



MDOT State Study 312 — Recycling PCC Pavements using Rubblization

Final Report

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16. Abstract Portland Cement Concrete (PCC) pavements still retain value after their useful design life. The most effective method to reuse PCC pavements is to recycle them as base/subbase layers. The Environmental Protection Agency's (EPA) Waste Reduction Model (WARM) estimates greenhouse gas (GHG) emissions savings of approximately 0.04 metric tons CO₂e per short ton of PCC not landfilled. Recycling PCC in place can achieve GHG emissions savings of 0.01 metric tons of CO₂e per short ton of aggregate replaced. PCC fracturing (FCP) methods generally include (1) rubblization (RCP) with steel to concrete bond ruptured, (2) crack-and-seat (C&S) for PCC without steel reinforcement, and (3) break-and-seat (B&S) for PCC with steel reinforcement. FCP structural coefficients, as shown below, are the recommended literature-reported values for the design of HMA overlays: RCP: 0.14 - 0.30, C&S: 0.20 - 0.35, B&S: 0.20 - 0.35 The AASHTO AASHTOWare Pavement ME Design characterizes pavement materials hierarchically, commensurate with their importance, size, and cost to the project. MEPDG Manual of Practice recommended FCP modulus values are: RCP: 50 ksi to 150 ksi, C&S, B&S: 150 ksi to 1,000 ksi The scheme used to estimate FCP layer coefficients is mathematically expressed as $a_{FCP} = 0.0045 \sqrt[3]{(E_{FCP})}$. The estimated FCP layer coefficients for Pike, Tate, and Forrest/Jones counties are similar to those reported in the literature.			
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List of Abbreviations

AI	Asphalt Institute
APT	Accelerated Pavement Testing
ASTM	American Society of Testing and Materials
B&S	Break and Seat
C&S	Crack and Seat
CRCP	Continuously Reinforced Concrete Pavement
DCP	Dynamic Cone Penetrometer
DOT	Department of Transportation
E _{FCP}	Fractured Slab Modulus
ERDC	US Army Engineer Research and Development Center
FCP	Fractured Concrete Pavement
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
HMA	Hot Mix Asphalt
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
ksi	Kips per Square Inch
LFWD	Light Falling Weight Deflectometer
MEPDG	Mechanistic Empirical Pavement Design Guide
MHB	Multi-Head Breaker
MPa	Mega Pascal
NAPA	National Asphalt Pavement Association
NCHRP	National Cooperative Highway Research Program
N _{MAX}	Nominal Maximum Aggregate Size
OMC	Optimum Moisture Content
PCC	Portland Cement Concrete
PCI	Pavement Condition Index
PDS	Prefabricated Drainage Systems
psi	Pounds per square inch
RCA	Recycled Concrete Aggregate
RCP	Rubblized Concrete Pavement
RDD	Rolling Dynamic Deflectometer
RFP	Resonant Frequency Breaker
SHRP	Strategic Highway Research Program
SN	Structural Number
SPS	Specific Pavement Studies
TRB	Transportation Research Board

Executive Summary

Portland Cement Concrete (PCC) pavements remain valuable assets beyond their design life. The benefit of recycling road construction materials can exceed \$41 per ton (1 ton = 0.91 metric ton); this estimate excludes processing and transportation costs but includes the avoided cost of virgin aggregates. The most effective method to reuse PCC pavements is to convert them into future pavements as base/subbase layers. The Environmental Protection Agency's (EPA) Waste Reduction Model (WARM) estimates greenhouse gas (GHG) emissions savings of approximately 0.04 metric tons CO_{2e} per short ton of concrete debris not transported to a landfill. Using recycled in-place concrete to replace virgin aggregate base/subbase can realize GHG emissions savings of 0.01 metric tons CO_{2e} per short ton of virgin aggregate material. PCC fracturing (FCP) methods for recycling generally include (1) rubblization (RCP) with steel to concrete bond ruptured, (2) crack-and-seat (C&S) for PCC without steel reinforcement, and (3) break-and-seat (B&S) for PCC with steel reinforcement.

Supporting the Mississippi Department of Transportation's (DOT) implementation of the mechanistic-empirical Pavement Design Guide (MEPDG), this research study investigates FCP techniques. It establishes layer coefficients for the current 1993 AASHTO design procedure. A hot-mix asphalt (HMA) overlay of FCP is an economically viable, environmentally friendly, and technically feasible rehabilitation approach for improving the performance of deteriorated PCC pavements.

This document briefly summarizes the state-of-the-art FCP techniques of RCP, B&S, and C&S and provides examples of successes and failures of HMA overlay of FCPs. The final study report includes the following sections:

- Study introduction and background
- Summary of literature review and FCP successes
- Description of test sections, locations, and backcalculated moduli
- Layer coefficient estimation schema and estimated layer coefficients
- Statistical analyses (rubblized vs C&S vs B&S) and recommended layer coefficients
- Conclusions and recommendations
- Implementation and technology transfer plan

Introduction/Background

Saeed and Akhtar⁽¹⁾ report that Portland Cement Concrete (PCC) pavements constitute an asset after their useful design life. The benefit of recycling road construction materials can exceed \$41 per ton (1 ton = 0.91 metric ton); this estimate does not include processing and transportation costs, but includes the avoided cost of virgin aggregate⁽⁵⁵⁾. Distress information (type, extent, severity, and causes) is essential for establishing candidate pavement rehabilitation strategies and selecting the optimal rehabilitation approach. HMA overlays of deteriorated PCC are the most used rehabilitation option. The performance of HMA overlays requires control of reflection cracks that appear within a couple of years of overlay placement over existing joints and cracks⁽²⁶⁾. The reflection crack distress in an HMA overlay at a PCC joint or crack is due to vertical load-induced stresses and strains⁽¹³⁾. PCC slab movements at the joints and/or cracks in PCC can be horizontal, which are induced due to moisture and/or temperature changes, or vertical, which are traffic load induced⁽²⁶⁾. Holt et al.⁽²⁾ emphasized that changes in crack width (the strain across the top of the crack) control the propagation of reflection cracking through new HMA overlays and demonstrated that the factors affecting crack movement were statistically significant in the development and propagation of reflection cracks.

Temperature variations are the primary climatic factor in the initiation and progression of reflection cracks; daily temperature fluctuations cause PCC pavement contraction and expansion, and slab curling (upward and downward). Freezing and thawing of pavements cause volume changes (e.g., expansion due to water freezing) and soften subgrade soils during thawing. Figure 1 shows expected reflective cracking propagation time in years after HMA overlay placement^(3, 18).

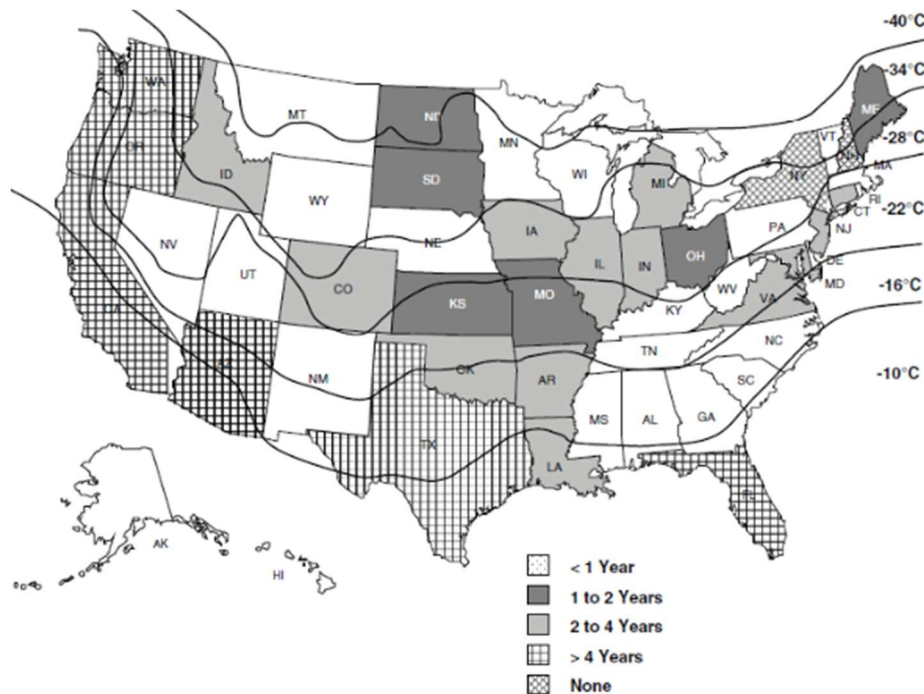


Figure 1. Time to observe reflective cracking after placing the HMA overlay^(3, 18).

Figure 2 shows reflective crack mitigation/prevention and repair techniques at joints and for the whole PCC pavement surface ⁽⁴⁾. The most effective method to reuse PCC pavements is to convert them into future pavements as base/subbase layers. Generally used PCC slab fracturing (fractured concrete pavement, FCP) methods include the following:

- Rubblization with steel to concrete bond ruptured (rubblized concrete pavement, RCP)
- Crack-and-seat (C&S) for PCC without steel reinforcement
- Break-and-seat (B&S) for PCC with steel reinforcement

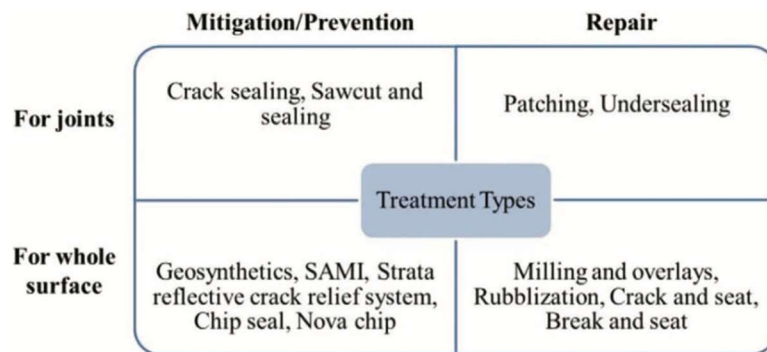


Figure 2. Classification of treatment types for reflective cracks ⁽⁴⁾.

Fracturing is the process of “in-situ” fracturing of an existing PCC pavement into small, interconnected pieces (FCP) that serve as a base course for a new hot mix asphalt (HMA) overlay. With no associated hauling or disposal costs, as all existing pavement material remains in place, rubblization is a cost-effective rehabilitation method. Crushing deteriorated PCC pavement, at a central plant or a temporary on-site plant, results in recycled concrete aggregate (RCA) that can be used as an unbound base/subbase layer in new pavement construction. Both C&S and B&S methods fracture the slab into large PCC pieces measuring 24 to 30 inches. Rubblization generally results in smaller particle sizes at the surface of rubblized PCC (approximately 3 inches) with 12-to-15-inch particles at the bottom of the rubblized layer ⁽¹⁾.

Research Objective

State Study 312 evaluated the successes and failures of fractured concrete as a base/subbase layer material, reviewed standards and specifications. It provided a procedure to estimate the fractured concrete layer coefficient. The results were summarized in a literature search report submitted earlier.

This report provides the FCP layer coefficient estimation schema and uses the recommended procedure to estimate the FCP layer coefficients for FCP in Pike, Tate, and Forrest counties. Statistical analyses are conducted to determine whether the three FCP techniques differ significantly from one another. Finally, conclusions are drawn from the work conducted, and recommendations are made to advance the work reported herein. A technology transfer and study implementation plan is proposed.

Literature Search Summary

Extent of HMA Overlays of PCC

Approximately 108,603 lane miles of PCC pavements overlaid with HMA (composite pavement), representing double the number of PCC-surfaced pavements in the United States ⁽⁵⁾. FHWA's data summarized in Table 1 (years 2011 through 2019) indicates approximately two-thirds of PCC pavements have been overlaid with HMA ⁽⁶⁾.

Table 1. Public Road Lane Miles by Type of Surface (National System)

Data Year	Concrete Surfaced (lane-miles)	Composite Pavement (lane-miles)
2011	77,347	84,954
2015	59,549	110,284
2016	57,066	111,882
2017	57,744	108,029
2018	55,942	88,589
2019	54,633	91,525

Performance of HMA Overlays of FCP

Many agencies in the United States have used FCP and RCA, followed by an HMA overlay, as an economically viable, environmentally friendly, and technically feasible approach to improving the performance of deteriorated PCC pavements. Mississippi DOT may glean limited guidance from research conducted by the Louisiana Department of Transportation and Development (LaDOTD) due to its proximity to Mississippi; however, specific information on the use of FCP will not be readily interchangeable for Mississippi conditions.

Witczak and Rada (1992) ⁽²⁶⁾ contend that HMA overlay of deteriorated PCC pavements is a viable rehabilitation strategy; associated problems include structural durability and premature ride quality losses due to reflective cracking. Pavement performance, estimated in accordance with (IAW) ASTM D6433 ⁽⁷⁾, triggers rehabilitation or reconstruction when PCC pavement performance drops below a certain level. However, satisfactory performance cannot be maintained, even with localized full-depth repairs and other performance-improvement techniques, as the PCC continues to deteriorate over time. Von Quintus et al. (2009) ⁽⁸⁾ proposed an *example framework* for evaluating existing jointed plain (JPCP) and jointed reinforced (JRCP) concrete pavements at airports for crack mitigation techniques.

Baladi and Svasdisant (2002) ⁽⁹⁾ conducted laboratory and field investigations and analyzed FWD and laboratory data to assert that the performance of HMA over RCP is greatly influenced by the project selection, including the RCP process, the construction procedure, the pavement design process, the pavement materials, and, finally, the traffic. The candidate project (1) should not include PCC pavements that have been cracked and sealed in the past, and (2) PCC pavements constructed on relatively soft soils with $E_{\text{Subgrade}} \leq 3$ ksi. Concrete pavements on subbases with $E_{\text{Subbase}} \leq 7$ ksi should not be rubblized. Additional recommendations include not rubblizing pavements with extensive discontinuities. RCP performance is sensitive to pavement variations, highlighting the need to tighten and enforce quality control procedures. Poor RCP performance is generally caused by inadequate rubblizing, poor construction of the asphalt overlay, or poor HMA mix design.

The State of Washington DOT demonstrated statewide network-level savings from cost-effective pavement assets management strategies, generating annual cost savings of \$80 million by 2025, including ⁽¹⁰⁾:

- Converting 3,000 lane miles (4,827 km) from HMA to chip seal resurfacing
- Planning \$850 million to \$1.0 billion for rehabilitation and reconstruction for the aging concrete pavement network for the next 10 years, with 50 percent of the concrete slated for reconstruction to be converted to flexible pavement using a C&S plus HMA overlay process ⁽¹¹⁾
- Strategically maintain flexible pavements to extend their service life, reducing their annual maintenance cost

Additional savings of \$1.6 million and 90 construction-closure hours per lane mile are expected with C&S and HMA overlays. By 2027, the State of Washington expects to convert about 260 lane miles (418 km) of current concrete pavement to C&S with an HMA overlay, saving approximately \$7 million annually ⁽¹⁰⁾.

The Asphalt Institute (AI) describes rubblization (fracturing) of PCC into small pieces, converting it into an interlaced, but flexible, base material ⁽¹²⁾. RCP is offered as a solution for the rehabilitation of deteriorated PCC using an HMA overlay, providing an excellent service life. The National Asphalt Pavement Association (NAPA) provides design and construction guidelines on rubblizing and overlaying deteriorated PCC pavements with HMA and discusses specifications from several states ⁽¹³⁾.

Many research studies did not emphasize the development of RCP/RCA MEDPG input parameters, though several states have used FCP with HMA overlays to rehabilitate highways of varying capacities. Transportation Research Circular E-C087 ⁽¹⁴⁾ on PCC rubblization summarizes:

- State of the practice of design and construction of HMA overlays on rubblized PCC pavements
- Rubblizing equipment
- Rubblization of airfield pavements
- Non-destructive testing of rubblized PCC
- Performance of RCP

PCC Fracturing Techniques

The objective of slab fracturing is to minimize temperature (horizontal) and traffic (vertical) induced movement of the slab joints and cracks ⁽²⁶⁾ by minimizing stress concentrations that are transferred to the

HMA overlay. Several methods are available to fracture concrete to minimize or eliminate reflective cracking in HMA overlays over deteriorated PCC pavements. Methods to safeguard against reflective cracking include:

- Sawing and sealing of joints (and large cracks $\geq 1/4$ inch wide) soon after HMA overlay
- Incorporating a stress-relieving membrane in the overlay design, including fabrics⁽¹⁵⁾
- Increasing HMA overlay thickness
- Using modified binders with enhanced stress-absorbing properties⁽¹⁶⁾
- Fracturing of PCC slab

The following PCC fracturing techniques are used to essentially reduce the PCC slab into a high-quality base/subbase material/layer using different techniques and equipment^(17, 25, 27, 13, 14, 38):

- RCP is used for any PCC pavement (steel to concrete bond ruptured)
- C&S is used for jointed plain concrete pavement (JPCP, PCC without steel reinforcement)
- B&S is used for jointed reinforced concrete pavement (JRCP, PCC with steel reinforcement)

It is well documented that the B&S technique is generally applicable to JRCP and C&S for rehabilitation of JPCP, while RCP can be recommended for any deteriorated PCC pavement^(5, 17, 25, 26, 27, 52, 53). As B&S and C&S result in FCP pieces ≥ 24 inch to ≤ 30 inch, HMA overlays constructed over fractured PCC slabs are still prone to reflective cracking. Since this size range is much larger than a typical base/subbase top-size aggregate, Florida DOT⁽¹⁸⁾ recommends a slightly larger B&S and C&S FCP slabs ranging from 2 to 6 ft in length (24 to 72 inch).

C&S produces short rigid slabs effectively 12- to 48-inches long^(25, 26). Pile drivers, guillotines (Figure 3), or spring arm (whip) hammers are used to produce C&S FCP. C&S produces closely spaced FCP pieces that reduce vertical and horizontal movements while maintaining full aggregate interlock and load transfer across the crack with little loss of structural capacity.

B&S requires more energy to produce FCP (relative to C&S) due to steel reinforcement and the need to eliminate the bond between PCC and steel in JRCP. B&S ensures that the FCP process does not result in a series of broken slabs tied together with bonded steel reinforcement^(25, 26, 27).

RCP breaks the PCC slab into small pieces ranging from sand-size particles to 4-inch by 8-inch pieces with smaller pieces towards the top of the pavement and larger pieces at the bottom of the slab (up to 15 inches⁽³²⁾). RCP fractures the PCC slab into angular pieces and severely degrades (or destroys) the PCC slab's structural integrity to prevent reflective cracking into the HMA overlay. The PCC slab is fractured through its thickness, and the bond between the concrete and any reinforcement is destroyed, while protecting the base/subbase from any damage⁽¹⁹⁾. The two types of equipment used for RCP are (1) the resonant frequency breaker, RFB (manufactured by Resonant Machines Company), and (2) the multi-head breaker, MHB (manufactured by Antigo Construction). The RFB, shown in Figure 4, operating at high frequency and low amplitude, fractures the PCC slab by generating a 45-degree shear plane⁽²⁰⁾. Figure 5 shows the MHB incorporating 12 to 16 drop hammers, weighing between 1,270 and 1,750 lbs, to fracture the PCC slab using free fall^(21, 22). The MHB is a single-pass machine with a breaking width of one vehicle-lane; the equipment operating characteristics of hammer weights, number of hammers, hammer drop

heights, and travel speed can be varied to produce a wide range of fracture energies to accommodate project conditions^(20, 23). Drop hammers break PCC slabs by impacting them, thereby placing the bottom of the slab in tension.

