Louisiana Transportation Research Center

Technical Assistance Report 17-01TA-C

Cracked Pavement Investigation

by

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July 31, 2017

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Tyson D. Rupnow, Ph.D., P.E.
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Re: H.003496
I-49 North: Segment J
Caddo Parish
DRAFT Cracked Pavement Investigation

Ms. Williams:

The LADOTD Pavement & Geotechnical Services section and LTRC have performed field and laboratory testing as well as engineering analyses to investigate the cause of the cracked pavement on the above captioned project.

From a geotechnical standpoint, three potential contributions to the pavement failure were analyzed: slope stability, differential settlement of the natural soils, and differential settlement or movement of the embankment fill.

- Slope stability analyses yielded high factors of safety, suggesting that even the worst-case analyses that were considered would not have caused a slope stability failure. A parametric study indicated that for a slope stability failure to have occurred through the pavement, all natural soils at the site would need to have a shear strength lower than any of the shear strengths observed in the borings or CPT soundings. Therefore, slope stability failure was ruled out as a contributing factor;
- A settlement analysis was performed to estimate the consolidation of existing soils due to the weight of new embankment fill. A magnitude of 2 to 3 inches of settlement was estimated, with the majority of the settlement occurring during fill placement. Within about a month of completing the fill placement, the amount of remaining settlement would have been negligible. Therefore, settlement due to the weight of the fill was ruled out as a contributing factor;
- The submitted field testing results from District 04 were reviewed to determine whether the placement, compaction, or composition of the embankment fill were adequate. Based on the Standard Specifications, the classification and compaction of the fill meet the relevant specifications. However, from a purely geotechnical standpoint, some of the accepted fill soils appear to be poor for embankment construction and the variability of compaction within the fill appears to be high. Based on the geotechnical investigation, the construction and/or composition

of the embankment fill appears to be the most likely contribution to the pavement cracking. However, there are inadequacies in the in both the Specifications and the TR Methods that make it impossible to say with certainty whether the fill is adequate.

As part of the pavement and roadway structure investigation, LTRC Concrete Research Laboratory personnel obtained a total of seven cores in the presence of District 04 personnel in February 2017. The cores extracted showed that the longitudinal joint performed as designed, with the formation of full-depth cracks below the saw cut. An examination of the cores over the longitudinal crack showed uniformly distributed concrete across the depth of the pavement, with no observable evidence of vibrator trails. Compressive tests showed sufficient strength concrete and soil cement.

Trenching operations showed consistent concrete across the cross-section from the centerline of roadway through the outside shoulder. A crack was observed in the granular layer directly underneath the pavement crack. In the soil cement layer, a crack was found approximately 12-15 inches towards the centerline of the lane from the existing pavement crack.

Falling Weight Deflectometer results indicated that all layers except the subgrade were in good condition. This supports the geotechnical findings, which indicate that the fill underlying the pavement structure is the most likely contributor to the crack. The results of the Walking Profiler indicated that some movement had occurred in the outside lane. The details of the pavement examination follow the report on the geotechnical investigation.

Attachments: Cracked Pavement Report & Appendices

JGR/TDR

DRAFT CRACKED PAVEMENT INVESTIGATION
PROJECT NO. H.003496
I-49 NORTH: SEGMENT J
CADDO PARISH

July 31, 2017

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Table of Contents

1.0 Project Information

This project consists of an investigation into the cause of the cracking of concrete pavement panels along I-49 North, Segment J in Caddo Parish. An approximate straight-line longitudinal crack spanning about 20 panels was observed within the right lane of the I-49 northbound section between LA-1 and Martin Luther King Drive in Shreveport. The approximate project location is shown on Figure 1. The following report describes the geotechnical and concrete testing and analyses made to evaluate the cracked pavement.

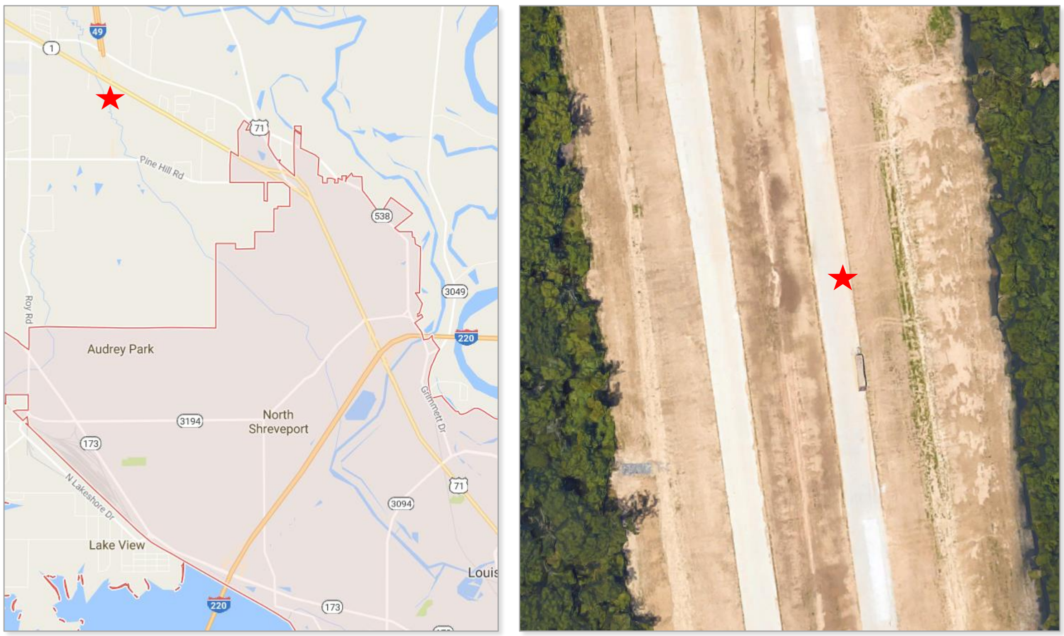


Figure 1: Project Location

2.0 Potential Geotechnical Contributions to Pavement Failure

Prior to performing any fieldwork or analyses, several potential geotechnical-related contributions to the pavement crack were identified. These modes of failure/movement were evaluated by the Pavement & Geotechnical Section, independent of the concrete investigation made by LTRC.

2.1 Slope Stability

One potential mode of failure is a deep-seated slope stability or block failure. This type of failure is characterized by a relatively deep failure surface that can penetrate into the natural soils beneath the embankment. A semicircular failure scarp is usually observable at the top of the slope where the failure has occurred. Toe scarps or bulges are also usually evident where the bottom of the failure surface exits the embankment or natural ground.

A deep-seated global stability failure is generally caused by undrained (short-term) shear failure of the subgrade soils. This happens when driving forces (traffic, surcharges, self-weight of the fill) exceeds the resisting forces (soil shear strength, self-weight of the fill at the toe). A high water table or introduction of runoff water into a tension crack can also exacerbate slope stability problems. A portion of the project along the cracked section included the filling and realignment of McCain Creek. Aerial photography also suggests that McCain Creek or other related channels have at one time passed across the final I-49 alignment within the cracked area. Therefore, it is possible that these abandoned channels are filled with relatively new, soft material that could provide a preferential path for a global stability failure surface.

Another potential mode of stability failure is a shallow slope stability or sloughing failure. These typically appear as shallow semicircular scarps in the side of the embankment. These failures typically do not extend down into the natural material below the fill. A shallow slope stability failure is generally caused by drained (long-term) shear failure of the embankment soils.

In embankments, this is usually caused by a loss of strength of the soil due to cracking that occurs with repeated seasonal wetting and drying cycles. As with a deep-seated stability failure, the introduction of water into the slope can exacerbate these types of failures. Due to the shallow nature of a drained type of failure, the failure surface is not usually expected to extend back through the pavement surface.

2.2 Differential Settlement of Natural Soils

Another possibility is that the weight of the new embankment fill caused the natural subgrade soils to consolidate in an uneven manner. Variability within the natural soils (such as the presence of filled channels, as discussed above) as well as variability of embankment height could lead to differential settlement.

It should be noted that some settlement is expected under all new embankment placed over natural soils. The embankment fill may distribute the differential settlement more evenly, and the rigid pavement structure will resist some degree of differential movement. Therefore, the magnitude of the residual (post-construction) differential settlement would have to be significant to manifest itself as a full-depth crack through the concrete and soil cement base.

2.3 Differential Settlement of the Embankment Fill

Similar to differential settlement of the natural soils, it may be possible that certain portions of the embankment fill have undergone differential movement. In general, consolidation settlement within properly constructed embankment is assumed minimal; however, poor moisture control, low compactive effort, or other construction issues could lead to differential movement within the embankment material.

3.0 FIELD INVESTIGATION

Representatives from the Pavement & Geotechnical section visited the subject site on April 18, 2017. The intent of the site visit was to examine the site as well as establish the scope for a geotechnical investigation to include soil borings and cone penetrometer test (CPT) soundings.

3.1 Site Conditions

At the time of the site visit, the pavement, median, and side slopes of the embankment appeared to be dry. Vegetation was present on the embankment. Rutting, significant signs of erosion, or scarps associated with slope stability failures were not evident. Large sections of concrete had already been removed near Stations 342+78 and 344+47 for concrete testing at LTRC. The full-depth crack in the concrete was evident at these locations, as was the crack at the top of the soil cement base. These observations are discussed in more detail in Section 5.7.

3.2 Geotechnical Investigation

The soil boring and CPT sounding locations and depths were selected to provide specific soil information to evaluate the potential modes of geotechnical related failure. Table 1 and the Boring Plan in Appendix 1 indicate the boring locations. Top of boring elevations were estimated using the cross-section sheets in the plans.

Table 1: Soil Boring & CPT Sounding Locations

Boring	Depth (ft)	Elev. (ft)	Station	Offset	Latitude	Longitude
B-01	47.0	+188.1	340+89	71.00	32.588111	-93.830309
B-02	21.0	+188.2	341+89	69.00	32.588379	-93.830364
B-03	53.0	+188.4	342+78	52.00	32.588609	-93.830454
B-04	53.0	+188.0	342+78	78.00	32.588618	-93.830375
B-05	56.0	+187.0	342+78	93.00	32.588624	-93.830325
B-06	21.0	+188.2	343+78	69.00	32.588883	-93.830446
B-07	53.0	+188.1	344+47	74.00	32.589086	-93.830466
CPT-01	28.3	+187.7	338+31	89.00	32.587423	-93.830145
CPT-02	40.0	+188.5	340+89	50.00	32.588104	-93.830376
CPT-03	38.1	+187.9	340+89	88.00	32.588115	-93.830259
CPT-04	44.6	+188.2	342+78	64.00	32.588614	-93.830420
CPT-05	47.4	+189.0	342+78	-44.00	32.588576	-93.830757
CPT-06	48.2	+188.2	344+47	64.00	32.589084	-93.830498
CPT-07	47.8	+188.2	346+48	86.00	32.589638	-93.830517
CPT-08	46.0	+187.5	344+97	90.00	32.589237	-93.830435

3.3 Groundwater

The borings were advanced with rotary wash drilling, which can obscure the presence of groundwater at the time of drilling. Long-term (14 to 18 hours after drilling) groundwater measurements were taken at two boring locations. Groundwater elevations were estimated at +179.4 ft and +180.6 ft in Borings B-04 and B-07, respectively. This corresponds to a depth of about 8 feet below the top of boring.

3.4 Geotechnical Conditions

The following sections characterize the materials encountered in the soil borings and CPT soundings. A fence diagram showing the general interpreted stratigraphy is included in Appendix 2. Soil boring logs and CPT parameter plots are furnished in Appendices 3 and 4, respectively. Note that the friction angle (ϕ), modulus, and undrained shear strength (S_u) values presented in the CPT plots are estimated based on published correlations. Different correlations with varying assumptions are plotted on the same set of axes.

3.4.1 Pavement

According to the plans, the pavement section at the subject site consists of 11 inches of concrete, followed by 4 inches of Class II stone base course, followed by 6 inches of Class II soil cement base, overlaying a 12-

inch treated subgrade layer. The soil borings and CPT soundings confirm the presence of dense to very dense soil cement and stone base material. Additional discussion regarding the pavement investigation is provided in Section 5.0.

3.4.2 Embankment

The cross-section sheets in the plans indicate fill thicknesses varying between 3 and 9 feet within the limits of this investigation. With the exception of Boring B-05, the upper 3 to 6 feet of all soil borings show the presence of soil cement along with silty sand fill having a fines (material passing the No. 200 sieve) content between about 30% and 50%. Boring B-05 was performed off the shoulder and encountered lean clay with sand and sandy lean clay within the apparent embankment portion of the boring.

In addition to classifying the soils using laboratory testing, the CPT data were also used to classify the soils. The Robertson & Campanella normalized Friction Ratio method generally classifies the upper 3 to 6 feet as "very stiff fine-grained soils," which are distinguished separately from the naturally occurring fine-grained clays and lean clays. This generally confirms the fill thicknesses shown in the plans.

3.4.3 Natural Soils

Below the fill, the natural soils generally consist of medium stiff to stiff fat and lean clays with some intermittent sand layers to an elevation of about +150 ft to +155 ft, followed by medium dense to very dense silty sand to the maximum exploration depth.

A zone of slightly softer, medium stiff lean clay was encountered approximately between the +160- and +170-foot elevations in several of the borings. However, this zone does not appear to be clearly defined in the CPT soundings.

CPT soundings 05 through 08 exhibit a zone of soft clay approximately between the +179- and +183-foot elevations. Based on the plans, this would appear to be near the interface between the fill and natural soils. At this elevation, Boring B-07 encountered a medium stiff fat clay having a plasticity index (PI) of 50 and a 10% organic content.

4.0 GEOTECHNICAL EVALUATION & ANALYSES

The following sections describe the analyses made to evaluate the potential modes of geotechnical related failure/movement identified above:

4.1 Slope Stability

A CPT sounding and a series of three borings were performed transverse to the roadway at Station 342+78. This location was selected to provide a cross-section for stability analyses at the approximate midpoint of the cracked pavement panels. This location also coincided with one of the locations where LTRC had trenched the concrete pavement (described in more detail in Section 5.7). A fence diagram of the soil stratigraphy at this location is shown in Appendix 5.

Slide version 7.023 by Rocscience Inc. was used to perform the stability analyses. The Spencer, Bishop, and Janbu methods were all used to evaluate the critical failure surface (the circular failure arc associated with the lowest factor of safety). Factor of safety (FoS) values obtained by the optimized Spencer method are reported herein. FoS values greater than 1.0 indicate a stable slope configuration, with a FoS of 1.3 to 1.5 typically being used in design of highway embankments.

The site geometry was modeled using the plan cross-section for the northbound lanes at Station 343+00. For the undrained analysis, geotechnical conditions were modeled using a composite of parameters from the relevant soil borings. No laboratory testing was performed to derive drained parameters; therefore, conservative values for the long-term friction angle in clays were selected for the drained analysis. Although the road was not opened to traffic at the time of cracking, a 250-psf traffic surcharge was modeled for all cases, as is typical during the design of embankments.

A "baseline" case representative of the typical conditions was run for the undrained and drained cases. Additional undrained cases were analyzed to model the potential effects of

- A completely saturated embankment (i.e., the water table at the ground surface); and
- The presence of the weaker zone (S_u = 400 psf) of soil encountered just beneath the embankment in CPT soundings 05 through 08 (as described in Section 3.4.3).

The minimum factors of safety from the analyses are shown in Table 2 and in Appendix 6. Note that the drained analysis was constrained to fail through the embankment; otherwise, the critical failure surface (still having a FoS greater than 1.0) did not pass through the embankment or pavement.

Table 2: Slope Stability Minimum Factors of Safety

Case	FoS
Undrained, baseline conditions	2.68
Undrained, saturated embankment	2.67
Undrained, saturated embankment with S_u = 400 psf zone beneath the fill	2.62
Drained, baseline conditions	2.96

As is typical during forensic slope stability investigations, an additional case was analyzed to force the FoS to an approximate value of 1.0. For this analysis, the undrained baseline case was used, and then the S_u value of the natural soils was reduced until the FoS value was just under 1.0. In order to make the critical failure surface pass through the pavement, the corresponding S_u was 370 psf. This is significantly lower than the average S_u value observed throughout the soil borings. The results of that analysis are also shown in Appendix 6.

4.1.1 Discussion of Results

Even with conservatively selected soil parameters, the baseline cases for the undrained and drained stability analyses yielded FoS values in excess of 2.6. The analyses with the saturated embankment and the weak soil layer represent the worst cases that may have reasonably existed, and these still yielded high FoS values. In addition, the following observations are noteworthy:

- A slope stability failure in an embankment typically forms a semicircular scarp; however, the crack was a straight line;
- No signs of sloughing or progressive stability failure were evident in the side slopes;

- Slope stability problems are greatly affected by water infiltration into the scarp. It would therefore be expected that the crack in the pavement would continue to open progressively with additional rainfall events. However, it is understood that the crack has not widened measurably since it was first observed;
- Similarly, the repaired sections of pavement have not shown any signs of new cracking. It would be expected that the progressive nature of a slope stability failure would lead to similar cracking until the slope has moved enough to reach equilibrium; and
- The average shear strength needed in the natural soils in order to create a failure surface that passes through the pavement is extremely low and not representative of the soil borings and CPT soundings made at the site.

In light of these observations, it is unlikely that a slope stability failure is the cause of the cracking.

4.2 Differential Settlement of Natural Soils

Geologically, the site is underlain by alluvium deposits of the Holocene age. These deposits consist of gray to brownish gray clay and silty clay with some sand and gravel locally. Based on geological maps, this alluvium deposit forms a band running approximately parallel to the project alignment within the cracked area. The band of alluvium is surrounded by deposits of the Wilcox Group of the Paleocene age. The alluvium deposits are younger and less overconsolidated than the surrounding Wilcox Group.

According to geological maps, three fault lines run approximately east-west near the project site. These fault lines are only shown in the Wilcox Group, and are bisected by the alluvium deposits that underlay the site. If it were continuous, the middle fault line would cross the project alignment near Station 343+00.

A consolidation settlement analysis was performed to estimate the amount of settlement induced in the natural soils due to the weight of the new embankment fill. Separate analyses were run using soil parameters from Borings B-03 and B-05. The total estimated primary consolidation settlement for a 7-foot fill height is 2 to 3 inches.

A time-rate analysis was also conducted to estimate the amount of residual settlement that would be expected to remain after the fill placement was complete. This analysis yielded an estimate of about ¼ inch of residual settlement within about one month of fill placement. This is because the soils mostly lie within the recompression portion of the settlement curve, and therefore settlement would have been expected to occur relatively quickly. This translates to an almost negligible differential settlement across the roadway.

4.2.1 Discussion of Results

Based on the settlement analyses, very little residual settlement would be expected after the placement of the embankment fill. Although LADOTD does not establish a specific post-construction settlement criterion for design, the estimated residual value of ¼ inch would certainly be acceptable for any typical roadway project. The residual settlement of ¼ inch would likely be distributed somewhat throughout the embankment fill, making differential settlement at the pavement level almost negligible.

It is also unlikely that the presence of fault lines in the project vicinity would have led to a straight line crack in the pavement. The fault lines depicted near the site on the geological map run approximately perpendicular to the crack in the pavement. The pavement distress, if caused by the faulting, would be expected to follow the fault line. A representative of the Louisiana Geological Survey indicated that the illustrated fault lines were based on unpublished mapping from 1970. During more recent geological mapping of the area, LGS found no evidence of fault movement at the surface.

4.3 Differential Settlement of the Embankment Fill

It is possible that the embankment fill itself settled due to poor site preparation, inadequate compaction and moisture control, excessive lift height, poor fill material, or combinations of these items ultimately leading to excessive void space in the fill.

4.3.1 Compaction

Most soils have a single maximum compacted dry density ($\gamma_{d\text{-}max}$), which is also associated with an optimum moisture content (W_{opt}). This is the concept behind a Proctor compaction curve, which establishes the moisture-density relationship of a specific type of soil by compacting multiple samples from the same soil type at different moisture contents (W) and dry densities (γ_{dry}). A soil that is too wet of W_{opt} has too much water in the void space to achieve $\gamma_{d\text{-}max}$. A soil that is too dry of W_{opt} needs more water to lubricate the soil particles, which allows them to be compacted more closely together. Furthermore, a soil that is compacted too dry of W_{opt} may be susceptible to volume change when moisture is introduced, which is unwanted after construction is complete. Therefore, it is desirable to achieve a level of compaction that maximizes the dry density and optimizes moisture content.

In theory, the same soil at any given moisture-density state should plot along a fully developed Proctor curve, with small deviations expected due to the heterogeneity of soil. Soils with different characteristics (corresponding to differing plasticity, grain size, specific gravity, etc.) will plot along different curves. Therefore, separate Proctor curves should be established for each different type of fill brought to the site.

Table 3: Typical Values for Compacted Fills

Soil Type	W_{opt}		$\gamma_{d\text{-}max}$ (pcf)	
	Min	Max	Min	Max
Lean clay (CL)	12	24	94	119
Silty sand (SM)	11	16	109	125
Clayey sand (SC)	11	19	106	125
Poorly graded sand (SP)	12	21	94	119

Table 5.17 of the FHWA Publication Geotechnical Aspects of Pavements Reference Manual (NHI-05-037) lists typical maximum and minimum values for $\gamma_{d\text{-}max}$ and W_{opt} of various types of compacted fill. Typically, the locally available usable embankment material consists of either silty sand (SM), clayey sand (SC), poorly graded sand (SP), or lean clay (CL). This is confirmed by the soil borings, which indicate most of the fill is comprised of silty sand. The typical compacted max/min values for those soil types as recommended by FHWA are tabulated in Table 3.

4.3.2 Soil Properties of Embankment Fill

Table 4 presents the results of embankment fill testing as furnished by District personnel. The tabulated properties include sample identification, percentages of sand, silt, and clay, liquid limit (LL), plasticity index (PI), AASHTO soil group, and soil type as classified by the Unified Soil Classification System (USCS). The AASHTO soil group is normally used to classify soil for use in pavement subgrade evaluation. The USCS system is typically used in general geotechnical engineering applications.

