Impact of Hurricane Katrina on Roadways in the New Orleans Area

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by

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ABSTRACT

On August 29, 2005, Hurricane Katrina devastated New Orleans and southeastern Louisiana, leaving hundreds of thousands either displaced or homeless. Nearly four weeks later, Hurricane Rita made landfall in the southwestern portion of the state, further damaging Louisiana's infrastructure and impacting the New Orleans area. In response, LTRC personnel conducted pavement testing on several on-going construction projects that were submerged to determine if contract modifications would be necessary to address damage impact. Damage was found in asphalt and concrete layers, and subgrades were found to be very weak. For one project, LA 46, LTRC had "before and after" data which indicated that the damage incurred was equivalent to three inches of asphalt concrete. As a result, LaDOTD contracted with Fugro Consultants, LP, to conduct testing on 238 miles of state highways in New Orleans at 0.1 mile intervals.

Fugro conducted Falling Weight Deflectometer, Ground Penetrating Radar, and Dynamic Cone Penetrometer testing along with coring selected locations for thickness and damage verification to determine the extent of structural damage to these pavements. Because there was no "before" data, a traditional forensic type analysis could not be undertaken. With the use of GIS mapping and NOAA flood mapping, data points could be identified as either submerged or non-submerged. The non-submerged data were then considered as a control set, and the submerged data were considered as the experimental set. In this manner, the data could be tested using standard analysis of variance techniques to test the hypothesis that the submerged pavements were weaker and therefore damaged as a result of the hurricanes. It is noted that this methodology does not imply that the non-submerged pavements were not damaged also, but provides a relative damage estimate.

Once weaker strength parameters were determined, standard pavement design methods were applied to the structural numbers and subgrade modulii to determine an equivalent amount of asphalt concrete for this strength loss.

In general, it was found that asphalt pavements had strength loss equivalent to about two inches of new asphalt concrete and that thinner asphalt pavements were weaker than the thicker pavements. Very little relative damage was detected for the PCC pavements. The composite pavements demonstrated no need for additional structure in the pavement layers; however a weaker subgrade for the submerged areas equivalent to nearly one inch of asphalt concrete was identified. Using recent bid prices in New Orleans of $250,000 per mile for a typical rehabilitation scenario (mill four inches/replace four inches of asphalt concrete), an estimated cost for the approximately 200 miles of submerged state highway pavements would be $50 million. There are another 300 miles of federal-aid and 1500 miles of non-federal aid roads that were submerged in the New Orleans area.

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INTRODUCTION

On August 29, 2005, Hurricane Katrina devastated New Orleans and southeastern Louisiana, leaving hundreds of thousands of people either displaced or homeless. Nearly four weeks later, Hurricane Rita made landfall in the southwestern portion of the state, further damaging Louisiana's infrastructure and, once again, bringing destruction to portions of the New Orleans area. While much of the damage to buildings and bridges was immediately obvious, the damage imparted to roadways would not be so easy to recognize. It was expected that the sustained flooding (three days or more) had damaged the roadway pavement structures below the surface in the submerged areas. Further, subsequent debris removal was expected to provide additional damage both to roads that were submerged and those that were not submerged. Such damage, if not repaired, is certain to have a profound impact on the recovery and future social and economic development of the New Orleans area.

LaDOTD's data collection efforts prior to the storm were designed largely to address pavement management and rehabilitation efforts and were, therefore, not suitable for evaluating the pavement structural damage that resulted from the submergence. Pavement distress data such as International Roughness Index (IRI), rutting, and cracking were available on some state routes, but vital structural data needed to determine the flood's impact, such as the resilient modulus of the pavement layers or overall structural number (SN), were not collected as part of the pavement management systems' biennial program. Such limitations meant that it would be impossible to conduct a comprehensive "before and after" style structural analysis to determine the reduction in strength of the pavement layers caused by the flood.

The Louisiana Transportation Research Center (LTRC) initially conducted structural damage testing on several roads that were under construction to determine any damage that might require additional work. Based on the preliminary results additional roads were tested in the New Orleans area. In all, a total of eight roadways were tested consisting of Falling Weight Deflectometer (FWD), Dynaflect, Dynamic Cone Penetrometer (DCP) and coring. The FWD provided pavement modulus and subgrade modulus through a back calculation method; the Dynaflect provided pavement structural number (SN) along with subgrade modulus; the DCP provided verification of the base and subgrade readings; and, the coring provided thicknesses and verification of moisture damage. Appendix A presents test results and a summary analysis for each project.

As presented in Appendix A, structural damage is indicated in the asphalt concrete pavement, concrete pavement, and base course along with weakened subgrades in most cases as related to the modulus of these layers. Some of the concrete pavements are showing voids at the joints with lost joint transfer efficiencies. In seven of eight cases there is no "before hurricane" data such that actual damage can be directly attributed to the submergence. However, there is "before" data for one project, LA 46, coming from a research project. In addition, the structural numbers (SN) for pavements under construction at the time of Katrina (I-610 and Metairie Road) are lower than the design SN, indicating damage directly caused from the submergence.

A direct "before-after" analysis of structural strengths for LA 46 is included in Appendix A. On this four-lane section (CSLM 1.4 to 3.1), testing provides an average reduction of 0.9 SN and a reduction of 1.6 ksi for the subgrade resilient modulus. The results of this testing indicated that the pavement structure had been adversely impacted by the flood waters equivalent to three inches of asphalt concrete.

On the basis of the LTRC investigation, LaDOTD contracted with Fugro Consultants, LP to conduct structural testing on 238 miles of state highways in New Orleans. FWD testing was undertaken every 0.1 mile and was correlated with Ground Penetrating Radar (GPR) test data which was collected continuously (0.5 foot intervals). In addition, DCP and cores were periodically taken to verify thicknesses and base course/subgrade moduli. This report evaluates the data obtained from the Fugro testing.

METHODOLOGY

FUGRO DATA

Fugro consultants performed tests on 238 miles of federally supported urban system pavements in New Orleans. Non-destructive testing was performed with the FWD every one-tenth of a mile. GPR was used to determine pavement layer thicknesses and identify areas with thickness variations or potential voids. GPR thickness data were calibrated by conducting coring tests through the pavement and base course with thickness measurements being taken of each layer. Coring also provided information to determine the type of pavement layers (i.e., asphalt, concrete, brick) as well as their condition, and the type of base course (i.e., soil cement, sand, sand shell). A visual survey of the subgrade was conducted to determine its type (i.e, sand, clay). DCP tests were conducted to provide an additional assessment of the base course and subgrade as well as to validate subgrade strength readings from the FWD.

The Fugro report provided the following data to LaDOTD. Appendix B provides details of the test factorial, descriptions of equipment used, analysis equations, and procedures.

- Subgrade resilient modulus (M_r)
- Effective pavement modulus (E_p)
- Modulus of subgrade reaction (k) for concrete pavements
- Effective structural number (SN_{eff}) based on deflections for flexible pavements
- Surface curvature index (SCI) values based on surface deflections
- California bearing ratio (CBR) values from DCP tests results
- Deflection basin analysis
- Dynamic cone penetrometer index (DCPI)

The FWD data provided by Fugro was reviewed to check for calculation and equipment errors. In order for the collected data to be considered valid for FWD testing, the deflections measured by the sensors were required to decrease as the distance of the sensor from the load plate increased. Any points collected that indicated “non-decreasing deflections” were considered invalid and removed from the data set.

It was decided to use the FWD data for the analysis conducted for this report in order to provide a timely assessment of the structural damage of the submerged pavements. Specifically, three test parameters, the first sensor deflection (D1), effective structural number (SN_{eff}), and subgrade resilient modulus (M_r) were selected.

- **First sensor deflection (D1):** The deflection of the pavement at the load plate reflects the strength of the overall pavement structure. High deflections represent weaker pavement structures.
- **Effective structural number (SN_{eff}):** The effective structural number represents the effective structural strength of the existing pavement and base course, which in this case was derived using formulas from the 1993 AASHTO design guide, deflections obtained from FWD testing, and pavement layer thicknesses determined by the GPR and validated through coring.
- **Subgrade resilient modulus (M_r):** The resilient modulus was derived using the AASHTO formula and deflection data from the FWD. The M_r was reduced by (0.33) to correlate to laboratory derived M_r as suggested in the AASHTO design guide.

RESEARCH APPROACH

Because there were no direct "before and after" comparison sites available other than LA 46 to show damage caused by Katrina, another methodology was chosen to demonstrate that structural damage had been incurred by those pavements subjected to submergence. The Fugro data set was divided and coded to distinguish those pavements that were submerged and those that were not. The non-submerged pavements could then be treated as a control section to test the hypothesis that damage was done to the submerged pavements. It should be noted that the control section designation does not imply that the non-submerged sections were not also damaged. In addition, further damage because of the volume and loads of the debris haul trucks that continue to travel over these weakened structures can not be determined from this data set as the Fugro evaluation was conducted several months after the waters had receded. A subsequent sampling of the same pavements in the future might reveal this additional damage. This investigation primarily attempts to determine if submergence increased the distress in the flooded areas to a greater degree than that in the non-flooded areas.

Methods employed to separate flooded areas from non-flooded areas

ArcGIS, a commercially available global information system (GIS) software package was used to import GIS referenced maps, data points, and perform basic spatial analysis.

Suitable maps for the New Orleans area were downloaded into ArcGIS from the United States Geological Survey (USGS) (http://seamless.usgs.gov/website/Seamless/). All test points from the Fugro data set were tagged with their respective GPS coordinates at the time of testing and were also imported into ArcGIS. Figure 1 shows the results of this integration. The red circles identify Portland Cement Concrete (PCC) pavements, the yellow circles identify AC pavements, and the blue circles identify composite pavements.

Detailed flood maps from FEMA (http:www.gismaps.fema.gov/2005pages) were imported into the ArcGIS system to separate the flooded areas from the non-flooded areas. As an example, Figure 2 presents the FEMA map which represents the maximum extent of flooding on September 2, 2005, and Figure 3 presents the point segregation.

Determining flood durations: The National Oceanographic and Atmospheric Association (NOAA) produced a series of modified False Color Infrared SPOT images (http://www.nhc.noaa.gov/) that were used to determine the duration of flooding. The specific images utilized from NOAA included images from August 31, and September 3, 5, 8, 10, 12, 14 to 20. The ArcGIS renderings for the dates of September 3 and 14 are provided in Figures 4 and 5 to serve as examples and to illustrate the ponding/differential de-watering effect described. For example, comparing Figures 4 and 5 clearly shows that the pocket of water south of Lakefront Airport in east New Orleans was drained much faster than the pocket south of Lake Pontchartrain in the Gentilly area. Using these maps, it was possible to segregate the test data according to flood duration.

The datasets were segregated into four flood duration groups. The first group represented points that had been submerged for a period of one week. The second group included points that were submerged for two weeks. The third group remained under water for a period of three weeks. The final group did not flood at all. One difficulty that the researchers encountered related to image drop-outs over time. Loss of coverage in some cases made it impossible to determine when certain test points became dry. Re-examination of Figure 4, for example, does show proper coverage of St. Bernard Parish on September 3, but the detail provided in Figure 6 shows that NOAA was no longer monitoring St. Bernard Parish by September 14. Data from pavements that dropped out of coverage were removed as the duration under water could not be established. Although such issues were problematic, there was enough coverage over time and over a large enough area to perform a proper analysis.

ANALYSIS

Statistical Methods

Statistical Analysis Software (SAS) version 9.1.3 was used for hypothesis testing. Analysis of Variance (ANOVA), and a comparisonwise test of the means were used for evaluation. For this study, three structural parameters from the Fugro data were analyzed: D1, SN_{eff}, and M_r. A confidence level of 95 percent was used for all testing.

The initial data set, identified as flooded and non-flooded (flood type), included all pavement types (PCC, Asphalt, Composite). The data were then further broken down into additional factors including pavement thickness and duration of submergence. The AC pavements were divided into < 7 in., 7 to 11 in., >11 in. thick groups', composite pavements were divided into < 16 in., > 16 in. thick groups', and PCC pavements were divided into < 10.5 in. and > 10.5 in. thick groups. The duration of submergence was identified as non-flooded, one week, two weeks, and three weeks or more. Typically the data set for duration of submergence was smaller because of coverage drop out. An Analysis of Variance (ANOVA) test was conducted using the main factors pavement type, flood type and their interactions.

Further analyses were conducted on thickness and duration of flooding. It was hypothesized that thinner pavements or pavements that were under water for longer periods of time would be more damaged than thicker sections or shorter duration. This analysis was conducted for each pavement type.

