Louisiana Transportation Research Center

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Field Validation of Equivalent Modulus for Stabilized Subgrade Layer

by

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16. Abstract

This report presents the findings associated with an effort to evaluate projects with weak subgrades utilizing the newly developed Equivalent Modulus Analysis (EMA) spreadsheet created by the Louisiana Department of Transportation and Development (DOTD). The EMA spreadsheet was developed to simplify the design process by allowing for easy application of the Method of Equivalent Thickness (MET) calculations that lies at the heart of the pavement design process. A validation was attempted by trying to compare the EMA spreadsheet's predictions to field collected data. Lime treatment projects were used to do the assessment as lime treatment is often employed in establishing working tables for construction equipment that cannot operate on very weak subgrades. This research attempted to accomplish two objectives: (1) to try to find a way to incorporate lime treatments into the design process in order to take advantage of the strength it offers and (2) to explore the strengths and weaknesses of the MET approach used in the design process through utilization of the EMA spreadsheet to see if there is a way to assess weak subgrades. Results, however, were both inconclusive and questionable because the untreated and lime-treated soils typically sheared during field testing which invalidated back-calculation efforts. Additionally, it proved problematic to assume that raw subgrades could be held as two-layer systems in order to carry out back-calculation. As such, the EMA spreadsheet could not be validated. More testing would be required to validate the approach.

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ABSTRACT

This report presents the findings associated with an effort to evaluate projects with weak subgrades utilizing the newly developed Equivalent Modulus Analysis (EMA) spreadsheet created by the Louisiana Department of Transportation and Development (DOTD). The EMA spreadsheet was developed to simplify the design process by allowing for easy application of the Method of Equivalent Thickness (MET) calculations that lies at the heart of the pavement design process. A validation was attempted by trying to compare the EMA spreadsheet's predictions to field collected data. Lime treatment projects were used to do the assessment as lime treatment is often employed in establishing working tables for construction equipment that cannot operate on very weak subgrades. This research attempted to accomplish two objectives: (1) to try to find a way to incorporate lime treatments into the design process in order to take advantage of the strength it offers and (2) to explore the strengths and weaknesses of the MET approach used in the design process through utilization of the EMA spreadsheet to see if there is a way to assess weak subgrades. Results, however, were both inconclusive and questionable because the untreated and lime-treated soils typically sheared during field testing which invalidated back-calculation efforts. Additionally, it proved problematic to assume that raw subgrades could be held as two-layer systems in order to carry out back-calculation. As such, the EMA spreadsheet could not be validated. More testing would be required to validate the approach.

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IMPLEMENTATION STATEMENT

The results of this study were intended to assist DOTD in improving the quality of its design and rehabilitation methodologies. Currently, material and thickness considerations are arrived at by standardized policy or by tedious trial-and-error methods using the traditional, iterative Odemark-Boussinesq (O-B) approach known, more commonly, as the MET. LTRC's recently developed EMA spreadsheet streamlines that calculation process and makes it possible to consider a wider array of design possibilities in a more efficient and comprehensive manner than is currently available. The results derived from this research effort, however, which looked at weak subgrade were both inconclusive and questionable principally because the weak untreated and lime-treated soils tested as part of the research effort had sheared in the field, rendering subsequent and integral back-calculation efforts invalid. In addition, the assumption that raw subgrades could be treated as two-layer systems, required to be able to carry out back-calculations, also proved to be problematic. All findings proved to be highly questionable. As such, they are not suitable for use in any implementation policy.

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INTRODUCTION

In 1949, Odemark derived his MET as a simplified method that could be used to calculate the response in a multi-layer system comprised of differing linear-elastic materials *[1]*. Odemark's method employed the assumption that the stresses, strains, and deflections below any given layer in a system will depend on the stiffness of that layer alone. According to the theory, any layer could be transformed into another of a differing modulus by multiplying its thickness by its modular ratio. Thus, in theory, the approach made it possible to convert the various layers of a system into a common material in order that Boussinesq's single-layer equations for stresses, strains, and displacements could be applied *[2]*.

However, Odemark's assumptions ignored certain aspects of linear-elastic theory. This fact led Ullidtz, in 1998, to introduce a system of correction factors that could be applied to Odemark equations in order to try and affect a correction *[3]*. Introduction of this system did improve matters somewhat. But, as many pavement materials are not linear-elastic, there were still cases where real world responses did not correlate well with theory.

A number of attempts have been made to refine the theory further or to automate the methodology. Despite this, most attempts have been largely academic exercises with their principal focus being on attempting to refine the approach to better accommodate the underlying linear-elastic theory *[4 - 9]*. The literature search did not uncover research efforts that attempted to empirically verify Odemark-Boussinesq (O-B) predictability on a network level. Neither was there literature represented that attempted to develop a network-level usage strategy based on empirical verification methods.

The literature shows that layers of similar material properties can present problems to O-B methods and the MET. It is suggested, for example, that to use the MET effectively, moduli should decrease with depth, preferably by a factor of at least two between consecutive layers *[10]*. Experience has shown that FWD and HWD testing on weak subgrades can also present problems as the equipment can shear the surface of the material being tested and render the results invalid. These issues were to be evaluated as part of the comparison analysis intended in Task 3.

In Louisiana, highway construction efforts often encounter field conditions that are so poor that conventional site development cannot proceed. That is to say, the structural capacity of the existing subgrade is not sufficient to support the introduction of construction equipment. In such cases, it is often the practice to treat the existing soil matrix by introduction of lime to a depth of 6 in. or more to help dehydrate the subgrade material. Such treatment is usually ignored from a design perspective. In such cases, the augmented strength is usually consigned to the margins of safety and, therefore, is not used to refine the pavement design structurally. It has been argued that this practice should be changed. Often, the added strength that results from subgrade treatment is considerable and its utilization in design would improve project cost-benefit ratios significantly if taken advantage of.

In analytical terms, designers must envision a site as a two-layer system if they are to take advantage of a lime treatment's added structural capacity. The left side of Figure 1 illustrates such an arrangement with Material-B (the portion of the existing soil matrix that is to be lime treated) serving as the first layer of such a two-layer system. Material-A (the semi-infinite remainder of the existing soil matrix that the treatment effort will not impact) serving as the second.

Analyzing such two-layer systems is a complex mathematical proposition. The traditional approach requires that the designer envision the multilayer system as a single monolithic structure having a single equivalent resilient modulus. Such an equivalency is illustrated on the right side of Figure 1. The nature of such an equivalency rests in the requirement that both the left and right sides of Figure 1 exhibit the same surface deflections under a given loading. The reason the equivalency is needed is because the MET traditional approach utilizes single-layer materials theory as its basis.

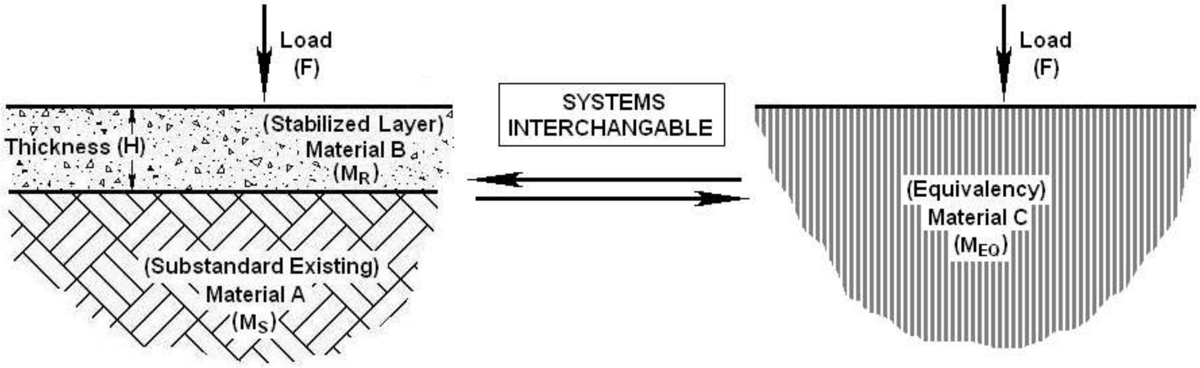


Figure 1
Pavement system equivalency

The MET used in conjunction with the Boussinesq Equation serve as the methodology by which an equivalency like that depicted in Figure 1 can be realized *[1, 2]*. There are numerous references available that thoroughly detail this methodology along with the theory that underlies the MET and, as such, the details will not be recounted here. For those interested, however, a fuller treatment has been provided in Appendix A.

The O-B methodology ensures that the left and right hand sides of the equivalency in Figure 1 are entirely interchangeable in a structural sense (i.e., both sides of the figure will have the same surface deflection under loading). As such, the O-B method provides the means of forgoing the complexity of the two-layer system on the left by replacing it with the analytically simpler one-layer equivalency on the right.

LTRC has developed an automation of the O-B method that affords the designer greater latitude when experimenting with design alternatives. This automation takes the form of an interactive spreadsheet, the EMA spreadsheet, wherein users can enter two out of a possible five governing variables that figure into an O-B analysis. Entering the two variables allows the missing variables to be read from a generated plot.

The five governing variables, as depicted in Figure 1, include the thickness of the treated layer (H), the treated layer's resilient modulus (M_R), the resilient modulus of the pre-existing subgrade material (M_S), the resilient modulus of the equivalency (M_{EQ}) and the load which the system is expected to support (F). The user is allowed to enter the load, F, as well as either the thickness of the treatment, H, or the resilient modulus of the pre-existing subgrade material, M_S, as inputs. An example of how the EMA spreadsheet is used is expressed in Figure 2, which is a screen capture taken from the spreadsheet.

The field at the top of Figure 2, highlighted in yellow and red, indicates where the user's inputs have been logged into the spreadsheet. The "2100, 6" figure shown in this field indicates that the user wishes to log a condition wherein a site is exhibiting a subgrade modulus of 6 ksi under a load application of 2100 lbf. The resulting five plots show the equivalency conditions that can be achieved when treatments to a given depth and modulus are applied.

For example, the plot shows that if the top 18 in. of subgrade material are treated so as to achieve 50 ksi performance then the resulting equivalency would be 20 ksi. Note, however, that the same 20 ksi equivalency can be achieved through the treatment of only 8 in., provided that this treatment achieves 250 ksi within that 8 in. The advantage of the EMA spreadsheet is that it permits the user to quickly explore design alternatives without having to directly carry out iterative O-B calculations.

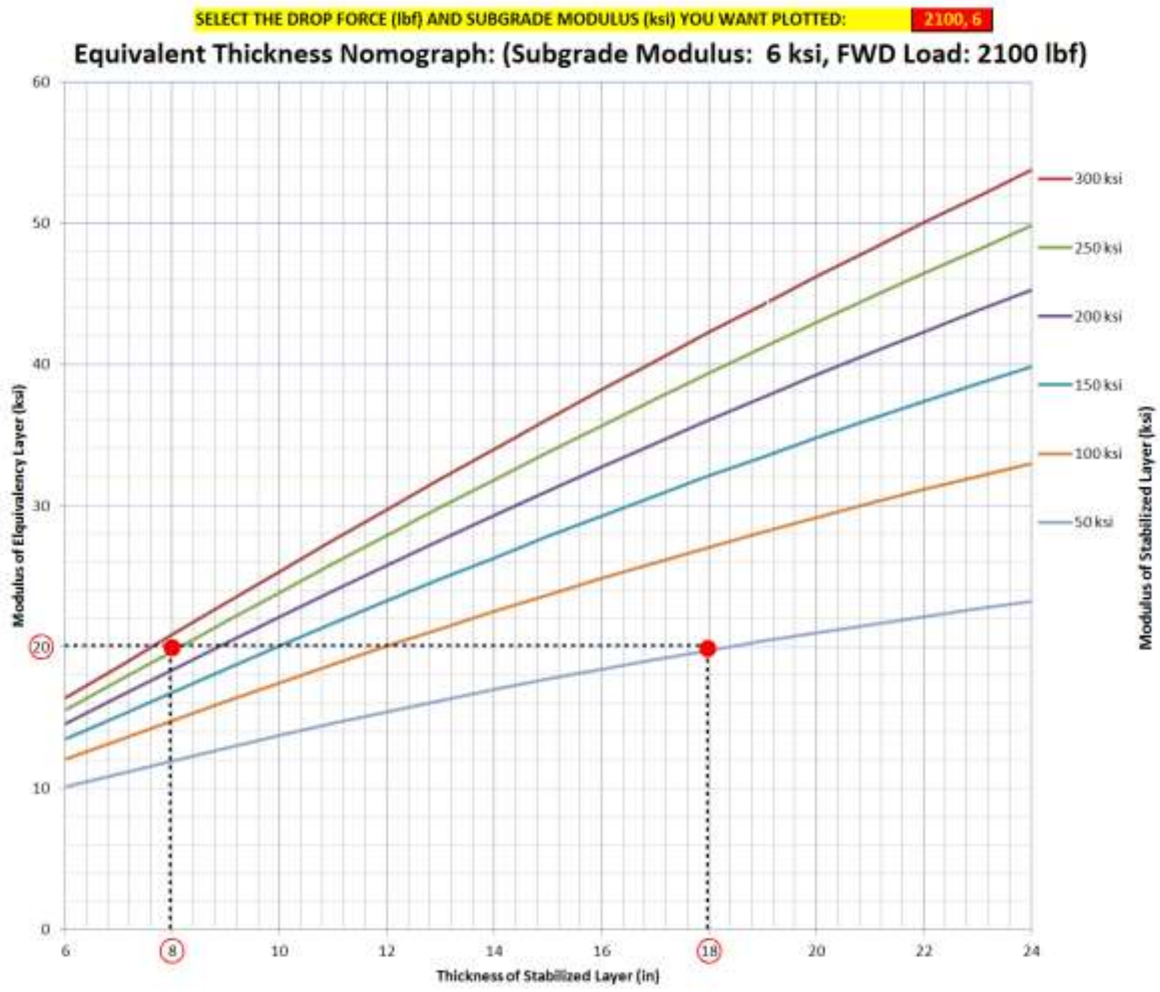


Figure 2

Output from LTRC's EMA spreadsheet

It should be noted that the O-B approach utilizes elastic materials theory that assumes all materials involved in an analysis are ideally elastic. Earth-materials, however, are not typically perfectly elastic in most cases (a fact that calls into question the overall validity of the O-B approach in those cases). To investigate this, an effort must be made to carry out a series of field evaluations in order to validate the O-B predictions where they appear to be the weakest, like on excessively weak subgrade projects.

The O-B theory, and the EMA spreadsheet, can be used to aid in the design of both flexible and rigid pavements. As already noted, subgrade modulus values, M_S, required for asphaltic design can be read directly off the spreadsheet plots. Modulus of Subgrade Reaction, more commonly referred to as the K-value, used in rigid pavement design can be read indirectly from the spreadsheet. That is, M_S figures read from the plots can be converted to K-values by first representing them in psi and then dividing the result by 22.5 in. This figure derived from the geometries associated with the standard 30-in. plate bearing test apparatus.

Determination of pavement layer and subgrade properties is typically arrived at in practice by the employment of a Dynamic Cone Penetrometer (DCP), a Falling Weight Deflectometer (FWD), and a Heavy Weight Deflectometer (HWD). The DCP test involves using a 17.8-lb. hammer to drive a 60-degree, 0.75-in. diameter cone tip into soil or other penetrable layers. The hammer is repeatedly dropped 22.6 in. onto an anvil and the depth of penetration is recorded for each drop. The collected penetration data is used to tabulate resilient modulus figures for the layers being investigated.

FWD and HWD testing involves dropping a known load onto a 6-in. or 12-in. diameter circular load plate. An array of nine geophones placed at 0, 8, 12, 18, 24, 36, 48, 60, and 72 in. from the point of impact of the load are used to monitor how the shockwave produced by the load propagates through the pavement cross-section. The load force is transferred through the plate where it imparts a deflection that simulates a wheel load. The geophones record this basin and software is used to calculate the stiffness-related parameters of the pavement structure via an iterative process called “back-calculation.” A proprietary software package developed by Dynatest called Evaluation of Layer Moduli and Overlay Design (ELMOD) automates this back-calculation process.

Special consideration has to be given to how to process FWD/HWD tests that are run on raw subgrades as they are essentially single layer systems, which can be a challenge because the MET methodology requires at least two layers for calculations to proceed. A possible work around of this problem is to arbitrarily divide the subgrade into a two-layer system and assume that each layer has the same subgrade material properties. This solution can present problems, though, as the back-calculation process can sometimes not resolve if layers are too similar in terms of their material properties.

OBJECTIVE

The central objective of the research is to validate the EMA spreadsheet through comparison of its predictions to field collected data so that current pavement design strategies and policies can be updated and modified in an effort to improve long-term performance and increase benefit-cost ratios on future pavement projects. It is also an objective of this research to develop a subgrade stabilization specification (lime and/or cement) for the Department that will allow the Department to take design advantage of the structural improvements that subgrade treatment applications provide.

SCOPE

Six in-state projects utilizing lime-treatment as a means of strengthening weak subgrades were examined as the basis of this research. Each of the six were DCP, FWD, and/or HWD tested. A control project that utilized cement treatment in place of the lime treatment that did not have weak subgrades was also DCP, FWD, and/or HWD tested. All projects were tested variously throughout their construction. That is, testing was conducted on the raw subgrade prior to lime or cement treatment and then again between the successive layers of construction through to placement of the wearing course. The lime projects examined were LA-8 near Simpson (Vernon Parish), US-171 near Leesville (Vernon Parish), LA-3177 near Butte LaRose (St. Martin Parish), LA-97 near Jennings (Jefferson Davis Parish), LA-73 near Dutchtown (Ascension Parish), and LA-91 near Gueydan (Vermilion Parish). The cement control project is US-90 near Lydia (Iberia Parish).

METHODOLOGY

A comprehensive series of before-and-after field evaluations were conducted wherein projected performance, as derived from the EMA spreadsheet, was compared to actual performance as derived from field testing. FWD, HWD, and DCP testing was conducted between successive stages of construction and rehabilitation as possible on existing and projected projects so that a dataset of actual performance could be compiled.

Correlations between expected performance and actual performance were determined by plotting the projected performance, as derived from the EMA spreadsheet, against the actual performance, as derived from FWD, HWD, and DCP. FWD testing, for example, was conducted at prepared subgrade sites prior to a lime or cement treatment so as to determine the site's "before" condition by direct measurement of subgrade properties. A "theoretical after" condition was then determined using the EMA spreadsheet to predict the modulus values that would be expected to appear once the project's subgrade had been treated.

At such time as the subgrade was actually treated, FWD testing was again performed on top of the treated layer in order to arrive at an "empirical after" condition that could be used to either validate or invalidate the "theoretical after" condition that was arrived at by EMA spreadsheet. A correlation was then established by plotting the "theoretical after" data against the "empirical after" data.

It was critical that the in-situ subgrade modulus and other site characteristics be assessed by DCP in order to check the FWD testing that was done directly on top of the treated and untreated subgrades as the practice of testing on such subgrades is unorthodox (shearing of the soil can often happen, calling into question the results). Throughout this research, resilient modulus values by DCP was based on research done at LTRC by Mohammad, Gaspard, Herath, and Nazzal *[11]*.

$$\boldsymbol{M_R = \frac{151.8}{(DCPI)^{1.096}}} \tag{1}$$

where,

M_R = Resilient modulus (ksi), and
DCPI = Dynamic cone penetration index (mm/blow).

The theoretical versus empirical correlation plots were then analyzed in order to find outliers and to parameterize how the theoretical figures deviated from the empirical. The reasons that may underlie why these deviations and outliers became manifest were considered and reported on where possible with the intent of establishing a more refined usage policy for the O-B Method.

Project selection was arrived at through the canvassing of prospective rehabilitation and new construction projects that fit research needs. Cross-sections and material properties of pavement layers were compiled from available sources (plans, cores, DCPs, and so forth) so that layer thicknesses, layer material, and other relevant information could be ascertained.

A Dynatest model 8000 FWD and, later, a Dynatest model 8081 HWD were used to evaluate pavement structural capacity (the model 8000 had been retired before the project concluded, which is why the model 8081 was employed). A 6-in. radius load plate was utilized on the model 8000 and a 12-in. radius load plate on the model 8081. Three drops were conducted at each location. On raw and lime treated subgrades, the FWD was run "empty" (i.e., without load plates), wherein the load applied would be on the order of 13 lbf/in^2 spread out over the 6-in. radius load plate. The "empty" load for the HWD was on the order of a similar 14 lbf/in^2 spread out over the 12-in. radius load plate. The 13 lbf/in^2 FWD figure was near enough to the 14 lbf/in^2 HWD figure as to make the FWD and HWD interchangeable in practice. The DCP was used principally to corroborate FWD and HWD findings.

FWD and HWD testing was conducted at all locations independent of soil conditions. The original intention was that FWD and HWD testing would use as wide a variety of loadings during each test as possible so as to enrich the data set and provide better coverage of the conditions that the EMA spreadsheet can embody. The EMA spreadsheet offered the following possibilities:

Table 1
Proposed FWD testing matrix

Parameter	Conditions to be Tested
FWD/HWD Drop Force (lbf)	1000, 1100, 1200, … , 3000
Subgrade Modulus (ksi)	2, 3, 4, … , 12
Depth of Treatment (in.)	6, 7, 8, … , 24

Since the majority of projects analyzed were lime projects with weak subgrades, it was expected that field logistics would not be able to adequately support testing over all the Table 1 possibilities. For this reason, the field testing effort was to limit itself to those force, modulus, and treatment thickness conditions that field circumstance and DOTD policy would allow. It was recognized that it would not be possible to know what testing regimen might be afforded before arriving at a site, so it was determined that the logistics and testing regimen proposed in Table 1 would have to be scrapped and testing on each project would be governed by circumstance.

All sites were retested over their successive stages of construction to the extent that field contractor operations permitted (i.e., atop embankment, atop stabilized layer, atop base course, etc…). This was done in order to evaluate the sensitivity of the O-B theory to multi-layered construction and in order to evaluate how strength properties distributed themselves within individual layers over time as construction progressed.

Dynatest's Evaluation of Layer Moduli and Overlay Design (ELMOD) software is an advanced and proprietary pavement analysis program that is widely accepted as the industry standard in modeling pavement performance. Version 6 of the ELMOD software was used to convert the raw FWD/HWD data collected in the field into empirical measurements (H, F, M_R, M_S, M_{EQ}). Theoretical equivalencies of these empirical measurements were arrived at by EMA spreadsheet.

The respective figures (ELMOD-actual vs. EMA-theoretical) were plotted against each other and statistically analyzed in an effort to determine how well the theoretical figures predicted the empirical ones. Outliers were isolated and reported on as they appeared.

FWD/HWD data collected on raw subgrades are single layer systems from a back-calculation standpoint. Such single-layer systems had to be coded into ELMOD as two-layer systems for ELMOD to be able to analyze them. As such, an arbitrary methodology had to be used to decide what the thicknesses of the two layers should be.

It was decided that it would be reasonable to set the upper layer thickness in ELMOD to whatever thickness the future lime or cement treatment would be. The thickness of the lower layers would be set to whatever remained of the embankment half-space. Because bedrock in Louisiana is so deep, it was known that the thickness of the full embankment thickness needed to set to 240 in. in ELMOD (recommended by Dynatest). As an example, ELMOD settings for a raw embankment on a future 6-in. lime treatment would consist of 6 in. of raw embankment material over 234 in. of raw embankment material for a total of 240 in.

ELMOD requires seed values when assessing systems with three or more layers. Seed values used in this study would be taken from a previous study *[12]*. Multilayer settings for the ELMOD "Structure" and "Moduli" screens that would be used for this study are shown in Appendix B. In all cases, the "Deflection Basin Fit" option would be chosen in the "Moduli Screen."

Relationships discovered in the comparative analysis (ELMOD vs. EMA) were used to the extent possible to comment on cases where the O-B theory deviated from field results. In such cases where the O-B theory failed, attempts were made to ascertain the underlying reasons.

DISCUSSION OF RESULTS

Attempting to run the proposed testing grid proved to be impractical as all sites tested had low subgrade modulus values. The depths of treatment on all projects except one were fixed at 12 in. The exception was a 10-in. lime treatment. In addition, because the subgrades were so weak, it was only possible to test using the lowest drop force values possible (i.e., the FWD weight chassis was made "empty"). This was only the case on raw subgrade and lime treated layers. Once the base layers and wearing course layers were built, the pavement structure was strong enough to support typical FWD loads.

The Dynatest model 8000 FWD was used in the earliest stages of FWD testing until it was replaced with a Dynatest 8081 HWD. The procurement of the model 8081 HWD was so that LTRC could test more robust pavement types than was possible with the model 8000. For project 12-11P, this presented a problem because the unloaded model 8081 is considerably heavier than the unloaded model 8000.

When testing on raw subgrades, drop weights must be very low to not detrimentally disturb the pre-lime treated material. It was discovered early during testing that the model 8000 FWD needed to be run with its weight-chassis "empty" (that is, without load plates) to not shear unprepared and prepared embankments. As such, it was clear that the heavier model 8081 HWD presented problems. For the model 8000, the empty chassis weight was roughly 1,500 lbf. For the model 8081, the "empty" chassis weight is around 6,500 lbf.

A work around was devised wherein the model 8081 HWD was fitted with a larger load plate. FWDs and HWDs both employ a circular load plate to help distribute the applied load. The diameter of the load plate employed while testing using the model 8000 was 6 in. The diameter of the load plate employed while testing using the model 8081 HWD was 12 in. The larger load plate fitted to the model 8081 provided for rig compatibility. That is, the lbf/in^2 loads being produced by the "empty" model 8081 HWD produced figures compatible with the lbf/in^2 loads that had been seen on the "empty" model 8000 FWD at select sites.

The left side of Table 2 provides a project summary of sites where FWD/HWD testing was carried out as part of the 12-11P research effort. It lists each project's DOTD project number, control section ID, Parish, beginning and ending log mile, project length, Route the project was located on, and the type of construction (A1: new asphalt, A2: asphalt widen and overlay, A3: asphalt overlay of asphalt, A5: asphalt overlay with and in-place base, and C2: new Portland cement concrete). Those sections that were lime treated were full-depth reconstructions. The only project that was not a lime job was Project H.002890.6. It was a cement-treated base project that was included to serve as a control section.

The right side of Table 2 provides a summary of the FWD/HWD testing schedule, showing the date of testing, the test direction, the starting and ending stations of the testing, the spacing between tests, the layer that was tested on the date given, and a summary of comments.

A particular problem encountered on Project H.009708.6 is alluded to in the comment column. The “uneven surface” comment found there reflects a condition wherein the contractor did not level the surface properly after lime placement between stations 223+00 and 222+10. This improper leveling caused FWD results on that layer to be corrupted because the geophones could not seat properly. To compensate for this, FWD testing was conducted at a nearby lime treated site that had the same cross-section (located between stations 223+50 and 223+20) yet was sufficiently smooth enough to get proper readings. This second location is labeled as “alternate” in Table 2. Although this alternate location is not directly over the location that the raw subgrade was tested, it was sufficiently close enough for the results to be compatible with what a properly leveled lime layer would have given.

Table 2
Summary of FWD/HWD testing

Proj. No.	Ctrl. Sect.	Parish	Begin. Log Mi.	End Log Mi.	Length (mi)	Route	Cons. Type	Test Date	Test Direct	Sta. From	Sta. To	Test Spacing	Surface	Comments
077-02-0020	077-02	(03) Ascen.	2.2	4.55	2.35	LA-73	A2	9/3/2013	South	217+50	214+50	10 ft.	Raw Subgrade	
								N/A	South	217+50	214+50	10 ft.	AC	
H.009631.6	850-31	(50) St. Martin	0	5.64	5.64	LA-3177	A3	8/29/2013	South	99+35	97+75	10 ft.	Raw Subgrade	
								7/26/2016	South	99+35	97+75	10 ft.	AC	
H.001875.6	134-01	(58) Vernon	0.027	7.2	7.173	LA-8	A3	7/16/2013	West	388+00	384+00	10 ft.	Raw Subgrade	
								7/16/2013	West	388+00	384+00	10 ft.	Lime Subgrade	
								7/24/2013	West	388+00	384+00	10 ft.	Soil Cement Base	
								7/27/2016	West	388+00	384+00	10 ft.	AC	
H.009708.6	024-06	(58) Vernon	10.29	16.95	6.66	US-171	A1	3/20/2013	South	220+40	219+50	10 ft.	Raw Subgrade	
								3/20/2013	South	223+00	222+10	10 ft.	Raw Subgrade	
								3/20/2013	South	220+40	219+50	10 ft.	Lime Subgrade	uneven surf.
								3/20/2013	South	223+50	223+20	10 ft.	Lime Subgrade	alternate
								7/27/2016	South	220+40	219+50	10 ft.	AC	uneven surf.
								7/27/2016	South	223+50	223+20	10 ft.	AC	alternate
H.002890.6	424-04	(23) Iberia	14.11	17.08	2.97	US-90	A1	11/8/2012	South	935+00	919+00	50 ft.	Raw Subgrade	Zone 29
								11/15/2012	South	935+00	920+50	50 ft.	Cement Subgrade	Zone 29
								6/10/2013	South	935+00	925+00	50 ft.	Soil Cement Base	Zone 29
								3/6/2017	South	935+00	925+00	50 ft.	AC	Zone 29
H.002095.6	201-01	(27) Jeff Davis	0.16	1.11	0.95	LA-97	C2	7/23/2013	South	145+00	141+00	10 ft.	Raw Subgrade	
								7/23/2013	South	116+50	115+50	10 ft.	Raw Subgrade	
								7/25/2013	South	145+00	141+00	10 ft.	Lime Subgrade	
								N/A	South	145+00	141+00	10 ft.	PCC	
H.010525.6	212-01	(57) Vermilion	2.95	7.02	4.07	LA-91	A5	9/22/2015	North	51+00	61+00	10 ft.	Raw Subgrade	
								3/9/2017	North	51+00	61+00	10 ft.	AC	

Table 3 is provided to show the number of projects that had been slated for testing but had to be dropped because the lime treatment clause in the contract was waived. Lime treatment is utilized on projects at the discretion of the contractor with the approval of the DOTD's project engineer. It is typically only used over short segments of a project wherein fine silts are found in abundance that contribute to excessive structural weakness when very wet. If the construction takes place when the weather is reasonably dry, then the need for lime treatment is often dropped. This was the case for the projects listed in Table 3.