Table 4: Embankment Fill Classification

Ident.	%Sand	%Silt	%Clay	LL	PI	Soil Type (USCS)	Soil Group
7A-14	8	43	49	56	25	Fat clay (CH)	A-7-5 (28)
7A-8	2	49	49	57	23	Fat clay (CH)	A-7-5 (29)
7A-24	31	35	34	52	25	Fat clay (CH)	A-7-6 (17)
7A-6	6	43	51	50	22	Fat clay (CH)	A-7-6 (24)
7A-30	23	36	41	34	16	Lean clay (CL)	A-6 (11)
7A-9	7	41	52	39	14	Lean clay (CL)	A-6 (14)
7B-02	17	42	43	43	20	Lean clay (CL)	A-7-6 (17)
7A-25	46	42	12	0	No Plasticity	Sandy silt (ML)	A-4 (00)
7A-4	47	21	32	25	4	Silty clay (CL-ML)	A-4 (00)
7A-32	46	29	25	25	7	Silty clay (CL-ML)	A-4 (01)
7A-5	75	22	3	0	No Plasticity	Silty sand (SM)	A-2-4 (00)
7A-16	66	29	5	0	No Plasticity	Silty sand (SM)	A-2-4 (00)
7A-17	68	28	4	0	No Plasticity	Silty sand (SM)	A-2-4 (00)
7A-18	71	26	3	0	No Plasticity	Silty sand (SM)	A-2-4 (00)
7A-22	65	22	13	25	7	Silty sand (SM)	A-2-4 (00)
7A-33	77	15	8	0	No Plasticity	Silty sand (SM)	A-2-4 (00)
7B-01	73	20	7	0	No Plasticity	Silty sand (SM)	A-2-4 (00)
7A-1	63	35	2	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-10	57	31	12	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-11	53	32	15	26	No Plasticity	Silty sand (SM)	A-4 (00)
7A-12	62	28	10	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-13	52	42	6	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-15	45	42	13	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-21	57	30	13	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-26	56	35	9	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-27	52	38	10	0	No Plasticity	Silty sand (SM)	A-4 (00)
7A-23	51	26	23	27	9	Silty sand (SM)	A-4 (02)

No other information regarding these samples was furnished by the District. It is assumed that the samples tabulated in Table 4 were accepted for use as fill. However, the exact placement of the material (station, offset, and lift) cannot be correlated to the sample identifications based on the furnished information.

According to the Louisiana Standard Specifications for Roads and Bridges, 2006 Edition, the submitted test results appear to meet the requirements for Usable Soils in Section 203.06.

4.3.3 Field Verification Testing

It is understood that W_{opt} and $\gamma_{d\text{-}max}$ of embankment fill were determined using LADOTD TR 415, Field Moisture-Density Relationships. This test method allows for either a 1-point Proctor/Family of Curves approach, or a 3-point Proctor/Field Curve approach. Both methods were employed for this project. Reference is made to the TR document for specific details on the performance of each method.

The 1-point approach involves compaction of a single soil sample slightly dry of optimum. If that point's moisture and density plots within a predetermined range on the Family of Curves, then the nearest curve's $\gamma_{d\text{-}max}$ and W_{opt} are used. This method assumes that the shape of the soil's actual moisture-density relationship would follow the predetermined curves on the Family of Curves plot. Documentation was furnished for ten tests using the 1-point method.

The 3-point approach involves compacting three samples from the same soil type at different moisture-density values. The intent is to test samples dry of optimum, wet of optimum, and near optimum in order to establish the shape of the curve. The test method allows utilization of the Proctor curve for multiple field verification tests, if the soil type is essentially the same. Samples whose moisture and dry density do not fall reasonably near a given Proctor curve are likely representative of a change in soil type, and should therefore require a new Proctor curve. Documentation was furnished for four tests using the 3-point method. These curves were replotted and are included in Appendix 7.

After establishing the moisture-density relationship for representative fill types, in-place density testing was conducted in accordance with LADOTD TR 401, The Determination of In-Place Density. This method allows for density testing using either a nuclear density device or a sand cone. It is understood that nuclear density tests were performed for this project. For this method, the device's probe is driven a distance into the compacted lift, and the W and γ_{dry} are determined three times. The three tests are then averaged for the final test results. In some cases, a field moisture content was determined using LADOTD TR 403, Determination of Moisture Content. Documentation was furnished for 28 nuclear density tests.

Section 203.07 of the Standard Specifications requires that embankment fill be compacted to within 95% of $\gamma_{d\text{-}max}$ and at a moisture content within ±2 of W_{opt}. The furnished in-place density test results indicate that all tests met the specified requirements.

4.3.4 Comparison of Soil Borings to In-Place Testing

Results from the in-place field testing were tabulated along with the geotechnical laboratory test results. This was done to compare the moisture content and density results from the soil borings with the nuclear testing done during the embankment construction. Note that the elevations assigned to the field density tests are estimated based on the reported lift number, test depth, and pavement surface elevation near the location of the test. Because the exact compacted lift thicknesses are unknown, and the total number of lifts along the alignment may vary, these elevations are only a rough estimate for comparison purposes in the figures below.

Figure 2 and Figure 3 show the field moisture contents and dry densities determined from the nuclear density testing plotted against approximate elevation. The values are divided into 3 groups: tests performed in the crack area (based on stationing), tests performed outside of the crack area, and tests performed within the treated subgrade and base material. Several min/max envelopes are also shown: the envelope bounding the in-place testing as well as the envelopes recommended by FHWA for compacted sands and lean clays from Table 3 above.

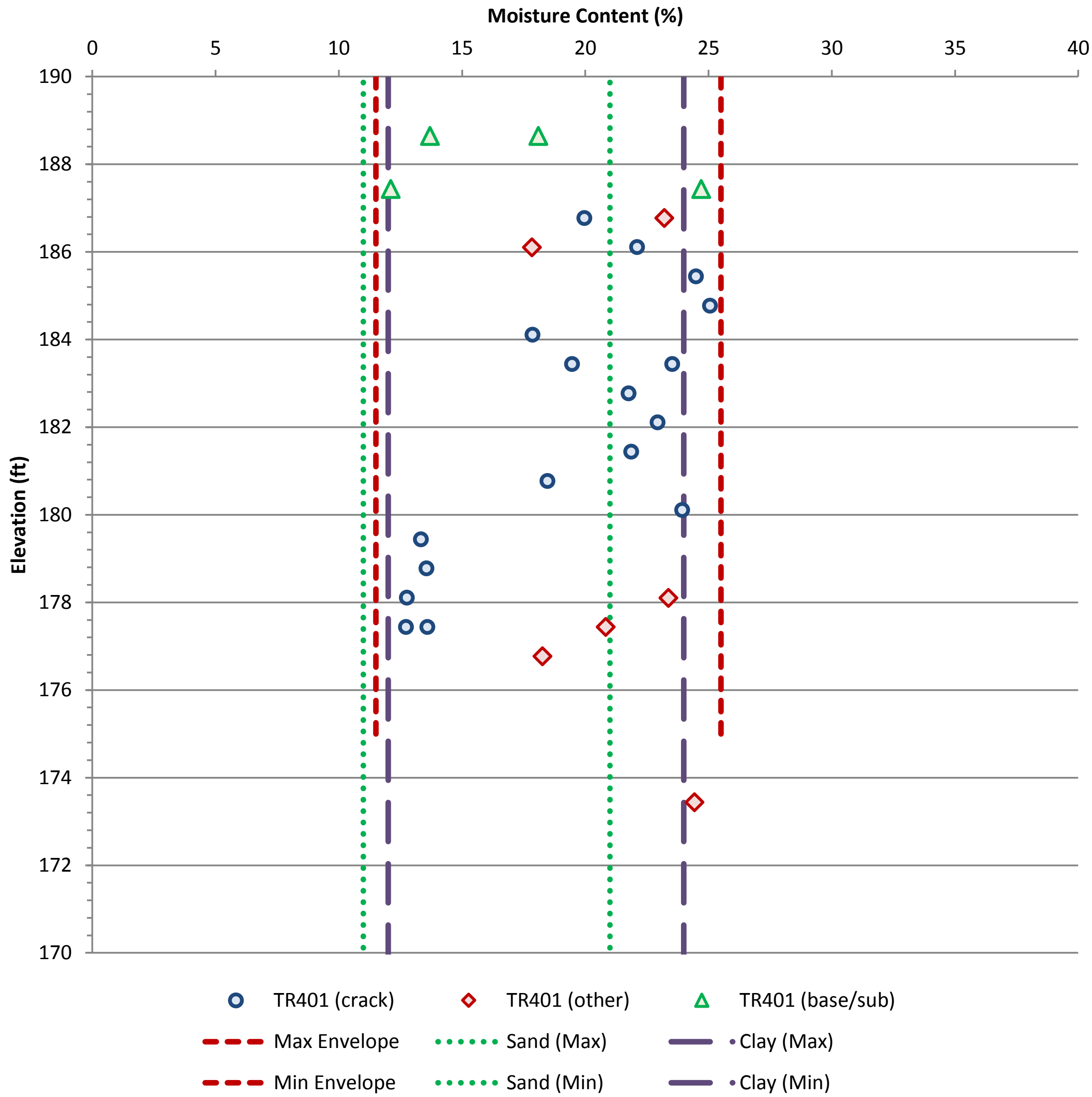


Figure 2: Moisture Content (from Nuclear Density Gauge) vs. Elevation

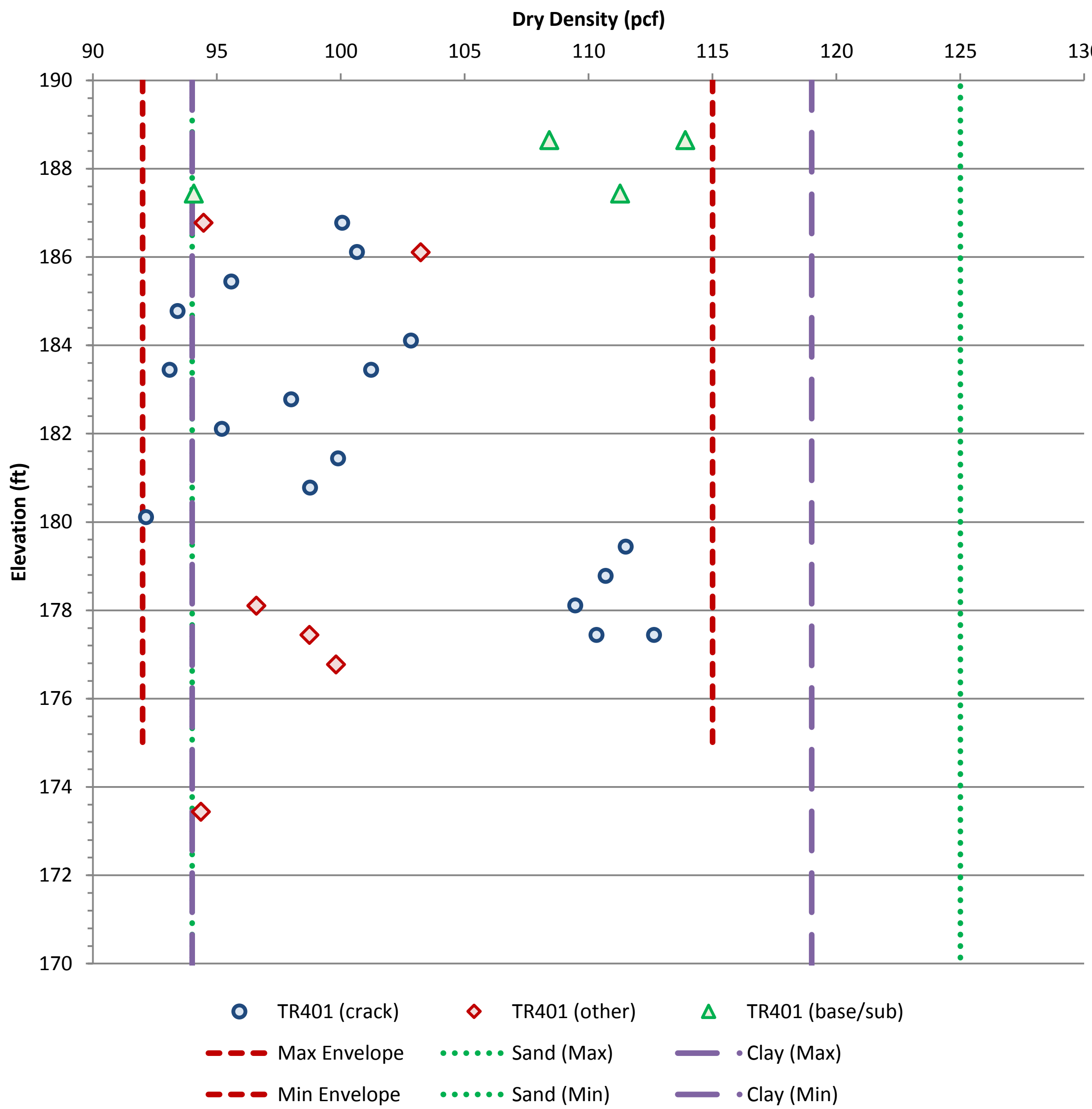


Figure 3: Dry Density (from Nuclear Density Gauge) vs. Elevation

Based on these plots, some of the test results indicate moisture contents higher than the upper bound established for compacted lean clays. Many of the tests fall significantly higher than the upper bound established for compacted sands. Similarly, several of the tests for dry density fall below the lower bound for both compacted sands and lean clays. It should be noted that the majority of the classification tests submitted for fill acceptance indicated that the borrow material was comprised of silty sand. Only a few of the tests were classified as lean clay. Unfortunately, the type of material in each tested lift was not recorded on the nuclear density test documentation, so it is not possible to separate each of the plotted points into clay and sand categories.

Also noteworthy are the values of W and γ_{dry} within the first three lifts in the crack area, as compared with the remaining lifts. The first three lifts have a significantly lower W, higher γ_{dry}, and lower coefficient of variation (CoV), as shown in Table 5.

Table 5: Variability in Lift Compaction

Lifts	W (%)		γ_{dry} (pcf)	
	Mean	CoV	Mean	CoV
0 through 3	13.2	3%	110.9	1%
4 through 14	21.8	10%	97.6	4%

The min/max envelope for the in-place nuclear density testing was then superimposed on the moisture content versus elevation data from the geotechnical laboratory, shown in Figure 4 below.

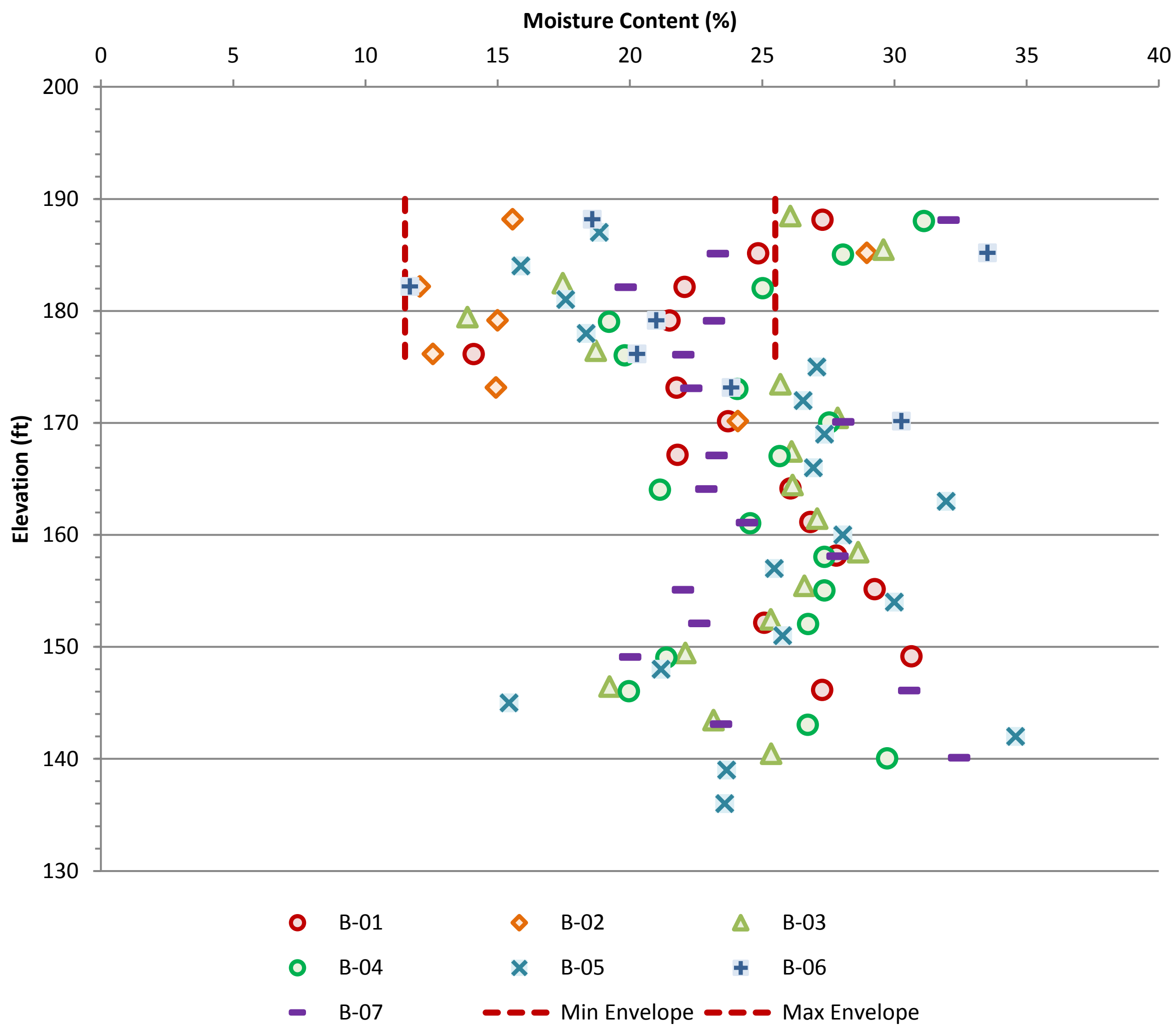


Figure 4: Moisture Content (from Laboratory Testing) vs. Elevation

Based on these results, the following observations can be made:

- Several of the samples tested in the lab had moisture content values significantly higher than any of the moisture contents from the nuclear density testing;
- The maximum and minimum moisture contents from the laboratory match those from the in-place testing below the approximate 185-foot elevation; and
- The moisture content variation within the controlled-compacted fill is clearly higher than that within the natural soils.

4.3.5 Comparison of Family of Curves to Field Curve Method

The District furnished four 3-point Proctor curves, which were used as field verification references for six in-place nuclear tests. In order to evaluate the compatibility of the Family of Curves and Field Curve methods, a single point from each of the 3-Point Proctors was plotted against the Family of Curves. The other two moisture-density points were ignored. In other words, points were discarded in order to convert the 3-point Proctors into 1-point Proctors. The nearest curve in the Family of Curves was then used to establish the theoretical W_{opt} and $\gamma_{d\text{-}max}$ for that sample, as per the TR Method.

It makes sense that if the Family of Curves method is compatible with the Field Curve method, the W_{opt} and $\gamma_{d\text{-}max}$ values should be approximately the same for both methods. Furthermore, because both methods are allowed in the TR, the test method used should not influence whether a test passes or fails. The acceptance criteria (percent of maximum dry density and deviation from optimum moisture content) were computed for both methods and compared. The results are shown in Table 6.

Table 6: Family of Curves vs. Field Curve Results

		In-place		*Field Curve*			*Family of Curves*		
Sample	**Station**	**W**	**γ_{dry}**	**Reference**	**% $\gamma_{d\text{-}max}$**	**± W_{opt}**	**Curve No.**	**% $\gamma_{d\text{-}max}$**	**± W_{opt}**
0040-001	340+25	13.6	112.6	0040-001	98.3%	-0.20	7	97.1%	0.7
0040-002	346+10	12.8	109.5	0040-001	95.5%	-1.03	7	94.4%	-0.1
0018-003	341+15	12.7	110.3	0040-001	96.3%	-1.07	7	95.1%	-0.2
0040-003	342+15	13.6	110.7	0040-003	96.2%	0.27	7	95.4%	0.7
0040-017	344+50	24.7	94.1	0040-017	95.1%	1.10	20	95.4%	2.8
0040-019	343+00	18.1	108.4	0040-019	99.2%	0.70	N/A	-	-

Of the six tests in the comparison, all tests pass both criteria when using the Field Curve method. However, using the Family of Curves, only three of the tests pass.

- Sample 0040-002 does not achieve 95% of the maximum dry density;
- Sample 0040-017 is more than 2% wet of the optimum moisture content; and
- Sample 0040-019 has no moisture-density points that fall within the shaded band on the Family of Curves plot.

Although the sample size is relatively small, only 50% of the test results remain passing when switching from the Field Curve to Family of Curves Method. This suggests that the two methods are incompatible and that the predetermined Proctor curves in the TR method are not representative of the soils at this site.

4.3.6 Discussion of Results

There are several aspects of the geotechnical investigation and the field testing that suggest that there could be inconsistencies within the embankment fill:

- The classification tests used for fill source acceptance show that the fill material is variable. As discussed, all of the submitted samples meet the Standard Specifications for Usable Soils. However, from a purely geotechnical standpoint, 26% of the samples submitted should not be used for fill. Four samples (15%) classify as Fat Clay (CH), which is susceptible to volume change as moisture content varies. One sample (4%) classifies as sandy silt (ML) and two samples (7%) classify as silty clay (CL-ML). These samples have a high silt content, which can lead to difficulty in compaction as well as pumping behavior when wet. It should be noted that the soil borings generally encountered silty sand within the embankment, which matches the majority of the fill source tests. Therefore, it is unclear where these unsuitable materials were used in the embankment;
- Some of the W and γ_{dry} values from the nuclear density tests fall outside the range of values for compacted fill suggested by FHWA. About half of the moisture content values exceed the maximum suggested value for sandy fill. Given the amount of silty sand fill encountered in the soil borings, this is noteworthy. As discussed, the classification of the fill being tested in-place is not recorded on the data sheet;
- The W values from the soil borings generally corroborate the results from the nuclear density tests, with the exception of a series of elevated moisture content values within the upper 5 feet of the soil borings. These tests indicated elevated W values as high as 34%, whereas the nuclear testing yielded a maximum of about 25%. These elevated W values appear in several borings and do not seem to be confined to a specific location along the alignment;
- There appears to be more scatter in the moisture contents within the controlled-compacted fill than within the naturally occurring underlying soils. Furthermore, there appears to be much better moisture control within the first three lifts of fill than in the remaining lifts;
- All of the furnished field density tests passed; however, this project calls into question the efficacy of LADOTD's methods for establishing the moisture-density relationships of soils. This is discussed in more detail in the following section; and
- The CPT soundings on the northern portion of the cracked section consistently encountered a zone of soft clay approximately 6 feet from the top of the sounding. This corresponds roughly with the bottom of the fill. It is possible that there is a zone of weaker soil at the base of the fill that should have been proof rolled and removed prior to fill placement. It is curious that the first three lifts appear to have achieved good compaction, which would have been difficult over a soft clay layer. However, it is impossible to correlate the CPT soundings and in-place density tests, since the offset distance is not recorded on the density test sheets.