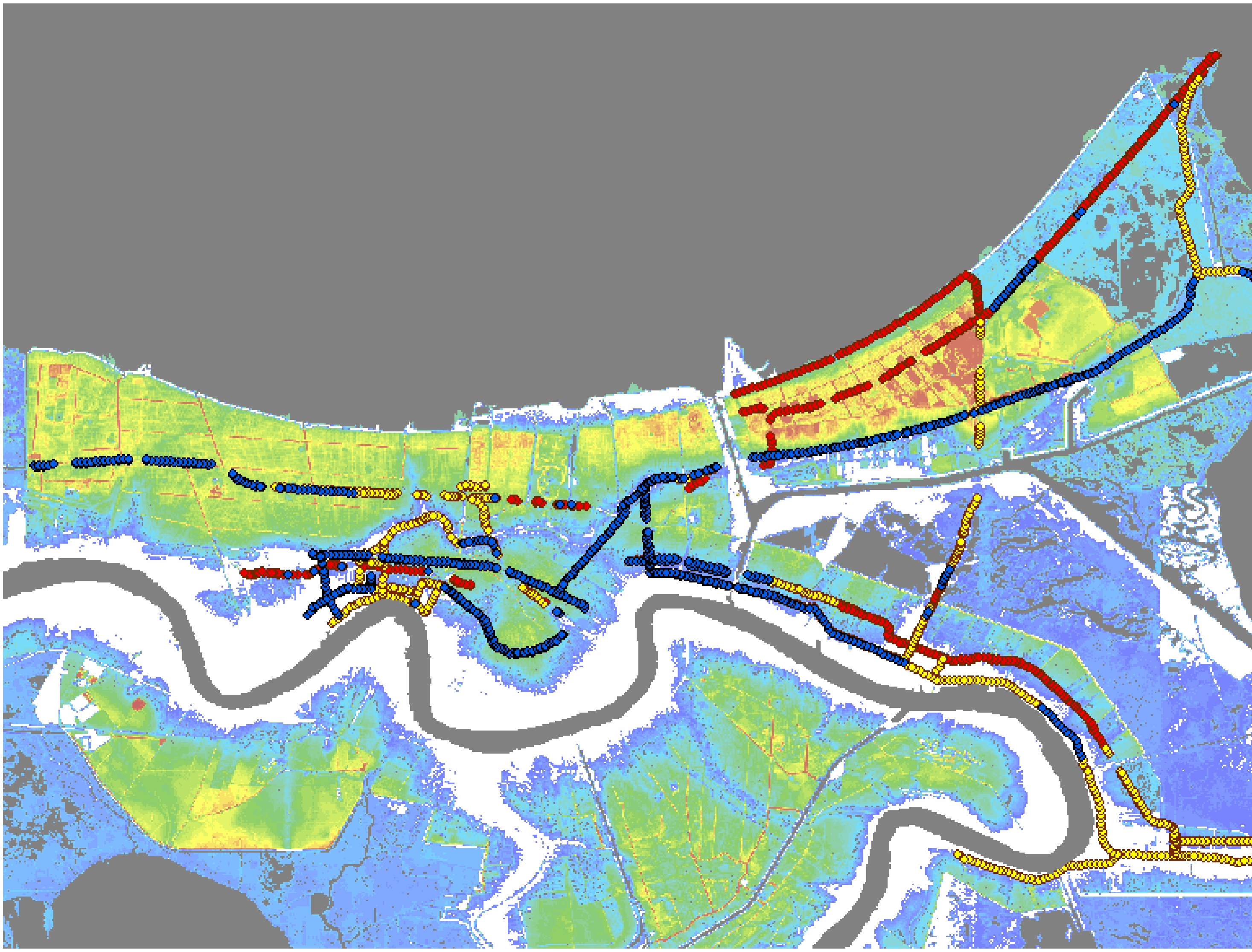


Figure 1: Integration of Fugro points with geo-referenced backplane in ArcGIS

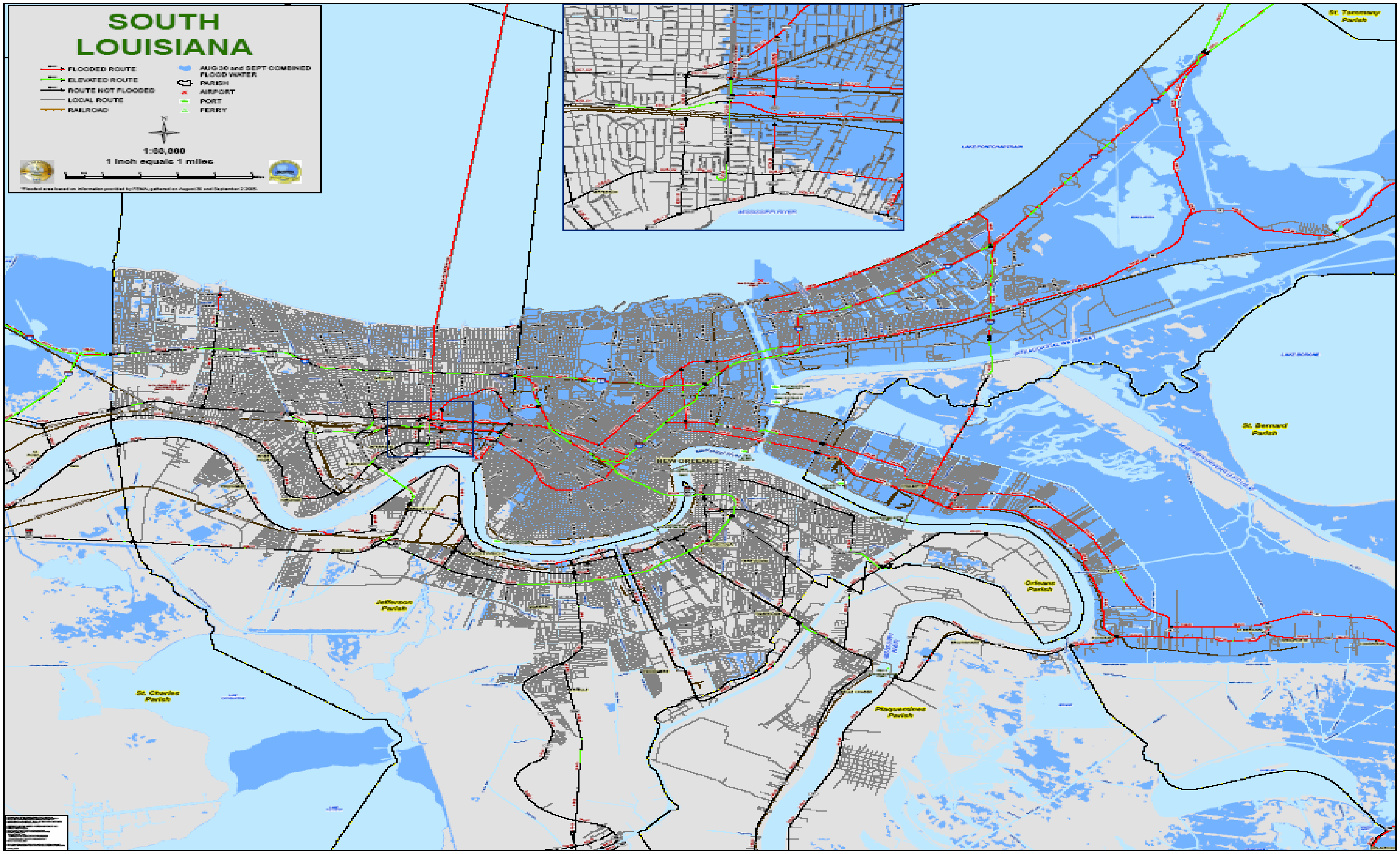


Figure 2: September 2nd flooding

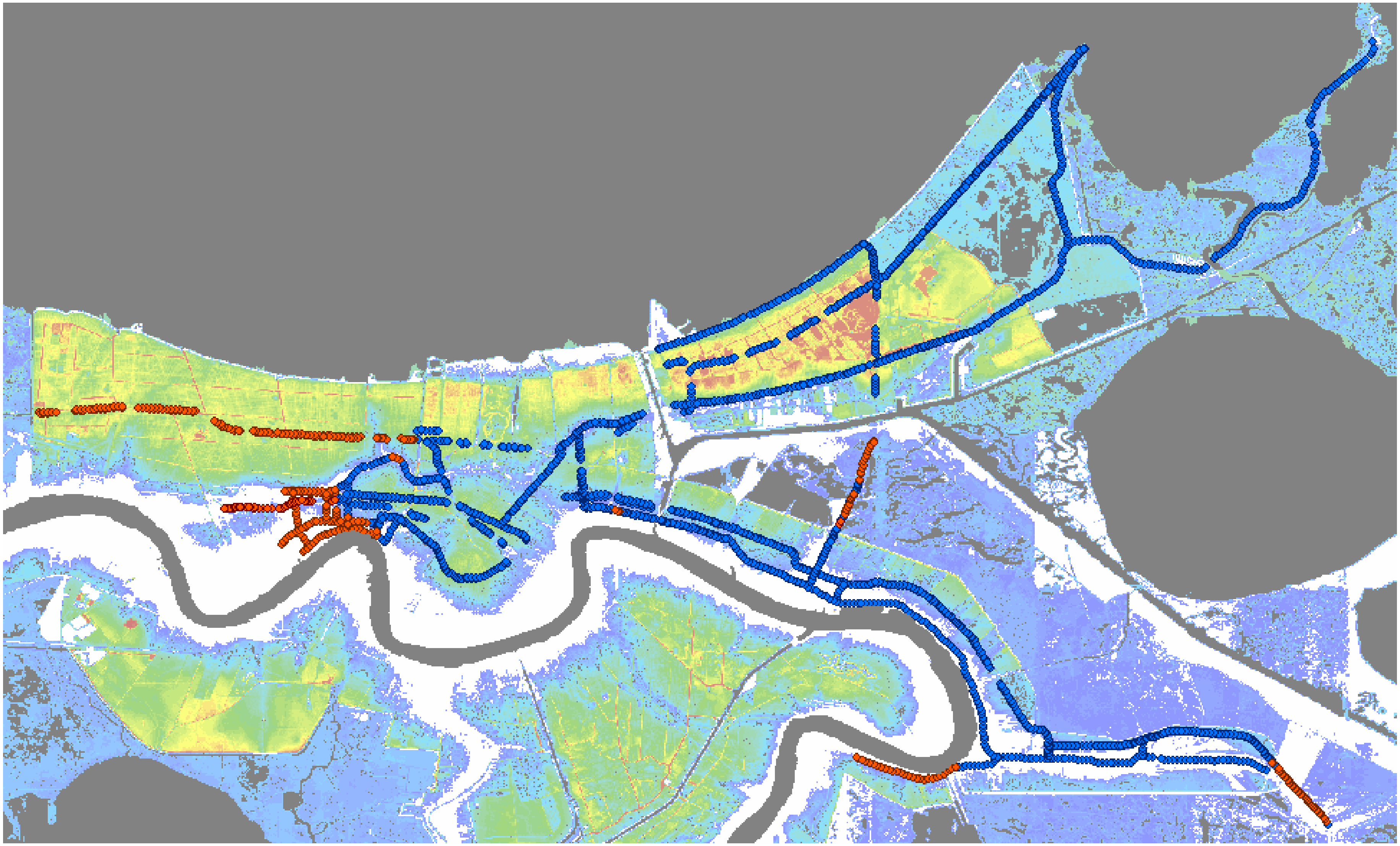


Figure 3: Segregation of Fugro points according to September 2nd flooding

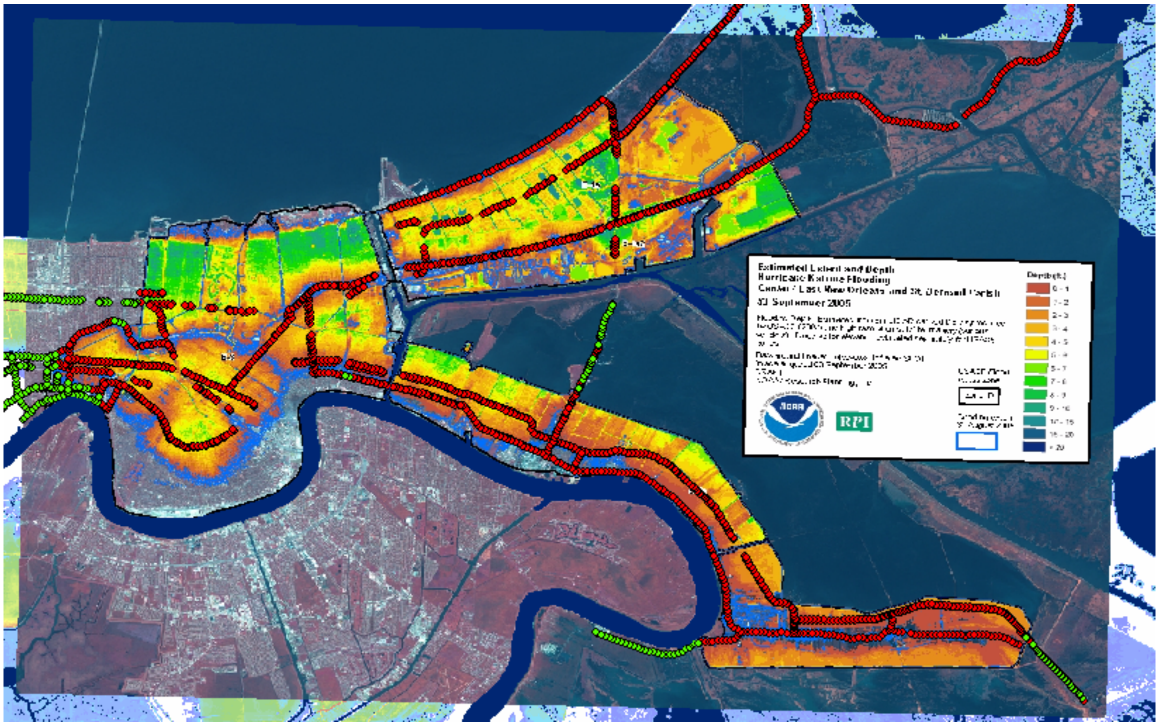


Figure 4: September 3 flooding (NOAA)

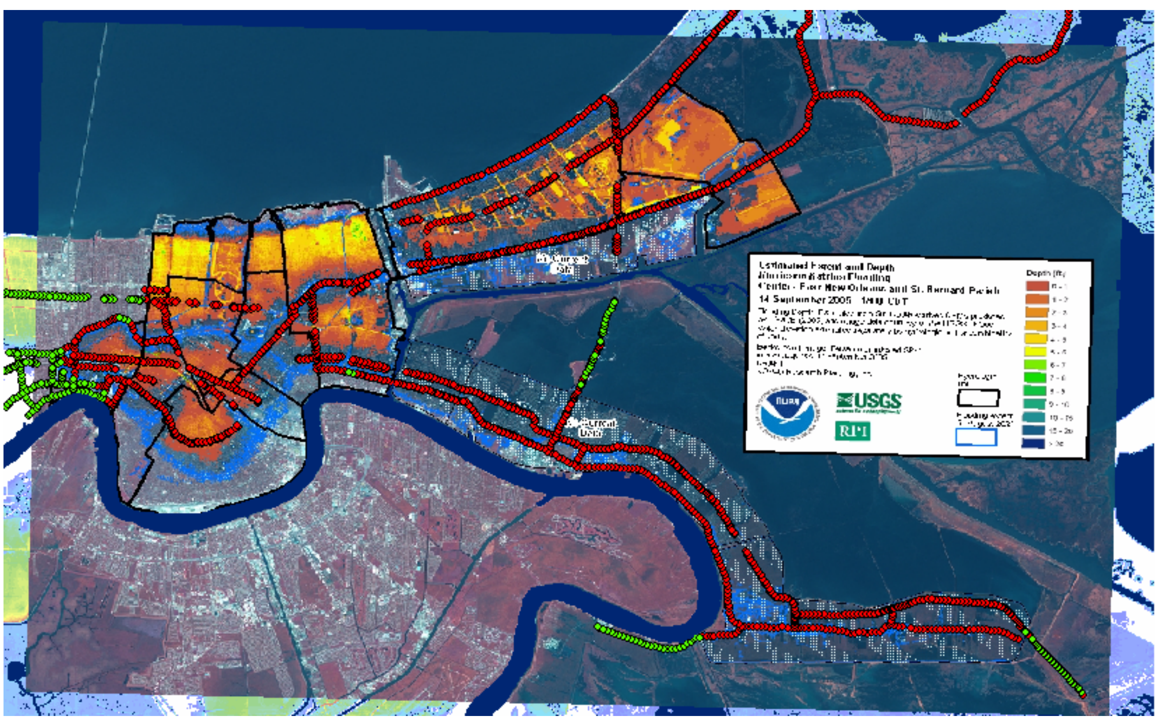


Figure 5: September 14 flooding (NOAA)