Table 3
Summary of projects not tested (lime requirement dropped)

Project No.	Control Section	Parish	Beginning Log Mile	End Log Mile	Length (mi)	Route	Construction Type
H.009490.6	117-01	43 - Sabine	4.71	8.62	3.91	LA-118	A5
H.002129.6	209-03	27 - Jeff Davis	0	5.77	5.77	LA-101	A5
H.010371.6	138-02	58 - Vernon	0	7.08	7.08	LA-399	A3
H.009560.6	139-06	58 - Vernon	5	13.3	8.3	LA-463	A5
H.009946.6	237-05	50 - St Martin	17.9	22.39	4.49	LA-352	A5
H.011426.6	386-02	01 - Acadia	0	4.5	4.5	LA-367	A5
H.011427.6	820-06	20 - Evangeline	3.32	8.08	4.76	LA-1172	A5
H.011652.6	213-05	28 - Lafayette	0.064	2.882	2.818	LA-92	A5
H.010373.6	840-08	40-Rapides	0.045	5.75	5.705	LA-497	A5
H.008331.6	849-11	49 - St Landry	0	8.35	8.35	LA-360	A5
H.010227.6	849-30	49- St Landry	0	1.83	1.83	LA-748	A3
H.001899.6	140-03	40 - Rapides	3.37	9.81	6.44	LA-113	A3
H.009527.6	332-04	62 - W Carroll	0	3.11	3.11	LA-585	A5

The left side of Table 4 provides a project summary of sites where DCP testing was carried out on projects that were examined as part of the 12-11P research effort. It lists each project's DOTD project number, control section ID, Parish, beginning and ending log mile, project length, Route the project was located on and the type of construction (A1: new asphalt, A2: asphalt widen and overlay, A3: asphalt overlay of asphalt, A5: asphalt overlay with and in-place base, and C2: new Portland cement concrete). Those sections that were lime treated were full-depth reconstructions. As previously stated, Project H.002890.6 was not a lime project. It was a cement treated base project that was included to serve as a control section.

The right side of Table 4 also provides a summary of the DCP testing schedule. It shows the date of testing, the longitudinal station where the test was conducted, the condition of the testing (the road condition that was tested) and the transverse location where the testing was conducted. Cells highlighted in yellow represent data that did not get recorded during testing leaving the date, station locations and comments unclear.

Table 4

Summary of DCP testing

Proj. No.	Ctrl. Sect.	Parish	Begin. Log Mi.	End Log Mi.	Length (mi)	Route	Cons. Type	Test Date	Location of Test (between)		Condition of Test (DCP pushed through)	Comments
077-02-0020	077-02	03 – Ascen.	2.2	4.55	2.35	LA-73	A2	2/2/2016	217+50	214+50	undisturbed full depth soil	4 ft. from shoulder
H.009631.6	850-31	50 - St Martin	0	5.64	5.64	LA-3177	A3	2/10/2016	99+35	97+75	undisturbed full depth soil	4 ft. from shoulder
H.001875.6	134-01	58 - Vernon	0.027	7.2	7.173	LA-8	A3	1/25/2016	388+00	384+00	undisturbed full depth soil	4 ft. from shoulder
H.009708.6	024-06	58 - Vernon	10.29	16.95	6.66	US-171	A1	1/25/2016	220+40	219+50	undisturbed full depth soil	4 ft. from shoulder
H.002890.6	424-04	23 - Iberia	14.11	17.08	2.97	US-90	A1	11/9/2012	935+00	919+00	raw subgrade	in wheelpath
								11/16/2012	935+00	920+50	cement treated subgrade	in wheelpath
								5/21/2013	935+00	925+00	soil cement base	in wheelpath
								5/28/2013	935+00	925+00	soil cement base	in wheelpath
								2/1/2016	935+00	925+00	undisturbed full depth soil	2 ft. from shoulder
H.002095.6	201-01	27 - Jeff Davis	0.16	1.11	0.95	LA-97	C2	2/10/2016	145+00	141+00	undisturbed full depth soil	4 ft. from shoulder
H.010525.6	212-01	57 - Vermilion	2.95	7.02	4.07	LA-91	A5	N/A	N/A	N/A	undisturbed full depth soil	N/A

Figure 3 is a map depicting all projects that were examined under the 12-11P study. Red circles indicate the Table 2 projects wherein testing was conducted and blue circles represent projects that were not tested because the lime treatment was waived.

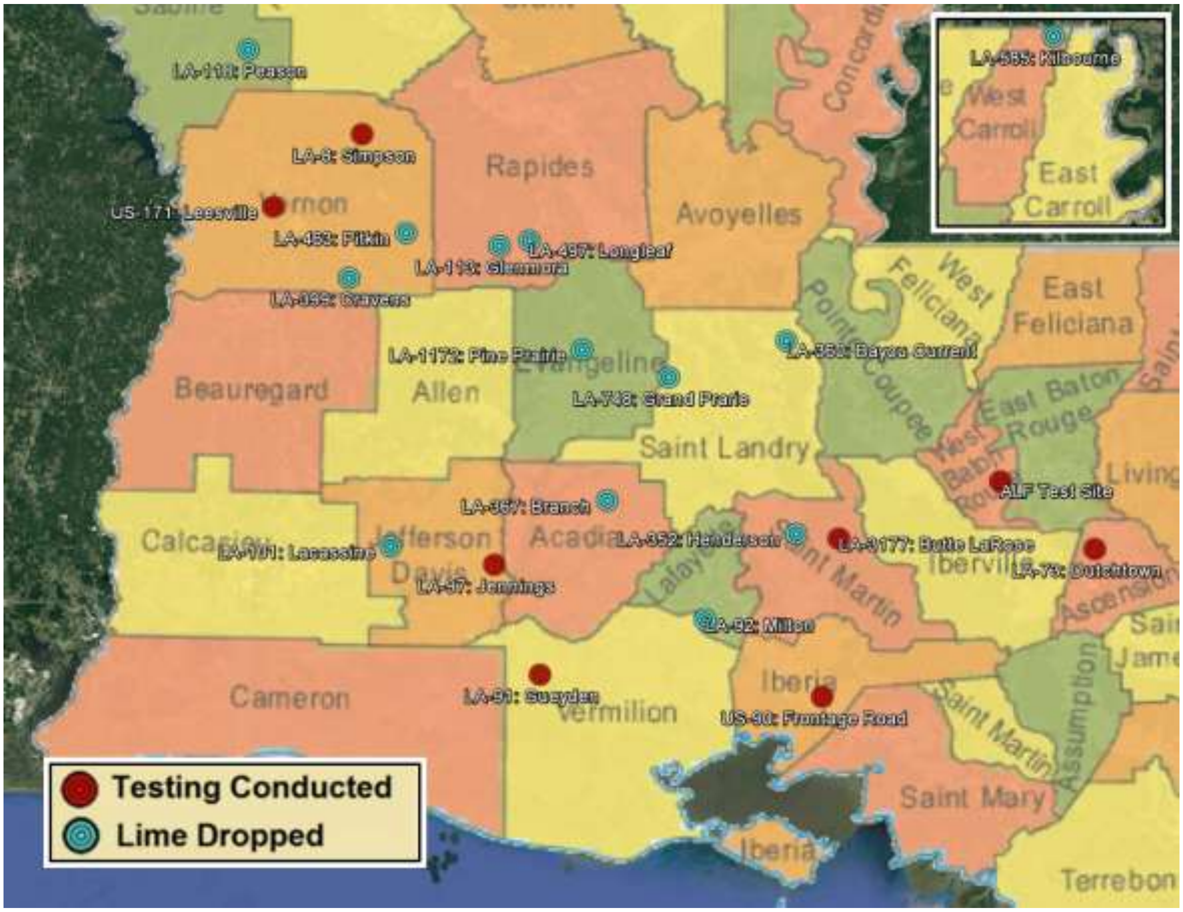


Figure 3
Projects map

Figure 4 depicts a detailed map of one of the sites tested as an example (LA 8 near Simpson in Vernon Parish). Figure 5 shows the layout of 29 FWD test points that were tested on LA 8 and Figure 6 provides a cross-section of the LA 8 pavement depicting the layer thicknesses and lime content. Figure 7 provides a detailed summary of LA 8's Natural Subgrade Properties as per NCHRP 9-23b and USDA NCRS Soil Unit Data.

Similar maps, cross-sections, and subgrade property summaries as those shown in Figures 4-7 have been prepared for the remaining projects listed in Table 2. To save space in the body of this report, these figures have been placed in Appendices C, D, and E. Appendix C provides the project maps and the FWD test location maps. Appendix D provides the project cross-sections along with thicknesses and material summaries and Appendix E provides the subgrade property summaries.

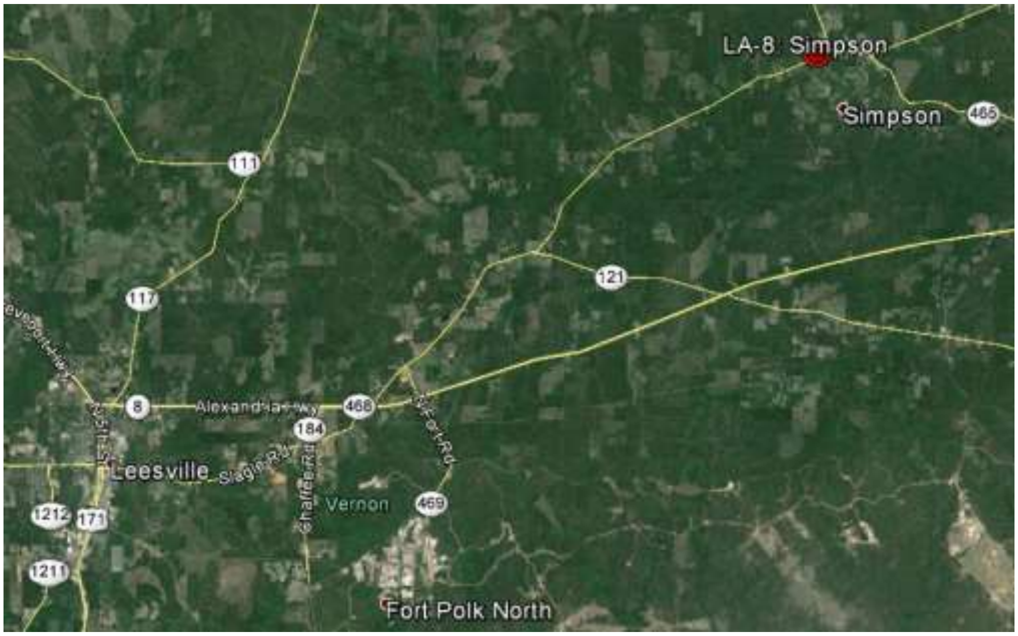


Figure 4

Map of LA 8 project (detail)

Figure 5

FWD test locations along LA 8

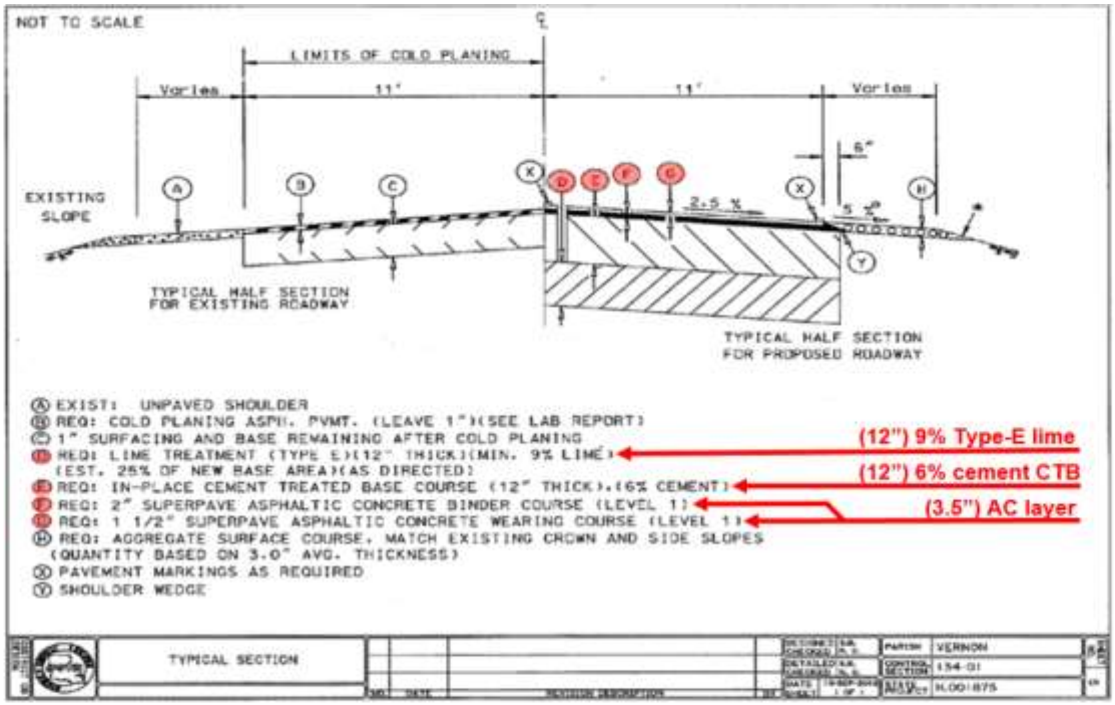
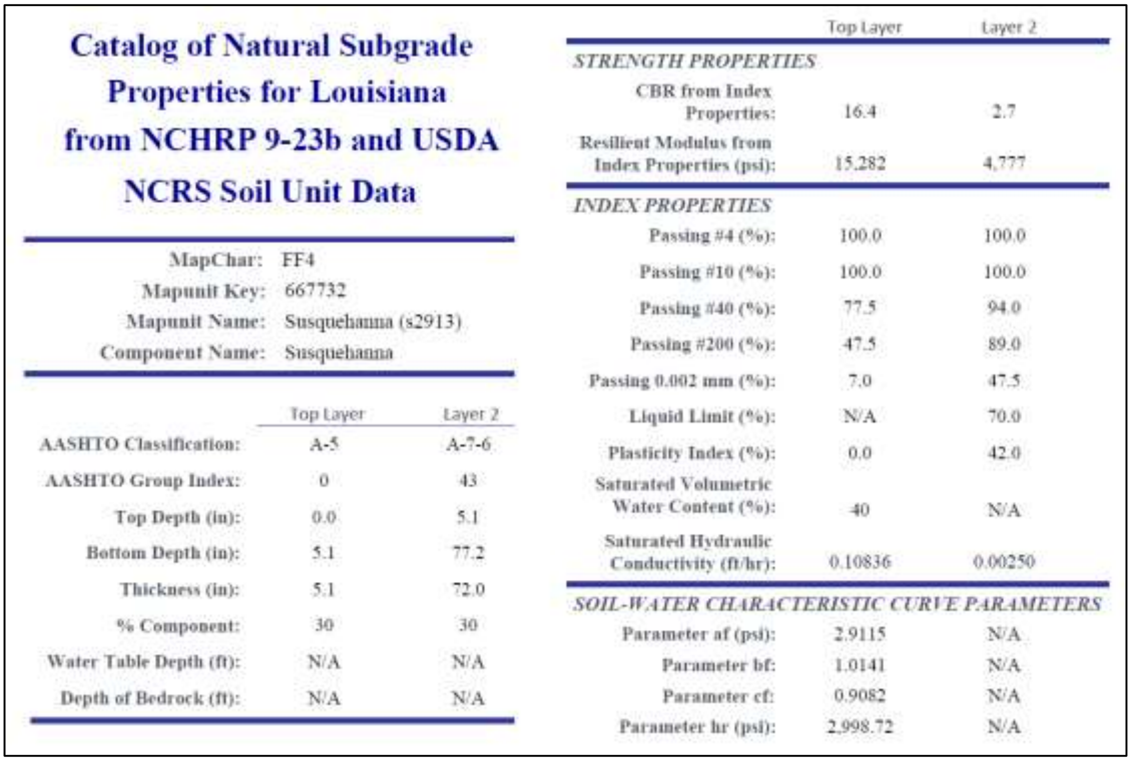


Figure 6

LA 8 cross-section



Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

MapChar:	FF4
Mapunit Key:	667732
Mapunit Name:	Susquehanna (s2913)
Component Name:	Susquehanna

	Top Layer	Layer 2
AASHTO Classification:	A-5	A-7-6
AASHTO Group Index:	0	43
Top Depth (in):	0.0	5.1
Bottom Depth (in):	5.1	77.2
Thickness (in):	5.1	72.0
% Component:	30	30
Water Table Depth (ft):	N/A	N/A
Depth of Bedrock (ft):	N/A	N/A

	Top Layer	Layer 2
STRENGTH PROPERTIES		
CBR from Index Properties:	16.4	2.7
Resilient Modulus from Index Properties (psi):	15,282	4,777
INDEX PROPERTIES		
Passing #4 (%):	100.0	100.0
Passing #10 (%):	100.0	100.0
Passing #40 (%):	77.5	94.0
Passing #200 (%):	47.5	89.0
Passing 0.002 mm (%):	7.0	47.5
Liquid Limit (%):	N/A	70.0
Plasticity Index (%):	0.0	42.0
Saturated Volumetric Water Content (%):	40	N/A
Saturated Hydraulic Conductivity (ft/hr):	0.10836	0.00250
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS		
Parameter af (psi):	2.9115	N/A
Parameter bf:	1.0141	N/A
Parameter cf:	0.9082	N/A
Parameter hr (psi):	2,998.72	N/A

Figure 7

LA 8 subgrade properties

The DCP results collected on LA 8 are presented in Figure 8 laid out to show what the DCP data of Table 4 encompasses. Most DCP tests were run approximately 4 ft. off of the pavement structure just clear of the shoulder through raw, full-depth soil at the same log mile that the FWD tests were run.

Figure 8 shows that there were two DCP tests that were run on LA 8. The blue curve represents the first DCP test (DCP1) and the green curve represents the second DCP test (DCP2). Roughly 40 in. of soil were penetrated as can be seen on the vertical axis. The horizontal axis displays the modulus values that were recorded with respect to depth.

Average M_S values on LA 8 as derived from the DCP curves can be read directly off of Figure 8. It can be seen, for example, that the average M_S for the full 40 in. for DCP1 was 30.8 ksi. For DCP2 it was 21.4 ksi. Combining DCP 1 and DCP2, the average full-depth M_S was 26.1 ksi.

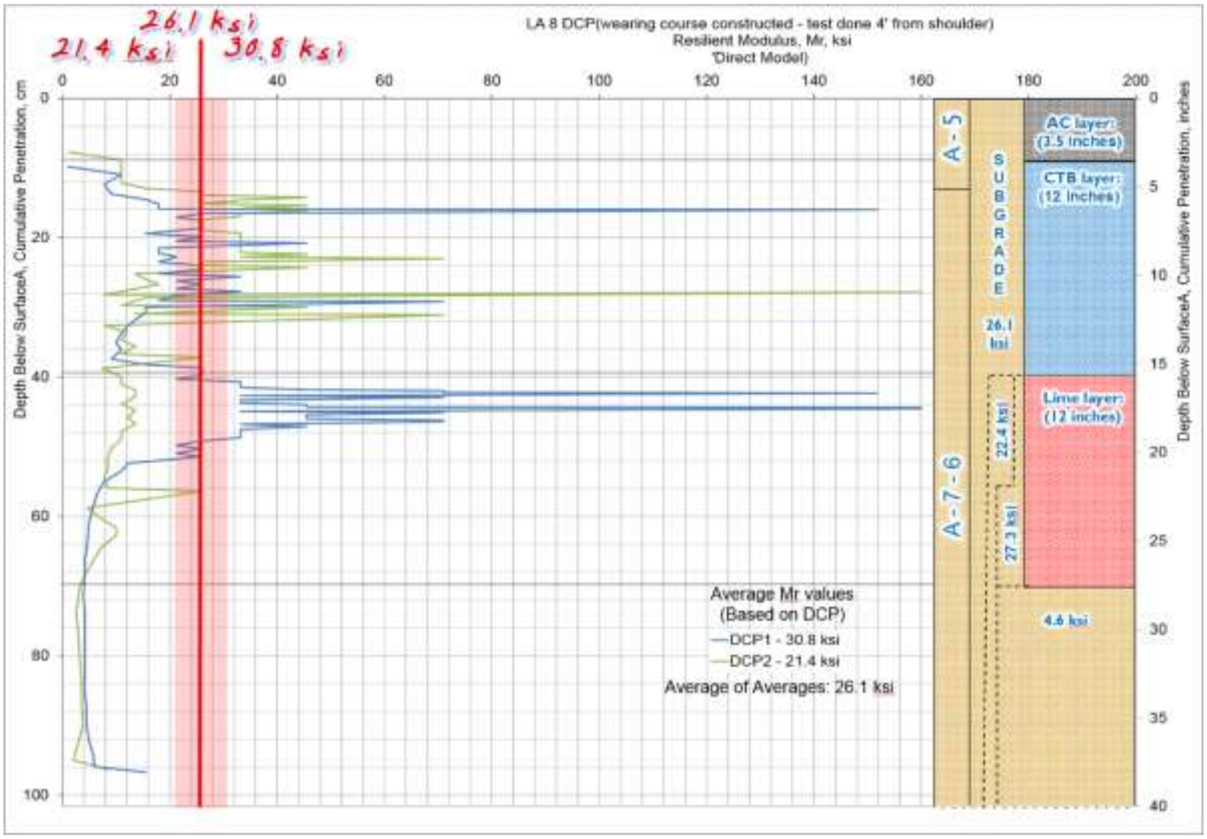


Figure 8
LA 8 DCP results

The colored area to the right of Figure 8 presents LA 8's cross-sectional layering referenced against the DCP plots. It shows, for example, that the DCP encountered the interface between the A-7-6 soil and the A-5 soil (see Figure 7) at a depth of about 5 in. very close to the location that the first large spike in the blue DCP1 curve of Figure 8 appeared.

The colored zone, repeated in Figure 9, illustrates where the pavement layer interfaces were located relative to the Figure 8 DCP plots. It can be seen, for example, that the heavily spiked region in the two DCP plots of Figure 8 are almost entirely spread out across those depths where the cement treated base (CTB) and lime layers were going to be built. The construction process, illustrated more comprehensively in Figure 9, shows that much of this strength was lost during construction because that stronger soil above what would become the working table was hauled away during Stage 1 excavation.

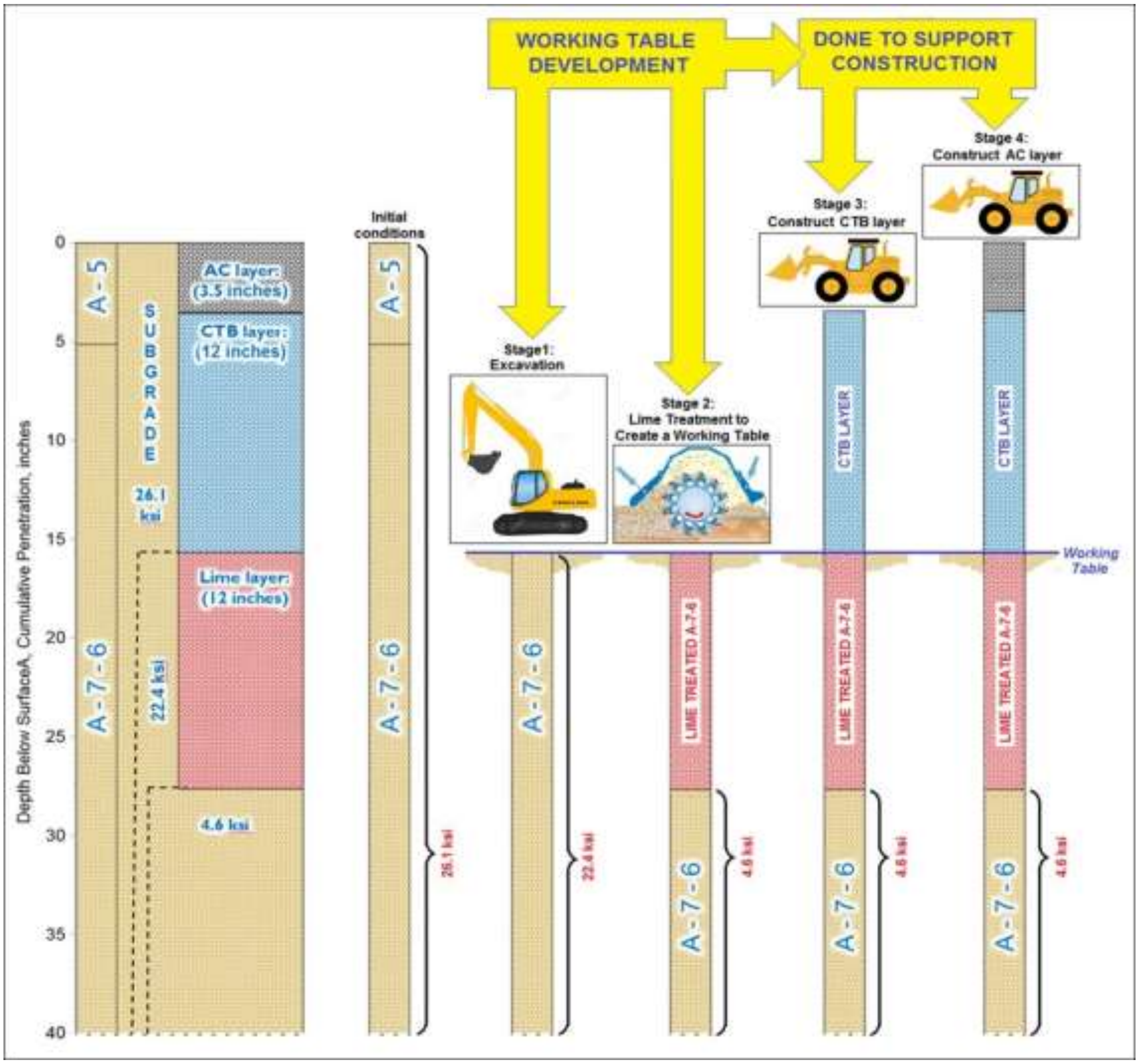


Figure 9
LA 8 stages of construction

Figure 9 demonstrates how the DCP curves in Figure 8 predicted the untreated subgrade would continue to weaken even further as construction progressed. Under “initial conditions,” it can be seen that the entire 40-in. column of raw soil had a M_S of 26.1 ksi on average. “Stage 1 excavation” called for the removal of the top 15 in. of overlying soil which reduced the M_S from 26.1 ksi to 22.4 ksi.

At the beginning of Stage 2 of construction, it was realized that a 12-in. lime treatment would be required to de-hydrate the embankment so as to create a working table strong enough to support the construction equipment needed to carry out Stages 3 and 4 of construction. This cutting of lime disturbed the projected 22.4 ksi of remaining raw soil, successfully reducing it to 4.6 ksi.

The compaction following the lime treatment at the completion of Stage 2 would be expected to contribute additional strength. But, as DCP testing between layers was not logistically possible, FWD testing would need to be utilized to assess what the contribution of the lime compaction would be.

It should be noted that the decision to use lime to de-hydrate the embankment could not have been known prior to construction. Thus, any strength derived from the lime layer could not have been incorporated into the design. In point of fact, it is not DOTD policy to utilize the strength attributed to lime even when it is part of design, so, any strength derived from lime is routinely ignored in design as a matter of policy.

Similar DCP plots like the one shown in Figure 8 have been prepared for the remaining projects listed in Table 2. They are presented in Appendix F.

Construction limitations allowed for only one project (the control section project, US 90) to be DCP tested between its various stages of construction. US 90 had a cement-treated subgrade rather than a lime treatment as used on the other projects of this study. As many as 10 DCP tests were conducted on US 90 between each stage of construction.

Figure 10 shows the DCP plots taken on the raw subgrade of US-90. Figure 11 shows the DCP plots taken on the US-90’s treated subgrade. Figure 12 shows the DCP plots taken on US-90’s treated Class II base.