Based on these points, the most likely geotechnical contribution to the pavement cracking is variability within the fill materials, compaction, and moisture control. However, all materials passed the various methods of acceptance employed by LADOTD. Therefore, due to the shortcomings of the specifications and test methods employed by the Department, it is not possible to say with any certainty that this variability is the sole reason for the cracked pavement.

4.4 Evaluation of TR 415

LADOTD currently uses LADOTD TR 415, Field Moisture-Density Relationships to establish the moisture-density relationships used for fill acceptance. Several aspects of the test method are inadequate, which render a definitive conclusion as to the source of the cracking impossible with the currently available data.

4.4.1 Field Curve Method

As discussed, the TR method allows the construction of a 3-point Proctor curve using fill soils. The intent is that the curve can be used as the target moisture-density criterion for nuclear testing within similar fill soils. Although the Field Curve method may be theoretically sound, its actual application in the field has several shortcomings, including:

- The method allows a minimum of three points to establish the Proctor curve. This is based on the assumption that a single point wet of the W_{opt} point is enough to establish the entire "wet leg" of the curve. That assumption is contingent upon the wet leg of the curve being parallel to the zero air voids line. However, this is not necessarily true for all soils, and could introduce error into the generation of the Proctor curve. Practically speaking, a minimum of four points should be used to draw the Proctor curve, with the points being spaced so that the slope of each leg of the curve can be clearly established;
- Some of the Proctor curves are not clearly defined, such as 040-001 in Appendix 7. That curve only covers a range of about 2 pcf of γ_{dry} and about 4% of W;
- Based on the documentation furnished by the District, it appears that certain 3-point Proctor curves are used as a reference for multiple in-place density tests. According to LADOTD TR 401, The Determination of In-Place Density, "When in-place density is to be compared with moisture-density relationships determined in accordance with DOTD TR 415 for percent compaction, the test site for in-place density shall be the same location as the original site that material was obtained for the moisture-density relationships." However, it appears that Proctor 040-0001 was used for a reference about 600 feet from the original site; and
- Building upon the previous point, most of the plotted in-place density points do not fall on either leg of the established Proctor curves. It could be argued that those curves are therefore not representative of the material that was tested in-place. Without any classification testing, it would be impossible for the field technician to know whether the reference Proctor is representative of the location being tested. This is why TR 401 requires the testing to be made at the same site. Because the soil types are not documented, they cannot be cross-referenced. Therefore, the referenced Proctors have little meaning relative to the in-place density tests. This creates a situation where it is possible to "curve shop," or select the available Proctor curve that yields passing results.

4.4.2 Family of Curves Method

The TR method also allows the use of a 1-point Proctor, which can then be matched to a predetermined Family of Curves to determine the theoretical W_{opt} and $\gamma_{d\text{-}max}$ values for use as an acceptance criterion.

This method is even faster than the Field Curve method, and was therefore used more frequently on this project. However, its adequacy from an engineering standpoint is questionable:

- It is unclear where the Family of Curves presented in the TR method actually came from. Other states that employ a similar method use a different set of curves, which can vary depending on soil composition. In other words, these predetermined Proctor curves are not universal. However, LADOTD is taking these curves at face value in order to accept embankments for our interstate system;
- Literature suggests that a Family of Curves approach is valid when using multiple field curves from a project to develop a site-specific set of curves. However, that is not the method being employed by the TR;
- The exercise in a previous section of this report shows that the Family of Curves method is not always compatible with the Field Curve method. In that exercise, only half of the tests that passed using the Family of Curves remained passing when applying the Field Curve method. Certainly, the Field Curve method is more acceptable as a control, since the curve is (or should be) fully developed using soil from the project site. The Family of Curves uses only a single test to then select a theoretical curve for acceptance; and
- The Family of Curves approach uses two sets of curves: one for sandy materials and one for clays. All of the curves referenced by the District appear to be the numbered set of curves established for clays. None of the supplemental (sand) curves appears to have been used for acceptance of in-place density testing. However, the soil borings and the fill source classification testing both indicate that the majority of the fill material is silty sand. Therefore, it is possible that inaccurate W_{opt} and $\gamma_{d\text{-}max}$ values were used for acceptance of a large portion of the embankment.

5.0 CONCRETE PAVEMENT INVESTIGATION

The following sections will detail the coring and pavement testing conducted within the cracked pavement. The objective of the coring was to (1) Verify the formation of full-depth cracks at the longitudinal joint and (2) To assess the concrete below the longitudinal crack.

The condition of the pavement was also tested with the Falling Weight Deflectometer (FWD) and the Walking Profiler.

5.1 Objective and Scope (concrete pavement investigation)

The objective of the study was to determine and document the cause of cracking noted in a section of Interstate 49 North located close to LA 1 in Shreveport, LA. To meet the objective, six cores were obtained for visual observation, three over the longitudinal crack and three from the longitudinal joint between the lanes. FWD tests were conducted at seven locations and the Walking Profiler was used to measure cross-slope changes at five locations in the distressed section.

5.2 Methodology

LTRC Concrete Research Laboratory personnel obtained a total of seven cores in the presence of District 04 personnel in February 2017. The location of the coring relative to the panels is shown in Figure 5. The cores were visually examined for the depth of saw cut and evidence of aggregate segregation, excessive air entrapment, and vibrator trails. The cores were also photographed and the visual observations are documented in the results section.

FWD test results as well as measurements obtained using the Walking Profiler are discussed in the results section.

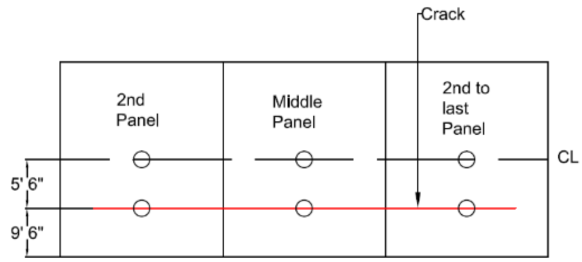


Figure 5: Core Locations

5.3 Discussion of Results

This section presents the photographs of the cores obtained and documents the visual observations associated with them.

5.3.1 Core Conditions

The cores retrieved were about eleven inches in depth. Cores over the longitudinal joint had full-depth cracks below the saw cut. The cores obtained at the longitudinal crack split open after they were extracted.

5.3.2 Visual Observations

Figure 6 presents a photograph of the cores obtained at the centerline longitudinal joint. The saw cuts are as specified in DOTD standard plan CP-01 Detail B, with a 4-inch deep saw cut and a 1-1/2-inch widened cut for the backer material and sealant. The cracks below the saw cut extend full-depth into the concrete, indicating that the joint functioned as designed.

Figure 6: Cores from the Centerline

Figure 7: Cores from the Longitudinal Joint

Figure 7 shows pictures of cores obtained over the longitudinal crack. The concrete in the core appears to be uniformly distributed, showing no evidence of aggregate segregation or vibrator trails. There is no evidence of excessive entrapped air voids.

On the roadway, the longitudinal crack extends over a length of 21 panels, with replacement panels on both ends of the crack. However, the longitudinal crack does not extend into the replacement panels. Close observation by the district personnel revealed a small crack on one of the replacement panels and a core was extracted at that location. However, visual observation of the core showed that the crack did not extend full depth into the concrete (Figure 8).

Figure 8: Core from one of the Replacement Panels

5.4 MIT-SCAN2-BT analysis

The MIT-SCAN2-BT system was used to verify dowel positions in the concrete pavement. All joints associated with the cracked panels were scanned and the collected data was analyzed. The dowels were found to be within tolerances specified in the DOTD standard plan CP-01.

5.5 Falling Weight Deflectometer

Tests were conducted at seven locations with the Falling Weight Deflectometer to assess the condition of the pavement layers. There were five layers in the pavement structure, 11 in. PCC, 4 in. stone, 6 in. soil cement, 12 in. cement treated subgrade layer, and untreated soil or subgrade. The backcalculation process has a limitation in that it is not able to provide accurate data for more than four layers. Because of this, two backcalculations were performed, so as to better assess the layers. Table 7 presents the results of testing.

The purpose of backcalculation 1 was to determine the strengths of the PCC, stone, and soil cement. Since the cement treated subgrade layer was merged with the subgrade, it made the subgrade appear to be stronger than it really was. When the PCC is stronger than 4500 psi, its condition is considered good. In this case, PCC values at all locations were in good condition. There was no significant difference between either side of the longitudinal crack. Regarding stone base course, values greater than 45 ksi are considered as indicators of good condition. As presented in Table 1, all stone resilient modulus values exceeded 45 ksi. The soil cement indicated a similar trend as seen in the PCC and stone layers. Values in the range of 300 ksi are considered good.

Backcalculation 2 was performed to determine the subgrade strength. Typically, when the section thicknesses are varied or merged, the backcalculation results begin to differ, which is the reason why the values for the PCC, stone, and the merged soil cement and subgrade layer (18 in. soil cement) differ from backcalculation 1. The Corr. Sub column represents the strength of the subgrade correlated to laboratory values. As presented in the table, subgrade resilient modulus values ranged from 2.5 to 3.1 ksi, indicating that it was very weak.

Based on the FWD assessment of the pavement structure, it appears that all layers except the subgrade were in good condition.

Table 7: FWD Test Results

			Backcalculation 1 (values in ksi)						Backcalculation 2 (values in ksi)				
Direction	**Lane**	**Identifier**	**11" PCC**	**4" ST**	**6 in SC**	**Sub**	**Corr Sub**		**11" PCC**	**4" ST**	**18 in SC**	**Sub**	**Corr Sub**
NB	Outside	LOC	8429.4	81.3	330.1	12.2	**4.8**		5425.9	71.1	162.1	6.5	**2.5**
NB	Outside	ROC	8333.6	79.8	331.8	12.7	**4.9**		5524.4	60.7	161.8	6.5	**2.5**
NB	Inside		8857.7	69.4	294.6	14.8	**5.8**		6563.7	50.2	169.1	8.0	**3.1**
SB	Inside		10670.9	74.1	298.8	12.2	**4.8**		7631.1	44.9	129.9	6.6	**2.6**
SB	Outside		8629.1	59.6	282.0	14.7	**5.7**		5926.2	45.2	167.1	7.9	**3.1**

Legend: LOC and ROC mean left and right of longitudinal crack. All other tests were performed at mid slab.

5.6 Walking Profiler

The walking profiler was used to measure changes in the cross slope of the pavement at panels, 2, 11, 16, 17, and 20 and the results are presented in Figures Figure 9 through Figure 13. Measurements began at the inside edge of the inside lane and ended near the outside edge of the outside lane. Rumble strips interfered with taking of readings on both shoulders. The blue line represents walking profile data while the red line represents the theoretical pavement surface, which should be straight from lane edge to lane edge in areas outside of horizontal curves and transitions. All panels were outside of curves and transitions. Most of the panels showed a slight depression occurring in the outside panel. The longitudinal crack was at 15 ft. The results indicated that some movement had occurred in the outside lane.

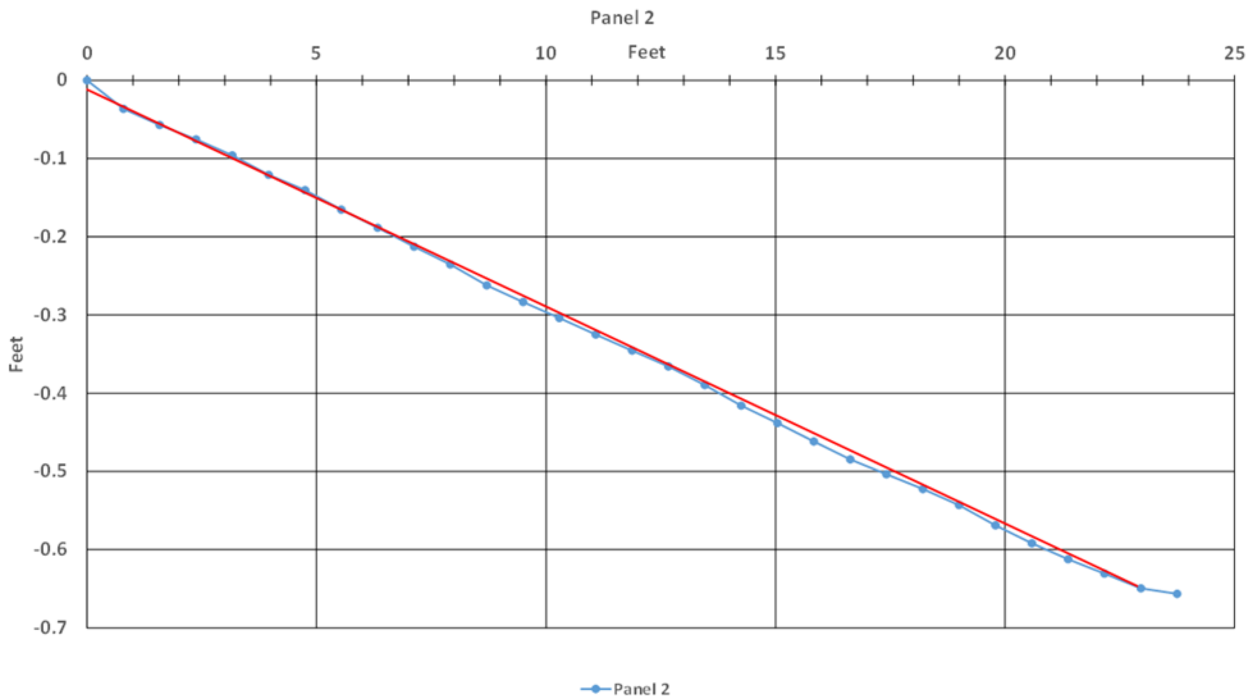


Figure 9: Walking Profiler Results from Panel 2

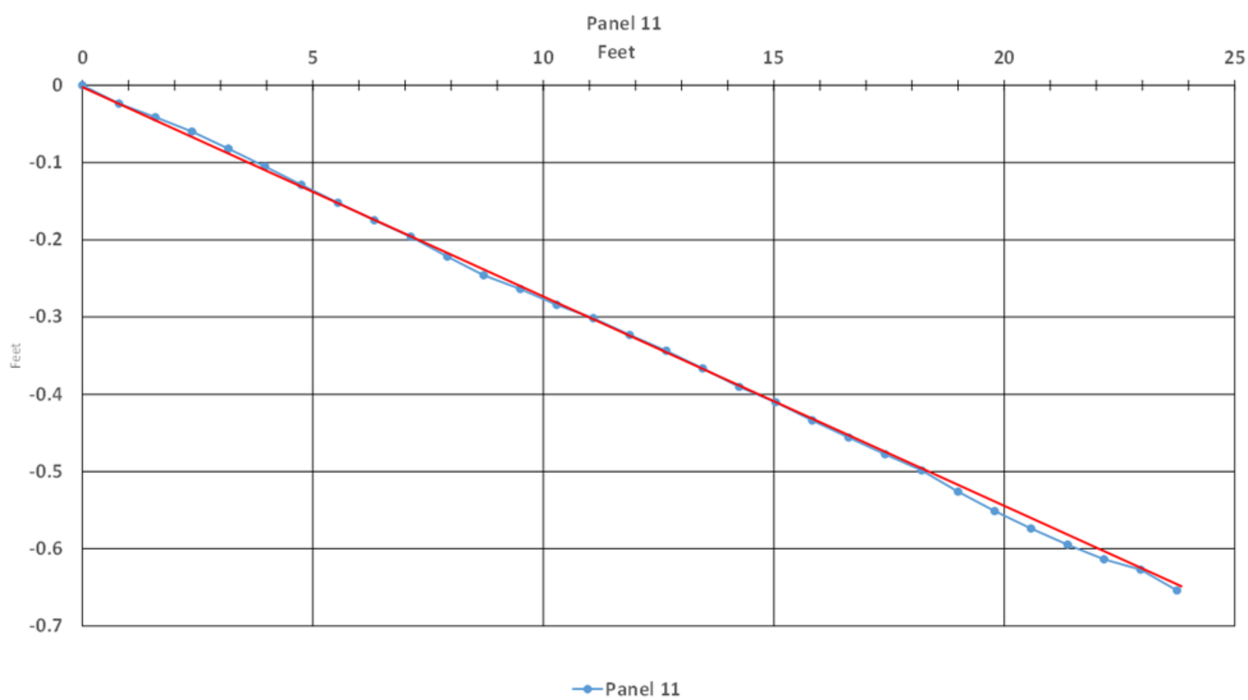


Figure 10: Walking Profiler Results from Panel 11

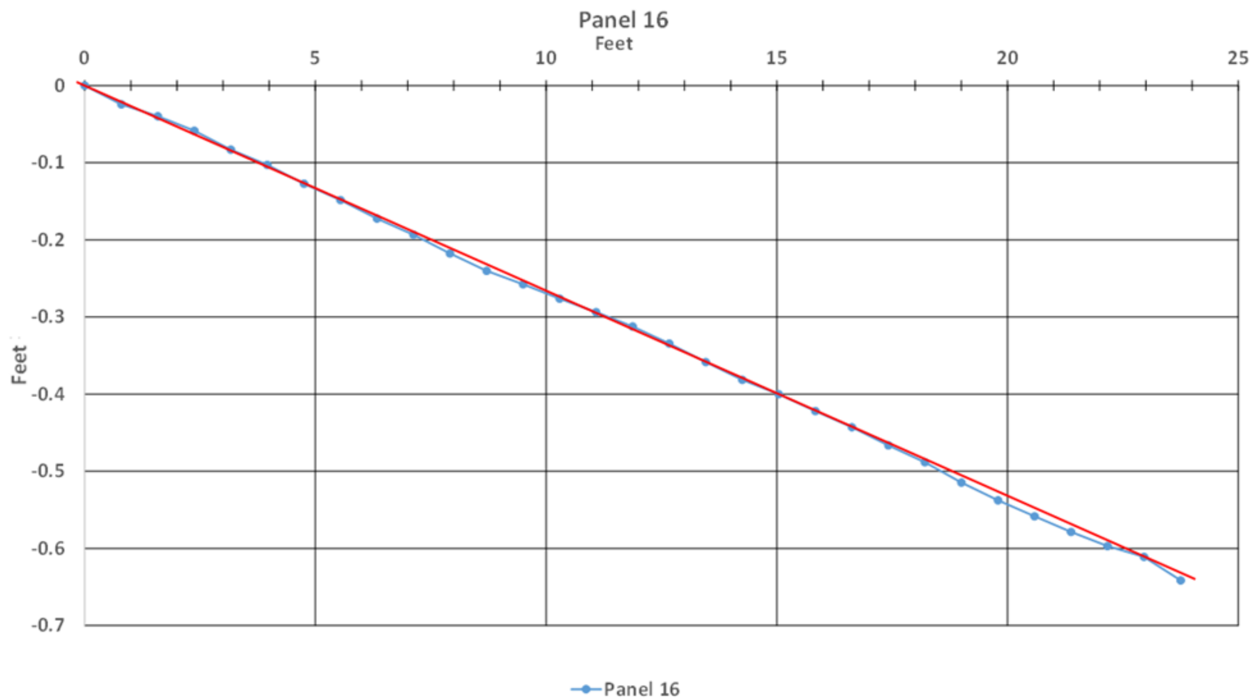


Figure 11: Walking Profiler Results from Panel 16

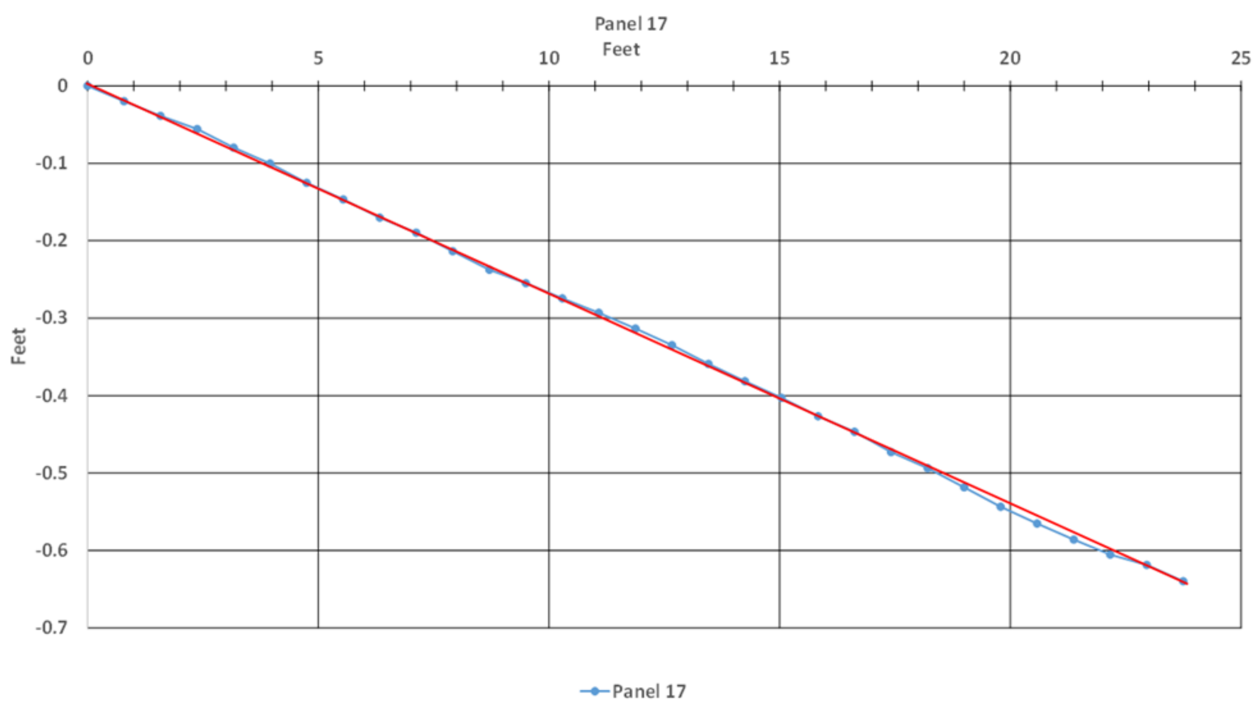


Figure 12: Walking Profiler Results from Panel 17

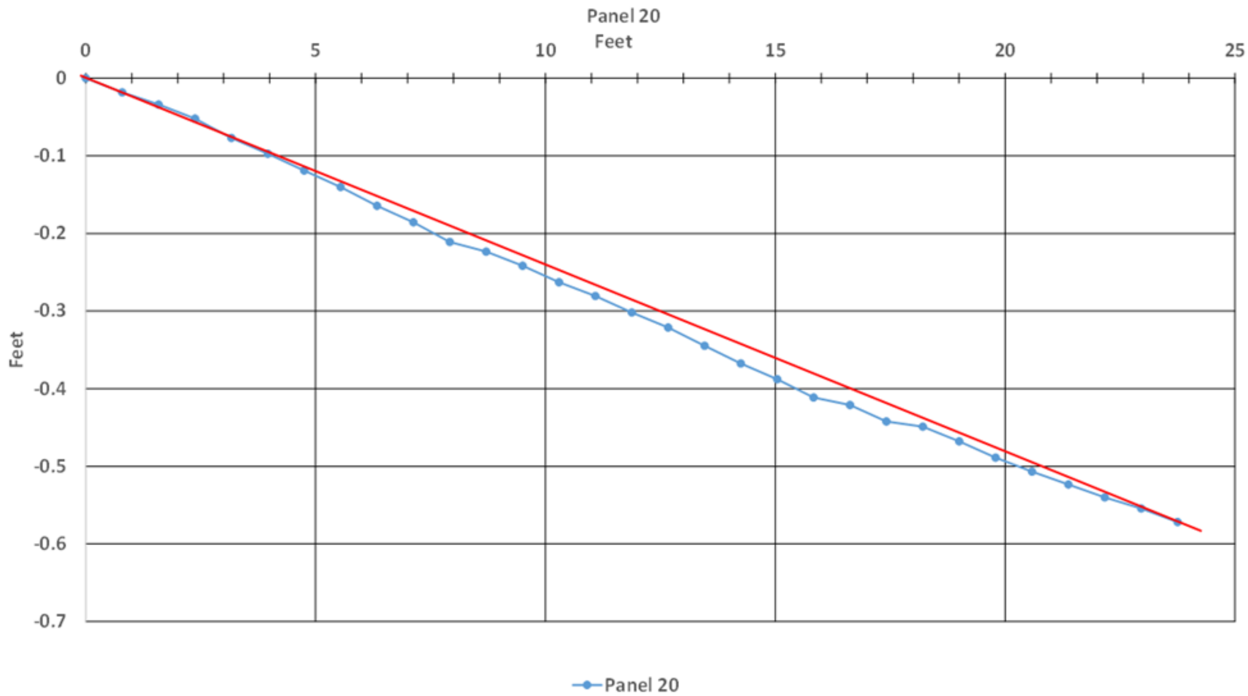


Figure 13: Walking Profiler Results from Panel 20

5.7 Concrete Trench

After walking profiler results showed discrepancies across the joint in a few locations, a decision was made to perform full depth trenching within the cracked pavement area. One location was in the center of the cracked area (Panel 11) and another was at the north end of the cracked pavement area (Panel 20). The trenching results showed three things:

- The concrete showed no segregation or abnormalities thought the depth or cross section (See Figure 14);
- The 4-inch granular layer was cracked directly below the pavement crack (See Figure 15); and
- The soil cement was cracked approximately 10-15 inches toward the centerline of the pavement compared to crack within the pavement (See Figure 16).

