the results for that point are unacceptable with an RMS error of over 30% (31.08%) and were removed from the data pool for final analysis.

After culling obviously incorrect data, the final results were aggregated for the east and west bound data sets. Incorrect data (see Figure A.8.7) are asphalt concrete moduli above 1,000,000 psi and subgrade moduli above 100,000 psi. Table A.8.11 shows the average data for the WB pavement section.

8.6 Conclusions

Overall, the work shows that when there are good understanding of the site conditions and layer thicknesses, backcalculation (field) matches closely with the resilient module (field). If the data were collected using good quality assurance and quality control (QA/QC), then the backcalculation results

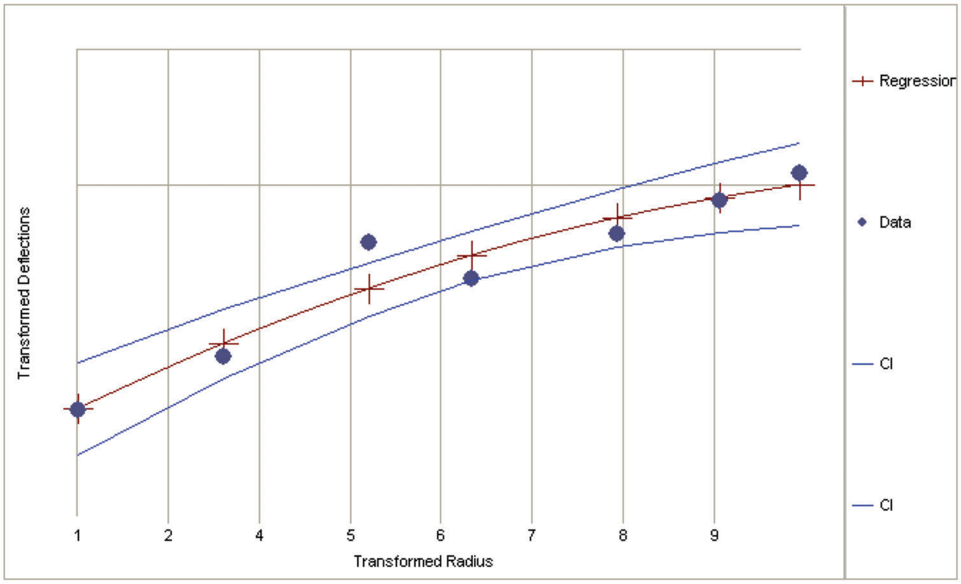


Figure A.8.7 SLIC anomaly at WB Station 43+05.

TABLE A.8.11
Moduli statistics SR 46 westbound

		Moduli in psi					
	RMS	**Layer 1**	**Layer 2**	**Layer 3**	**Layer 4**	**Layer 5**	**Layer 6**
Average	0.979	410,418	265,708	24,923	24,590	14,670	500,000
Std. Dev	0.772	148,221	139,983	22,182	19,453	5,784	—

would be able to be used to determine the modulus for design.

However, there are many issues that need to be reviewed in the testing protocol to make this feasible. First, there needs to be a strong complete *FWD* testing protocol. The LTPP protocol is recommended for research level work, with a small modification for production level testing. (FHWA, 2000) The number of drops is increased from the current INDOT protocol, but the total time of testing is still dominated by the time to move between the stations. If INDOT prefers, there are very good protocols available from other agencies including Colorado, Virginia, and Federal Lands Highways (Hossain, Bendana, & Yang, 1995).

A recommended *FWD* setup and testing protocol is listed below.

FWD *Equipment Setup*

This setup assumes a DYNATEST *FWD* with nine geophones. Setup of the *FWD* equipment for testing consists of setting up: the geophones at the proper distances, the weight drop heights to obtain the target load levels, and the computer software to obtain the data in the proper format and perform needed data checks.

Computer Software

The data obtained uses the following units and formats for the data.

Units – metric
Distance – meters
Pressure – kPa
Load – kN
Deflections – microns (μ)

Research data must be valid and accurate. For the deflection data, the *FWD* software uses up to five quality control checks as the data are collected. The checks to be used are listed below. Details on handling data failing any quality control checks are described in the LTPP Manual for Falling Weight Deflectometer Measurements: Operational Field Guidelines (FHWA, 2000).