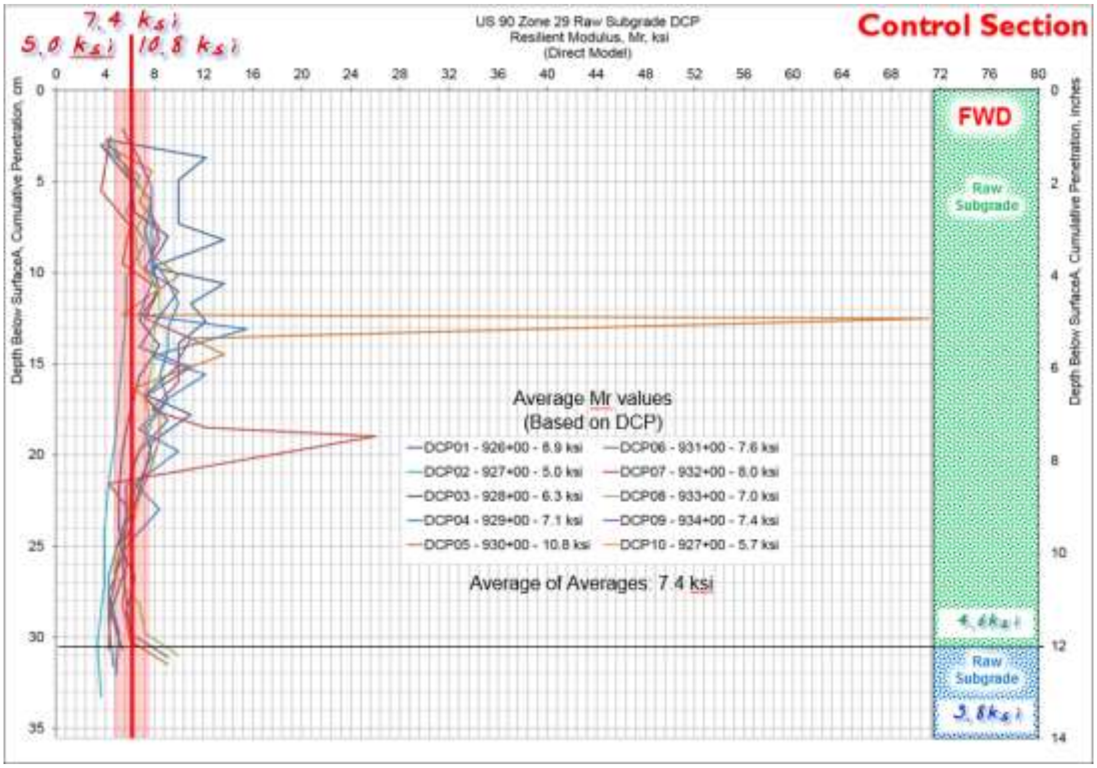
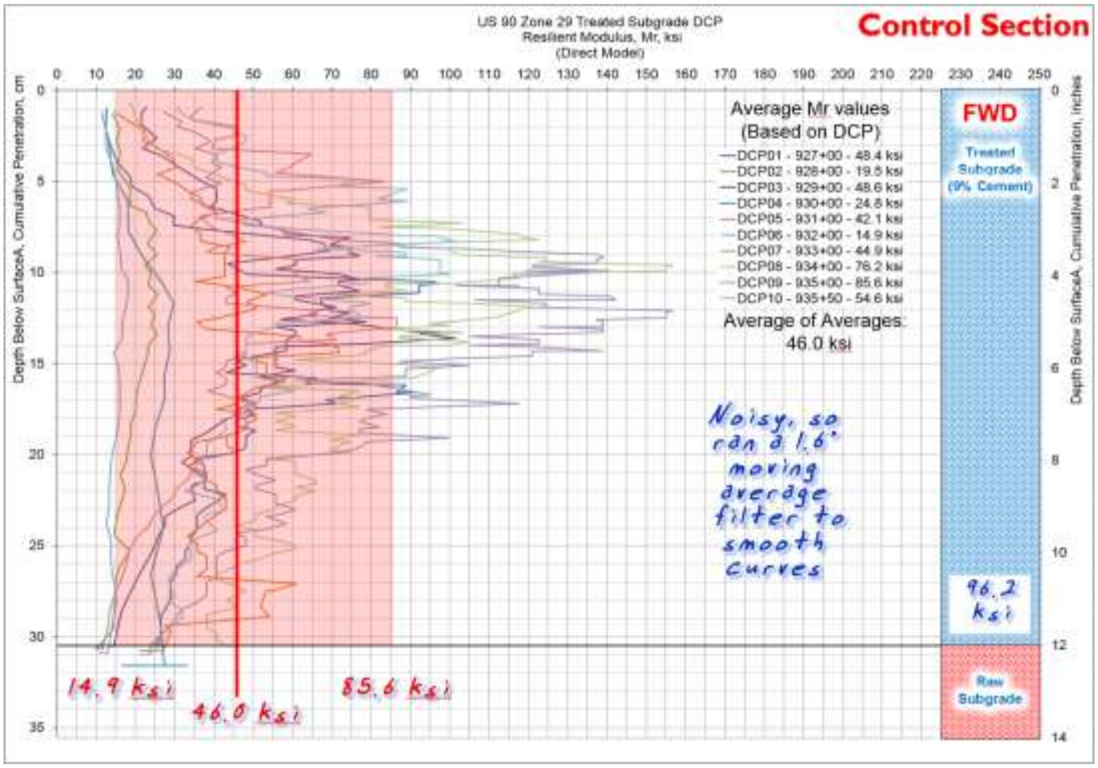


Figure 10

US-90 DCP results on raw subgrade



Noisy, so ran a 1.6" moving average filter to smooth curves

Figure 11

US-90 DCP results on treated subgrade

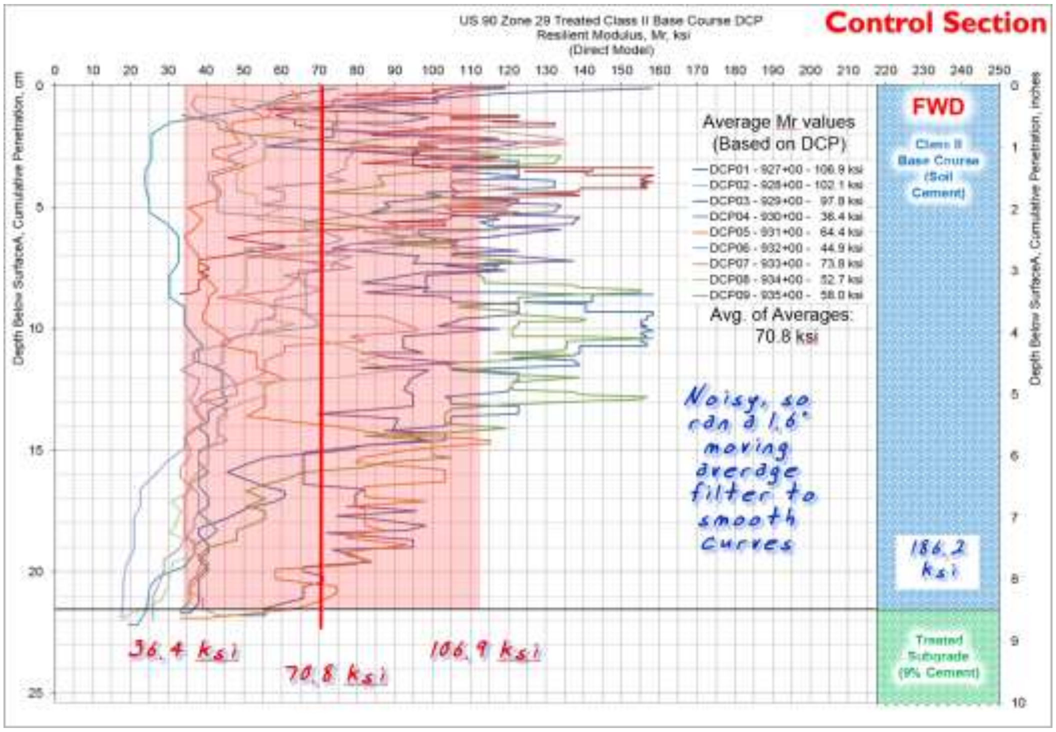


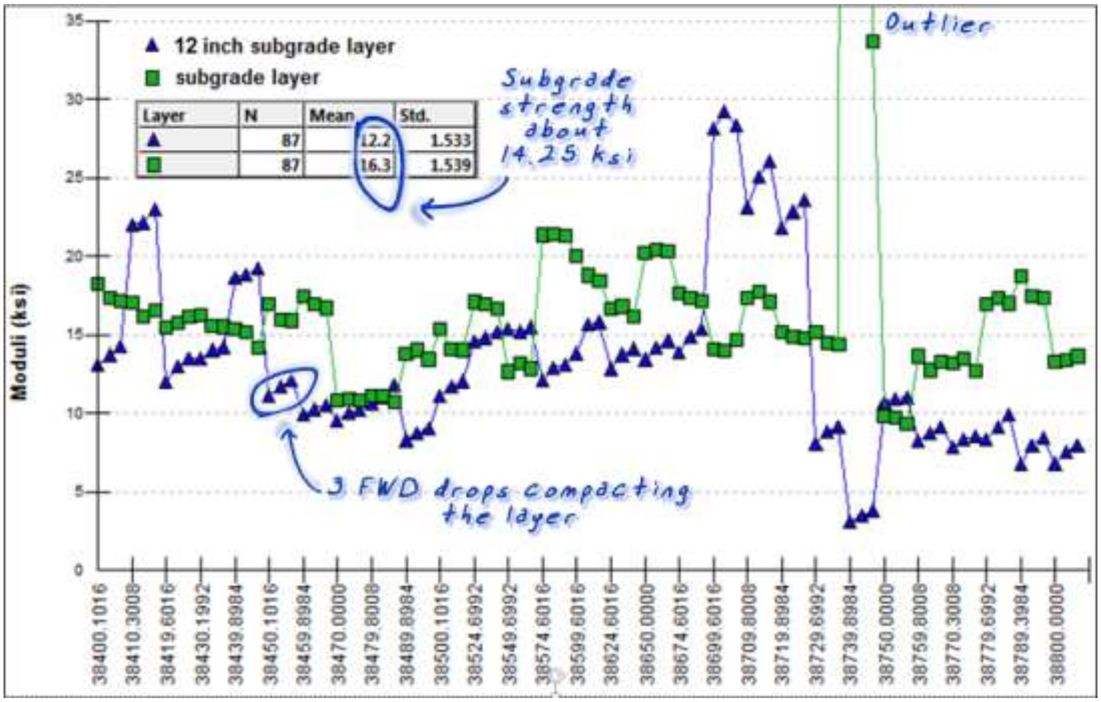
Figure 12

US-90 DCP results on treated Class II base

The FWD results collected on LA 8, presented in Figures 13-16, show what the FWD data collected as detailed in Table 2 looked like. Figure 13 shows the FWD results as tabulated from tests run atop the raw subgrade, Figure 14 from tests run atop the lime treatment, Figure 15 from tests run atop the soil cement layer, and Figure 16 from tests run atop the AC wearing course.

Figure 15 mapped out the 29 FWD drop sites visited on LA 8. Each of these sites underwent three drops. In Figures 13 - 16, the three drops can be seen clustered sequentially. For example, the three solid blue triangles circled in Figure 13 shows one clustering of these three drops. The increase in modulus figures seen in the circled cluster is indicative of a slight compaction and possible shearing of the material during the three drops. Shearing and compaction violates back-calculation theory. This behavior is seen repeating in Figure 14.

The various curves shown in Figures 14-16 represent strength figures derived from “true” distinguishable layers (distinct layers having measurable differing properties). It must be noted, however, that the two curves shown in Figure 13 do not represent truly distinguishable layers. As explained in the methodology, all single-layer subgrade systems had to be divided into two arbitrary layers for ELMOD to be able to run. On LA 8, it was known that lime was to be cut into the subgrade to a depth of 12 in. So, that thickness was entered into ELMOD. But, in reality, the two layers implied in Figure 13 are not actual.



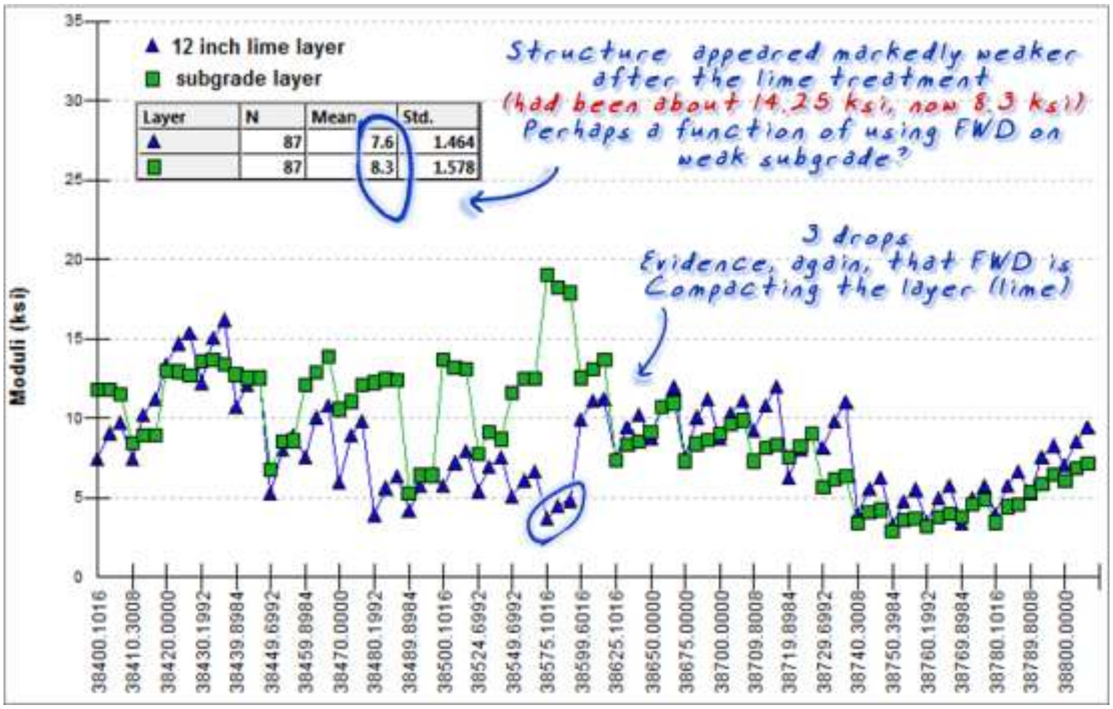
Outlier

Subgrade strength about 14.25 ksi

3 FWD drops compacting the layer

Figure 13

LA-8 FWD results on raw subgrade

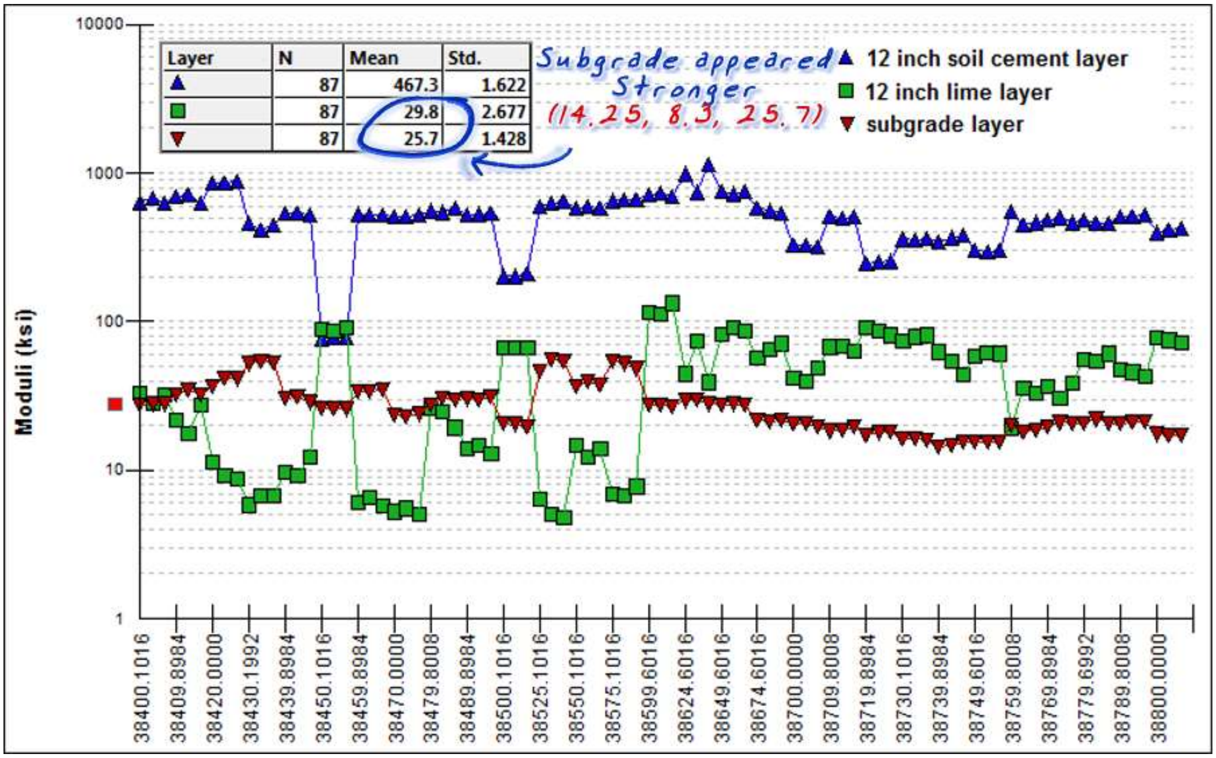


Structure appeared markedly weaker after the lime treatment (had been about 14.25 ksi, now 8.3 ksi) Perhaps a function of using FWD on weak subgrade?

3 drops Evidence, again, that FWD is Compacting the layer (lime)

Figure 14

LA-8 FWD results on lime treatment

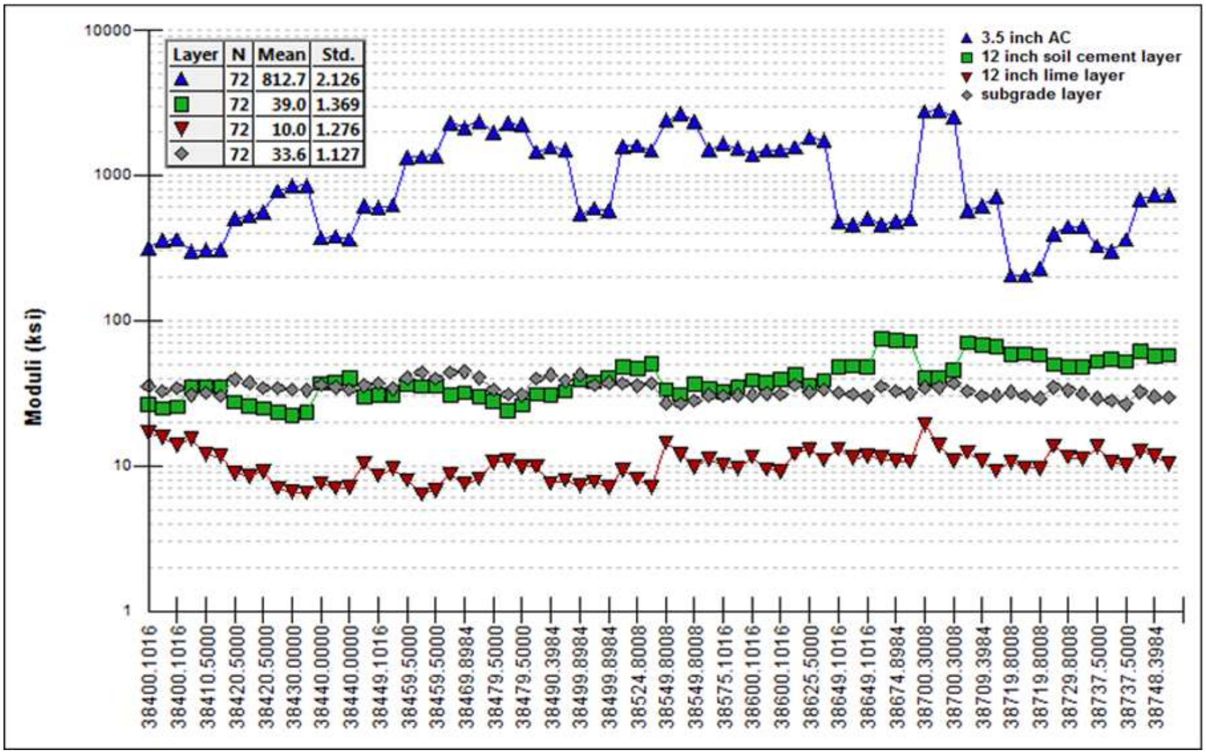


Layer	N	Mean	Std.
▲	87	467.3	1.622
■	87	29.8	2.677
▼	87	25.7	1.428

Subgrade appeared Stronger (14.25, 8.3, 25.7)

Figure 15

LA-8 FWD results on soil cement



Layer	N	Mean	Std.
▲	72	812.7	2.126
■	72	39.0	1.369
▼	72	10.0	1.276
◆	72	33.6	1.127

Figure 16

LA-8 FWD results on AC wearing course

A statistical analysis of these tests shows that the 12-in. layer in Figure 13 had a mean M_S of 12.2 ksi with a standard deviation of 1.533. The remaining subgrade material in Figure 13 had a mean M_S of 16.3 ksi with a standard deviation of 1.539. Combining these two layers suggests the raw subgrade had a strength of 14.25 ksi according to FWD testing.

Looking at Figures 13-16 successively, it can be seen that, according to FWD, the subgrade strength was originally around 14.25 ksi before construction began. It then weakened to 8.3 ksi after lime treatment. Once the soil cement layer was added, the strength in the subgrade increased to 25.7 ksi. And, once the AC layers were added, the strength in the subgrade increased again to a final 33.6 ksi.

For the lime layer, Figures 13-16 show that the strength dropped from 14.25 ksi to 7.6 ksi as a result of the lime treatment. It then increased to 29.8 ksi after the CTB layer was constructed. Then it dropped to 10.0 ksi after the AC layer was constructed.

For the CTB layer, Figures 15 and 16 show that the strength dropped from 467.3 ksi to 39.0 ksi after the AC layer was constructed.

Similar FWD plots like those shown in Figures 13 through 16 have been prepared for the remaining projects listed in Table 2. To save space in the body of this report, these figures are presented in Appendix G.

A summary of the DCP and FWD findings presented in Figures 7-16 as well as in Appendices E, F, and G is presented in Table 5. For each project, it presents the soil type according to NCHRP 9-23b, the date of all FWD and DCP testing, and modulus values in ksi observed for each layer tested on a given FWD or DCP test date.

Table 5
(a) Summary of DCP and FWD testing results

Route	Soil Type NCHRP 9-23b and USDA NCRS (type, thickness)	Date of Test	Mr based on FWD		
			Subgrade		
			(ksi)	(ksi)	(ksi)
	top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (12 in.)	Subgrade
LA-8	A-5 (5.1 in.) A-7-6 (72 in.)	7/16/2013	16.3	12.2	14.3
		7/16/2013			8.3
		7/24/2013			25.7
		1/25/2016			
		7/27/2016			33.6
	top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (12 in.)	Subgrade
US-171	A-5 (5.1 in.) A-7-6 (72 in.)	3/20/2013	4.0	5.6	4.8
		3/20/2013			8.2
		1/25/2016			
		7/27/2016			37.3
	top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (10 in.)	Subgrade
LA-3177	A-4 (14.2 in.) A-4 (45.7 in.)	8/29/2013	5.5	7.9	6.7
		2/10/2016			
		7/26/2016			8.3
	top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (12 in.)	Subgrade
LA-97	A-4 (16.1 in.) A-7-6 (13 in.) A-7-6 (30.7 in.)	7/23/2013	6.5	3.7	5.1
		7/25/2013			6.8
		2/10/2016			
	top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (12 in.)	Subgrade
LA-73	A-4 (7.1 in.) A-4 (18.9 in.) A-4 (28.0 in.) A-4 (16.9 in.)	9/3/2013	2.7	1.8	2.2
		2/2/2016			
	top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (12 in.)	Subgrade
LA-91	A-4 (9.8 in.) A-6 (6.3 in.) A-7-6 (48.8 in.)	9/22/2015	4.5	3.6	4.0
		3/9/2017			12.1
	CONTROL SECTION top bottom		Raw Subgrade not to be Treated	Raw Subgrade to be Treated (12 in.)	Subgrade
US-90	A-4 (5.9 in.) A-6 (34.3 in.) A-6 (19.7 in.)	11/8/2012	3.8	4.6	4.2
		11/15/2012			8.9
		6/10/2013			10.2
		2/1/2016			
		3/6/2017			12.3
		M_r based on DCP for US-90 (summary of results): (due to logistics, LA-91 and US-90 were the only projects that had DCP testing done in the wheelpath during construction)			
		11/9/2012			7.4 (DCP)
		11/16/2012			
		5/28/2013			

Table 5

(b) Summary of DCP and FWD testing results

Route	Soil Type NCHRP 9-23b and USDA NCRS (type, thickness)	Date of Test	Mr based on FWD: Pavement Structure (ksi)	(ksi)	(ksi)	(ksi)	Mr based on DCP: Subgrade (below Lime) (ksi)	Untreated Lime Layer (ksi)	Layers Combined (ksi)
	top / bottom		9% Type-E Lime (12 in.)	6% CTB (12 in.)		AC (3.5 in.)	Raw Soil (228 in.)	Raw Soil (12 in.)	Raw Soil (240 in.)
LA-8	A-5 (5.1 in.) A-7-6 (72 in.)	7/16/2013							
		7/16/2013	7.6						
		7/24/2013	29.8	467.3					
		1/25/2016					4.6	27.3	22.4
		7/27/2016	10.0	39.0		812.7			
	top / bottom		9% Type-E Lime (12 in.)	9% Type-C Lime w/CTB (12 in.)	Class II Base (6in.)	AC (9.0 in.)	Raw Soil (228 in.)	Raw Soil (12 in.)	Raw Soil (240 in.)
US-171	A-5 (5.1 in.) A-7-6 (72 in.)	3/20/2013							
		3/20/2013	10.8						
		1/25/2016					8.3	7.6	7.8
		7/27/2016	49.3	98.2	118.8	537.4			
	top / bottom		10% Type-E Lime (10 in.)	12% CSB (8.5 in.)		AC (4.0 in.)	Raw Soil (230 in.)	Raw Soil (10 in.)	Raw Soil (240 in.)
LA-3177	A-4 (14.2 in.) A-4 (45.7 in.)	8/29/2013							
		2/10/2016					2.4	5.7	4.7
		7/26/2016	528.3	1878.5		784.1			
	top / bottom		9% Type-E Lime (12 in.)	Class II Base (soil cement) (10 in.)		PCC (8 in.)	Raw Soil (228 in.)	Raw Soil (12 in.)	Raw Soil (240 in.)
LA-97	A-4 (16.1 in.) A-7-6 (13 in.) A-7-6 (30.7 in.)	7/23/2013							
		7/25/2013	10.6						
		2/10/2016					6.3	7.5	7.8
	top / bottom		9% Type-E Lime (12 in.)		AC (15 in.)		Raw Soil (228 in.)	Raw Soil (12 in.)	Raw Soil (240 in.)
LA-73	A-4 (7.1 in.) A-4 (18.9 in.) A-4 (28.0 in.) A-4 (16.9 in.)	9/3/2013							
		2/2/2016					22.5	8.3	16.0
	top / bottom		9% cement mixed with 9% lime (12 in.)	In-Place SCB (6 % cement) (8.5 in.)		AC (3.5 in.)	Raw Soil (228 in.)	Raw Soil (12 in.)	Raw Soil (240 in.)
LA-91	A-4 (9.8 in.) A-6 (6.3 in.) A-7-6 (48.8 in.)	9/22/2015							
		3/9/2017	389.4	442.9		3174			
	CONTROL SECTION top / bottom		9% Cement Treated Subgrade (12 in.)	Class II Base (soil cement) (8.5 in.)		AC (3.5 in.)	Raw Soil (228 in.)	Raw Soil (12 in.)	Raw Soil (240 in.)
US-90	A-4 (5.9 in.) A-6 (34.3 in.) A-6 (19.7 in.)	11/8/2012							
		11/15/2012	96.2						
		6/10/2013	107.7	186.2					
		2/1/2016					10.3	8.0	9.2
		3/6/2017	709.0	202.7		2838.5			
		M_r based on DCP for US-90 (summary of results): (due to logistics, LA-91 and US-90 were the only projects that had DCP testing done in the wheelpath during construction)							
		11/9/2012							
		11/16/2012	46.0 (DCP)						
		5/28/2013		70.8 (DCP)					

FWD Assessment of LA-8

Table 6 summarizes how strength figures in the subgrade and lime layers changed during construction based on FWD testing (development comes from Table 5).

Table 6

FWD summary of subgrade strength development on LA-8 (with lime)

	Subgrade	12-in. Lime Layer	Average
Initial Condition	16.3 ksi	12.2 ksi	14.25 ksi
Lime Treated	8.3 ksi[1]	7.6 ksi[1]	7.95 ksi
Post CTB	25.7 ksi	29.8 ksi	27.75 ksi
Post AC	33.6 ksi	10.0 ksi[2]	21.80 ksi
Overall Gain in Strength during Construction (w/ lime):			21.80 – 14.25 = 7.55 ksi

1. The decrease in layer strength was likely due to the soil matrix being disturbed during lime cutting.
2. It is not clear why the strengths in the LIME and CTB layers dropped after the AC was added.

Effect If There Had Been No Lime Treatment on LA-8 (Based on FWD). It is clear from Table 6 that the lime treatment had a detrimental impact on both the 12-in. lime layer and the underlying subgrade. This loss can be approximated by subtracting the “Initial Condition” figure in Table 6 from the “Lime Treated” figure in Table 6:

1. M_R losses in the lime layer due to lime treatment: 7.6 ksi – 12.2 ksi = -4.6 ksi
2. M_R losses in the sub-lime layer due to lime treatment: 8.3 ksi – 16.3 ksi = -8.0 ksi

Approximating what the strength figures would have been had the lime treatment not occurred can be estimated by applying the -4.6 ksi and -8.0 ksi losses to the Table 6 values. The results are provided in Table 7.

It is recognized that it would have been better to have had non-lime treated control sections on LA-8 (along with the other projects in the study) to compare lime treatment results against. This was not possible, though, as project needs could not be placed in construction contracts. As such, the foregoing method of approximating non-lime strengths became a necessity.

Table 7

FWD summary of subgrade strength development on LA-8 (without lime)

	Subgrade	12-in. Lime Layer	Average
Initial Condition	16.3 ksi	12.2 ksi	14.25 ksi
Not-Lime Treated	16.3 ksi	12.2 ksi	14.25 ksi
Post CTB	25.7 - (- 8.0) = 33.7 ksi	29.8 - (- 4.6) = 34.4 ksi	34.05 ksi
Post AC	33.6 - (- 8.0) = 41.6 ksi	10.0 - (- 4.6) = 14.6 ksi	28.10 ksi
Overall Gain in Strength during Construction (w/o lime): 28.10 – 14.25 = +13.85 ksi			

Table 6 shows that the overall increase in strength because of the lime was 7.55 ksi. Table 7 shows that projected overall increase in strength would have been 13.85 ksi if the lime hadn't been used. It can, therefore, be asserted that, according to FWD results, the lime treatment caused a loss in strength equaling 7.55 ksi – 13.85 ksi = -6.30 ksi.

It is believed that this -6.30 ksi drop in strength is attributable to the subgrade material being sandier and drier than is typical for lime usage. There was not enough natural water within the subgrade at the site for the lime to properly hydrate. In addition, the subgrade was not the kind of silty clay that lime is most effective on. According to FWD testing, then, the cutting of lime only served to weaken the existing subgrade strength on LA-8.

FWD Assessment of Remaining Projects

The same methodology can be used to evaluate the impact of lime treatment as reported by FWD on two of the remaining projects, US-171 and US-90. Summaries for these can be found in Tables 8 and 9. There were problems with being able to assess LA-3177, LA-97, LA-73, and LA-91 for reasons that will be elaborated on.

Table 8 shows that the overall increase in strength because of the lime was 38.50 ksi. It also shows that the projected overall increase in strength would have been 33.80 ksi if the lime hadn't been used. It can therefore be asserted that, according to FWD results, the lime treatment caused an overall gain in strength equaling 38.50 ksi – 33.80 ksi = +4.70 ksi.

Table 8
FWD summary of subgrade strength development on US-171

(With Lime)	Subgrade	12-in. Lime Layer	Average
Initial Condition	4.0 ksi	5.6 ksi	4.80 ksi
Lime Treated	8.2 ksi	10.8 ksi	9.50 ksi
Post AC	37.3 ksi	49.3 ksi	43.30 ksi

Overall Gain in Strength during Construction (w/ lime): 43.30 – 4.80 = 38.50 ksi

Strength gains in the lime layer due to lime treatment: 10.8 ksi – 5.6 ksi = +5.2 ksi
Strength gains in the sub-lime layer due to lime treatment: 8.2 ksi – 4.0 ksi = +4.2 ksi

(Without Lime)	Subgrade	12-in. Lime Layer	Average
Initial Condition	4.0 ksi	5.6 ksi	4.80 ksi
Not-Lime Treated	4.0 ksi	5.6 ksi	4.80 ksi
Post AC	37.3 - (+4.2) = 33.1 ksi	49.3 - (+5.2) = 44.1 ksi	38.60 ksi

Overall Gain in Strength during Construction (w/o lime): 38.60 – 4.80 = 33.80 ksi

Table 9 shows that the overall increase in strength because of the lime was 356.45 ksi. It also shows that the projected overall increase in strength would have been 308.10 ksi if the lime hadn’t been used. It can, therefore, be asserted that, according to FWD results, the lime treatment caused an overall GAIN in strength equaling 356.45 ksi – 308.10 ksi = +48.35 ksi.

Table 9
FWD summary of subgrade strength development on US-90

(With Lime)	Subgrade	12-in. Lime Layer	Average
Initial Condition	3.8 ksi	4.6 ksi	4.20 ksi
Lime Treated	8.9 ksi	96.2 ksi	52.55 ksi
Post In-Place SCB	10.2 ksi	107.7 ksi	58.95 ksi
Post AC	12.3 ksi	709.0 ksi	360.65 ksi

Overall Gain in Strength during Construction (w/ lime): 360.65 – 4.20 = 356.45 ksi

Strength gains in the lime layer due to lime treatment: 96.2 ksi – 4.6 ksi = +91.6 ksi
Strength gains in the sub-lime layer due to lime treatment: 8.9 ksi – 3.8 ksi = +5.1 ksi

(Without Lime)	Subgrade	12-in. Lime Layer	Average
Initial Condition	3.8 ksi	4.6 ksi	4.20 ksi
Not-Lime Treated	3.8 ksi	4.6 ksi	4.20 ksi
Post In-Place SCB	10.2 - (+5.1) = 5.1 ksi	107.7 - (+91.6) = 16.1 ksi	10.60 ksi
Post AC	12.3 - (+5.1) = 7.2 ksi	709.0 - (+91.6) = 617.4 ksi	312.30 ksi

Overall Gain in Strength during Construction (w/o lime): 312.30 – 4.20 = 308.10 ksi

FWD testing on the lime layer of LA-3177 was not logistically possible as the contractor constructed the overlying CSB layer before FWD tests could be conducted on top of the lime.