These results show that the cracking is most likely top down and is behaving like a cantilevered beam with the weaker soil cement layers failing at shorter distance from centerline versus the stronger concrete layer failing at the longer distance from centerline as expected.

Figure 14: Concrete cross section

Figure 15: 4-inch granular crack

Figure 16: Soil cement base crack

6.0 Conclusions

Three main potential geotechnical modes of failure/movement were investigated as possible contributors to the pavement cracking observed at the site. Those included slope stability, differential settlement of the underlying natural soils, and differential settlement of the embankment fill. A geotechnical investigation, which included soil borings, cone penetrometer soundings, and laboratory testing, was conducted in order to develop soil parameters for evaluation of these potential modes of failure. The District also furnished field verification documentation, which included fill source classification testing, in-place nuclear density testing, and moisture-density relationship testing.

Undrained (short-term) and drained (long-term) models were evaluated for slope stability. Relatively conservative cases were modeled, and the critical factors of safety computed for all models were quite high. Furthermore, slope stability failure is typically progressive and exacerbated by the introduction of water, although progressive opening of the crack has not been noted. Therefore, based on the information available at this time, slope stability was ruled out as a contributor to the pavement cracking.

Differential settlement of the underlying soils was also analyzed. Although the site is located over relatively young Holocene age alluvium deposits, a settlement analysis indicated that the amount of residual differential settlement after fill placement should have been minimal. Therefore, differential settlement of the natural soils was ruled out as a contributor to the pavement cracking.

Finally, differential settlement of the embankment fill itself was evaluated. Based on the information available at this time, it appears that there is a relatively high variability associated with the embankment fill in terms of soil type, moisture content, and in-place density. Furthermore, a zone of soft clay was encountered in several of the cone penetrometer soundings at the interface between the fill and natural grade. Therefore, it appears that there may be zones of poor moisture control, compaction, and site preparation within the embankment. However, according to records furnished by the District, all fill material within the cracked zone meets the required specifications for classification, moisture content, and dry density.

Further evaluation of the specifications and test methods used reveals that there are shortcomings in both that make it impossible to render a definite opinion as to the quality of the embankment fill. Specifically, the Standard Specifications allow poor soils (fat clay, silty clay, and sandy silt) to be used as Usable Soils. These types of soils could be susceptible to volume change, pumping, and difficulty achieving proper moisture control and compaction. Furthermore, LADOTD TR 415, Field Moisture-Density Relationships is inadequate for establishing compaction criteria in certain scenarios, several of which may have arisen on this project.

Thus, it is possible that although the fill was installed in accordance with LADOTD specifications, portions of the embankment may not be adequately compacted when analyzed from a more rigorous geotechnical standpoint. Although the variability within the fill (including the soft zone at the bottom of the fill) is the most likely geotechnical-related contributor to the crack, it is not possible to know with certainty based on the limited information available at this time. It is assumed that all embankments in the state are constructed using these same specifications and test methods, so a more detailed comparative investigation would be needed to determine whether the same variable conditions are present in all recently constructed embankment fills, and whether they correlate with observable problems such as embankment instability or cracked pavement.

Visual examination of the extracted cores revealed uniformly distributed concrete in the location of the longitudinal crack, with no evidence of vibrator trails. The cores obtained at the centerline longitudinal joint show the formation of full-depth cracks beneath the saw cut, indicating that the joint performed as expected. FWD results indicated that all layers except the subgrade were in good condition. This supports the results of the geotechnical investigation, which suggest that the quality of the embankment fill is the most likely cause of the cracking. The results of the walking profiler indicated that some movement had occurred in the outside lane.

In addition, the crack does not extend into the panels that were replaced at both ends of the crack. Therefore, replacing the cracked panels may solve the observed problem. The panel replacement option is also supported by the apparent absence of a slope stability type of failure that, if present, could be progressive in nature and continue to undermine replacement panels.

APPENDICES

1. Boring Plan
2. Fence Diagram Along I-49 Cracked Section
3. Soil Boring Logs
4. CPT Parameter Plots
5. Fence Diagram Transverse to Interstate at 342+78
6. Slope Stability Analysis Output
7. 3-Point Proctor Curves


H.003496: I-49 North, Seg J
Soil Boring Locations for Cracked Pavement Investigation
Legend
20' Boring (Pavement)
50' Boring (Grass)
50' Boring (Pavement)
50' CPT (Grass)
50' CPT (Pavement)
CPT-05 (47.4')
CPT-02 (40.0')
B-03 (53')
CPT-06 (48.2')
B-01 (47')
B-02 (21')
B-04 (53')
B-06 (21')
B-07 (53')
CPT-08 (46')
CPT-07 (47.8')
CPT-01 (28.3')
CPT-03 (38.1')
B-05 (56')
N
100 ft
Google Earth
© 2016 Google

LADOTD
1201 Capitol Access Road
Baton Rouge, LA 70804

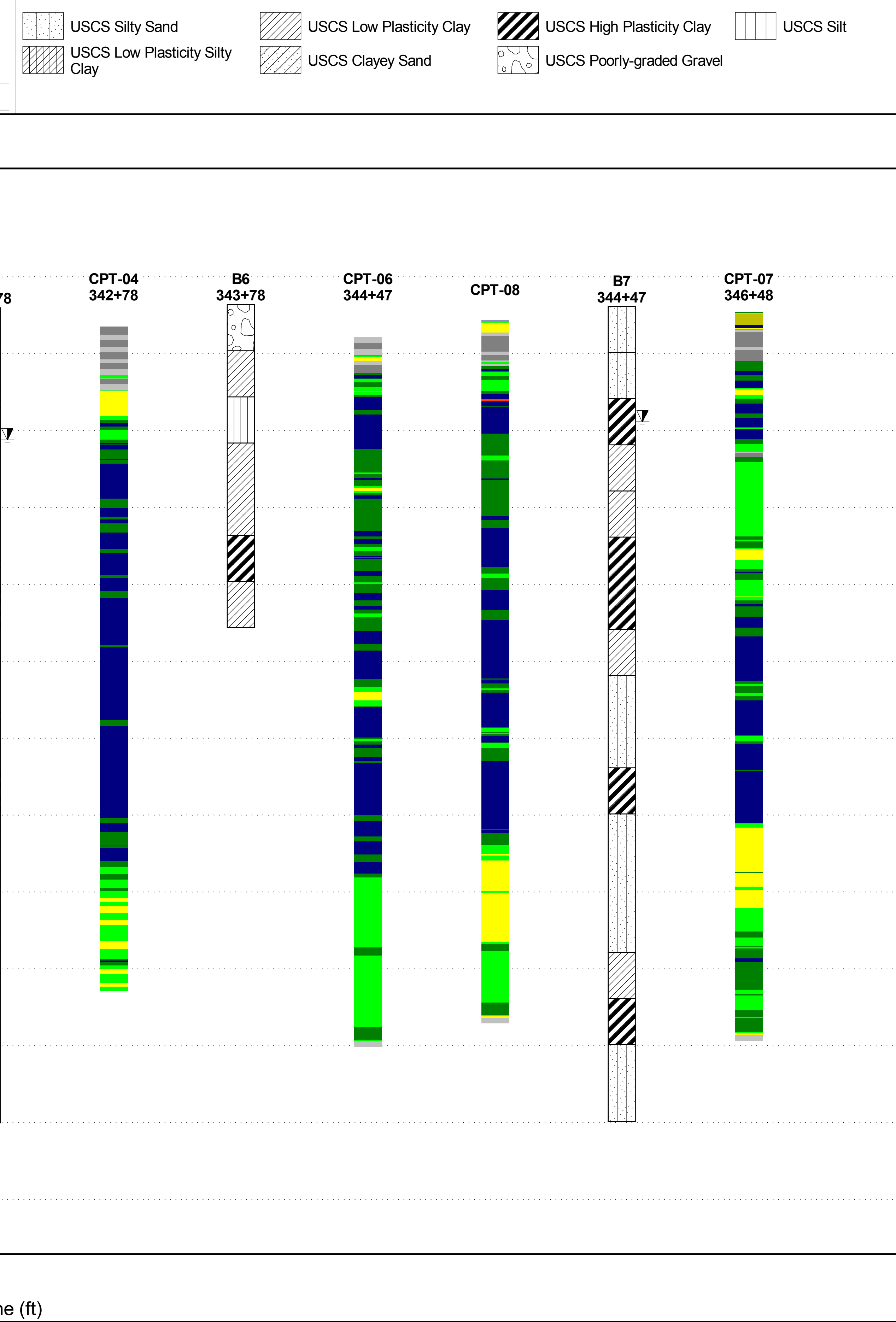
SUBSURFACE DIAGRAM

CLIENT

PROJECT NUMBER H.003496

PROJECT NAME I-49 North, Segment J - Cracked Pavement

PROJECT LOCATION Shreveport, LA



BORING NO. B1

DRILL RIG AND EQUIPMENT
DRILL RIG MODEL: Simco
DRILLING METHOD: Wet - Rotary
HOLE DIAMETER: 4"
SPT HAMMER / ETR:Automatic

DEPTH (Feet)	ELEVATION	SOIL TYPE AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	188.1	Very dense, gray, SILTY SAND, (SM) (soil cement)		27		NP	46.3	50/4"	1	D84
				25	38	16	57.5	12-16-18 (34)	4	D85
		Hard, gray, SANDY LEAN CLAY, (CL) -- w/soil cement								
		Very stiff, brown, LEAN CLAY WITH SAND, (CL) with iron oxide	121	22	45	21	76.6	2.37@6.66	M.S.	C86
10	178.1	Medium stiff, brown, FAT CLAY, (CH)	111	22	56	34	91.5	0.93@9.16	M.S.	C87
		Stiff to very stiff, gray, LEAN CLAY, (CL) with sand, with iron oxide	134	14	33	18	81.4	1.74@11.66	M.S.	C88
			129	22	45	25	91.2	2.51@14.16	M.S.	C89
		With sand	129	24	39	24	79.1	1.05@16.66	M.S.	C90
20	168.1									
		Medium stiff, gray, SANDY LEAN CLAY, (CL)	127	22	44	27	68.5	0.54@14.16	M.S.	C91
		Very stiff, gray, FAT CLAY, (CH) Org=6%	125	26	63	39	97.4	3.06@21.66	S/S	C92
			127	27	71	47	99.1	2.43@24.16	S/S	C93
30	158.1	Very stiff, gray, LEAN CLAY, (CL)	121	28	47	24	95.4	2.03@26.66	M.S.	C94
		Very stiff to hard, gray, FAT CLAY, (CH)	122	29	61	37	96.6	2.33@29.16	M.S.	C95
			123	25	59	32	98.8	4.55@30.83	S/S	C96
40	148.1	Very dense, gray, SILTY SAND, (SM)		31		NP	37.3	40-50	1	D97
				27		NP	19.3	50/5"	1	D98
		Bottom of hole at 47 feet Backfilled with cement bentonite slurry upon completion.								
50	138.1									
60	128.1									

PL NM LL — 10 20 30 40 50 60 70 80 90

BORING NO. B1	STATION: 340+89 ft	WATER LEVEL: No Water Level Taken ATD
LATITUDE: 32.58811	OFFSET: 71 R	
LONGITUDE: -93.83031	DATE TAKEN: 4/25/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID: FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

BORING NO. B2

DRILL RIG AND EQUIPMENT
DRILL RIG MODEL: Simco
DRILLING METHOD: Wet - Rotary
HOLE DIAMETER: 4"
SPT HAMMER / ETR:Automatic

DEPTH (Feet)	ELEVATION	SOIL TYPE AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	188.2	Very dense, gray, SILTY SAND, (SM) (soil cement)		16		NP	47.2	50/5"	1	D77
				29		NP	63.9	20-22-25 (47)	4	D78
		Dense, gray, SANDY SILT, (ML) -- w/soil cement								
		Very stiff, gray, SANDY LEAN CLAY, (CL)	128	12	25	9	60.3	2.37@5.83	M.S.	C79
10	178.2	Medium dense, gray, SANDY SILT, (ML) with iron oxide		15		NP	50.5	3-4-6 (10)	4	D80
				13		NP	55.5	4-5-8 (13)	4	D81
				15		NP	56.7	2-6-4 (10)	4	D82
		Medium dense, brown, SILTY SAND, (SM)		24		NP	25.1	6-7-11 (18)	4	D83
20	168.2									
		Bottom of hole at 21 feet Backfilled with cement bentonite slurry upon completion.								
30	158.2									
40	148.2									
50	138.2									
60	128.2									

BORING NO. B2	STATION: 341+89 ft	WATER LEVEL: No Water Level Taken ATD
LATITUDE: 32.58838	OFFSET: 69 R	
LONGITUDE: -93.83036	DATE TAKEN: 4/25/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID: FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

BORING NO. B3

DRILL RIG AND EQUIPMENT
DRILL RIG MODEL: Simco
DRILLING METHOD: Wet - Rotary
HOLE DIAMETER: 4"
SPT HAMMER / ETR:Automatic

DEPTH (Feet)	ELEVATION	SOIL TYPE AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	188.4	Dense to very dense, brown, SILTY SAND, (SM) -- w/stone		26		NP	33.9	50	1	D42
				30		NP	28.4	15-15-18 (33)	4	D43
		Stiff, brown, LEAN CLAY WITH SAND, (CL)	123	17	39	23	72.9	1.60@5.83	M.S.	C44
10	178.4	Very stiff, brown, SANDY SILTY CLAY, (CL-ML) with iron oxide	131	14	21	6	62.0	2.35@9.16	M.S.	C45
		Very stiff, brown, LEAN CLAY, (CL) Org = 5%	126	19	49	33	87.1	2.43@10.83	M.S.	C46
		Very stiff, brown, FAT CLAY, (CH) with iron oxide	129	26	62	41	94.0	2.17@14.16	M.S.	C47
		Medium stiff to stiff, gray and brown, LEAN CLAY, (CL)	124	28	45	26	89.3	0.66@15.83	M.S.	C48
20	168.4	With sand	128	26	36	19	77.0	1.10@19.16	M.S.	C49
		Stiff, tan, FAT CLAY, (CH) with iron oxide	129	26	60	39	96.0	1.74@20.83	M.S.	C50
		Medium stiff, brown, SANDY LEAN CLAY, (CL) with iron oxide	127	27	26	8	64.7	0.63@24.16	M.S.	C51
30	158.4	Medium stiff, brown, FAT CLAY, (CH) with iron oxide	121	29	59	38	94.5	0.90@25.83	M.S.	C52
		Stiff, brown, LEAN CLAY, (CL)	124	27	43	25	85.4	1.75@30.83	M.S.	C53
		Brown, CLAYEY SAND, (SC)	126	25	25	10	45.7	0.56@30.83	M.S.	C54
40	148.4	Medium dense, brown, SILTY SAND, (SM)		22		NP	27.9	5-8-10 (18)	4	D55
				19		NP	28.9	7-9-12 (21)	4	D56
		Very dense, brown and grayish, SILTY SAND, (SM)		23		NP	17.3	20-32-48 (80)	2	D57
50	138.4			25		NP	26.1	44-48-8/1"	2	D58
		Bottom of hole at 53 feet Backfilled with cement bentonite slurry upon completion.								
60	128.4									

BORING NO. B3	STATION: 342+78 ft	WATER LEVEL: No Water Level Taken ATD
LATITUDE: 32.58861	OFFSET: 52 R	
LONGITUDE: -93.83045	DATE TAKEN: 4/19/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID: FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

STANDARD ABBREVIATIONS & DEFINITIONS

SM CL CH

N.P. = Non-Plastic
ORG. = Organic

FAILURE MODE:
M.S. = Multiple Shear SL. = Slump
S/S = Slickensides YLD. = Yield
V.S. = Vertical Shear 60 S. = Shear Angle

SPT TERMINATION, AASHTO T 206
1 = 7.2.1 - 50 Blows Within A 6" Interval
2 = 7.2.2 - 100 Blows Total
3 = 7.2.3 - No Advancement for 10 Blows
4 = 7.2.4 - Sampler Driven the Entire 18"
5 = Non-standard
6 = Weight of Rods (WOR)
7 = Weight of Hammer (WOH)

SOIL PROPERTIES

⊗ = SOIL TYPE nomenclature is based on ASTM D 2487
WET DENSITY = Wet density of in-place soil, (pounds per cu. ft.) determined by AASHTO T 208.
MOISTURE CONTENT = Moisture Content of in-place soil, expressed as a percentage of the dry weight of the soil, (%), determined by DOTD TR 403, Method B.
LIQUID LIMIT & PLASTICITY INDEX = Atterberg limits and indices, DOTD TR 428
SPT = Standard Penetration Test, AASHTO T 206, number of blows per each 6 inch increment, unless amount of penetration is shown
UU = Unconsolidated Undrained triaxial test, AASHTO T 296, compressive strength (tons per sq. ft.), of one specimen confined at noted pressure (pounds per sq. in.)
C = Soil cohesion (tons per sq. ft.)
ϕ = Soil angle of internal friction (degrees)
△ = Unconsolidated Undrained triaxial test, AASHTO T 296, three specimens, (c - ϕ).
+ = Consolidated drained direct shear test, AASHTO T 236, (c - ϕ).
* = Hydrometer test performed

MISCELLANEOUS:

C-3 = Location and Identification of thin-walled tube sample, AASHTO T 207
C-3 = Location and Identification of thin-walled tube sample, AASHTO T 207, with a portion of the sample saved for consolidation testing
D-3 = Location and Identification of SPT sample, AASHTO T 206
G-3 = Location and Identification of sample recovered using an auger as per ASTM D1452
G-3 = Grab Sample, unable to recover undisturbed sample for strength testing and material retained for classification.
NO RECV. = No Recovery, unable to recover sample for testing or classification.
DIST. = Disturbed sample recovered with thin-walled tube sampler.
▽ = Water Table depth below ground surface at time of drilling
▼ = Water Table depth below ground surface after drilling as noted
ETR = Energy Transfer Ratio determined according to ASTM D4633

CORRELATION OF PENETRATION RESISTANCE AND SOIL PROPERTIES

SOIL		DESIGNATION	"N" (blows per ft.)
SAND AND SILT	RELATIVE DENSITY	VERY LOOSE	LESS THAN 4
		LOOSE	4 - 10
		MEDIUM DENSE	10 - 30
		DENSE	30 - 50
		VERY DENSE	OVER 50
CLAY	CONSISTENCY	VERY SOFT	LESS THAN 2
		SOFT	2 - 4
		MEDIUM STIFF	4 - 8
		STIFF	8 - 15
		VERY STIFF	15 - 30
		HARD	OVER 30

THIS SIGNATURE AND SEAL IS AFFIXED TO THIS DRAWING AS CERTIFICATION THAT THE LABORATORY TESTING AND ANALYSIS WAS PERFORMED ACCORDING TO THE LISTED PROCEDURES. NO DESIGN COMPUTATIONS WERE PERFORMED OR REVIEWED BY ME.