- **Roll-off (Roll Off)** – Enabled or Smart
- **Decreasing (Decrease)** – Enabled or Smart
- **Out of Range (Overflow)** – Enabled or Smart
- **Load Variation (Repeatability)** – Enabled or Smart
  The tolerance range for load is set at:

$$X \pm (0.18\ \text{kN} + 0.02X) \text{ or } X \pm (40\ \text{lbs} + 0.02X)$$

- **Deflection Variation (Repeatability)** – Enabled or Smart
  The tolerance range for deflections is set at:

$$X \pm (2\ \text{microns} + 0.01X) \text{ or } X \pm (0.08\ \text{mils} + 0.01X)$$

where X = average ***normalized*** deflection for all drops at that height

TABLE A.8.12
***FWD* load configuration**

Height	Target Load (pounds ± 10%)
1	6,000
2	9,000
3	12,000
4	16,000

TABLE A.8.13
***FWD* loading sequence**

Height	Number of Drops (Research)	Number of Drops (Production)	Store Peak Deflection
3	3	3	No
1	4	0	Yes
2	4	3	Yes
3	4	3	Yes
4	4	3	Yes (store time history for last drop in set with research data)

In addition to the data checks, the following options were set within the field software.

- **Sampling window** – 60 mSec
- **Smoothing** – off
- **Preserve temperatures** – off
- **Automated prompts** – Surface temperature (if no infrared device on *FWD*)

Deflection Sensor Spacing

Deflection sensors are placed at the following spacing in U.S. Customary units close to the measurements:

0, 8, 12, 18, 24, 36, 48, 60, 72 (all distances in inches).

Drop Heights

The drop heights are set up to produce the loads shown in Table A.8.12 when the *FWD* is properly warmed-up. The *FWD* should be warmed-up at a location outside the experimental section.

Prior to performing *FWD* measurements on a test section, the load levels from the drop height setting are verified as part of warm up.

Loading Sequence

The loading sequence consist of a total of seventeen drops for research and twelve for production level work. The first three drops are seating drops from drop height 3 and are not stored. The complete load-deflection time history (60 m-sec) is recorded for the last drop at the forth drop height for research work. Table A.8.13 summarizes the loading sequence.

These drops are needed to test for linearity in the subgrade soil and reduce overall error by performing extra drops at each drop height.

FWD *Data and Backup Procedures*

FWD operators also have the responsibility to safeguard the *FWD* data files by keeping copies of the data in more than one location. All deflection data files should be backed up before leaving the site.

Layer Thickness

Layer thickness is the other field issue that should be resolved to obtained high quality data. Using construction plans and overall specifications is not enough. Error due to failure to have proper thickness can exceed 30% in some cases. (Irwin, Yang, & Stubstad, 1988) Generally, the maximum error in the thickness should be about one-quarter the thickness of the layer; ±1 inch for asphalt layers, ±3 inches for unbound base layers, and ±6 inches for upper subgrade layers. The less error in thickness, the less error in the backcalculation analysis.

If good as-built drawings showing thickness are available, then they may be used. If not, ground penetrating radar (GPR) tests of the project with cores or test pits to provide ground truth are recommended.

A.9. CONCLUSIONS AND IMPLEMENTATION

Since INDOT adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) at the beginning of 2009, obtaining accurate and representative values of the resilient modulus needed for the design has proven to be difficult. This is the particularly the case when designing the reconstruction of the pavement of existing roads. The reason is the need to direct sampling of the subgrade soil and access to the equipment required to perform the resilient modulus tests in the laboratory following the standard AASHTO T-307-99 (2007). The problem is compounded by the length of the project, as it requires a large number of representative soil samples. An alternative that would be efficient and cost effective is to obtain the resilient modulus from indirect, non-destructive tests. The goal of the project is to assess the potential of the following tests to estimate the resilient modulus of the subgrade: Falling Weight Deflectometer, *FWD*, Light Weight Deflectometer, LWD, and Dynamic Cone Penetrometer, DCP.

