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Analysis of Repeated Network-Level Testing by the Falling Weight Deflectometer on I-81 in the Virginia Department of Transportation's Bristol District

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Final Report VTRC 17-R6

VIRGINIA TRANSPORTATION RESEARCH COUNCIL
530 Edgemont Road, Charlottesville, VA 22903-2454
www.VTRC.net

Standard Title Page - Report on Federally Funded Project

1. Report No.: FHWA/VTRC 17-R6	2. Government Accession No.:	3. Recipient's Catalog No.:
4. Title and Subtitle: Analysis of Repeated Network-Level Testing by the Falling Weight Deflectometer on I-81 in the Virginia Department of Transportation's Bristol District		5. Report Date: November 2016 6. Performing Organization Code:
7. Author(s): James M. Bryce, Ph.D., Samer W. Katicha, Ph.D., Brian K. Diefenderfer, Ph.D., P.E., And Gerardo W. Flintsch, Ph.D., P.E.		8. Performing Organization Report No.: VTRC 17-R6
9. Performing Organization and Address: Virginia Tech Transportation Institute 3500 Transportation Research Plaza Blacksburg, VA 24061 Virginia Transportation Research Council 530 Edgemont Road Charlottesville, VA 22903		10. Work Unit No. (TRAIS): 11. Contract or Grant No.: 100985
12. Sponsoring Agencies' Name and Address: Virginia Department of Transportation 1401 E. Broad Street Richmond, VA 23219 Federal Highway Administration 400 North 8th Street, Room 750 Richmond, VA 23219-4825		13. Type of Report and Period Covered: Final Contract 14. Sponsoring Agency Code:
15. Supplementary Notes:		

16. Abstract:

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The objective of this study was to analyze the results from the 2011 testing and compare them to the results obtained from the 2006 study to determine if the previously completed FWD survey of VDOT's entire interstate network needed to be repeated. First, deflection values that were obtained from pavement segments that received treatments between the two sets of tests were identified and omitted from any comparison. Second, the two datasets were compared directly (i.e., without accounting for errors) and were modeled to account for the expected errors in the data defined as the root mean square of the difference between 2006 and 2011 measurements.

The results of the 2011 testing showed lesser deflection and greater structural number values when compared to the data collected in 2006. A characterization of the errors implicit in each set of measurement showed that the errors outweigh the changes in deflection values from the two datasets. Therefore, it was not possible to quantify a recommended time between subsequent rounds of deflection testing on the pavement network. Since the literature shows significant benefits to conducting pavement deflection testing on the network, VDOT will continue this practice based on local needs and as budgetary constraints allow.

17 Key Words: Pavement; Deflection; Falling Weight Deflectometer (FWD); Interstate		18. Distribution Statement: No restrictions. This document is available to the public through NTIS, Springfield, VA 22161.	
19. Security Classif. (of this report): Unclassified	20. Security Classif. (of this page): Unclassified	21. No. of Pages: 37	22. Price:

FINAL REPORT

ANALYSIS OF REPEATED NETWORK-LEVEL TESTING BY THE FALLING WEIGHT DEFLECTOMETER ON I-81 IN THE VIRGINIA DEPARTMENT OF TRANSPORTATION'S BRISTOL DISTRICT

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In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

Charlottesville, Virginia

November 2016
VTRC 17-R6

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Each contract report is peer reviewed and accepted for publication by staff of Virginia Transportation Research Council with expertise in related technical areas. Final editing and proofreading of the report are performed by the contractor.

ABSTRACT

This study was undertaken in an effort to determine the required time between subsequent rounds of network-level pavement deflection testing using a falling weight deflectometer (FWD) on the Virginia Department of Transportation's (VDOT's) interstate system. Network-level deflection testing was conducted in two separate years (2006 and 2011) on Interstate 81 in VDOT's Bristol District. The testing was conducted using the FWD at an interval of 0.2 miles in the right-hand lane (travel lane) of the interstate.

The objective of this study was to analyze the results from the 2011 testing and compare them to the results obtained from the 2006 study to determine if the previously completed FWD survey of VDOT's entire interstate network needed to be repeated. First, deflection values that were obtained from pavement segments that received treatments between the two sets of tests were identified and omitted from any comparison. Second, the two datasets were compared directly (i.e., without accounting for errors) and were modeled to account for the expected errors in the data defined as the root mean square of the difference between 2006 and 2011 measurements.

The results of the 2011 testing showed lesser deflection and greater structural number values when compared to the data collected in 2006. A characterization of the errors implicit in each set of measurement showed that the errors outweigh the changes in deflection values from the two datasets. Therefore, it was not possible to quantify a recommended time between subsequent rounds of deflection testing on the pavement network. Since the literature shows significant benefits to conducting pavement deflection testing on the network, VDOT will continue this practice based on local needs and as budgetary constraints allow.

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INTRODUCTION

The Virginia Department of Transportation (VDOT) uses the results of automated video distress surveys to assist in developing maintenance priorities to manage pavements on Virginia's interstate and primary roadways. Totaling nearly 27,000 lane-miles, these roadways consist of flexible, rigid, and composite (flexible over rigid) pavements. The video-based surface distress data consist of quantities of distresses that are visually observable at the pavement surface. Currently, VDOT's Maintenance Division determines typical maintenance treatments and calculates average costs based on a combination of the structural design data (e.g., pavement structural number) and visual condition indices. It is from this process that a needs-based budget is developed. While these values are transformed into a condition index, they do not provide information regarding changes in structural capacity of the pavement system on a network level.

Zaghloul (1998) stated that including falling weight deflectometer (FWD) testing in pavement rehabilitation decision-making could yield significant cost savings for a highway agency. While the FWD has been widely used for project-level structural assessment of pavements, only a few agencies have implemented or considered implementing a network-level structural survey using the FWD (Scullion, 1988; Zaghloul et al., 1998; Hossain et al., 2000; Noureldin et al., 2005; Diefenderfer, 2008; Crook et al., 2012).

The primary drawbacks to using the FWD for network-level surveys are that the testing is time-consuming and the FWD device must be stationary during the test (Flintsch et al., 2012). Production rates ranging from 80 to 120 test points per day are achievable, and lane mileage covered depends on the test spacing employed. Diefenderfer (2008) found that typical production rates could cover approximately 30 lane-miles per day at a 0.2-mile test spacing.

Additionally, if an agency wishes to pursue FWD testing on a network basis, there is no consensus on the time interval between test cycles. It was found that the temporal frequency for repeated FWD tests was generally based on budgetary constraints or management and policy decisions. Stubstad et al. (2012) stated that structural testing on flexible and rigid pavement networks can be repeated on 5-year and 10-year cycles, respectively, based on analysis of LTPP data. Noureldin et al. (2005) suggested that the Indiana DOT test approximately 20% of their network each year. Hossain et al. (2000) stated for pavements similar to those tested in Kansas, an interval between test cycles of up to 3 years was acceptable. Damnjanovic and Zhang (2006) stated that 25% of those highways in Texas selected for network-level structural testing could be tested each year, resulting in a 4-year test cycle. Crook et al. (2012) summarized a literature review and telephone survey and stated that testing could be repeated every 3 to 5 years. Carvalho et al. (2012) stated that often the agency budget determines the amount of FWD testing conducted because of many factors they described. Thus, the optimum timing between subsequent rounds of network FWD testing has not been quantified by previous studies.

PURPOSE AND SCOPE

Two previous studies by VDOT used the FWD to collect structural capacity data of Virginia's entire interstate system between 2005 and 2008 (Galal et al., 2007; Diefenderfer, 2008). Following these studies, it was unclear if, or when, subsequent testing would be needed. Rather than conducting a second round on the entire interstate system, the purpose of this current project was to conduct deflection testing using the FWD in VDOT's Bristol District and specifically within the right lane of northbound and southbound I-81. These newer data were compared to the original data set to determine if additional testing would be needed statewide.

METHODS

Data Collection

Testing was performed using a Dynatest model 8000 FWD in the travel (right-hand) lane of the roadway. The FWD load plate was located in the right wheel path during testing. The FWD was equipped with 9 deflection sensors at radial distances of 0, 8, 12, 18, 24, 36, 48, 60, and 72 in from the center of the load plate. Testing was conducted at 0.2-mile intervals and at three load levels (9,000; 12,000; and 16,000 lbf). At each load level, two deflection basins (i.e., the deflection response measured by the array of sensors from a single load application) were recorded. Pavements of similar thickness (that is, having a thickness variation of less than 2 inches) were grouped together for computational efficiency. Additional details of the test procedure and data analysis can be found in Diefenderfer (2008). The FWD was calibrated according to its standard calibration procedure and schedule.

Analysis

The analysis was conducted by removing potential data outliers and results from those sections that had received maintenance between the two deflection test cycles and then the agreement between the two datasets was assessed using regression techniques. A test for outliers was conducted to remove those individual observations from the 2006 and 2011 datasets that were found to deviate extremely from the expected condition. As a "test of reasonableness," the data were checked for data points in which the individual data value was outside a range defined by the mean plus and minus 3.49 standard deviations. Any data values outside this range were removed from analysis.

Linear Regression

First, a linear regression was conducted along with an evaluation of the sources of error within the dataset, wherein the shortcoming of applying traditional regression to these data was demonstrated. Linear regression is a method for comparing two sets of data that is typically used in engineering practice, but the results are often misleading when assessing the agreement between datasets (de León Izeppi et al., 2012). Second, a model that takes into account errors in the measurements from both years was evaluated, and orthogonal regression was used to analyze the datasets. Measurement errors where defined based on the standard deviation of the difference between the 2006 and 2011 measurements. In addition, a discussion of potential sources of errors and the impacts of the distributions of the errors on calculating SN and M_R was developed.

Assessing Agreement Between Survey Years

Analyzing the agreement between the 2006 and 2011 data is critical when comparing the two rounds of testing. This is because the data are expected to contain a certain level of error, and this error may lead to a misinterpretation of the repeatability of the values calculated from deflection testing (particularly the SN and M_R). For example, if two sets of measurements (*A* and *B*) are taken at a location that has a true value of *a* and the measurements contain a normally distributed error with mean zero and standard deviation σ [$\varepsilon \sim N(0, \sigma)$], then the data would take the form $A = a + \varepsilon_A,\ B = a + \varepsilon_B$. It is assumed that the error standard deviation (i.e. accuracy) is the same for 2006 and 2011 measurements (there is no indication to assume one set of measurements was more accurate than the other and the natural assumption is to consider both set of measurements having the same quality); with this assumption, the variance of the difference between the two sets of measurements is equal to two times the variance of the measurement error [$Var(A-B) = Var(\varepsilon_A - \varepsilon_B) = 2Var(\varepsilon_A) = 2Var(\varepsilon_B) = 2\sigma^2$]. The reason it is important to evaluate the measurement error standard deviation is to be able to develop a relationship between the two sets of measurements. Traditional linear regression performed with least squares makes an (often forgotten) implicit assumption that the independent variable is measured with no error (or at least an error that is much smaller than the error in the dependent variable and therefore can be neglected). In the context of the two sets of FWD measurements analyzed in this study, performing a linear regression with 2006 measurements as the independent, *x*, variable, and 2011 measurements as the dependent, *y*, variable, is equivalent to saying that 2006 measurements are not affected by any error (they are the ground truth) and all observed differences between the 2011 and 2006 sets of measurements are due to error in 2011 measurements (the same would be implied, but with all error assigned to 2006 measurements, if the sets of measurements are switched in the analysis). Furthermore, using linear regression, the relationship between the 2006 and 2011 sets of measurements will be different depending on how the data are analyzed (i.e. which variable is considered dependent or independent). The researchers think that arbitrarily assigning all of the error to one of the two sets is not a realistic assumption (furthermore, there is no rational way to decide to which data set the error should be assigned). A more realistic assumption is to consider the same level of error in both sets of measurements. In this case, orthogonal regression can be used to determine a unique relationship between the two sets of measurements. The comparison between linear regression and orthogonal regression is further discussed with examples later in the report.