Table 2 provides the C&S, B&S and RCP slab recommended crack spacing and expected aggregate sizes in terms of nominal maximum aggregate size (N_{MAX}). N_{MAX} is typically defined as the largest sieve that retains no more than 10 percent of the aggregate by weight.

Table 2. Expected aggregate size and recommended crack spacing for slab fracturing methods.

Fracturing Method	Recommended Crack Spacing	Nominal Max. Particle Size ^b , N_{MAX} ^(17, 25, 26)	Typical Cost
Crack & Seat	≤ 30 inch	$18 \text{ inch} \leq N_{max} \leq 24 \text{ inch}$ $\leq 20\% \text{ FCP} \leq 24 \text{ inch}$ occasionally 30-inch square piece	CY 2002 ⁽²⁵⁾ : \$0.35 - \$0.45 /yd ²
Break & Seat	≤ 30 inch	≥ 12 inch with ruptured reinforcement ⁽²⁴⁾	CY 2002 ⁽²⁵⁾ : \$0.35 - \$0.45 /yd ²
Rubblization ^a	≤ 12 inch	Surface: $2 \text{ inch} \leq N_{max} \leq 3 \text{ inch}$ Slab Bottom: $9 \text{ inch} \leq N_{max} \leq 12 \text{ inch}$	CY 2008 ⁽²³⁾ : \$1.41 - \$2.05 /yd ²

^a \$ range provided based on information from eight states

^b N_{MAX} is the largest sieve that retains no more than 10 percent of aggregate by weight



Figure 3. Guillotine breakers are used for the C&S FCP methodology.

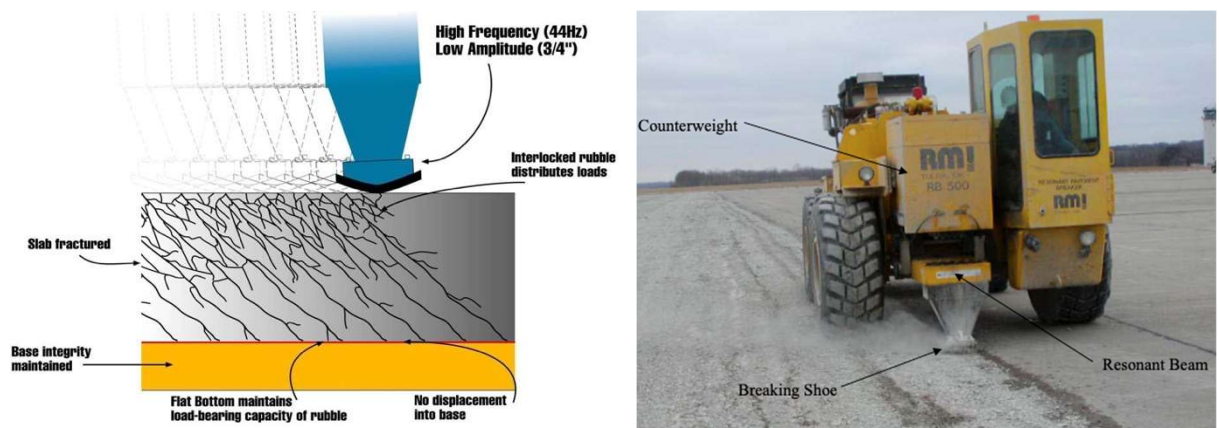


Figure 4. The RCP process rubblizes the PCC slab into angular pieces using RFB⁽¹⁹⁾.



Figure 5. Variations of MHB equipment created by Antigo Construction, Inc. ⁽²¹⁾.

Seating Fractured Concrete

Seating presses FCP fragments downward to ensure firm contact with the supporting layers ^(25, 26, 27). FCP is seated by rolling to maintain a stable foundation (enhance support with base/subbase) using pneumatic-tired, steel drum, vibratory, and Z-pattern rollers. Pneumatic rollers are most effective for seating, as steel drum and vibratory rollers tend to bridge over FCP. Over rolling of FCP tends to increase the measured FCP surface deflection ⁽²⁴⁾.

RCP produced using different equipment (e.g., multi-head breakers (MHB) versus resonant frequency breakers (RFB)) has different seating needs ⁽²⁸⁾. RFB-produced RCP is seated using a 10-ton tandem vibratory roller, in low amplitude and high frequency mode, very similar to a thin HMA lift ($\approx$ 2-inch). MHB-produced RCP is seated using a Z-patterned or Elliott grid vibratory roller, as shown in Figure 6 ^(29, 30). Figure 8 shows RFB-produced RCP before and after rolling to seat the rubblized PCC ⁽³¹⁾. Guillotine or hammer-produced C&S FCP is seated using several passes of a 30-to-50-ton rubber-tired roller. An 8- to 10-ton pneumatic roller smooths up the RCP surface before HMA overlay placement (Figure 7). In areas with subgrade drainage problems, characterized by weak or wet subgrades, the vibratory amplitude should be reduced to prevent subgrade damage. It is effective to use water during FCP seating, and usually, there is no need to apply a prime coat on FCP before the HMA overlay ⁽²²⁾.



Figure 6. Vibratory roller with Z-patterned or Elliott grid attachment (www.elliottattach.com).



Figure 7. Frequently the Z-grid roller is followed by a steel-wheeled or pneumatic tire roller ^(29, 30).



Figure 8. RFB-rubblized PCC before and after seating ⁽³¹⁾.

Most state DOTs require most RCP particles to be between 1 and 3 inches in size; however, N_{MAX} particles near the edges and under the steel reinforcement can be as large as 15 inches ⁽³²⁾. Pneumatic and steel vibratory rollers are the most widely used for compacting (seating) the RCP; only two states reported using pneumatic rollers to seat RCP. Table 3 lists FCP N_{MAX} allowed by various state DOTs (76 percent response rate).

Table 3. N_{MAX} reported by state DOTs responding to the 1998 Florida DOT survey of RCP practices ⁽⁴⁵⁾.

Maximum Particle Size, N_{MAX}	Responding States	Number of States
5 to 8 inch	Alabama, Indiana, Michigan, Massachusetts, Missouri, Mississippi, North Carolina, North Dakota, Ohio, Oklahoma, Virginia	11
9 to 15 inch	Arkansas, Illinois, Kansas, Louisiana, Maryland, Minnesota, Nevada, New York, Pennsylvania, Wisconsin	10

Weak Subgrade Considerations in Selecting Fracturing Technique

Weak subgrades create problems during rubblization due to a lack of support and ultimately may result in premature pavement failure⁽³²⁾. Subgrade (and base/subbase) support significantly affects the long-term performance of the HMA overlay of FCP⁽¹⁸⁾. The Illinois DOT summarizes a method for evaluating the success of an HMA overlay over RCP rehabilitation⁽³³⁾ based on more than 10 years' experience with RCP. The Illinois DOT, based on its experience with 10 projects, found that overlaying RCP performed better than patching and overlaying alone. Heckel⁽³³⁾ documents three projects utilizing RFB and four projects using the MHB with recommendations (shown in Figure 9) as:

- Method I - Use the MHB
- Method II - Use RFB with high floatation tires with tire pressure ≤ 60 psi
- Method III - Use RFB with restrictions on tire pressure
- Method IV - Use either the MHB or RFB

The Texas Transportation Institute (TTI) documents the suitability of RCP followed by construction and performance evaluations of two recently completed RCP projects^(34, 35) and proposed a modified FCP process in Texas (shown in Figure 9). The procedure summarizes that high risk for RCP should translate to moderate risk for C&S, and moderate risk for RCP should translate to low risk for C&S or saw-cut and C&S. For example, an eight-inch-thick JPCP on a 3 CBR subgrade would be high risk for RCP but moderate risk for C&S.

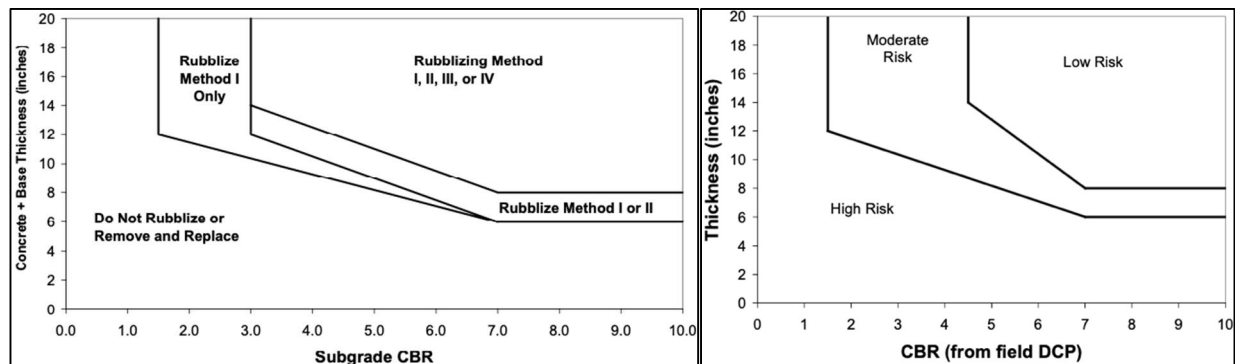


Figure 9. Illinois DOT rubblization risk chart^(35, 33) and TTI-modified chart⁽³⁴⁾.

Factors essential for successful RCP performance include, but are not limited to, the following^(12, 27, 25, 33, 34, 35, 36, 45, 46, 47, 48, 51):

- thickness of the pavement structure
- subgrade condition
- presence of excessive subgrade moisture
- pavement drainage infrastructure
- presence of the shoulder

The Michigan DOT evaluated the performance of geocomposite drains (Prefabricated Drainage Systems [PDS]) installed as underdrains on HMA overlays for C&S, B&S, RCP, and recycled concrete rehabilitation construction projects⁽³⁷⁾. Although the Michigan DOT observed additional bottom distress and occasional bending of the PDS top, these PDS systems were evaluated to be performing well.

Several factors affect E_{RCP} , including the following⁽²²⁾:

- age of the existing PCC (E_{RCP} increased with time, possibly due to the cement stiffening effect)
- underlying support (E_{RCP} increased with increasing underlying support)
- edge drains effectively prevented moisture from building up under RCP
- rubblization efforts (E_{RCP} increases with particle size and dimension of fractured slab, working contrary to reflective crack mitigation).

The US Army Engineer Research and Development Center (ERDC) summarized successful RCP performance on military airfields based on evaluations using the FWD, finite element analysis, and cost estimates. They developed reliable criteria for the design of asphalt overlays over RCP⁽³⁸⁾.

Structural Factors

The 1993 AASHTO Guide recommended FCP structural coefficients are the most preferred method to establish the design layer coefficients⁽⁵⁾:

- RCP: 0.14 - 0.30
- C&S: 0.20 - 0.35
- B&S: 0.20 - 0.35

Figure 10 compares other methods to establish FCP structural coefficients⁽⁵⁾.

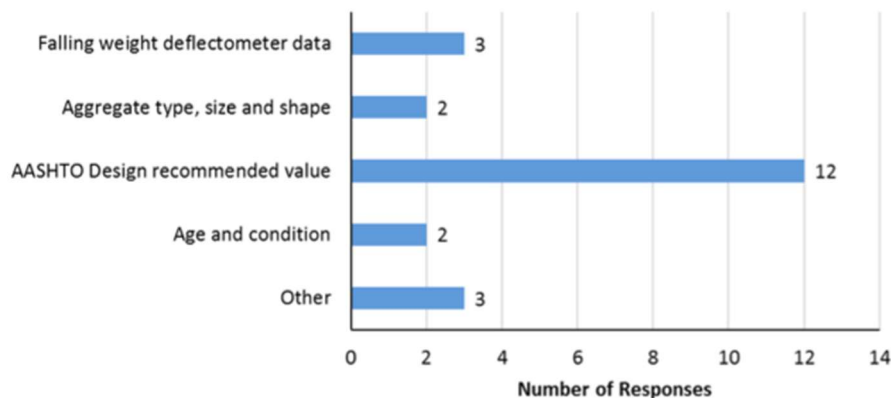


Figure 10. Layer coefficient determination techniques used by respondents⁽⁵⁾.

State Study 105⁽³⁹⁾ determined a layer coefficient for rubblized CRCP for use in the design of overlay thickness over rubblized CRCP. The study involved a 14.2-mile-long CRCP rehabilitation/reconstruction project on a divided four-lane section of Interstate 55 in Montgomery, Carroll, and Grenada Counties in

North Mississippi. The rehabilitation design for the southbound lanes involved repairing working punchouts and then overlaying with HMA. The reconstruction design for the two northbound lanes included installing edge drains, rubblizing the 8-inch-thick CRCP surface, and placing 6.5 inches of HMA over the rubblized concrete. The example E_{RCP} versus reliability level shown in Figure 11 was developed for each year non-destructive tests were conducted; these are reported in Table 4 and were used in conjunction with Figure 12 to estimate layer coefficients. Based on work conducted for State Study 105, Barstis et al⁽³⁹⁾ selected a FCP layer coefficient of 0.25 for pavement overlay design.

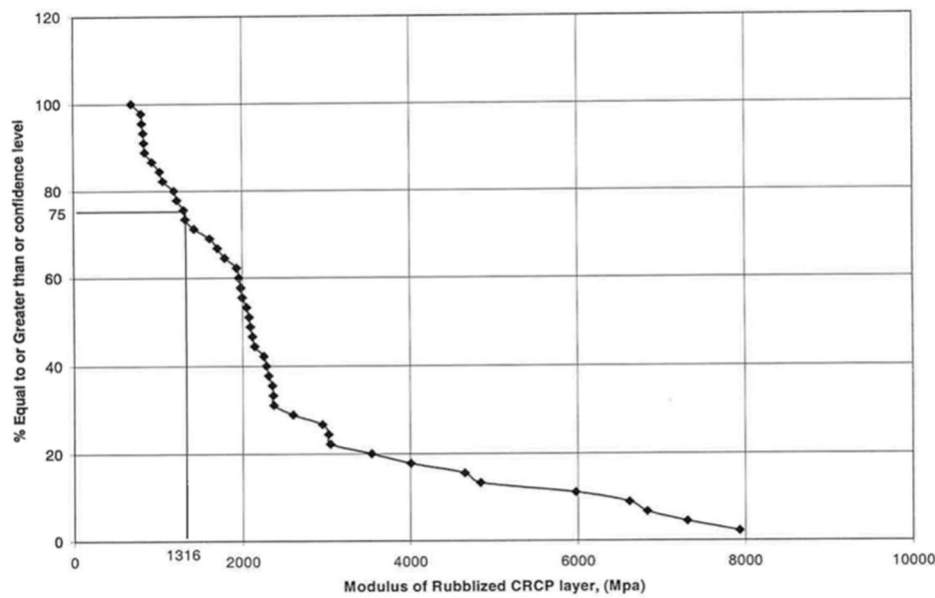


Figure 11. State Study 105 selected E_{RCP} of 1,316 MPa at 75 percent reliability⁽³⁹⁾.

Table 4. Reliability (confidence) level versus E_{FCP} Used to estimate the AASHTO layer coefficient⁽³⁹⁾.

Confidence Interval, %	1994 Data	1996 Data	1997 Data	Average E_{FCP} , MPa	Design Layer Coefficient
75	1316	2241	3586	2431	0.31
85	1005	1909	915	1276	0.25
90	844	1687	725	1086	0.24
95	767	1316	535	873	0.22

The Iowa DOT estimated an average rubblized PCC layer coefficient of 0.19 and backcalculated E_{RCP} to be 539 MPa (78 ksi)⁽⁴⁰⁾ based on data from seven test sections. The Wisconsin DOT⁽⁴¹⁾ surveyed state DOTs (14 respondents) to determine layer coefficient values used to characterize C&S and RCP layers (shown in Table 5). Forty-three percent of respondents reported using C&S, with layer coefficients ranging from 0.25 to 0.40, a mean of 0.30, and a median of 0.28. The RCP layer coefficient ranged from 0.14 to 0.29 with a mean of 0.21 and a median of 0.20. The Indiana DOT evaluated pavement sections before and after an HMA overlay to estimate the AASHTO layer coefficient of RCP; test results established the project RCP layer coefficient at 0.25⁽⁴²⁾.

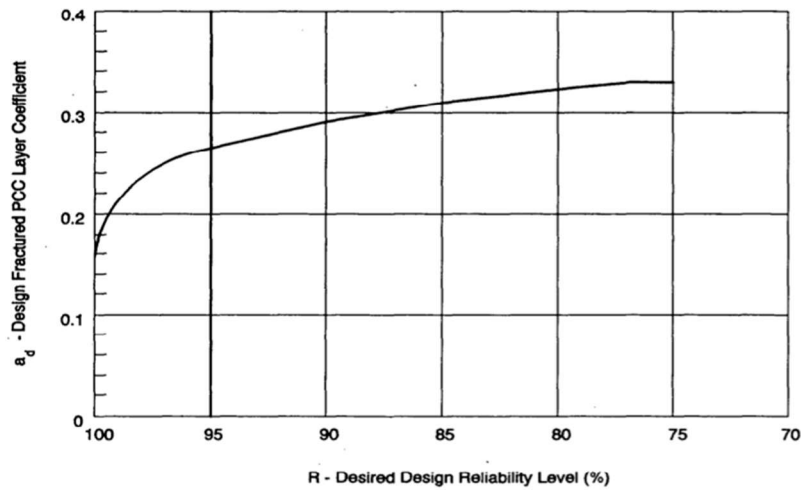


Figure 12. Recommended AASHTO layer coefficients for RCP and C&S layers.

Table 5. Layer coefficients reported by state DOTs responding to the Wisconsin DOT survey ⁽⁴¹⁾.

State DOT	C&S Layer Coefficient	RCP Layer Coefficient	Remarks
Maryland	0.25	0.20	C&S: 0.20 to 0.35; RCP: 0.15 to 0.30
Ohio	0.27	0.14	
South Dakota	0.28	0.24	
North Dakota	0.29	N/A	No RCP use; B&S: 0.25
Arkansas	0.30	0.29	
Kansas	0.40	0.20	
Maine	N/A	0.20	Literature-based RCP coefficient
Indiana	N/A	Not provided	
California	N/A	N/A	Layer coefficients not used in design
Montana	N/A	Not provided	Layer coefficients not used in design
Georgia	N/A	N/A	No experience with C&S or RCP
Minnesota	N/A	GE of 1.5	Gravel equivalence (GE); no C&S
Illinois	N/A	N/A	ME process used for RCP; no C&S
Mississippi	N/A	Not provided	No C&S experience
Mean	0.30	0.21	
Median	0.28	0.20	

Table 5 values closely follow those recommended and developed by NAPA Information Series 117 (NAPA IS 117) ⁽⁴³⁾ and NCHRP 1-37A ⁽⁴⁹⁾ and used by Barstis et al ⁽³⁹⁾ for State Study 105. The Wisconsin DOT ⁽³⁶⁾ recommend a value of 0.22 for RCP to ensure sufficient HMA thickness over the RCP layer for overlay performance and to mitigate reflective cracking. For ME design, an E_{RCP} of 65 ksi is recommended (mathematically derived based on RCP performance in Wisconsin), which is lower than those recommended in NAPA SS 117 ⁽⁴³⁾ and NCHRP 1-37A ⁽⁴⁹⁾. With a 76 percent response rate, the FCP layer coefficients used by various state DOTs are summarized in Table 6.

Table 6. E_{RCP} reported by state DOTs responding to the 1998 Florida DOT survey of RCP practices ⁽⁴⁵⁾.