The following types of subgrade were specifically targeted for the project: untreated soils type A-6 and A-7-6. This objective has proven challenging; first, because these soils are usually chemically treated to improve their stability and engineering properties and so it has not been easy to identify the right project; and second, because the actual type of soil placed in-situ may not fit into these categories. In addition, coordination with the job contractor, subcontractor, and technical personnel from INDOT and others to access the site and perform all the tests at the same time has been challenging. In other occasions, the weather or equipment availability or equipment trouble have delayed the work. Fortunately, four sites had been available for testing, thanks to the work and help of INDOT personnel. The first site was on US 31 around Kokomo, Indiana. The soil is classified as A-4, according to AASHTO, with 58% passing No. 200 sieve and *PI*, Plastic Index, 8.5%. The second site was on SR 37 around Paoli, Indiana. The soil is defined as A-7-6, with 88% of soil passing the No. 200 sieve and $PI = 23.8\%$. The third site was on SR 641 at Terre Haute. The soil is classified as A-6 according to AASHTO with 89% passing #200 sieve and $PI = 20.2\%$. The last site was Ramp line A connecting SR 641 and SR 46 at Terre Haute. The soil has 72% of passing #200, and 30.6% *PI*, so it is classified as A-6.

FWD, *LWD*, and *DCP* tests were performed on the four selected sites. A representative 90 m long section at each site was chosen. In each section, eleven points at 9 m intervals were identified to run the three tests. After pavement construction, *FWD* tests were conducted on US 31 and SR 37. In addition, at each of the eleven points on each site, in-situ water content, optimum moisture content, maximum dry unit weight, granulometry, Atterberg limits and resilient modulus tests were performed.

The scope of the project was expanded to further investigate relations between field *FWD* and laboratory resilient modulus tests using the data repository of INDOT to obtain additional geotechnical and pavement information. The ARC GIS program was used to visualize M_R and *FWD* data so that the two independent data sets were paired. The data collected was classified into three tier categories, based on the degree of uncertainty associated with the data, which originated mostly from difficulties in determining whether the data paired originated at the same location. Tier one data, having the highest confidence in how the data was paired, showed good agreement between *FWD* and M_R, similar to the results from the first phase of the project. Tier two and three data did not show a strong correlation due to the higher uncertainty associated with how the data points were paired.

Based on the results from all the field and laboratory tests, the following conclusions can be reached:

1. The subgrade modulus obtained from *FWD* tests conducted on top of the pavement compares very well with the resilient modulus of the subgrade, i.e. $M_R = E_{FWD}$.
2. Results from *FWD* tests conducted directly on top of the subgrade are not reliable, likely due to the lack of confinement of the soil.
3. The stiffness obtained from *LWD* tests performed on top of the subgrade does not compare well with the resilient modulus of the soil obtained in the laboratory. The values obtained from *LWD* are too low.
4. There is not a good relation between the soil stiffness obtained from *DCP* and from the laboratory using the correlation by Salgado and Yoon (2003), which was deemed appropriate in this study.
5. While *LWD* and *DCP* have not provided acceptable estimates of soil stiffness, they can be used to estimate quality consistency of the subgrade. The research has shown that the field measurements using either method are sensitive to the quality of the construction and can be used to identify those areas with lower quality than others.
6. When good quality *FWD* data is obtained, its results, in terms of stiffness of the subgrade, can be used to estimate the resilient modulus, M_R, of the subgrade. If the data were collected using good quality assurance and quality control (QA/QC), then the backcalculation results would be able to be used to determine the modulus for design.
7. Good quality *FWD* data requires a strong complete *FWD* testing protocol. The LTPP protocol is recommended for research level work, with a small modification for production level testing. There are however very good protocols available that INDOT could explore for their use.
8. Good quality *FWD* data can only be achieved when pavement layer thickness is accurate. Using construction plans and overall specifications may not be sufficient. If good as-built drawings showing thickness are available, then they may be used. If not, ground penetrating radar (GPR) or other non-destructive tests may be performed in conjunction with the *FWD* tests to determine the geometry of the pavement.
9. The correlations proposed between *FWD* and M_R are based on limited, yet highly reliable, data. It would be desirable to extend the database used in the project to further confirm such an important conclusion. This could be done by identifying sites under construction where a campaign of tests similar to those completed under phase one of the project could be conducted.

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On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

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