RESULTS AND DISCUSSION

Data Collection

VDOT's FWD was used to collect structural capacity data on southbound Interstate 81 in the Bristol District during July 2011 and on northbound Interstate 81 in the Bristol District between June and July of 2011. Following this testing, the FWD device became inoperable and additional testing was not possible with the same device. In addition, it was deemed too cost-prohibitive at the time to attempt additional network testing via a third party vendor. Thus, only data from I-81 were collected for this study, as shown in Table 1.

Table 1. Summary of Test Locations, I-81, Bristol District

County	Direction	Miles	No. of Data Points	
			2006	2011
Wythe	Southbound	29.85	143	148
Smyth	Southbound	23.23	117	118
Washington	Southbound	34.67	174	171
Wythe	Northbound	29.77	144	147
Smyth	Northbound	23.38	119	117
Washington	Northbound	34.30	173	172

Analysis

Following the analysis processes used in Diefenderfer (2008), the collected data were processed and the pavement effective structural number (SN_{eff}) and subgrade resilient modulus (M_R) were the primary outputs. These collected data are referred to as *2011 data*, and the data collected in September 2006 on the same routes are referred to as *2006 data*. Tables 2 and 3 show a summary of the collected data.

Table 2. Data Summary, 2006 Data

County	SN_{eff} (Northbound; Southbound)		M_R, psi (Northbound; Southbound)	
	Average	Standard Deviation	Average	Standard Deviation
Wythe	6.89; 6.76	0.58; 0.60	12,826; 12,898	4,824; 5,003
Smyth	7.06; 7.12	0.94; 0.98	16,051; 14,933	6,116; 6,653
Washington	7.84; 7.83	1.20; 0.99	14,241; 13,708	5,736; 6,253

Table 3. Data Summary, 2011 Data

County	SN_{eff} (Northbound; Southbound)		M_R, psi (Northbound; Southbound)	
	Average	Standard Deviation	Average	Standard Deviation
Wythe	7.63; 7.31	0.64; 1.12	13,773; 14,598	4,436; 5,921
Smyth	7.62; 7.48	0.83; 0.92	16,164; 16,428	5,834; 7,065
Washington	8.32; 8.68	1.48; 1.32	14,793; 14,534	7,367; 6,161

From Tables 2 and 3 it can be seen that the average SN_{eff} and M_R for all three counties increased slightly between 2006 and 2011 in both directions. Closer inspection of the data showed two situations that were thought to adversely affect the comparison of the 2006 and the 2011 data. The first situation included the presence of outliers in the data. Visual inspection showed that some SN_{eff} and M_R results did not pass the "test of reasonableness," as will be described later in this report, and so an outlier removal procedure was developed and applied to both datasets. Second, the averaged data shown in Tables 2 and 3 did not account for any pavement rehabilitation work that may have occurred between 2006 and 2011. Pavement rehabilitation efforts could improve the structural condition (depending on the rehabilitation efforts used), and thus those segments affected would not be useful in determining if the 5-year period between the 2006 and 2011 surveys was too short or sufficiently long to detect a difference in structural behavior within the collected data. The outlier removal process and pavement rehabilitation accounting procedures are described in the following sections.

Removal of Outliers

Within the SN_{eff} and M_R data, a test for outliers was conducted to remove those individual observations from the 2006 and 2011 datasets that were found to deviate extremely from the expected condition. As a test of reasonableness, the data were checked for data points in which the individual data value was outside a range defined by the mean plus and minus 3.49 standard deviations. Assuming normally distributed data, the defined range would be expected to exclude only 0.02% of the data. This operation was performed individually for the SN_{eff} and M_R data obtained from each county. Using this process, the number of data points removed is shown in Table 4. From the 2006 data, 3 and 7 data points were removed from SN_{eff} and M_R, respectively. From the 2011 data, 6 and 8 data points were removed from SN_{eff} and M_R, respectively. In total, 24 data points were removed, which represents 1.4% of all data.

Table 4. Number of Data Points Removed as Outliers

County	2006 Data			2011 Data		
	SN_{eff}	M_R	% Removed (SN_{eff} ; M_R)	SN_{eff}	M_R	% Removed (SN_{eff} ; M_R)
Wythe	0	3	0% ; 1.0%	1	2	0.3% ; 0.7%
Smyth	2	2	0.8% ; 0.8%	1	2	0.4% ; 0.9%
Washington	1	2	0.3% ; 0.6%	4	4	1.2% ; 1.2%

Accounting for Pavement Rehabilitation Efforts

Following the removal of outliers, the effects of pavement rehabilitation efforts between the years 2006 and 2011 were investigated. The structural data for these sections needed to be removed from the analysis so any changes in structural capacity could be attributed to something other than rehabilitation. Annual (visual) condition survey data available from VDOT's Maintenance Division were used to identify the year of last rehabilitation, condition index, and load-related distress index. While this process was performed to identify areas of significant rehabilitation, the potential effects of routine patching were not included. This is because the locations where routine patching is applied are not documented using specific location references but rather a general area. Because of this, it was not possible to identify pavement sections where routine patching may have been applied.

Through a contract with a third-party vendor, VDOT collects condition data on the pavement network annually by using continuous digital imaging and performs crack detection through automated image analysis (VDOT, 2010). In addition, pavement surface roughness and rutting data are simultaneously collected by vehicle-mounted sensors. The data are analyzed to quantify the pavement network condition and the process is performed on all of Virginia's interstate and primary networks and approximately 20% of its secondary network each year, such that the entire secondary system is characterized on a 5-year cycle. An index calculation methodology is employed to quantify the distresses observed in terms of a critical condition index (CCI). The CCI is determined as the lesser (i.e., worse) of the load-related distress rating (LDR) and the non-load-related distress rating (NDR). The LDR incorporates load-related distresses such as wheel-path cracking, patching, rutting, etc., while the NDR includes non-load-related distresses such as transverse and longitudinal cracking (observed outside the wheel path), bleeding, etc.

The structural data for each county were organized by segment as identified within VDOT's pavement management system (PMS). The PMS groups pavements by homogeneous surface segment. For this study, 14, 18, and 15 segments were identified for Washington, Wythe, and Smyth counties, respectively, from the 2012 condition survey (the 2012 condition survey was used since these data would reflect the condition during the 2011 FWD testing). These segments are shown in Table 5 for the southbound directions and in Table 6 for the northbound direction.

Table 5. Number of PMS Segments, Southbound Interstate 81

Segment No.	Washington County		Wythe County		Smyth County	
	Beginning milepost*	End milepost*	Beginning milepost*	End milepost*	Beginning milepost*	End milepost*
1	0.00	3.28	0.00	3.14	0.00	0.12
2	3.28	8.40	3.14	4.63	0.12	1.15
3	8.40	12.98	4.63	5.84	1.15	2.50
4	12.98	13.52	5.84	6.60	2.50	7.27
5	13.52	15.28	6.60	7.31	7.27	9.10
6	15.28	16.28	7.31	8.58	9.10	13.72
7	16.28	18.43	8.58	9.42	13.72	14.14
8	18.43	20.01	9.42	10.54	14.14	15.40
9	20.01	25.40	10.54	13.40	15.40	16.10
10	25.40	29.80	13.40	15.02	16.10	17.24
11	29.80	30.73	15.02	15.45	17.24	18.87
12	30.73	32.07	15.45	17.08	18.87	20.64
13	32.07	32.86	17.08	21.52	20.64	21.22
14	32.86	34.67	21.52	23.18	21.22	21.89
15			23.18	23.48	21.89	23.23
16			23.48	24.29		
17			24.29	28.17		
18			28.17	29.85		

*County relative.

Table 6. Number of PMS Segments, Northbound Interstate 81

Segment No.	Washington County		Wythe County		Smyth County	
	Beginning milepost*	End milepost*	Beginning milepost*	End milepost*	Beginning milepost*	End milepost*
1	0.00	0.78	0.00	3.18	0.00	1.15
2	0.78	1.12	3.18	4.60	1.15	4.94
3	1.12	2.18	4.60	6.45	4.94	7.23
4	2.18	8.36	6.45	8.69	7.23	7.87
5	8.36	9.04	8.69	9.89	7.87	8.77
6	9.04	18.14	9.89	10.63	8.77	9.19
7	18.14	19.66	10.63	11.50	9.19	10.36
8	19.66	21.25	11.50	12.42	10.36	11.05
9	21.25	22.45	12.42	13.53	11.05	11.35
10	22.45	23.30	13.53	14.06	11.35	13.70
11	23.30	26.17	14.06	15.92	13.70	15.45
12	26.17	30.26	15.92	16.36	15.45	16.70
13	30.26	32.72	16.36	17.12	16.70	17.32
14	32.72	34.30	17.12	17.42	17.32	18.94
15			17.42	18.20	18.94	19.89
16			18.20	19.46	19.89	21.90
17			19.46	20.31	21.90	22.68
18			20.31	21.62	22.68	23.36
19			21.62	24.28		
20			24.28	25.50		
21			25.50	29.78		

*County relative.

Following segmentation by PMS homogeneous section, the following PMS data for each county were tabulated: year of last rehabilitation, average CCI, and average LDR. This information for Washington County is shown in Table 7 for the southbound direction and in Table 8 for the northbound direction. The bolded year in the *year of last rehabilitation* column indicates construction activity that occurred between 2006 and 2011, or between the cycles of FWD testing. Also bolded are large differences (taken to be an increase greater than 10 points on a 100-point scale for CCI or LDR) in the Average CCI and Average LDR, which could indicate a rehabilitation activity that was not reflected in the PMS. As shown in Table 7, construction activity for the southbound direction was noted within Segments 7, 9, and 13 with possible construction activity in Segment 1 (as denoted by a large positive difference [improvement] between the 2007 and 2012 CCI values). Construction activity was noted in the northbound direction in Segments 8, 9, 10, 11, and 12 with possible construction activities in Section 1.

The same process was followed for Wythe and Smyth Counties as shown in Tables A1 through A4 in the Appendix. Tables A1 and A2 show construction activity within Segment 1 for Wythe County and possible construction activity in Segments 2, 3, and 5 for the southbound direction and Segments 1, 2, 4, 6, 9, and 11 for the northbound direction. Table A3 shows construction activity within Segments 2, 6, 7, and 15 and possible construction activity in Segment 3 for the southbound direction in Smyth County. Table A4 shows construction activity within Segments 1, 2, 5, 8, 9, 14, and 18 and possible activity at Segments 15 and 17 for the northbound direction in Smyth County.

Table 7. PMS Data for Washington County Along Southbound I-81

Segment No.	**Year of Last Rehab**	**Average CCI**		**Difference**	**Average LDR**		**Difference**
		2007	**2012**		**2007**	**2012**	
1	2001	76	93	**17**	90	94	3
2	2002	65	42	-22	85	55	-30
3	2006	95	96	0	97	98	1
4	2005	89	97	8	97	98	0
5	2002	84	78	-6	97	78	-19
6	2003	84	79	-5	95	85	-10
7	**2007**	86	84	-2	91	97	5
8	2006	92	61	-31	94	61	-33
9	**2009**	57	97	**40**	74	100	**25**
10	1996	75	73	-2	95	88	-7
11	1999	98	97	-1	98	97	-1
12	2004	93	97	3	95	97	2
13	**2010**	83	96	**12**	89	97	8
14	2003	95	91	-5	97	93	-5

Table 8. PMS Data for Washington County Along Northbound I-81

Segment No.	Year of Last Rehab	Average CCI		Difference	Average LDR		Difference
		2007	**2012**		**2007**	**2012**	
1	2001	80	90	**10**	80	90	**10**
2	2005	99	96	-3	99	96	-3
3	2001	90	80	-10	92	80	-12
4	2002	91	71	-20	91	71	-20
5	2004	84	84	0	99	91	-8
6	2003	97	94	-3	97	94	-3
7	2005	83	80	-3	83	80	-3
8	**2011**	83	100	**17**	83	100	**17**
9	**2011**	100	100	0	100	100	0
10	**2011**	96	100	4	99	100	1
11	**2008**	63	95	**32**	83	98	**15**
12	**2010**	63	91	**28**	83	100	**17**
13	1999	84	82	-2	99	90	-9
14	2003	99	84	-15	100	97	-3

Analysis of M_R

Following the segmentation procedure and removal of suspect data and data from segments that included construction activity, the calculated M_R was tabulated for each county. Tables 9 and 10 show the M_R for each county and the percentage change by segment for the southbound and northbound directions, respectively. A positive percentage change indicates the M_R increased from 2006 to 2011. For the data from Washington County, 5 segments were seen to have a lesser M_R while 10 segments were found to have a greater M_R in 2011 versus 2006. The absolute value of the percentage change ranged from 1% to 32%. For the data from Wythe County, 12 segments were seen to have a lesser M_R while 16 segments were found to have a greater M_R in 2011 versus 2006, and 1 segment indicated no change in value. The absolute value of the percentage change ranged from 0% to 45%. For the data from Smyth County, 6 segments were seen to have a lesser M_R while 12 segments were found to have a greater M_R in 2011 versus 2006. The absolute value of the percentage change ranged from 1% to 73%.