NO.	DATE	LOG UPDATE DESCRIPTION	BY

SHEET NUMBER

PARISH: CADDO
CONTROL SECTION
STATE PROJECT

DESIGNED: B. FERNANDEZ
CHECKED: J. RAUSER
DETAILED
CHECKED: JGR
DATE: 6/1/2017
SHEET ___ OF ___

NO.	DATE	REVISION DESCRIPTION	BY

STATE OF LOUISIANA · UNION · JUSTICE · CONFIDENCE

GEOTECHNICAL EXPLORATION LOGS

I-49 North, Segment J - Cracked Pavement

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

Report Form: LADOTD 2016 - ENGLISH - BEN COMPUTER Project File: T:\GEOTECHNICAL PROJECTS_H.003000 THRU H.003999\H.003496 I-49 N (MLK TO LA 1) SEG. J\FAILED PAVEMENT\CPT\GINT\H.003496 - CRACKED PAVEMENT.GPJ

GEOTECHNICAL EXPLORATION LOGS

I-49 North, Segment J - Cracked Pavement

CADDO

DRILL RIG AND EQUIPMENT

DRILL RIG MODEL: Simco
DRILLING METHOD: Wet - Rotary
HOLE DIAMETER: 4"
SPT HAMMER / ETR:Automatic

BORING NO. B4

DEPTH (Feet)	ELEVATION	SOIL TYPE AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	188.0	Dense to very dense, brown, SILTY SAND, (SM) -- w/stone		31		NP	47.2	48-32-20/5"	2	D1
				28		NP	43.9	17-14-19 (33)	4	D2
		Stiff, brown, SANDY SILTY CLAY, (CL-ML) with iron oxide	131	25	24	7	66.9	1.57@5.83	M.S.	C3
10	178.0	Brown, SANDY SILT, (ML)	138	19		NP	65.5	4.05@9.16	M.S.	C4
		Stiff to very stiff, brown, LEAN CLAY WITH SAND, (CL) with iron oxide, Org=4%	132	20	35	19	71.5	2.16@10.83	M.S.	C5
			123	24	39	21	76.6	1.17@14.16	M.S.	C6
20	168.0	Stiff, brown, FAT CLAY, (CH) with iron oxide	125	28	66	43	97.0	1.39@15.83	M.S.	C7
		Medium stiff to stiff, brown, LEAN CLAY WITH SAND, (CL) Org = 4%	126	26	33	17	71.3	0.74@19.16	M.S.	C8
			133	21	36	19	73.2	0.80@20.83	M.S.	C9
			129	25	28	14	71.4	1.54@24.16	M.S.	C10
30	158.0	Stiff, brown, FAT CLAY, (CH)	124	27	57	36	91.7	1.55@25.83	M.S.	C11
		Medium dense, brown with grayish, SILTY SAND, (SM)		27		NP	27.5	5-6-10 (16)	4	D12
				27		NP	30.2	7-9-12 (21)	4	D13
40	148.0			21		NP	31.2	6-8-11 (19)	4	D14
				20		NP	32.0	6-9-12 (21)	4	D15
		Very dense, gray, SILTY SAND, (SM)		27		NP	20.2	20-50	1	D16
50	138.0	Very dense, gray, SANDY SILT, (ML)		30		NP	55.6	25-40-35/3"	2	D17
60	128.0	Bottom of hole at 53 feet Backfilled with cement bentonite slurry upon completion.								

BORING NO. B4	STATION: 342+78 ft	WATER LEVEL:
LATITUDE: 32.58862	OFFSET: 78 R	8.6' 18 hours after drilling
LONGITUDE: -93.83038	DATE TAKEN: 4/18/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID: FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

BORING NO. B5

DEPTH (Feet)	ELEVATION	SOIL TYPE AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	187.0	Stiff, gray, LEAN CLAY WITH SAND, (CL) with iron oxide	129	19	39	20	81.1	1.76@1.66	M.S.	C59
		Hard, tan, SANDY LEAN CLAY, (CL) with iron oxide, Org = 5%		16	32	16	65.1			C60
		Stiff, gray, LEAN CLAY WITH SAND, (CL) with iron oxide	132	18	33	16	78.3	1.92@6.66	M.S.	C61
10	177.0	Very stiff, gray, SANDY LEAN CLAY, (CL) with iron oxide	130	18	38	21	60.0	2.64@9.16	M.S.	C62
		Stiff, gray, LEAN CLAY, (CL) with iron oxide	124	27	47	27	88.5	1.32@10.83	M.S.	C63
		Stiff, gray, FAT CLAY, (CH)	124	27	57	33	99.0	1.87@14.16	M.S.	C64
20	167.0		126	27	65	43	96.6	1.32@15.83	M.S.	C65
		Stiff, gray, LEAN CLAY WITH SAND, (CL)	127	27	45	27	81.4	1.23@19.16	M.S.	C66
		Stiff, gray, FAT CLAY, (CH)	125	32	65	43	97.3	1.34@20.83	M.S.	C67
		Medium stiff, brown, LEAN CLAY WITH SAND, (CL)	126	28	35	17	75.7	0.88@24.16	M.S.	C68
30	157.0	Stiff to very stiff, brown and grayish, SANDY LEAN CLAY, (CL)	123	25	26	8	53.6	1.31@25.83	M.S.	C69
			125	30	30	14	64.0	1.10@29.16	M.S.	C70
			129	26	32	17	58.5	2.27@30.83	M.S.	C71
40	147.0	Loose to medium dense, gray, SILTY SAND, (SM)		21		NP	25.9	2-2-4 (6)	4	D72
				15		NP	25.7	4-6-12 (18)	4	D73
				35		NP	21.0	7-8-9 (17)	4	D74
50	137.0	Hard, gray, LEAN CLAY WITH SAND, (CL)		24	33	14	80.0	20-35-45/5"	2	D75
				24	28	11	76.1	37-50/5"	1	D76
60	127.0	Bottom of hole at 56 feet Backfilled with cement bentonite slurry upon completion.								

BORING NO. B5	STATION: 342+78 ft	WATER LEVEL: No Water Level Taken ATD
LATITUDE: 32.58862	OFFSET: 93 R	
LONGITUDE: -93.83033	DATE TAKEN: 4/25/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID: FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

BORING NO. B6

DEPTH (Feet)	ELEVATION	SOIL TYPE AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	188.2	Very dense, brown, POORLY GRADED GRAVEL WITH SAND, (GP)		19		NP	3.4	50/5"	1	D35
		Very stiff, brown, LEAN CLAY, (CL)	131	34	45	25	86.0	2.32@4.16	M.S.	C36
		Brown, SANDY SILT, (ML)	130	12		NP	51.1	0.76@5.83	M.S.	C37
10	178.2	Very stiff, brown, SANDY LEAN CLAY, (CL)	132	21	38	22	50.7	3.64@9.16	M.S.	C38
			125	20	38	21	63.9	2.09@10.83	M.S.	C39
		Stiff, gray, FAT CLAY, (CH)	127	24	61	40	95.3	1.28@14.16	M.S.	C40
20	168.2	Stiff, gray, LEAN CLAY, (CL)	124	30	39	22	86.3	1.09@15.83	M.S.	C41
		Bottom of hole at 21 feet Backfilled with cement bentonite slurry upon completion.								
30	158.2									
40	148.2									
50	138.2									
60	128.2									

BORING NO. B6	STATION: 343+78 ft	WATER LEVEL: No Water Level Taken ATD
LATITUDE: 32.58888	OFFSET: 69 R	
LONGITUDE: -93.83045	DATE TAKEN: 4/19/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID: FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

STANDARD ABBREVIATIONS & DEFINITIONS

SM — ML — CH — CL-ML — CL

N.P. = Non-Plastic
ORG. = Organic

FAILURE MODE:
M.S. = Multiple Shear SL. = Slump
S/S = Slickensides YLD. = Yield
V.S. = Vertical Shear 60 S. = Shear Angle

SPT TERMINATION, AASHTO T 206
1 = 7.2.1 - 50 Blows Within A 6" Interval
2 = 7.2.2 - 100 Blows Total
3 = 7.2.3 - No Advancement for 10 Blows
4 = 7.2.4 - Sampler Driven the Entire 18"
5 = Non-standard
6 = Weight of Rods (WOR)
7 = Weight of Hammer (WOH)

SOIL PROPERTIES

⊗ = SOIL TYPE nomenclature is based on ASTM D 2487
WET DENSITY = Wet density of in-place soil, (pounds per cu. ft.) determined by AASHTO T 208.
MOISTURE CONTENT = Moisture Content of in-place soil, expressed as a percentage of the dry weight of the soil, (%), determined by DOTD TR 403, Method B.
LIQUID LIMIT & PLASTICITY INDEX = Atterberg limits and indices, DOTD TR 428
SPT = Standard Penetration Test, AASHTO T 206, number of blows per each 6 inch increment, unless amount of penetration is shown
UU = Unconsolidated Undrained triaxial test, AASHTO T 296, compressive strength (tons per sq. ft.), of one specimen confined at noted pressure (pounds per sq. in.)
C = Soil cohesion (tons per sq. ft.)
ϕ = Soil angle of internal friction (degrees)
△ = Unconsolidated Undrained triaxial test, AASHTO T 296, three specimens, (c - ϕ).
+ = Consolidated drained direct shear test, AASHTO T 236, (c - ϕ).
* = Hydrometer test performed

MISCELLANEOUS:

C-3 = Location and Identification of thin-walled tube sample, AASHTO T 207
C-3 = Location and Identification of thin-walled tube sample, AASHTO T 207, with a portion of the sample saved for consolidation testing
D-3 = Location and Identification of SPT sample, AASHTO T 206
G-3 = Location and Identification of sample recovered using an auger as per ASTM D1452
G-3 = Grab Sample, unable to recover undisturbed sample for strength testing and material retained for classification.
NO RECV. = No Recovery, unable to recover sample for testing or classification.
DIST. = Disturbed sample recovered with thin-walled tube sampler.
▽ = Water Table depth below ground surface at time of drilling
▼ = Water Table depth below ground surface after drilling as noted
ETR = = Energy Transfer Ratio determined according to ASTM D4633

CORRELATION OF PENETRATION RESISTANCE AND SOIL PROPERTIES

SOIL		DESIGNATION	"N" (blows per ft.)
SAND AND SILT	RELATIVE DENSITY	VERY LOOSE	LESS THAN 4
		LOOSE	4 - 10
		MEDIUM DENSE	10 - 30
		DENSE	30 - 50
		VERY DENSE	OVER 50
CLAY	CONSISTENCY	VERY SOFT	LESS THAN 2
		SOFT	2 - 4
		MEDIUM STIFF	4 - 8
		STIFF	8 - 15
		VERY STIFF	15 - 30
		HARD	OVER 30

THIS SIGNATURE AND SEAL IS AFFIXED TO THIS DRAWING AS CERTIFICATION THAT THE LABORATORY TESTING AND ANALYSIS WAS PERFORMED ACCORDING TO THE LISTED PROCEDURES. NO DESIGN COMPUTATIONS WERE PERFORMED OR REVIEWED BY ME.

NO.	DATE	LOG UPDATE DESCRIPTION	BY

SHEET NUMBER

PARISH: CADDO | CONTROL SECTION | STATE PROJECT

DESIGNED: B. FERNANDEZ
CHECKED: J. RAUSER
DETAILED
CHECKED: JGR
DATE: 6/1/2017
SHEET ___ OF ___

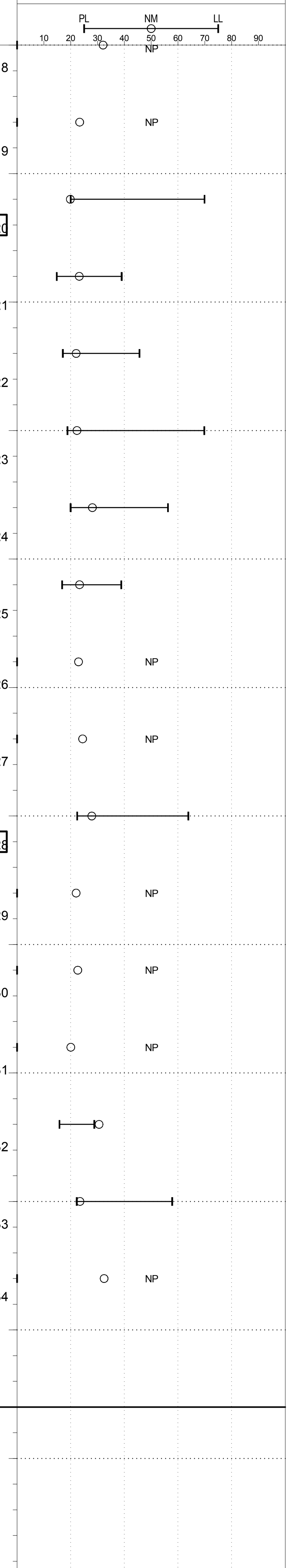
NO.	DATE	REVISION DESCRIPTION	BY

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

DRILL RIG AND EQUIPMENT

DRILL RIG MODEL: Simco
DRILLING METHOD: Wet - Rotary
HOLE DIAMETER: 4"
SPT HAMMER / ETR: Automatic

DEPTH (Feet)	ELEVATION	GRAPHIC	SOIL TYPE ⊗ AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200	SPT or UU	FAILURE MODE/ SPT TERMINATION	SAMPLE TYPE NUMBER
0	188.1	SM	Dense, brown, SILTY SAND, (SM) -- w/stone		32		NP	29.8	47-24-15 (39)	4	D18
		SM	Brown, SILTY SAND, (SM)	121	23		NP	42.1	1.94@4.16	M.S.	C19
▼		CH	Medium stiff, brown, FAT CLAY, (CH) Org=10%	127	20	70	50	86.6	0.85@5.83	M.S.	C20
10	178.1	CL	Very stiff, brown, SANDY LEAN CLAY, (CL) Org=4%	126	23	39	24	65.5	2.43@9.16	M.S.	C21
		CL	Stiff, brown, LEAN CLAY WITH SAND, (CL) Org=7%	125	22	46	29	83.6	1.27@10.83	M.S.	C22
		CH	Stiff, brown, FAT CLAY, (CH)	124	22	70	51	94.9	1.82@14.16	M.S.	C23
20	168.1			126	28	56	36	92.9	1.38@15.83	M.S.	C24
		CL	Stiff, brown, LEAN CLAY, (CL)	128	23	39	22	85.3	1.61@19.16	M.S.	C25
		SM	Medium dense, brown, SILTY SAND, (SM)		23		NP	36.6	6-6-8 (14)	4	D26
					24		NP	33.4	9-8-6 (14)	4	D27
30	158.1	CH	Stiff, brown, FAT CLAY, (CH) with iron oxide	121	28	64	42	99.0	1.12@25.83	M.S.	C28
		SM	Medium dense, brown and grayish, SILTY SAND, (SM)		22		NP	28.5	6-7-10 (17)	4	D29
					23		NP	37.1	6-9-12 (21)	4	D30
40	148.1				20		NP	24.2	4-4-7 (11)	4	D31
		CL	Hard, gray, SANDY LEAN CLAY, (CL) with iron oxide		31	29	13	50.1	10-17-21 (38)	4	D32
		CH	Hard, gray, FAT CLAY, (CH)		23	58	36	98.7	15-22-25 (47)	4	D33
50	138.1	SM	Very dense, gray, SILTY SAND, (SM)		32		NP	40.9	35-48-17/2"	2	D34
60	128.1		Bottom of hole at 53 feet Backfilled with cement bentonite slurry upon completion.								



BORING NO. B7		STATION: 344+47 ft	WATER LEVEL:
LATITUDE: 32.58909		OFFSET: 74 R	7.5' 14 hours after drilling
LONGITUDE: -93.83047		DATE TAKEN: 4/18/2017	BACKFILL METHOD: Cement Bentonite Slurry
LRS ID:	FIELD BOOK:	STRUCTURE NO.:	DRILLER: Mike Anderson

STANDARD ABBREVIATIONS & DEFINITIONS

SM
CH
CL

N.P. = Non-Plastic
ORG. = Organic

FAILURE MODE:
M.S. = Multiple Shear SL. = Slump
S/S = Slickensides YLD. = Yield
V.S. = Vertical Shear 60 S. = Shear Angle

SPT TERMINATION, AASHTO T 206
1 = 7.2.1 - 50 Blows Within A 6" Interval
2 = 7.2.2 - 100 Blows Total
3 = 7.2.3 - No Advancement for 10 Blows
4 = 7.2.4 - Sampler Driven the Entire 18"
5 = Non-standard
6 = Weight of Rods (WOR)
7 = Weight of Hammer (WOH)

SOIL PROPERTIES

⊗ = SOIL TYPE nomenclature is based on ASTM D 2487
WET DENSITY = Wet density of in-place soil, (pounds per cu. ft.) determined by AASHTO T 208.
MOISTURE CONTENT = Moisture Content of in-place soil, expressed as a percentage of the dry weight of the soil, (%), determined by DOTD TR 403, Method B.
LIQUID LIMIT & PLASTICITY INDEX = Atterberg limits and indices, DOTD TR 428
SPT = Standard Penetration Test, AASHTO T 206, number of blows per each 6 inch increment, unless amount of penetration is shown
UU = Unconsolidated Undrained triaxial test, AASHTO T 296, compressive strength (tons per sq. ft.), of one specimen confined at noted pressure (pounds per sq. in.)
C = Soil cohesion (tons per sq. ft.)
ϕ = Soil angle of internal friction (degrees)
△ = Unconsolidated Undrained triaxial test, AASHTO T 296, three specimens, (c - ϕ).
\+ = Consolidated drained direct shear test, AASHTO T 236, (c - ϕ).
* = Hydrometer test performed

MISCELLANEOUS:

C-3 = Location and Identification of thin-walled tube sample, AASHTO T 207
C-3 = Location and Identification of thin-walled tube sample, AASHTO T 207, with a portion of the sample saved for consolidation testing
D-3 = Location and Identification of SPT sample, AASHTO T 206
G-3 = Location and Identification of sample recovered using an auger as per ASTM D1452
G-3 = Grab Sample, unable to recover undisturbed sample for strength testing and material retained for classification.
NO RECV. = No Recovery, unable to recover sample for testing or classification.
DIST. = Disturbed sample recovered with thin-walled tube sampler.
▽ = Water Table depth below ground surface at time of drilling
▼ = Water Table depth below ground surface after drilling as noted
ETR = = Energy Transfer Ratio determined according to ASTM D4633

CORRELATION OF PENETRATION RESISTANCE AND SOIL PROPERTIES

SOIL		DESIGNATION	"N" (blows per ft.)
SAND AND SILT	RELATIVE DENSITY	VERY LOOSE	LESS THAN 4
		LOOSE	4 - 10
		MEDIUM DENSE	10 - 30
		DENSE	30 - 50
		VERY DENSE	OVER 50
CLAY	CONSISTENCY	VERY SOFT	LESS THAN 2
		SOFT	2 - 4
		MEDIUM STIFF	4 - 8
		STIFF	8 - 15
		VERY STIFF	15 - 30
		HARD	OVER 30

THIS SIGNATURE AND SEAL IS AFFIXED TO THIS DRAWING AS CERTIFICATION THAT THE LABORATORY TESTING AND ANALYSIS WAS PERFORMED ACCORDING TO THE LISTED PROCEDURES. NO DESIGN COMPUTATIONS WERE PERFORMED OR REVIEWED BY ME.

NO.	DATE	LOG UPDATE DESCRIPTION	BY

SHEET NUMBER

PARISH: CADDO
CONTROL SECTION
STATE PROJECT

DESIGNED B. FERNANDEZ
CHECKED J. RAUSER
DETAILED JGR
CHECKED
DATE 6/1/2017
SHEET ___ OF ___

NO.	DATE	REVISION DESCRIPTION	BY

GEOTECHNICAL EXPLORATION LOGS

I-49 North, Segment J - Cracked Pavement

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

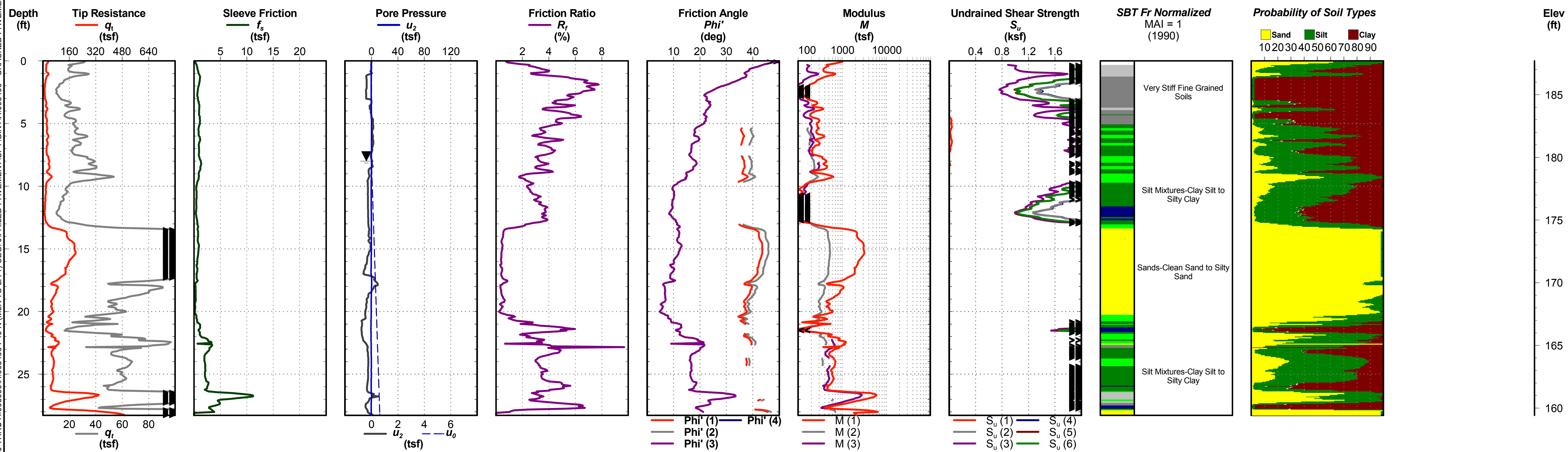
Cone Penetration Test

CPT-01

Date: Apr. 19, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 761859.1
Easting: 2871124.0
Elevation: 187.7

Elevation: 187.7
Water Depth: 8
Total Depth: 28.3 ft



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Electronic File Name: CP1.cpt