This missing lime layer data made the calculations outlined in Tables 6, 7, and 8 impossible to carry out for LA-3177.

FWD testing on the AC layer of LA-97 was not logistically possible as the FWD was malfunctioning at the time the tests should have been run. This missing lime layer data made the calculations outlined in Tables 6, 7, and 8 impossible to carry out for LA-97.

FWD testing on both the lime layer and the AC layer of LA-73 was not logistically possible. Contractor scheduling and FWD malfunction made it possible to only collect FWD data on the raw subgrade. This missing data made the calculations outlined in Tables 6, 7, and 8 impossible to carry out for LA-73.

FWD testing on the lime layer of LA-91 was not logistically possible as the contractor constructed both the SCB and AC layers before FWD tests could be conducted. This missing lime layer data made the calculations outlined in Tables 6, 7, and 8 impossible to carry out for LA-91.

Table 10 summarizes Tables 6-9 showing the impact that lime treatment had on the various projects according to FWD.

Table 10

Overall subgrade strength development summary according to FWD

Project Name	Overall Impact of Lime Treatment	
	Strength Increased	Strength Decreased
LA-8 (12 in. layer)		-6.30 ksi
US-171 (12 in. layer)	+4.70 ksi	
LA-3177 (10 in. layer)	could not be tabulated	
LA-97 (12 in. layer)	could not be tabulated	
LA-73 (12 in. layer)	could not be tabulated	
LA-91 (12 in. layer)	could not be tabulated	
US-90 (12 in. layer)	+48.35 ksi	

DCP Assessment of LA-8

The last three columns of Table 5b show the DCP based M_R results for the “subgrade below lime” (4.6 ksi), the 12-in. “untreated lime layer” (27.3 ksi) and the “layers combined” (22.4 ksi). Arranging these values as they were in Table 6 produces the figures shown in Table 11.

Table 11
DCP summary of subgrade strength development on LA-8 (with lime)

	Subgrade	12-in. Lime Layer	Layers Combined
Initial Condition	4.6 ksi	27.3 ksi	22.4 ksi
Lime Treated	8.3 ksi	7.6 ksi[1]	7.95 ksi
Post CTB	25.7 ksi	29.8 ksi	27.75 ksi
Post AC	33.6 ksi	10.0 ksi[2]	21.80 ksi
Overall Loss in Strength during Construction (w/ lime):			21.80 – 22.4 = -0.60 ksi

1. The decrease in layer strength was likely due to the soil matrix being disturbed during lime cutting.
2. It is not clear why the strengths in the lime and CTB layers dropped after the AC was added.

Effect If There Had Been No Lime Treatment on LA-8 (Based on DCP). The DCP test indicates that the lime treatment caused the 12-in. lime layer to lose strength and caused the subgrade material below the lime to gain strength. This can be demonstrated by subtracting the "Initial Condition" figure in Table 11 from the "Lime Treated" figure:

1. Strength loss in the lime layer due to lime treatment: 7.6 ksi – 27.3 ksi = -19.7 ksi
2. Strength loss in the sub-lime layer due to lime treatment: 8.3 ksi – 4.6 ksi = +3.7 ksi

Approximating what the strength figures would have been had the lime treatment not occurred can be estimated by applying the -19.7 ksi and 3.7 ksi figures to the Table 11 values. The results are provided in Table 12.

Table 12
DCP summary of subgrade strength development on LA-8 (without lime)

	Subgrade	12-in. Lime Layer	Combined
Initial Condition	4.6 ksi	27.3 ksi	22.4 ksi
Not-Lime Treated	4.6 ksi	27.3 ksi	22.4 ksi
Post CTB	25.7 - (+ 3.7) = 22.0 ksi	29.8 - (- 19.7) = 49.5 ksi	35.75 ksi
Post AC	33.6 - (+ 3.7) = 29.9 ksi	10.0 - (- 19.7) = 29.7 ksi	29.80 ksi
Overall Gain in Strength during Construction (w/o lime):			29.80 – 22.4 = 7.40 ksi

Table 11 shows that the lime treatment caused an overall decrease in strength equaling -0.60 ksi. Table 12 shows that projected overall increase in strength would have been 7.4 ksi if the lime hadn't been used. It can, therefore, be asserted that, according to DCP results, the lime treatment caused a LOSS in strength equaling -0.60 ksi – 7.40 ksi = -8.00 ksi.

DCP Assessment of Remaining Projects

A similar methodology can be used to evaluate the impact of lime treatment as reported by DCP on US-171 and US-90. Summaries for these can be found in Tables 13 and 14. LA-3177, LA-97, LA-73, and LA-91 could not be DCP evaluated as the methodology detailed in Tables 11 and 12 required the use of FWD data that was unavailable for reasons detailed earlier.

Table 13 shows that the overall increase in strength because of the lime was 35.50 ksi. It also shows that the projected overall increase in strength would have been 33.95 ksi if the lime hadn't been used. It can, therefore, be asserted that, according to DCP/FWD results, the lime treatment caused an overall gain in strength equaling 35.50 ksi – 33.95 ksi = +1.55 ksi.

Table 13

DCP summary of subgrade strength development on US-171

<table>
<tr><th>(With Lime)</th><th>Subgrade</th><th>12-in. Lime Layer</th><th>Combined</th></tr>
<tr><td>Initial Condition</td><td>8.3 ksi</td><td>7.6 ksi</td><td>7.8 ksi</td></tr>
<tr><td>Lime Treated</td><td>8.2 ksi</td><td>10.8 ksi</td><td>9.50 ksi</td></tr>
<tr><td>Post AC</td><td>37.3 ksi</td><td>49.3 ksi</td><td>43.30 ksi</td></tr>
<tr><td colspan="4">Overall Gain in Strength during Construction (w/ lime): 43.30 – 7.8 = 35.50 ksi</td></tr>
<tr><td colspan="4">Strength gains in the lime layer due to lime treatment: 10.8 ksi – 7.6 ksi = +3.2 ksi
Strength gains in the sub-lime layer due to lime treatment: 8.2 ksi – 8.3 ksi = -0.1 ksi</td></tr>
<tr><th>(Without Lime)</th><th>Subgrade</th><th>12-in. Lime Layer</th><th>Combined</th></tr>
<tr><td>Initial Condition</td><td>8.3 ksi</td><td>7.6 ksi</td><td>7.8 ksi</td></tr>
<tr><td>Not-Lime Treated</td><td>8.3 ksi</td><td>7.6 ksi</td><td>7.8 ksi</td></tr>
<tr><td>Post AC</td><td>37.3 - (-0.1) = 37.4 ksi</td><td>49.3 - (+3.2) = 46.1 ksi</td><td>41.75 ksi</td></tr>
<tr><td colspan="4">Overall Gain in Strength during Construction (w/o lime): 41.75 – 7.8 = 33.95 ksi</td></tr>
</table>

Table 14 shows that the overall increase in strength because of the lime was 351.45 ksi. It also shows that the projected overall increase in strength would have been 308.05 ksi if the lime hadn't been used. It can, therefore, be asserted that, according to FWD results, the lime treatment caused an overall gain in strength equaling 351.45 ksi – 308.05 ksi = +43.40 ksi.

Table 14
DCP summary of subgrade strength development on US-90

(With Lime)	**Subgrade**	**12-in. Lime Layer**	**Combined**
Initial Condition	10.3 ksi	8.0 ksi	9.2 ksi
Lime Treated	8.9 ksi	96.2 ksi	52.55 ksi
Post In-Place SCB	10.2 ksi	107.7 ksi	58.95 ksi
Post AC	12.3 ksi	709.0 ksi	360.65 ksi
Overall Gain in Strength during Construction (w/ lime): 360.65 – 9.2 = 351.45 ksi			
Strength gains in the lime layer due to lime treatment: 96.2 ksi – 8.0 ksi = +88.2 ksi			
Strength gains in the sub-lime layer due to lime treatment: 8.9 ksi – 10.3 ksi = -1.4 ksi			
(Without Lime)	**Subgrade**	**12-in. Lime Layer**	**Combined**
Initial Condition	10.3 ksi	8.0 ksi	9.2 ksi
Not-Lime Treated	10.3 ksi	8.0 ksi	9.2 ksi
Post In-Place SCB	10.2 - (-1.4) = 11.6 ksi	107.7 - (+88.2) = 19.5 ksi	15.55 ksi
Post AC	12.3 - (-1.4) = 13.7 ksi	709.0 - (+88.2) = 620.8 ksi	317.25 ksi
Overall Gain in Strength during Construction (w/o lime): 317.25 – 9.2 = 308.05 ksi			

Table 15 summarizes Tables 11 through 14 showing the impact that lime treatment had on the various projects according to FWD.

Table 15
Overall subgrade strength development summary according to DCP

Project Name	**Overall Impact of Lime Treatment**	
	Strength Increased	**Strength Decreased**
LA-8 (12-in. layer)		-8.00 ksi
US-171 (12-in. layer)	+1.55 ksi	
LA-3177 (10-in. layer)	could not be tabulated	
LA-97 (12-in. layer)	could not be tabulated	
LA-73 (12-in. layer)	could not be tabulated	
LA-91 (12-in. layer)	could not be tabulated	
US-90 (12-in. layer)	+43.40 ksi	

CONCLUSIONS

- The increase in modulus figures of Figures 13-16 and in Appendix G are indicative of compaction and shearing taking place when FWD testing was conducted on raw subgrade and atop lime treated material. Such shearing and compaction violates back-calculation theory and draws into question the results derived from FWD tests conducted on these materials.

- There were a number of questionable results that FWD testing reported. FWD testing, for example, had reported that the lime layer strength on LA-8 had dropped unexpectedly from 29.8 ksi to 10 ksi once the AC layer was in place. A similar unexpected drop in strength occurred in the CTB layer from 467.3 ksi to 39.0 ksi once the AC was in place according to FWD. It is not clear what caused this apparent weakening nor if it can be believed.

- Similar questionable FWD results were also observed that called into question much of the remaining FWD analysis. The strength figures on LA-3177 for the lime and CSB layers (528.3 ksi and 1878.5 ksi, respectively) appeared to be too high to be believable. The strength figures on LA-91 for the lime-cement, in-place SCB and AC layers (389.4 ksi, 442.9 ksi and 3174 ksi, respectively) also appeared to be too high to be believable as did the strength figure for the AC layer on US-90 (2838.5 ksi).

- As shown in Tables 10 and 15, only three projects underwent enough FWD and DCP testing for the effects of lime treatment to be assessed. LA-8 showed a 6.30 ksi weakening according to FWD (8.00 ksi according to DCP). US-171 showed a 4.70 ksi strengthening according to FWD (1.55 ksi according to DCP). US-90, the control section where lime was not used, showed a 48.35 ksi strengthening according to FWD (43.40 ksi according to DCP). Only US-171 did not show any of the questionable strength characteristics detailed in the previous bullet.

- The literature suggests that there are problems with assuming that homogeneous subgrades can be treated as two layer systems as was done in this project. Mallick and El-Korchi state that to use the MET, moduli should be decreasing with depth, preferably by a factor of at least 2 between consecutive layers *[10]*. This was not the case for any of the subgrades that were examined for this project and might explain some of the questionable results that were observed.

RECOMMENDATIONS

- No further work should be done on this research as results are inconclusive.
- It proved very difficult to coordinate research needs with contractor scheduling. If any further testing is to be undertaken, it is recommended that provisions be made for said testing contractually.
- Testing of raw subgrades and lime-treated subgrades, especially when excessively weak, should be avoided as it violates MET theory.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AC	Asphaltic Concrete
CTB	Cement Treated Base
DCP	Dynamic Cone Penetrometer
DOTD	Louisiana Department of Transportation and Development
ELMOD	Evaluation of Layer Moduli and Overlay Design
EMA	Equivalent Modulus Analysis
F	Load that the System is expected to Support
FWD	Falling Weight Deflectometer
H	Thickness of the Treated Layer
HWD	Heavy Weight Deflectometer
K-value	Modulus of Subgrade Reaction
Ksi	Kips per in. sq.
LTRC	Louisiana Transportation Research Center
M_{EQ}	Resilient Modulus of the Equivalency
MET	Method of Equivalent Thicknesses
M_R	Resilient Modulus of the Treated Layer
M_S	Resilient Modulus of the Pre-Existing Subgrade Material
O-B	Odemark-Boussinesq
PCC	Portland Cement Concrete
SCB	Soil Cement Base

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APPENDIX A

Method of Equivalent Thickness Summary

The Equivalent Modulus Analysis Spreadsheet is designed to allow pavement designers to model two-layer pavement systems as single layer equivalents. Typically, when a weak subgrade is encountered in the field, the practice is to overlay the weak layer with a strengthening layer. The objective of this practice is to cause the composite system to achieve an intended design strength. Functionally and for reasons relating to practicality, most designers prefer to envision such two-layer systems as a single monolithic structure having a single equivalent resilient modulus.

Determination of this Single Equivalent Resilient Modulus is a complex proposition as there are four parameters which govern the equivalency. These four factors include the Subgrade Modulus of Elasticity (M_S), the Overly Material Modulus of Elasticity (M_R), the Thickness of the Overlay (H), and the Drop Force (F) as applied during Falling Weight Deflectometer (FWD) testing. A Falling Weight Deflectometer is used to stimulate the pavement so that the other variables can be evaluated. A diagram which shows these four variables as well as the single layer equivalent they can be modeled as is provided in Figure A1. The Equivalent Resilient Modulus (M_{EQ}) on the right side of the figure summarizes the equivalency:

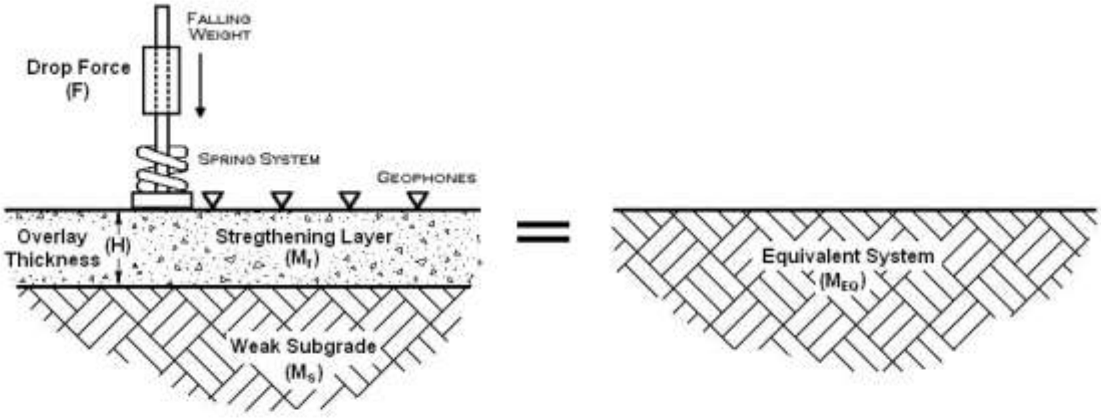


Figure A1
Representation of variables associated with pavement system equivalency

In practice, the equivalency is arrived at through iterative applications of Odemark's Method of Equivalent Thickness and the Boussinesq Equation *[3]*:

1. ***Odemark:***

$$z_{EQ} = 0.9*(H)\left[\frac{M_R}{M_S}\right]^{1/3} \tag{1}$$

2. ***Boussinesq:***

$$\delta[z, M_i] = \frac{(1+\mu)PR}{M_i} * \left[\frac{1}{\sqrt{1+\left(\frac{z}{R}\right)^2}} + (1-2\mu)\left[\sqrt{1+\left(\frac{z}{R}\right)^2} - \left(\frac{z}{R}\right)\right]\right] \tag{2}$$

3. ***Modified Boussinesq***
(sets z=0 and solves for M_i):

$$M_{EQ} = \frac{(1+\mu)PR}{\delta_{TOT}} * \left[1 + (1-2\mu)\right] \tag{3}$$

where,

M_S	Subgrade Modulus of Elasticity (ksi);
M_R	Overly Material Modulus of Elasticity (ksi);
H	Overlay Thickness (in.);
z	Depth below pavement surface (in.);
z_{EQ}	Odemark based equivalent depth (in.);
R	Radius of FWD Impact Plate (assumed to be 8.85 in. for all calculations);
P	FWD Plate Pressure (psi);
F	FWD Applied Load (assumed to be 2000 lbf for all calculations);
μ	Poisson's Ratio (assumed to be 0.35 for all calculations); and
$\delta[z,M_i]$	Displacement of material with Modulus M_i (ksi) and at depth z (in.) due to load F.

The FWD Plate is circular. As such, Plate Pressure (P) can be arrived at as follows:

$$P = \frac{F}{\pi * R^2} \tag{4}$$

Since F and R are constants, the FWD Plate Pressure (P) will also be a constant: 8.13 psi.

Equivalent Resilient Modulus values (M_{EQ}) for a given set of conditions is arrived at in the following manner:

Step 1: Assume values for M_S, M_R, F, and H.
Step 2: The Boussinesq Equation is used to calculate $\delta[0, M_R]$.

Step 3: The Boussinesq Equation is used to calculate $\delta[H, M_R]$.

Step 4: Compression in the Overlay Layer resulting from application of Force (F) is calculated:

$$\delta_1=\delta[0, M_R]- \delta[H, M_R]. \tag{5}$$

Step 5: The Odemark Equation is used to calculate z_{EQ}.

Step 6: The Boussinesq Equation is used to calculate $\delta[z_{EQ.},M_S]$.

Step 7: The Total Displacement seen at the surface is calculated: $\delta_{TOT}=\delta_1+\delta[z_{EQ.}, M_S]$.

Step 8: The Modified Boussinesq Equation is used to calculate M_{EQ}.

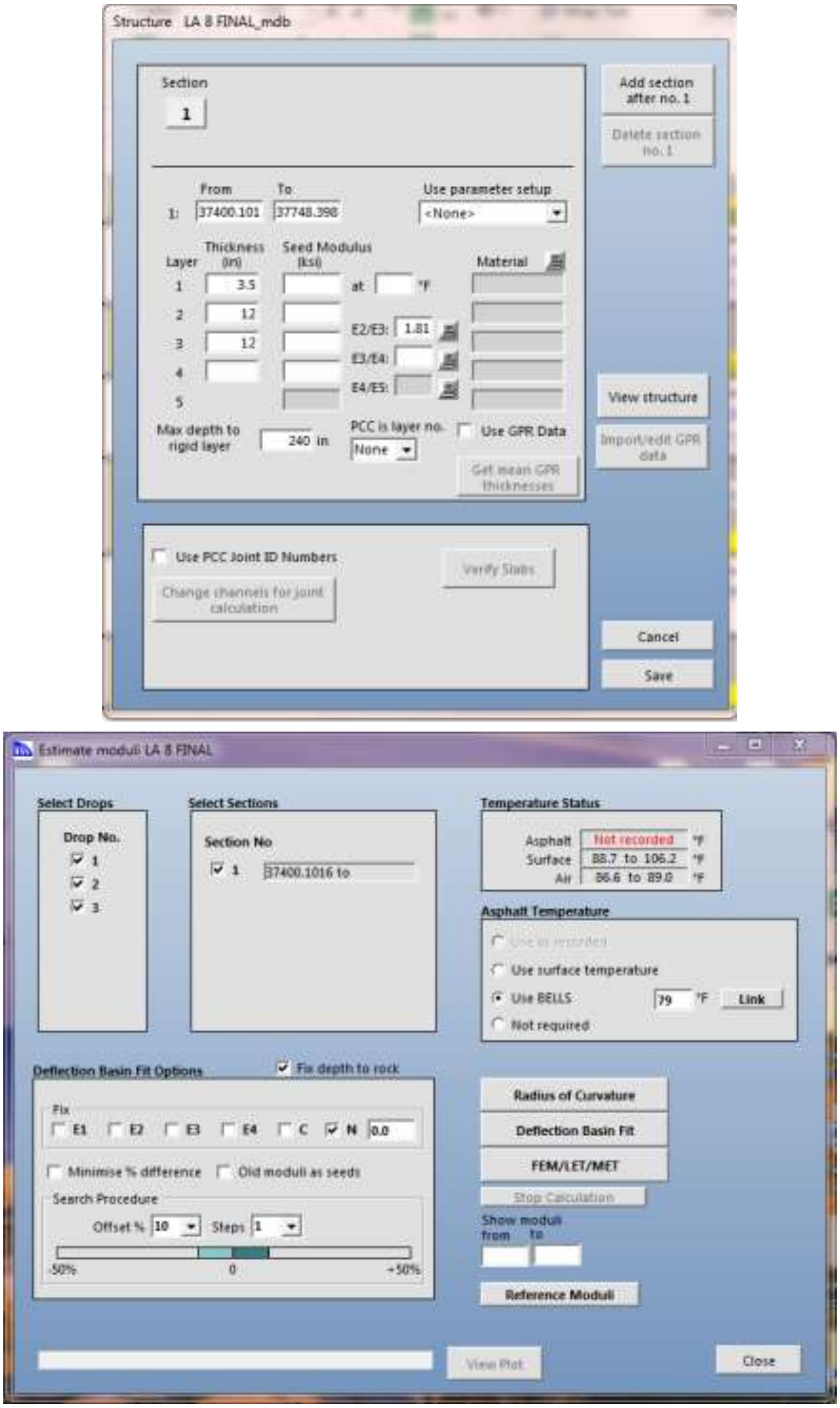
Step 9: A new set of values are assumed for M_S, M_R, F, and H and the steps are repeated.

Varying one variable repeatedly in Step 9 while recording the iterations produces a curve. Iterating two or more variables repeatedly in Step 9 while recording the iterations produces a family of curves. This was the method used to generate the Equivalent Modulus Analysis Spreadsheet.

The Graph A tab of the Equivalent Modulus Analysis Spreadsheet allows the user to examine the relationship that exists between variables wherein Drop Force and Base Thickness can be selected. The Graph B tab allows the user to examine the relationship that exists between variables wherein the Drop Force and Subgrade Modulus can be selected.

APPENDIX B

ELMOD Settings



Structure LA 8 FINAL_mdb

Section 1

1: From 37400.101 To 37748.398

Use parameter setup: <None>

Layer	Thickness (in)	Seed Modulus (ksi)
1	3.5	
2	12	
3	12	
4		
5		

at °F

E2/E3: 1.81

E3/E4:

E4/E5:

Max depth to rigid layer: 240 in

PCC is layer no.: None

☐ Use GPR Data

☐ Use PCC Joint ID Numbers

Estimate moduli LA 8 FINAL

Select Drops — Drop No.: ☑ 1 ☑ 2 ☑ 3

Select Sections — Section No: ☑ 1 37400.1016 to

Temperature Status:

Asphalt: Not recorded °F

Surface: 88.7 to 106.2 °F

Air: 86.6 to 89.0 °F

Asphalt Temperature:

○ Use as recorded

○ Use surface temperature

● Use BELLS 79 °F

○ Not required

Deflection Basin Fit Options — ☑ Fix depth to rock

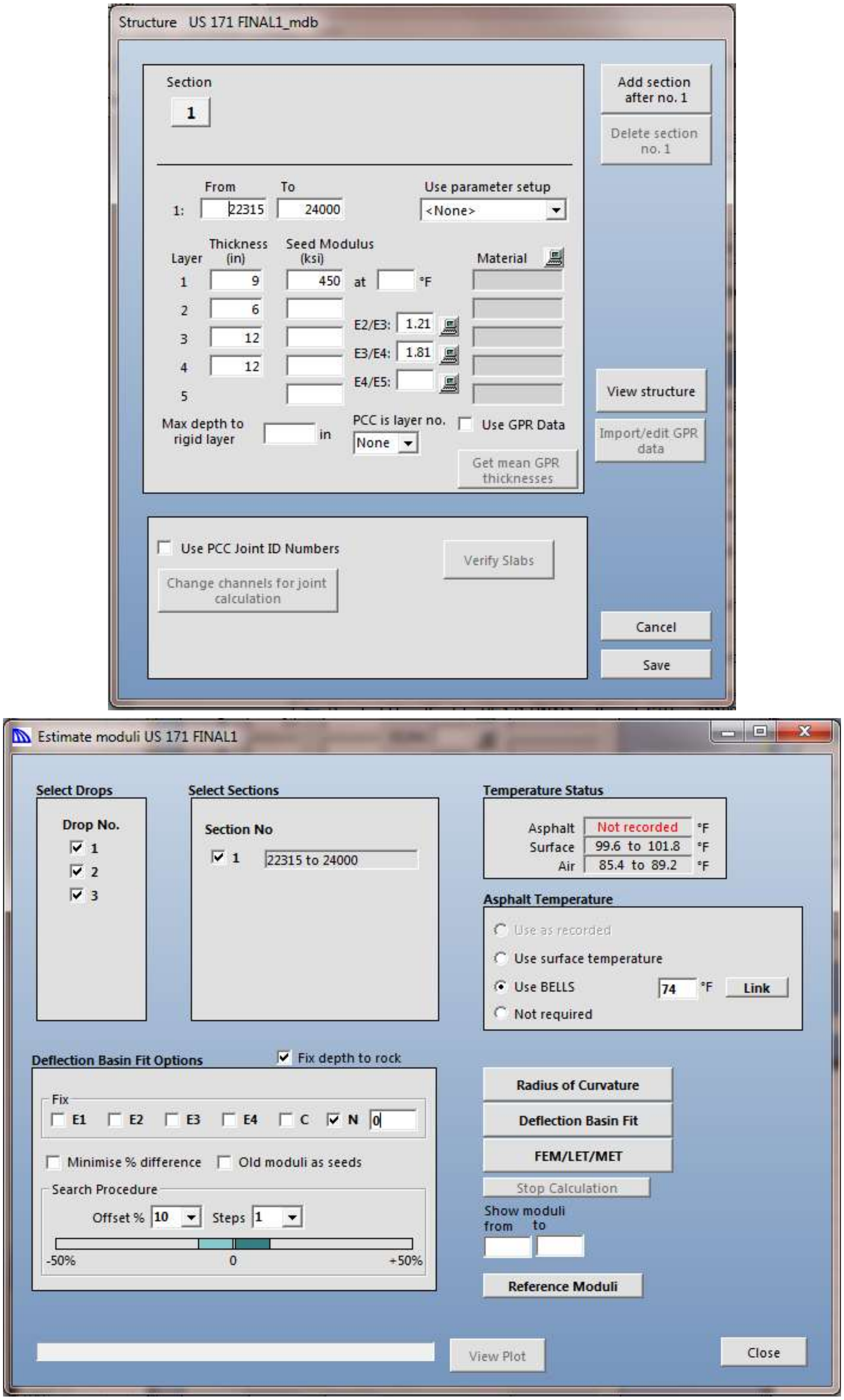
Fix: ☐ E1 ☐ E2 ☐ E3 ☐ E4 ☐ C ☑ N 0.0

☐ Minimise % difference ☐ Old moduli as seeds

Search Procedure: Offset % 10 Steps 1

Figure B1

ELMOD multi-layer settings for LA-8


Structure US 171 FINAL1_mdb

Section 1

	From	To	Use parameter setup
1:	22315	24000	<None>

Layer	Thickness (in)	Seed Modulus (ksi)	
1	9	450	at °F
2	6		E2/E3: 1.21
3	12		E3/E4: 1.81
4	12		E4/E5:
5			

Max depth to rigid layer in

PCC is layer no. None

☐ Use GPR Data

☐ Use PCC Joint ID Numbers

Estimate moduli US 171 FINAL1

Select Drops

Drop No.
☑ 1
☑ 2
☑ 3

Select Sections

Section No
☑ 1 22315 to 24000

Temperature Status

Asphalt	Not recorded	°F
Surface	99.6 to 101.8	°F
Air	85.4 to 89.2	°F

Asphalt Temperature
○ Use as recorded
○ Use surface temperature
● Use BELLS 74 °F
○ Not required

Deflection Basin Fit Options ☑ Fix depth to rock

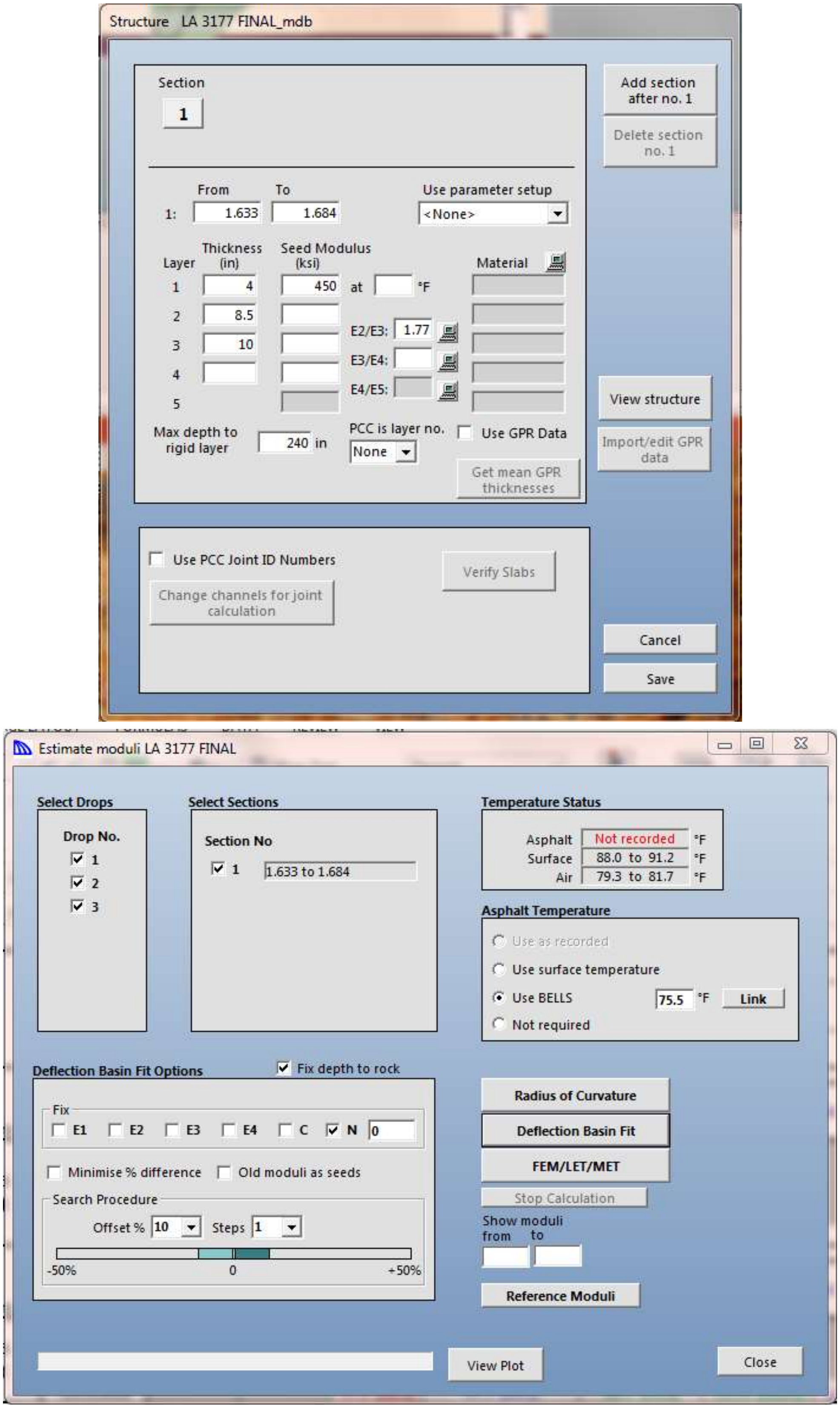
Fix ☐ E1 ☐ E2 ☐ E3 ☐ E4 ☐ C ☑ N 0

☐ Minimise % difference ☐ Old moduli as seeds

Search Procedure
Offset % 10 Steps 1

Figure B2

ELMOD multi-layer settings for US-171


Structure LA 3177 FINAL_mdb

Section 1

From 1.633 To 1.684

Use parameter setup: <None>

Layer	Thickness (in)	Seed Modulus (ksi)
1	4	450
2	8.5	
3	10	
4		
5		

E2/E3: 1.77

E3/E4:

E4/E5:

Max depth to rigid layer 240 in

PCC is layer no. None

☐ Use GPR Data

☐ Use PCC Joint ID Numbers

Estimate moduli LA 3177 FINAL

Select Drops

Drop No.
☑ 1
☑ 2
☑ 3

Select Sections

Section No
☑ 1 1.633 to 1.684

Temperature Status

Asphalt Not recorded °F
Surface 88.0 to 91.2 °F
Air 79.3 to 81.7 °F

Asphalt Temperature

○ Use as recorded
○ Use surface temperature
● Use BELLS 75.5 °F
○ Not required

Deflection Basin Fit Options ☑ Fix depth to rock

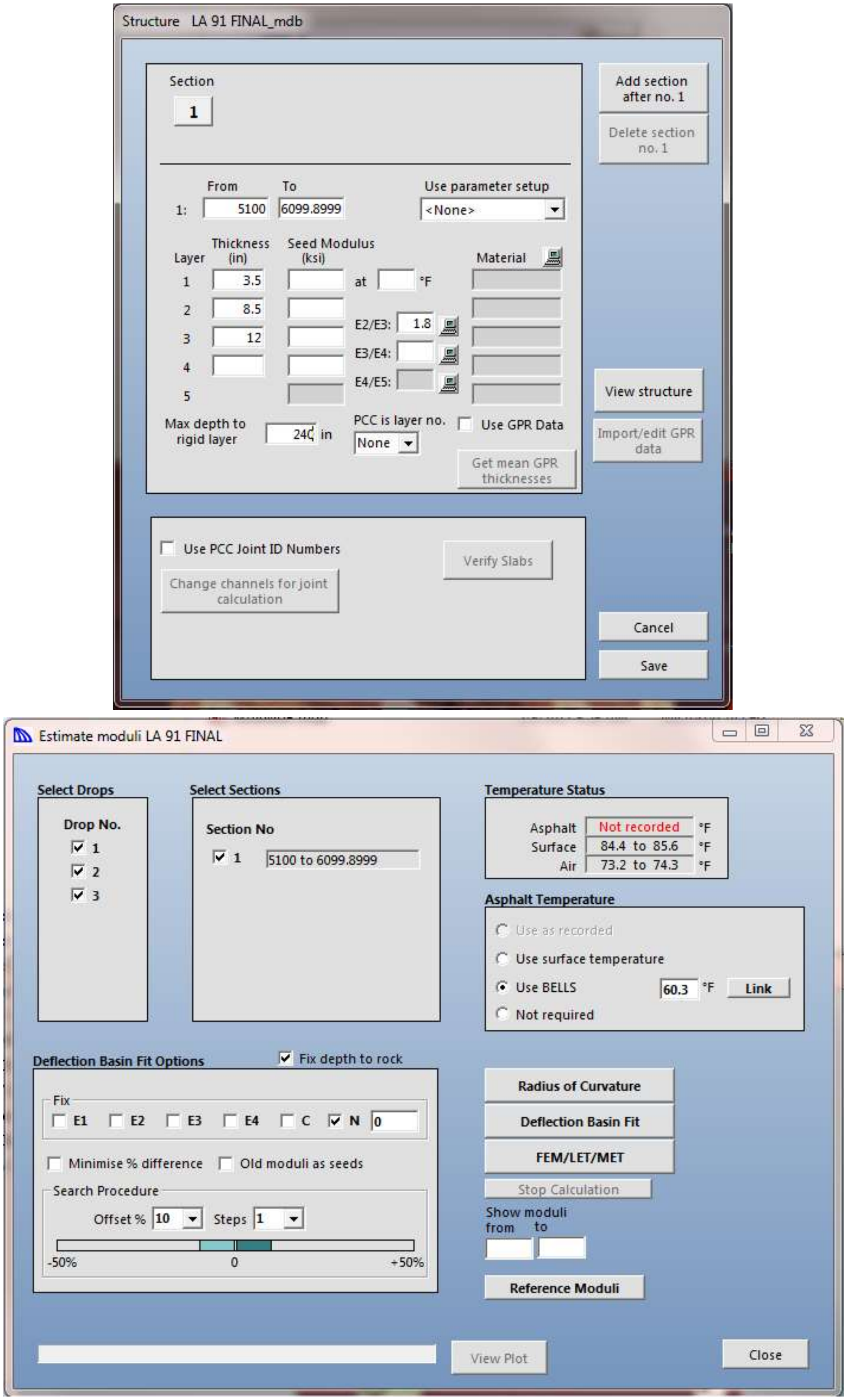
Fix: ☐ E1 ☐ E2 ☐ E3 ☐ E4 ☐ C ☑ N 0

☐ Minimise % difference ☐ Old moduli as seeds

Search Procedure: Offset % 10 Steps 1

Figure B3

ELMOD multi-layer settings for LA-3177


Structure LA 91 FINAL_mdb

Section 1

From 5100 To 6099.8999

Use parameter setup <None>

Layer	Thickness (in)	Seed Modulus (ksi)
1	3.5	
2	8.5	
3	12	
4		
5		

E2/E3: 1.8

E3/E4:

E4/E5:

Max depth to rigid layer 240 in

PCC is layer no. None

☐ Use GPR Data

☐ Use PCC Joint ID Numbers

Estimate moduli LA 91 FINAL

Select Drops

Drop No.
☑ 1
☑ 2
☑ 3

Select Sections

Section No
☑ 1 5100 to 6099.8999

Temperature Status

Asphalt	Not recorded	°F
Surface	84.4 to 85.6	°F
Air	73.2 to 74.3	°F

Asphalt Temperature

○ Use as recorded
○ Use surface temperature
● Use BELLS 60.3 °F
○ Not required

Deflection Basin Fit Options

☑ Fix depth to rock

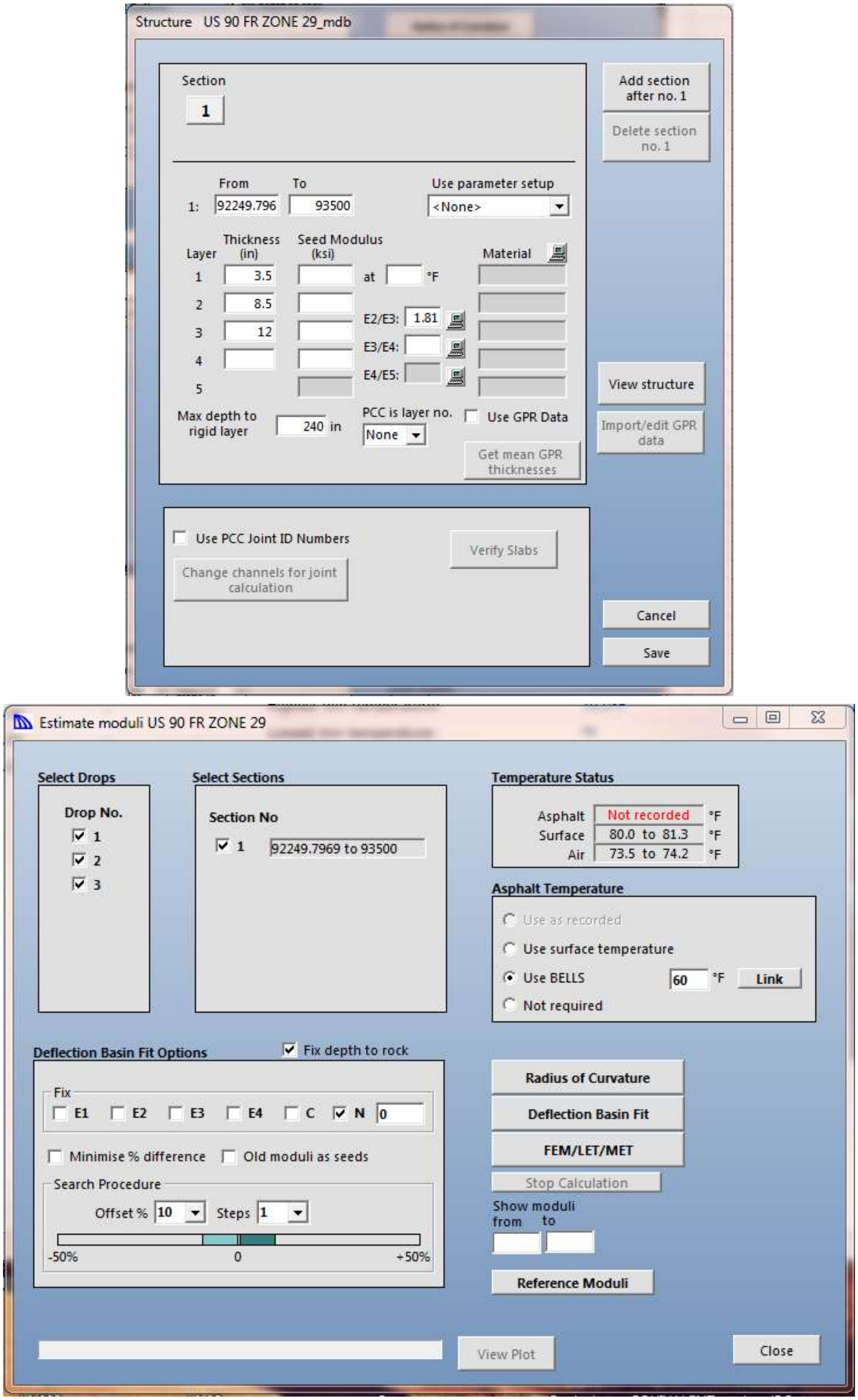
Fix: ☐ E1 ☐ E2 ☐ E3 ☐ E4 ☐ C ☑ N 0

☐ Minimise % difference ☐ Old moduli as seeds

Search Procedure: Offset % 10 Steps 1

Figure B4

ELMOD multi-layer settings for LA-91



Structure US 90 FR ZONE 29_mdb

Section 1

	From	To	Use parameter setup
1:	92249.796	93500	<None>

Layer	Thickness (in)	Seed Modulus (ksi)
1	3.5	
2	8.5	
3	12	
4		
5		

E2/E3: 1.81

E3/E4:

E4/E5:

Max depth to rigid layer 240 in

PCC is layer no. None

☐ Use GPR Data

☐ Use PCC Joint ID Numbers

Estimate moduli US 90 FR ZONE 29

Select Drops

Drop No.

☑ 1

☑ 2

☑ 3

Select Sections

Section No

☑ 1 92249.7969 to 93500

Temperature Status

Asphalt	Not recorded	°F
Surface	80.0 to 81.3	°F
Air	73.5 to 74.2	°F

Asphalt Temperature

○ Use as recorded

○ Use surface temperature

● Use BELLS 60 °F

○ Not required

Deflection Basin Fit Options ☑ Fix depth to rock

Fix ☐ E1 ☐ E2 ☐ E3 ☐ E4 ☐ C ☑ N 0

☐ Minimise % difference ☐ Old moduli as seeds

Search Procedure

Offset % 10 Steps 1

Figure B5

ELMOD multi-layer settings for US-90

APPENDIX C

Project and FWD Test Location Maps

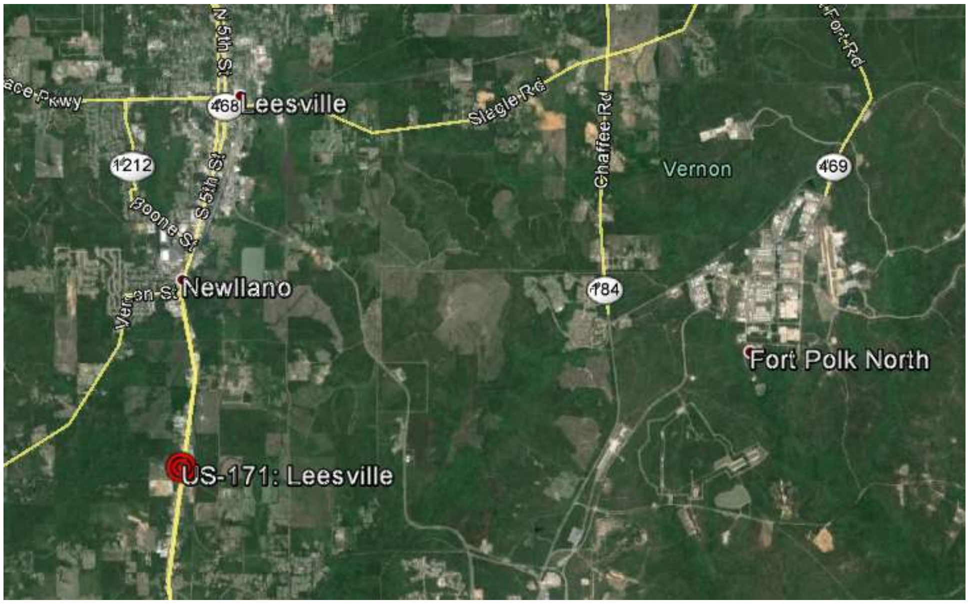


Figure C1
Map of US 171 project (detail)

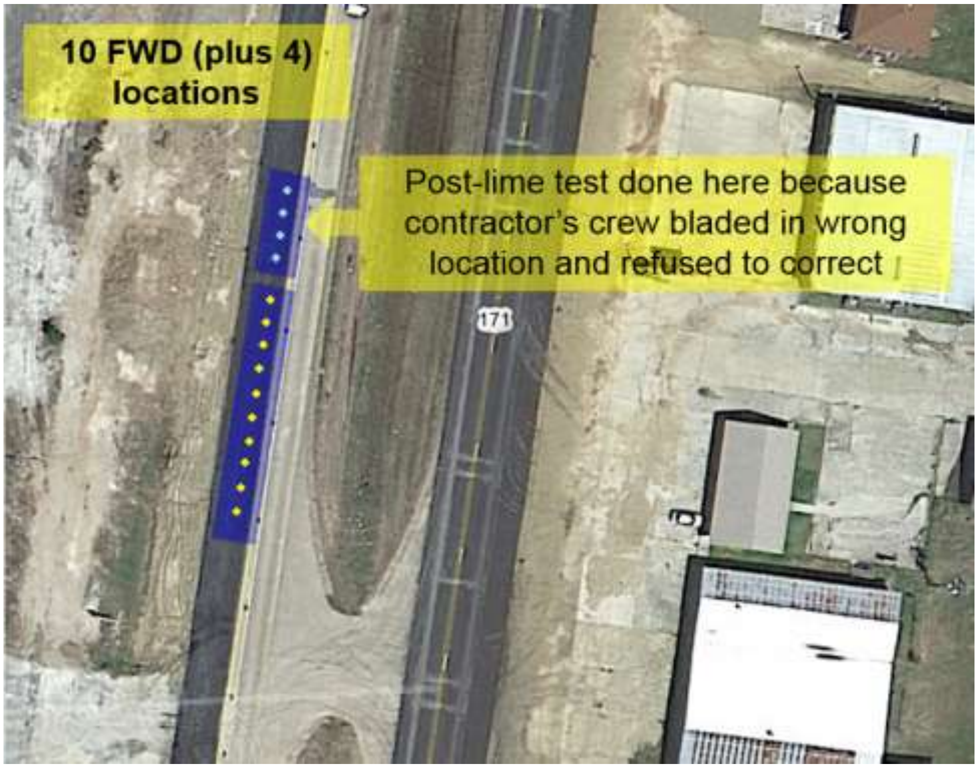


Figure C2
FWD test locations along US 171

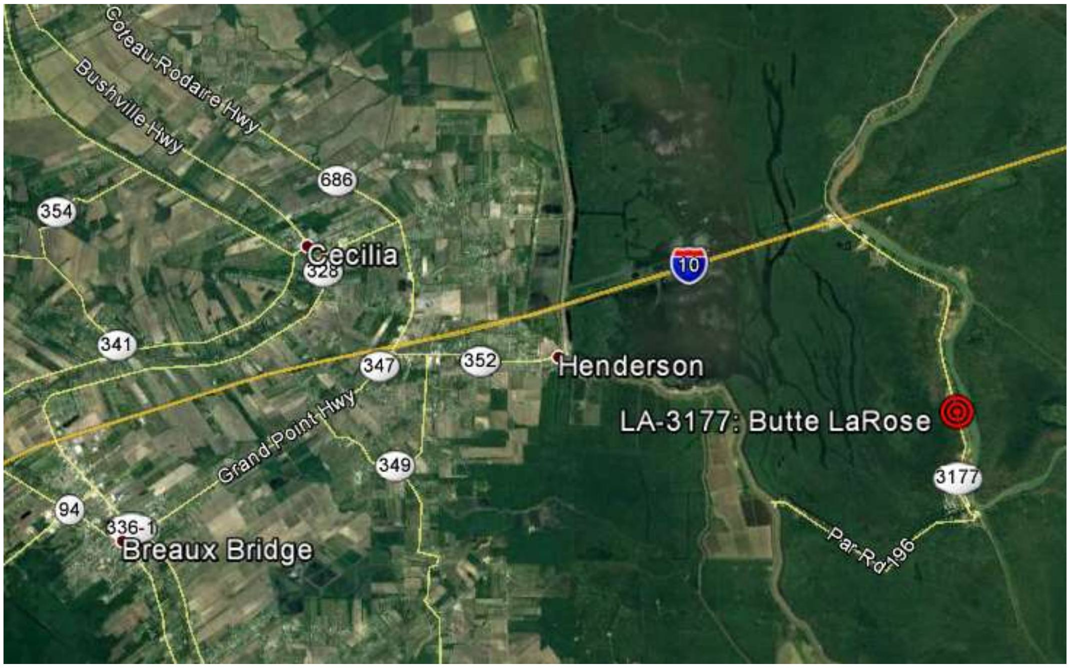


Figure C3
Map of US 3177 project (detail)

Figure C4
FWD test locations along US 3177

Figure C5
Map of LA 97 project (detail)

Figure C6
FWD test locations along LA 97

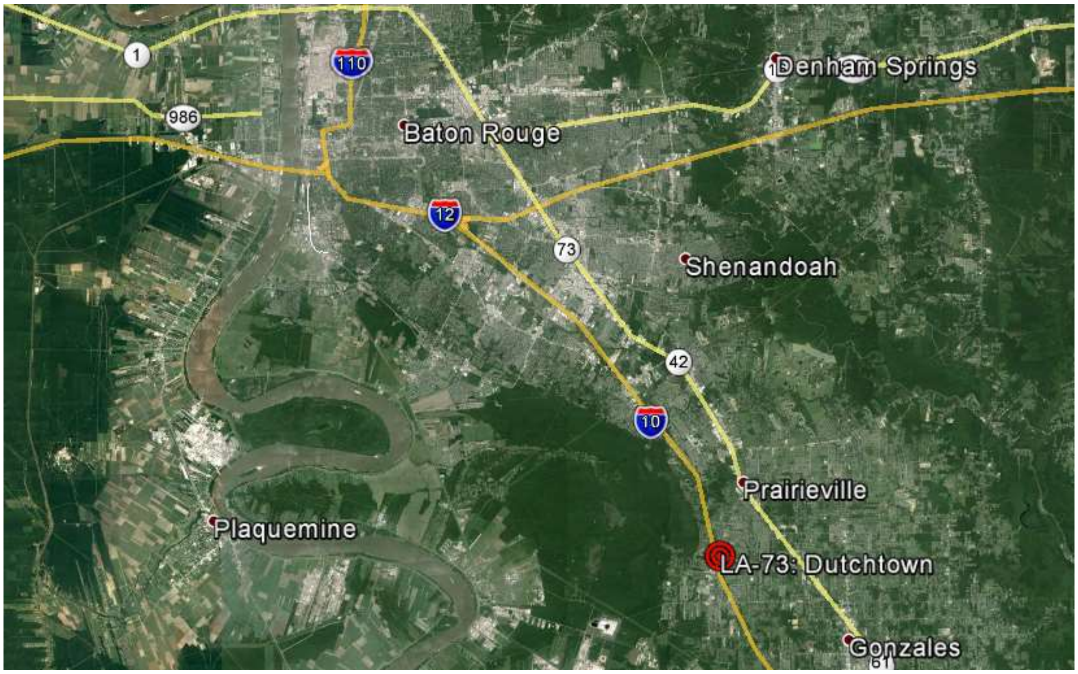


Figure C7
Map of LA 73 project (detail)

Figure C8
FWD test locations along LA 73

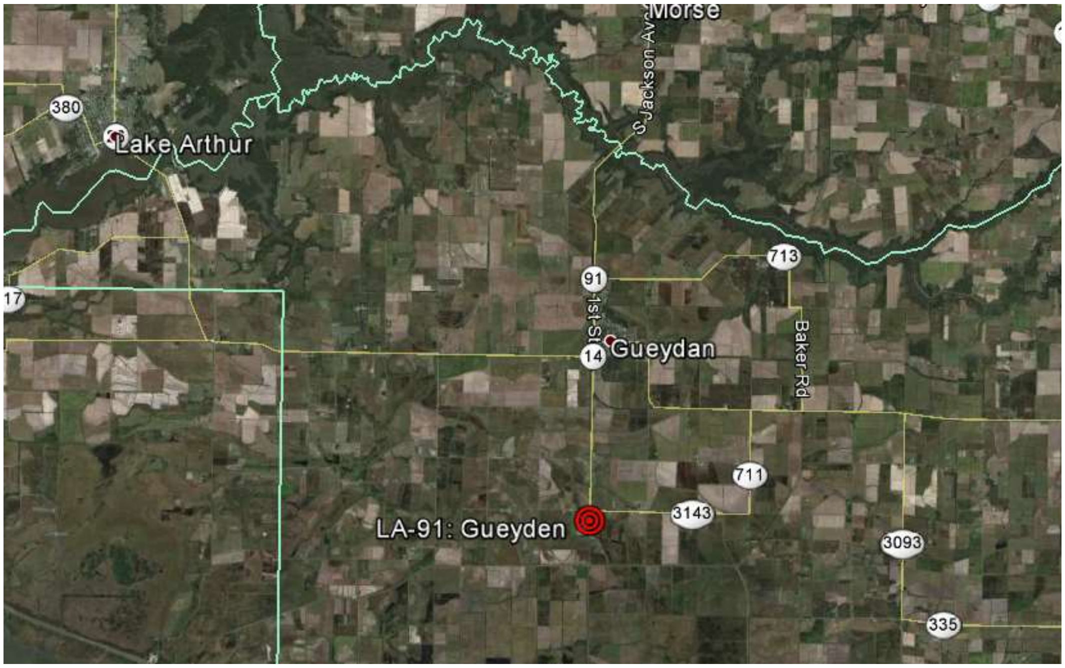


Figure C9

Map of LA-91 project (detail)

Figure C10

FWD test locations along LA 91

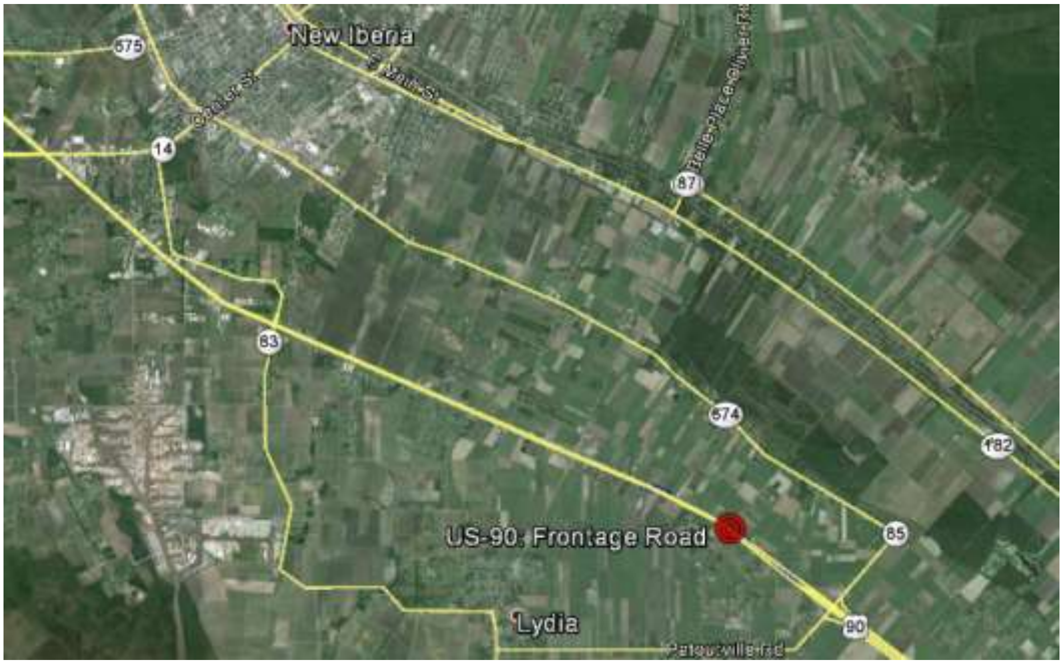


Figure C11

Map of US 90 project (detail)