Table 9. Summary of M_R Results for Southbound Interstate 81

Segment No.	Washington County			Wythe County			Smyth County		
	2006 M_R (psi)	2011 M_R (psi)	Change	2006 M_R (psi)	2011 M_R (psi)	Change	2006 M_R (psi)	2011 M_R (psi)	Change
1							-*	-	-
2	21,514	21,930	2%						
3									
4				11,505	13,556	18%	17,650	16,150	-8%
5	14,558	15,705	8%				20,082	21,150	5%
6	11,659	12,860	10%	13,150	13034	-1%			
7				13,515	16,612	23%			
8				14,868	16,386	10%	12,165	14,270	17%
9				12,091	13,237	9%	11,466	13,541	18%
10	13,277	14,229	7%	10,949	11,582	6%	14,718	15,627	6%
11	11,469	15,125	32%	14,271	20,788	46%	11,374	12,109	6%
12	13,368	11,626	-13%	22,158	17,169	-22%	14,475	11,471	-21%
13				14,177	13,266	-6%	14,430	14,622	1%
14	12,663	10,302	-19%	11,889	16,711	41%	17,144	14,619	-15%
15				13,250	11,776	-11%			
16				12,622	12,440	-1%			
17				12,676	15,356	21%			
18				12,636	13,884	10%			

*No data were collected within this segment.

Table 10. Summary of M_R Results for Northbound Interstate 81

Segment No.	Washington County			Wythe County			Smyth County		
	2006 M_R (psi)	2011 M_R (psi)	Change	2006 M_R (psi)	2011 M_R (psi)	Change	2006 M_R (psi)	2011 M_R (psi)	Change
1									
2	20,789	18,189	-13%						
3	13,343	15,300	15%	12,081	11,988	-1%	18,478	20,876	13%
4	18,878	18,682	-1%				16,723	17,288	3%
5	10,737	12,990	21%	15,744	20,179	28%			
6	12,896	12,966	1%				18,982	18,758	-1%
7	12,691	13,912	10%	16,090	15,858	-1%	22,919	20,621	-10%
8				11,801	11,326	-4%			
9									
10				10,304	10,698	4%	16,532	18,386	11%
11							13,174	16,513	25%
12				9,710	10,199	5%	11,616	16,419	41%
13	11,803	11,127	-6%	19,531	18,503	-5%	12,167	21,070	73%
14	11,706	12,175	4%	14,705	9,811	-33%			
15				11,798	17,059	45%			
16				14,058	14,544	4%	11,021	10,667	-3%
17				12,542	11,405	-9%			
18				12,327	12,326	0%			
19				16,335	19,791	21%			
20				19,696	18,907	-4%			
21				13,493	15,517	15%			

Analysis of SN_{eff}

The SN_{eff} values were also tabulated for each pavement segment following the removal of suspect data (e.g., data that included construction activity), and the results are shown in Tables 11 and 12 for the southbound and northbound directions, respectively. A positive percent change indicates the SN_{eff} increased from 2006 to 2011. In Washington County, none of the segments had a decrease in SN_{eff}. For Wythe County, Segment 15 in the southbound direction and Segment 12 in the northbound direction had a decrease in SN_{eff} values from 2006 to 2011. For Smyth County, only Segment 10 in the southbound direction had a decrease in the SN_{eff} value. The range of SN_{eff} values was found to be 5.84 to 10.44, and the percent change ranged from a 13% decrease to a 25% increase.

Table 11. Summary of SN_{eff} Results for Southbound Interstate 81

	Washington County			Wythe County			Smyth County		
Segment No.	**2006 SN_{eff}**	**2011 SN_{eff}**	**Change**	**2006 SN_{eff}**	**2011 SN_{eff}**	**Change**	**2006 SN_{eff}**	**2011 SN_{eff}**	**Change**
1							-*	-	-
2	9.48	9.90	4%						
3									
4				6.90	7.33	6%	7.21	7.67	6%
5	7.82	7.99	2%				7.80	8.29	6%
6	7.69	8.00	4%	6.83	7.79	14%			
7				7.10	7.71	9%			
8				6.68	7.71	15%	6.88	7.54	10%
9				6.41	7.06	10%	7.06	7.85	11%
10	7.93	8.83	11%	6.73	7.30	9%	6.98	8.00	15%
11	7.16	7.57	6%	6.35	7.09	12%	6.23	6.87	10%
12	6.74	6.87	2%	6.16	7.38	20%	6.02	6.15	2%
13				6.78	7.57	12%	6.35	6.45	2%
14	6.50	6.76	4%	6.70	7.64	14%	6.72	7.10	6%
15				7.41	6.79	-8%			
16				6.65	7.48	13%			
17				7.26	8.23	13%			
18				7.93	8.41	6%			

*No data were collected within this segment.

Table 12. Summary of SN_{eff} Results for Northbound Interstate 81

Segment No.	Washington County			Wythe County			Smyth County		
	2006 SN_{eff}	2011 SN_{eff}	Change	2006 SN_{eff}	2011 SN_{eff}	Change	2006 SN_{eff}	2011 SN_{eff}	Change
1									
2	8.05	8.09	1%						
3	7.89	8.98	14%	6.61	7.19	9%	7.43	8.34	12%
4	9.14	10.44	14%				6.55	8.21	25%
5	7.31	7.70	5%	7.42	7.61	3%			
6	7.69	8.65	12%				7.62	8.76	15%
7	8.01	9.42	18%	6.65	6.83	3%	7.83	8.72	12%
8				6.97	6.82	2%			
9									
10				6.76	6.95	3%	7.55	7.43	-2%
11							7.25	7.74	7%
12				7.36	6.38	-13%	6.87	7.32	7%
13	6.97	7.19	3%	7.32	7.70	5%	7.59	8.30	9%
14	6.82	7.36	8%	5.84	7.26	24%			
15				6.44	7.78	21%			
16				6.63	7.63	15%	6.22	6.56	6%
17				5.99	6.48	8%			
18				6.66	7.37	11%			
19				6.58	7.83	19%			
20				6.55	7.51	15%			
21				7.06	8.23	17%			

Assessing the Agreement Between Survey Years

The relationship between the 2006 and 2011 datasets was evaluated using several techniques including linear and orthogonal regression. Orthogonal regression allows for consideration of the potential sources of error in calculating SN_{eff} and M_R.

Linear Ordinary Least Squares Regression

The results of the regression for the temperature-corrected Center Deflection (D_0) can be seen in Figure 1. Two relationships are shown in Figure 1: Regression 1 is the case where the 2006 data are taken as the regressor (i.e., taken as the independent variable and assumed to contain no error); and Regression 2 is the case where the 2011 data are taken as the regressor (i.e., the 2006 data are treated as the dependent variable).

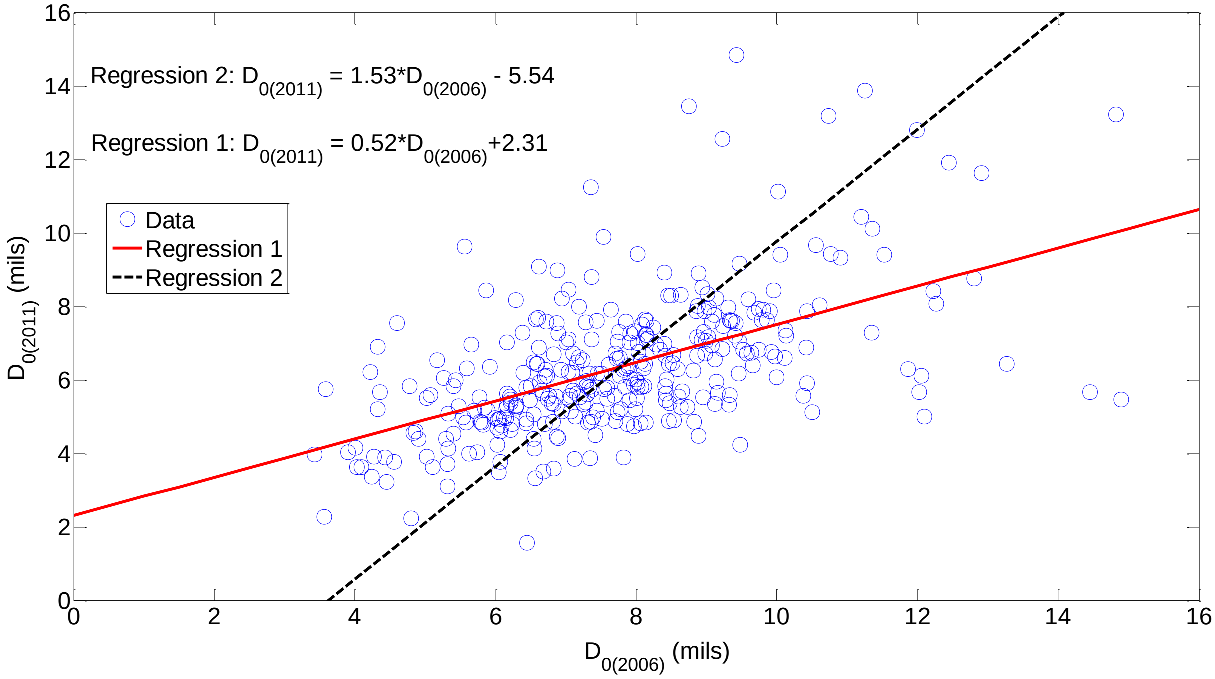


Figure 1. Linear Regression Model for Center Deflection (mils)

The reason for plotting the two relationships in Figure 1 is to demonstrate the impact of violating the assumption in linear regression that the regressor (i.e., the independent variable) does not contain error. For example, assuming the $D_{0(2006)}$ data to be the regressor, and performing ordinary least squares regression to determine the relationship between the $D_{0(2011)}$ and $D_{0(2006)}$, the following relationship is given to predict the $D_{0(2011)}$ data:

$$D_{0(2011)} = 0.52 \times D_{0(2006)} + 2.31 \tag{1}$$

The relationship in Equation 1 implies that if the $D_{0(2011)}$ is measured (with some measurement error ξ), the $D_{0(2006)}$ can be determined as follows:

$$1.91 \times \left(D_{0(2011)} + \xi\right) - 4.43 = D_{0(2006)} \tag{2}$$

However, when the $D_{0(2011)}$ is treated as the regressor (as is the case for Regression 2 in Figure 1), the following relationship is given to predict the $D_{0(2006)}$ data from the $M_{R(2011)}$ data:

$$D_{0(2006)} = 0.66 \times D_{0(2011)} + 3.63 \tag{3}$$

The fact that the coefficient in Equation 3 (0.66) is closer to zero than the coefficient in Equation 2 (1.91) is known as attenuation bias and can be attributed to the fact that the 2006 data also contain measurement errors. This can be demonstrated by first assuming that given the true value of the regressor, the value of the independent variable can be determined as:

$$Y_{true} = a \times X_{true} + b \quad (4)$$

where Y_{true} is the value for the independent variable, a and b are the model coefficients, and X_{true} is the true value for the regressor. Given that the regressor in the case of Figure 1 can only be known within a given amount of error, Equation 4 can be written as:

$$Y_{true} = a \times X_{meas} + b - a\xi \quad (5)$$

where X_{meas} is the measured value of the regressor and ξ is the error term such that X_{meas} and X_{true} can be related by $X_{meas} = X_{true} + \xi$. Bound and Krueger (1991) discuss the attenuation bias in the case that the covariance between the regressor and the error is not zero (as is the case in Equation 5), and present the formula to calculate the attenuation bias as 1-λ, where λ is calculated as:

$$\lambda = \frac{\mathrm{cov}(Y_{true}, X_{meas})}{\mathrm{var}(Y_{true})} \quad (6)$$

where cov(Y_{true},X_{meas}) is defined as the covariance between Y_{true} and X_{meas}, and var(Y_{true}) is the variance of Y_{true}. Equation 6 can be simplified to (Frost, 2000):

$$\frac{\mathrm{cov}(Y_{true}, X_{meas})}{\mathrm{var}(Y_{true})} = \frac{\mathrm{var}(X_{true})}{\mathrm{var}(X_{true}) + \mathrm{var}(\xi)} a \quad (7)$$

Thus, it can be shown that the value for the slope (a) will always be underestimated by an amount that depends on the error in the regressor. The implication is that even if the true values are equivalent to each other (i.e., $Y_{true} = X_{true}$), the error in the regressor will force the model to indicate a value for a that is different than one. Therefore, methods other than linear regression must be used to compare the 2006 data with the 2011 data.