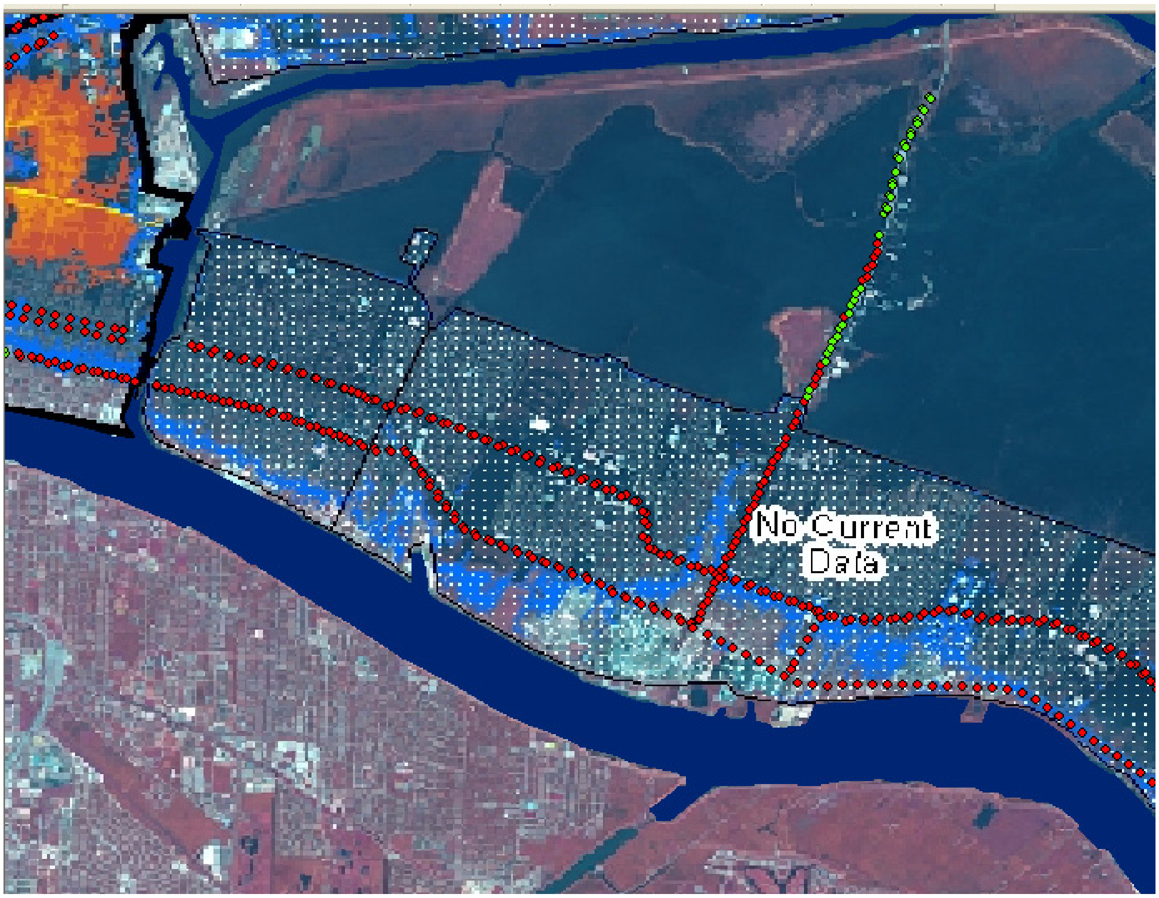


Figure 6: Detail of St. Bernard Parish on September 14 (taken from Figure 5)

Pavement Damage Analysis

LaDOTD has historically used Dynaflect-generated SN values for pavement design. AASHTO pavement design coefficients were developed based on Marshall Mix design properties as correlated to SN. FWD derived SN has not been used in Louisiana. In order to convert the SN derived from the FWD to Dynaflect SN, a correlation equation was developed using data from previous projects. Three hundred and thirty-four (334) points from 15 projects were used for this correlation. Appendix C provides the development of the correlation.

If the statistical analysis demonstrates that the submerged pavements by pavement type had structural damage through lower SN, then this loss of SN can be converted to an equivalent pavement thickness, using the coefficient for asphalt concrete to represent the structural loss. For example, if flooded pavements had an SN of 3 and the non-flooded pavements had an SN of 4, then the loss of structure is an SN of 1. In Louisiana, a layer coefficient of 0.44 is generally assigned to asphalt mixtures. Therefore, the equivalent in. of AC due to the distresses in this example would be 2.3 in. (1/0.44). This conversion of SN to equivalent inches of asphalt concrete (AC) provided a mechanism to estimate a cost associated with the damaged pavements.

In addition, to weakened pavement structures, weakened subgrades should also be considered. The DARWIN pavement analysis procedure is used to infer the damage caused by decreasing the subgrade resilient modulus. DARWIN 3.1 is the designation for a series of AASHTO's computer software programs for pavement design and was part of the implementation of the 1993 AASHTO Guide for Design of Pavement Structures. DARWIN 3.1 is divided into four modules: Flexible Structural Design, Rigid Structural Design, Overlay Design and Life Cycle Cost. Each module addresses a specific item in the overall pavement design process.

The thickness of pavement structures for design purposes is typically determined using the DARWIN computer program. The main input parameters for the program are the highway’s subgrade resilient modulus (M_r) and the traffic loads. The output from the analysis is the required structural number (SN) that the design pavement would need in order to support the intended traffic and protect the subgrade.

Using the output from the DARWIN pavement design, a curve can be created that shows how SN would vary as the subgrade M_r is changed for a given traffic load level. Figure 7 provides an example using data generated from the LA 46 pavement which is the only pavement in the study for which before and after data exists (example DARWIN tables are presented in Appendix D). If the subgrade resilient modulus is 6 ksi, then the required pavement thickness would be 10.5 in. If the subgrade resilient modulus is reduced to 3 ksi, then the required pavement thickness would be 13 in. Using this methodology, an equivalent amount of pavement thickness can be assigned due to the reduction of subgrade modulus. In this example, it would be 2.5 inches of pavement.

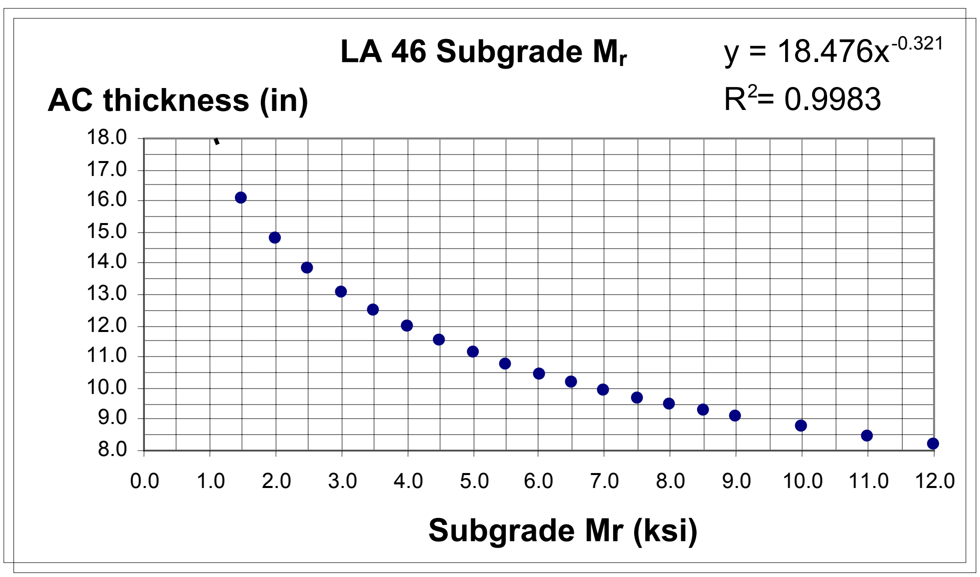


Figure 7
AC verses subgrade M_r (LA 46)

It is recognized that this methodology for the damage attributable to lower subgrade modulus is project-specific because of the dependence on traffic loading. However, if extended to all the pavements submerged, it can provide at least an estimate of the total damage. Once a particular project has been identified for rehabilitation, the actual traffic can be used to determine the thickness required.

Figure 8 presents a flow chart of the statistical methods and methodology used to analyze the Fugro data.

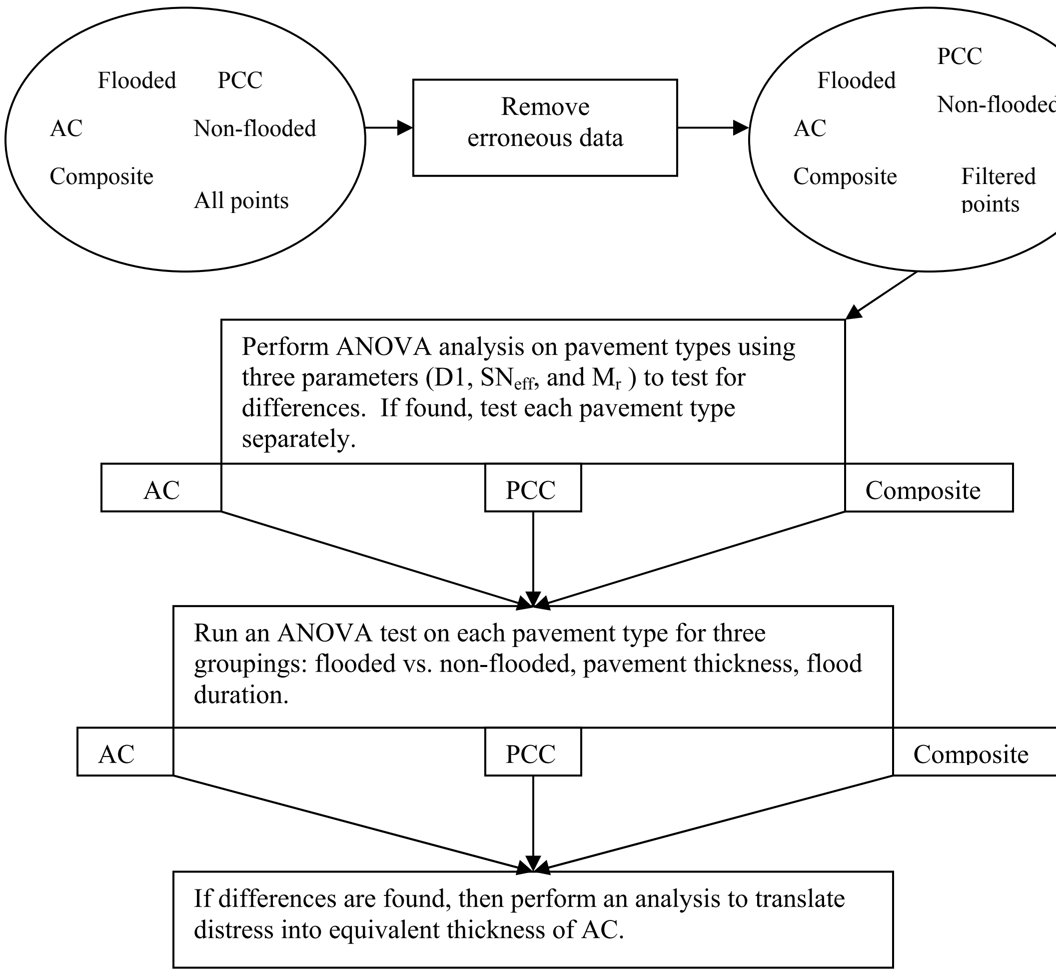


Figure 8

Analysis flow chart

DISCUSSION OF RESULTS

STATISTICAL ANALYSIS

Analysis of Pavement Type and Flood Type

A two-way ANOVA was performed on the entire data set for each of the test parameters. The main factors were pavement type and flood type with the interaction also examined. Table 1 presents the p-values. P-values less than 0.05 indicate that the factors are significantly different. These values have been bolded in the table. All parameters are significant for the two main factors of pavement type and flood type. This indicates that for each parameter, the pavement type and whether or not the pavement was submerged, the values do not come from the same data set; that is, they are different. The interaction factor shows no significant difference for the M_r parameter. This makes sense as the M_r parameter is probably independent of the pavement type. Because of this influence of pavement type, the remaining analyses were conducted by pavement type.

Table 1.

Two way ANOVA for Pavement Type and Flood Type

	p-value (n=2274)		
	Pavement type	Flood type	Ptype *Ftype
D1	**<0.0001**	**<0.0001**	**0.0005**
SN_{eff}	**<0.0001**	**0.0006**	**<0.0001**
Mr	**<0.0001**	**<0.0001**	0.7503

Analysis of Asphalt Pavements

Three factors were analyzed for the AC pavements: flood type (flooded vs. non-flooded), thickness of pavement structure, and duration of flooding. Table 2 presents the results of the ANOVA analysis. Significant differences were found in all three parameters for the flood type and thickness groups, while in the duration group, significant difference was found only in the M_r parameter. The physical meaning is that the pavement strengths, as reflected by D1 and SN, were not affected by the length of time being submerged, while the duration of submergence does appear to affect the subgrade resilient modulus.

Table 2

AC Pavement ANOVA Testing

	p-value		
	Flood type (n=881)	Thickness (n=881)	Duration (n=428)
D1	**<0.0001**	**<0.0001**	0.5660
SN_{eff}	**<0.0001**	**<0.0001**	0.9432
M_r	**<0.0001**	**<0.0001**	**0.0455**

As it was believed that the thinner asphalt pavements might be more susceptible to the flood waters than thicker pavements, ANOVA was used to look at three levels of thickness as defined in Table 3. Significant differences in performance between the flooded and non-flooded pavements were found at all levels of thickness except the thin pavements (<7in.) for the D1 and M_r parameters. This means that the D1 and M_r for thin pavements are similar for flooded or non-flooded pavements. However, a closer look at the data shows that there were only 18 points in the non-flooded category which is less than desirable for this type of analysis. Further, it noted that for the D1 parameter, the standard deviation for the flooded and the non-flooded categories are very high. This high variation in the data sets also leads to the lack of significant difference found in the statistical test. That the standard deviation for this parameter is so high in the flooded pavements indicates there is quite a difference in strength values in these thin sections and that any future rehabilitation should be evaluated on a project by project basis. Similarly, there was high variation found in the rather small data set for the non-flooded pavements for the M_r parameter. A test of the means for these thickness levels provided in Table 4 indicates that for the D1 and M_r parameters, the asphalt pavements could probably be defined into two groups as thin pavements (<7") and thick pavements (>7"), although the SN_{eff} had three distinct levels.

Table 3

ANOVA for Asphalt Pavements by Thickness

	Thickness Group	D1						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
AC	<7"	192	15.42	12.25	18	10.92	7.58	0.1390
	7" - 11"	303	8.48	5.67	118	7.26	4.73	**0.0394**
	> 11"	180	7.26	5.27	70	5.47	2.23	**0.0087**
	Thickness Group	**SN_{eff}**						
		Flooded			**Non-flooded**			**p-value**
		n	**mean**	**std dev**	**n**	**mean**	**std dev**	
AC	< 7"	192	4.54	2.02	18	5.56	3.65	**0.0477**
	7" - 11"	303	6.47	1.92	118	6.95	2.41	**0.0306**
	> 11"	180	7.40	2.29	70	8.27	2.25	**0.0123**
	Thickness Group	**Mr**						
		Flooded			**Non-flooded**			**p-value**
		n	**mean**	**std dev**	**n**	**mean**	**std dev**	
AC	< 7"	192	4.65	2.38	18	5.46	4.88	0.1892
	7" - 11"	303	5.66	2.38	118	6.86	2.81	**<0.0001**
	> 11"	180	5.83	2.45	70	6.86	2.89	**0.0046**

Table 4
Means Test for Asphalt Pavement Thickness

Group	Thickness Range	D1	SN_{eff}	M_r
1	<7" (n=210)	A	C	B
2	7"~11" (n=421)	B	B	A
3	>11" (n=250)	B	A	A
Note		"A" means highest D1	"A" means highest SN_{eff}	"A" means highest M_r

Analysis of PCC Pavements

Table 5 presents the results from the ANOVA tests. There were no significant differences found for any of the parameters based on flood type or duration under water. The thinner versus thicker PCC slabs do show a difference in performance for each parameter. This is verified in Table 6 where the ANOVA is conducted by pavement thickness. Within each thickness the parameters are similar. Table 6 provides the analysis by thickness for each parameter. As presented in Table 7, when testing the means for the two levels of thickness, each of the parameters is different. It appears that the PCC pavements do not appear to have sustained relative structural damage due to flooded versus non-flooded conditions. Similar to the asphalt data set, the number of samples for the non-flooded category is smaller than desired. The mean SN_{eff}'s is lower for the submerged pavements than the non-flooded pavements. The resilient modulus of the subgrade shows similar results between flooded and non-flooded. This may be because the subgrades were stabilized for these higher type pavements. It is noted that the mean values of M_r are higher than for the asphalt pavements. Similar to the asphalt pavements, duration of submergence is a non-factor for the PCC pavements.