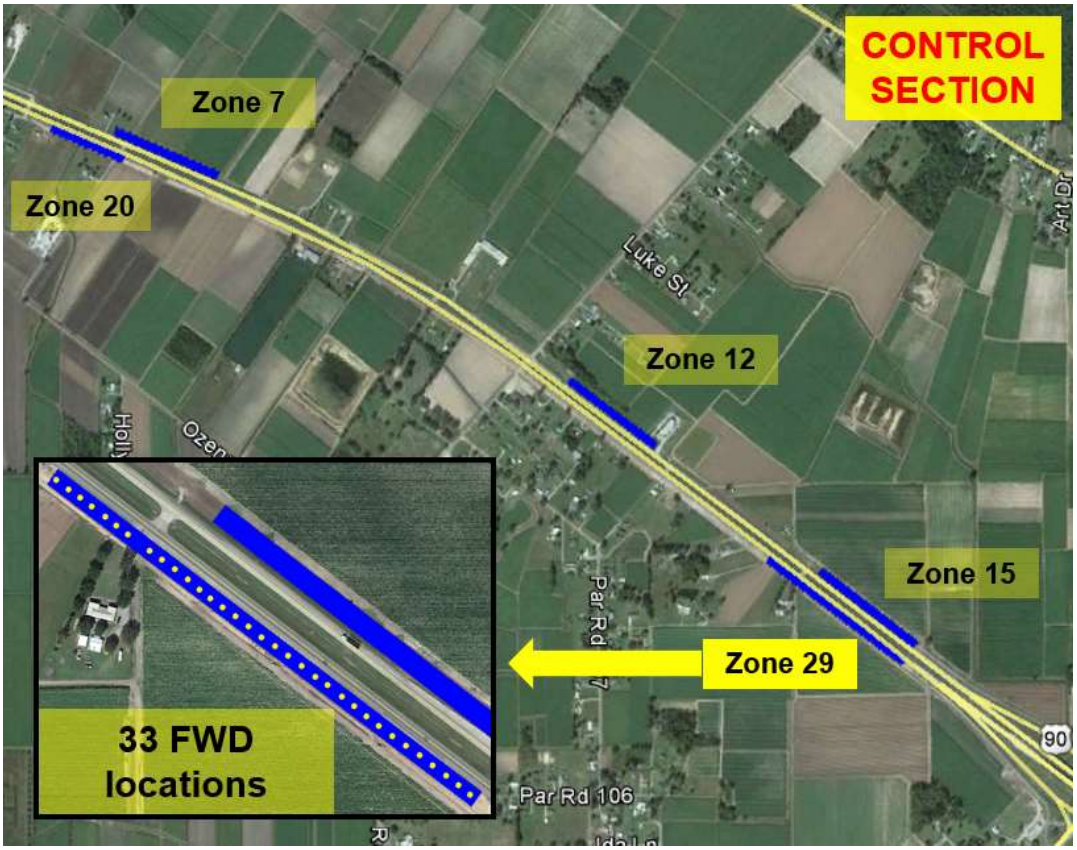


Figure C12

FWD test locations along US 90

APPENDIX D

Project Cross-Sections

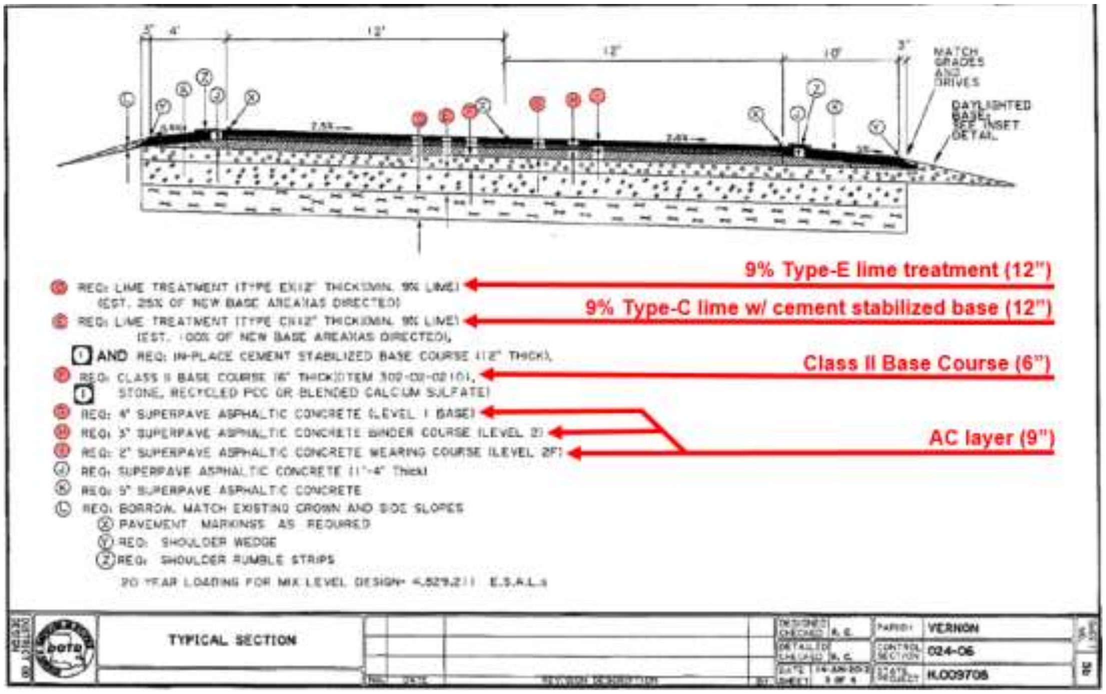


Figure D1
US 171 cross-section

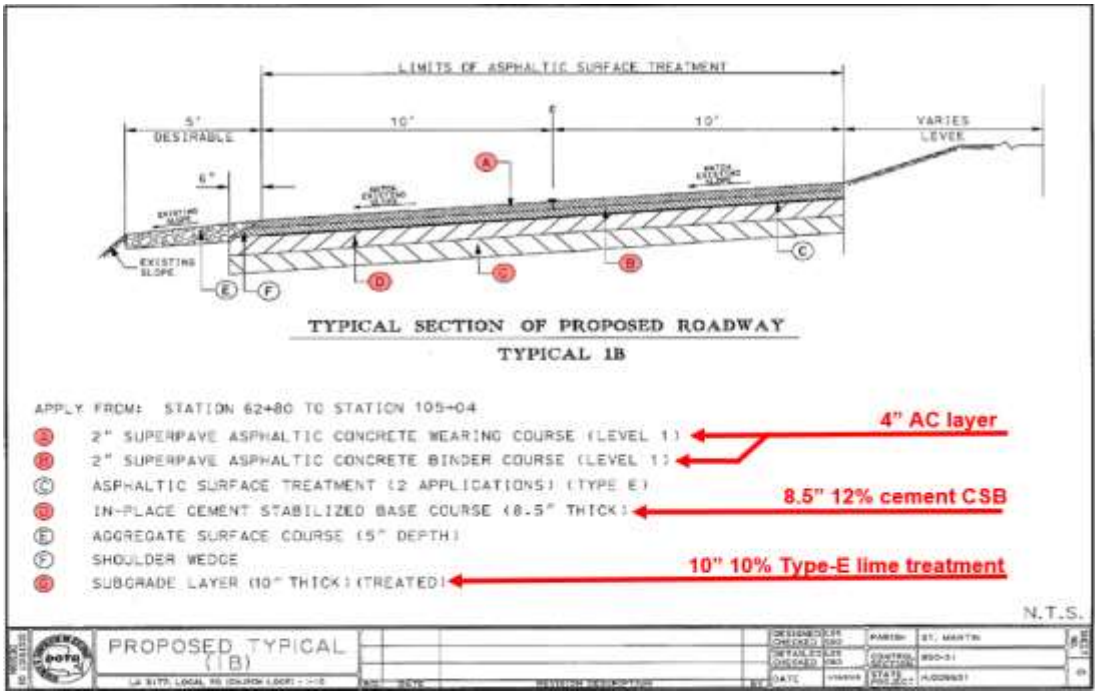


Figure D2
LA 3177 cross-section

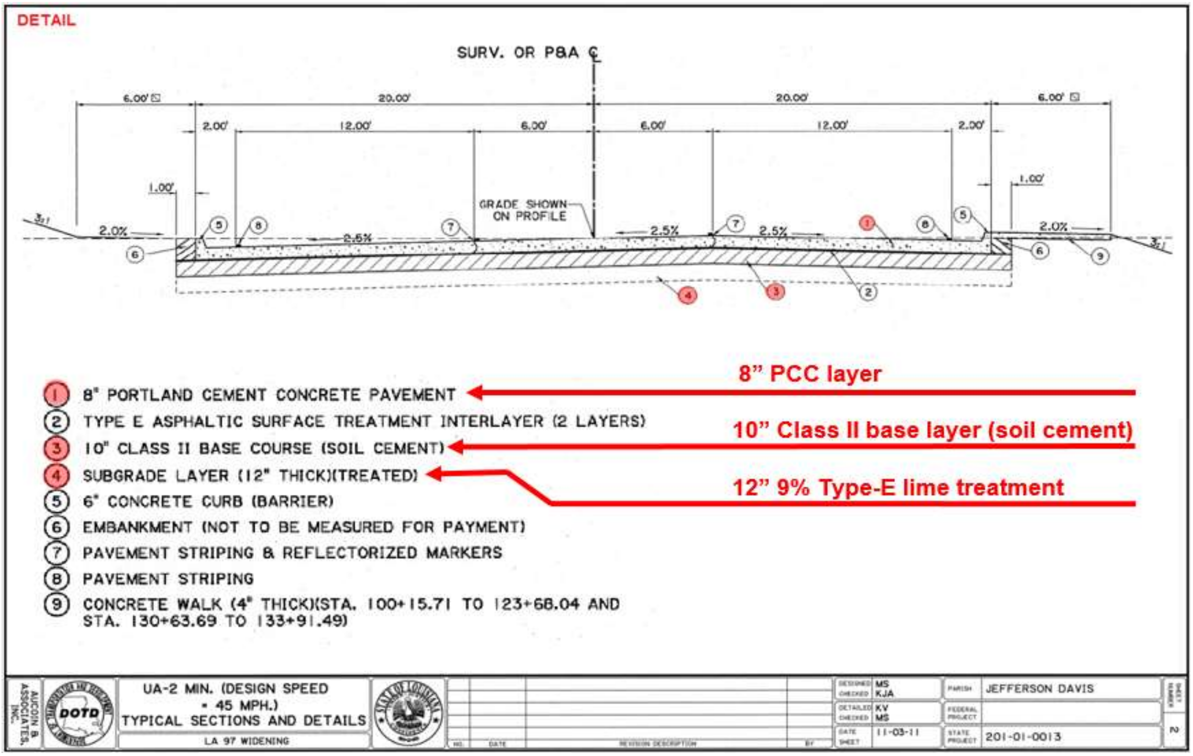


Figure D3

LA 97 cross-section

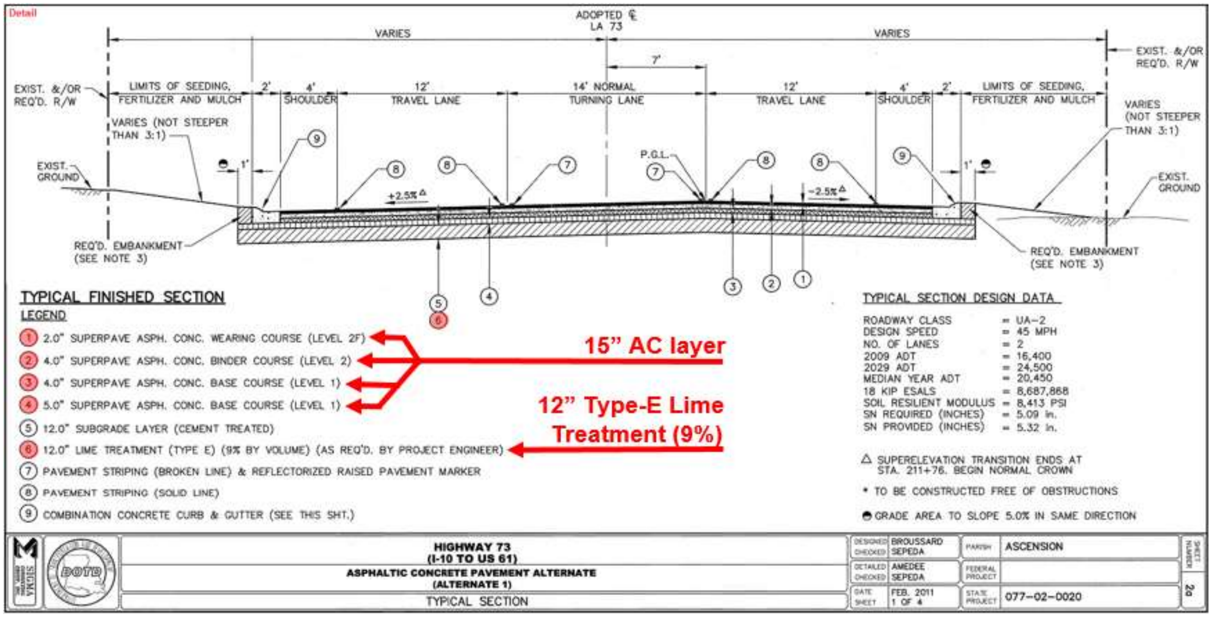


Figure D4

LA 73 cross-section

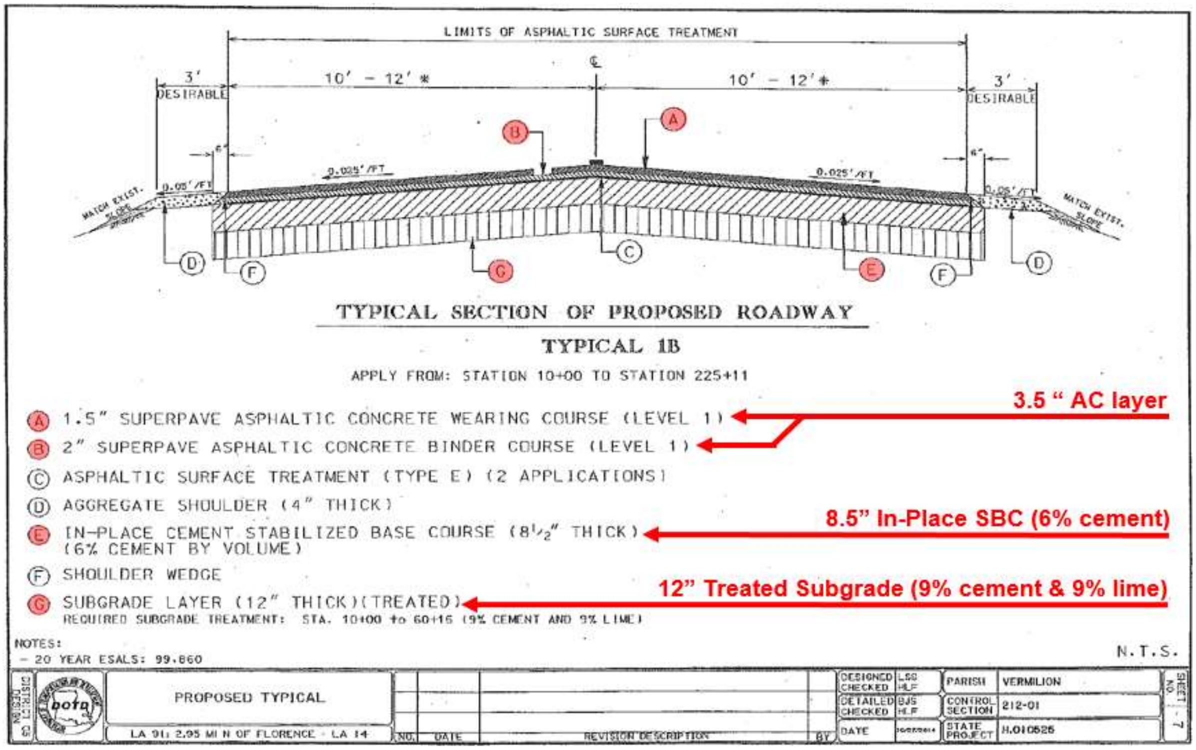


Figure D5

LA 91 cross-section

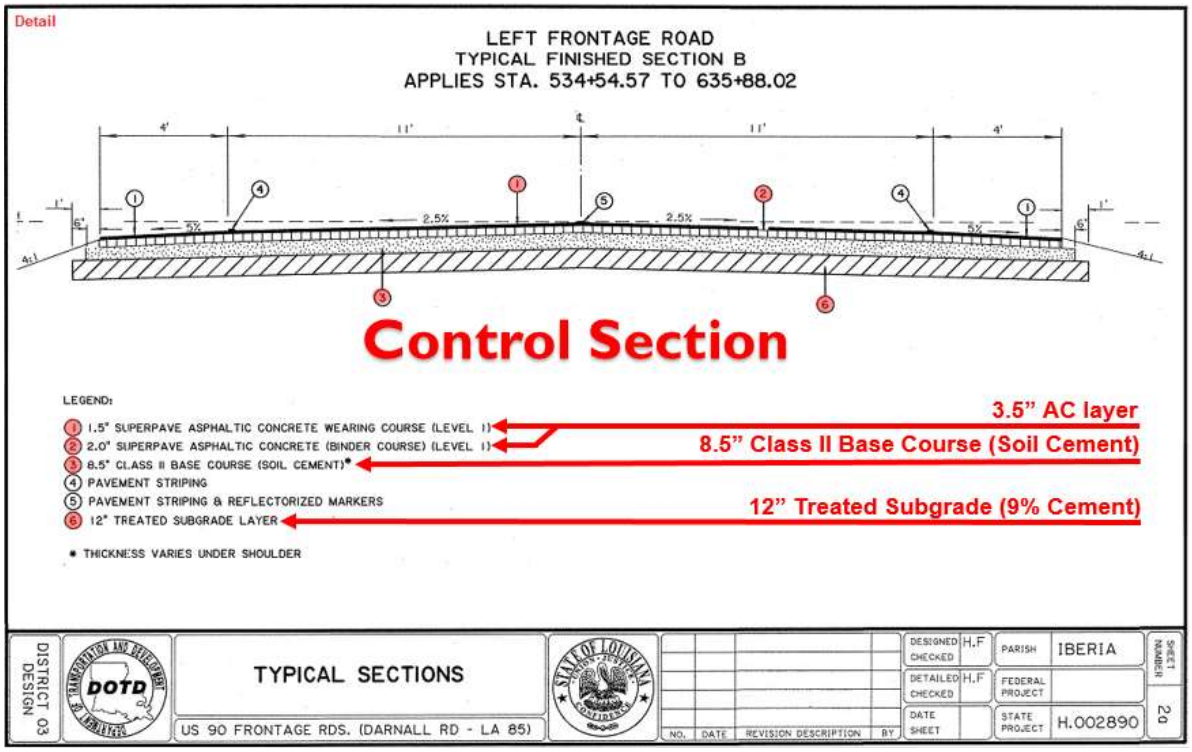
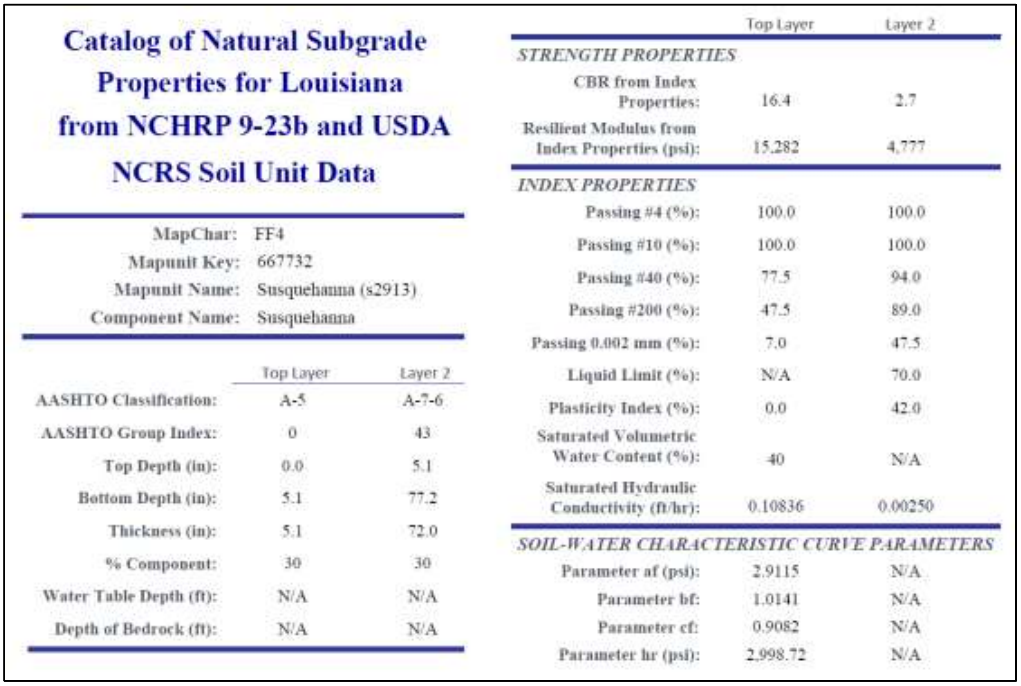


Figure D6

US 90 cross-section

APPENDIX E

Project Subgrade Properties



Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

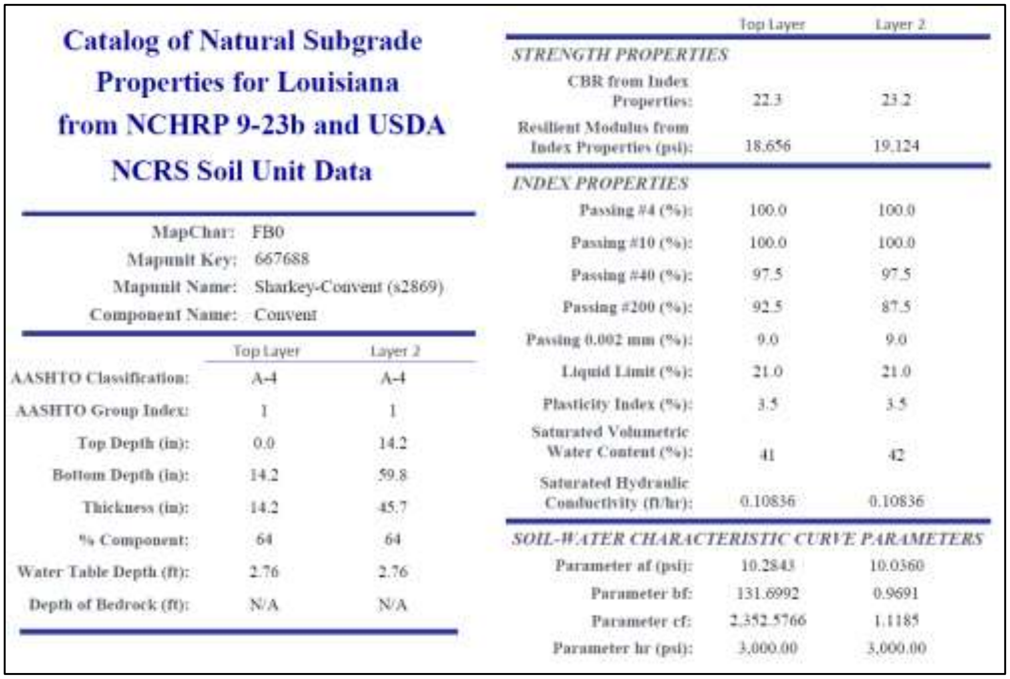
MapChar: FF4
Mapunit Key: 667732
Mapunit Name: Susquehanna (s2913)
Component Name: Susquehanna

	Top Layer	Layer 2
AASHTO Classification:	A-5	A-7-6
AASHTO Group Index:	0	43
Top Depth (in):	0.0	5.1
Bottom Depth (in):	5.1	77.2
Thickness (in):	5.1	72.0
% Component:	30	30
Water Table Depth (ft):	N/A	N/A
Depth of Bedrock (ft):	N/A	N/A

	Top Layer	Layer 2
STRENGTH PROPERTIES		
CBR from Index Properties:	16.4	2.7
Resilient Modulus from Index Properties (psi):	15,282	4,777
INDEX PROPERTIES		
Passing #4 (%):	100.0	100.0
Passing #10 (%):	100.0	100.0
Passing #40 (%):	77.5	94.0
Passing #200 (%):	47.5	89.0
Passing 0.002 mm (%):	7.0	47.5
Liquid Limit (%):	N/A	70.0
Plasticity Index (%):	0.0	42.0
Saturated Volumetric Water Content (%):	40	N/A
Saturated Hydraulic Conductivity (ft/hr):	0.10836	0.00250
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS		
Parameter af (psi):	2.9115	N/A
Parameter bf:	1.0141	N/A
Parameter cf:	0.9082	N/A
Parameter hr (psi):	2,998.72	N/A

Figure E1

US 171 subgrade properties



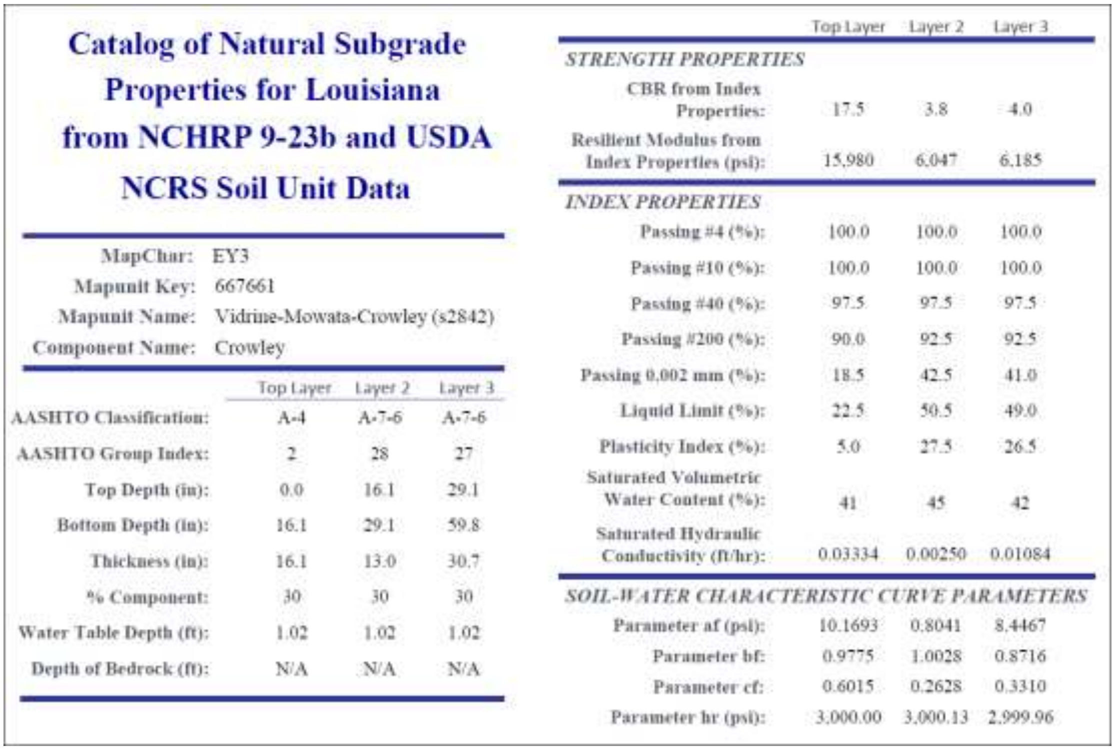
Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

MapChar: FB0
Mapunit Key: 667688
Mapunit Name: Sharkey-Convent (s2869)
Component Name: Convent

	Top Layer	Layer 2
AASHTO Classification:	A-4	A-4
AASHTO Group Index:	1	1
Top Depth (in):	0.0	14.2
Bottom Depth (in):	14.2	59.8
Thickness (in):	14.2	45.7
% Component:	64	64
Water Table Depth (ft):	2.76	2.76
Depth of Bedrock (ft):	N/A	N/A

	Top Layer	Layer 2
STRENGTH PROPERTIES		
CBR from Index Properties:	22.3	23.2
Resilient Modulus from Index Properties (psi):	18,656	19,124
INDEX PROPERTIES		
Passing #4 (%):	100.0	100.0
Passing #10 (%):	100.0	100.0
Passing #40 (%):	97.5	97.5
Passing #200 (%):	92.5	87.5
Passing 0.002 mm (%):	9.0	9.0
Liquid Limit (%):	21.0	21.0
Plasticity Index (%):	3.5	3.5
Saturated Volumetric Water Content (%):	41	42
Saturated Hydraulic Conductivity (ft/hr):	0.10836	0.10836
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS		
Parameter af (psi):	10.2843	10.0360
Parameter bf:	131.6992	0.9691
Parameter cf:	2,352.5766	1.1185
Parameter hr (psi):	3,000.00	3,000.00

Figure E2
LA 3177 subgrade properties



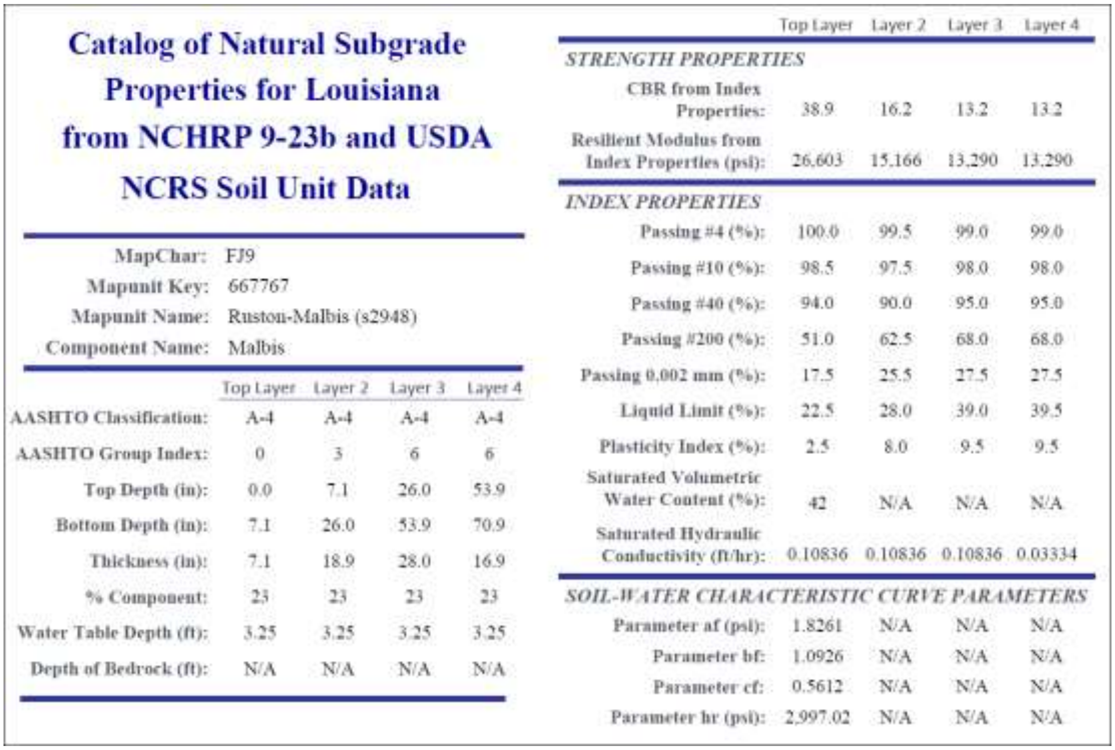
Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

MapChar: EY3
Mapunit Key: 667661
Mapunit Name: Vidrine-Mowata-Crowley (s2842)
Component Name: Crowley

	Top Layer	Layer 2	Layer 3
AASHTO Classification:	A-4	A-7-6	A-7-6
AASHTO Group Index:	2	28	27
Top Depth (in):	0.0	16.1	29.1
Bottom Depth (in):	16.1	29.1	59.8
Thickness (in):	16.1	13.0	30.7
% Component:	30	30	30
Water Table Depth (ft):	1.02	1.02	1.02
Depth of Bedrock (ft):	N/A	N/A	N/A

	Top Layer	Layer 2	Layer 3
STRENGTH PROPERTIES			
CBR from Index Properties:	17.5	3.8	4.0
Resilient Modulus from Index Properties (psi):	15,980	6,047	6,185
INDEX PROPERTIES			
Passing #4 (%):	100.0	100.0	100.0
Passing #10 (%):	100.0	100.0	100.0
Passing #40 (%):	97.5	97.5	97.5
Passing #200 (%):	90.0	92.5	92.5
Passing 0.002 mm (%):	18.5	42.5	41.0
Liquid Limit (%):	22.5	50.5	49.0
Plasticity Index (%):	5.0	27.5	26.5
Saturated Volumetric Water Content (%):	41	45	42
Saturated Hydraulic Conductivity (ft/hr):	0.03334	0.00250	0.01084
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS			
Parameter af (psi):	10.1693	0.8041	8.4467
Parameter bf:	0.9775	1.0028	0.8716
Parameter cf:	0.6015	0.2628	0.3310
Parameter hr (psi):	3,000.00	3,000.13	2,999.96

Figure E3
LA 97 subgrade properties



Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

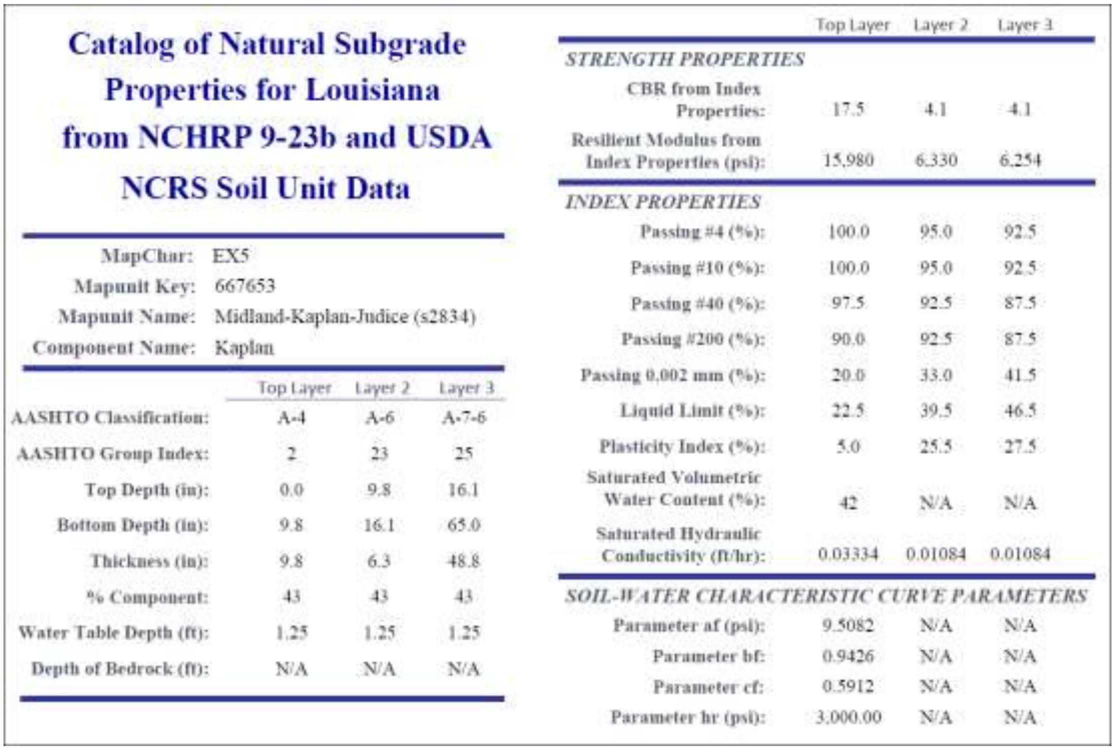
MapChar:	FJ9
Mapunit Key:	667767
Mapunit Name:	Ruston-Malbis (s2948)
Component Name:	Malbis