Modeling the 2006 and 2011 Data with Errors in Both Datasets

To compare the 2006 and 2011 data, all sources of error in the data must be investigated. To investigate the impact of errors on the data, the '*true*' D_0 values (D_j for year j) can be written as functions of the measured deflection values (d_j for year j) combined with errors (Equations 8 and 9). In this case, the '*true*' deflection values refer to the value that would be obtained from taking the mean of a significantly large (n approaching infinity) number of tests over a short period of time such that the errors could be averaged out. This also implies that the errors in any given year are randomly distributed with a mean value of zero.

$$d_{2011} = D_{2011} + \varepsilon_1 \quad (8)$$

$$d_{2006} = D_{2006} + \varepsilon_2 \quad (9)$$

where ε is the measurement error, D_j for is the *'true'* deflection for year *j*, and d_j is the measured deflection for year *j*. This error can be composed of several sources, such as FWD repeatability errors, error associated with the temperature correction, or error due to spatial variability (difference in 2011 deflection testing locations relative to the 2006 deflection testing locations). Instead of adding the error due to spatial variability to only the 2011 data, it will be split between the two datasets (i.e., the location of the *'true'* deflection is not at either deflection testing location). The deflection values may be expected to change over time such that $\Delta D_{2006} = D_{2011} - D_{2006}$. Thus, d_{2006} and d_{2011} can be related as;

$$d_{2011} = d_{2006} + \Delta D_{2006} + \varepsilon_1 + \varepsilon_2 \tag{10}$$

The first step is to evaluate the agreement between the two sets of deflection data following the methods outlined in Bryce et al. (2012) and de León Izeppi et al. (2012). Taking the difference in the two sets of readings removes the deflection values, leaving only the error term along with ΔD_{2006}. The differences can then be plotted against the mean of the readings to evaluate the variance and its dependence on the value of the deflections. The results are shown in Figure 2.

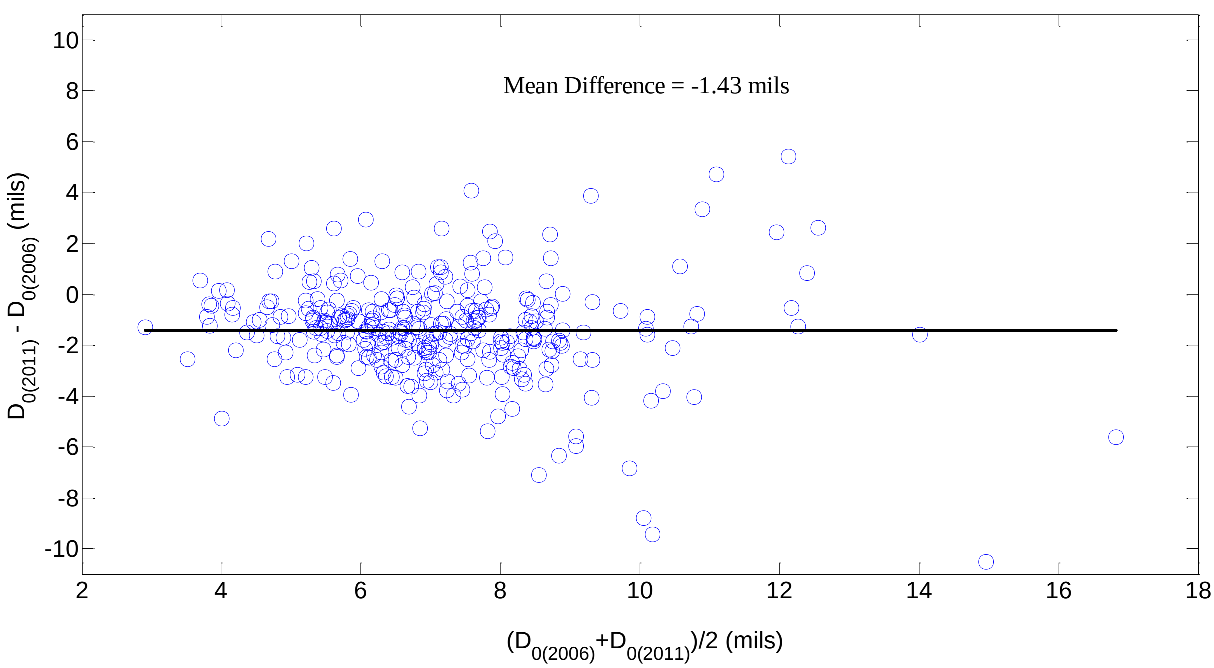


Figure 2. Difference Versus Mean for Deflection Readings (mils)

The first issue to note from Figure 2 is the heteroscedasticity (the dependence of the variance on the mean). Any model to compare the deflections will need to account for the heteroscedasticity in order to be considered valid. Secondly, it is important to note the consistent differences (bias) that exist between the two datasets that indicate a lower deflection value on average for the year 2011. The value for the bias is taken as the mean of the differences, and indicates that a significant difference exists between the two sets of measurements that is not explained as random error. The bias that is shown is described as both the consistent error and the potential changes in the actual deflection values (ΔD_{2006} from Equation 10).

As previously discussed, error exists in both sets of data, leaving ordinary least squares regression as an invalid method for comparison of the data set. In order to account for the error in both datasets, orthogonal regression is typically used. Carroll et al. (2006) provided a discussion of orthogonal regression, and the importance of accounting for the ratio of the errors when using orthogonal regression to compare two datasets. Take the case where **VV** and **W** are the ground truth values each year and **Y** and **X** the corresponding measurements, containing error, obtained for **V** and **W**, respectively. The relationship between **V** and **W** is sought such that $\mathbf{V} = \beta_0 + \beta_x \mathbf{W} + \varepsilon$. Since both sets of measurements include error, **V** and **W** are not observed but rather $\mathbf{Y} = \mathbf{V}+\mathbf{U_V}$ and $\mathbf{X} = \mathbf{W}+\mathbf{U_W}$, where $\mathbf{U_V}$ and $\mathbf{U_W}$ are the errors in V and W, respectively. The basic concept of orthogonal regression is to minimize the Euclidean distance between a data point defined by $\{\mathbf{X}, \mathbf{Y}\}$, and the line defined by $\beta_0 + \beta_x \mathbf{X}$, weighted by the ratio of the variances of $\mathbf{U_V}$ and $\mathbf{U_W}\mathbf{U_W}$ as illustrated in Figure 3. In the case of the FWD measurements for the two years, it is expected to be reasonable to assume a ratio between the variances of the error terms, $\mathbf{U_V}$ and $\mathbf{U_W}$, of one and therefore orthogonal regression was used.

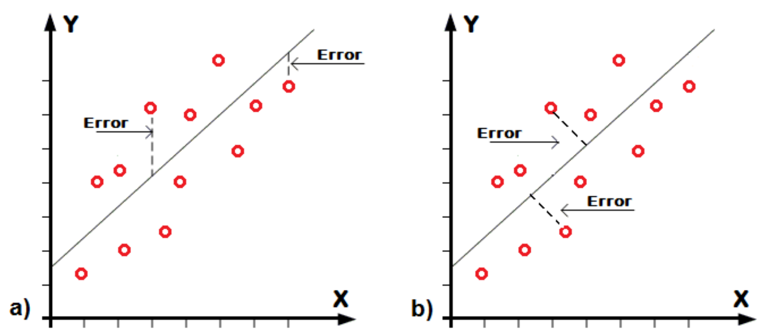


Figure 3. Comparison of Regression Errors: a) Ordinary regression error; b) Orthogonal Regression Error

A $\log_{10}$ transform was applied to each data set in order to adjust for the heteroscedasticity, and the resulting differences in the values plotted against the mean value can be seen in Figure 4. It can be seen from Figure 4 that a consistent bias exists in the data such that

$$\{\mathrm{Mean}[\log_{10}(D_{0(2011)})] - \mathrm{Mean}[\log_{10}(D_{0(2006)})]\} = -0.0908.$$

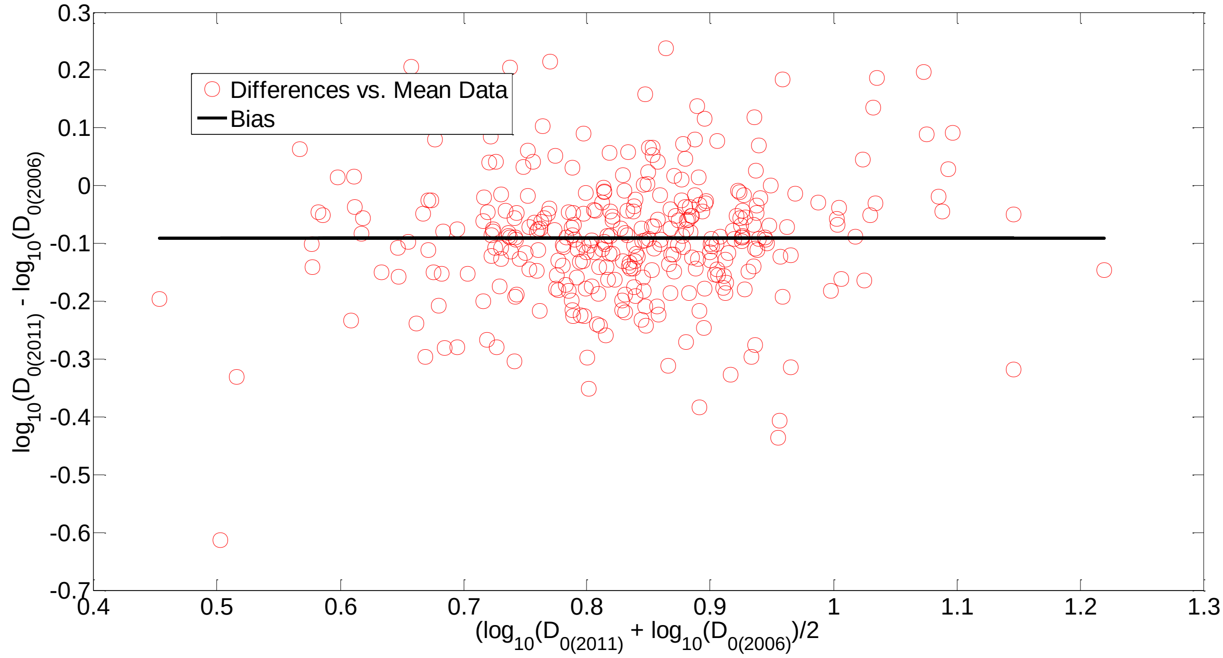


Figure 4. Differences Against the Mean for log10 Transformed Deflections

Using the $\log_{10}$ transform, the orthogonal regression model will be set up as:

$$\begin{aligned} \log_{10} d_{2011} &= a \times (\log_{10} d_{2006} - 0.0908) + b \ldots \\ \log_{10}(D_{2011} + \varepsilon_1) &= a \times (\log_{10}(D_{2006} + \varepsilon_2 + \Delta D_{2006}) - 0.0908) + b \end{aligned} \tag{11}$$

The results of the orthogonal regression can be seen in Figure 5.