AASHTO Layer Coefficient	Responding States	Number of States
0.10 to 0.15	Michigan, Maryland, Mississippi, Missouri, Ohio, Vermont	6
0.16 to 0.20	Indiana, Michigan, Minnesota, North Dakota, Pennsylvania	5
0.21 to 0.25	Alabama, Illinois, Kansas, Massachusetts, Michigan, Wisconsin	6
0.26 to 0.30	Arkansas, Louisiana, North Carolina, Nevada, Oklahoma	5
No response	New York	1

Structural deficiency is an alternate approach endorsed by the 1993 AASHTO Guide for the design of HMA overlays of FCP (both C&S and RCP) ⁽⁴⁴⁾. The structural deficiency between the structural number (SN_F) required to support traffic over some future design period, and the effective structural number (SN_{EFF}) of the existing FCP must be satisfied by the HMA overlay. Assigning a structural coefficient to the FCP layer has the greatest impact on future performance, as it influences both the HMA overlay thickness and the probability of reflection cracking.

The required design inputs for the mechanistic procedures for the structural design of HMA overlays of C&S pavements include the design E_{HMA} , the $MR_{Subgrade}$, and the $E_{Equivalent-C\&S}$ ^(45, 27); NCHRP Synthesis No. 144 ⁽⁴⁵⁾ provides complete guidance. Using the finite element program ILLI-PAVE, the HMA bending strains for a range of overlay thicknesses were estimated; these were used to develop transfer functions for the number of repetitions to failure as a function of bending strain for typical Illinois DOT Class I asphalt concrete mixtures, using the procedure summarized by Schutzach ^(46, 47).

Work conducted at the U.S. Army Engineer Research and Development Center (ERDC) demonstrated that PCC pavement with an intact elastic modulus of about 6×10^3 ksi was reduced by C&S to an $E_{Effective-C\&S}$ of about 1 to 1.5×10^3 ksi. Field FWD based values were confirmed using layered elastic methods. Table 7 shows that after C&S, the average predicted subgrade modulus decreased because the subgrade was carrying an additional load, and a higher stress was being applied to the top of the subgrade because of the reduced strength of PCC due to C&S. The subgrade modulus then increased with each additional asphalt overlay, which corresponds to a lower stress on the subgrade ⁽⁴⁸⁾.

Table 7. Predicted modulus values and vertical stress on top of the subgrade ⁽⁴⁸⁾.

Pavement Condition	$E_{Subgrade}$, psi	$E_{Concrete}$, psi	E_{HMA} , psi	Maximum Vertical Stress on Subgrade, psi
Intact PCC	21,618	5,896,165		2.02
C&S	12,106	1,143,948		3.77
with 4-inch HMA	15,642	1,276,666	271,885	2.97
with 6-inch HMA	17,598	1,541,334	361,063	2.33

The MEPDG Manual of Practice ⁽⁴⁹⁾ summarizes the general hierarchical approach for determining design inputs for materials based on the philosophy that the level of engineering effort applied to describe the pavement materials should be commensurate with the importance, size, and cost of the project. Material inputs at Level 1 are determined through dedicated laboratory tests, whereas at Level 3 they are

estimated based on experience, with little or no laboratory testing. Level 2 inputs are intermediate to levels 1 and 3 and are estimated using correlations with other common field- or laboratory-measured material properties. AASHTOWare pavement mechanistic-empirical design (PMED) software runs the same analysis irrespective of the input level selected and includes Level 3 default values for PCC materials:

- Intact PCC slabs (high-strength and lean concrete mixes)
- Fractured slabs, FCP (C&S, B&S and RCP)

At Level 1, the elastic modulus of fractured concrete is estimated using non-destructive deflection testing IAW AASHTO T 256⁽⁵⁰⁾. At Level 3, RCP is considered characteristically different from B&S and C&S FCP and is assigned different modulus ranges as shown below⁽⁴⁹⁾:

- Rubblized concrete, RCP 50 ksi to 150 ksi
- C&S and B&S concrete 150 ksi to 1,000 ksi

A study of 100 selected projects nationwide found that the RCP and C&S methods outperformed the B&S method. B&S's poor performance may be due to deficiencies in eliminating the PCC and the reinforcement bond. Based on performance, FCP techniques can be ordered as RCP, C&S, and B&S⁽²⁶⁾. Reasonable predictive models developed by Witczak and Rada⁽²⁶⁾ for the fractured PCC modulus (E_{FCP}) for RCP and C&S is shown in Table 8. Figure 13 shows the frequency distribution of average project FCP modulus (E_{FCP}) investigated by Witczak and Rada, and E_{FCP} 's relation to sound PCC set at 5 ksi ($E_{sound} = 5,000$ psi) as E_R ($E_R = E_{FCP} / E_{sound}$). FCP techniques are generally recommended for pavements in fair to failed conditions ($PCI \leq 50$ ⁽⁵¹⁾), defined as $PCI \leq 55$ IAW ASTM D6433⁽⁷⁾, or pavement remaining life (R_{LX}) ≤ 20 percent.

E_{RCP} 's and $E_{C\&S}$'s frequency distributions of project mean modulus were quite similar, ranging from 400 to 500 ksi, and exhibited a between-project coefficient of variation of approximately 35 percent. The $E_{B\&S}$ was uniformly distributed across a wide range from 250 to 2,750 ksi, clearly reinforcing the assertion that JRCP's B&S process does not uniformly eliminate the PCC and reinforcing steel bond.

Table 8. FCP Modulus Prediction Models.

FCP Type	PCI Deterioration Model	R ²
RCP	$E_{RCP} = 1690 - 15.4 \times \text{Precip}_{\text{average annual, in}} - 17.34 \times \text{Temp}_F + 2.2 \times E_{\text{subgrade, ksi}} + 1.9 \times \text{Spacing}_{\text{Joint, inch}} + 34.7 \times \text{Spacing}_{\text{Crack, inch}}$	0.603
C&S	$E_{C\&S} = -968.39 + 20.34 \times E_{\text{subgrade, ksi}} + 34.89 \times \text{Spacing}_{\text{crack, inch}} + 5.37 \times \text{Seating Load}_{\text{ton}}$	0.776
B&S	-- ¹	-- ¹

¹ Reasonable model could not be developed with an $R^2 > 20$.

² Witczak and Rada⁽²⁶⁾ do not provide models developed using the Light Weight Deflectometer (LWD); LWD-based models could not be found in the literature. State Study 308 is investigating procedures for backcalculating FCP layer moduli. This reference and discussion are included for the overall interest of the study.

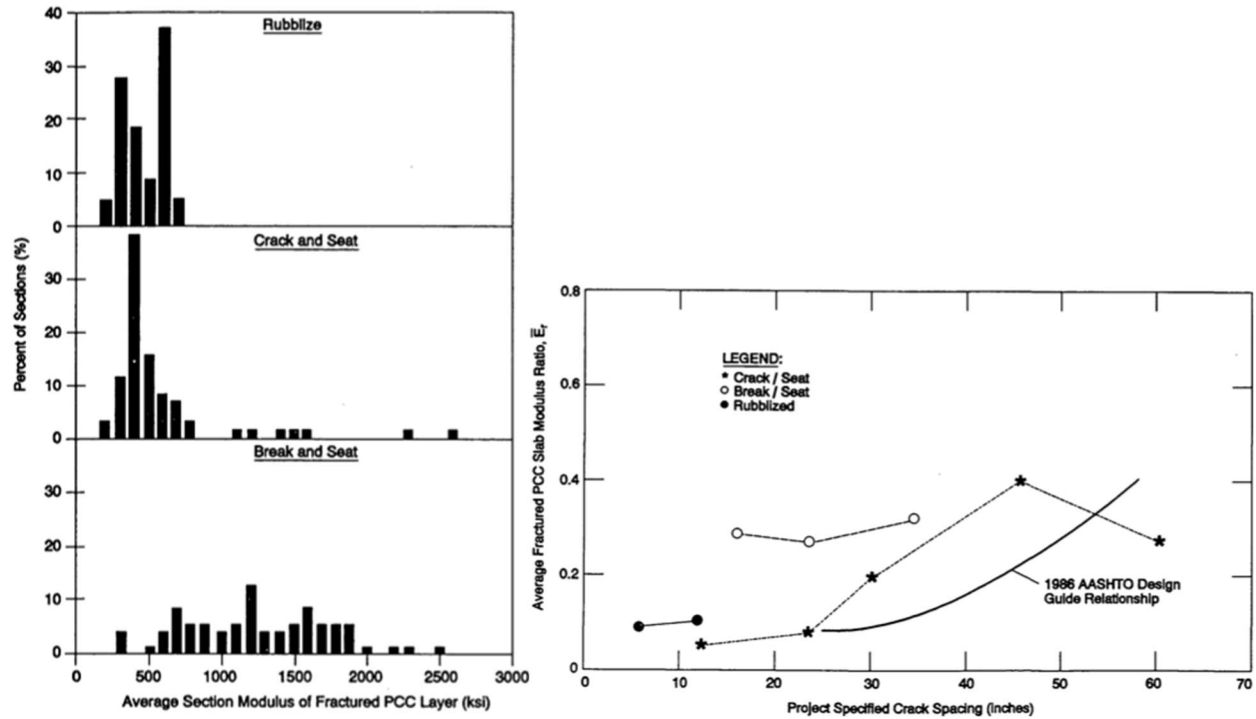


Figure 13. Frequency Distribution of FCP techniques and Modulus Ratio ($E_r = E_{FCP} / E_{sound}$)

Witczak and Rada⁽⁵¹⁾ calculated the layer coefficient of the FCP using Equation 1:

$$a_{FCP} = a_s \sqrt[3]{\frac{E_{FCP}}{E_s}}$$

..... Equation 1

where a_{FCP} is related to the E_{FCP} in terms of the standard arbitrary material whose a_s and E_s were established for AASHTO Road Test materials. Using a dense-graded crushed stone base a_s of 0.14 and E_s of 30,000 psi, the a_{FCP} can be represented as Equation 2⁽⁵¹⁾.

$$a_{FCP} = 0.14 \sqrt[3]{\frac{E_{FCP}}{30,000}} = 0.0045 \sqrt[3]{E_{FCP}}$$

..... Equation 2

For the Huchinping Highway rehabilitation project in China, the average E_{RCP} immediately after rubblization and immediately after HMA overlay were 55 ksi and 200 ksi, respectively, indicating a significant increase in E_{RCP} (+300 percent) after HMA overlay due to overburden pressure from the HMA overlay⁽²²⁾ (Figure 14). The KimShan Broadway rehabilitation project showed similar results (Figure 15). Chen et al⁽²²⁾ recommend that E_{RCP} use the measured value after an HMA overlay, not the value immediately after rubblization, before the HMA overlay.

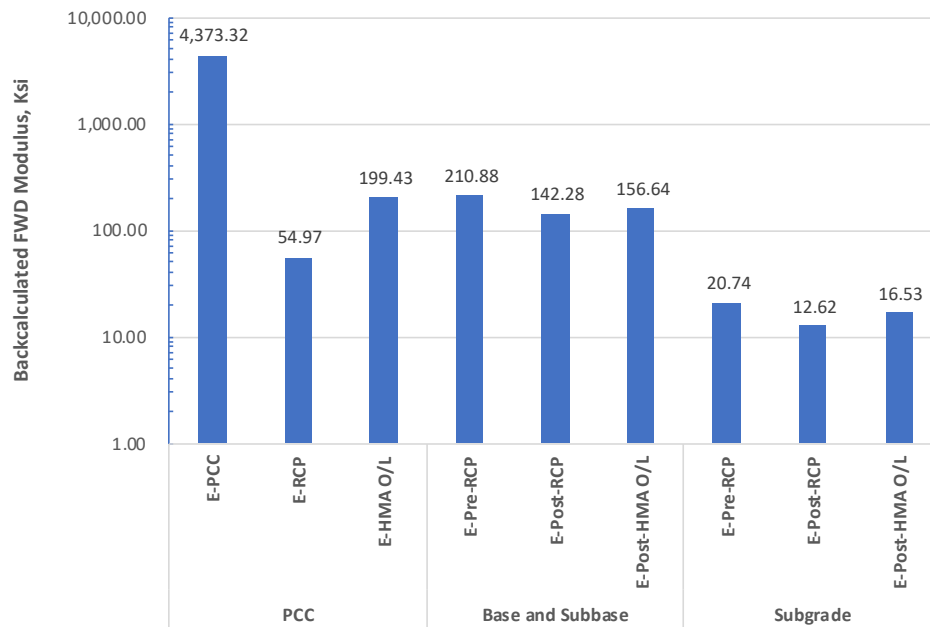


Figure 14. Backcalculated FWD moduli (ksi) for Huchinping Highway.

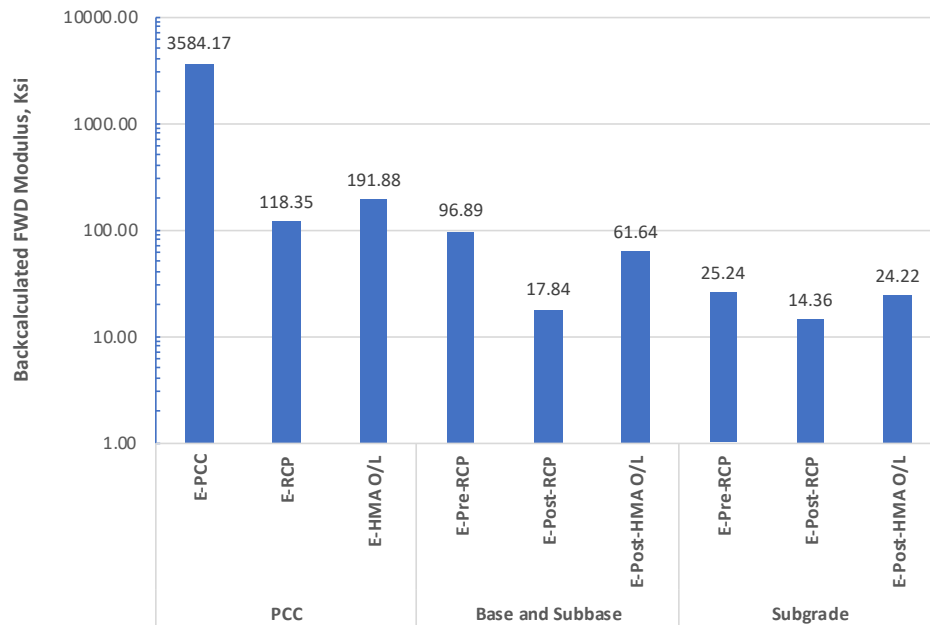


Figure 15. Backcalculated FWD moduli (ksi) for KimShan Broadway.

Though the RCP process is not viable over weak subgrades, many state DOTs have indicated a preference for RCP over C&S or B&S. The process also reduces construction time (enhancing safety) and is less expensive than PCC reconstruction and/or restoration. LaDOTD has estimated that the modulus of RCP exceeds that of a virgin base, leading to improved performance from an equivalent (thickness) pavement structure⁽⁵²⁾. Xin et al⁽⁵³⁾ used the backcalculated moduli to develop mechanistic-empirical (ME) pavement layer thickness designs. The backcalculated moduli for rubblized pavement composite subgrade

(consisting of fly ash-treated base, gravel subbase, and clay roadbed soil) were lower than those of intact pavement composite subgrade (attributed to disturbing the base/subbase/subgrade during rubblization). The Illinois DOT, while demonstrating the use of staged construction with RCP ⁽⁵⁴⁾, noted that the backcalculated modulus of RPC varied significantly due to inherent inconsistencies in particle size and packing.

Successes and Failures of Fractured Concrete Pavements

Sufficient published data demonstrate that RCA is a viable alternative to virgin aggregate for unbound base course construction ⁽⁵⁵⁾. Overall, RCA as an aggregate for unbound base/subbase layers performs excellently. Mississippi DOT State Study 238 (Project No. SPR-1(59) 106002 160000) ⁽⁵⁶⁾ evaluated data from seven RCA tests and reported that RCA can be successfully used as unbound base/subbase layers in Mississippi. Ten-year performance evaluations of seven deteriorating PCC pavements rehabilitated with an HMA overlay over C&S along the I-10 corridor crossing five north Florida counties indicated satisfactory performance ⁽⁵⁷⁾.

Proper drainage before RCP is important for successful overlay performance. Mehta and Choubane ⁽⁵⁸⁾ demonstrated that current RCP techniques are inadequate for RCP over subgrade with a high water table. FWD data analysis indicated that the development of subgrade pore pressures during rubblization was a possible cause of failure. The RCP induced pore-water pressure, mobilizing water from the subgrade into the sandy-clay base beneath the RCP, resulting in poor compaction post-rubblization and severe rutting with localized potholes immediately after the first high-traffic-volume incident. Wolters et al ⁽⁵⁹⁾ observed longitudinal cracking at the pavement construction joint and at the pavement edge of the HMA overlay of RCP. Most distress observed on pavement sections appeared to be related to poor or insufficient drainage, overlay construction issues, or HMA mixture problems. FWD tests on HMA overlays over RCP demonstrated that rehabilitated pavements (HMA over RCP) were structurally comparable to HMA over PCC pavements and performed better on strength parameters ^(60, 61).

The Strategic Highway Research Program (SHRP) Long-Term Pavement Performance Program (LTPP) includes Specific Pavement Studies 6 (SPS-6), which investigates the performance of jointed (JPCP and JRCP) pavements after rehabilitation as C&S, B&S, and RCP test sections ^(62, 63). The RCP test sections did not perform well, and the average back-calculated E_{RCP} for the two sections was 90 ksi and 225 ksi, respectively. Sebesta et al ⁽³⁵⁾ did not identify the causes of low E_{RCP} , which led to considerable fatigue cracking in one of the sections within five years of service. After about nine years of service, both sections had to be rehabilitated due to prevalent distress. The literature review summarizes additional examples ⁽¹⁾.

Layer Coefficient Estimation

FCP Layer Coefficient Estimation Schema

The FCP layer coefficient schema was developed and documented in a technical memorandum submitted to Mississippi DOT in October 2021⁽⁶⁴⁾. Data from Pike County FWD tests were used to demonstrate FCP estimation, and the technical memorandum is attached as Appendix A. The schema is summarized here.

The procedure described by Witczak and Rada⁽⁵¹⁾ (Equation 1 and Equation 2) was used to analyze LFWD deflection data and to determine the RCP layer coefficient. Data from a nationwide study of nearly 500 projects were downselected to 100 projects to better understand HMA overlays of FCP as a rehabilitation technique.