Table 5

PCC Pavement ANOVA Testing

	p-value		
	Flood type (n=486)	Thickness (n=486)	Duration (n=251)
D1	0.0944	**0.0475**	0.5972
SN_{eff}	0.0782	**0.0233**	0.2455
Mr	0.7146	**0.0020**	0.9545

Table 6

ANOVA for PCC Pavements by Thickness

	Thickness Group	D1						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
PCC	< 10.5"	265	5.17	1.90	4	4.24	0.76	0.3322
	> 10.5"	176	4.78	2.07	41	4.53	1.09	0.4842
	Thickness Group	SN_{eff}						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
PCC	< 10.5"	265	7.67	1.57	4	8.27	1.27	0.4496
	> 10.5"	176	8.30	1.52	41	8.40	1.19	0.8851
	Thickness Group	M_r						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
PCC	< 10.5"	265	5.33	1.55	4	6.12	0.72	0.3113
	> 10.5"	174	5.85	1.89	41	5.55	1.15	0.3714

Table 7
Means Test for PCC Pavement Thickness

Level	Thickness Range	D1	SN_{eff}	Mr
1	<10.5" (n=269)	A	B	B
2	>10.5" (n=217)	B	A	A
Note		"A" means highest D1	"A" means highest SN	"A" means highest Mr

Analysis of Composite Pavements

Three factors were analyzed for the composite pavements including flood type, thickness, and duration of flooding. Table 8 presents the results from the ANOVA tests. These results are more complex than either the asphalt or PCC pavements. Table 9 presents the

ANOVA by pavement thickness. Table 10 provides the overall structures that represent these pavements. There is much masking of results occurring because of the relative strengths of the asphalt, PCC, brick and base course materials that constitute these pavements and their overall thicknesses. For the first time, duration of submergence provides significant differences in performance of D1 and M_r. These same parameters also show performance differences between flooded and non-flooded pavements. The SN_{eff} parameter seems to be masked by the strengths of the PCC layers although the thinner sections demonstrate significant difference in performance.

Table 8

Composite Pavement ANOVA Testing

	p-value		
	Flood type (n=907)	Thickness (n=907)	Duration (n=558)
D1	**0.0002**	0.2597	**0.0021**
SN_{eff}	0.5125	**0.0084**	0.5701
Mr	**<0.0001**	0.1101	**0.0211**

Table 9

ANOVA for Composite Pavements by Thickness

	Thickness Group	D1						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
COMP	< 16"	272	7.10	3.10	55	6.00	2.02	0.0059
	> 16"	439	5.61	2.72	132	5.02	2.53	0.0274
	Thickness Group	SN_{eff}						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
COMP	< 16"	272	7.00	1.81	55	7.13	1.54	0.0297
	> 16"	439	9.11	2.16	132	8.66	2.14	0.5656
	Thickness Group	M_r						
		Flooded			Non-flooded			p-value
		n	mean	std dev	n	mean	std dev	
COMP	< 16"	272	4.39	2.59	55	5.24	1.71	0.0280
	> 16"	439	5.20	1.89	132	6.82	2.24	<0.0001

Table 10

Composite Pavement Types

No:	Pavement sections
1	AC-PCC-BASE-PCC
2	AC-BRICK-PCC
3	AC-CRCP
4	AC-PCC-AC
5	AC-PCC-AC-PCC
6	AC-PCC
7	AC-PCC-BASE
8	AC-PCC-BASE-SUBBASE
AC thickness ranged from 1.5 to 19 in.	
PCC thickness ranged from 4 to 15 in.	

PAVEMENT STRUCTURAL DAMAGE ANALYSIS

Asphalt Pavement Analysis

Table 11 presents the amount of asphalt concrete mix that would be equivalent to the loss of structure in both the pavement and the subgrade for the submerged pavements using calculations provided in the Methodology section. Effective SN's from the FWD readings were converted to Dynaflect SN's according to the correlation provided in Appendix 3 for asphalt mixtures. The change in SN because of flooded versus non-flooded status was converted to inches of asphalt using an asphalt coefficient of 0.44. The subgrade modulus was converted to equivalent asphalt thickness according to Figure 7.

It is noted that the thinner pavement section requires more structure than the thicker sections. The overall equivalent thickness of 1.95 inches is similar to the analysis completed for LA 46 where "before and after" data was available. The loss of structure due to subgrade modulus is similar regardless of pavement thickness.

Table 11

Equivalent Thickness for Asphalt Pavements

Parameter	Thickness		
	All points	>7"	<7"
SN	1.23	0.64	1.51
M_r	0.72	0.60	O.57
Total	1.95	1.24	2.08

PCC Pavement Analysis

A cursory review of Table 6 shows that the thicker pavements have no loss of strength for SN_{eff} or M_r. For the thinner PCC pavements, the equivalent thickness of asphalt to account for the loss of structure because of the pavement and the subgrade are 0.43 inches and 0.47 inches, respectively. Other types of distress such as smoothness damage caused by debris haul trucks would be reason for heavier overlays.

Composite Pavement Analysis

Similar to the PCC pavements, the composite pavements which typically include one or two layers of PCC pavement demonstrate no need for additional structure in the pavement layers. However, the subgrade loss of structure accounts for the equivalent of 0.9 inches of asphalt concrete.

SUMMARY OF DAMAGE ASSESSMENT

For each pavement type, generally the thinner pavements experienced more relative damage for the submerged pavements than the non-submerged pavements. For asphalt pavements the damage analysis for the thinner sections would require at least two inches of asphalt and the actual before-after analysis would indicate three inches. Because of the variation in the D1 parameter for these pavements, more asphalt thickness may be required when evaluated on a project by project basis. Additionally, the analysis covered herein only accounts for the state highways New Orleans. There is another 300 miles of federal aid and 1500 miles of non-federal aid roads that were submerged in the greater New Orleans area.

Approximately 200 of the 238 miles evaluated in this study were submerged. Recent bid prices in New Orleans were received for $250,000 per mile for four inches of milling and a four inch overlay. If this is assumed typical, then the cost of rehabilitating the 200 miles of submerged state roads would be $50 million.

This study only considered the structural damage from a relative perspective of submerged and non-submerged pavements. There may well be damage to the non-submerged roads at the time the Fugro data was obtained, and there may be additional data because of the continuing debris truck hauling. No visual distress or smoothness damage has been addressed in this report because the "after" data has not yet been collected. As such, the findings of this study should be considered conservative.

CONCLUSIONS

This report evaluates data obtained under contract to Fugro to conduct structural testing of 238 miles of state highway pavements in the greater New Orleans area at 0.1 mile intervals. The results to date examine three structural parameters, D1, SN_{eff} and M_r. The data was divided into those pavements that were submerged under water for periods over three days and those pavements that were not submerged. This is not to imply that those pavements not submerged were not damaged by the hurricanes. The number and overweight loading of debris haul trucks immediately after the storms up until the time of testing several months later extracted a toll on the roadway system. Additional debris hauling continuing until today is causing additional damage.

The parameters were tested using analysis of variance techniques and testing of the means to test the hypothesis that the submerged pavements were weaker than the non-submerged pavements. Once this was accomplished, standard design methods were used to convert the difference in strengths to an equivalent depth of asphalt concrete representing that lost strength. These results were verified with an actual before-after style analysis on LA 46 in St Bernard Parish as before data was available from a research project on the roadway.

This study does not examine visual distress and smoothness data such as taken from the Pavement Management System as the after data is not yet available. Such an analysis should uncover additional damage. The results of this study should be considered conservative. Specific findings include:

1. Overall, pavements that were submerged were found to be weaker than non-submerged pavements for each of the strength parameters tested. There was a difference in strength values for each of the pavement types, asphalt concrete, PCC and composite pavements evaluated.

2. For the asphalt pavements, each of the strength parameters was weaker for the submerged pavements. Also there was a difference in these parameters depending on the thickness of the pavement.

3. The variation in the thinner asphalt pavements was very high for the D1 parameter indicating that any future rehabilitation or reconstruction design should be completed on a project by project basis.

4. The duration of submergence was not a factor for the asphalt pavements. Damage was sustained regardless of the length of time the pavement was submerged.

5. The overall equivalent strength loss for the asphalt pavements is similar to two inches of new asphalt concrete. This is similar to the three inch equivalency found for the before-after analysis conducted on LA 46. Note that the thinner pavements required more asphalt concrete than the thicker pavements.

6. PCC pavements demonstrated little relative loss of strength between those pavements that were submerged and the non-submerged pavements. While not significantly different, there is a reduced SN_{eff} for the submerged pavements. Similarly, duration of submergence was not a factor for the PCC pavements. As could be assumed, there was a difference in strength parameters based on thickness.

7. The M_r for PCC pavements is similar between the submerged and non-submerged pavements. In general, the M_r for the PCC pavements is higher than the asphalt pavements.

8. Although the loss of strength of the PCC pavements was minimal, other factors such as pavement smoothness, might require a thicker overlay.

9. The composite pavements demonstrated no need for additional structure in the pavement layers due to submergence. However, a weaker subgrade for the submerged areas is equivalent to 0.9 inches of asphalt concrete.

APPENDIX A

This appendix summarizes the LTRC testing results for eight roadways that were impacted by Hurricanes Katrina and Rita. Pavement evaluations were performed by LTRC on the eight routes as presented in Table A-1. LA 46 was the only route for which LTRC had pre-hurricane data.

Four methods were used to assess the pavement structure (FWD, Dynaflect, DCP, and Coring). Appendix B contains a description of each device and its pavement assessment capabilities.

Table A-1
Projects tested by LTRC

Route	Parish	Begin CSLM	End CSLM	Data prior to hurricane
LA 46	Orleans	1.400	4.500	Yes
I-10	Orleans	18.319	24.000	No
Tulane Ave	Orleans	0.000	1.080	No
S. Caliborne Ave	Orleans	2.040	3.030	No
Metarie Road	Jefferson	0.000	2.850	No
Metarie Road	Orleans	0.000	0.680	No
I-10	Jefferson	5.890	9.490	No
I-610	Orleans	0.000	4.520	No

LA 46
Orleans Parish

LA 46 was submerged by flood waters caused by Hurricane Katrina on August 29, 2005. Tests had been previously conducted with the Dynaflect on this roadway in August 2002, so a comparison of pavement conditions before and after the submersion based on Dynaflect data was possible. Table A-2 presents the existing typical sections as well as the limits of project testing. The typical section thicknesses were determined by coring.

Table A-2

LA 46 project limits and typical sections

4 lane section	3 lane section	
CSLM 1.4 to 3.1	CSLM 3.2 to 3.9	CSLM 4.0 to 4.35
4.5" to 6.5" AC	9.5" AC	16" AC
9" PCCP	7" PCCP	7" PCCP
Soil	Soil	Soil

Tests were conducted on October 18, 2005, at 0.1 mile intervals in the east-bound lane to match the locations that were tested in August 2002. There was a change in typical section due to a recent widening project from CSLM 3.2 to 4.35. Because of this, only comments about the change in subgrade conditions will be offered for this section.

In the four-lane section (CSLM 1.4 to 3.1), there was an average reduction of 0.9 SN/ in. in the structural number and 1.6 ksi in the subgrade, as presented in Table A-3. In the three-lane section, there was an average reduction of 0.6 ksi in the subgrade as presented in Tables A-4 and A-5.

It is evident from the test results that this roadway was weakened by the hurricane-flood waters. It would take an asphalt concrete overlay of approximately 3 in. to mitigate the damages ((0.9 SN/0.44) + (10 in-9.0 in. for M_r from Figure 7) = 2.04 + 1.0 = 3.0 in.).

Table A-3

LA 46 East Bound (4 lane section)												
	FWD				Dynaflect (AFTER)		Dynaflect (BEFORE)		DCP			
CSLM	5.5" AC	9" PCC	Soil	Void	SN	Soil (ksi)	SN	Soil (ksi)	Base		Soil	
1.400	1553.8	1899.6	7.9	0.2	5.1	5.4	5.3	5.4	DCPI	Mr (ksi)	DCPI	Mr (ksi)
1.500	1979.8	1325.6	8.1	0.3	4.9	4.9	4.4	6.9				
1.602	3271.6	1847.2	9.3	0.4	5.2	6.0	5.7	6.9				
1.701	803.0	59.0	11.8	1.8	2.5	5.0	5.3	6.3	31.5	4.0	16.5	7.6
1.800	1498.8	738.4	7.7	-0.4	4.2	4.6	5.5	6.5				
1.925	826.4	962.2	7.7	-0.1	4.8	4.3	5.3	6.1				
2.000	875.2	721.0	5.7	-0.4	4.3	3.8	5.2	6.0				
2.100	551.4	63.3	10.2	1.2	2.8	4.5	4.2	5.7				
2.200	1386.1	429.4	6.7	0.3	4.5	4.7	5.1	5.8				
2.301	2208.5	304.4	5.0	1.5	4.3	3.8	4.6	5.5				
2.400	1548.8	583.9	7.3	0.2	4.8	4.3	5.2	6.5				
2.500	1152.3	970.0	8.3	0.7	4.3	5.8	5.3	6.8				
2.600	2768.4	873.3	8.9	0.8	4.7	4.9	5.2	6.5	38	3.3	17.8	7
2.700	3685.6	132.3	7.4	1.0	4.0	4.5	5.1	7.4				
2.800	1535.1	583.9	6.0	0.7	3.9	4.1	5.0	6.9				
2.900	548.3	169.2	8.2	0.8	3.3	4.6	5.3	7.2				
3.000	1494.8	1011.8	10.6	0.6	4.5	6.0	5.6	5.7				
3.100	319.8	1140.7	13.3	0.7	4.3	4.9	5.1	6.8				
Avg	1556.0	767.5	8.3		4.2	4.8	5.1	6.4				

Table A-4

LA 46 East Bound (3 lane section)												
	FWD				Dynaflect (AFTER)		Dynaflect (BEFORE)		DCP			
CSLM	9.5" AC	7" PCCP	Soil	Void	SN	Soil (ksi)		Soil	Base		Soil	
3.200	83.9	34.1	14.7	4.9	3.7	6.4		7.7	DCPI	Mr (ksi)	DCPI	Mr (ksi)
3.300	664.7	9155.2	12.5	0.3	6.0	7.6		6.3				
3.400	444.3	6093.7	13.8	0.3	5.3	6.1		8.0				
3.500	317.4	10366.6	10.6	0.8	5.8	6.7		8.2				
3.600	415.1	12260.3	12.5	0.3	6.3	8.0		7.3				
3.700	583.9	8035.1	12.0	0.2	5.9	7.0		8.0				
3.801	885.7	994.2	17.1	0.5	5.4	7.0		5.2	37.0	3.4	24.1	5.2
3.900	638.5	8732.4	11.8	0.4	5.9	7.0		7.3				
Avg	504.2	6958.9	13.1	1.0	5.5	7.0		7.3				

Table A-5

LA 46 East Bound (3 lane section)												
	FWD				Dynaflect (AFTER)		Dynaflect (BEFORE)		DCP			
CSLM	16" AC	7" PCCP	Soil	Void				Soil	Base		Soil	
4.000	608.4	1267.5	11.8	0.4	5.7	7.8		7.7	DCPI	Mr (ksi)	DCPI	Mr (ksi)
4.100	825.0	923.2	8.2	0.5	5.6	6.4		7.9				
4.200	502.1	3546.3	8.9	0.6	5.9	7.2		7.8				
4.301	496.3	1662.9	11.3	0.5	5.3	7.4		7.8	75	1.7	45.1	2.8
4.350	1817.5	97.1	9.7	0.0	5.9	6.1		7.8				
Avg	849.8	1499.4	10.0	0.4	5.7	7.0		7.8				

I-10 (Michoud Blvd. to Lake Ponchatrain)

This section of I-10 was submerged for several weeks by the storm surge resulting from Hurricane Katrina on August 29, 2005. The existing roadway has three lanes in each direction. Its typical section, according to the plans, is 10 in. PCCP, 6 in. cement-treated sand shell, 6 in. sand shell, and sand embankment. The pavement thickness was verified by cores and a conversation with the District 07 laboratory.