	Top Layer	Layer 2	Layer 3	Layer 4
AASHTO Classification:	A-4	A-4	A-4	A-4
AASHTO Group Index:	0	3	6	6
Top Depth (in):	0.0	7.1	26.0	53.9
Bottom Depth (in):	7.1	26.0	53.9	70.9
Thickness (in):	7.1	18.9	28.0	16.9
% Component:	23	23	23	23
Water Table Depth (ft):	3.25	3.25	3.25	3.25
Depth of Bedrock (ft):	N/A	N/A	N/A	N/A

	Top Layer	Layer 2	Layer 3	Layer 4
STRENGTH PROPERTIES				
CBR from Index Properties:	38.9	16.2	13.2	13.2
Resilient Modulus from Index Properties (psi):	26,603	15,166	13,290	13,290
INDEX PROPERTIES				
Passing #4 (%):	100.0	99.5	99.0	99.0
Passing #10 (%):	98.5	97.5	98.0	98.0
Passing #40 (%):	94.0	90.0	95.0	95.0
Passing #200 (%):	51.0	62.5	68.0	68.0
Passing 0.002 mm (%):	17.5	25.5	27.5	27.5
Liquid Limit (%):	22.5	28.0	39.0	39.5
Plasticity Index (%):	2.5	8.0	9.5	9.5
Saturated Volumetric Water Content (%):	42	N/A	N/A	N/A
Saturated Hydraulic Conductivity (ft/hr):	0.10836	0.10836	0.10836	0.03334
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS				
Parameter af (psi):	1.8261	N/A	N/A	N/A
Parameter bf:	1.0926	N/A	N/A	N/A
Parameter cf:	0.5612	N/A	N/A	N/A
Parameter hr (psi):	2,997.02	N/A	N/A	N/A

Figure E4

LA 73 subgrade properties



Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

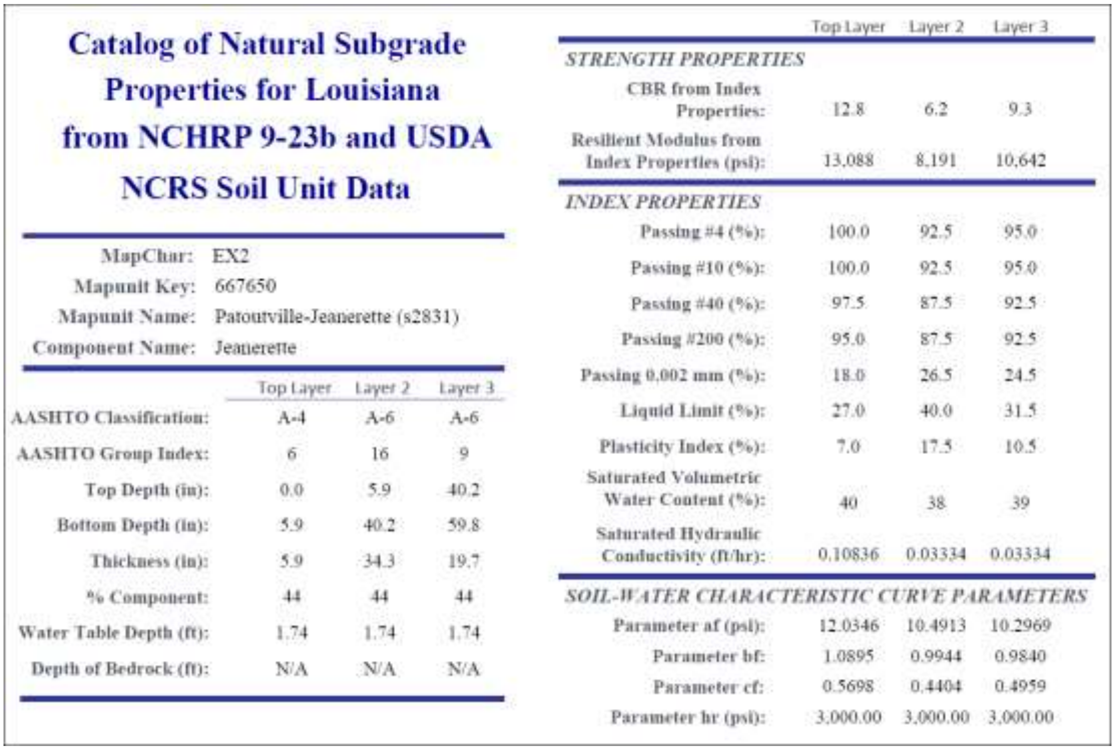
MapChar:	EX5
Mapunit Key:	667653
Mapunit Name:	Midland-Kaplan-Judice (s2834)
Component Name:	Kaplan

	Top Layer	Layer 2	Layer 3
AASHTO Classification:	A-4	A-6	A-7-6
AASHTO Group Index:	2	23	25
Top Depth (in):	0.0	9.8	16.1
Bottom Depth (in):	9.8	16.1	65.0
Thickness (in):	9.8	6.3	48.8
% Component:	43	43	43
Water Table Depth (ft):	1.25	1.25	1.25
Depth of Bedrock (ft):	N/A	N/A	N/A

	Top Layer	Layer 2	Layer 3
STRENGTH PROPERTIES			
CBR from Index Properties:	17.5	4.1	4.1
Resilient Modulus from Index Properties (psi):	15,980	6,330	6,254
INDEX PROPERTIES			
Passing #4 (%):	100.0	95.0	92.5
Passing #10 (%):	100.0	95.0	92.5
Passing #40 (%):	97.5	92.5	87.5
Passing #200 (%):	90.0	92.5	87.5
Passing 0.002 mm (%):	20.0	33.0	41.5
Liquid Limit (%):	22.5	39.5	46.5
Plasticity Index (%):	5.0	25.5	27.5
Saturated Volumetric Water Content (%):	42	N/A	N/A
Saturated Hydraulic Conductivity (ft/hr):	0.03334	0.01084	0.01084
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS			
Parameter af (psi):	9.5082	N/A	N/A
Parameter bf:	0.9426	N/A	N/A
Parameter cf:	0.5912	N/A	N/A
Parameter hr (psi):	3,000.00	N/A	N/A

Figure E5

LA 91 subgrade properties



Catalog of Natural Subgrade Properties for Louisiana from NCHRP 9-23b and USDA NCRS Soil Unit Data

MapChar:	EX2
Mapunit Key:	667650
Mapunit Name:	Patoutville-Jeanerette (s2831)
Component Name:	Jeanerette

	Top Layer	Layer 2	Layer 3
AASHTO Classification:	A-4	A-6	A-6
AASHTO Group Index:	6	16	9
Top Depth (in):	0.0	5.9	40.2
Bottom Depth (in):	5.9	40.2	59.8
Thickness (in):	5.9	34.3	19.7
% Component:	44	44	44
Water Table Depth (ft):	1.74	1.74	1.74
Depth of Bedrock (ft):	N/A	N/A	N/A

	Top Layer	Layer 2	Layer 3
STRENGTH PROPERTIES			
CBR from Index Properties:	12.8	6.2	9.3
Resilient Modulus from Index Properties (psi):	13,088	8,191	10,642
INDEX PROPERTIES			
Passing #4 (%):	100.0	92.5	95.0
Passing #10 (%):	100.0	92.5	95.0
Passing #40 (%):	97.5	87.5	92.5
Passing #200 (%):	95.0	87.5	92.5
Passing 0.002 mm (%):	18.0	26.5	24.5
Liquid Limit (%):	27.0	40.0	31.5
Plasticity Index (%):	7.0	17.5	10.5
Saturated Volumetric Water Content (%):	40	38	39
Saturated Hydraulic Conductivity (ft/hr):	0.10836	0.03334	0.03334
SOIL-WATER CHARACTERISTIC CURVE PARAMETERS			
Parameter af (psi):	12.0346	10.4913	10.2969
Parameter bf:	1.0895	0.9944	0.9840
Parameter cf:	0.5698	0.4404	0.4959
Parameter hr (psi):	3,000.00	3,000.00	3,000.00

Figure E6

US 90 subgrade properties

APPENDIX F

Project DCP Results

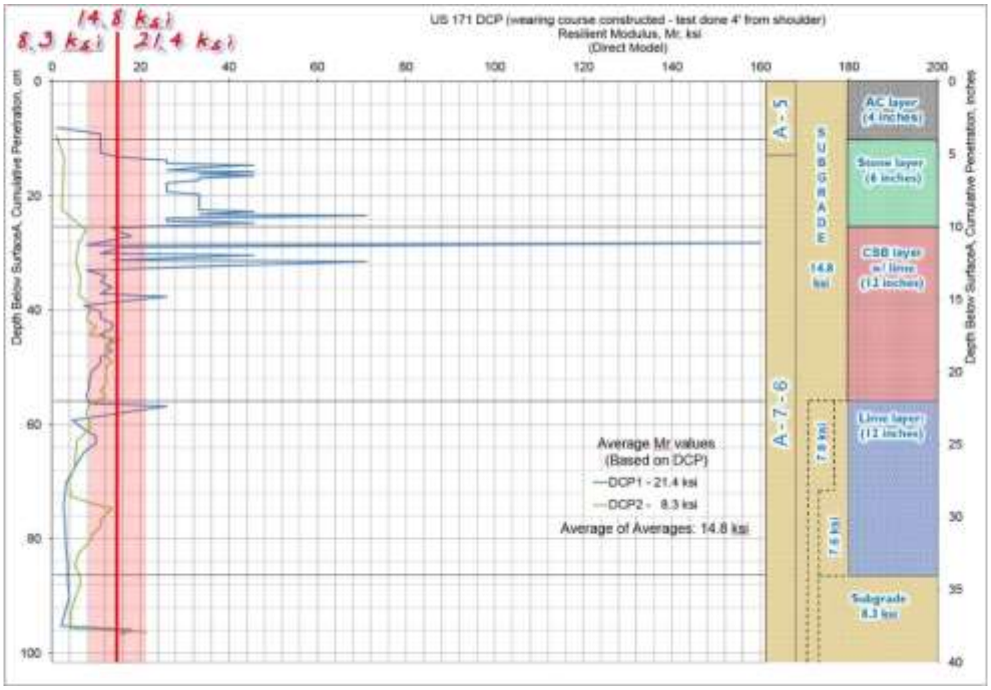


Figure F1

US 171 DCP results

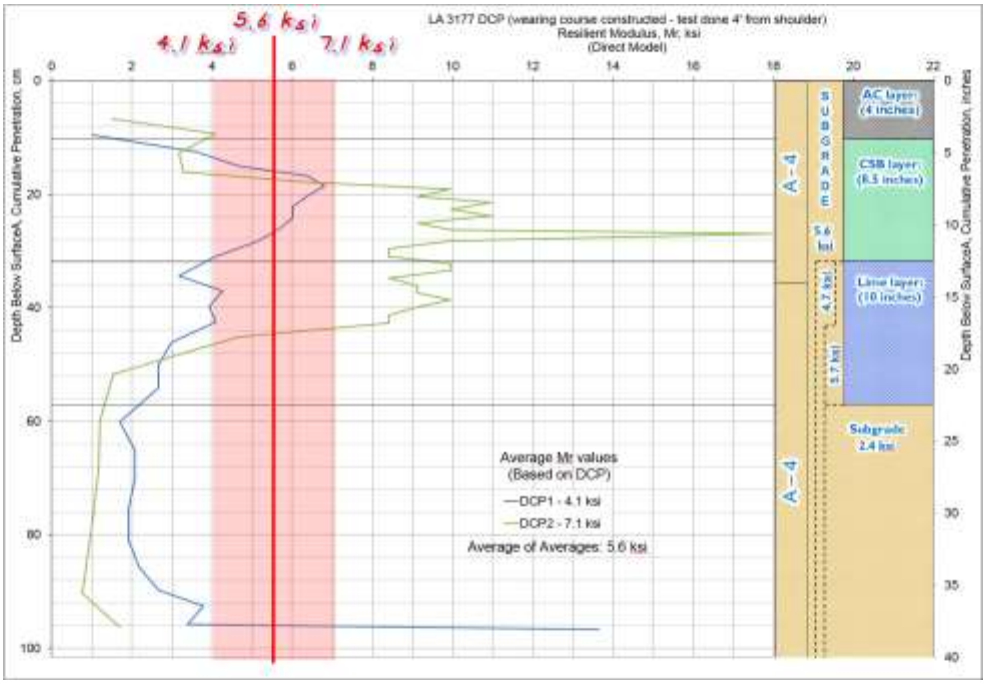


Figure F2

LA 3177 DCP results

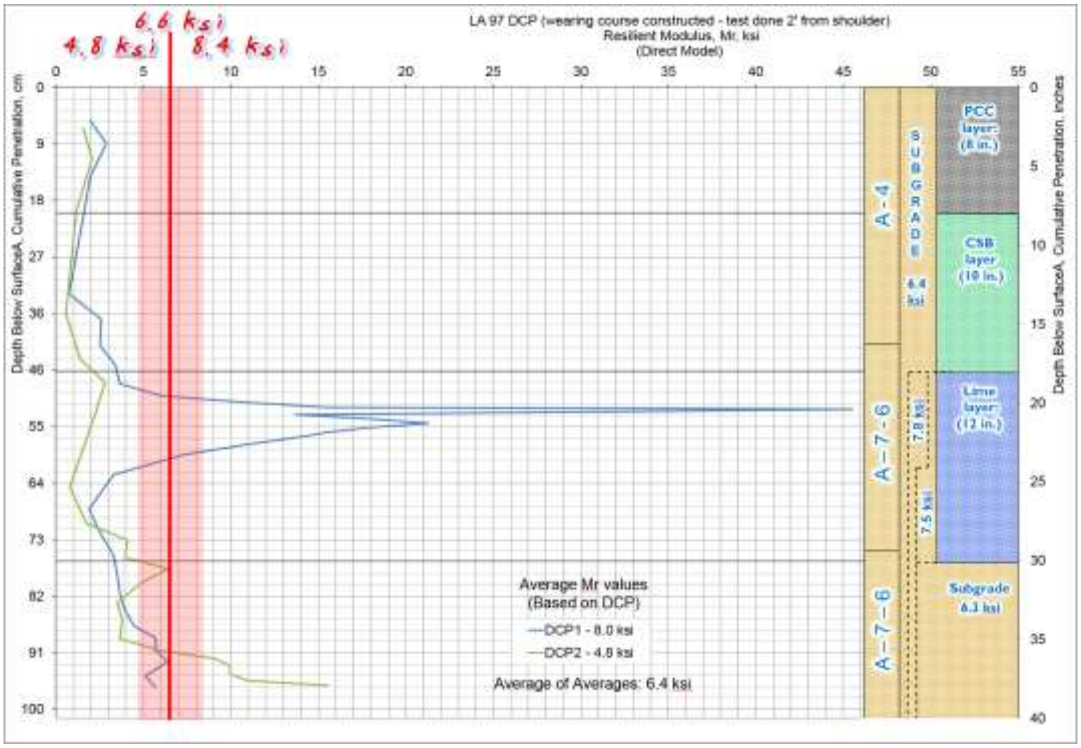


Figure F3
LA 97 DCP results

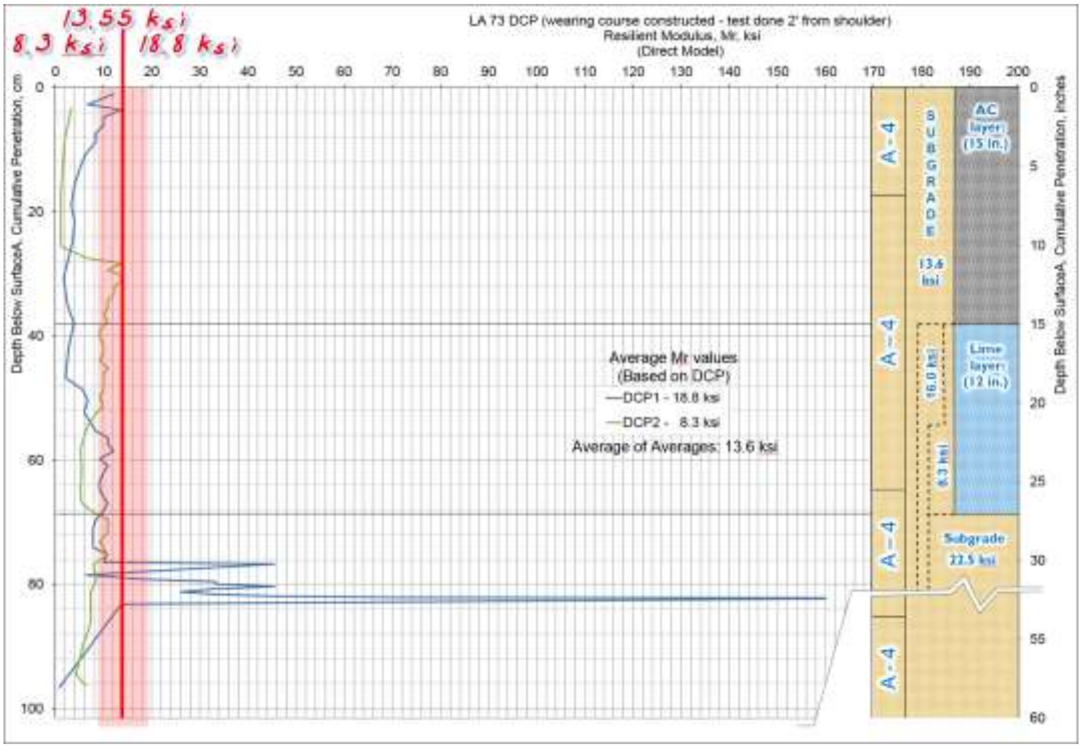


Figure F4
LA 73 DCP results

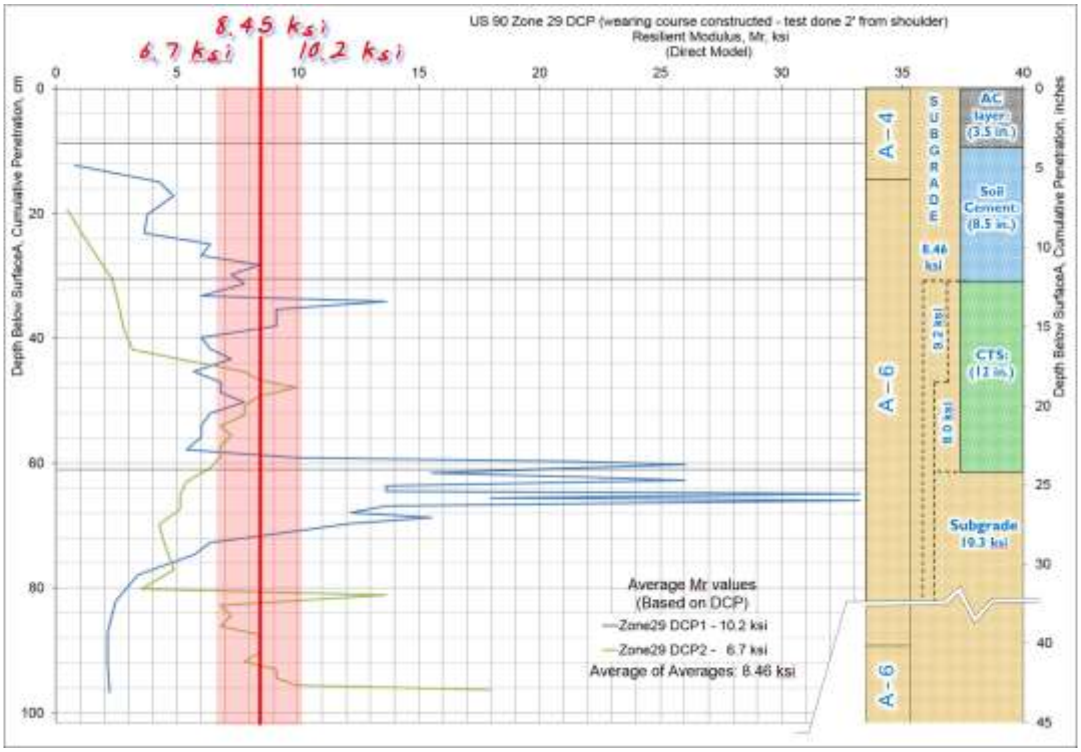
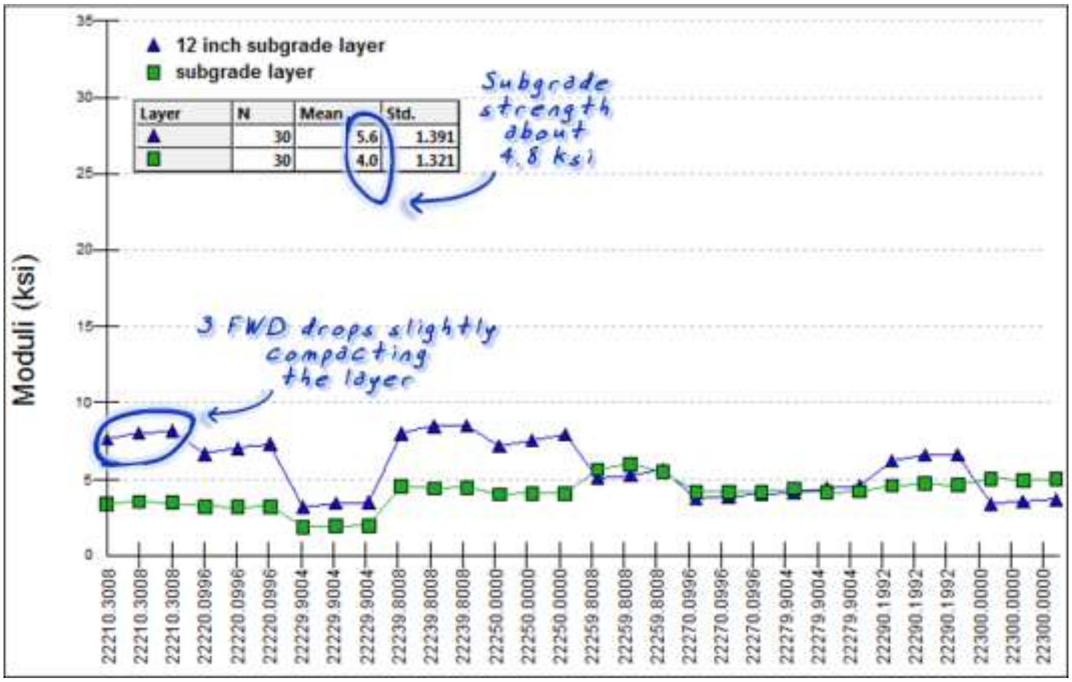


Figure F5

US 90 DCP results

APPENDIX G

Project FWD Results

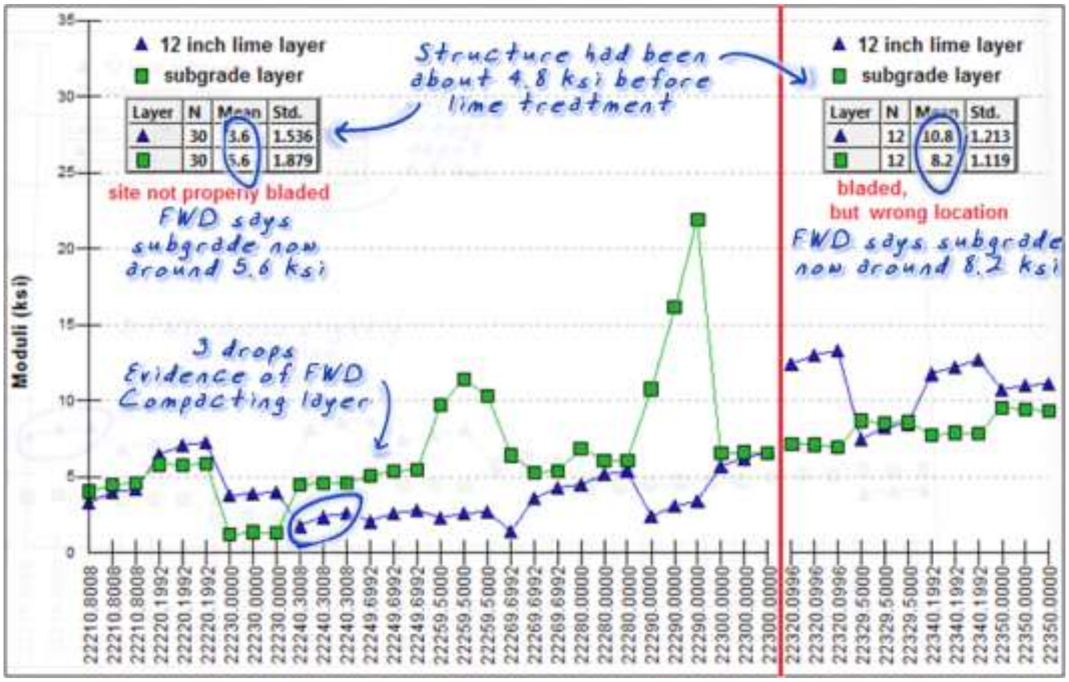


Layer	N	Mean	Std.
▲	30	5.6	1.391
🟩	30	4.0	1.321

Subgrade strength about 4.8 ksi

3 FWD drops slightly compacting the layer

Figure G1
US-171 FWD results on raw subgrade



Layer	N	Mean	Std.
▲	30	3.6	1.536
🟩	30	5.6	1.879

Layer	N	Mean	Std.
▲	12	10.8	1.213
🟩	12	8.2	1.119

Structure had been about 4.8 ksi before lime treatment

site not properly bladed

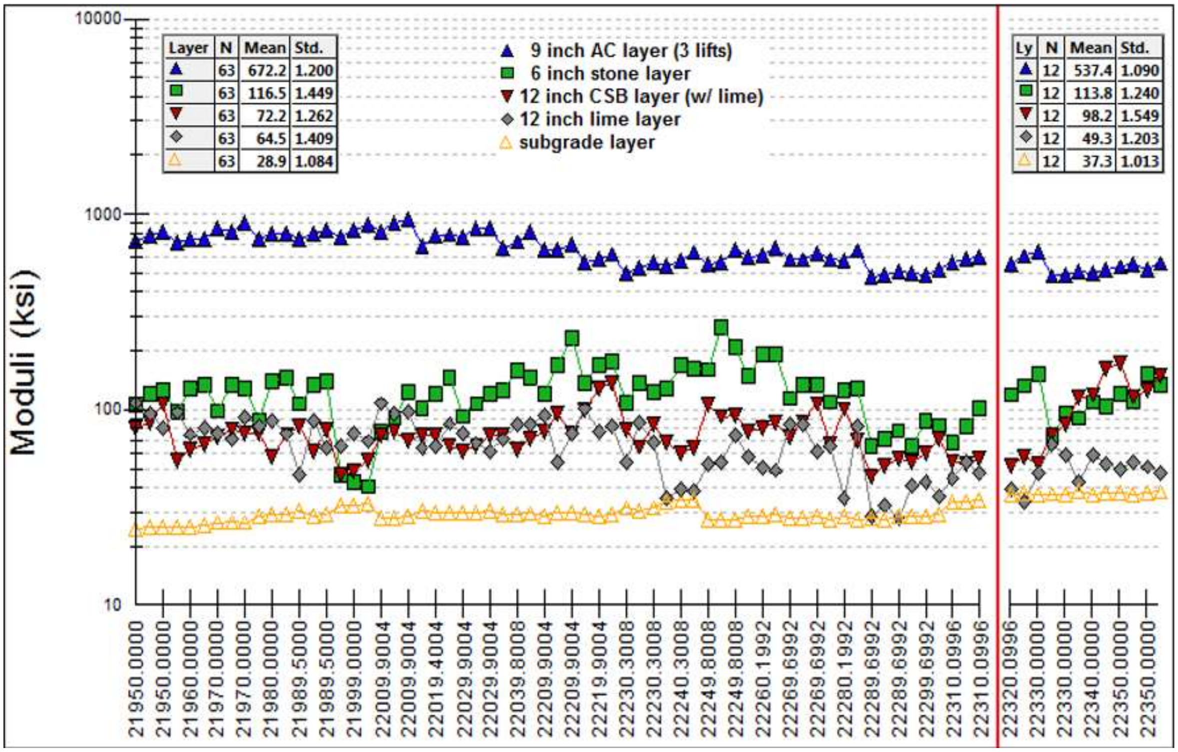
FWD says subgrade now around 5.6 ksi

3 drops Evidence of FWD Compacting layer

bladed, but wrong location

FWD says subgrade now around 8.2 ksi

Figure G2
US-171 FWD results on lime layer

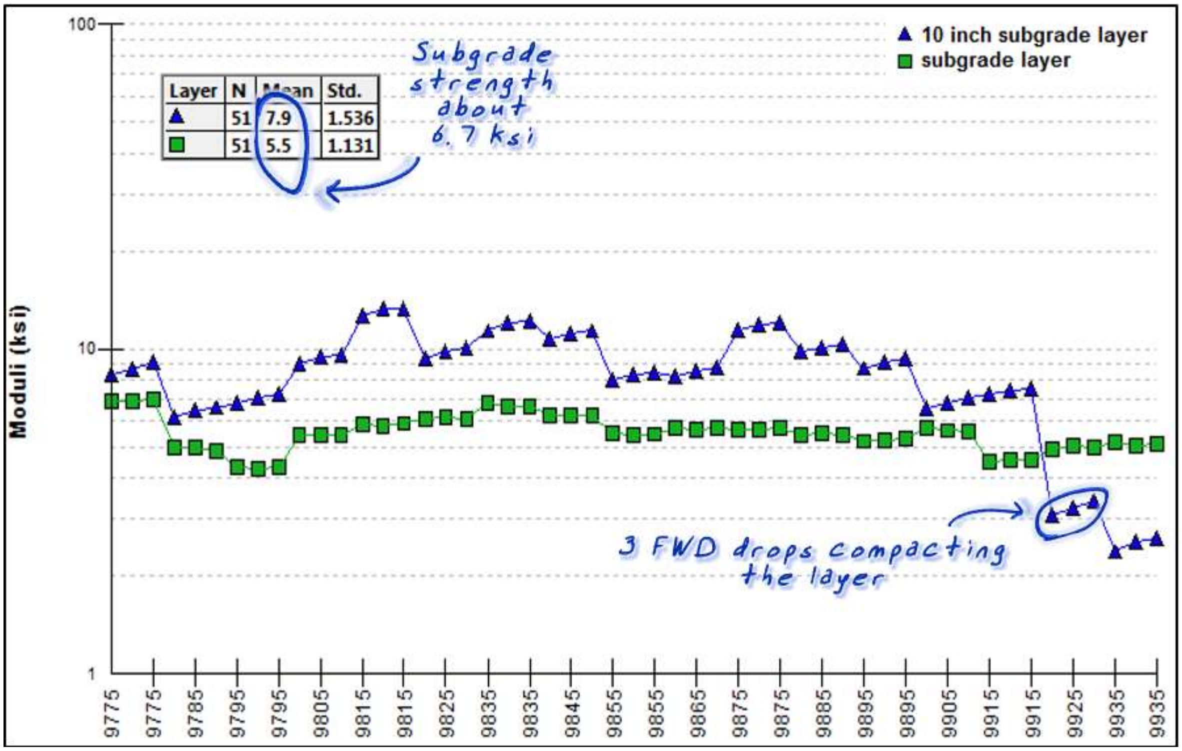


Layer	N	Mean	Std.
▲	63	672.2	1.200
■	63	116.5	1.449
▼	63	72.2	1.262
◆	63	64.5	1.409
△	63	28.9	1.084