FCP resilient modulus (E_{MR-FCP}) was calculated from FWD data at two falling heights, with three impacts per height. The E_{MR-FCP} for each height was taken as the mean of the three measurements and the stress for each point was defined to be the mean of the two falling height values. The effort produced predictive equations, in terms of present condition index (PCI)⁽⁷⁾, and backcalculated effective moduli⁽⁶⁵⁾ of fractured PCC slabs for each technique. The layer coefficient was calculated using Equation 3.

$$a_{FCP} = a_s \sqrt[3]{\frac{E_{FCP}}{E_s}}$$

..... Equation 3

where a_{FCP} is related to the E_{FCP} in terms of the standard arbitrary material whose a_s and E_s were established for AASHO Road Test materials. Using a dense-graded crushed stone base a_s of 0.14 and E_s of 30,000 psi, the a_{RCP} can be represented as Equation 4⁽⁵¹⁾.

$$a_{FCP} = 0.14 \sqrt[3]{\frac{E_{FCP}}{30,000}} = 0.0045 \sqrt[3]{E_{FCP}}$$

..... Equation 4

Kim et al⁽⁴⁰⁾ report on successfully using the equation developed by Galal et al⁽⁴²⁾ for a_{RCP} as shown in Equation 3. Kim et al⁽⁴⁰⁾ surmise that Equation 5 provides layer coefficients very similar to those derived from AASHO Road Test data.

$$a_{RCP} = 0.0045 \sqrt[3]{E_{RCP}}$$

..... Equation 5

I-55 Pike County Non-destructive Tests

SS 308 describes the test sections and non-destructive tests conducted with the FWD and the GPR and summarizes the backcalculated moduli⁽⁶⁶⁾. NDT data were collected after rubblization and placement of an HMA overlay. The information provided by SS 308 on the test section location, non-destructive testing

equipment and procedures, and backcalculated moduli is attached as Appendix B. Figure 16 shows the project extent from Mississippi State Road (SR) 24 (Station 431+26) to SR 568 (Station 25+00) on I-55, approximately 7.7 miles. NDTs were accomplished in November 2020, and details are summarized in Appendix B. Three hundred forty FWD tests were conducted at 250-ft intervals at three load levels applied at 9,000 lbs., 12,000 lbs., and 16,000 lbs. Deflections measured at different load levels were normalized to 9,000 lbs. GPR data were collected using the GSSI SIR-30 GPR, equipped with an air-horn antenna at 2.0 GHz and a ground-coupled antenna at 900 MHz. GPR data were used to estimate pavement layer thickness after calibration with pavement cores obtained at selected locations (see Figure 17).

Cores were obtained at 12 selected locations to provide ground-truth pavement layer thicknesses and pavement-type information. The cores were used for backcalculation and as ground-truth data for calibrating the GPR measurements. Table 9 provides pavement characterization information gleaned from coring data.

The FWD data helped establish uniformity of the test section, which was split into several uniform subsections using the accumulated difference of the normalized center deflections determined at 9,000 lb. Figure 18 provides the Northbound (a) and Southbound (b) segmentation limits.

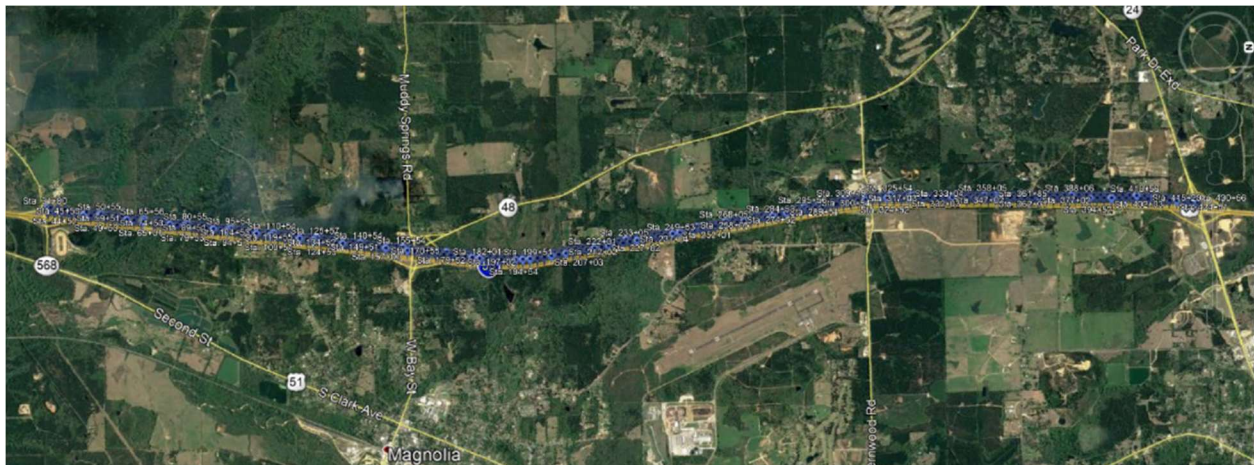


Figure 16. Location of I-55 Pike County NDTs (photo courtesy of SS 308⁽⁶⁶⁾).

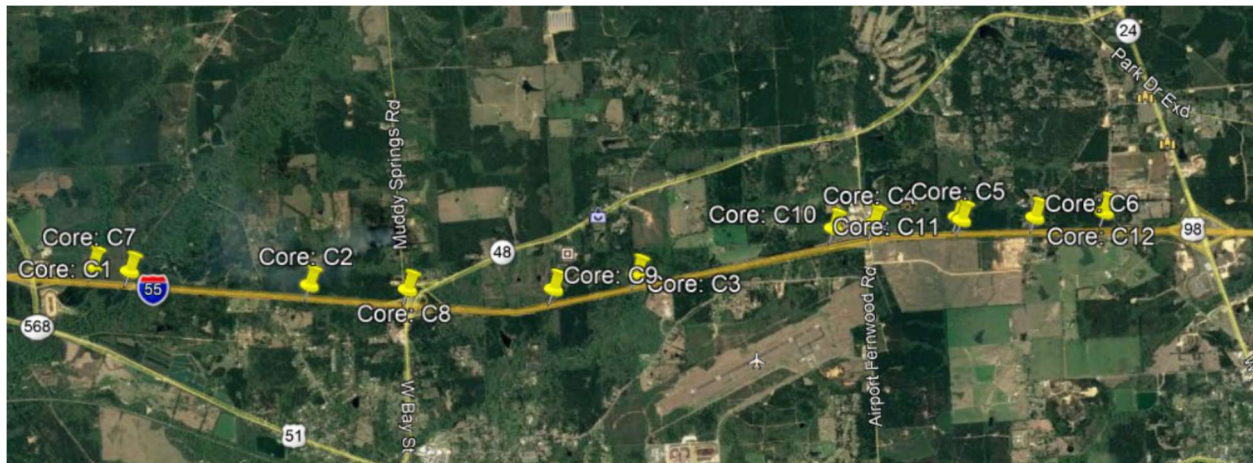


Figure 17. I-55 Pike County Coring locations (photo courtesy of SS 308⁽⁶⁶⁾).

Table 9. I-55 Pike County coring locations and layer type information (data courtesy of SS 308⁽⁶⁶⁾).

Core	Layers					Core Location		Notes
	Thickness (inches)					Direction	Station	
1	Asphalt	C/Conc	Fabric	C/Sand		NB	42+50	C/Conc washed out with coring operations. No sample obtained.
	10.5	13.5	@24	cont.				
2	Asphalt	R/Conc	S/Cement	C/Sand		NB	104+25	Total depth to subgrade was 22 inches.
	8.25	9.75	4	cont.				
3	Asphalt	R/Conc	S/Cement	C/Sand		NB	217+28	
	7.5	9.5	6.5	cont.				
4	Asphalt	C/Conc	C/Sand			NB	300+10	
	11	15	cont.					
5	Asphalt	R/Conc	S/Cement	C/Gravel	C/Sand	NB	333+00	very weak stabilized layer, fell apart
	8	9.5	8.5	6	cont.			
6	Asphalt	C/Conc	Fabric	C/Sand		NB	355+37	Asphalt debonded at 7.5 in depth
	11	11	@22	cont.				
7	Asphalt	R/Conc	S/Cement	C/Gravel	C/Sand	SB	466+00	Depth to subgrade was 23 inches. Thickness of R/Conc and S/Cement was adjusted from original measurements.
	10.5	±8	±5	<1	cont.			
8	Asphalt	C/Conc	Fabric	C/Sand		NB	137+85	Core location changed by MDOT. See MDOT for location.
	10.875	10.5	@22	cont.				
9	Asphalt	R/Conc	S/Cement	C/Gravel	C/Sand	SB	204+87	
	7.5	9.5	6	<1	cont.			
10	Asphalt	C/Conc	Fabric	C/Gravel	C/Sand	SB	306+82	
	11.5	9.5	@21	7	cont.			
11	Asphalt	R/Conc	C/Gravel	C/Sand		SB	345+63	Gravel in treated layer breaking up the sample. switched to auger.
	7.5	9.5	6	@23				
12	Asphalt	C/Conc		Fabric	C/Sand	SB	395+90	
	10.5	22		@33	cont.			

R/Conc: Rubblized Concrete, C/Gravel: Red Clay Gravel, C/Sand: Red Clay Sand (Subgrade), C/Conc: Crushed Concrete, S/Cement: Cement Stabilized Granular Material

(a) Northbound					(b) Southbound				
Section #	From Station	To Station	Length (ft)	# FWD Points	Section #	From Station	To Station	Length (ft)	# FWD Points
1	25+00	40+03	1,503	8	1	25+00	46+24	2,124	10
2	42+50	57+53	1,503	7	2	48+74	56+25	751	4
3	62+51	97+53	3,502	15	3	58+75	91+59	3,284	15
4	100+02	145+05	4,503	19	4	93+76	113+75	1,999	9
5	147+51	202+52	5,501	23	5	116+26	146+26	3,000	13
6	205+01	245+01	4,000	17	6	148+73	148+73	-	1
7	247+52	312+53	6,501	27	7	151+26	168+74	1,748	8
8	315+03	335+05	2,002	9	8	171+23	213+76	4,253	19
9	337+53	360+01	2,248	10	9	212+63	233+74	2,111	8
10	362+51	372+51	1,000	6	10	236+25	311+26	7,501	33
11	375+04	387+52	1,248	6	11	313+76	323+76	1,000	5
12	390+02	397+53	751	4	12	326+24	358+74	3,250	15
13	400+01	410+02	1,001	6	13	361+24	383+75	2,251	10
14	415+02	424+73	971	5	14	386+24	401+25	1,501	7
					15	403+75	424+46	2,071	11

Figure 18. Northbound (a) and Southbound (b) segmentation limits (data courtesy of SS 308⁽⁶⁶⁾).

Given the variability shown in Figure 19, the FCP layer coefficients were estimated for sections where the three layers (HMA, FCP and subgrade) were indicated and the FCP layer approximated the expected layer thickness⁽⁶⁴⁾.

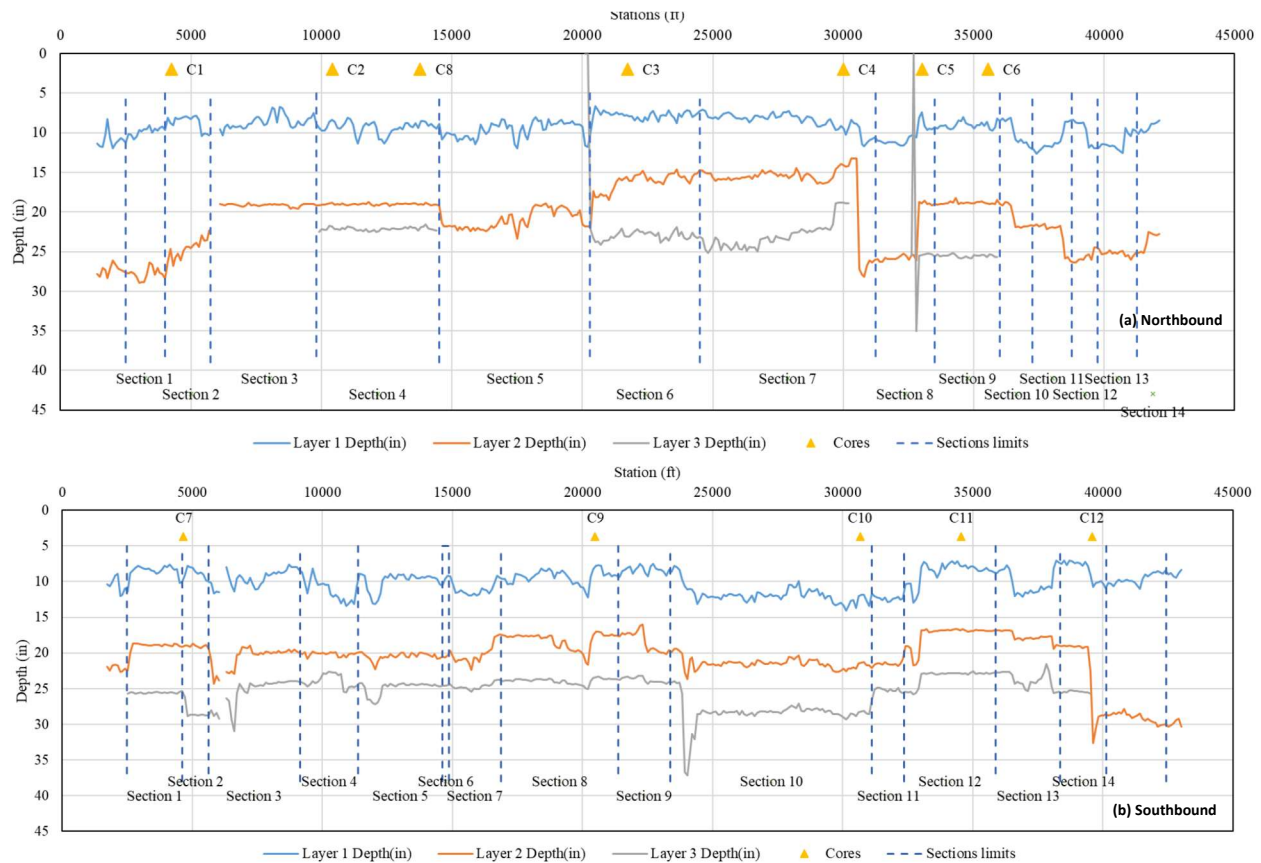


Figure 19. Northbound (a) and Southbound (b) layer profiles (Figure courtesy of SS 308⁽⁶⁶⁾).

Equation 5 was used to estimate the RCP layer coefficient at the mean E_{RCP} and 84th percentile E_{RCP} provided by SS 308. The layer coefficients estimated in Table 10 are within the generally accepted range of 0.14 to 0.30. The north-bound RCP layer coefficient of 0.26 is slightly lower than the south-bound RCP layer coefficient of 0.27. The 84th percentile (conservative) values are 0.21 and 0.22, respectively ⁽⁶⁴⁾.

Table 10. RCP layer coefficient estimated from FWD deflection data for I-55 Pike County.

Section	Station	N	Mean E_{RCP} , ksi	84th percentile E_{RCP} , ksi	Mean a_{RCP}	84th percentile a_{RCP}
Northbound I-55						
3	62+52 to 97+53	15	117.2	67.8	0.22	0.18
4	100+02 to 145+05	19	169.6	102.4	0.25	0.21
5	147+52 to 202+51	23	152.1	70.4	0.24	0.19
6	205+02 to 245+01	17	184.2	135	0.26	0.23
7	247+53 to 312+52	27	337.1	194.4	0.31	0.26
9	337+54 to 360+01	10	192.7	104.3	0.26	0.21
Northbound Mean					0.26	0.21
Southbound I-55						
1	25+00 to 46+24	10	123.2	64.7	0.22	0.18
2	48+75 to 56+24	4	183.3	113.9	0.26	0.22
3	58+76 to 91+59	15	137.3	70.8	0.23	0.19
4	93+77 to 11+374	9	213	113.5	0.27	0.22
5	116+26 to 146+25	13	153.4	99.5	0.24	0.21
6	148+74	1	1650.1	N/A	Outlier; excluded from analysis	
7	151+27 to 168+74	8	238.4	120.6	0.28	0.22
8	171+24 to 213+76	19	139.8	93.8	0.23	0.20
11	313+77 to 323+75	5	437.2	266.1	0.34	0.29
12	326+24 to 358+74	15	199.9	157.4	0.26	0.24
13	361+25 to 383+74	10	205.3	81.1	0.27	0.19
14	386+25 to 401+24	7	550.1	147.3	0.37	0.24
South Bound Mean					0.27	0.22

Mean E_{RCP} and 84th percentile E_{RCP} provided by SS 308 ⁽⁶⁶⁾

I-55 Tate County Non-destructive Tests

SS 308 describes the test sections and non-destructive tests conducted on I-55 in Tate County using the FWD and GPR, and summarizes the backcalculated moduli ⁽⁶⁷⁾. NDTs were conducted

pre- and post-fracturing using the C&S methodology. The project extends from Rifle Range Road (Station 477 +20) to Highway 4 (Station 398 + 43) Southbound near the town of Senatobia in North Mississippi. Pre-rubblization tests were conducted in February 2022, and post-rubblization tests were completed in July 2022. The original construction comprised a 9-inch-thick PCC slab placed over a cement-stabilized material (CSM) layer, which in turn was placed over a clay-gravel layer, which in turn was placed over a CSM layer, all over the subgrade. After rehabilitation, the pavement structure consisted of an HMA layer over an 8-inch-thick aggregate base layer, a 6-inch-thick CSM layer, a 48-inch-thick fill soil, a 9-inch-thick RCP layer, and the subgrade. Post-construction NDTs were performed on top of the aggregate base layer. The extra-thick fill soil layer post-rubblization added complexity to identifying the interfaces between rubblized concrete and fill soil, and between rubblized concrete and support (subgrade). SS 308 used data from test points where the fill layer was absent for analysis ⁽⁶⁷⁾.

Figure 21 shows the GPR-processed pavement layer profiles used for modulus backcalculation using FWD deflection data. FWD and GPR data analysis procedures were summarized earlier and are also detailed in Appendix C ⁽⁶⁷⁾.

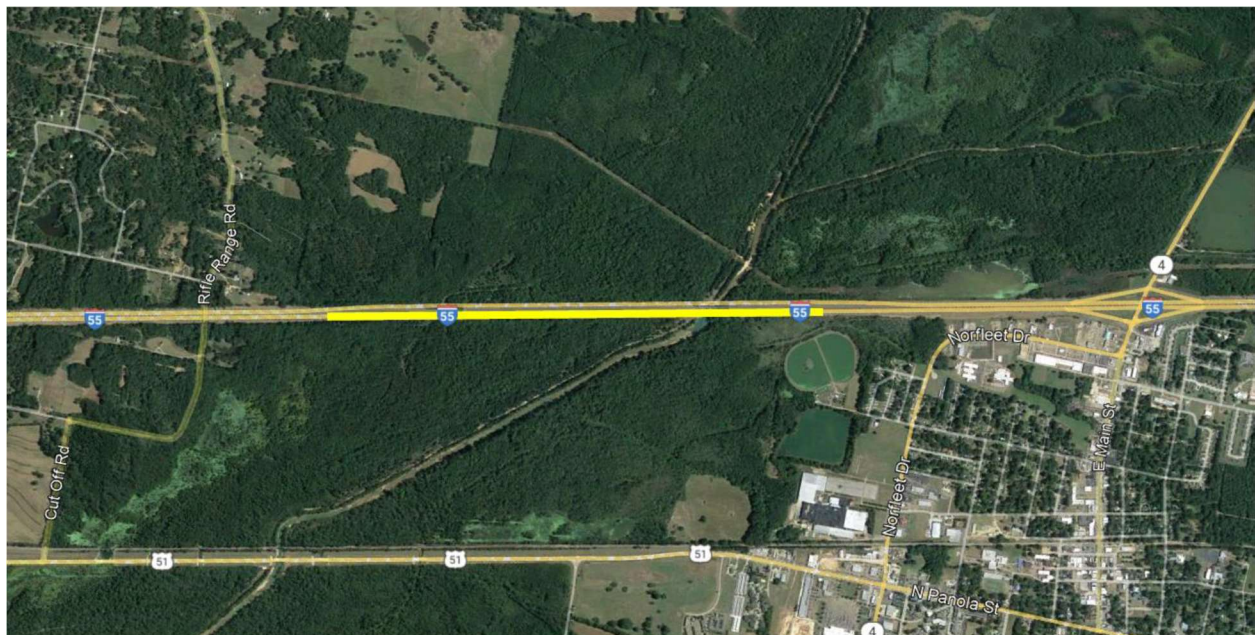


Figure 20. Location of I-55 Tate County NDTs (photo courtesy of SS 308 ⁽⁶⁷⁾).