Satellite imagery taken on August 31, 2005, two days after Hurricane Katrina, was reviewed. The images indicated that the entire road had been submerged and some portions of the roadway were still submerged during the day of testing. Tests with the FWD and Dynaflect were conducted on both the east and west bound lanes at 0.3 mile intervals on September 29, 2005. Tests were conducted in the outside wheel path of the outside lane. Once the data were reduced, DCP tests were selected at specific locations to validate FWD and Dynaflect results. DCP tests were conducted on October 4, 2005.

Most of the project site had water near or on the shoulder during testing. Three zones were observed during testing. Zone 1 began at CSLM 18.319 and ended at approximately CSLM 22.500. This zone appeared to have little embankment fill and water was observed at or near the shoulder. Zone 2 began at approximately CSLM 22.500 and ended at approximately 23.200. It had an embankment fill ranging from 3 to 5 ft. The soil cement and subgrade were weaker than Zone 3. The last zone, Zone 3 began at 23.000 and ended at 24.300. It had an embankment fill ranging from 3 to 5 ft.

Zone 1 (CSLM 18.319 to 22.500)

The modulus values of the concrete pavement and sand embankment for both the east and west bound lanes are adequate, see Tables A-6, A-7.

The average cement treated sand shell base course modulus was adequate in the east-bound lanes, except for one area (CSLM 20.000 to 21.500). The west-bound lanes have extremely weak cement treated sand shell base course. The westbound pavement SN is 0.4 – 1.0 SN weaker than all other sections because of the reduced structure provided by this base course.

Zone 2 (CSLM 22.500 to 23.200)

The modulus values of the concrete pavement for both the east and west bound-lanes were adequate. The average cement treated sand shell base course modulus was acceptable in the east bound lane while the west bound lane was lower than normal. The average modulus values for the sand embankment are representative for that material. Overall, the westbound roadway was weaker than the eastbound roadway.

Zone 3 (CSLM 23.000 to 24.300)

The modulus values of the concrete pavement for both the east and west bound lanes were adequate. The average cement treated sand shell base course modulus was above standard. The average modulus values for the sand embankment are representative for that material, but lower in the westbound roadway in comparison with the eastbound roadway.

Load transfer at joints (all zones)

Load transfer efficiency values greater 70 percent are considered good, 50 to 70 percent are considered fair, and less than 50 percent are considered poor. Based on the test results, no joints were in the poor range, (Tables A-7, A-9). Joints with less than 70 percent load transfer efficiency should be repaired prior to asphalt concrete overlay.

Voids under concrete pavement (All zones)

Testing was conducted at both the joints and midslab. Only one location (CSLM 21.416, east bound lane, joint) out of the 68 test points, indicated that voids may be present.

Table A-6

I-10 East Bound (midslab)										
	FWD				Dynaflect		DCP			
CSLM	10" PCCP	6" CTB	Soil	Voids	SN	soil	Base		Soil	
	(ksi)	(ksi)	(ksi)			(ksi)	DCPI		DCPI	
18.319	4837.8	239.3	11.6	0.3	5.2	7.9	*0.3*		*2.2*	
18.696	4629.5	298.9	11.6	-0.2	5.3	7.9				
18.974	5331.2	203.0	14.1	0.1	5.2	8.7				
19.316	4610.3	522.6	13.0	0.1	5.5	8.1	*X*		*2.6*	
19.561	3724.2	452.2	14.2	0.4	5.1	8.5				
20.090	8916.4	226.0	11.8	0.2	5.9	9.0				
20.331	4991.9	161.7	11.7	0.3	4.9	7.6	*X*		*3.2*	
20.632	8329.7	7.4	15.1	0.0	5.8	8.4	*1.1*		*2.4*	
21.120	5793.2	6.2	14.9	-0.3	4.9	7.2				
21.421	4359.2	259.6	8.1	0.6	4.8	6.9	*X*		*2.4*	
21.724	5079.5	292.4	12.6	0.2	5.7	9.3				
22.001	4816.5	526.5	12.7	0.0	5.7	8.8	*0.2*		*3.7*	
22.357	4505.6	384.3	13.8	0.0	5.6	9.0				
	5378.8	275.4	12.7		5.4	8.3				
22.755	4519.0	194.3	17.7	0.0	5.5	14.0	*X*		*3.6*	
23.065	5045.7	212.2	15.5	0.1	5.6	12.0				
	4782.4	203.3	16.6		5.6	13.0				
23.287	4704.7	249.3	15.5	0.1	5.8	12.0	*1.8*		*3.9*	
23.999	2528.1	939.8	18.9	0.1	5.6	13.0				
	3616.4	594.5	17.2		5.7	12.5				

Table A-7

I-10 East Bound (joints)				
FWD			Dynaflect	
CSLM	Voids	LTE	SN	Soil
		%		(ksi)
18.314	0.8	93	3.8	8.5
18.691	0.4	88	2.9	5.8
18.968	0.4	86	2.1	6.1
19.312	1.0	82	2.8	7.2
19.556	0.1	74	2.8	6.0
20.084	0.2	86	2.5	5.0
20.326	0.4	76	3.2	6.3
20.626	0.6	81	3.0	6.4
21.114	1.1	82	3.2	10.0
21.416	2.0	66	2.6	8.0
21.718	0.7	99	3.1	8.2
21.996	0.9	80	2.7	7.6
22.351	0.8	79	3.2	4.2
			2.9	6.9
22.750	-0.1	84	3.2	10.0
23.059	0.9	65	2.6	8.0
			2.9	9.0
23.281	0.5	82	3.1	8.3
23.991	0.0	76	3.1	8.2
			3.1	8.3

Table A-8

I-10 West Bound (midslab)										
	FWD				Dynaflect		DCP			
CSLM	10" PCCP	6" CTB	Soil	Voids	SN	soil	Base		Soil	
	(ksi)	(ksi)	(ksi)			(ksi)	DCPI		DCPI	
18.576	5565.6	10.6	13.3	0.1	4.8	7.1				
19.031	4213.5	40.1	11.9	0.1	4.4	6.0				
19.312	2647.2	237.0	12.4	0.7	4.5	7.0				
19.673	3171.8	156.6	13.1	0.5	4.8	6.5				
19.974	5657.4	14.4	12.3	0.3	4.9	6.9				
20.416	4248.2	20.9	10.6	0.5	4.4	6.3				
20.779	3337.9	22.0	9.9	0.1	4.0	4.5				
21.119	5523.4	2.9	15.0	0.2	5.0	8.1				
21.432	5803.2	7.1	12.8	0.0	4.9	7.3				
21.764	4946.5	52.0	12.4	0.4	4.9	7.9				
22.074	3482.7	1.3	26.2	-0.1	4.7	7.6	0.4		2.7	
22.386	3598.8	23.8	14.2	0.2	4.5	6.6				
	4349.7	49.1	13.7	0.1	4.7	6.8				
22.774	3549.6	124.9	13.6	0.1	5.0	8.9				
23.081	4370.5	58.5	16.7	0.1	5.1	10.0				
	3960.0	91.7	15.1		5.1	9.5				
23.403	2608.6	262.3	12.3	0.2	4.8	8.1				
23.753	2850.8	823.9	14.2	0.3	5.3	9.2				
24.130	3549.7	954.4	6.4	0.2	5.1	7.5				
	3003.0	680.2	11.0		5.1	8.3				

Table A-9

I-10 West Bound (joints)				
FWD			Dynaflect	
CSLM	Voids	LTE	SN	Soil
		%		(ksi)
18.581	0.6	68	3.4	5.6
19.036	-0.1	76	3.2	7.0
19.317	0.1	71	3.8	7.9
19.680	0.1	90	4.0	8.5
19.980	-0.1	97	4.0	5.8
20.421	-0.3	97	4.0	8.0
20.781	0.3	66	4.1	7.4
21.125	-0.1	87	3.6	5.0
21.438	-0.2	96	4.9	12.0
21.770	0.1	100	3.6	9.8
22.080	-0.2	71	3.8	9.8
22.391	0.7	58	3.6	7.0
			3.8	7.8
22.780	-0.2	93	4.9	12.0
23.087	0.0	92	4.2	9.6
			4.6	10.8
23.409	0.1	73	3.6	9.8
23.758	-0.1	88	3.8	9.8
24.134	-0.1	92	5.0	13.0
			4.1	10.9

Tulane Ave
Orleans Parish

Tulane Ave. was submerged by flood waters resulting from Hurricane Katrina on August 29, 2005. Testing was conducted from Broad St. (CSLM 0.000) to S. Carrolton (CSLM 1.080). The existing roadway consists of two lanes in each direction. It is a composite section consisting of 2 in. of asphalt concrete over 8 in. of PCC. It has a base course of sandy material, which ranges from 9 to 19 in. The thickness of the pavement was verified with cores, and the base course thickness was extrapolated from DCP readings.

Tests with the FWD and Dynaflect were conducted on both the east and west bound lanes at 0.1 mile intervals on October 13, 2005. Tests were conducted in the outside wheel path of the outside lane. DCP tests were selected at specific locations to validate FWD and Dynaflect results. DCP tests and coring were conducted on October 17, 2005.

The test results indicated the subgrade soil was extremely weak in both the east and west bound lanes as presented in Table A-10. The weakness in the subgrade could be attributed to the saturation in the soil caused by being submerged for several weeks. The base course on the east and west bound lanes was in a weakened condition in 50 percent of the locations tested. The composite pavement has reasonable values and voids may be present at three locations.

Table A-10

Tulane Ave										
	FWD				Dynaflect		DCP			
CSLM	10" Comp	11" Base	Soil	Void	SN	Soil	Base		Soil	
EB	(ksi)	(ksi)	(ksi)				DCPI	Mr (ksi)	DCPI	Mr (ksi)
0.115	3964.4	25.8	3.5	1.6	4.0	2.6				
0.229	1049.9	150.4	4.0	1.7	4.4	2.6				
0.351	2860.3	56.4	3.5	1.7	5.0	3.3				
0.464	1446.2	206.0	4.1	1.6	4.5	2.7	8	15.8	79	2.2
0.651	2321.3	251.4	2.6	0.3	4.2	2.7				
0.783	1211.4	228.5	3.5	0.4	4.1	2.6				
0.911	3286.3	13.4	3.0	1.6	4.7	3.0				
Avg	**2305.7**	**133.1**	**3.5**		**4.4**	**2.8**				
WB										
0.051	2758.4	116.5	1.9	1.0	4.5	2.6				
0.157	2323.3	3.3	8.5	**2.7**	3.9	2.3				
0.265	3268.3	61.4	1.7	1.5	4.6	2.5	13	9.7	82	1.7
0.385	1735.9	157.2	2.1	1.7	4.3	2.3				
0.483	1760.9	132.0	3.4	1.1	4.4	2.4				
0.577	4714.2	25.9	2.7	0.7	4.8	3.3				
0.696	4574.4	19.6	3.1	1.1	4.7	2.7	9.0	14	74	2.3
0.821	1919.5	111.0	4.0	**2.2**	4.9	3.2				
0.911	2474.0	3.2	6.7	**2.1**	3.7	2.9				
	2836.5	**70.0**	**3.8**		**4.4**	**2.7**				

S. Claiborne Ave.
Orleans Parish

S. Claiborne Ave. was submerged by flood waters resulting from Hurricane Katrina on August 29, 2005. Testing was conducted from State St. (CSLM 2.040) to Washington St. (CSLM 3.030). The existing roadway consists of three lanes in each direction. It was a composite section consisting of 9 in. of asphalt concrete over 8 in. of PCC. It appears to have no base course according to DCP readings. The thickness of the pavement was verified with cores.

The test results indicate the subgrade soil was extremely weak in both the east and west bound lanes. Both the asphalt concrete and underlying concrete pavement show signs of serious distress in the majority of the test areas as presented in Table A-11.

Voids may be present at 50 percent of the locations tested. The voids should be verified by cores.

Table A-11

S. Claiborne Ave								
	FWD				Dynaflect		DCP	
CSLM	9" AC	8" PCC	Soil	Voids	SN	soil	soil	Mr
EB	(ksi)	(ksi)	(ksi)			(ksi)	DCPI	Mr (ksi)
2.072	2376.8	4.9	5.0	2.2	3.4	2.0		
2.243	299.3	145.8	6.5	2.1	3.3	3.1		
2.389	143.3	683.8	7.2	2.2	3.8	3.8	45	2.8
2.495	838.5	1752.3	5.6	0.8	4.7	3.3		
2.636	144.3	77.7	7.2	1.7	2.9	2.2	51	2.5
2.773	82.0	449.3	5.1	3.0	3.0	1.2		
2.891	399.2	5615.6	6.2	0.5	5.3	4.4		
3.058	115.2	30.9	6.3	4.9	2.2	2.3		
AVG	549.8	1095.0	6.1		3.6	2.8		
WB								
2.167	1358.0	752.8	3.6	0.8	3.4	2.2		
2.354	276.7	6472.4	7.3	0.5	3.7	2.9		
2.513	144.6	67.2	7.2	2.3	3.1	1.2	13	9.6
2.679	548.2	128.0	6.6	1.1	3.4	1.1		
2.844	220.4	723.6	2.7	2.3	3.9	3.0		
3.028	80.2	86.0	5.2	4.9	2.8	3.3		
3.185	99.7	869.2	6.2	1.7	5.3	5.0		
3.243	160.1	415.8	5.4	1.9	4.5	2.8		
Avg	361.0	1189.4	5.5		3.8	2.7		

I-610
Orleans Parish

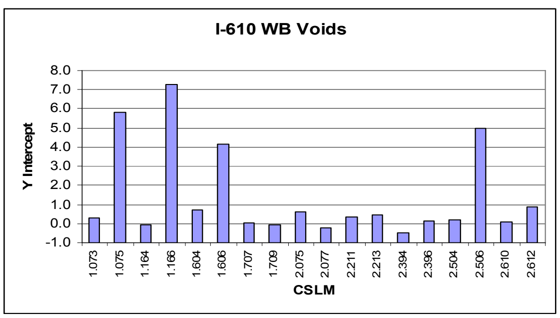
A pavement assessment was performed on I-610 from CSLM 0.000 to CSLM 4.520 on November 29, 2005. According to the as-built plans, the typical section for this project consists of 10 in. PCC, 6 in. cement treated sand-shell, and sand embankment. Tests were conducted on the inside lane on both the east and west bound roadways.

The test results as shown in Table A-12 indicated that the pavement structure was sound on both roadways. There are a few locations indicating damage to the cement treated sand shell base course (<100 ksi); the subgrade strength was acceptable. The concrete pavement layer modulus was acceptable and the overall structural number was representative of a pavement of its age.