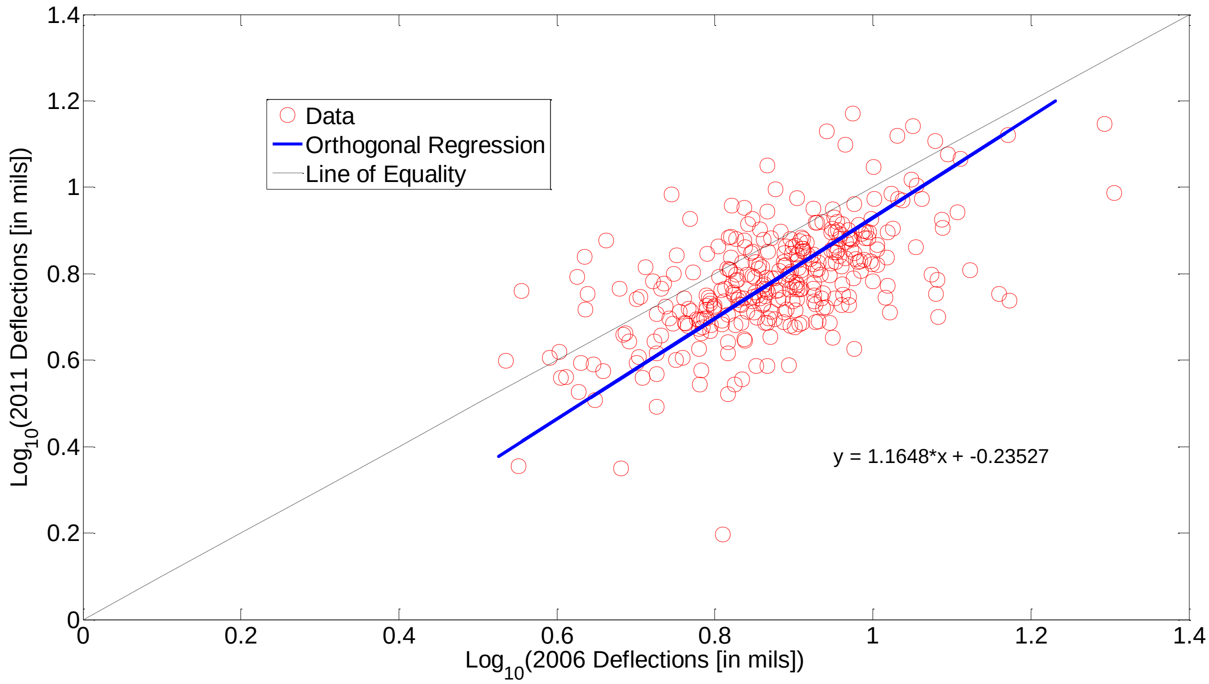


Figure 5. Orthogonal Regression Model for $\log_{10}$ Transformed Deflections (mils)

From the relationship in Figure 5, the following relationships between the deflections (Figure 6) are obtained:

$$\begin{aligned} \log_{10} d_{2011} &= 1.1648 \times \log_{10} d_{2006} - 0.2353\ldots \\ d_{2011} &= 0.5817\, d_{2006}^{\;1.1648} \end{aligned} \tag{12}$$

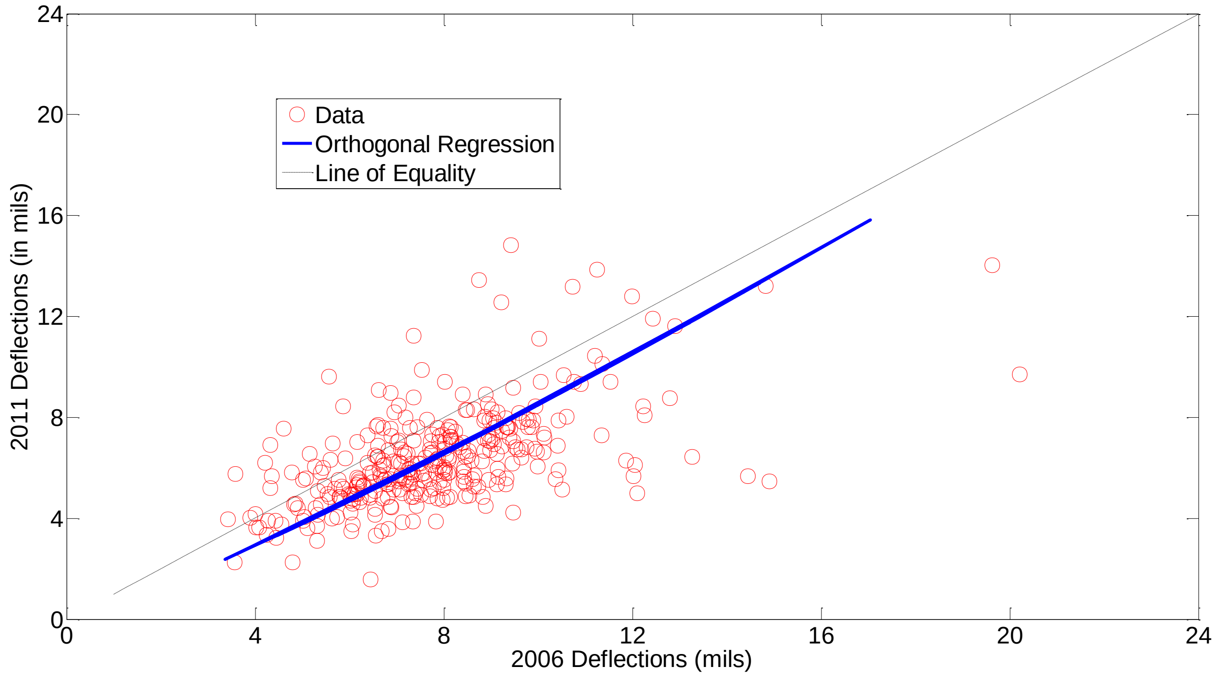


Figure 6. Orthogonal Regression Model for Deflections (mils)

It can be seen in Figure 6 that the 2011 deflection values are consistently lower than the 2006 deflection values. Thus, the question becomes whether the same segment of pavement is consistently found as the weakest relative to other pavement sections in a given year. In order to determine this, the D_0 values were averaged over the pavement segments defined in Tables 5 and 6 (disregarding the segments that received treatments), and then the Spearman Rank Correlation was evaluated between the averaged data. The Spearman Rank Correlation is a non-parametric statistic that measures ranks in the differences in the data (Zimmerman et al., 2003). The resulting Spearman Rank Correlation was found to be 0.79 with a *p*-value of <0.0001 (indicating that the correlation is statistically significant). This indicates that the rank ordering of the relative strength of the pavement segments in each year of testing is strongly related. In practical terms, most of the weaker (alternatively stronger) segments according to the 2006 data are also the weaker (alternatively stronger) segments according to the 2011 data.

Determining the Significance of the Model Variability

In order to determine whether the differences in the 2006 and 2011 data from the Bristol District are significant or may be explained by biases and random variance, a comparison was made between the data from the sections where no pavement rehabilitation was conducted and data from the sections where rehabilitation was conducted. First, the differences in deflections were plotted as a function of the mean of the data over the sections that had rehabilitation conducted between years of testing, as shown in Figure 7. An interesting note is that the bias is lower than that from the sections that had no rehabilitation performed (-0.94 in sections that had rehabilitation versus -1.43 in sections with no rehabilitation performed). Second, the orthogonal regression was conducted using the data from sections that had rehabilitation performed, and the results are shown in Figure 8.

The next step was to determine the 95th percentile confidence interval for each regression model for both the case where no rehabilitation was conducted and the case where rehabilitation was conducted. The 95th percentile confidence interval for the orthogonal regression model is shown in Figure 9 (the case where no rehabilitation was conducted) and Figure 10 (the case where rehabilitation was conducted). By comparing Figure 9 and Figure 10, it can be concluded that the data from pavement segments with no work done on them more closely follow the line of equality than the segments where work was performed, which should be expected. Furthermore, the slope of the regression in Figure 9 very closely follows the line of equality, which indicates that the two datasets represent similar measurements within a certain amount of error. However, the intercept seen in Figure 9 is significantly different from zero, which indicates that the two sets of measurements have a significant consistent difference (i.e., bias). It is expected that the bias is composed of two components: consistent errors between the two rounds of testing and actual changes in deflection values (presumably due to changes in stiffness) between the 2 years, although it is not possible to decouple the two in this analysis.

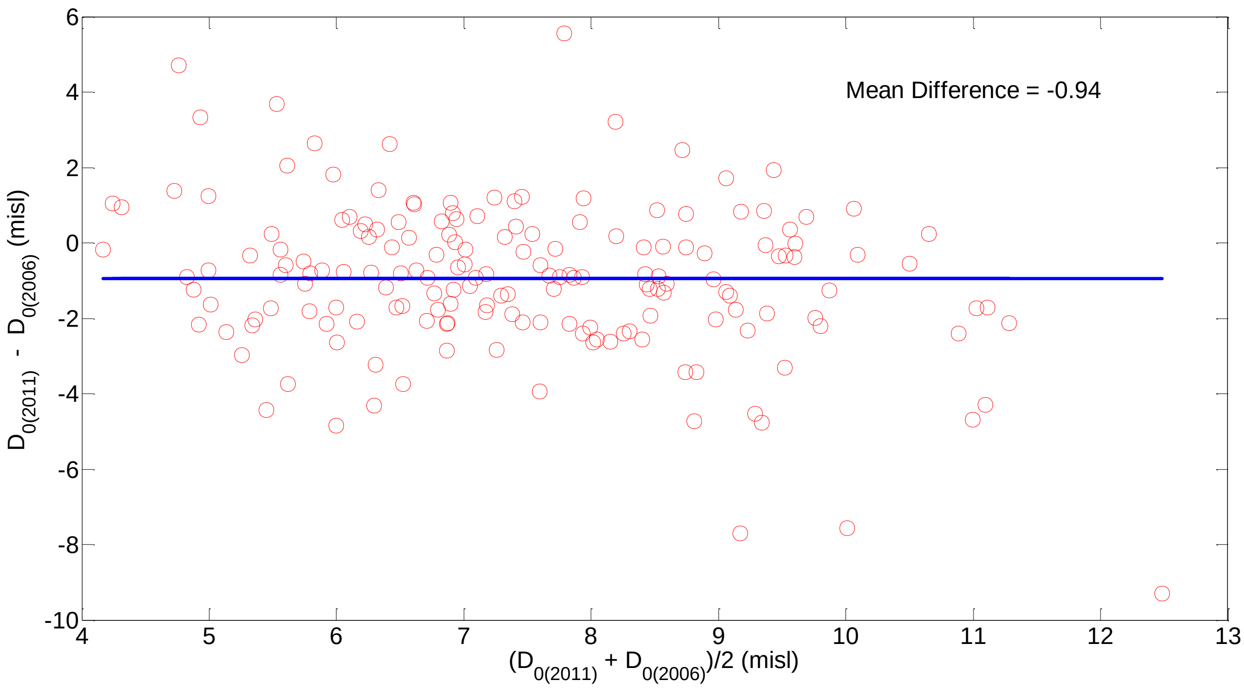


Figure 7. Differences in Deflection (mils) Versus the Mean for 2006 and 2011 Deflections