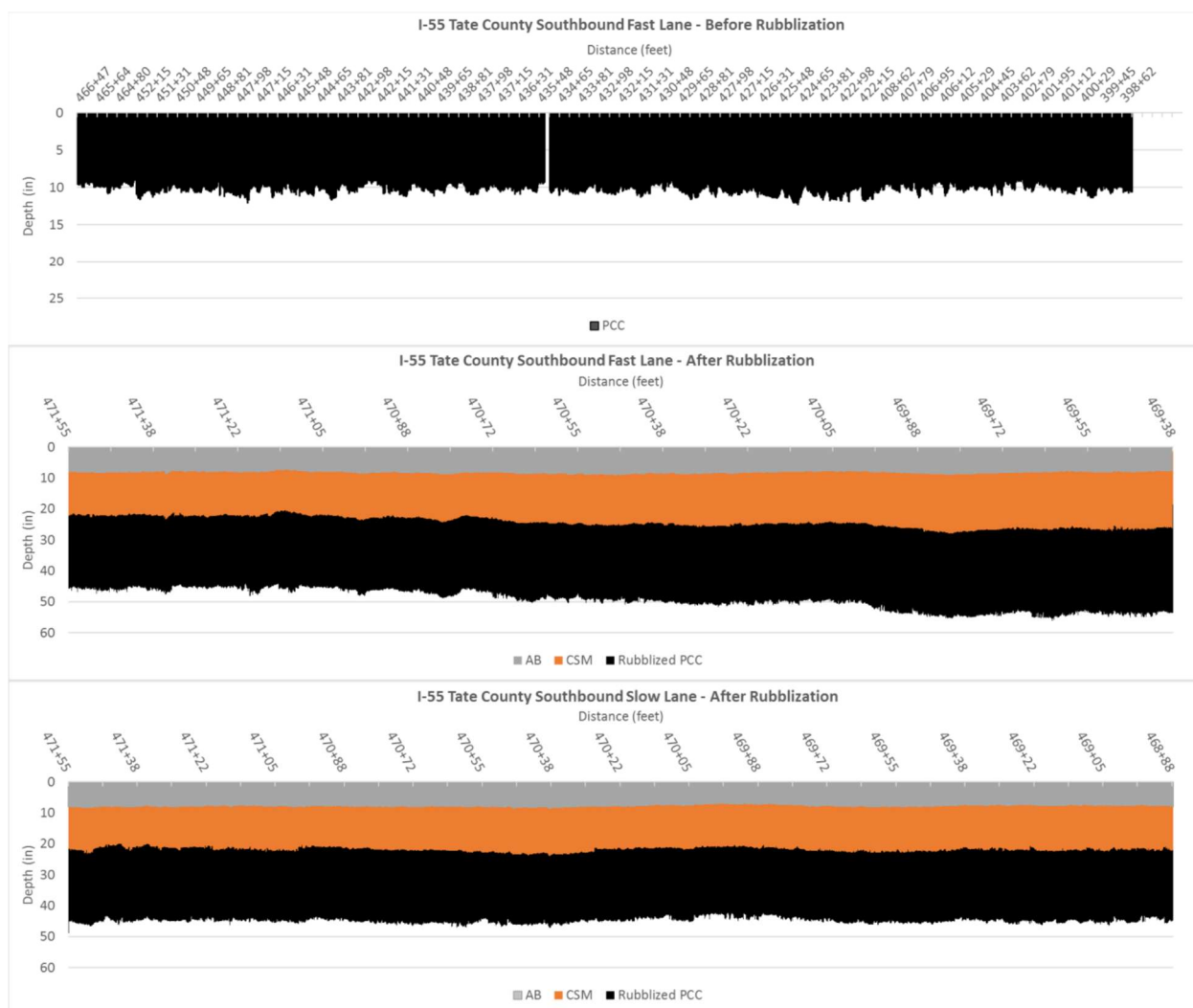


Figure 21. GPR processed pavement layer profiles (courtesy of SS 308⁽⁶⁷⁾).

Table 11. RCP layer coefficient estimates from FWD deflection data for I-55, Tate County.

Lane	Station	N	Mean E_{RCP} , ksi	84th percentile E_{RCP} , ksi	Mean a_{RCP}	84th percentile a_{RCP}
South Bound – Pre-rubblization						
Fast	398+43 to 466+68 (N = 78)	PCC	3,588	1,851	N/A	N/A
Fast		Support	26.9	18.2	N/A	N/A
North Bound Mean					N/A	N/A
South Bound – Post-rubblization						
Fast+Slow	468+88 to 471+55 (N=13) (N _{Slow} =10) (N _{Fast} =3)	AB	64.8	29.3	N/A	N/A
Fast+Slow		CSM	74.3	29.9	N/A	N/A
Fast+Slow		RCP	233.9	97.4	0.28	0.21
Fast+Slow		Support	62.2	49.2	N/A	N/A
South Bound Mean					0.27	0.22

Mean E_{RCP} and 84th percentile E_{RCP} provided by SS 308⁽⁶⁷⁾

NDTs after placement of HMA were not performed; the E_{RCP} is expected to increase after placement of the HMA overlay due to overburden pressure due to the HMA overlay ⁽²²⁾.

SS 308 explains the 84th percentile ⁽⁶⁸⁾ shown in Table 11 (see Appendix D). For conservatism, backcalculated moduli are reported as the mean minus one standard deviation; this is essentially the 16th percentile but is reported as the 84th percentile. The reported 84th (16th) percentile is a conservative and practical measure indicating the percentage of normalized deflections or backcalculated layer moduli below (above, in the case of the 16th percentile) for which 84 percent of the results may be found. The statistical analysis results are presented to provide a better understanding of the overall stiffness of the pavement layers. The 84th percentile is used to account for the inherent variability of the material properties, pavement thickness, and pavement responses, which in turn is useful to be able to compare the pavement structures between lanes ⁽⁶⁸⁾.

I-59 Forrest and Jones Counties Non-destructive Tests

SS 308 describes the test sections and non-destructive tests conducted on I-59 in Forrest and Jones Counties using the FWD and GPR, and summarizes the backcalculated moduli ⁽⁶⁹⁾. NDT data were collected on (1) intact PCC (pre-rubblization), (2) rubblized PCC, (3) first HMA lift, and (4) final HMA overlay.

The information provided by SS 308 on the test section location, non-destructive testing equipment, procedures, and backcalculated moduli is attached as Appendix D. Figure 22 shows the project extent for 9.7 centerline miles on I-59 in Forrest and Jones Counties, Mississippi.

NDTs comprised 1,240 tests across five testing rounds between February 2022 and September 2025. NDTs on intact PCC (February 2022) and after placement of the final HMA layer were deemed reliable. NDTs on rubblized PCC and after placement of the 1st HMA layer were considered unreliable ⁽⁶⁹⁾. FWD NDTs on rubblized concrete produced unreliable results due to irregular, non-uniform PCC pieces produced during the rubblization process. A thin HMA layer further increases data unreliability, as summarized in the SS 308 submittal in Appendix D ⁽⁶⁹⁾.

NDTs deemed unreliable by SS308 are not further discussed here; however, the estimated layer coefficients are provided.

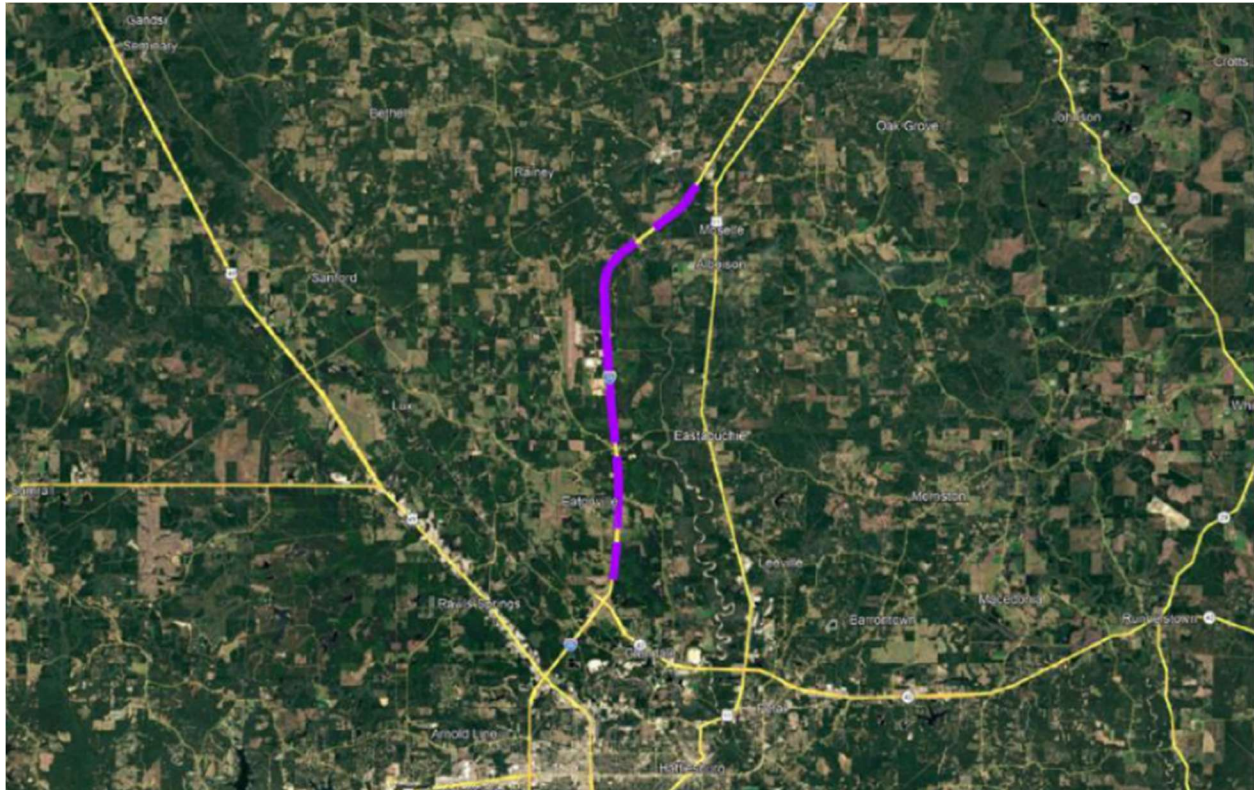


Figure 22. Location of I-59 NDTs in Forrest and Jones Counties (photo courtesy of SS 308⁽⁶⁹⁾)

The original structure had a 10.2-inch-thick PCC over a 5.4-inch cement-stabilized base over the subgrade. Figure 23 shows the pavement layer thickness summary for the northbound lanes estimated using cores extracted in the field. Figure 24 shows similar information for the southbound lanes.

SS 308 examined the FWD data for each testing round for uniformity using the accumulated difference method of the normalized center deflections at 9,000 lb and established subsection boundaries at locations where significant changes in the slope of the accumulated difference occurred ⁽⁶⁹⁾. GPR and coring data further supplemented the FWD data for the segmentation. Using the slope of the accumulated differences, each traveling direction was divided into 13 uniform sections (see Figure 25).

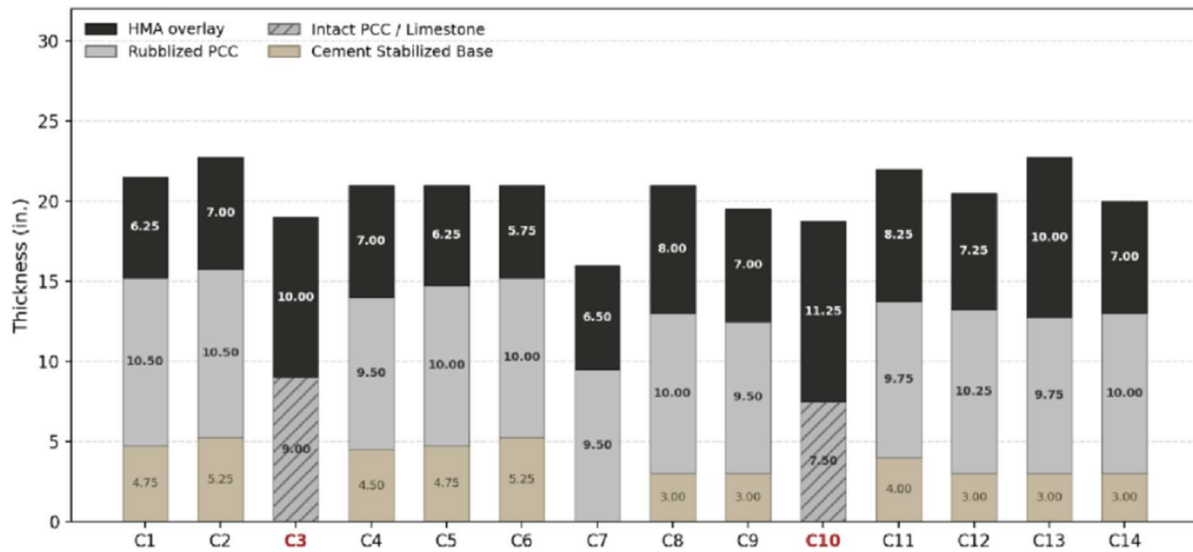


Figure 23. I-59 pavement layer thickness summary for northbound lanes using cores ⁽⁶⁹⁾.

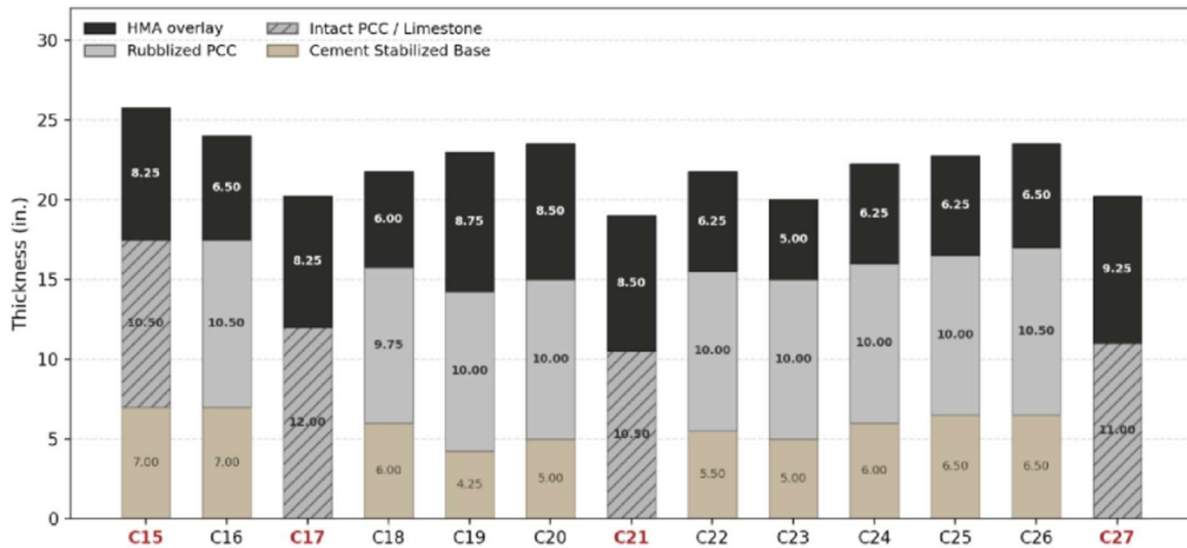


Figure 24. I-59 pavement layer thickness summary for Southbound lanes, based on cores ⁽⁶⁸⁾.

Equation 5 was used to estimate the RCP layer coefficient at the mean E_{RCP} and the 16th-percentile E_{RCP} provided by SS 308 for the final round of testing after the HMA overlay has been placed. The layer coefficients estimated in Table 12 for north and southbound lanes are within the generally accepted range of 0.14 to 0.30. The northbound RCP layer coefficient of 0.28 is slightly higher than the southbound RCP layer coefficient of 0.25. The 16th percentile

(conservative) values are 0.24 and 0.22, respectively. Reporting the 16th percentile, calculated as the geometric mean divided by the Stdev Factor, is consistent with the statistical procedure described in ELMOD documentation by SS 304 ⁽⁶⁹⁾.

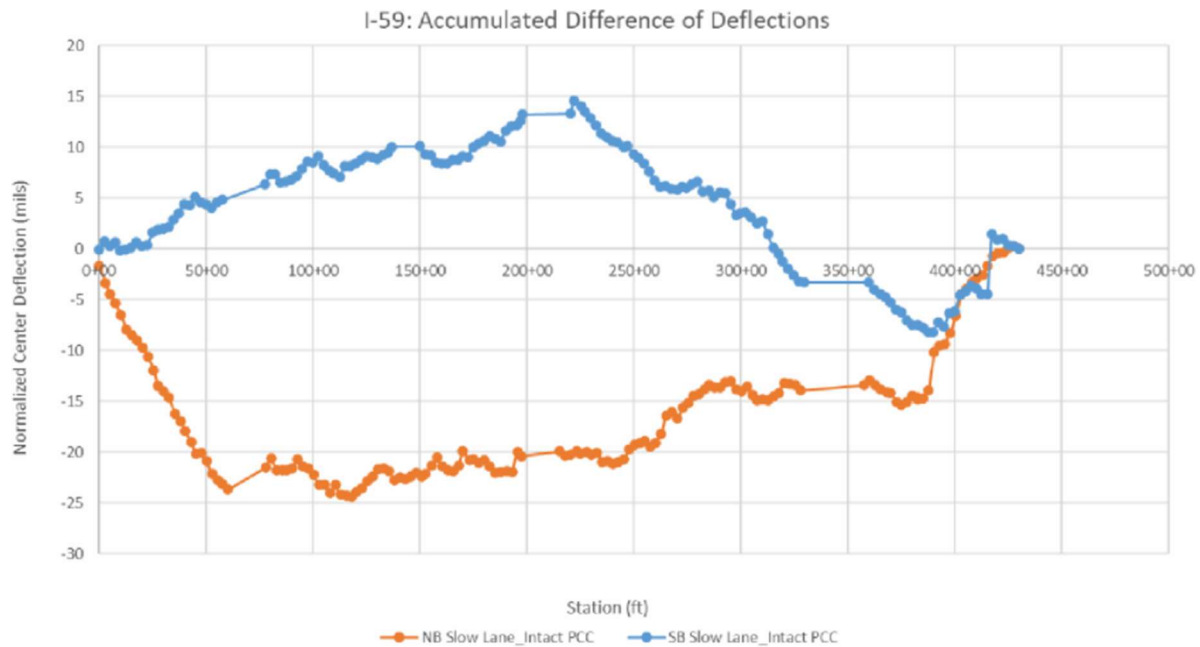


Figure 25. Northbound and southbound segmentation based on FWD deflection data ⁽⁶⁹⁾.

Table 12. RCP layer coefficient estimated from FWD deflection data for I-59 Forrest/Jones Counties.