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

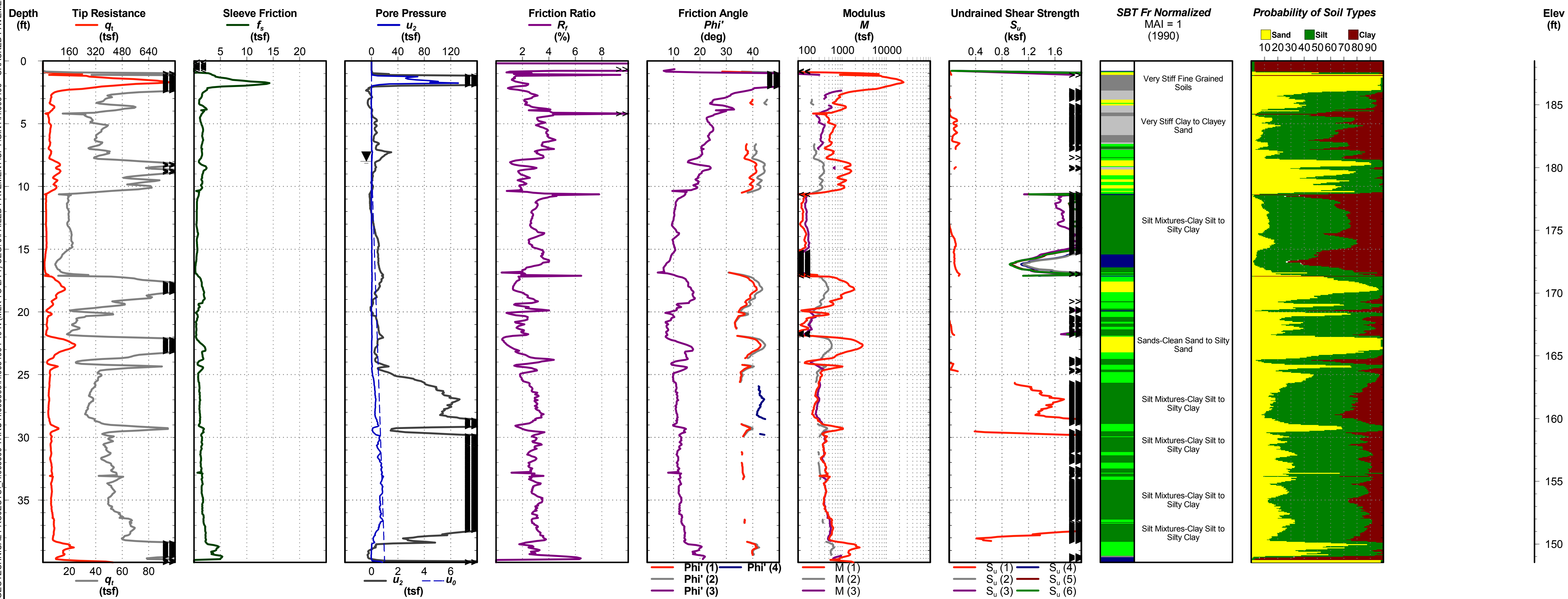
Cone Penetration Test

CPT-02

Date: Apr. 19, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 762114.6
Easting: 2871062.4
Elevation: 188.5

Elevation: 188.5
Water Depth: 8
Total Depth: 40.0 ft



Electronic File Name: CP2.cpt

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

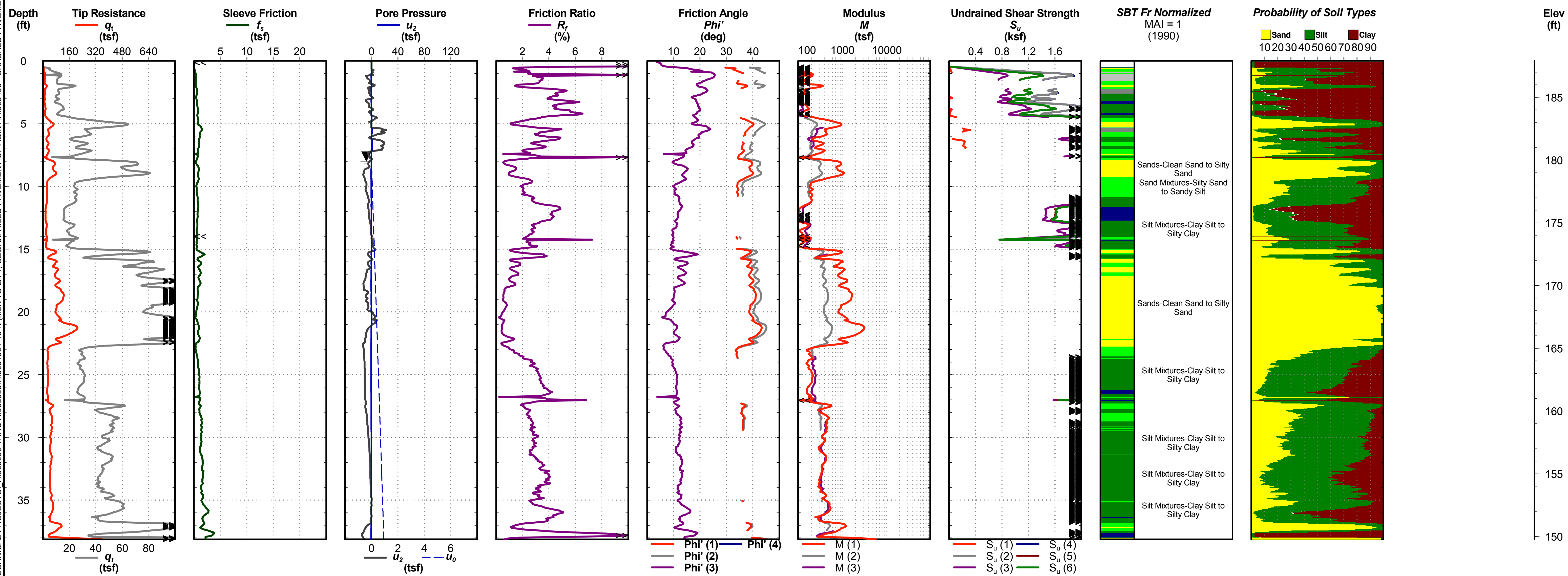
Cone Penetration Test

CPT-03

Date: Apr. 19, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 762125.0
Easting: 2871105.7
Elevation: 187.9

Elevation: 187.9
Water Depth: 8
Total Depth: 38.1 ft



Electronic File Name: CP3.cpt

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

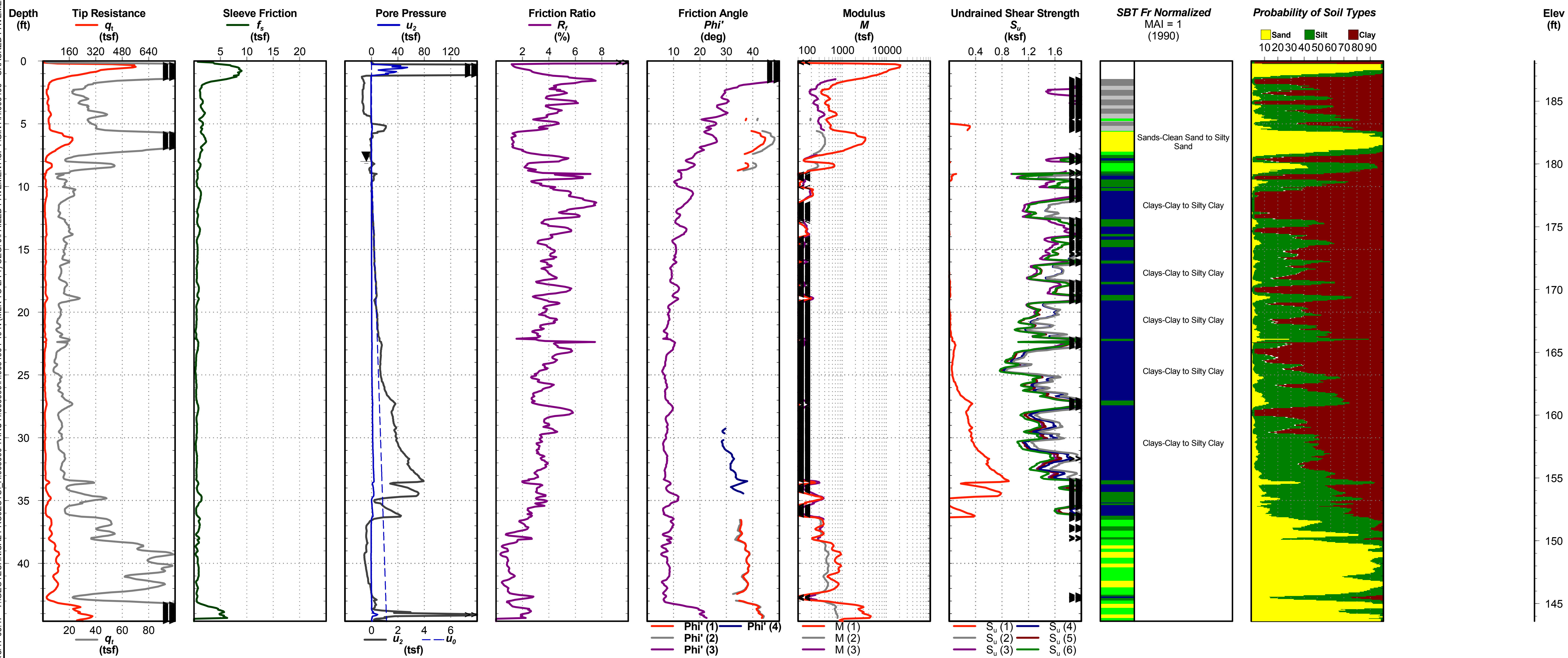
Cone Penetration Test

CPT-04

Date: Apr. 18, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 762391.6
Easting: 2871025.8
Elevation: 188.2

Elevation: 188.2
Water Depth: 8
Total Depth: 44.6 ft



Electronic File Name: excel import.xls

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

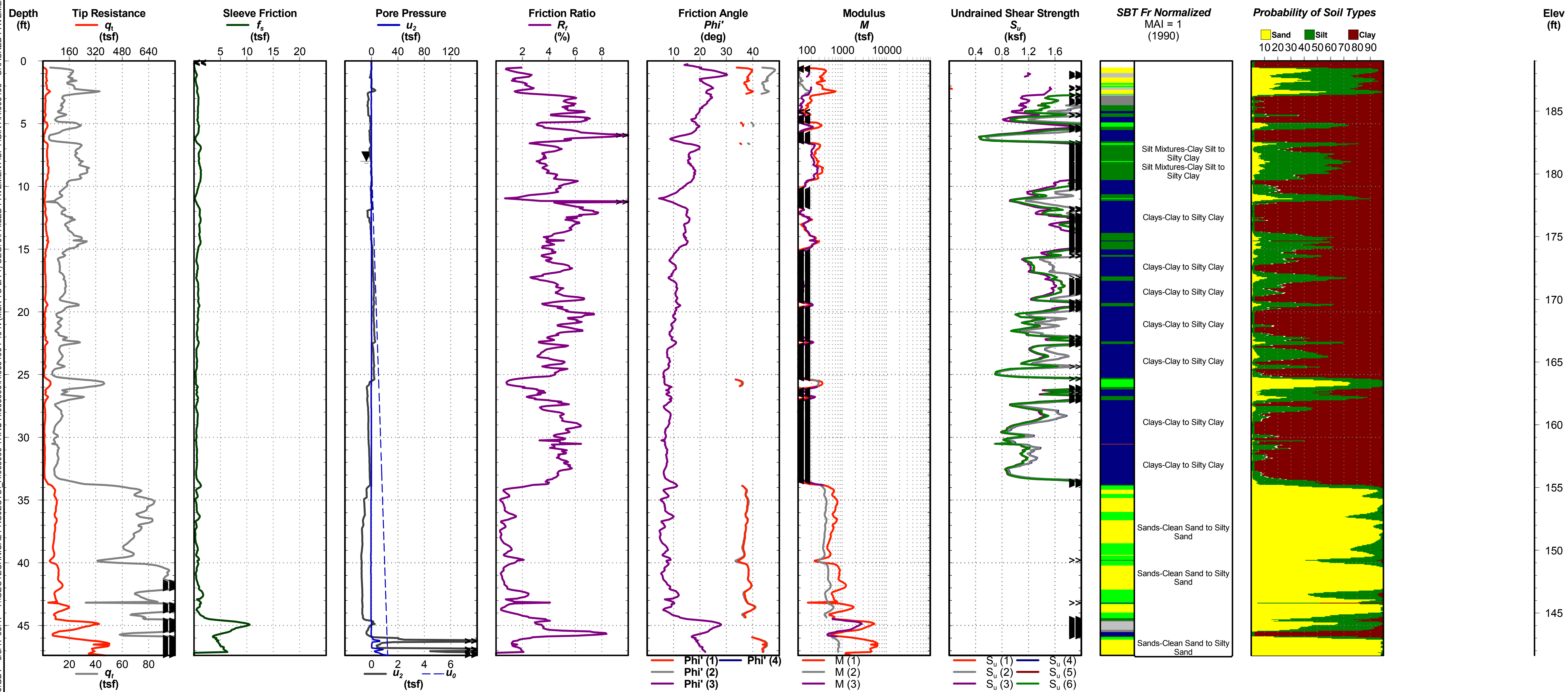
Cone Penetration Test

CPT-05

Date: Apr. 19, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 762296.8
Easting: 2871040.0
Elevation: 189.0

Elevation: 189.0
Water Depth: 8
Total Depth: 47.4 ft



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Electronic File Name: CP5.cpt

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

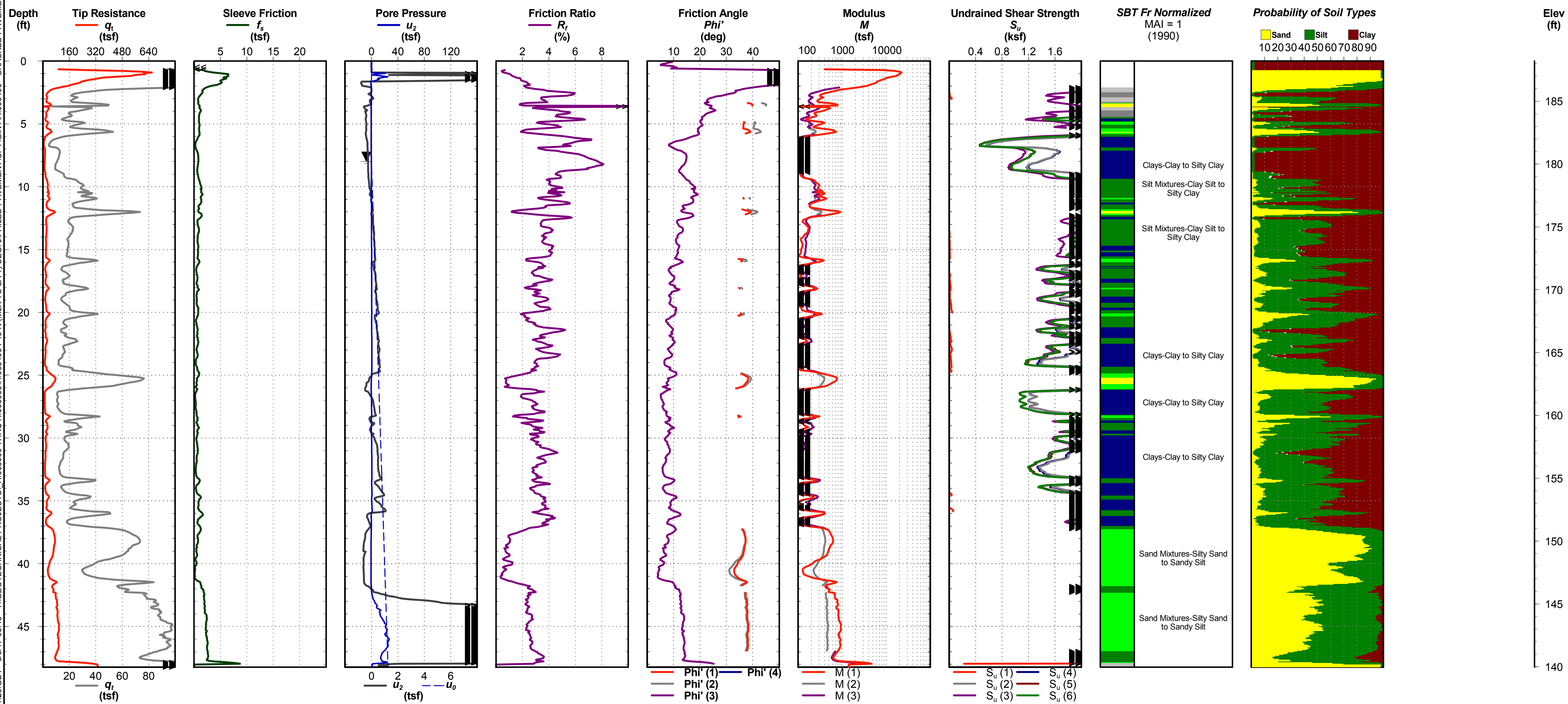
Cone Penetration Test

CPT-06

Date: Apr. 19, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 762467.9
Easting: 2871032.9
Elevation: 188.2

Elevation: 188.2
Water Depth: 8
Total Depth: 48.2 ft



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Electronic File Name: CPT6.cpt

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

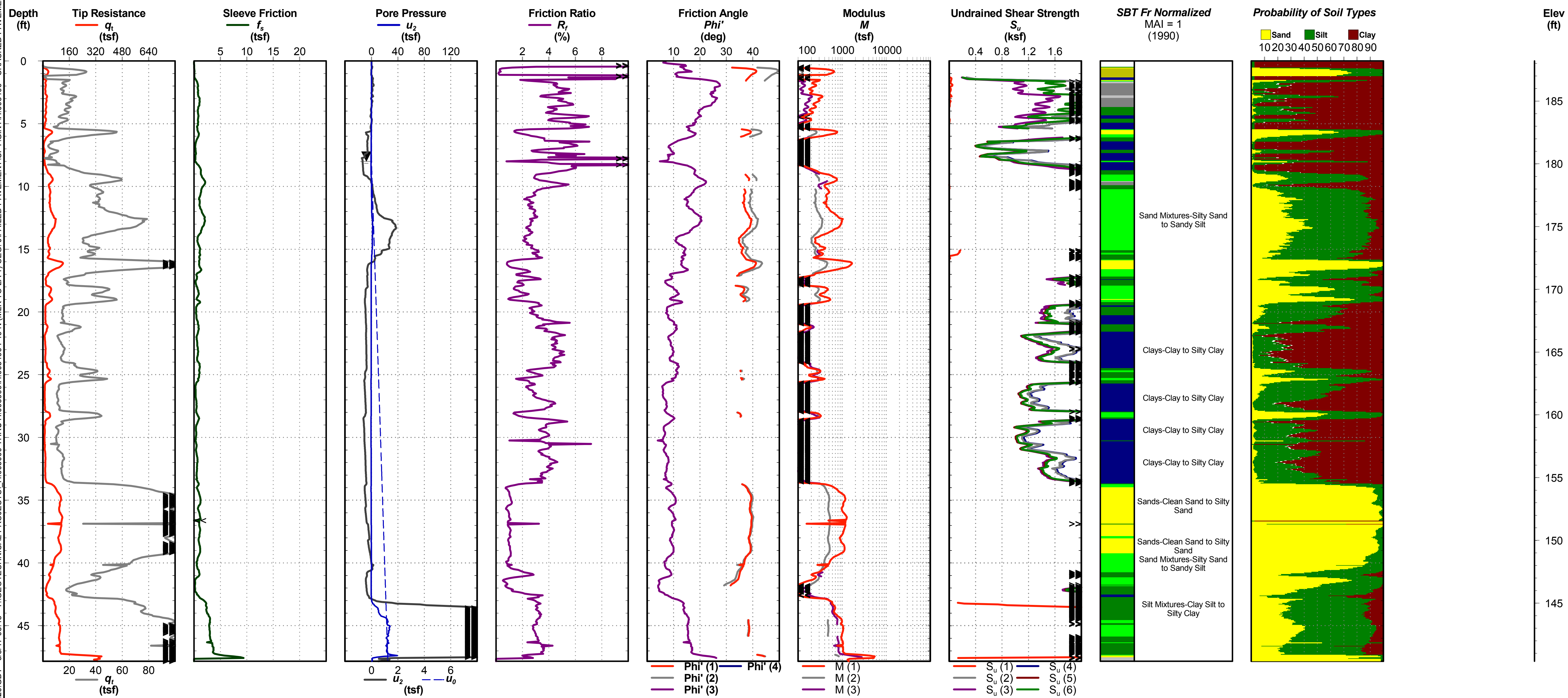
Cone Penetration Test

CPT-07

Date: Apr. 19, 2017 Project No: H.003496
Operator:
Drilling Agency:

Northing: 762685.6
Easting: 2871084.8
Elevation: 188.2

Elevation: 188.2
Water Depth: 8
Total Depth: 47.8 ft



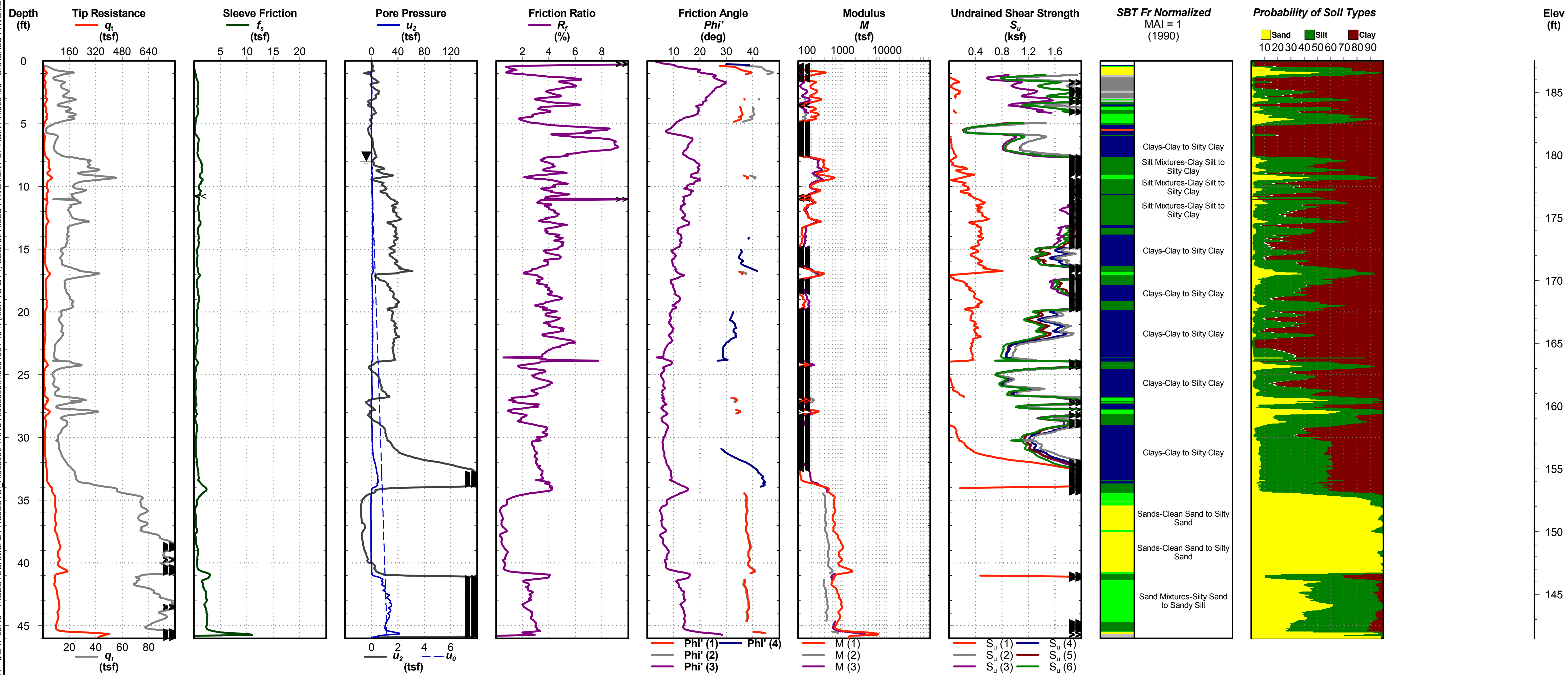
Electronic File Name: CP7.cpt

I-49 North, Segment J - Cracked Pavement
Shreveport, LA

Cone Penetration Test

CPT-08

Date:	Apr. 25, 2017 Project No: H.003496	**Northing:**	762471.7	**Elevation:**	187.5
Operator:		**Easting:**	2871017.5	**Water Depth:**	8
Drilling Agency:		**Elevation:**	187.5	**Total Depth:**	46.0 ft