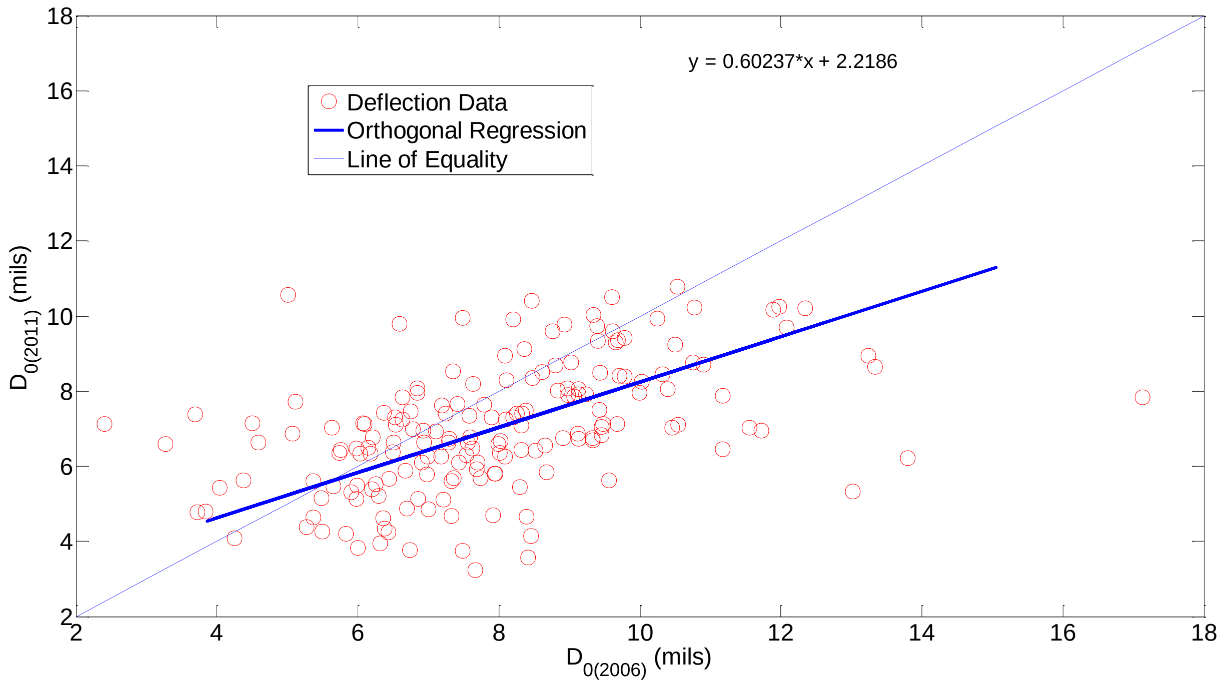


Figure 8. Orthogonal Regression Model for Deflections (mils)

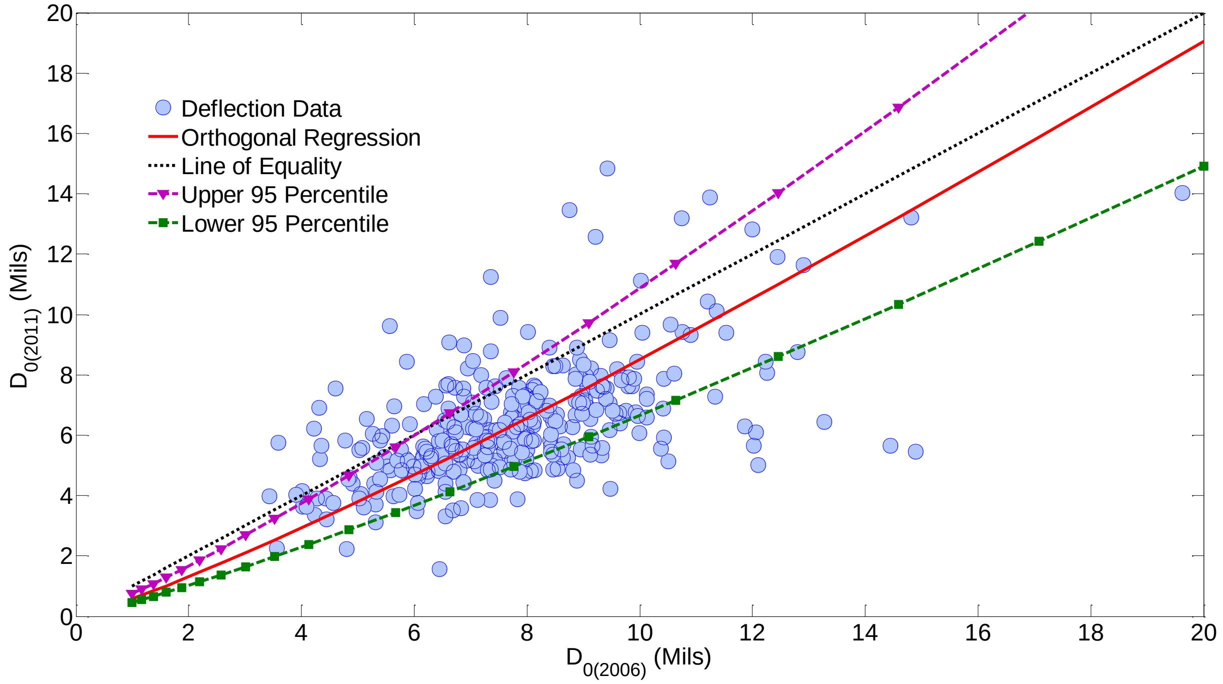


Figure 9. Orthogonal Regression Model for Deflections With 95th Percentile Confidence Intervals on Segments Where No Rehabilitation Was Performed

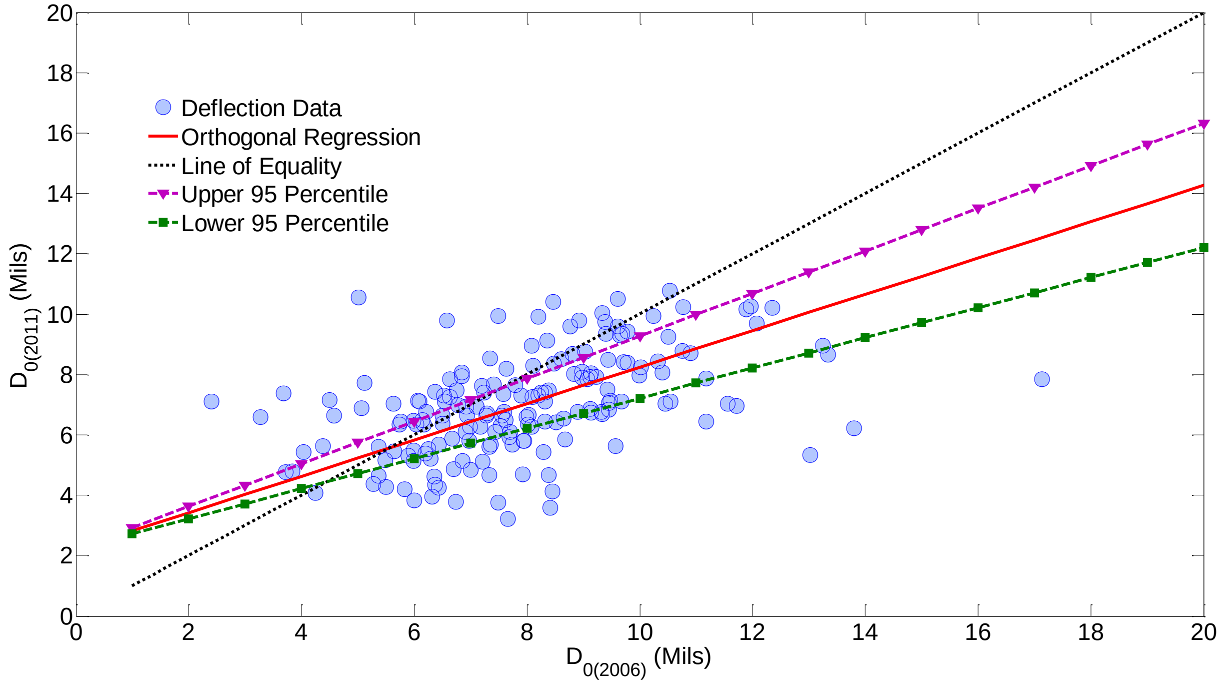
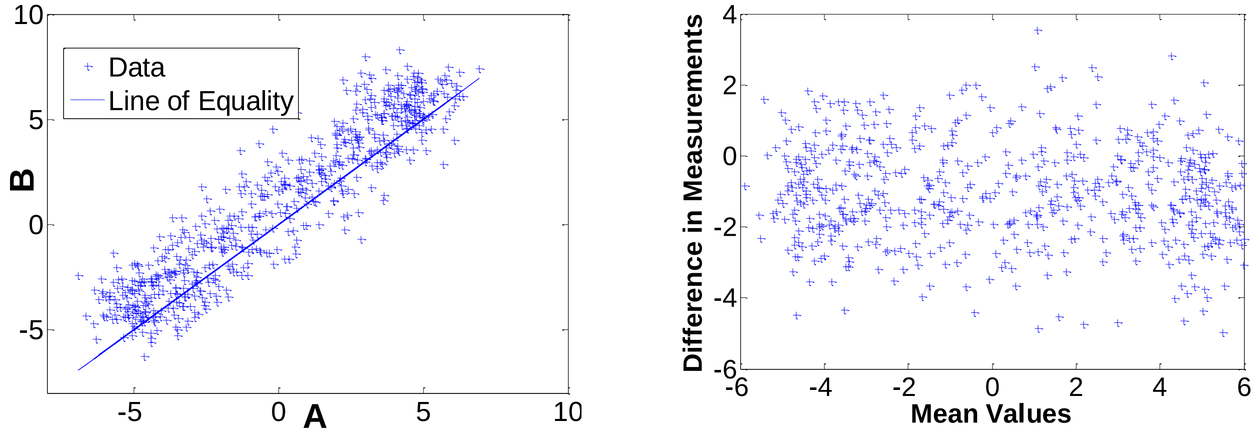


Figure 10. Orthogonal Regression Model for Deflections With 95th Percentile Confidence Intervals on Segments Where Rehabilitation Was Performed

Impact of Errors in Deflection Measurements on Calculated SN_{eff} and M_R

Although it was shown that when the errors and consistent differences in the center deflection are accounted for, the agreement between the $D_{0(2011)}$ and $D_{0(2006)}$ is relatively good, the same may not hold true for the SN and MR values calculated for each year. This is because as the error is carried through the calculations, the distribution of the errors will be altered (particularly if there is a consistent bias in one of the sets of measurements). For example, consider the case where two sets of measurements (**A** and **B**) are taken, and it is found that each set of measurements are the same values with normally distributed errors and a bias exists between the measurements, such that $\mathbf{A} = f(x) + \mathrm{N}[0,1]$, $\mathbf{B} = f(x) + \mathrm{N}[1,1]$. The plot of the measurements can be seen in Figure 11a and the plot of the differences against the mean values can be seen in Figure 11b.



(a) (b)

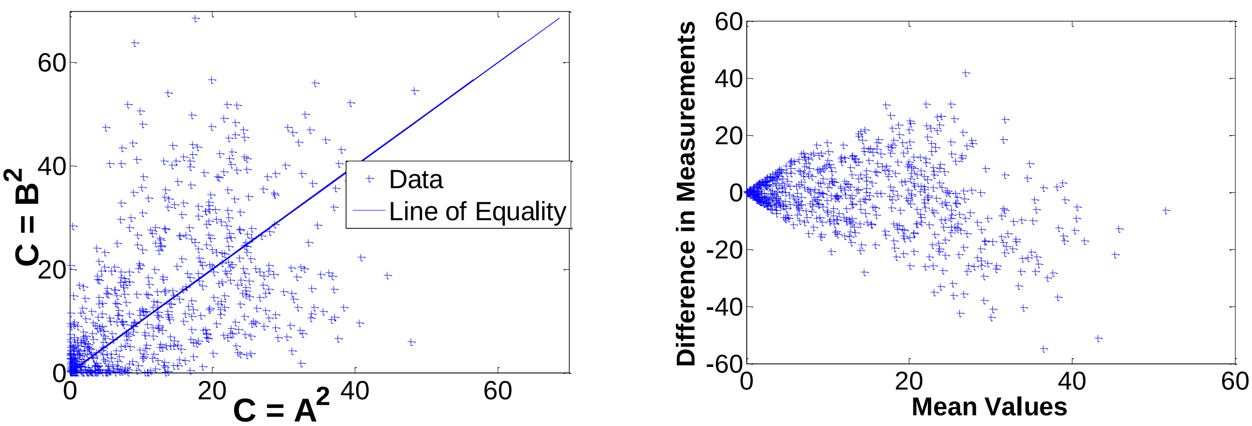
Figure 11. Measurements with Normally Distributed Errors

It can be seen in Figure 11a that the relationship between the two sets of measurements is parallel to the line of equality, and biased such that the measurements labeled **B** are consistently larger than the measurements labeled **A**. Furthermore, it can be seen in Figure 11b that the differences in the measurement are random and normally distributed over the set of measurements, and the range of the differences is equal to the error term. However, if the two sets of measurements are used to predict another variable (**C**) such that **C** = $f(x)^2$, the error tends to distort any relationship between the two calculated values (Figure 12).