Section	Station	N	Mean E_{RCP} , ksi	16th percentile E_{RCP} , ksi	Mean a_{RCP}	16th percentile a_{RCP}
Northbound I-59, Final Round						
1	22 – 6,049	25	133	92	0.23	0.20
2	7,816 – 10,835	13	173	120	0.25	0.22
3	11,089 – 17,615	27	155	111	0.24	0.22
4	17,866 – 19,873	9	84	55	0.20	0.17
5	21,383 – 24,400	13	316	189	0.31	0.26
6	24,657 – 29,431	20	228	145	0.27	0.24
7	29,677 – 31,943	10	Incomplete rublization reported		---	---
8	32,204 – 32,698	3	295	269	0.30	0.29
9	35,968 – 37,976	9	337	226	0.31	0.27
10	38,231 – 38,984	4	242	186	0.28	0.26
11	39,239 – 41,248	9	377	277	0.33	0.29

12	41,496 – 42,257	4	179	95	0.25	0.21
13	42,503 – 43,001	3	394	292	0.33	0.30
Northbound Mean					0.28	0.24
Southbound I-59, Final Round						
1	69 – 5,553	23	132	86	0.23	0.20
2	7,616 – 8,617	5	86	67	0.20	0.18
3	8,874 – 11,389	11	206	123	0.27	0.22
4	11,643 – 13,899	10	161	98	0.24	0.21
5	15,158 – 17,421	11	210	146	0.27	0.24
6	17,678 – 19,181	7	340	282	0.31	0.30
7	19,437 – 21,951	3	80	60	0.19	0.18
8	22,201 – 25,222	13	151	99	0.24	0.21
9	25,475 – 29,494	17	186	96	0.26	0.21
10	29,746 – 31,757	9	278	213	0.29	0.27
11	32,006 – 33,015	5	210	136	0.27	0.23
Southbound Mean					0.25	0.22

Table 13. RCP layer coefficient for FWD tests deemed unreliable for I-59 Forrest/Jones Counties.

Section	Station	N	Mean E _{RCP} , ksi	16th percentile E _{RCP} , ksi	Mean a _{RCP}	16th percentile a _{RCP}
Northbound I-59, Round 2a						
12	41,806 – 42,257	15	106	74	0.21	0.19
13	42,282 – 42,995	32	78	36	0.19	0.15
Northbound Mean					0.20	0.17
Southbound I-59, Round 2a						
8	22,435 – 25,126	12	115	82	0.22	0.20
9	25,372 – 29,361	17	162	98	0.25	0.21
10	29,612 – 31,857	10	137	85	0.23	0.20
11	32,105 – 32,850	4	97	67	0.21	0.18
12	36,088 – 40,575	20	126	74	0.23	0.19
13	40,823 – 42,938	10	115	60	0.22	0.18
Southbound Mean					0.22	0.19
Northbound I-59, Round 3						
1	0 – 4,475	173	60	37	0.18	0.15
Northbound I-59, Round 4						
1	0 – 4,512	183	76	59	0.19	0.18
2	6,080 – 9,701	125	47	30	0.16	0.14
South Bound Mean					0.18	0.16

Comparison Between Concrete Fracturing Technologies

Purpose of Statistical Comparisons

Statistical tests differentiate between fracturing techniques and recommend whether layer coefficients are statistically different enough to warrant different layer coefficients for pavement structural design purposes.

Selection of Null and Alternate hypothesis

Setting up the research question and the null and alternative hypotheses ⁽⁷⁰⁾ is as important as conducting the test statistics and arriving at a solution. The research question, null and alternative hypotheses are as follows:

Research Question: Do concrete fracturing techniques have an impact on the FCP layer coefficient?

Null Hypothesis (H_0): Fracturing technique does not affect the estimated FCP layer coefficient ($a_{RCP} = a_{B\&S} = a_{C\&S}$)

Alternate Hypothesis (H_A): Fracturing technique influences the FCP layer coefficient ($a_{RCP} \neq a_{B\&S} \neq a_{C\&S}$)

The selection of the null hypothesis should be such that rejecting it (no effect) leads us to our alternative hypothesis, which we expect to be supported. The tests are conducted at a significance level of 0.05 ($\alpha = 0.05$), meaning there is a 5 percent probability of incorrectly rejecting the null hypothesis (a false positive). Discussion of Type I (false positive) and Type II (false negative) errors is beyond the scope of this document.

The calculated test statistic (P-value), if less than the significance level, indicates that there is sufficient statistical evidence to reject the null hypothesis (H_0).

Comparison of FCP Coefficients from Pike and Tate Counties

Test statistics and P-values were calculated for a two-tailed t-test assuming unequal variances using Microsoft 365 Excel version 2409 (15 October 2024). Test statistics shown in Table 14

indicate that there is no difference between layer coefficients (a_{RCP}) calculated for Pike and Tate Counties; this was expected, as both Pike and Tate County NDTs were conducted on rubblized concrete. When comparing the layer coefficients at the 84th percentile, the P-value (0.7194) indicates that the H_0 cannot be rejected; the calculated layer moduli are statistically similar. Similarly, when comparing the layer coefficients using mean values, the results indicate they are not statistically different ($P=0.3468$). Finally, Table 14 shows the t-test results for layer coefficients estimated from mean values, without averaging the Tate County layer coefficients ($N=13$ instead of $N=2$).

SS 308 could not conduct Pike County NDTs due to scheduling conflicts; therefore, tests on the RCP before placement of the HMA layer are unavailable. NDTs after HMA placement in Tate County are to be conducted by SS 308; data from these tests will help compare the RCP layer coefficient before and after placement of the HMA layer. This exercise will help establish if the placement of an HMA overlay impacts the RCP layer coefficient.

Table 14. Results of two-sample, two-tailed unequal variance t-test at 0.05 significance level (Pike and Tate Counties).

Statistic	a_{FCP} , 84 th percentile values		a_{FCP} , mean values		a_{FCP} , mean values	
	Pike County _{RCP}	Tate County _{C&S}	Pike County _{RCP}	Tate County _{C&S}	Pike County _{RCP}	Tate County _{C&S}
Mean	0.2167	0.2369	0.2649	0.2845	0.2649	0.2870
Variance	0.000802682	0.003570125	0.001566407	0.00034322	0.001566407	0.004994604
Observations	18	2	18	2	18	13
Hypo. Mean Diff.	0		0		0	
df	1		2		17	
t Stat	-0.47180732		-1.21997266		-1.020624828	
P(T<=t) one-tail	0.359676214		0.173403808		0.160868969	
t Critical one-tail	6.313751515		2.91998558		1.739606726	
P(T<=t) two-tail	0.719352428	H₀ supported	0.346807617	H₀ supported	0.321737938	H₀ supported
t Critical two-tail	12.70620474		4.30265273		2.109815578	

Comparison of FCP Coefficients from Pike and Forrest/Jones Counties

Test statistics shown in Table 15 indicate that there is no difference between layer coefficients (a_{RCP}) calculated for Pike and Forrest/Jones Counties. The H_0 could not be rejected because the P-statistic exceeds the significance level. This is true for both the mean and the 84th percentile values.

Comparison of FCP Coefficients from Tate and Forrest/Jones Counties

Test statistics shown in Table 16 indicate that there is no difference between layer coefficients (a_{RCP}) calculated for Tate and Forrest/Jones Counties. The H_0 could not be rejected as the P statistic is larger than the test significance level.

Table 15. Results of two-sample, two-tailed unequal variance t-test at 0.05 significance level (Pike and Forrest/Jones Counties).

Statistic	a_{FCP}, mean values		a_{FCP}, 84th percentile values	
	Pike County_{RCP}	Jones/Forrest_{RCP}	Pike	Jones/Forrest_{RCP}
Mean	0.26488	0.263704	0.216669	0.231856
Variance	0.001566	0.001513	0.000803	0.001459
Observations	18	25	18	25
Hypo. Mean Diff.	0		0	
df	36		41	
t Stat	0.096844		-1.49673	
P(T<=t) one-tail	0.461694		0.071061	
t Critical one-tail	1.688298		1.682878	
P(T<=t) two-tail	0.923388	H₀ supported	0.142122	H₀ supported
t Critical two-tail	2.028094		2.019541	

Table 16. Results of two-sample, two-tailed unequal variance t-test at 0.05 significance level (Tate and Forrest/Jones Counties).

Statistic	a_{FCP}, mean values	
	Tate County_{C&S}	Jones/Forrest_{RCP}
Mean	0.287036	0.263704
Variance	0.004995	0.001513
Observations	13	25
Hypo. Mean Diff.	0	
df	16	
t Stat	1.106374	
P(T<=t) one-tail	0.142461	
t Critical one-tail	1.745884	
P(T<=t) two-tail	0.284922	H₀ supported
t Critical two-tail	2.119905	

Summary and Conclusions

Study Summary

Portland Cement Concrete (PCC) pavements remain a valuable asset after their design life. The most effective method to reuse PCC pavements is to convert them into future pavements as base/subbase layers. The Environmental Protection Agency's (EPA) Waste Reduction Model (WARM) estimates

greenhouse gas (GHG) emissions savings of approximately 0.04 metric tons CO₂e per short ton of concrete debris not transported to a landfill. Using recycled in-place concrete to replace a virgin aggregate base/subbase can achieve GHG emissions savings of 0.01 metric tons of CO₂e per short ton of virgin aggregate. PCC fracturing (FCP) methods for recycling generally include (1) rubblization (RCP) with steel to concrete bond ruptured, (2) crack-and-seat (C&S) for PCC without steel reinforcement, and (3) break-and-seat (B&S) for PCC with steel reinforcement.

FCP structural coefficients, as shown below, are the most preferred values for the design of HMA overlays:

- RCP: 0.14 - 0.30
- C&S: 0.20 - 0.35
- B&S: 0.20 - 0.35

The AASHTO AASHTOWare Pavement ME Design ⁽⁴⁹⁾ approaches ME Pavement design hierarchically by characterizing pavement materials commensurate with the importance, size, and cost of the project. Material inputs at Level 1 are determined using dedicated laboratory tests, while at Level 3, these are estimated based on experience with little or no laboratory testing. Level 2 inputs are intermediate to levels 1 and 3 and are estimated using correlations with other common field- or laboratory-measured material ⁽⁷¹⁾. MEPDG Manual of Practice recommended FCP modulus values are:

- RCP 50 ksi to 150 ksi
- C&S, B&S 150 ksi to 1,000 ksi

The scheme developed to estimate FCP layer coefficients is based on a literature search reported by Saeed et al ⁽¹⁾ and summarized in this report as well. Equation 6 shows the mathematical representation of the FCP layer coefficient schema.

$$a_{RCP} = 0.0045 \sqrt[3]{E_{RCP}}$$

..... Equation 6

Overall, HMA overlays of FCP, especially RCP, have performed well when the PCC pavement is properly fractured and the HMA overlay is designed and constructed appropriately. The performance of HMA overlays on FCP has been well documented as excellent, with no visible load-related distresses, based on field condition surveys.

Statistical analyses conducted using limited NDT data from three field test sections indicate that there are no statistically significant differences between fracturing techniques employed during these projects, such as RCP and C&S. MDOT has no specifications for B&S, and no test sections were available for field NDTs.

Study Conclusions and Recommendations

SS312 conclusions are as follows:

- Tests on B&S sections could not be conducted as no test sections were available. MDOT does not have a specification for B&S FCP.
- Estimated layer coefficient for Pike ($a_{RCP-Pike}$, 0.22), Tate ($a_{C\&S-Tate}$, 0.21), and Forrest/Jones ($a_{RCP-Forrest/Jones}$, 0.24) counties are similar to those reported in the literature (0.01 - 0.30)
- Estimated layer coefficient for Pike ($a_{RCP-Pike}$), Tate ($a_{C\&S-Tate}$), and Forrest/Jones ($a_{RCP-Forrest/Jones}$) counties are statistically similar. This also supports the assertion that overburden pressures from the thin overlay do not significantly affect estimated layer coefficients, as NDTs in Pike and Forrest/Jones counties were conducted after placement of the HMA rehabilitation.
- NDTs could not be conducted reliably on the Forrest/Jones counties test section representing RCP before and after PCC fracturing. Data after placement of the HMA were considered in this study. The a_{RCP} after HMA placement is slightly higher but is not statistically significant at the 0.05 significance level.
- Estimated layer coefficient for Tate ($a_{RCP-Tate}$) county for slow and fast lanes is statistically similar.
- Current limited NDT data indicate that the FCP layer coefficient before and after placement of the HMA is statistically similar.

SS 312 recommendations are as follows:

- MDOT should develop B&S specifications if PCC with steel reinforcement is expected to be fractured.
- MDOT should use the layer coefficient estimated using the schema prepared for SS 312, as the results are within the generally accepted norms.
- Additional tests should be conducted to conclusively establish if FCP layer coefficients are statistically similar before and after placement of the HMA overlay.

Implementation and Technology Transfer Plan

The technology transfer plan, prepared in consultation with the Mississippi DOT Research Division, focuses on “training the trainers” to disseminate study results to DOT districts. At completion of the study, a meeting should be held to discuss the developed FCP layer coefficient estimation schema and estimated layer coefficients. During this meeting, layer coefficient estimates should be demonstrated using field NDT data.

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Appendix A

Layer Coefficient Estimation Schema

MDOT State Study 312 — Recycling PCC Pavements using Rubblization

Final Report

Preliminary Rubblized Concrete Pavement Layer Coefficient using I-55 Pike County Non-Destructive Tests

Background

Literature review conducted for State Study 312 demonstrated that the most effective method to reuse Portland Cement Concrete (PCC) pavements is to convert them into future pavements as base/subbase layers⁽¹⁾. Generally used PCC slab fracturing (fractured concrete pavement, FCP) methods include the following:

- Rubblization with steel to concrete bond ruptured (rubblized concrete pavement, RCP)
- Crack-and-seat (C&S) for PCC without steel reinforcement
- Break-and-seat (B&S) for PCC with steel reinforcement

Hot mix asphalt (HMA) over FCP pavement performance is influenced by the project selection besides the fracturing process, the construction procedure, the pavement design process, the pavement materials and, finally, the traffic⁽²⁾. The candidate project should not include PCC pavements that have been cracked and seated in the past; PCC pavements constructed on relatively soft soils with $E_{\text{Subgrade}} \leq 3$ ksi should not be a candidate for FCP. Concrete pavements on subbases with $E_{\text{Subbase}} \leq 7$ ksi should not be rubblized.

RCP Structural Factor for I-55 Pike County

Mechanistic-Empirical Pavement Design Guide (MEPDG) Manual of Practice⁽³⁾ summarizes the general hierarchical approach for determining design inputs for materials based on the philosophy that the level of engineering effort applied to describe the pavement materials should be commensurate with the importance, size and cost of the project. A similar approach is adopted for State Study 312. Material inputs at Level 1 are determined using dedicated laboratory tests while at Level 3 these are estimated based on experience with little or no laboratory testing. Level 2 inputs are a combination of Levels 1 and 3 and are estimated using correlations with other commonly field- or laboratory-measured material properties.

A national survey with 58 responses reported by West et al⁽⁴⁾ in 2002 indicated that the 1993 AASHTO Guide recommended FCP structural coefficients (shown below) were the most preferred method to establish the design layer coefficients.

- RCP: 0.14 - 0.30
- C&S: 0.20 - 0.35
- B&S: 0.20 - 0.35

Barstis et al⁽⁵⁾ summarize State Study 105 that determined a layer coefficient of 0.14 for rubblized CRCP for use in design of overlay thickness over rubblized CRCP. State Study 105 used a Dynaflect and a falling weight deflectometer (FWD) to obtain deflection measurements. Kim et al⁽⁶⁾ report on a study conducted to characterize E_{RCP} from seven test sections in Iowa with the objective to establish RCP layer modulus and

layer coefficients. The average rubblized PCC layer coefficient was estimated to be 0.19 and backcalculated E_{RCP} were found to be 539 MPa (78 ksi). Wisconsin DOT⁽⁷⁾ reports on a survey of state DOTs to determine layer coefficients values used to characterize C&S and RCP layers. Of the 14 respondents, the RCP layer coefficient ranged from 0.14 to 0.29 with a mean of 0.21 and a median of 0.20.

Galal et al⁽⁸⁾ report on an Indiana DOT contract to apply the RCP to rehabilitate a portion of US-41 in Benton County. The pavement sections, evaluated before and after HMA overlay to estimate the AASHTO layer coefficient of RCP, helped establish the project RCP layer coefficient at 0.25. The results of this study facilitated raising the RCP layer coefficient from 0.20 to 0.22 which was still relatively conservative compared to the US-41 estimated value. The reported limited-in-scope study did not include PCC slabs of different thicknesses.

Experience Based RCP Layer Coefficient

RCP layer coefficient of 0.20 to 0.25 is indicated based on previous experience at DOTs including Mississippi. Design memorandum for I-59 from the Forrest/Lamar County Line to 0.3 miles south of Moselle Forrest & Jones Counties uses an RCP layer coefficient of 0.25⁽⁹⁾.

NDT Based RCP Layer Coefficient

Witczak and Rada⁽¹⁰⁾ undertook a nationwide study of nearly 500 projects which were down selected to 100 projects to develop a better understanding of HMA overlays of FCP as a rehabilitation technique. The effort produced performance predictive equations, in terms of present condition Index (PCI)⁽¹¹⁾, and backcalculated effective moduli⁽¹²⁾ of fractured PCC slabs for each technique.

Witczak and Rada⁽¹³⁾ calculated the layer coefficient of the FCP using Equation 1.

$$a_{FCP} = a_s \sqrt[3]{\frac{E_{FCP}}{E_s}}$$

..... Equation 1

where a_{FCP} is related to the E_{FCP} in terms of the standard arbitrary material whose a_s and E_s were established for AASHTO Road Test materials. Using a dense-graded crushed stone base a_s of 0.14 and E_s of 30,000 psi, the a_{FCP} can be represented as Equation 2⁽¹³⁾.

$$a_{FCP} = 0.14 \sqrt[3]{\frac{E_{FCP}}{30,000}} = 0.0045 \sqrt[3]{E_{FCP}}$$

..... Equation 2

Kim et al⁽⁶⁾ report on successfully using the equation developed by Galal et al⁽⁸⁾ for a_{RCP} as shown in Equation 3. Kim et al⁽⁶⁾ surmise that Equation 3 provides layer coefficients that are very similar to those derived using AASHTO Road Test data.

$$a_{RCP} = 0.0045 \sqrt[3]{E_{RCP}}$$

..... Equation 3

Using Equation 3, RCP layer coefficients for the Pike County sections are shown in Table 1. Discussion of FWD tests and protocols is beyond the scope of this document and are available elsewhere⁽¹⁴⁾.

Table 1. RCP layer coefficient estimated from FWD deflection data.

Section	Station	N	Mean E_{RCP} , ksi	84th percentile E_{RCP} , ksi	Mean a_{RCP}	84th percentile a_{RCP}
North Bound I-55						
3	62+52 to 97+53	15	117.2	67.8	0.22	0.18
4	100+02 to 145+05	19	169.6	102.4	0.25	0.21
5	147+52 to 202+51	23	152.1	70.4	0.24	0.19
6	205+02 to 245+01	17	184.2	135	0.26	0.23
7	247+53 to 312+52	27	337.1	194.4	0.31	0.26
9	337+54 to 360+01	10	192.7	104.3	0.26	0.21
North Bound Mean					0.26	0.21
South Bound I-55						
1	25+00 to 46+24	10	123.2	64.7	0.22	0.18
2	48+75 to 56+24	4	183.3	113.9	0.26	0.22
3	58+76 to 91+59	15	137.3	70.8	0.23	0.19
4	93+77 to 11+374	9	213	113.5	0.27	0.22
5	116+26 to 146+25	13	153.4	99.5	0.24	0.21
6	148+74	1	1650.1	N/A	Outlier; excluded from analysis	
7	151+27 to 168+74	8	238.4	120.6	0.28	0.22
8	171+24 to 213+76	19	139.8	93.8	0.23	0.20
11	313+77 to 323+75	5	437.2	266.1	0.34	0.29
12	326+24 to 358+74	15	199.9	157.4	0.26	0.24
13	361+25 to 383+74	10	205.3	81.1	0.27	0.19
14	386+25 to 401+24	7	550.1	147.3	0.37	0.24
South Bound Mean					0.27	0.22

Summary and Conclusions

I-55 Pike County RCP layer coefficient are within the generally accepted range of 0.14 to 0.30. The north bound RCP layer coefficient of 0.26 is slightly lower than the south bound RCP layer coefficient of 0.27. The 84th percentile (conservative) values are 0.21 and 0.22, respectively.