As presented in Figure A-1, 8 out of the 35 test points indicated voids were present under the pavement. Figure A-2 presents the results of the load transfer efficiency of the joints. Fifty-five percent of the joints in the west bound lane and 89 percent of the joints in the east bound lane were in failure mode.

Table A-12

I-610 Orleans Paish 11-29-2005					
	FWD			DYNAFLECT	
CSLM	10" PCC	6" CTB	Soil	SN	Soil
EB	ksi	ksi	ksi		ksi
1.101	3179.1	561.1	8.6	4.9	7.9
1.154	5541.5	130.6	9.1	5.2	8.8
1.587	4706.6	367.9	10.8	4.9	6.6
1.606	6100.4	222.9	7.3	5.0	6.4
1.659	5393.3	550.1	8.4	4.9	6.1
2.18	6905.8	41.5	12.4	4.9	5.5
2.211	5192.0	27.1	10.1	4.6	4.9
2.37	5730.4	865.9	11.7	5.1	6.7
2.517	4537.0	96.6	9.8	4.8	5.6
AVG	**5254.0**	**318.2**	**9.8**	**4.9**	**6.5**
WB					
1.073	5333.7	100.9	7.9	5.1	5.9
1.164	5315.6	144.7	9.8	5.1	6.9
1.604	8080.0	28.0	8.0	5.5	8.1
1.707	6252.7	59.0	11.0	4.7	6.7
2.075	4069.6	353.9	11.8	4.8	6.1
2.211	4100.5	422.7	7.0	4.7	5.3
2.394	5202.6	164.0	17.6	4.9	6.2
2.504	3272.9	581.2	8.2	4.4	4.9
2.610	4726.5	556.8	12.9	4.4	6.9
AVG	**5150.5**	**267.9**	**10.5**	**4.8**	**6.3**



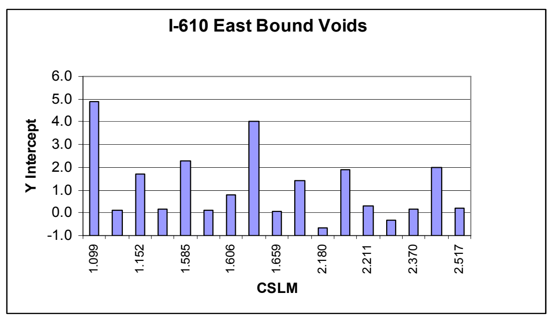
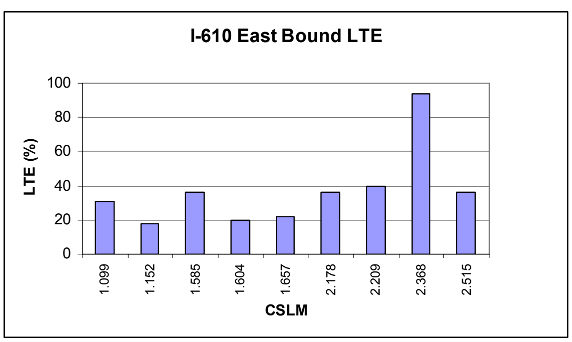


Figure A-1



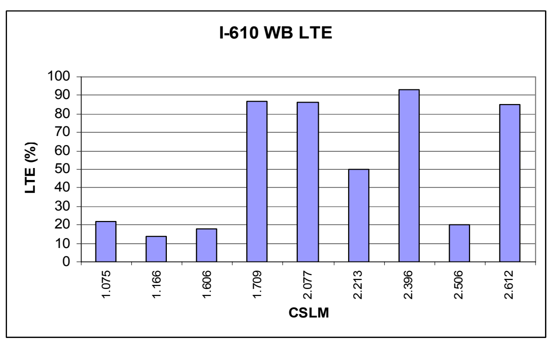


Figure A-2

Metairie Road

Jefferson Parish

A pavement assessment was performed on Metairie Road from Severn Ave. (CSLM 0.00 to the Orleans Parish line (CSLM 2.850) on November 2, 2005. This project was under construction. According to the typical section design provided by the LaDOTD Pavement Design Engineer, a structural number of 3.87 is required for the projected traffic loading and a subgrade with a resilient modulus of 8 ksi.

The construction plans included the removal and replacement of 2 in. of asphalt from CSLM 0.000 to 0.080 and 3.5 in. of asphalt for the remainder of the project.

Testing was conducted with FWD, Dynaflect, DCP and coring. A core taken on this roadway indicated an 11.5 in. thick asphalt concrete pavement. The top 2.5 in. of asphalt was in good condition while the remaining 8.5 in. was stripped and crumbled when touched. The FWD test results indicated that the top 2.5 in. of asphalt was in good condition and that the remaining 8.5 in. was in poor condition for asphalt pavement as presented in Table A-13. In fact, the bottom 8.5 in. of asphalt was performing similar to a stone base course due to the poor condition of the asphalt. If standard design values were used for asphalt (0.44 SN/in.) and stone (0.14 SN/in.), then the existing typical section should have an SN of 2.38 (2.5 in. x 0.44 + 8.5 in. x 0.14 = 2.38 SN). According to the Dynaflect readings, the average SN for both the east and west bound lanes was 2.8 and that was within the range that was predicted (2.38) based upon observations from the core. If the 11.5 in. of asphalt were in good condition, then the overall SN would be approximately 5.6.

Since the required SN was 3.87 and the existing SN was 2.38, there was a 1.49 loss in SN which equates to the structural equivalent of 3.5 in. of asphalt concrete.

The typical section design was based on a subgrade resilient modulus of 8 ksi and the test results indicate that the existing resilient modulus was 4 ksi. A Darwin analysis reducing the design subgrade resilient modulus by 4 ksi would require a total structural equivalency of 6 in. of asphalt concrete.

Table A-13

Metarie Road (826-04) Jefferson Parish									
	FWD			DYNAFLECT		DCP			
CSLM	3" AC	8.5" stone	Soil	SN	Soil	Base		Soil	
EB	ksi	ksi	ksi		ksi	DCPI	Mr (ksi)	DCPI	Mr (ksi)
0.134	4574.3	330.0	9.6	3.1	3.9				
0.485	2260.8	32.1	9.4	2.4	4.3				
0.697	4353.6	558.3	9.4	3.5	4.3				
0.879	1410.9	45.3	9.1	2.2	4.7				
1.114	2694.2	154.2	10.4	3.0	4.1				
1.360	4385.9	210.4	8.9	3.3	4.0				
1.701	3301.4	47.7	9.5	2.4	4.9				
1.938	4528.9	1104.2	9.8	3.8	4.2				
2.129	542.5	44.3	11.7	2.7	5.8				
2.432	1279.1	64.9	9.1	2.3	4.0				
2.689	3649.6	908.7	7.9	3.7	3.6	3.8	N/A	36	3.5
AVG	**2998.3**	**318.2**	**9.5**	**2.9**	**4.3**				
WB									
0.355	697.4	50.1	8.4	1.6	3.5				
0.578	909.6	14.1	6.8	1.2	3.2				
0.757	1138.5	54.0	10.0	1.1	4.8				
0.992	5630.3	1172.7	10.8	3.9	4.2				
1.243	1079.2	15.9	9.2	1.2	4.4				
1.474	2481.6	65.4	10.8	2.8	4.3				
1.779	1424.1	55.4	9.1	2.1	3.7				
1.971	3968.5	511.4	9.7	3.4	3.7				
2.301	6595.8	1230.5	12.0	4.1	4.3				
2.529	945.5	44.1	7.4	4.8	4.4				
2.717	747.1	41.1	11.0	3.8	2.6				
AVG	**2328.9**	**295.9**	**9.6**	**2.7**	**3.9**				

Metairie Road
Orleans Parish

A pavement assessment was performed on Metairie Road from the Orleans Parish line (CSLM 0.000) to I-10 (CSLM 0.680) on November 2, 2005. This area was submerged during Hurricane Katrina. Unlike Metairie Road in Jefferson Parish, neither traffic data nor the typical section design was available. One core was taken in the east bound lane. It consisted of 15 in. of asphalt that appeared to be in good condition. The west bound lane was a composite pavement section consisting of asphalt concrete and PCC. Since cores were not taken in the composite section, the thickness of the asphalt and concrete is unknown and comments about the west bound lane will not be offered.

If it is assumed that Metairie Road in Orleans Parish has traffic similar to Jefferson Parish, the required SN would be 4.92 for a subgrade resilient modulus 4 ksi. According to the test results, which are presented in Table A-14, the average SN was 1.5 and the subgrade was approximately 4 ksi. The FWD results indicate that the asphaltic concrete layer modulus was completely deteriorated (75 ksi). For a pavement that has 15 in. of asphalt, which equates to an SN of 6.6 for new pavement, this pavement structure was extremely weak.

Table A-14

Metairie Road (836-05) Orleans Parish								
CSLM	FWD		Dynaflect		DCP			
EB	15" AC	Soil	SN	Soil	Base		Soil	
	ksi	ksi		ksi	DCPI	Mr (ksi)	DCPI	Mr (ksi)
0.165	82.7	5.7	1.6	3.7				
0.340	80.2	5.3	1.7	3.4				
0.430	78.0	5.4	1.2	3.9				
0.580	57.2	5.9	1.3	3.9	11.0	N/A	21.0	6.0
AVG	**74.5**	**5.6**	**1.5**	**3.7**				

I-10 (Causeway Blvd. to 17Th Street Canal)
Jefferson Parish

A pavement assessment was performed on I-10 from the Causeway Blvd. (CSLM 5.890) to the 17th Street Canal (CSLM 9.490) on November 29, 2005. The existing pavement typical section on the inside lane of the west bound lane is 25 in. full-depth asphalt concrete, and untreated subgrade. The plans indicate that there is approximately 9 in. of wearing course, 6 in. of binder course and 12 in. of base course. The typical section varies in the other lanes due to slope correction, asphalt overlay of the existing concrete pavement, and the addition of ramps. Tests were conducted on the inside lane only.

The asphalt pavement is aged and showing signs of distress. If this were a newly constructed pavement, an SN of 10.56 (9 in. AC wearing * 0.44 SN/in. + 6 in. AC binder* 0.44 SN/in. + 12 in. AC base course * 0.33 SN/in) would be expected.

According to the current construction plans, an SN of 6.96 is required and approximately 4 in. of the existing pavement will be removed and replaced under this project. The Dynaflect test results indicated that the average SN for both the east and west bound lanes is 4.8 as shown in Table A-15. Based on the thickness of this section, the corresponding existing SN is 0.19.

The current asphalt replacement plan for this project will not address the structural deficiency (SN 6.96 required – SN 4.8 existing = SN 2.16 deficit). Using a coefficient for the existing asphalt of 0.19 SN/in. and 0.44 SN/in. for the new pavement, would require that 9 in. of the existing asphalt concrete pavement needs to be removed and replaced to meet the required SN of 6.96 for the rehabilitated full-depth asphalt concrete section. A similar analysis would need to be conducted for the other lanes which are composite pavement sections.

Table A-15

I-10 SP 450-15-0089 Jefferson Parish				
Location	**FWD**		**Dynaflect**	
East	**25" AC**	**Soil**	**SN**	**Soil**
Bound	ksi	ksi		ksi
7.700	507.9	10.9	5.3	12.0
7.800	955.7	5.1	5.5	9.8
7.900	270.4	12.1	4.2	7.6
8.000	994.9	9.0	5.1	6.4
8.100	245.2	6.4	n/a	n/a
8.500	507.3	8.4	5.0	6.9
8.602	339.5	10.0	4.1	4.5
8.700	1287.1	6.1	5.2	4.6
9.200	838.5	9.1	5.4	11.0
9.301	1941.1	12.8	n/a	n/a
AVG	**788.8**	**9.0**	**5.0**	**7.9**
West				
Bound				
7.600	777.3	10.9	5.2	9.9
7.700	494.3	11.7	5.2	12.0
7.800	744.0	11.1	5.3	9.8
7.900	724.4	10.3	4.8	7.5
8.000	719.0	7.7	4.4	4.7
8.100	787.6	6.9	4.3	3.8
8.500	637.9	7.1	4.6	5.6
8.600	500.3	10.0	4.4	4.7
8.700	699.9	5.2	3.8	4.0
AVG	**663.4**	**8.8**	**4.6**	**6.5**

APPENDIX B

THE FOLLOWING TASKS WERE PERFORMED BY FUGRO CONSULTANTS FOR LADOTD:

1 GROUND PENETRATING RADAR TESTING
2 NONDESTRUCTIVE DEFLECTION TESTING (FWD)
3 CORING AND DCP
4 DATA ANALYSIS AND REPORTING

This document was prepared to summarize the essential activities performed for completing each of the tasks noted above. A detailed QC Plan was prepared separately to specify how each of these tasks was carried out and checks to confirm they were properly completed.

1. Ground Penetrating Radar (GPR) Testing

The following activities were completed during the proposed GPR testing

1.1 Testing procedures
1.2 Equipment setup
1.3 Control sections
1.4 Event markers
1.5 File naming
1.6 File back-up

Objective: The purpose of the GPR test program was to establish structural profiles of the selected pavement structures, voids beneath the concrete pavements, as well as variability of the structural profile along the pavement facilities.

1.1 Testing Procedures

Data was available at ½ foot intervals. The GPR data used was then matched to the GPS coordinate of the FWD testing every FWD drop point to the middle of the outside lane. The acceleration/deceleration lanes were not tested. Divided highways were tested in both primary and alternate directions. Undivided roadways were tested in the primary direction. Sections were driven from start to end with some cushion before the start and beyond the end of the section to ensure the entire section was tested. Data that lied within the section was then exported in the office and DMIs assigned to GPR data.

1.2 Equipment Setup

The GPR survey was carried out using air-coupled 1 GHz horn antenna and air-coupled 2-GHz antenna. The antennae were as mounted to a vehicle traveling at normal driving speed. The GPR equipment consisted of a GSSI SIR-20 radar control and data acquisition unit, a Model 4108 or Model 4108f 1-GHz antenna and a GSSI Model 4105f 2-GHz antenna. This equipment was approved and licensed by the FCC. GPS data was collected concurrently with the GPR data collection. The data collection rate was controlled by the same DMI controlling the GPR data collection. Prior to the start of the survey, the vehicle DMI was calibrated to a known distance. During the survey, markers were placed in the GPR data at mileposts and other event markers.

1.3 Control Sections

All testing was performed based on control section boundaries. Control section maps provided by the DOT were used to determine the section limits. Operators ensured that testing was performed over the entire test section. Comments were provided for any deviation, either in length or travel lane from the original specification of the testing procedure.

1.4 Event Markers

It was understood that the following additional items were to be noted (in the GPR field notes) during the course of the GPR testing:

1.4.1 Surface Type:
1.4.1.1 A - Asphalt Concrete (AC)
1.4.1.2 J - Jointed Portland Cement Concrete (JCP)
1.4.1.3 C - Continuously Reinforced Concrete (CRC)
1.4.1.4 COMP – Composite Pavement
1.4.2 Date - Include the date that the GPR data was collected
1.4.3 Events - The following events on the DEPARTMENT's highway network shall be marked on the corresponding GPR trace number
1.4.3.1 Mileposts on Interstates
1.4.3.2 Every surface type change
1.4.3.3. The beginning and ending points of every bridge
1.4.3.4 The beginning and ending point of any segment of highway that is under construction or marked for construction along the highway

1.5 File Naming

The file name included the route number and was stored as an .mdb file (i.e. LA 4789 LM XXX to LM XXXX).