Electronic File Name: CPT8.cpt

SUBSURFACE DIAGRAM

LADOTD
1201 Capitol Access Road
Baton Rouge, LA 70804

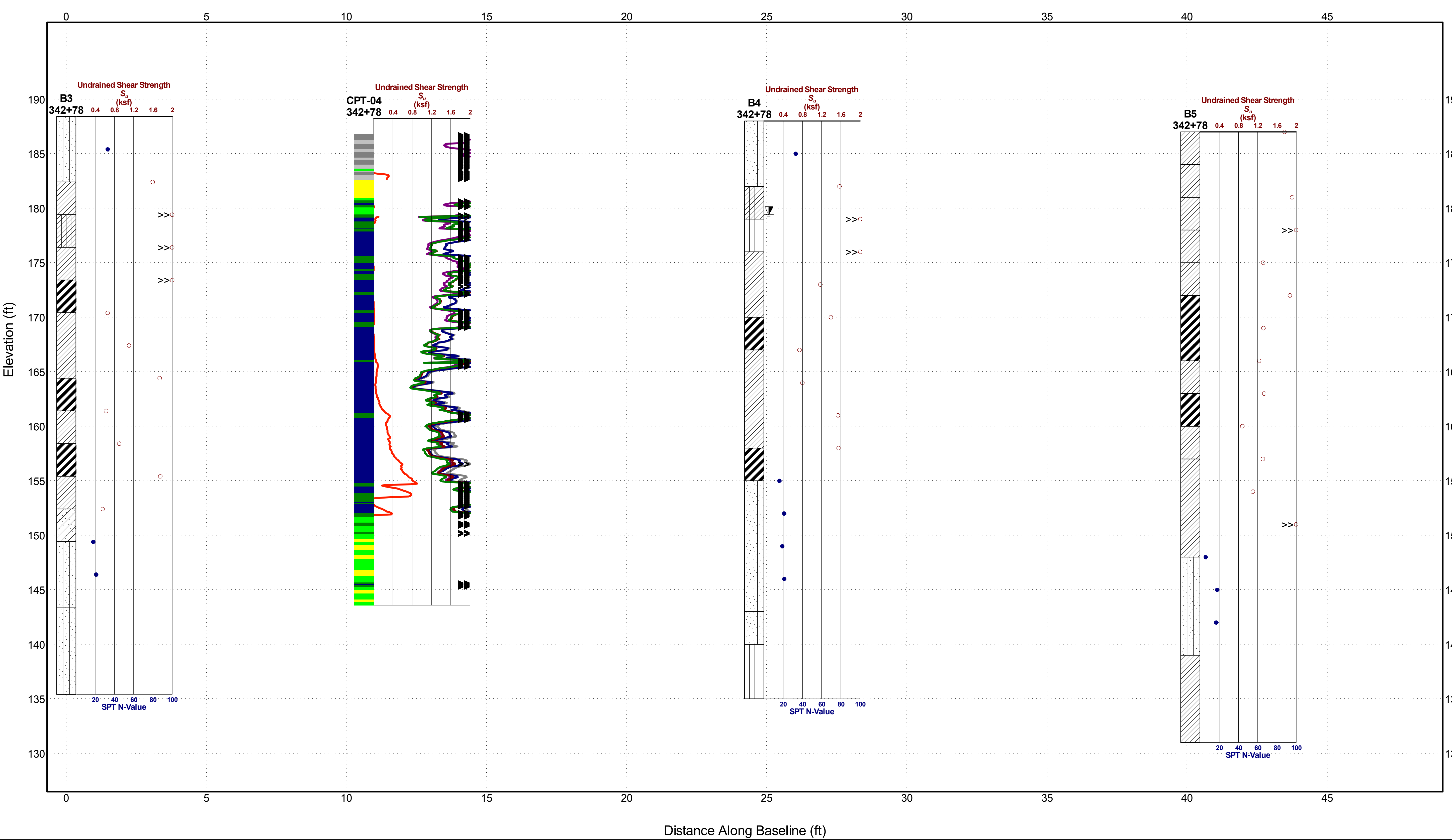
CLIENT ____

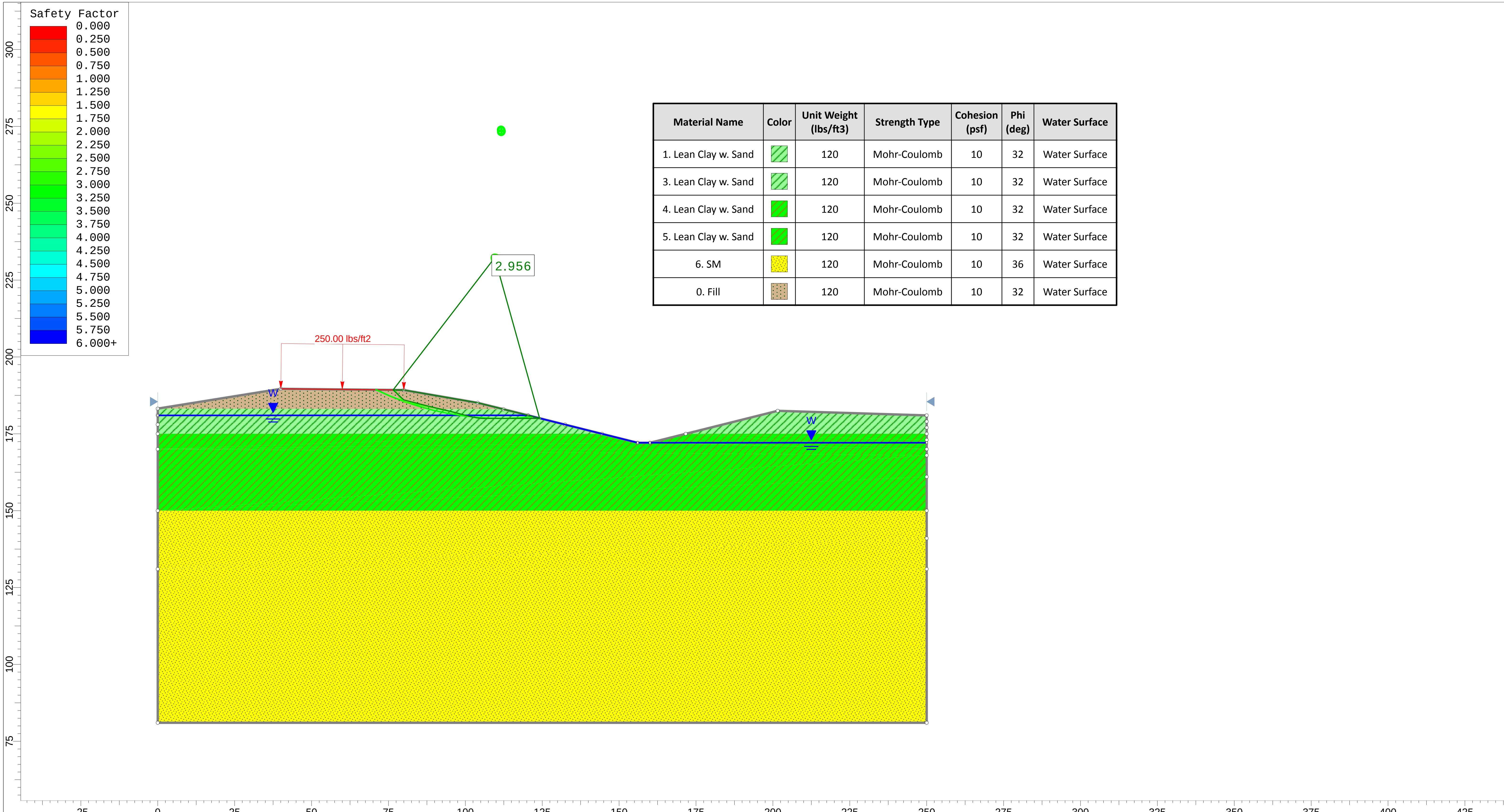
PROJECT NUMBER H.003496

PROJECT NAME I-49 North, Segment J - Cracked Pavement

PROJECT LOCATION Shreveport, LA

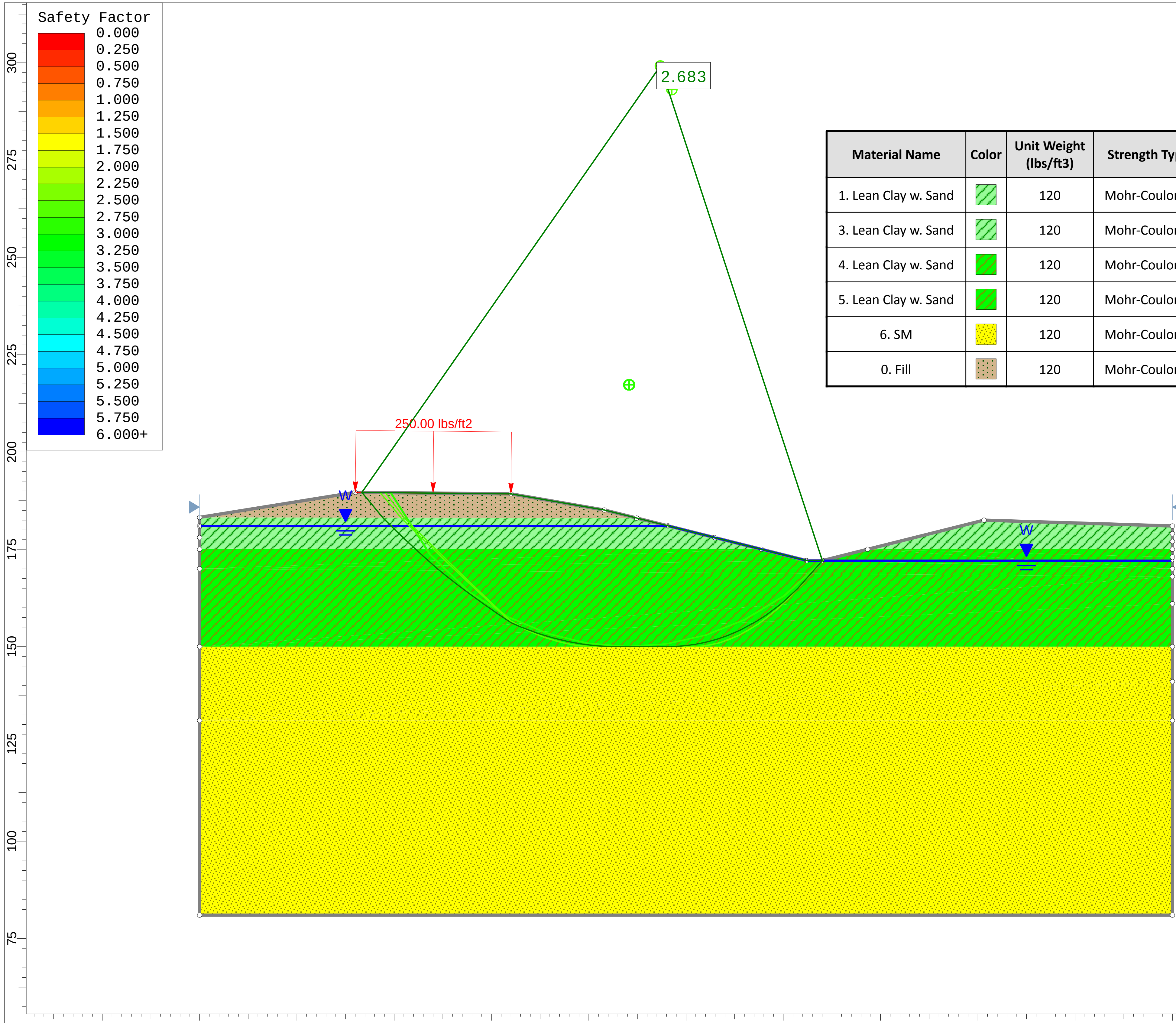
USCS Silty Sand
USCS Clayey Sand
USCS Low Plasticity Clay
USCS Silt
USCS Low Plasticity Silty Clay
USCS High Plasticity Clay





Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
1. Lean Clay w. Sand		120	Mohr-Coulomb	10	32	Water Surface
3. Lean Clay w. Sand		120	Mohr-Coulomb	10	32	Water Surface
4. Lean Clay w. Sand		120	Mohr-Coulomb	10	32	Water Surface
5. Lean Clay w. Sand		120	Mohr-Coulomb	10	32	Water Surface
6. SM		120	Mohr-Coulomb	10	36	Water Surface
0. Fill		120	Mohr-Coulomb	10	32	Water Surface

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT	Project	H.003496 - I-49 North, Seg J	
	Analysis Description	Sta 342+78, Long Term Baseline	
	Drawn By: JGR	Scale: 1:358	Company: LADOTD
SLIDEINTERPRET 7.023	Date: 4/6/2017, 1:52:15 PM		File Name: 343+00 LT - Baseline.slim

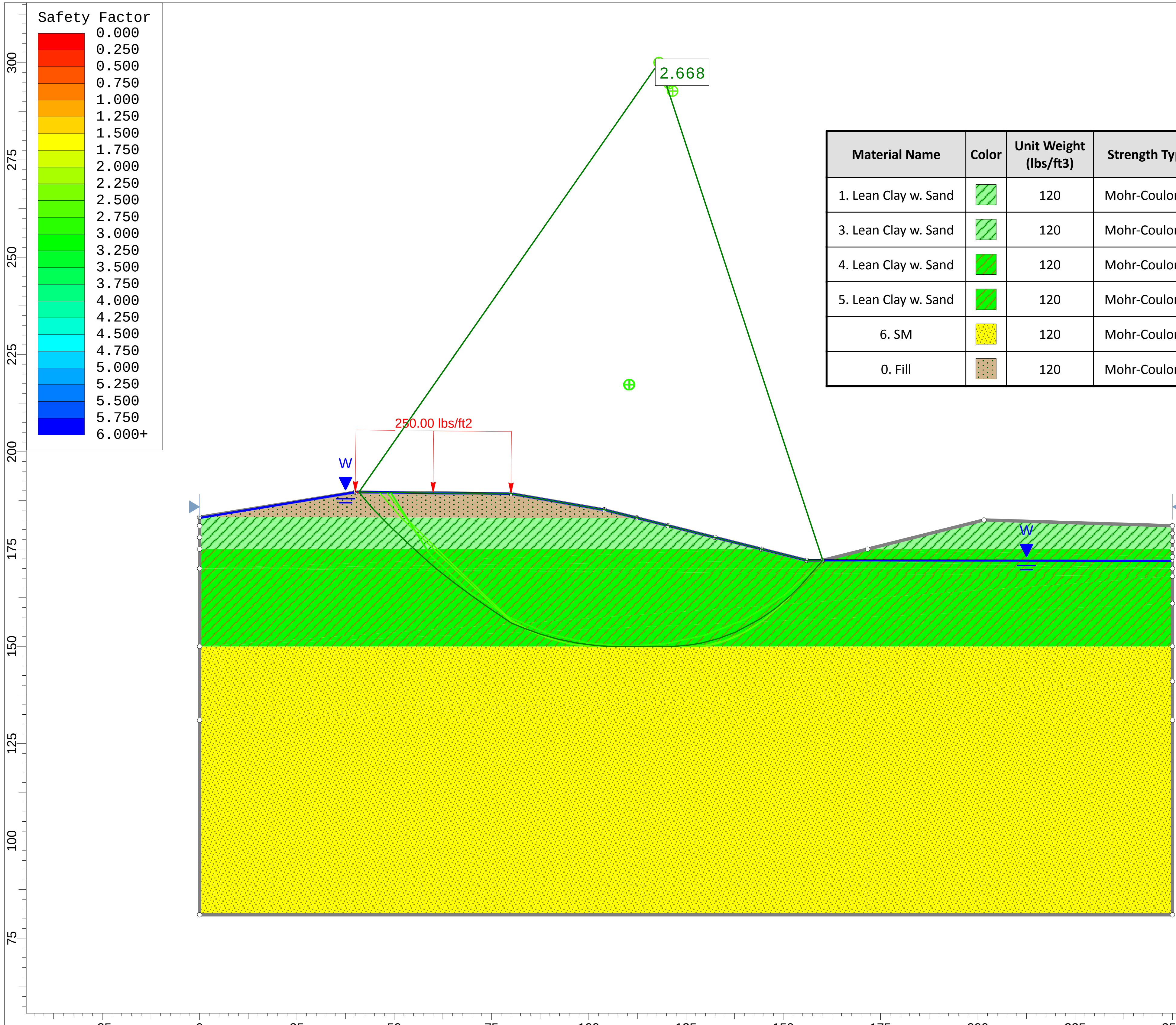


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
1. Lean Clay w. Sand		120	Mohr-Coulomb	1500	0	Water Surface
3. Lean Clay w. Sand		120	Mohr-Coulomb	1200	0	Water Surface
4. Lean Clay w. Sand		120	Mohr-Coulomb	1350	0	Water Surface
5. Lean Clay w. Sand		120	Mohr-Coulomb	900	0	Water Surface
6. SM		120	Mohr-Coulomb	10	36	Water Surface
0. Fill		120	Mohr-Coulomb	50	36	Water Surface

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

SLIDEINTERPRET 7.023

Project	H.003496 - I-49 North, Seg J
Analysis Description	Sta 342+78, Short Term Baseline
Drawn By	JGR
Scale	1:358
Company	LADOTD
Date	4/6/2017, 1:52:15 PM
File Name	343+00 ST - Baseline.slim

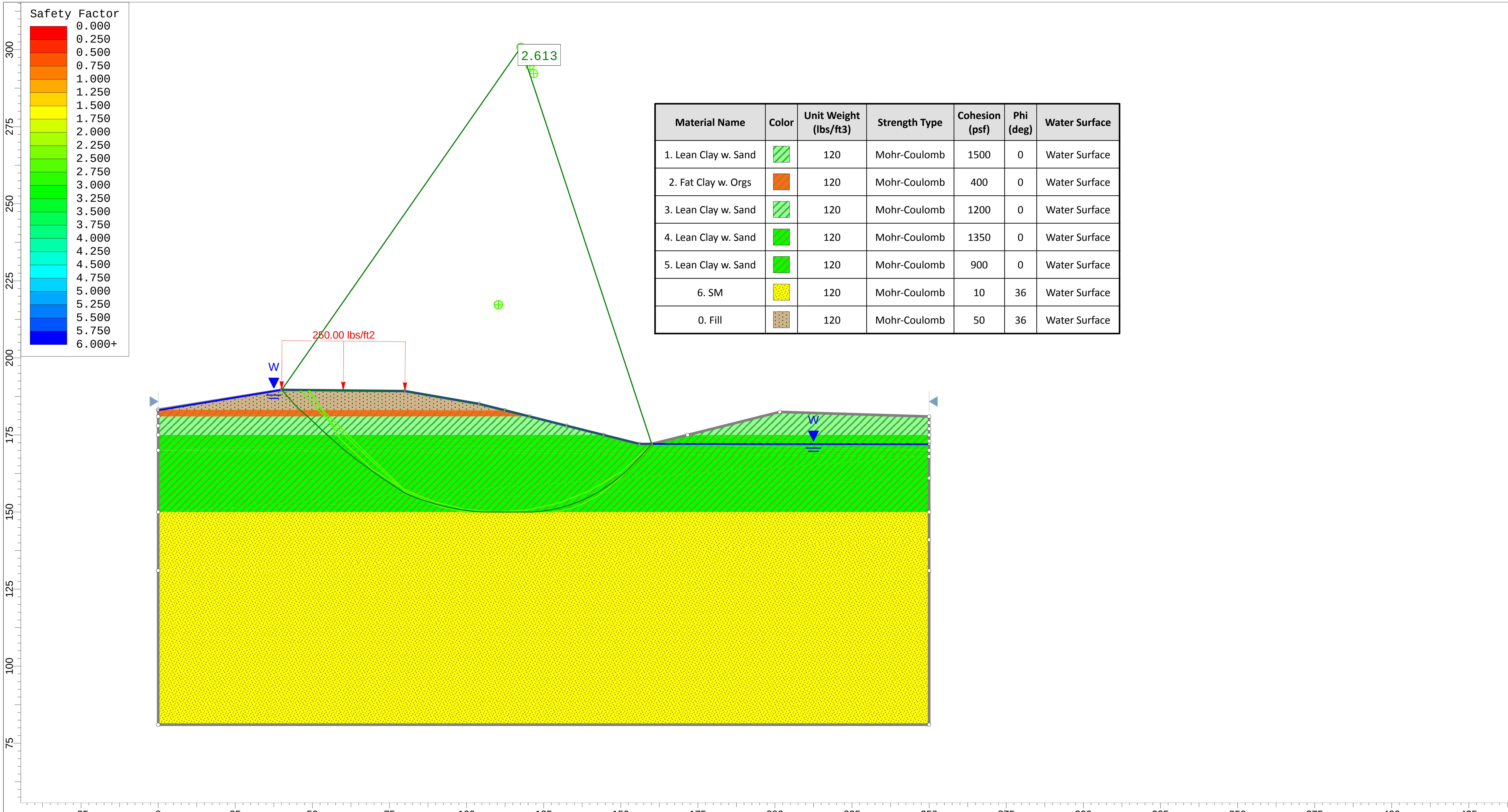


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
1. Lean Clay w. Sand		120	Mohr-Coulomb	1500	0	Water Surface
3. Lean Clay w. Sand		120	Mohr-Coulomb	1200	0	Water Surface
4. Lean Clay w. Sand		120	Mohr-Coulomb	1350	0	Water Surface
5. Lean Clay w. Sand		120	Mohr-Coulomb	900	0	Water Surface
6. SM		120	Mohr-Coulomb	10	36	Water Surface
0. Fill		120	Mohr-Coulomb	50	36	Water Surface