In order to estimate the impact of the errors in the deflection measurements on the calculations of SN and M_R, a set of deflection values were generated, and the deflection values were used to calculate a resulting SN and M_R (using the methods described in Diefenderfer, 2008). The resulting values of SN and M_R can be seen in Figure 13. Next, a normally distributed error (similar to the error seen in Figure 2) that was proportional to the deflection values was added to each of the deflection values such that

$$D_{iMeas} = {D_i}/{D_0} \times \mathrm{N}[0,1] + D_i \tag{13}$$

where D_{iMeas} is the deflection value that contains the normally distributed error, D_i is the deflection at distance i from the center deflection (D_0). Two sets of deflections with the same magnitude error were then used to calculate the values of SN and $M_{R,}$ and the results are shown in Figure 14.



(a) (b)

Figure 12. Relationship Calculated from the Measurements with Normally Distributed Errors

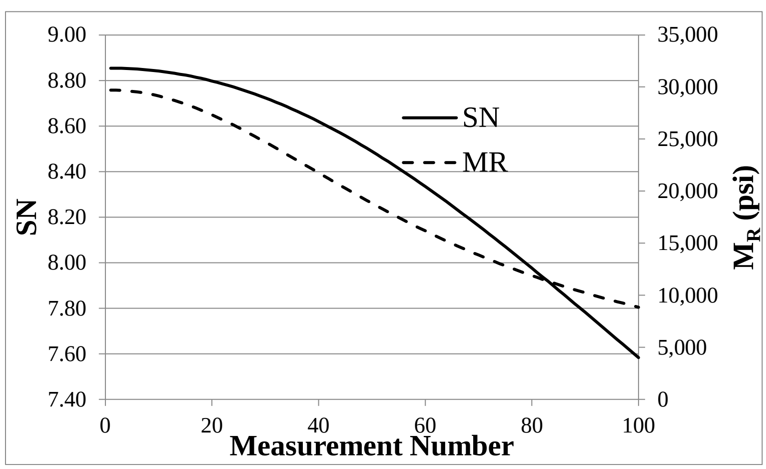


Figure 13. Resulting SN and M_R From Simulated Deflection Values

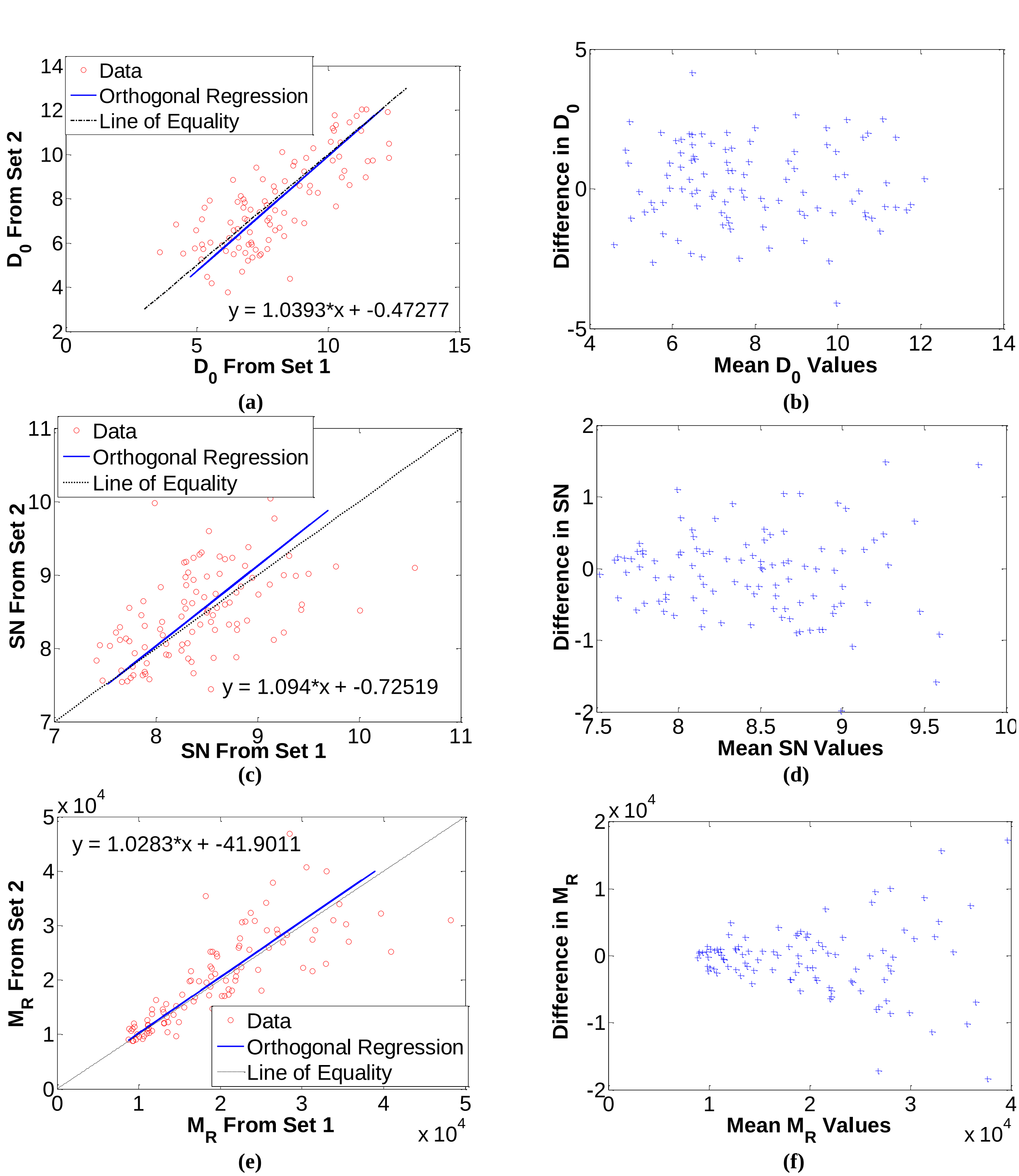


Figure 14. Resulting D_0, SN and M_R from Simulated Deflection Data With Errors

It was found that when a normally distributed error (of similar magnitude to that in Figure 2) is added to the same set of measurements, the resulting SN and M_R calculations can vary considerably, and the variance was highly dependent on the magnitude of the deflection. For example, the resulting SN calculation varied up to ±1.5, and the MR varied up to almost ±2,000 psi at the same testing location. Thus, it can be expected that two pavements with the same SN and M_R may be expected to show significant variations in these values based on relatively small random errors in the deflection values. A comparison of the SN and M_R values was conducted for the 2006 and 2011 data (excluding data where it was determined maintenance was performed between the two rounds of testing), and the results are shown in Figure 15.

The first thing to note in Figure 15 is that the differences in the SN and M_R values calculated between the two years are similar to the differences when the normally distributed errors were added to the same signal as seen in Figure 14d and Figure 14f. This indicates that a significant amount of the variance may be due to the random errors in the deflections. Secondly, using orthogonal regression to compare the $M_{R(2011)}$ values to the $M_{R(2006)}$ values results in a relationship that is very close to equality (with a consistent bias) as seen in Figure 14c. Finally, it was found that adding a normally distributed error to the same dataset that was used to simulate the SN and M_R from Figure 13, and then calculating the results for SN and M_R with the errors in the deflections (Figure 14) produces differences similar to those seen in Figure 14b and Figure 14d. This indicates that the normally distributed errors assumed in the simulated data match the distribution of the errors in the actual deflection data (though the magnitude may be different), and thus the errors seen in Figure 15 may be expected as a result of normally distributed errors.

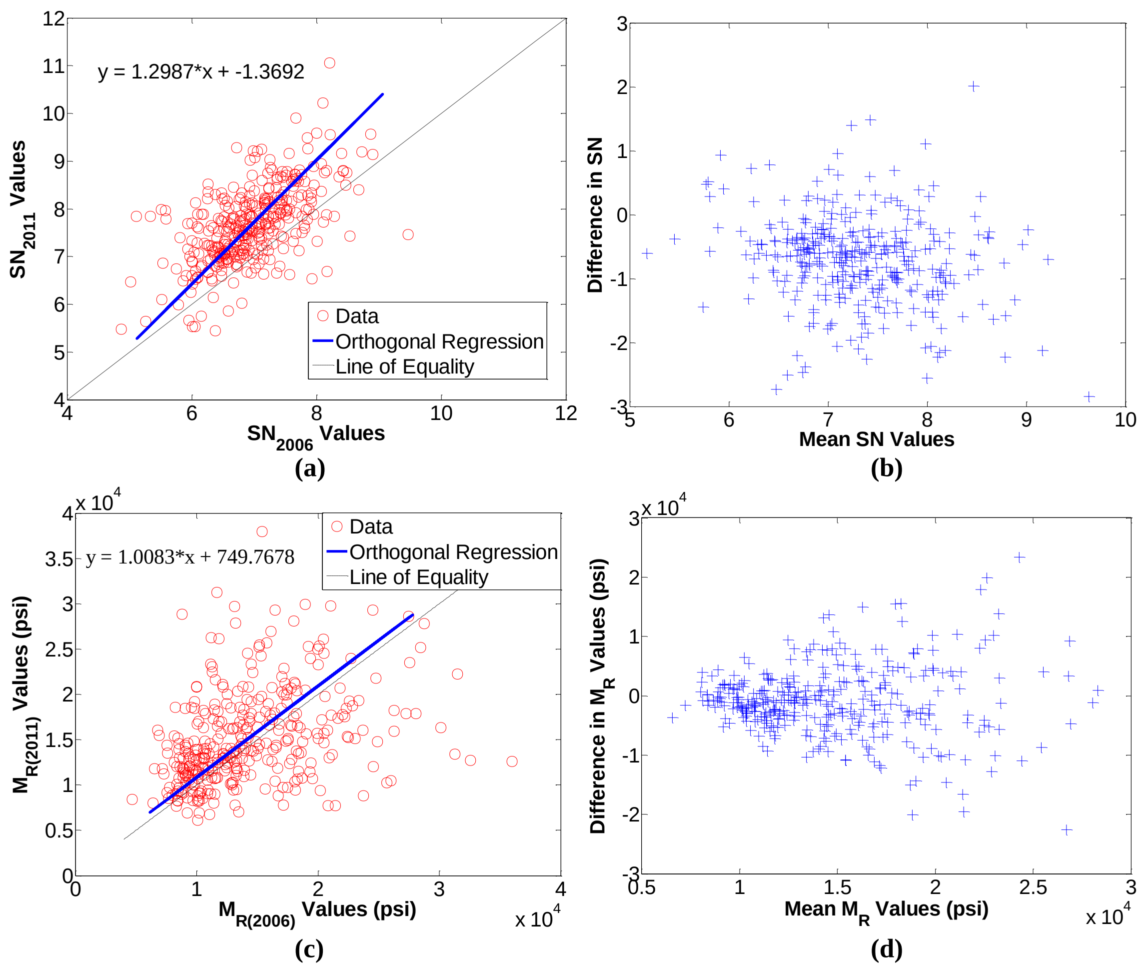


Figure 15. SN and M_R Compared from 2011 and 2006 Deflection Values

The previous analysis demonstrates the importance of correcting for random errors in the deflection values before using the deflections to estimate SN or M_R. Second, it was determined that the errors in both years of deflection testing must be accounted for, rendering a technique such as orthogonal regression as a more valid option than ordinary least squares regression. Third, it is important to understand the potential impact of random errors (which are implicit in every measurement) on the final outcomes of calculating SN and M_R in order to understand whether the variance seen in the final values of SN and M_R are potentially due to random errors, or may be due to changes in the actual values of SN or M_R.