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Appendix B

I-55 Pike County Non-Destructive Tests

MDOT State Study 312 — Recycling PCC Pavements using Rubblization

Final Report

STUDY 308 FWD TESTING AND BACKCALCULATION OF RUBBLIZED AND FRACTURED SLAB PCC PAVEMENT MODULI FOR MDOT –I-55- PIKE COUNTY

Introduction

ARA has performed nondestructive testing (NDT) on approximately 7.7 centerline miles of I-55 from station 25+00 (intersection with SR 568) to station 431+26 (intersection with SR 24). The project scope included Falling Weight Deflectometer (FWD) testing, Ground Penetrating Radar (GPR) testing, coring, and backcalculation analysis of the pavement layer moduli. Figure 1 presents the overall project location.

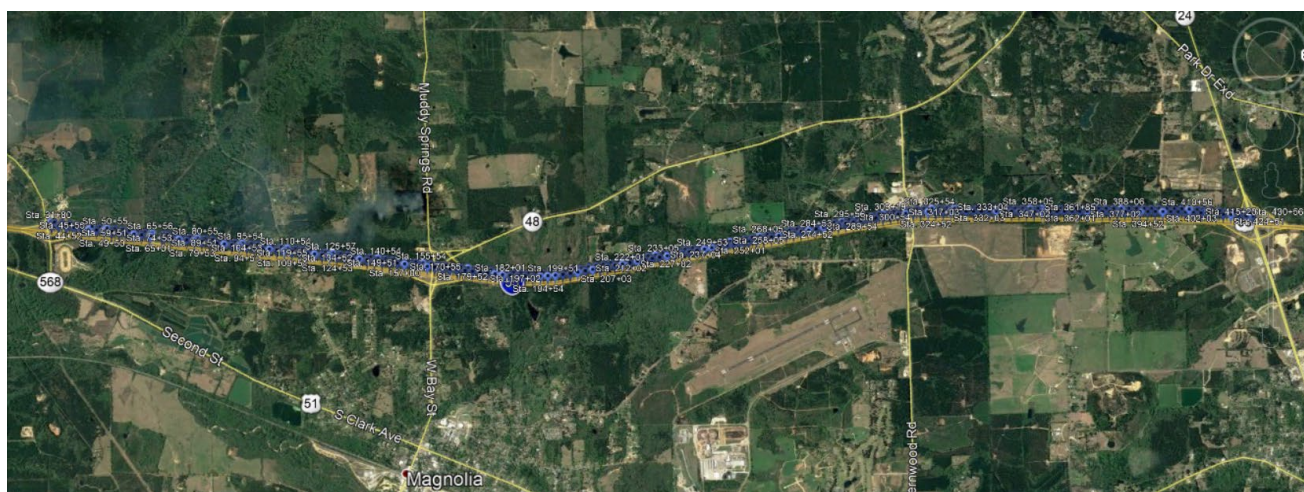


Figure 1. Overall Project Location

Falling Weight Deflectometer Testing

FWD testing was performed on the outer lane in both traveling directions on I-55 in Pike County, Mississippi between stations 25+00 to 431+26. FWD testing was performed on November 3rd 2020. Each FWD test location was conducted at 250-ft intervals, and 340 FWD stations were tested. At each tested station three FWD load levels were applied 9,000 lbs., 12,000 lbs., and 16,000 lbs. Figure 2 and Figure 3 illustrates the FWD load normalized center deflection corresponding to a FWD load level of 9,000 lbs. on the northbound and the southbound traveling directions, respectively. In addition, Table 1 presents the statistical summary of the load normalized center deflection data corresponding to a FWD load level of 9,000 lbs.

From Table 1, it is important to mention that section 6 on the southbound traveling direction only shows one single FWD station. At this location, the backcalculation results showed significantly higher moduli values than the sections 5 and 7, so that it couldn't be combined with neither of them. It is not a common practice to have a single FWD point on a uniform section but in this unique case, it was preferred to report this point individually to reflect the difference in pavement responses.

Table 1. Center Deflection Statistical Summary

Direction	Lane	Section	Station (ft)		Normalized Center Deflections (mils)		
			From	To	Average	St. Deviation	84th Percentile
NB	Slow	1	25+00	40+03	3.37	1.18	4.55
NB	Slow	2	42+50	57+52	3.96	1.50	5.46
NB	Slow	3	62+52	97+53	4.02	1.85	5.87
NB	Slow	4	100+02	145+05	3.65	0.70	4.36
NB	Slow	5	147+52	202+51	3.68	1.25	4.93
NB	Slow	6	205+02	245+01	4.58	0.75	5.33
NB	Slow	7	247+53	312+52	4.02	0.82	4.83
NB	Slow	8	315+03	335+05	2.94	1.40	4.34
NB	Slow	9	337+54	360+01	3.97	0.80	4.77
NB	Slow	10	362+51	372+51	2.78	1.33	4.11
NB	Slow	11	375+04	387+52	2.70	0.74	3.43
NB	Slow	12	390+02	397+52	4.13	0.58	4.71
NB	Slow	13	400+01	410+02	3.69	1.18	4.87
NB	Slow	14	415+03	424+72	3.61	0.26	3.88
SB	Slow	1	25+00	46+24	5.71	1.10	6.81
SB	Slow	2	48+75	56+24	4.21	1.50	5.71
SB	Slow	3	58+76	91+59	4.88	1.87	6.75
SB	Slow	4	93+77	113+74	3.55	0.79	4.33
SB	Slow	5	116+26	146+25	3.94	0.81	4.75
SB	Slow	6	148+74	148+74	2.17*	N/A	N/A
SB	Slow	7	151+27	168+74	3.80	1.76	5.55
SB	Slow	8	171+24	213+76	5.27	1.12	6.38
SB	Slow	9	216+24	233+74	6.88	1.28	8.16
SB	Slow	10	236+25	311+25	4.76	0.94	5.70
SB	Slow	11	313+77	323+75	2.48	0.25	2.73
SB	Slow	12	326+24	358+74	4.51	0.95	5.46
SB	Slow	13	361+25	383+74	3.60	1.23	4.84
SB	Slow	14	386+25	401+25	3.85	1.94	5.80
SB	Slow	15	403+76	424+46	3.33	0.81	4.14

* Average of 1 point

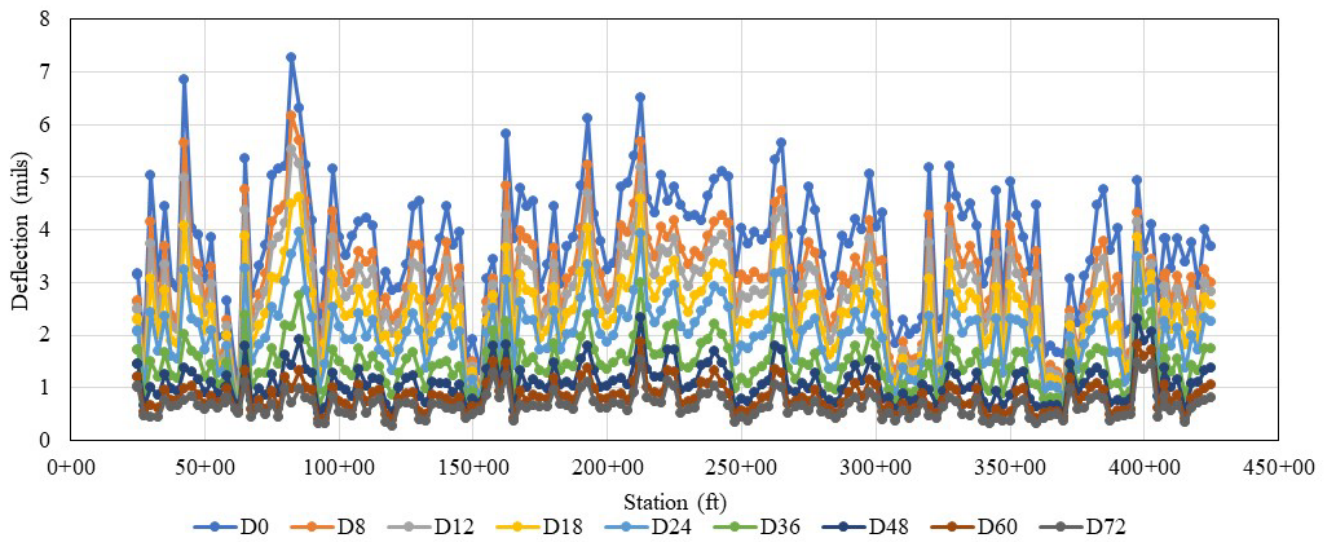


Figure 2. I-55 NB Normalized Deflections at 9,000 lbs.

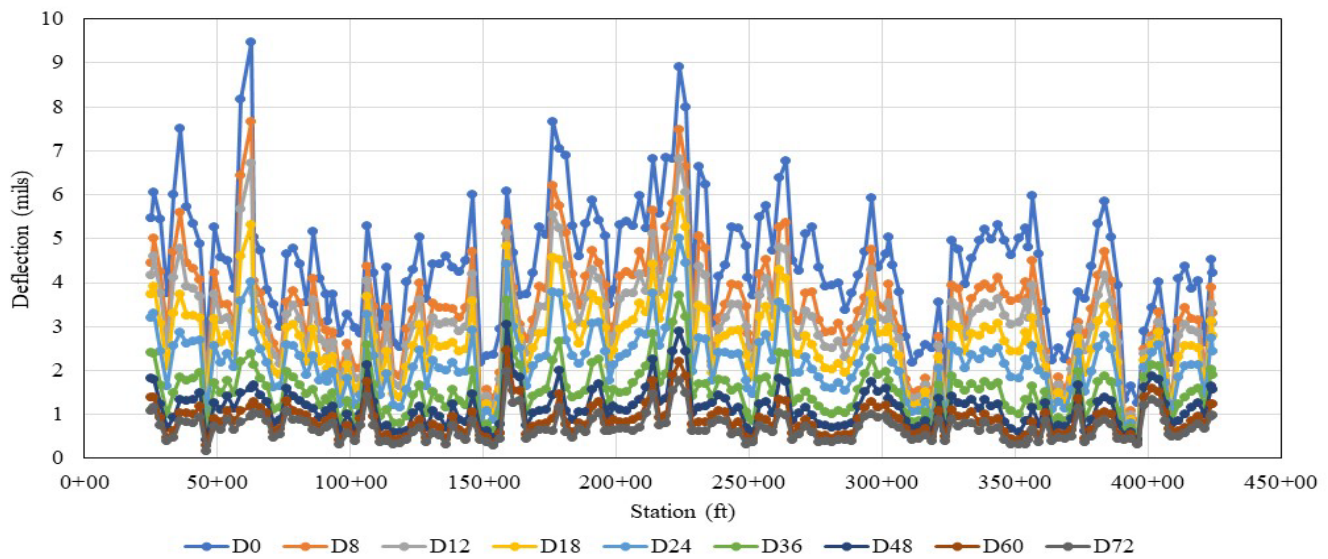


Figure 3. I-55 SB Normalized Deflections at 9,000 lbs.

Ground Penetrating Radar Testing

Continuous GPR testing was performed on the same pavement sections on I-55. GPR data was then processed to estimate the pavement layer thickness after being calibrated with ground truth data based on pavement cores. The testing was performed using the GSSI SIR-30 GPR. Two antennas were utilized in the data collection: an air-horn antenna with a frequency of 2.0 GHz and a ground-coupled antenna with a frequency of 900 MHz. The Air-horn antenna provides high accuracy measurements of the surface layer thickness, while the ground-coupled antenna provides measurements at up to 2-ft deep. Figure 4 presents the GPR equipment utilized for this project, while Figure 5 presents a screenshot from the raw GPR data on I-55 northbound.



Figure 4. GPR Unit with Double Antennas

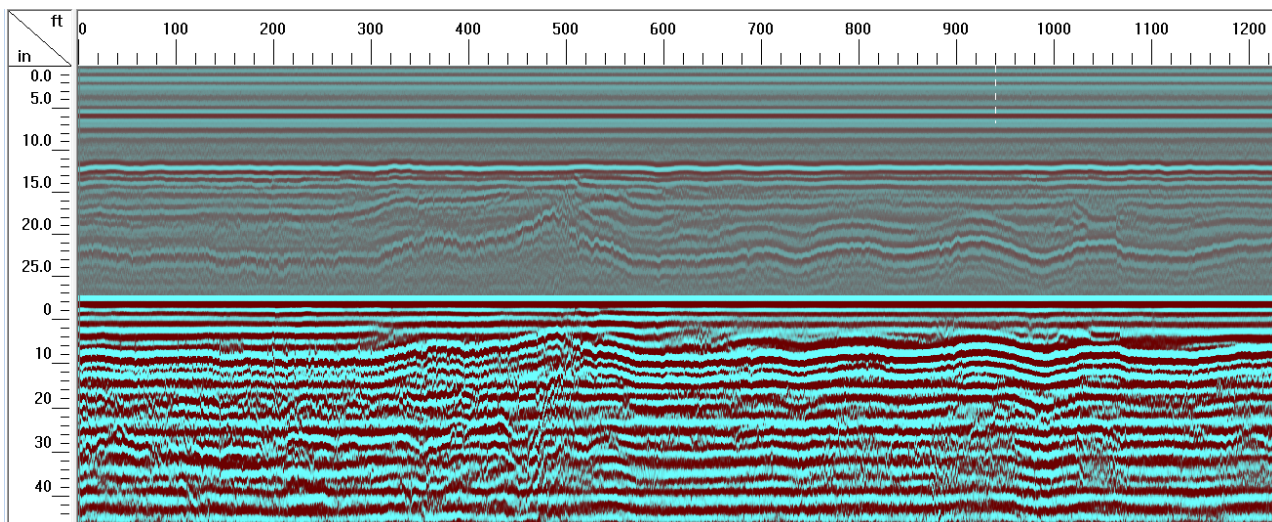


Figure 5. Raw GPR Data Example from I-55 Northbound

Field Coring

Burns Cooley Dennis, Inc. (BCD) performed pavement coring on 12 different locations to determine pavement layer thicknesses and types. The cores were used for the backcalculation analysis and as ground truth data in calibration process of the GPR measurements. Table 2 provides the layer thickness and material type information corresponding to the 12 pavement cores, and Figure 6 presents an example of the core photos provided by BCD. Appendix A shows all details pertaining to the pavement coring effort.

Table 2. Coring Thickness and Layer Type Information

Core	Layers					Core Location		Notes
	Thickness (inches)					Direction	Station	
1	Asphalt	C/Conc	Fabric	C/Sand		NB	42+50	C/Conc washed out with coring operations. No sample obtained.
	10.5	13.5	@24	cont.				
2	Asphalt	R/Conc	S/Cement	C/Sand		NB	104+25	Total depth to subgrade was 22 inches.
	8.25	9.75	4	cont.				
3	Asphalt	R/Conc	S/Cement	C/Sand		NB	217+28	
	7.5	9.5	6.5	cont.				
4	Asphalt	C/Conc	C/Sand			NB	300+10	
	11	15	cont.					
5	Asphalt	R/Conc	S/Cement	C/Grave 1	C/Sand	NB	333+00	very weak stabilized layer, fell apart
	8	9.5	8.5	6	cont.			
6	Asphalt	C/Conc	Fabric	C/Sand		NB	355+37	Asphalt debonded at 7.5 in depth
	11	11	@22	cont.				
7	Asphalt	R/Conc	S/Cement	C/Grave 1	C/Sand	SB	466+00	Depth to subgrade was 23 inches. Thickness of R/Conc and S/Cement was adjusted from original measurements.
	10.5	±8	±5	<1	cont.			
8	Asphalt	C/Conc	Fabric	C/Sand		NB	137+85	Core location changed by MDOT. See MDOT for location.
	10.875	10.5	@22	cont.				
9	Asphalt	R/Conc	S/Cement	C/Grave 1	C/Sand	SB	204+87	
	7.5	9.5	6	<1	cont.			
10	Asphalt	C/Conc	Fabric	C/Grave 1	C/Sand	SB	306+82	
	11.5	9.5	@21	7	cont.			
11	Asphalt	R/Conc	C/Gravel	C/Sand		SB	345+63	Gravel in treated layer breaking up the sample. switched to auger.
	7.5	9.5	6	@23				

Core	Layers				Core Location		Notes
	Thickness (inches)				Direction	Station	
12	Asphalt	C/Conc	Fabric	C/Sand	SB	395+90	
	10.5	22	@33	cont.			

R/Conc= Rubblized Concrete, C/Gravel= Red Clay Gravel,
C/Sand= Red Clay Sand (Subgrade), C/Conc= Crushed Concrete, and
S/Cement= Cement Stabilized Granular Material.

The figure below presents the locations of the 12 cores:

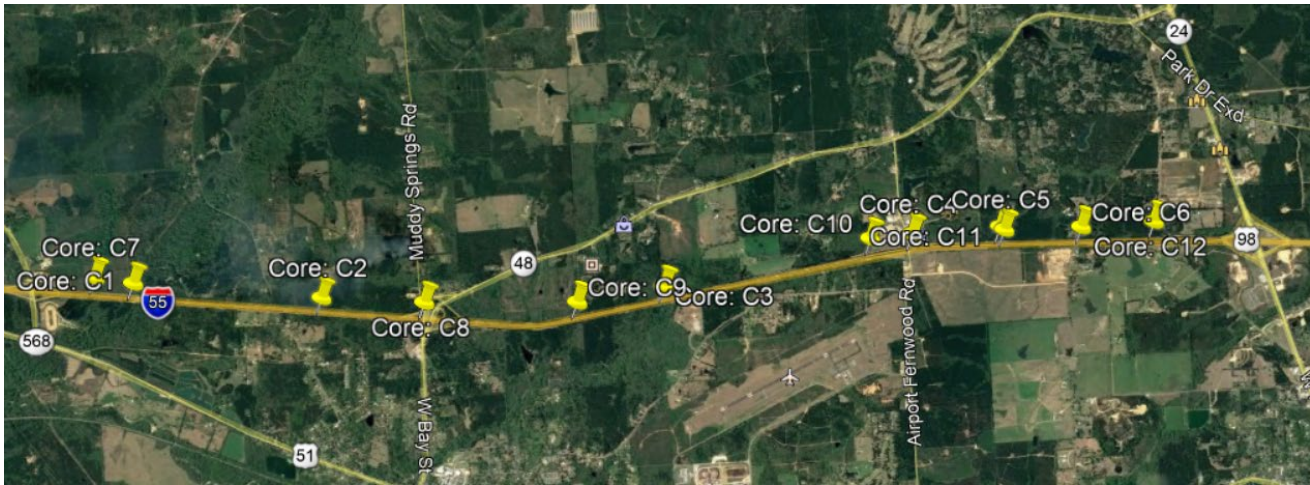


Figure 6. I-55 Coring Locations

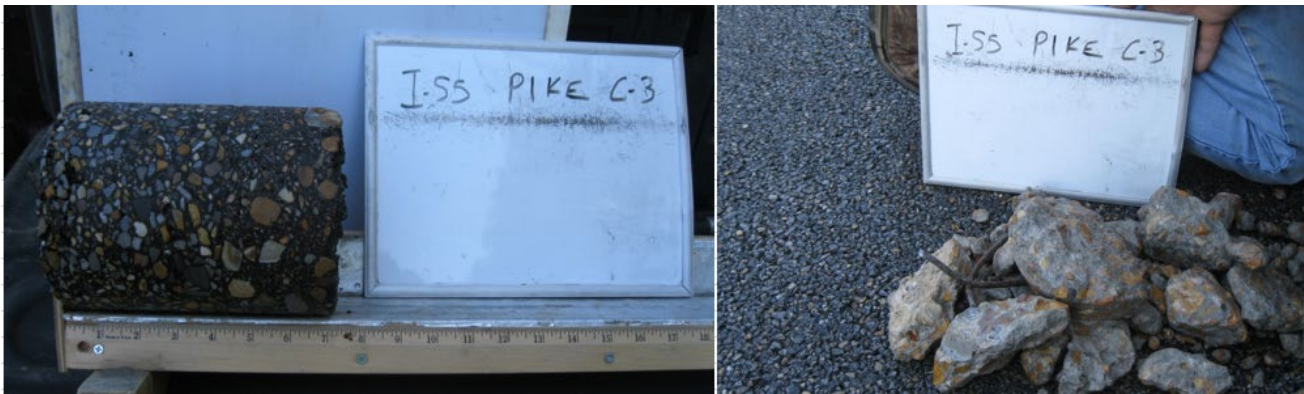


Figure 7. Example from Coring Location #3

As shown on the extracted material from core #3 shown in Figure 7, the reinforcement steel bars on the rubblized concrete layer are visible. The figure below shows another example from core #6 where no steel bars are visible on the crushed concrete.