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

SLIDEINTERPRET 7.023

Project	H.003496 - I-49 North, Seg J
Analysis Description	Sta 342+78, Short Term Saturated Embankment
Drawn By	JGR
Scale	1:358
Company	LADOTD
Date	4/6/2017, 1:52:15 PM
File Name	343+00 ST - Saturated Embankment.slim

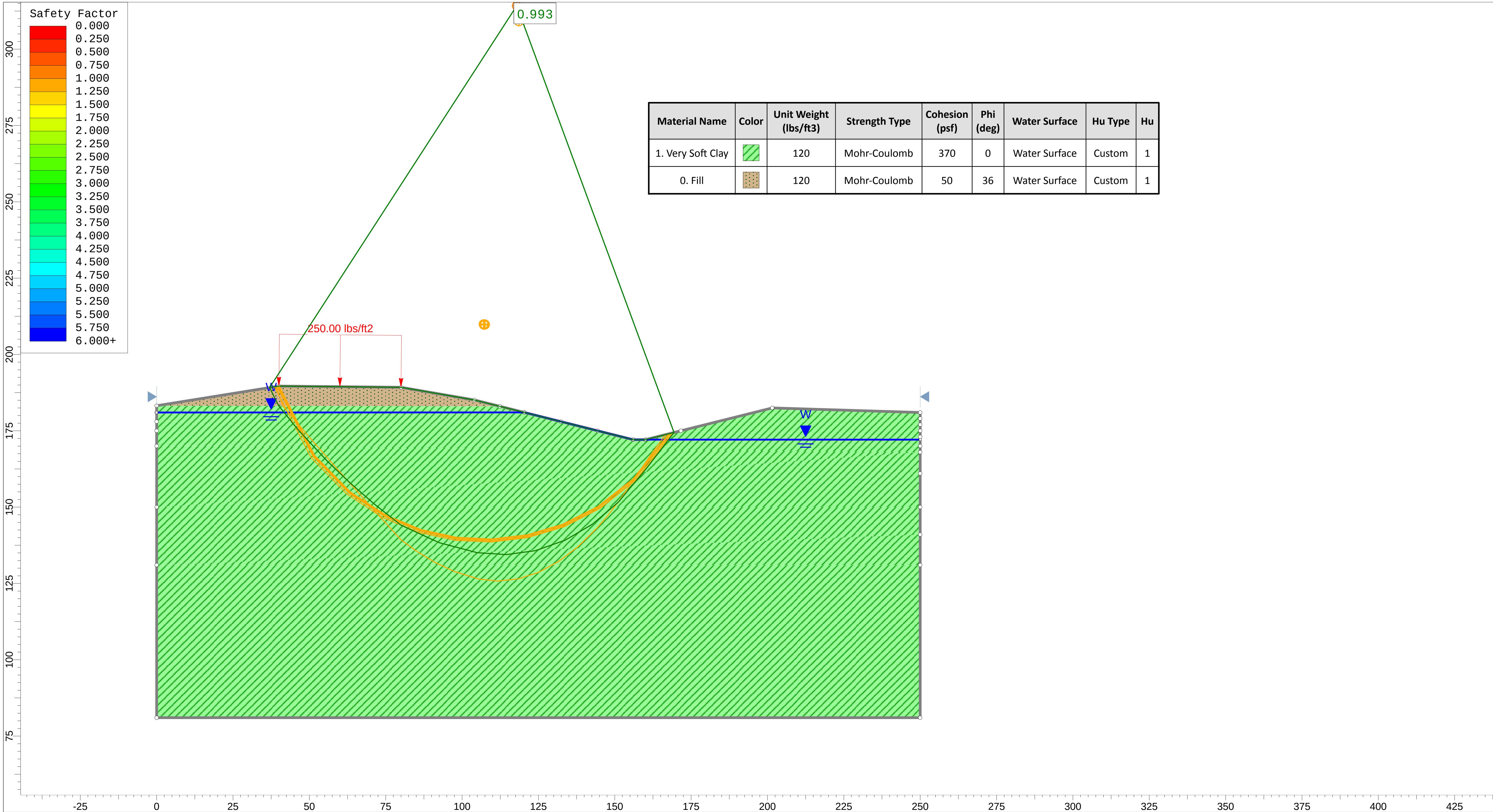


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
1. Lean Clay w. Sand		120	Mohr-Coulomb	1500	0	Water Surface
2. Fat Clay w. Orgs		120	Mohr-Coulomb	400	0	Water Surface
3. Lean Clay w. Sand		120	Mohr-Coulomb	1200	0	Water Surface
4. Lean Clay w. Sand		120	Mohr-Coulomb	1350	0	Water Surface
5. Lean Clay w. Sand		120	Mohr-Coulomb	900	0	Water Surface
6. SM		120	Mohr-Coulomb	10	36	Water Surface
0. Fill		120	Mohr-Coulomb	50	36	Water Surface

LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

SLIDEINTERPRET 7.023

Project	H.003496 - I-49 North, Seg J
Analysis Description	Sta 342+78, Short Term Saturated Embankment w/Weak Layer
Drawn By	JGR
Scale	1:358
Company	LADOTD
Date	4/6/2017, 1:52:15 PM
File Name	343+00 ST - Saturated Embankment w-Soft Layer.slim

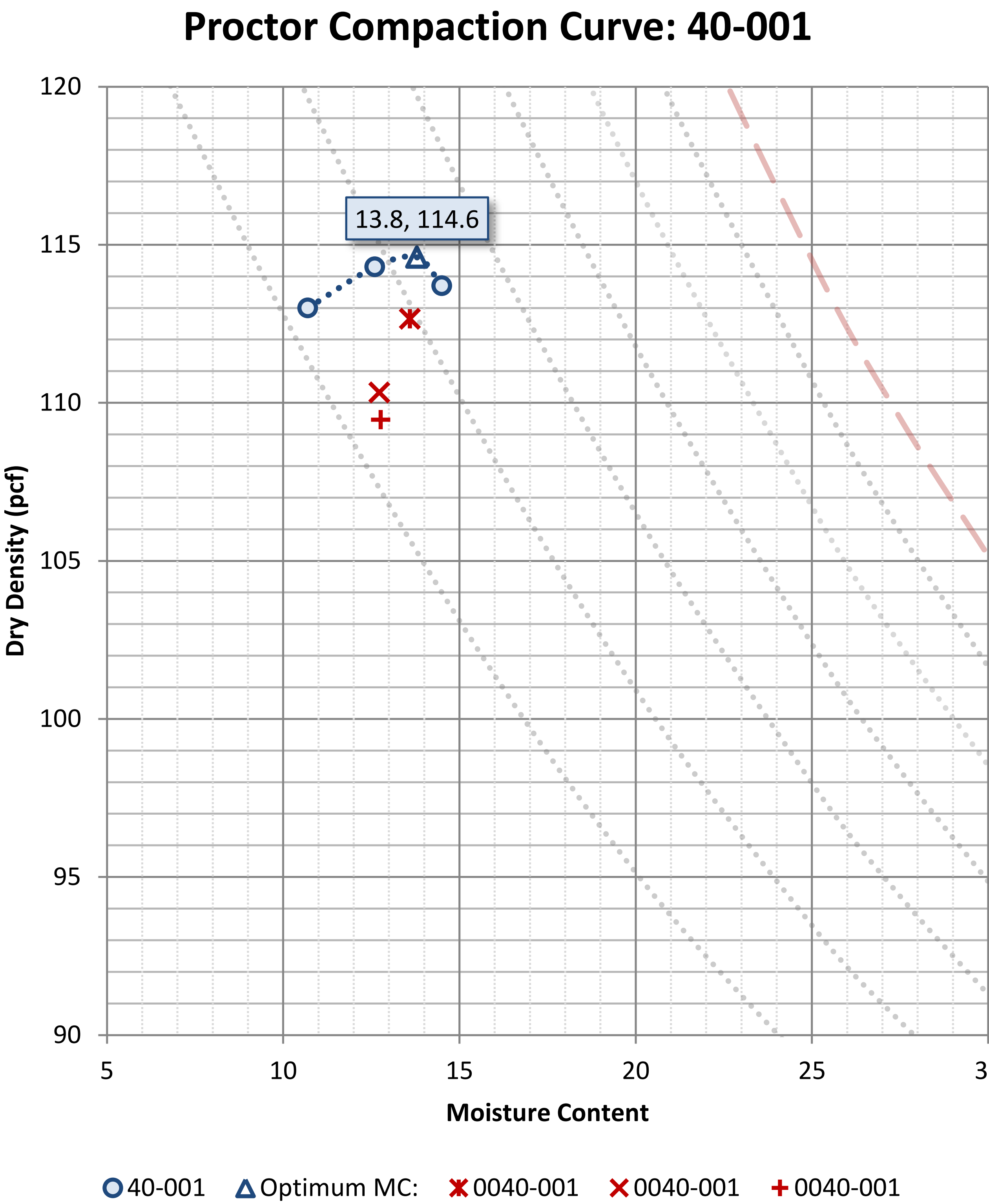


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
1. Very Soft Clay		120	Mohr-Coulomb	370	0	Water Surface	Custom	1
0. Fill		120	Mohr-Coulomb	50	36	Water Surface	Custom	1

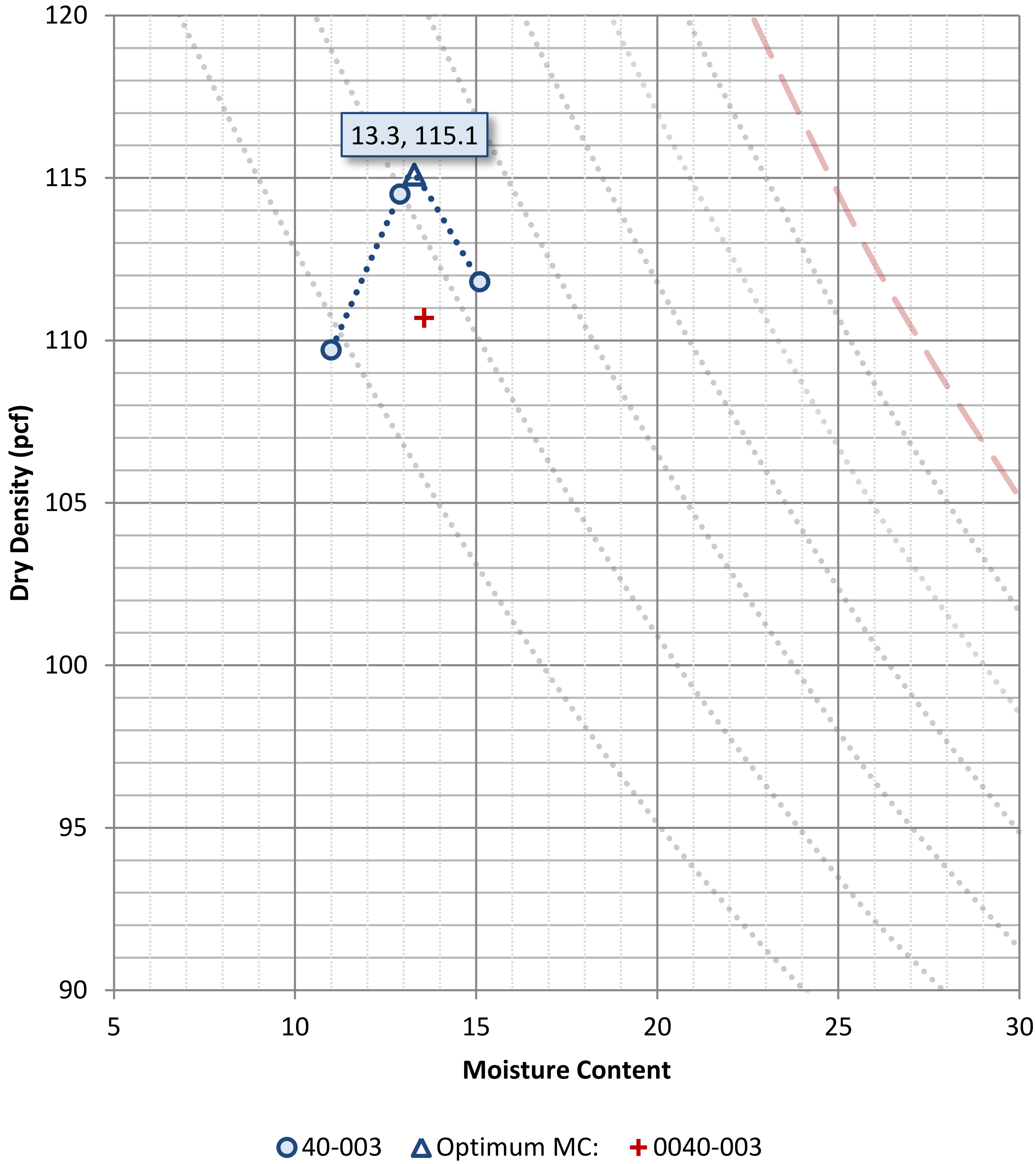
LOUISIANA DEPARTMENT OF TRANSPORTATION & DEVELOPMENT

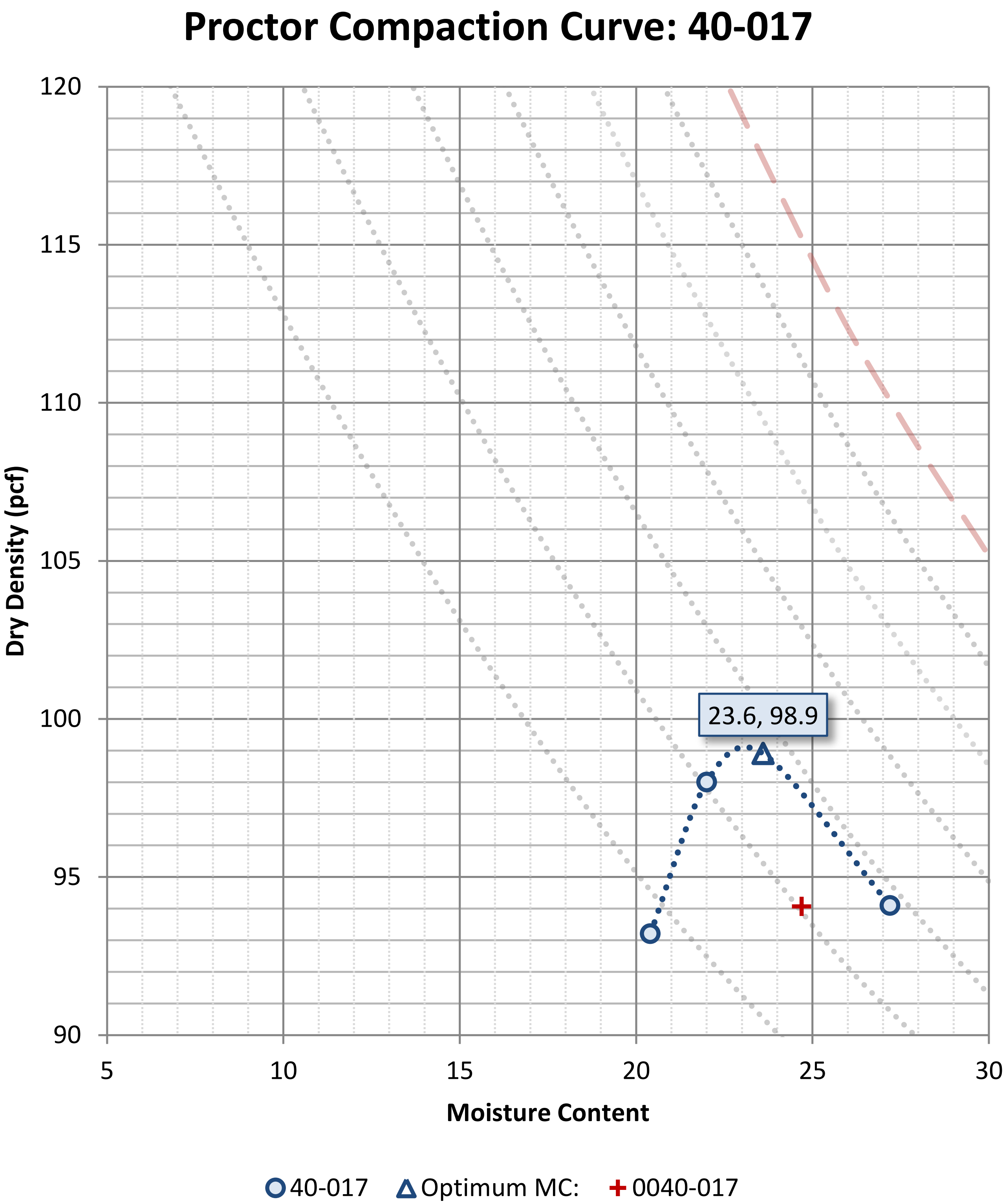
SLIDEINTERPRET 7.023

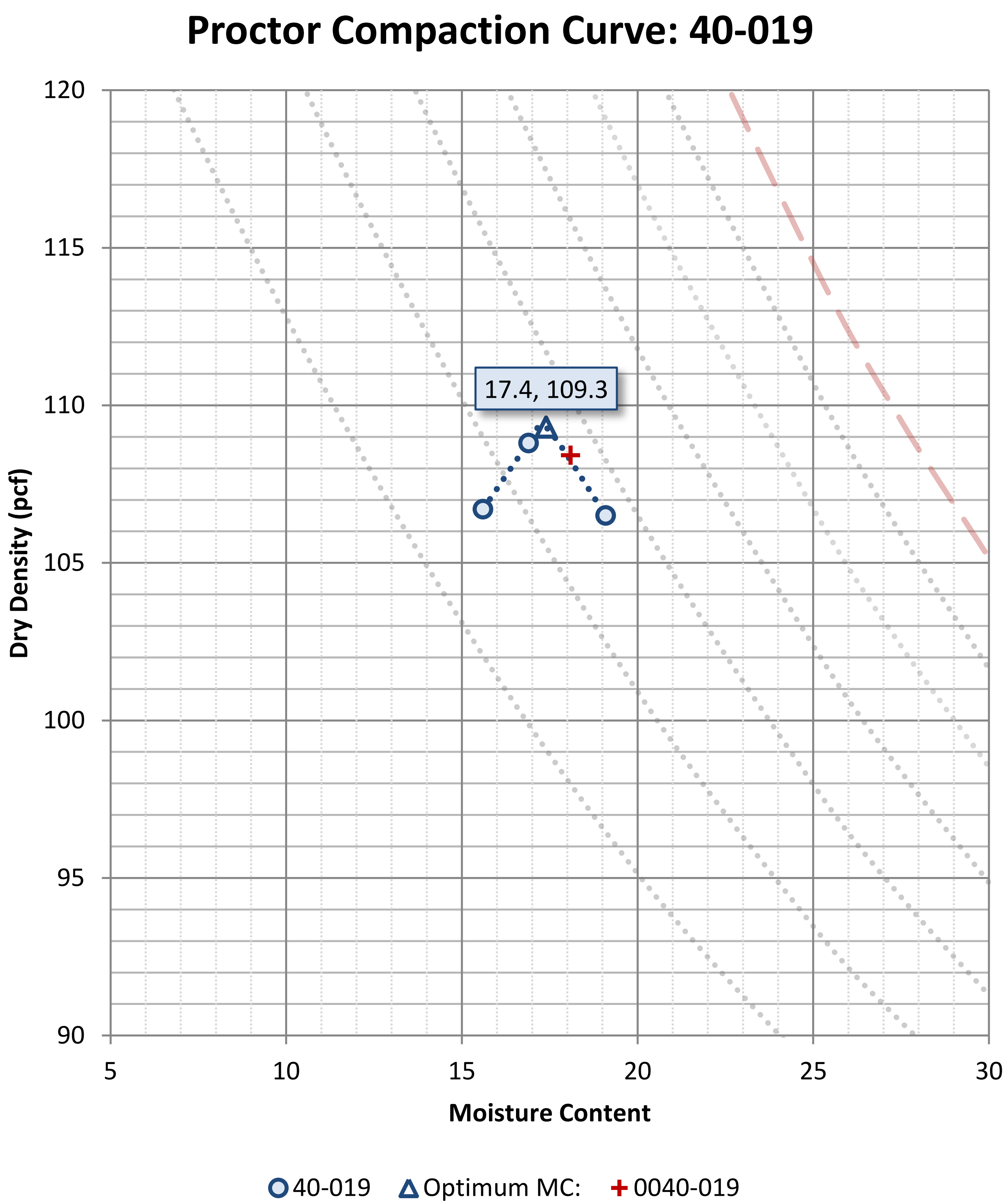
Project	H.003496 - I-49 North, Seg J
Analysis Description	Sta 342+78, Short Term Forced Failure
Drawn By	JGR
Scale	1:358
Company	LADOTD
Date	4/6/2017, 1:52:15 PM
File Name	343+00 ST - Failure Case.slim


Proctor Compaction Curve: 40-001
13.8, 114.6
Dry Density (pcf)
120
115
110
105
100
95
90
5
10
15
20
25
30
Moisture Content
40-001
Optimum MC:
0040-001
0040-001
0040-001

Proctor Compaction Curve: 40-003




Proctor Compaction Curve: 40-017
23.6, 98.9
Dry Density (pcf)
120
115
110
105
100
95
90
5
10
15
20
25
30
Moisture Content
40-017
Optimum MC:
0040-017


Proctor Compaction Curve: 40-019
17.4, 109.3
Dry Density (pcf)
120
115
110
105
100
95
90
5
10
15
20
25
30
Moisture Content
40-019
Optimum MC:
0040-019