Ly	N	Mean	Std.
▲	12	537.4	1.090
■	12	113.8	1.240
▼	12	98.2	1.549
◆	12	49.3	1.203
△	12	37.3	1.013

Figure G3

US-171 FWD results on AC wearing course



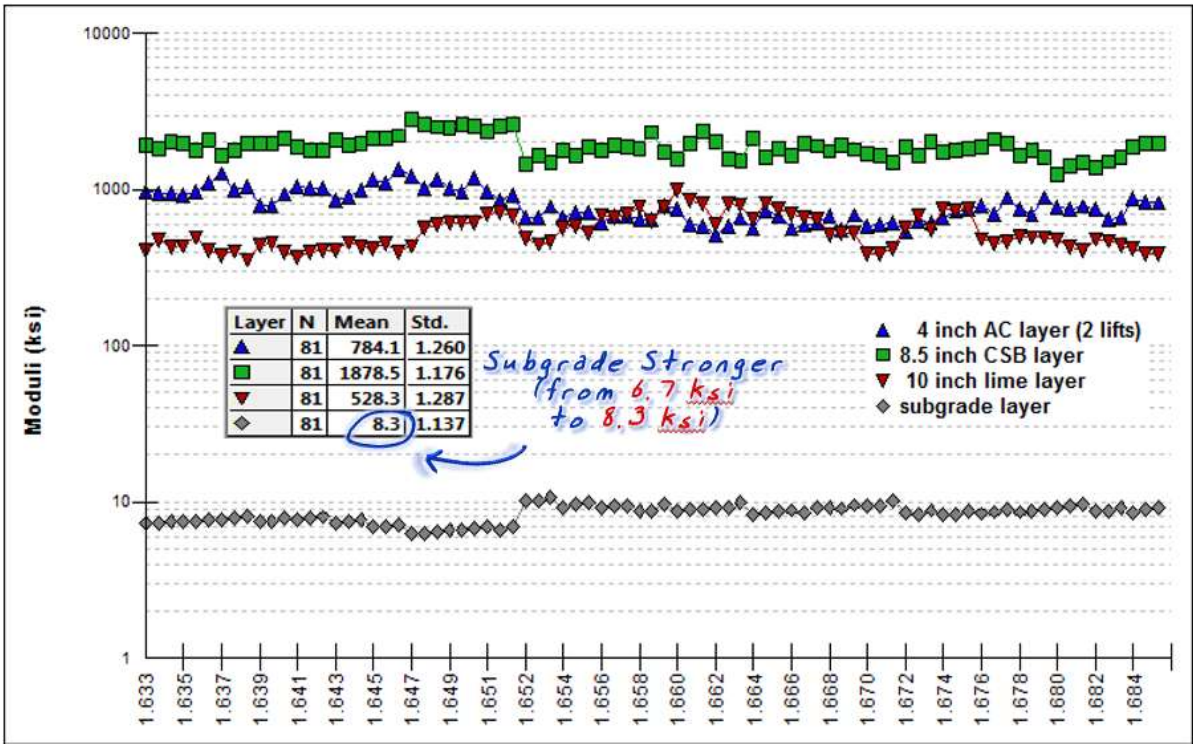
Layer	N	Mean	Std.
▲	51	7.9	1.536
■	51	5.5	1.131

Subgrade strength about 6.7 ksi

3 FWD drops compacting the layer

Figure G4

LA-3177 FWD results on raw subgrade

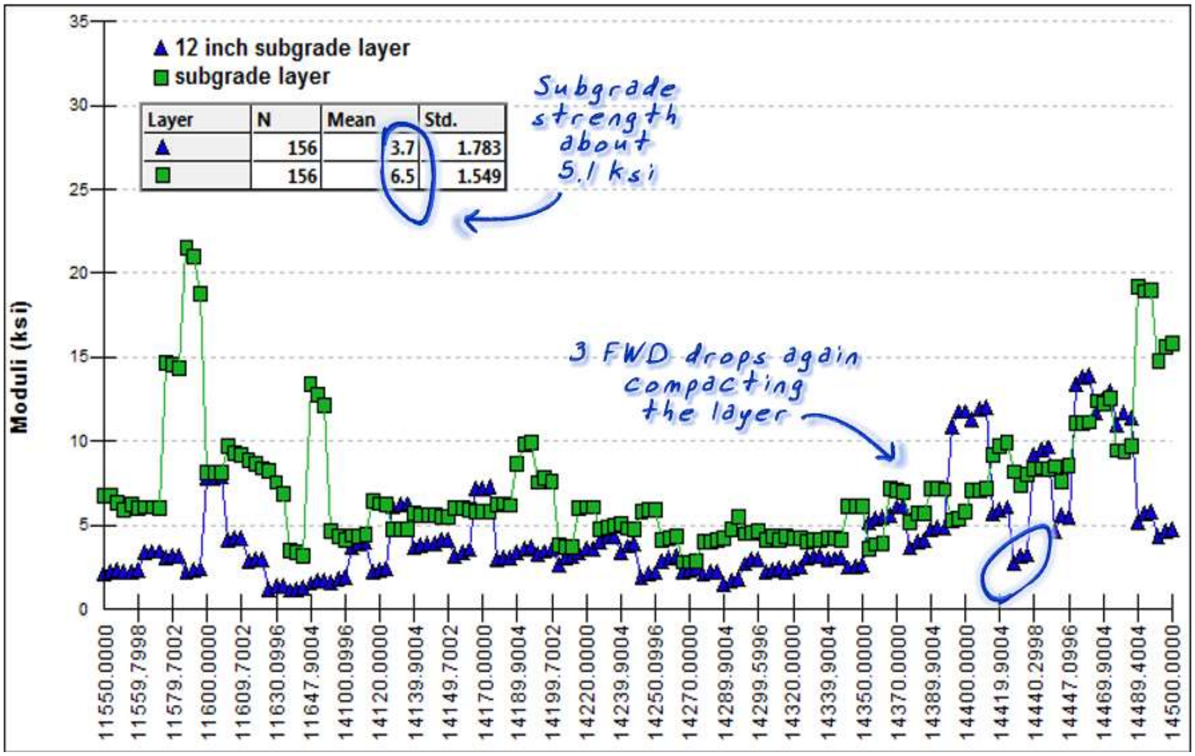


Layer	N	Mean	Std.
▲	81	784.1	1.260
🟩	81	1878.5	1.176
▼	81	528.3	1.287
◆	81	8.3	1.137

Subgrade Stronger (from 6.7 ksi to 8.3 ksi)

Figure G5

LA-3177 FWD results on AC wearing course



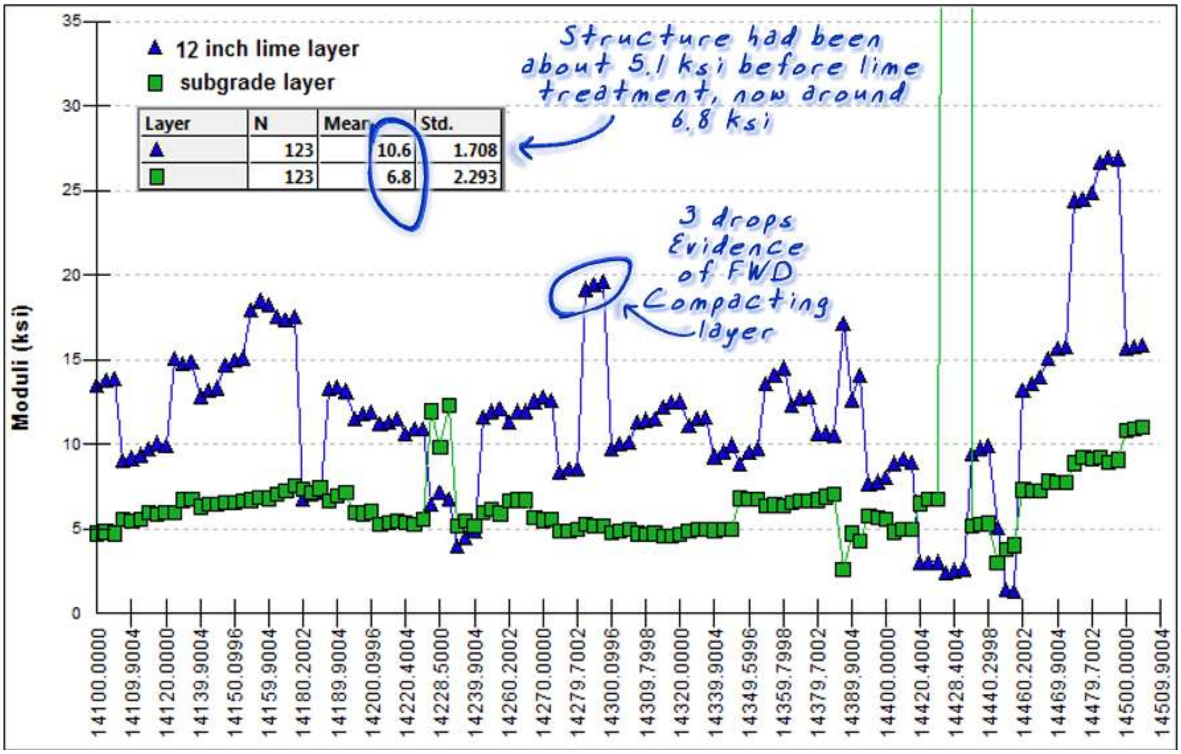
Layer	N	Mean	Std.
▲	156	3.7	1.783
🟩	156	6.5	1.549

Subgrade strength about 5.1 ksi

3 FWD drops again compacting the layer

Figure G6

LA-97 FWD results on raw subgrade



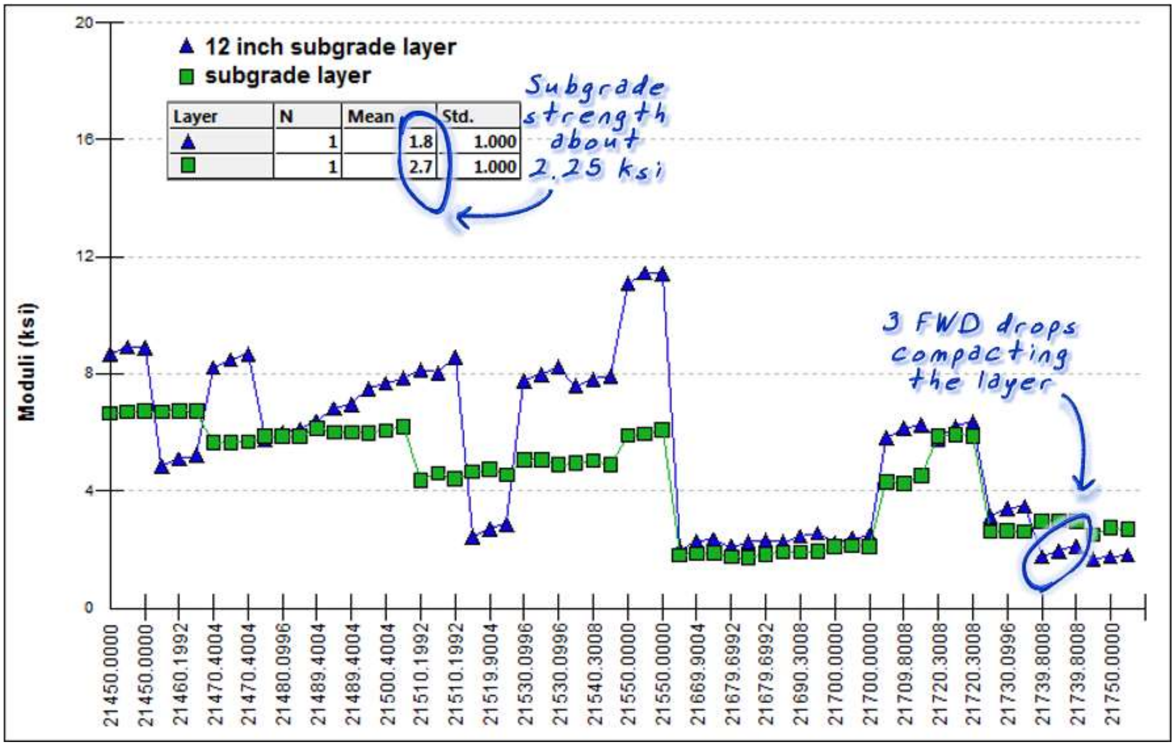
Layer	N	Mean	Std.
▲	123	10.6	1.708
🟩	123	6.8	2.293

Structure had been about 5.1 ksi before lime treatment, now around 6.8 ksi

3 drops Evidence of FWD Compacting layer

Figure G7

LA-97 FWD results on lime layer



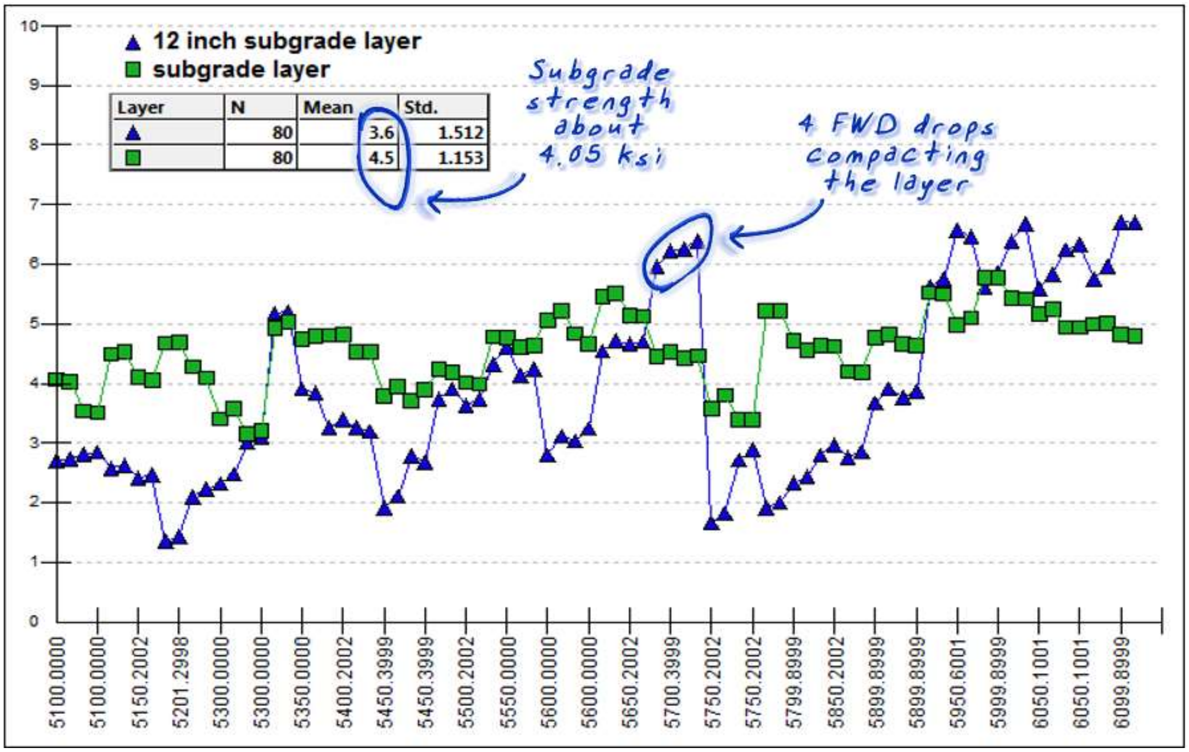
Layer	N	Mean	Std.
▲	1	1.8	1.000
🟩	1	2.7	1.000

Subgrade strength about 2.25 ksi

3 FWD drops compacting the layer

Figure G8

LA-73 FWD results on raw subgrade



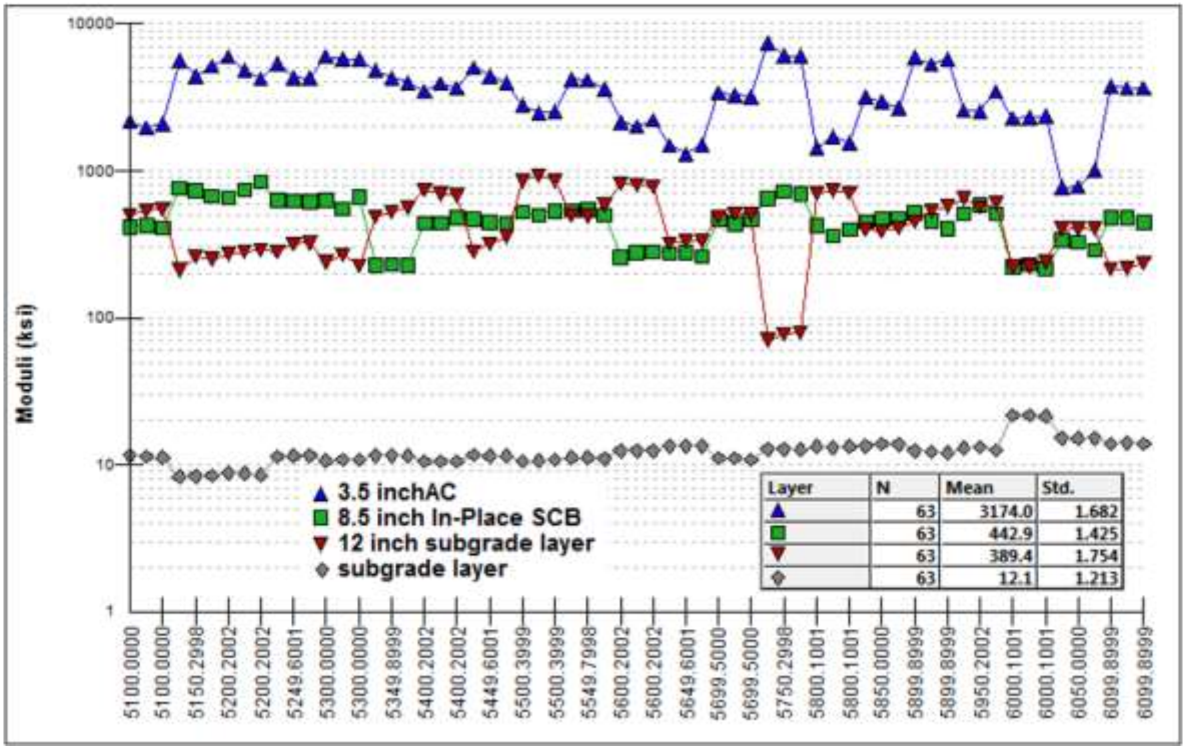
Layer	N	Mean	Std.
▲	80	3.6	1.512
■	80	4.5	1.153

Subgrade strength about 4.05 ksi

4 FWD drops compacting the layer

Figure G9

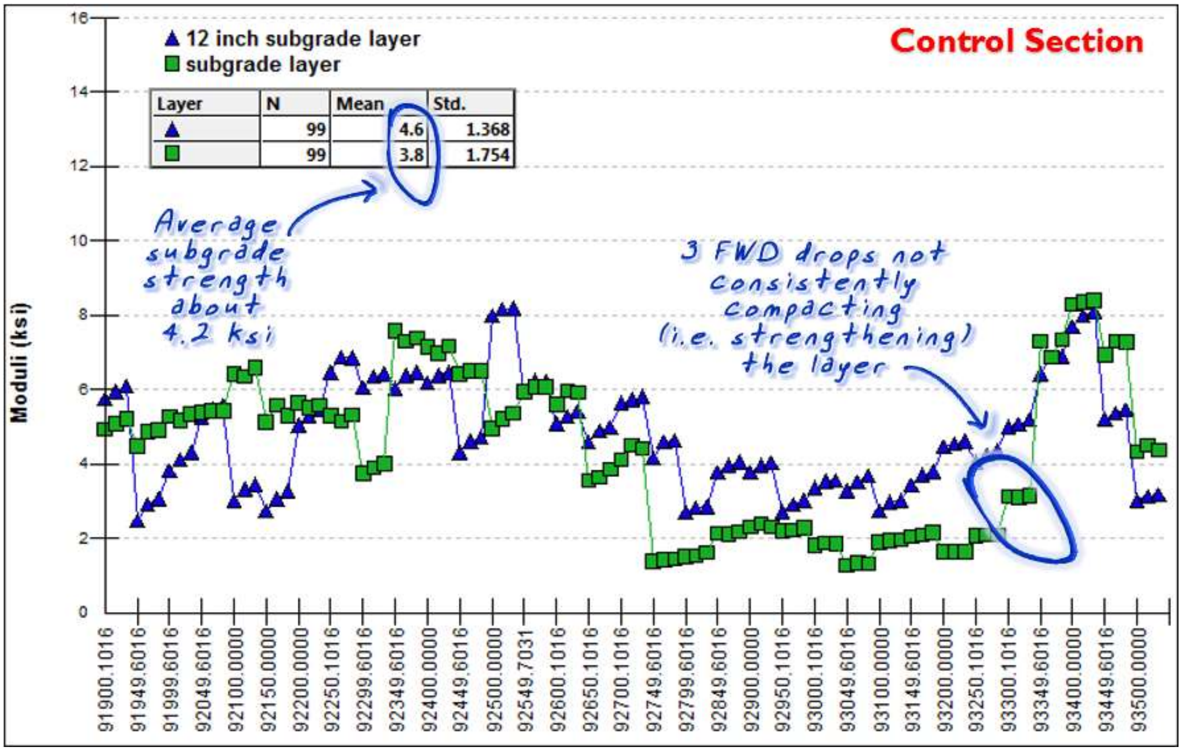
LA-91 FWD results on raw subgrade



Layer	N	Mean	Std.
▲	63	3174.0	1.682
■	63	442.9	1.425
▼	63	389.4	1.754
◆	63	12.1	1.213

Figure G10

LA-91 FWD results on AC wearing course

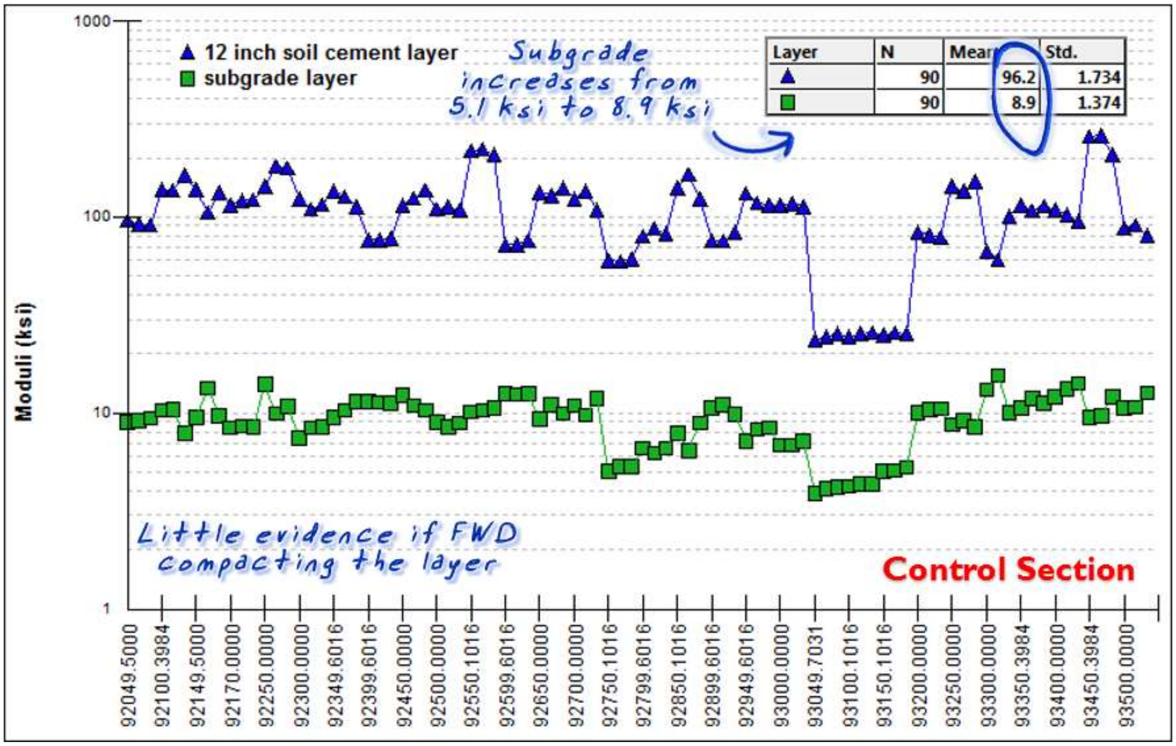


Average subgrade strength about 4.2 ksi

3 FWD drops not consistently compacting (i.e. strengthening) the layer

Figure G11

US-90 FWD results on raw subgrade

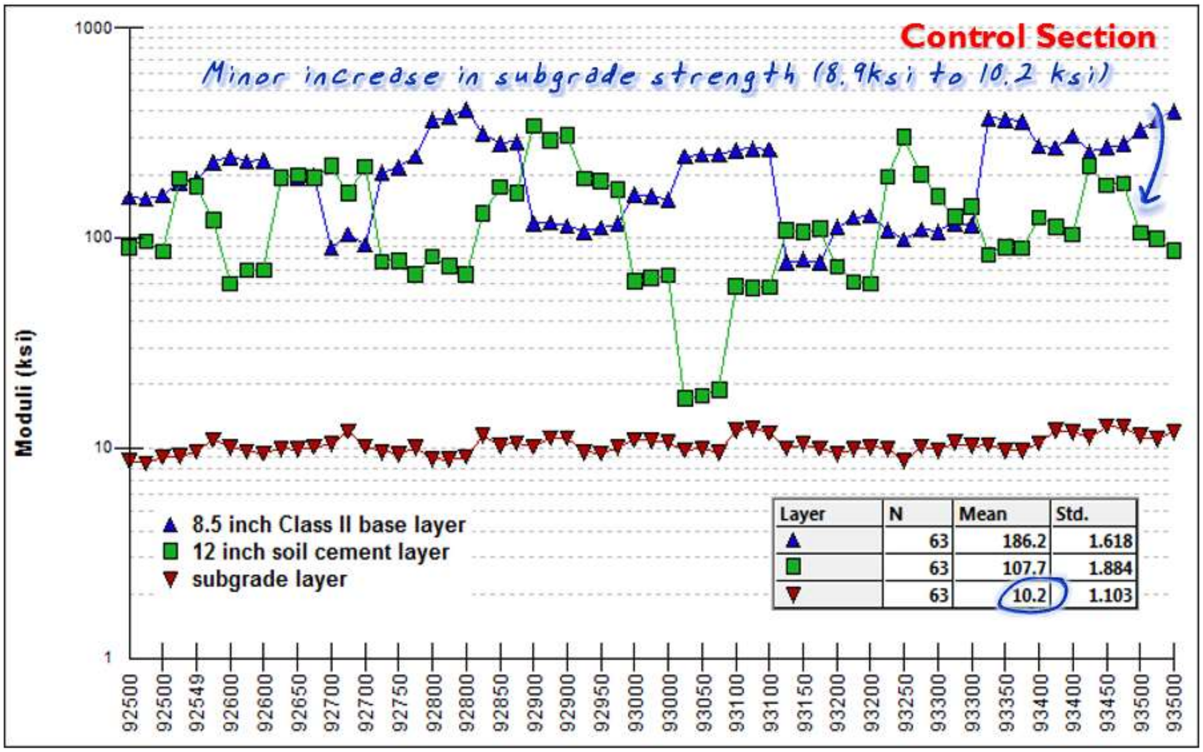


Subgrade increases from 5.1 ksi to 8.9 ksi

Little evidence if FWD compacting the layer

Figure G12

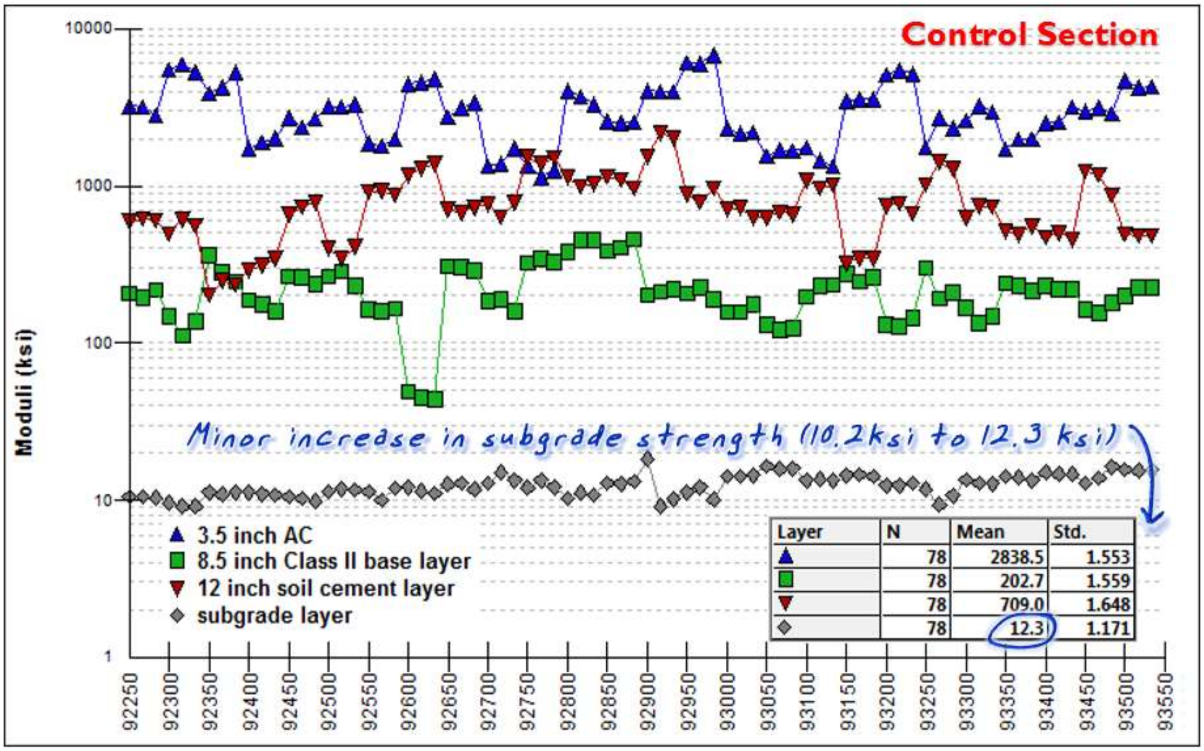
US-90 FWD results on soil cement layer



Layer	N	Mean	Std.
▲	63	186.2	1.618
🟩	63	107.7	1.884
▼	63	10.2	1.103

Figure G13

US-90 FWD results on Class II base layer



Layer	N	Mean	Std.
▲	78	2838.5	1.553
🟩	78	202.7	1.559
▼	78	709.0	1.648
◆	78	12.3	1.171

Figure G14

US-90 FWD results on AC wearing course

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