Similar to comparing the D_0 values, the question becomes whether the same segment of pavement (or segment of subgrade) is consistently found as the weakest relative to other pavement sections in a given year. In order to determine this, the M_R and SN values were averaged over the pavement segments (omitting the segments found to have received a treatment between the two rounds of testing), and then the Spearman Rank Correlation was evaluated between the averaged data. The Spearman Rank Correlation for the SN values is 0.71 with a *p*-value of <0.0001 (indicating the correlation is statistically significant). The Spearman Rank Correlation for the M_R value is 0.67 with a *p*-value of <0.0001 (indicating the correlation is statistically significant). Thus, the results of the deflection testing indicated the same weak sections relative to all other sections in 2006 and 2011.

SUMMARY OF FINDINGS

- The deflection values from the 2011 testing (excluding the segments that received a rehabilitation treatment between 2006 and 2011) are lower on average than the deflection values from the 2006 testing, which indicate stronger pavements in 2011.

- The testing in 2011 generally identified the same weak locations relative to other locations tested in 2011 as the 2006 testing.

- Although the deflection values (and corresponding M_R and SN values) differ between the two rounds of testing, the majority of the differences in this study resulted from errors that occurred when repeated deflection measurements were conducted.

- Previous studies, identified in the literature search, suggested repeating network-level FWD deflection testing on a 3- to 5-year basis. Based on analysis of LTPP data, Stubstad et al. (2012) suggested that structural testing on flexible and rigid pavement networks can be repeated on 5-year and 10-year cycles, respectively.

CONCLUSIONS

- *The literature showed that significant benefits are obtainable by network-level structural testing.*

- *The optimum time between rounds of network-level FWD testing could not be quantified because of the inherent sources of error in FWD data collection described herein.* Similar observations have been reported by others in the literature.

RECOMMENDATIONS

1. *VDOT's Maintenance Division should continue to use the network deflection testing results to determine network-level needs. VDOT's Materials Division and district pavement design staff should also continue to use the network deflection testing results to determine project-level needs.*

2. *VDOT's Maintenance Division should consider subsequent rounds of network-level deflection testing based on local needs and available budget rather than a timed interval.*

BENEFITS AND IMPLEMENTATION

This study was unable to quantify the time required between subsequent rounds of network-level pavement deflection testing. However, examples from the literature (e.g., Diefenderfer, 2008) showed that significant benefits are gained from pavement deflection testing on a pavement network. This is because several previous studies have shown significant cost savings can be derived from implementing the results of network-level pavement deflection testing within an agency pavement rehabilitation decision-making process.

Since no quantifiable time between subsequent rounds of network-level pavement deflection testing was identified, VDOT's Maintenance Division should continue to collect deflection data on the pavement network using the FWD as needed and as available budgets allow.

ACKNOWLEDGMENTS

The authors greatly appreciate the assistance of Affan Habib, Bipad Saha, and David Thacker, VDOT Materials Division; Steven Mullins and Chuck Ring, VDOT Bristol District; and David Lee and Travis Higgs, VDOT Salem District; and Trenton Clark, Virginia Asphalt Association. The authors acknowledge Linda Evans of the Virginia Transportation Research Council (VTRC) for assistance with the editorial process.

Affan Habib, VDOT Materials Division; Raja Shekharan, VDOT Maintenance Division; Steven Mullins, VDOT Bristol District; and Kevin McGhee, VTRC; served as the Technical Review Panel for this study.

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APPENDIX
PMS AND STRUCTURAL CONDITION DATA

Table A1. PMS Data for Wythe County Southbound I-81

Segment No.	Year of Last Rehab	Average CCI		Difference	Average LDR		Difference
		2007	**2012**		**2007**	**2012**	
1	**2009**	33	95	**62**	35	98	**63**
2	2000	40	70	**30**	41	70	**30**
3	2002	71	85	**14**	71	87	**16**
4	2006	94	91	-2	94	93	-1
5	2002	71	85	**14**	72	89	**17**
6	2003	94	86	-9	94	86	-9
7	2003	98	88	-9	98	88	-9
8	2006	91	67	-24	91	69	-23
9	2005	97	93	-4	99	98	-1
10	2006	91	56	-35	93	64	-29
11	2004	99	88	-11	99	88	-11
12	2000	86	80	-5	91	82	-9
13	2000	97	85	-12	98	87	-11
14	1998	82	58	-25	86	76	-11
15	2004	93	87	-7	96	90	-6
16	1998	82	54	-28	83	69	-13
17	1994	72	40	-32	75	41	-34
18	1999	82	75	-7	82	76	-7

Table A2. PMS Data for Wythe County Northbound I-81

Segment No.	Year of Last Rehab	Average CCI		Difference	Average LDR		Difference
		2007	**2012**		**2007**	**2012**	
1	2008	70	85	15	70	99	29
2	2011	92	99	7	92	99	7
3	2004	93	78	-15	93	78	-15
4	2009	30	86	56	30	98	68
5	2003	94	97	3	100	97	-3
6	2011	93	87	-6	93	87	-6
7	2004	91	91	0	97	91	-6
8	2003	91	92	1	91	97	6
9	2009	38	92	54	49	94	45
10	2004	88	79	-9	95	91	-4
11	2008	58	98	40	58	100	42
12	2004	80	47	-33	93	47	-46
13	2000	88	88	0	95	94	-1
14	2000	64	24	-40	76	24	-52
15	2005	73	80	7	96	80	-16
16	2000	44	43	-1	61	43	-18
17	2003	78	69	-9	87	69	-18
18	2006	95	70	-25	98	70	-28
19	2000	62	29	-33	67	29	-38
20	2006	94	78	-16	94	78	-16
21	2000	84	73	-11	92	73	-19

Table A3. PMS Data for Smyth County Southbound I-81

Segment No.	Year of Last Rehab	Average CCI		Difference	Average LDR		Difference
		2007	**2012**		**2007**	**2012**	
1	2003	81	88	7	93	88	-5
2	**2009**	53	93	**40**	78	98	**20**
3	1997	63	84	**20**	80	90	**10**
4	2006	74	80	6	94	85	-9
5	1994	98	88	-10	99	94	-5
6	**2010**	64	98	**34**	85	99	**14**
7	**2009**	53	75	**23**	84	75	-9
8	2003	47	44	-3	77	45	-32
9	1996	78	76	-2	94	82	-11
10	2003	60	32	-27	78	33	-45
11	2001	78	46	-32	81	48	-33
12	2003	79	54	-25	82	60	-22
13	1999	85	64	-21	91	68	-22
14	2003	90	85	-4	93	91	-2
15	**2011**	59	97	**38**	68	100	**32**

Table A4. PMS Data for Smyth County Northbound I-81

Segment No.	Year of Last Rehab	Average CCI		Difference	Average LDR		Difference
		2007	**2012**		**2007**	**2012**	
1	2009	92	95	3	92	98	6
2	2008	36	97	61	37	97	60
3	1996	86	77	-9	95	77	-18
4	2003	96	79	-17	96	79	-17
5	2008	51	97	46	63	100	37
6	2003	96	87	-9	96	87	-9
7	1994	72	98	26	97	98	1
8	2011	36	100	64	36	100	64
9	2009	91	97	6	91	97	6
10	1994	99	59	-40	99	59	-40
11	2009	49	73	24	49	73	24
12	1997	60	59	-1	85	59	-26
13	2006	99	87	-12	100	87	-13
14	2011	96	100	4	96	100	4
15	2004	61	81	20	69	81	12
16	2000	83	55	-28	87	55	-32
17	2003	55	97	42	55	98	43
18	2008	76	86	10	76	100	24

Table A5. Structural Data for Segments in Smyth County

Direction	Segment No.	2006 SNeff	2006 MR (psi)	2006 No. of Data	2011 SNeff	2011 MR (psi)	2006 No. of Data
Northbound	3	7.43	18,478	12	8.34	21,744	12
Northbound	4	6.55	16,723	3	8.21	17,830	3
Northbound	6	7.62	18,982	2	8.76	19,977	2
Northbound	7	7.83	26,474	5	8.72	20,214	6
Northbound	10	7.55	16,532	12	7.43	17,561	10
Northbound	11	7.25	13,174	9	7.74	15,450	9
Northbound	12	6.87	11,616	6	7.32	16,213	6
Northbound	13	7.59	12,167	3	8.30	19,963	3
Northbound	16	6.22	11,021	10	6.56	11,146	10
Southbound	4	7.21	17,650	24	7.67	17,077	24
Southbound	5	7.80	20,082	9	8.29	20,501	9
Southbound	8	6.88	12,165	7	7.54	14,338	7
Southbound	9	7.06	11,466	3	7.85	14,160	3
Southbound	10	6.98	14,718	6	8.00	15,573	6
Southbound	11	6.23	11,374	8	6.87	12,430	8
Southbound	12	6.02	14,475	9	6.15	11,837	9
Southbound	13	6.35	14,430	3	6.45	15,543	3
Southbound	14	6.72	17,144	3	7.10	14,938	3

Table A6. Structural Data for Segments in Washington County

Direction	Segment No.	2006 SNeff	2006 MR (psi)	2006 No. of Data	2011 SNeff	2011 MR (psi)	2006 No. of Data
Northbound	2	8.05	20,789	2	8.10	17,981	2
Northbound	3	7.89	13,343	5	8.98	16,878	5
Northbound	4	9.07	18,878	29	10.44	19,530	27
Northbound	5	7.31	10,737	3	7.70	13,382	3
Northbound	6	7.69	12,896	46	8.65	13,441	46
Northbound	7	8.01	12,691	7	9.41	14,814	7
Northbound	13	6.97	11,803	12	7.19	11,390	13
Northbound	14	6.82	11,706	9	7.36	12,570	8
Southbound	2	9.48	21,615	25	9.90	22,536	23
Southbound	5	7.82	14,558	9	7.99	15,700	8
Southbound	6	7.69	11,659	5	8.00	12,857	5
Southbound	10	7.93	13,277	22	8.83	14,452	22
Southbound	11	7.16	11,469	4	7.57	15,120	5
Southbound	12	6.74	13,368	7	6.87	13,227	6
Southbound	14	6.50	12,663	10	6.76	10,508	10

Table A7. Structural Data for Segments in Wythe County

Direction	Segment No.	2006 SNeff	2006 MR (psi)	2006 No. of Data	2011 SNeff	2011 MR (psi)	2006 No. of Data
Northbound	3	6.61	12,081	9	7.19	12,512	9
Northbound	5	7.42	15,744	6	7.61	20,288	6
Northbound	7	6.65	16,090	5	6.83	17,302	4
Northbound	8	6.97	11,801	5	6.82	11,380	5
Northbound	10	6.76	10,304	3	6.95	11,246	3
Northbound	12	7.36	9,710	1	6.38	10,796	2
Northbound	13	7.32	19,531	2	7.70	20,530	3
Northbound	14	5.84	14,705	2	7.26	10,124	2
Northbound	15	6.44	11,798	4	7.71	17,709	4
Northbound	16	6.63	14,058	6	7.63	13,652	6
Northbound	17	5.99	12,542	4	6.48	12,434	4
Northbound	18	6.66	12,327	7	7.37	12,864	7
Northbound	19	6.58	16,335	10	7.83	21,166	11
Northbound	20	6.55	19,696	5	7.51	19,420	4
Northbound	21	7.06	13,493	22	8.23	16,291	22
Southbound	4	6.90	11,505	4	7.33	15,476	4
Southbound	6	6.83	13,150	6	7.79	13,114	6
Southbound	7	7.06	13,515	4	7.71	17,607	5
Southbound	8	6.68	14,868	5	7.71	16,086	5
Southbound	9	6.41	12,091	15	7.06	13,901	15
Southbound	10	6.73	10,949	8	7.30	11,909	9
Southbound	11	6.35	14,271	1	7.09	20,750	1
Southbound	12	6.16	22,158	2	7.38	17,619	6
Southbound	13	6.78	14,178	21	7.57	13,521	21
Southbound	14	6.70	11,889	8	7.64	17,110	8
Southbound	15	7.41	13,250	1	6.79	12,609	1
Southbound	16	6.65	12,622	4	7.48	12,644	4
Southbound	17	7.26	12,676	19	8.23	16,271	19
Southbound	18	7.93	12,636	10	8.41	15,126	10