1.6 File Back-Up

At the end of each day of testing, a backup copy of all data collected was made. Verification that all data was collected and stored on the hard drive of the data collection computer was performed after the testing of each control section was complete before leaving the area of that control section. The file back-up was conducted each evening. All files were copied to a portable USB hard drive.

All hard drives were kept secure, with original paperwork and activity forms filled out during the day of testing. Operators stored all data drives and forms until the end of data collection, at which time they took them back to the office after a thorough QC review that all data was collected.

2. Nondestructive Deflection Testing (FWD)

The following activities were anticipated for completing the FWD testing proposed:

2.1 Testing procedures
2.2 Equipment setup
2.3 Control sections
2.4 Event markers
2.5 File naming
2.6 File back-up

Objective: The purpose of the deflection test program was to determine the structural response characteristics of the pavement structure and the underlying subgrade materials to wheel loads as well as variability of the structural properties along the pavement facilities and to provide LaDOTD with the resources to perform back-calculation analyses at their discretion.

2.1 Testing Procedures

The deflection testing program was performed in accordance with ASTM Test Standard D4694 (Standard Test Method for Deflections With a Falling Weight-Type Impulse Load Device) and D4695 (Standard Guide for General Pavement Deflection Measurements). The type of testing conducted was a Level 1 program, for a network level evaluation of pavement condition.

Test Spacing was conducted every tenth mile or three-tenths mile (as determined by LaDOTD) offset to the middle of the outside lane. The acceleration/deceleration lanes were not tested. Divided highways were tested in both primary and alternate directions. Undivided roadways were tested in the primary direction. DMI settings were set to '0' at the beginning of each control section in the primary direction and not reset until testing was complete for that control section and direction. The DMI was set to ending point on the return for the secondary direction and counted downward (the stationing was maintained, increasing in primary direction for both directions tested).

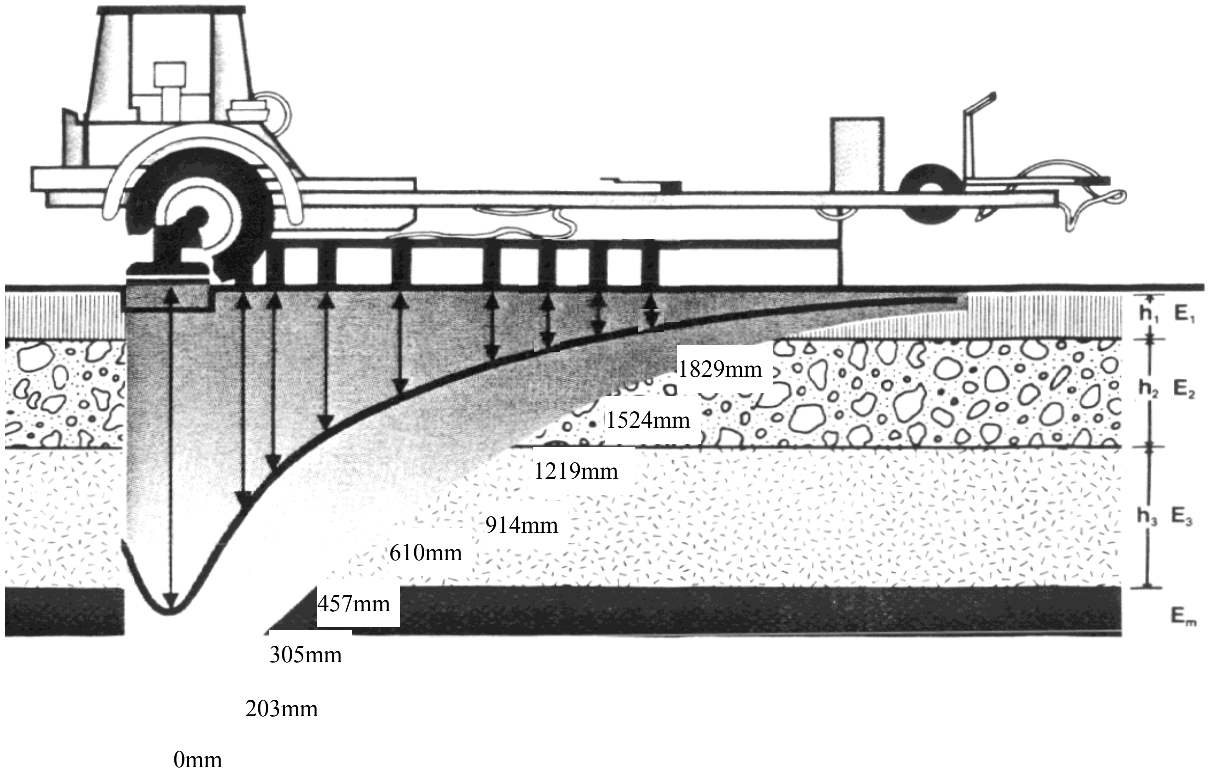
2.2 Equipment Setup

The operator ensured that the FWD is set up for testing in the following manner.

The drop sequence was as follows:

No. of Drops	Force	Stored
1	9K	No (seating)
2	9K	Yes
1	12k	Yes
1	16k	Yes

Nine sensors were used at a spacing of 0", 8", 12", 18", 24", 36", 48", 60" and 72" from the load plate as illustrated.



2.3 Control Sections

All testing was performed based on control section boundaries. Control section maps were provided by the LaDOTD and used to determine the section limits. Operators ensured that testing was offset 250' from all control section begin & end points and bridge limits (beginning and ending). They also avoided test points in the immediate vicinity of underpasses (where possible). A comment was provided for any offset with the extent of the offset.

2.4 Event Markers

The following additional items were noted (in the comments field) during the course of the FWD testing:

- Milepoint - The number of miles from the beginning of the control section regardless of the direction of the travel
- Direction - The direction of the travel relative to the direction of the control section:
  - N – North
  - S – South
  - E – East
  - W – West

- Surface Type:
    - A - Asphalt Concrete (AC)
    - J - Jointed Portland Cement Concrete (JCP)
    - C - Continuously Reinforced Concrete (CRC)
- Date - Includes the date that the FWD was collected
- Events - The following events on the DEPARTMENT's highway network were marked on the corresponding 0.10-mile record
    - Mileposts on Interstates
    - Every surface type change
    - The beginning and ending points of every bridge
    - The beginning and ending point of any segment of highway that is under construction or marked for construction along the highway

2.5 File Naming

All files were named using the following format as described by LaDOTD. The data acquisition files contain the route number (i.e. LA 4789 LM XXX to LM XXXX).

2.6 File Back-Up

At the end of each day of testing, two backup copies of all data collected was made. The back-up was done before leaving the area where the testing was completed. All files were Win-zipped onto a removable flash USB drive. The zip files were named to show the section tested.

All flash drives were kept secure, with original paperwork and activity forms filled out during each day of testing. Operators stored all data drives and forms until the end of the week, at which time:

Copy 1 of the week's data drives and originals of all paper forms were forwarded to the home office for processing.

Copy 2 of the data drives and a copy of all forms were kept with the operator until the end of the project.

3. Coring and Special Testing

Objective: The purpose of the Materials Sampling is to confirm layer thicknesses and material types identified from GPR testing.

3.1 Coring Procedures

The following procedures were anticipated for completing the Materials Sampling proposed:

Boring locations were selected in consultation with appropriate LaDOTD staff. Coring locations varied depending on the structural response, radar profile and/or areas of different surface type and/or structural capacity. Fugro identified the recommended locations to be sampled using the deflection and/or GPR profiles.

Coring location included:

Station and/or control section log mile
Route Number and Control Section
GPS coordinates

During the material sampling, notes were taken to identify any seepage of water in the underlying pavement and soil layers during the materials sampling program. The specific number of cores taken was coordinated with the LaDOTD staff. All unbound layers were augured down to the original subgrade. Four (4) inch pavement core logs that include the surface, base, and sub-base courses sampled were obtained.

The logs documented:

Measurements of thickness
Types and characteristics of each layer
Any deterioration of layer materials
Stripping in asphalt
Separation noticed
Honeycomb in PCC
"D" cracking in PCC

Digital photographs with a scale of each core and core hole were included with all core logs. Fugro provided proper core disposal. Fugro ensured the core holes were properly patched and adequate clean up was conducted on the project site.

3.2 Special Testing (DCP)

The DCP consisted of a 60 degree cone connected to a 5/8 in.-diameter steel rod, which was advanced into the soil by repeatedly dropping a 17.67 pound hammer. Hammer drills were used to drill through the bound surface layers. The DCP was driven 3 ft. below the base course or subbase surface and the number of blows recorded with depth. If the DCP encountered zero penetration for 20 blows, the DCP was stopped, and the boring advanced through the treated/stone layer in order for the DCP to be advanced into the subgrade.

The data contained the route number, direction, distance from centerline, lane mile, DMI and GPS. The readings were recorded in centimeters with every blow.

4. Data Analysis and Reporting

The following parameters were provided to LaDOTD:

The calculation of the subgrade resilient modulus (M_R) and effective pavement modulus (E_p) as outlined in the AASHTO 1993 Guide for Design of Pavement Structures.
Modulus of subgrade reaction, k-value, for concrete pavements.

Effective Structural Number (SN_{eff}) based on deflections for flexible pavements.
Surface Curvature Index (SCI) values based on surface deflections.
California Bearing Ratio (CBR) values from Dynamic Cone Penetrometer (DCP) test results.
Deflection Basin Analysis.

4.1 General Comments Regarding the Analysis:
One of the primary issues encountered in applying the AASHTO guide to compute the above values was how to analyze pavements with AC over PCC. Three typical types of pavement structures may be encountered:

AC pavements over a non-PCC layer
PCC pavements over any layer type
Composite Pavements (AC overlays over a PCC pavement)

For computation purposes regarding the SN and k-value, these cases were handled as such (refer to case definitions above):

Only an SN calculated
Only a SN and k-value calculated
Only a SN and k-value calculated
All pavements will have the M_r and E_p calculated. The M_r and E_p were calculated based on the deflections at Sensor 1 (0 in. from load plate) and Sensor 9 (72 in. from load plate).

4.2 Temperature Correction
A temperature correction factor will be incorporated when computing the Effective Pavement Modulus to account for variations in the asphalt modulus due to temperature. (There was both a graphical and a numeric method for calculating E_p.) The data will be processed using the numeric method. But the tools for using both methods can be provided to LaDOTD. The temperature correction is for 68° F.

4.3 Computation of M_r and E_p - Limitations
The M_r of subgrade and the E_p that were computed based on deflection from the 1993 AASHTO Design Guide page III-97.

$$M_R = \frac{0.24\,P}{d_r r}$$

P = load
d_r = deflection at radius r
r = radius

The 1993 AASHTO Pavement Design Guide recommends a load of 9000 lbs. The LaDOTD may request that the back-calculation be done at an alternate load level, if desired.

There were reduction factors to the Back-calculated M_r values to convert them to laboratory M_r values used in the 1993 AASHTO Pavement Design Guide. (For AC, a factor of 0.33 was used to obtain the design (laboratory) resilient modulus and for PCC this factor is 0.25). The analysis that was performed to calculate E_p used the unaltered M_r

value. These factors were found in the sections where the deflection methods for determining a subgrade resilient modulus were discussed (AASHTO Design Guide, pg. III-101 and III-111). The k-value computation for rigid pavements will incorporate the 0.25 reduction factor for the M_r for PCC pavements.

For the computation of E_p, the deflection at d_0 was used and requires a temperature correction to adjust the deflection. The deflections were adjusted to 68° F. Two classes of systems were considered in the AASHTO Design Guide for Flexible Pavements (AASHTO III-97 for flexible and III-109 for rigid).

AC over Granular or Asphalt-Treated Base
AC over Cement- or Pozzolanic-Treated Base

The computation of E_p for Rigid pavements does not require a temperature correction.

AC over PCC was not mentioned in the AASHTO Guide. To account for sections where AC over PCC was present, the PCC can be considered a Cement-Treated base when calculating the temperature correction factor.

Iterating the E_p value until the calculated deflection matches the corrected field deflection will find a solution for E_p. This can be done easily in Microsoft Excel using the Solver function. The equation used was in the AASHTO Design Guide on page III-97 (III-109 for rigid).

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

d_0 = deflection at center of load at 68^0 F in inches (calculated deflection)
p = NDT plate pressure in psi
a = NDT plate radius in inches (5.91 in.)
D = total thickness of pavement layers above subgrade in inches
M_R = subgrade resilient modulus in psi
E_p = effective pavement modulus of all layers above subgrade in psi

The field deflection at d=0 will be corrected to 68° F using Figures 5.6 or 5.7 from the AASHTO Design Guide pages III-99 and III-100. Equations were derived from these two graphs so that the conversion factors could be programmed. [**Note**: The equations were fitted to the curves to perform the double interpolation in order to obtain the temperature correction factor for the deflection at Sensor 1. This double interpolation requires a thickness and a temperature, which was part of the field data.**]**

The curves used to generate equations for the computation of the temperature correction factor were valid for certain temperature and asphalt thickness ranges. It was expected the limits of asphalt thickness and temperature might be exceeded at times (although less frequently than the thickness). For values of either temperature or thickness that exceed the maximum limit used to generate the equations, the maximum limit will be used instead. Using values beyond the limit often result in corrected deflections that were either negative or unrealistically small. This was a mathematical issue inherent to the curve fitting process. The models will be tested through comparison to the graphs they were generated from to verify they were within the temperature and thickness limits of the graphs. To ensure that temperature correction factors will be within a reasonable range, for temperatures above 120° F, 120° F was used, and for AC thicknesses greater than 12 in., 12 in. was used. Based on prior experience, AC pavements above 12 in. thick only need to be corrected for temperature in the top 12 in. of the layer.

4.4 Computation of SN_{eff} from Deflection Testing

Two procedures exist for determining the SN_{eff} for flexible pavements.

SN_{eff} from deflections (E_p, M_R)

The effective structural number can be calculated using the computed effective pavement modulus. The equation is as follows. The E_p in this equation was from the E_p calculated from the deflections (see above). This equation is found in the AASHTO Guide on page III-102. An alternative to the equation is Figure 5.8 on page III-103. The guide describes both the equation and Figure 5.8 to be used for AC pavements.

$$SN_{eff} = 0.0045 D \sqrt[3]{E_p}$$

As requested by LaDOTD, effective SN values will be calculated for rigid pavements as well.

[Note: The procedure for the E_p and M_r were the same for flexible and rigid pavements. The only difference was this last step, the SN calculation, which was not specified for PCC pavements. This equation can be applied to rigid pavements, but must be used with caution as the equation was developed for flexible pavements.

4.5 Computation of k-value

An automated algorithm for computing the static modulus of subgrade reaction and k-value was the most ideal method in terms of time; however there were some considerations that were weighed.

Options for Computing k-value:

1) Based on the AASHTO Design Guide Section 3.2.1 pg. II-37-44. Follow the procedure found in section 3.2.1 - there were two conditions specified. They were:

a) If the [PCC] slab was directly on the subgrade, the following equation can be used. This will obtain a **composite k-value**.

$$k = {M_R}/{19.4}$$

b) If the slab was on a subbase or base layer, Figures 3.3 through 3.6 from the AASHTO Pavement Design Guide can be utilized to compute the k-value. This was a very time-consuming manual process for the amount of sections and would also require the computation of the elastic modulus of the subbase layer and resilient modulus of the subgrade layer prior to the procedure of computing the k-value. It was not recommended to do this process manually.

2) This method for backcalculating a dynamic effective k-value from NDT that can be converted to a static effective k-value was from the 1993 AASHTO Guide, III-117 and III-131. It can be used for PCC, and AC over PCC pavements. This procedure was found on page L-13 to L-21.

-The deflection bowl AREA was computed for either PCC or AC/PCC pavements. A correction will need to be applied to the AC/PCC modulus to the deflection at 0 in. if necessary. These correction equations were straightforward and were found on page L-19. One was for unbonded and the other for bonded interfaces between the AC and PCC.

-Next, the dense liquid radius of relative stiffness, l_k, was calculated as an intermediate step in obtaining the dynamic effective k-value. The l_k was a function of the AREA.

-From the l_k, load, d_0, Euler's constant, and plate radius, the dynamic effective k-value can be calculated. This equation is on page L-14.

-To obtain the static effective k-value the dynamic effective k-value was divided by two.

The third option for computing a static k-value was from the Federal Aviation Administration's equation (FAA Advisory Circular 150/5370-11A):

$E_{Subgrade} = 26k^{1.284}$ which through inversion becomes

$$k = \left(\frac{E_{Subgrade}}{26} \right)^{1/1.284}$$

The $E_{Subgrade}$ value is equal to the backcalculated M_r from the deflection method discussed in a previous section.

To facilitate the process, the following assumptions were made:

Assumption 1: To correct the deflection at d = 0 for Method 2 for AC over PCC pavements, an AC elastic modulus of 500,000 psi was assumed. This is a typically used seed value for back-calculation. Furthermore, the effort was made to keep this computation as independent from the back-calculation as possible, so we opted not to use a back-calculated AC modulus. The effect of changing this assumed AC modulus value was minimal on the correction to d_0.

Assumption 2: Similar to a fixed modulus of 500,000 psi being assumed, the interface between the AC and the PCC was assumed to be bonded (as opposed to unbonded). The difference in the corrected deflection was minimal and information as to the bonding or lack thereof of the layers was not available.

Assumption 3: This was less of an assumption as much as a description of the calculation. For Method 1 the M_r used was multiplied by 0.25 to convert it to a theoretical lab value prior to performing the k-value computation. The factor of 0.25 was from the 1993 AASHTO Design Guide and was recommended for rigid pavements.

Assumption 4: The simple equation (from Method 1), if applied, was used for rigid pavements despite the presence of any subgrade layers. Hence the k-value reported is representative of a composite of all layers below the PCC.

Any or all of the methods could be reported as the calculation involves a simple spreadsheet calculation. The k-value from AASHTO M_r provides values that were typically higher than those from the Area Method. There was some scatter when correlating these results. The k-value from the FAA M_r was a more conservative estimate of the k-value than the AASHTO method. The correlation between the AASHTO M_r and the FAA M_r was close at low M_r values but the FAA method was more conservative as the M_r increases. Fugro will provide the k-value from these methods and will recommend which values to use once the range in k-value was determined.

4.6 Deflection Basin Analysis

FWD data was used to differentiate between sound pavement structures and those requiring reconstruction or major rehabilitation. This differentiation was made, in part, based on the FWD deflection interpretation scheme provided in TTI report 409-3F "Incorporating a Structural Strength Index Into the Texas Pavement Evaluation System" provided to Kevin Gaspard and the Modulus 4.2 Users Manual and shown in the following table. This table was developed for a target load of 9000 pounds.

The typical TxDOT setup for FWD testing has 7 sensors from 0" to 72", at 12" spacing. The setup that was used on the LaDOTD project adds sensors at 8" and 18". The SCI value was computed as the difference in deflection between the deflection at the center of the load, and the deflection 12" from the load location. For the proposed sensor spacing, this was the 3rd sensor. The Sensor 9 spacing was 72" from the load location.

While the SCI and Sensor 9 readings will give an indication of strength of the base and subgrade, they were not used to determine remaining life of the pavement or the required rehabilitation. Additional analysis of the deflection data would be required to determine remaining life and rehabilitation requirements. This analysis includes the back-calculation of layer moduli along with the determination of the design or required SN from traffic data.

Sensor 9 (mils)	SCI	Pavement Diagnosis
≤ 1.2	SCI ≤ 20	Good base, Stiff subgrade
	20 < SCI > 40	Marginal base, Stiff subgrade
	SCI ≥ 40	Thin and/or soft base, Stiff subgrade
> 1.2 and ≤ 2.0	SCI ≤ 20	Good base, Marginal subgrade
	20 < SCI > 40	Marginal base, Marginal subgrade
	SCI ≥ 40	Thin and/or soft base, Marginal subgrade
> 2.0	SCI ≤ 20	Good base, Soft or wet subgrade
	20 < SCI > 40	Marginal base, Soft or wet subgrade
	SCI ≥ 40	Thin and / or soft base, Soft or wet subgrade

4.7 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) was used to assess in situ strength characteristics of undisturbed soil and/or compacted materials up to three feet below the bottom of the pavement. The DCP consists of a 60-degree cone connected to a 5/8 inch-diameter steel rod. This rod penetrates the soil by repeatedly dropping a 17.67-pound hammer onto a fixed anvil located on the rod. The raw data for each DCP location was Road Name, Direction, Distance From Centerline, Lane Mile, Station Number, GPS, Typical Cross Section Information, Reference Reading Prior to Initiating Blows, and Moisture Content for Each Layer Strata. On sections where subgrade modulus or k-value varied dramatically, the DCP was used to determine the DCP Index, which was the depth in penetration divided by the number of blows to reach that penetration.

LaDOTD has requested that Fugro use the LaDOTD equation for analysis of DCP results:

$$M_R \text{ (ksi)} = 122.4/\text{DCPI}, \text{ where DCPI is mm/blow.}$$

In addition to the M_r from DCP, the DCP results were graphed as requested by LaDOTD. The ASTM D6951-03 specification was used for DCP testing with a modification of the number of blows for refusal. The number of blows was changed from 5 to 20 blows for refusal.

4.1 Deflection Analysis

Using the non-destructive deflection testing data and GPR profiles, the roadways were analyzed to identify those areas responding differently to loads. Deflection profile plots were used to assist in determining core locations. The Surface Curvature Index (SCI) and Sensor 9 readings (representing subgrade response) were used to assess the strength and weakness of the base/subbase and subgrade layers. The deflection data was also used to determine the M_r E_p and K-value of the pavement. This information was used in determining DCP test locations.

A complete set of electronic files was provided to LaDOTD on a USB drive. These included the following file structure and contents:

Core Log Database

LaDOTD Core Database.mdb

This is a database containing all data for each core log

All Core Logs.pdf

There is a pdf sheet for each core log

All Photos

This folder contains a picture of each core and associated hole

DCP

DCP Results

This folder contains an excel sheet for each DCP location and associated plots

Summary Files

This folder contains an excel file summarizing the DCP data for each control section

Elmod Thickness Files

Space Delimited

Tab Delimited

These folders contain the thickness data for each section ready for input into Elmod in the described format

Final Database

LaDOTD Structural Analysis.mdb

This database contains all the information relating to structural analysis, including GPR thickness and material descriptions

FWD Input Files

This folder contains the FWDb database file for each control section

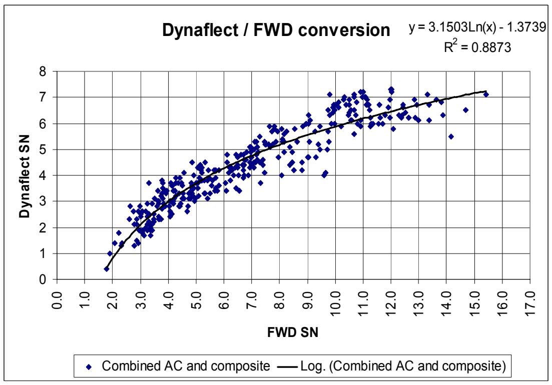
Power Point Plots

This folder contains each of the power point plots of FWD deflections and GPR traces for 5 miles on each sheet

Raw GPR Files

This folder contains all of the .DZT raw GPR files collected in the field.

APPENDIX C



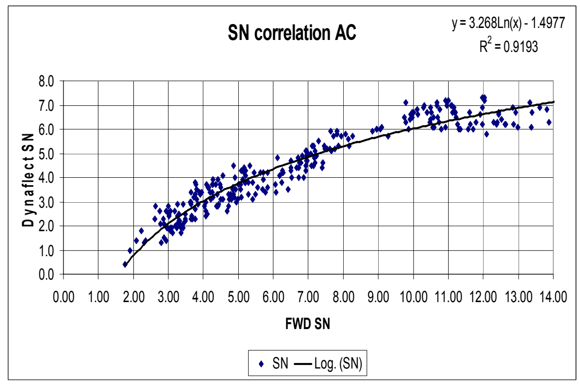


Figure C-1
SN correlation curves all points

Figure C-2
SN correlation curves AC points only

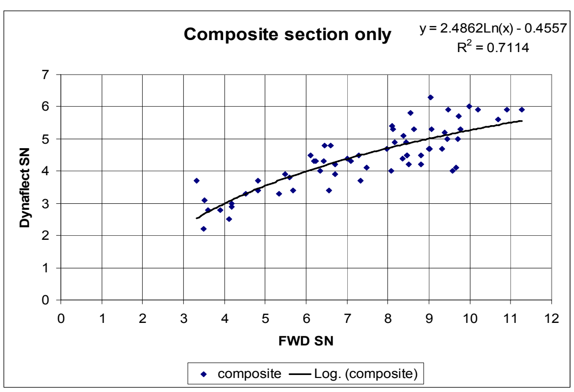


Figure C-3
SN correlation curves composite points only

Table C-1
Summary of sites used for establishment of correlation

Route	Parish	Type	Pavement				Base Course		Subbase		Number
			Layer 1		Layer 2		Layer 1				of data
			Type	Thick	Type	Thick	Type	Thick	Type	Thick	Points
La 333	Vermilion	AC	AC	6			SC	8.5			9
US 171	Beauregard	AC	AC	5			Stone	10	CTB	12	9
LA 991	Iberville	AC	AC	4			CTB	12			9
LA 22	Ascension	AC	AC	17							3
			AC	13							6
LA 28	Vernon	AC	AC	5			Stone	10.75			9
LA 344	Iberia	AC	AC	6			SC	7			9
LA 182	Lafourche	AC	AC	2.5			SC	8.25			9
LA 652	Lafourche	AC	AC	3.5			SC	9			9
LA 28	Rapides	AC	AC	6			SC	8			66
I-12	St Tammany	AC	AC	20			SC	6			56
I-10 Road	St Tammany	AC	AC	21			Sand/shell	12			25
I-10 Shoulder		AC	AC	11			Sand/shell	12			25
US 190	WBR	AC	AC	10			BCS	10			27
								Total AC			271
Tulane Ave	Orleans	Composite	AC	2	PCC	8	Sand				16
US 61											
S. Claiborne	Orleans	Composite	AC	9	PCC	8					16
US 90											
LA 46	Orleans	Composite	AC	5.5	PCC	9					31
								Total Composite			63
								Total all points			334

APPENDIX D

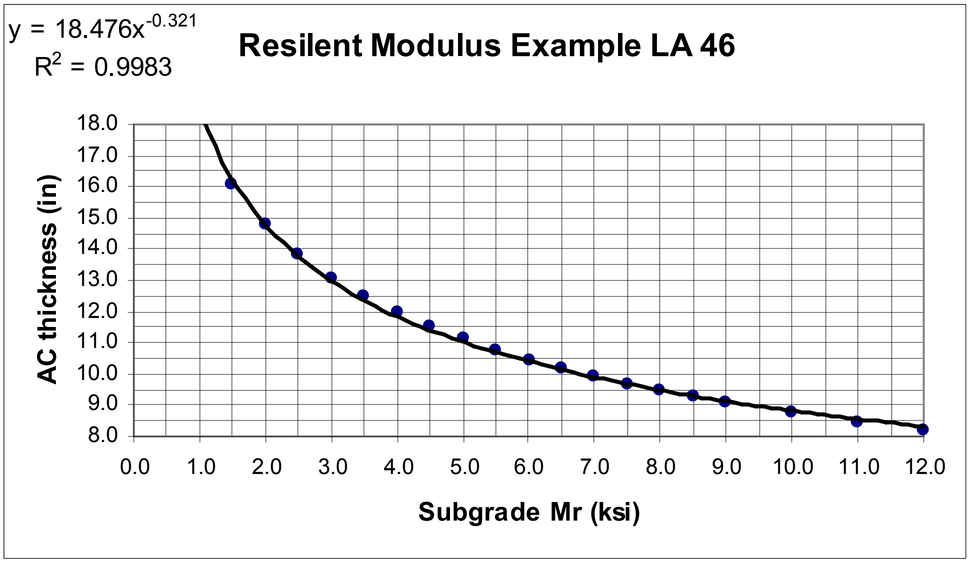


Figure D-1
AC verses subgrade M_r (LA 46)

AC thickness = SN / 0.44 SN/in

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	1,802,650
Initial Serviceability	4.3
Terminal Serviceability	2.5
Reliability Level	97 %
Overall Standard Deviation	0.47
Roadbed Soil Resilient Modulus	4,000 psi
Stage Construction	1
Calculated Design Structural Number	5.20 in

Rigorous ESAL Calculation

Performance Period (years)	8
Two-Way Traffic (ADT)	27,500
Number of Lanes in Design Direction	2
Percent of All Trucks in Design Lane	80 %
Percent Trucks in Design Direction	50 %

Figure D-2
Sample of DARWIN output

Table D-1
Traffic data for LA 46

DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
DATA COLLECTION AND ANALYSIS SECTION
P. O. BOX 94245
BATON ROUGE, LOUISIANA 70804-9245
(225) 379=1925

TRAFFIC ASSIGNMENT

DATE:	08-Dec-99
PROJECT NO.	046-03-0064
NAME:	St. Bernard / Orleans Line - Paris Road
DESCRIPTION:	District 02 Overlays
ROUTE:	La. 46
FUNCTIONAL CLASS:	Urban Principal Arterial
PARISH:	St. Bernard

2000 ADT =	27,500	ANN. GROWTH	1.0%
2008 ADT =	29,800		
D =	55%		
K =	10%		
T =	7%		

AXLE DISTRIBUTION

VEHICLE TYPE	PERCENT	2000 ADT	2008 ADT	MEDIAN YEAR
1 MOTORCYCLES	0.20%	55	60	57
2 PASSENGER CARS	75.60%	20,790	22,529	21,659
3 2A-4T SINGLE UNIT	17.00%	4,675	5,066	4,871
4 BUSES	0.40%	110	119	115
5 2A-6T SINGLE UNIT	2.00%	550	596	573
6 3A SINGLE UNIT	1.00%	275	298	287
7 4A SINGLE UNIT	0.10%	28	30	29
8 4A SINGLE TRAILER	1.00%	275	298	287
9 5A SINGLE TRAILER	2.20%	605	656	630
10 6A SINGLE TRAILER	0.20%	55	60	57
11 5A MULTI-TRAILER	0.10%	28	30	29
12 6A MULTI-TRAILER	0.10%	28	30	29
13 7A MULTI-TRAILER	0.10%	28	30	29
TOTALS		27,500	29,800	28,650