Figure 8. Example from crushed concrete layer from Core #6 (no steel bars present)

The figure below shows two side by side photos showing what was considered to be rubblized concrete (left hand side) versus what was considered to be crushed concrete (right hand side).



Figure 9. Example from Rubblized vs Crushed Concrete

Right of Way Images

Simultaneously with the FWD and GPR testing, right of way (ROW) images were collected every 25-ft. These images help in making engineering judgments on the backcalculation results by comparing the moduli values with the pavement condition. For example, if the asphalt surface shows significant deformations, the asphalt modulus is expected to be relatively low, on the other hand if no distresses are visible on the pavement surface and the resulting backcalculated asphalt modulus is within expected range, then there is no reason to further inspect the results. Figure 10 shows an example of the ROW at station 52+75 NB.



Figure 10. Example ROW Image at Station 52+75 NB

Backcalculation Procedure

The following steps were performed in order to conduct the backcalculation analysis using ELMOD 6.0 software:

Step 1: FWD data for each section was checked for uniformity. The method of accumulated difference of the normalized center deflections at 9,000 lb. was used for this purpose. If necessary, the test section was split into several uniform subsections. The accumulated difference deflection basin was plotted using ELMOD 6 software to divide the testing stretch into homogeneous pavement sections. Figure 11 presents the accumulated difference deflection basin plotted on ELMOD 6 on the southbound traveling direction.

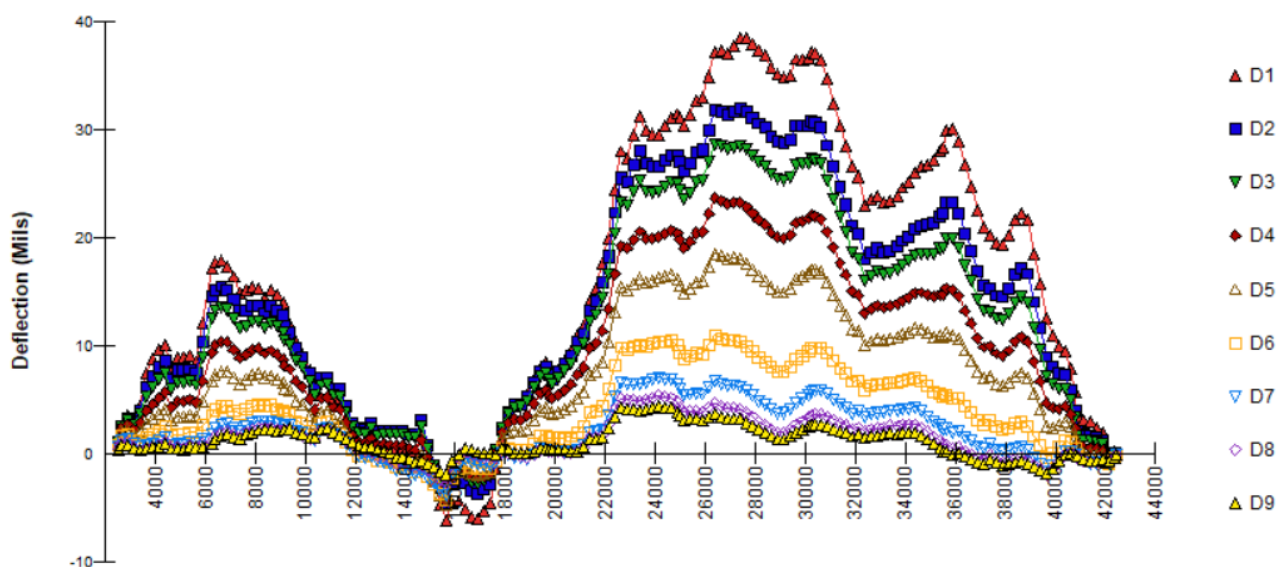


Figure 11. Accumulated Difference in Deflections SB Traveling Direction

The segments were divided at each significant change in slopes of the figure above. Also, coring and GPR data were utilized in the segmentation procedure. Following this process, the northbound traveling direction was divided into 14 subsections, while the southbound traveling direction was divided into 15 subsections. Table 3 and Table 4 provide the limits of each pavement subsections for the northbound and the southbound directions, respectively.

Table 3. Northbound Segmentation Limits

Section #	From Station	To Station	Length (ft)	# FWD Points
1	25+00	40+03	1,503	8
2	42+50	57+53	1,503	7
3	62+51	97+53	3,502	15
4	100+02	145+05	4,503	19
5	147+51	202+52	5,501	23
6	205+01	245+01	4,000	17
7	247+52	312+53	6,501	27
8	315+03	335+05	2,002	9
9	337+53	360+01	2,248	10
10	362+51	372+51	1,000	6
11	375+04	387+52	1,248	6
12	390+02	397+53	751	4
13	400+01	410+02	1,001	6
14	415+02	424+73	971	5

Table 4. Southbound Segmentation Limits

Section #	From Station	To Station	Length (ft)	# FWD Points
1	25+00	46+24	2,124	10
2	48+74	56+25	751	4
3	58+75	91+59	3,284	15
4	93+76	113+75	1,999	9
5	116+26	146+26	3,000	13
6	148+73	148+73	-	1
7	151+26	168+74	1,748	8
8	171+23	213+76	4,253	19
9	212+63	233+74	2,111	8
10	236+25	311+26	7,501	33
11	313+76	323+76	1,000	5
12	326+24	358+74	3,250	15
13	361+24	383+75	2,251	10
14	386+24	401+25	1,501	7
15	403+75	424+46	2,071	11

Step 2: ARA processed the GPR data and calibrated it with pavement cores provided by BCD. Then, the data was averaged for each subsection shown in Table 3 and 4 and were utilized in the backcalculation process. The figures below present the calibrated GPR thickness measurements with the assigned coring locations and the segmentation limits.

Also, Figure 14 shows a screenshot from the RADAN 7 software illustrating an example of the GPR data calibration procedure using cores. The black dots represent the ground truth depth from the cores, the yellow area corresponds to layer 1 depth, while the red area corresponds to layer 2 depth. It can be noticed from the figure that after GPR calibration the black dots are in line with its corresponding yellow and red interfaces, which in turn indicates that the GPR depth is matching the core depth at the core location.

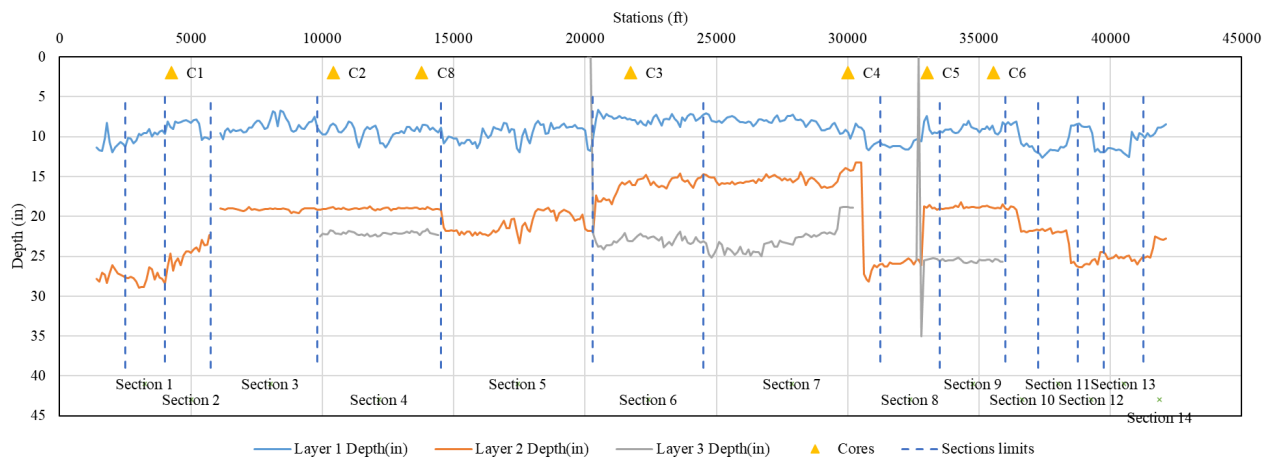


Figure 12. Processed GPR Results Northbound Traveling Direction

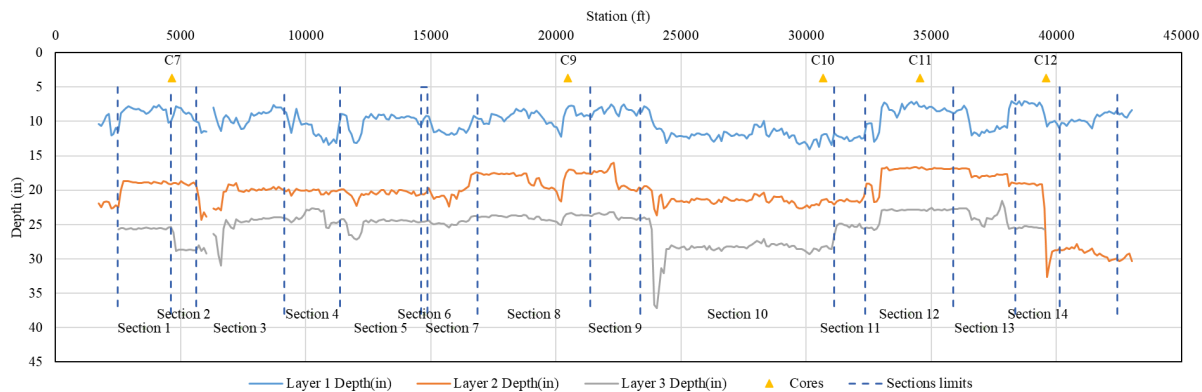


Figure 13. Processed GPR Results Southbound Traveling Direction

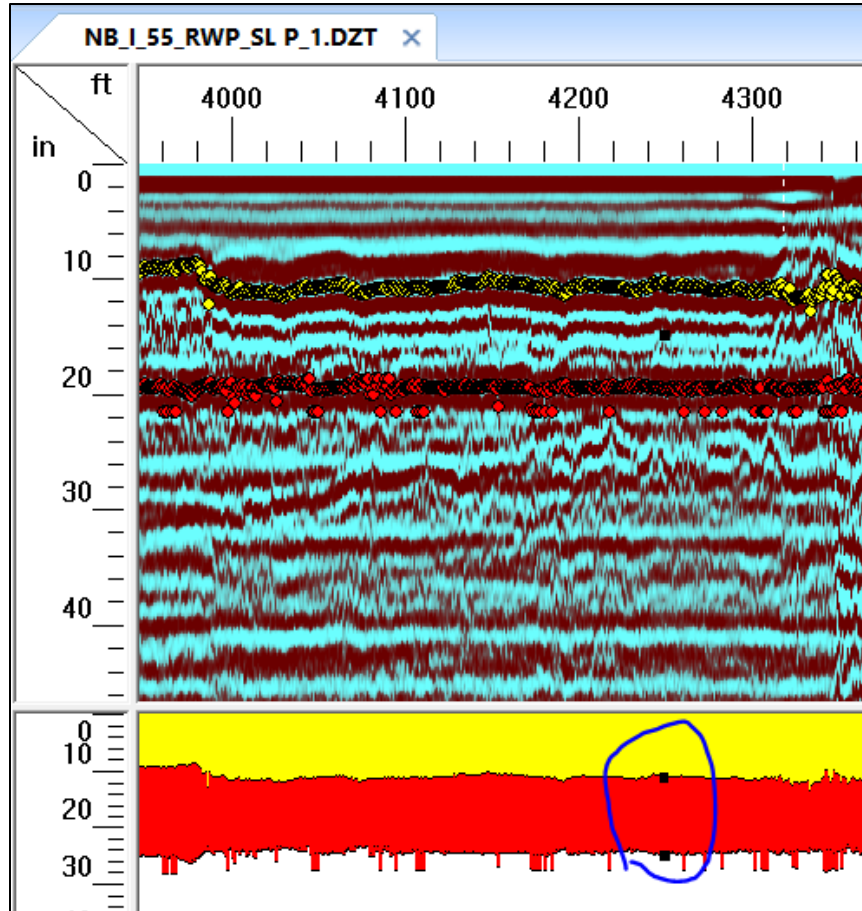


Figure 14. GPR Calibration Example

Step 3: The mid-depth temperature of the HMA layers was determined using the BELLS equation. The BELLS equation requires the average air temperature of the ‘previous day’ (one day prior to FWD data collection). Temperature data was obtained from www.wunderground.com website using the closest weather stations to each pavement test site. The primary objective of this step was to normalize the resulting HMA layer moduli to a reference temperature of 77°F.

Step 4: The surface modulus plot for each test point was checked for abnormalities. Typically, the surface modulus basin must decrease with increase in distance from load and then slightly increase again if the subgrade is non-linear or a bedrock is present. Abnormal points can be excluded from the analysis or the entire station can be excluded if most of the geophone readings must be removed. Examples of normal and abnormal surface modulus basins are shown in Figure 15 and Figure 16, respectively.

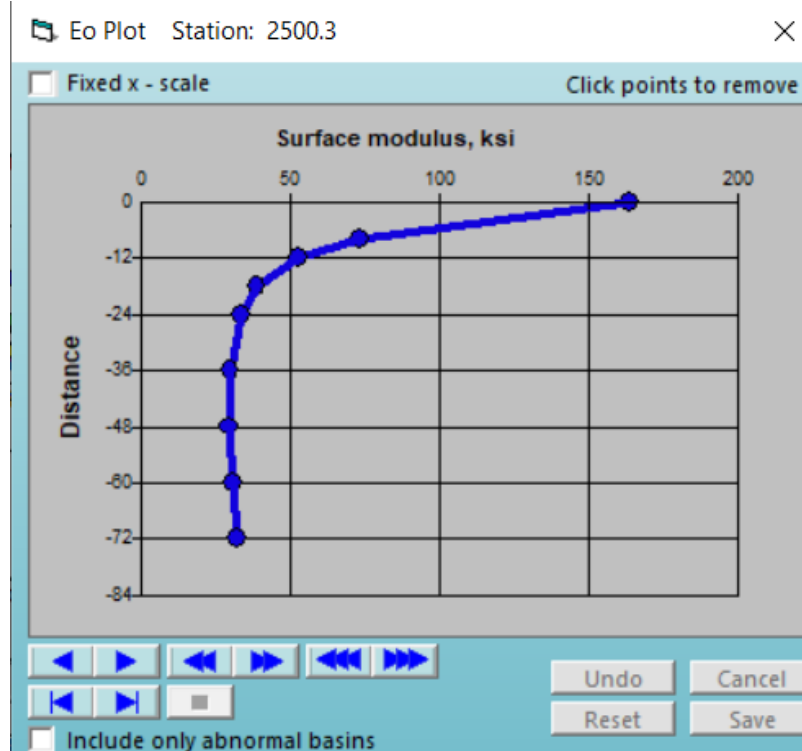


Figure 15. Example of Normal Surface Modulus Basin

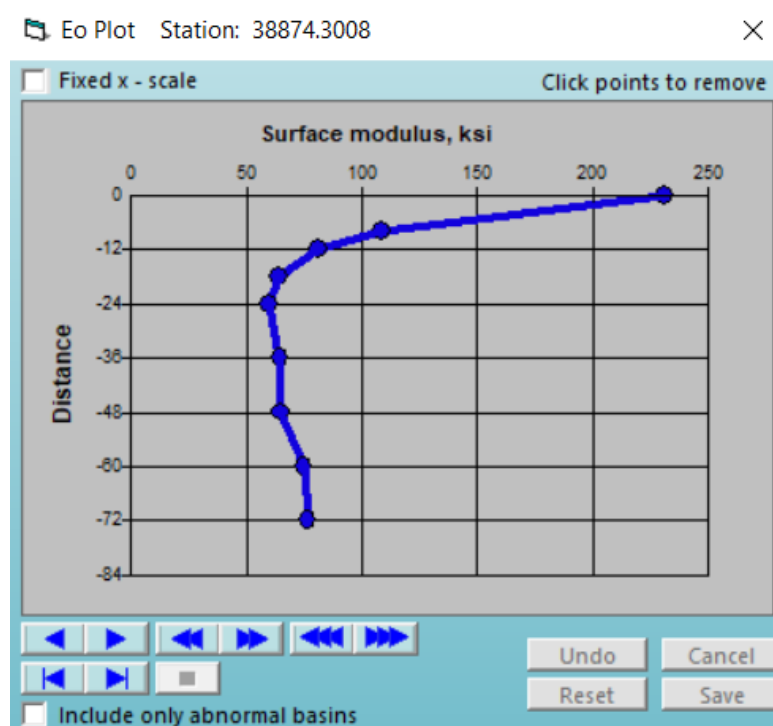


Figure 16. An Abnormal Surface Modulus at 72 in Away from the Load

Step 6: A parameter file in ELMOD was setup with the different layer types prior to performing backcalculation. This will ensure that the backcalculated moduli for temperature dependent materials, such as HMA, will be adjusted to reference temperature.

The layer thickness information from Figure 10 and Figure 11 were used. The FWD drop corresponding to the second 9,000-lb load level was used in the analysis. Several trials were made for each section using the Finite Element (FE) module and the Deflection Basin Fit module in ELMOD for each of the test sections. A mesh of 24 columns and rows was selected for the FE analysis and non-linearity of the subgrade was considered. The average percent differential deflections for all geophones was used to compare the different runs and select the trial with the best deflection basin fit.

It was concluded that when the provided seed moduli were used in the FE module and then the average moduli results from the FE analysis for each layer underneath the HMA layer and above the subgrade layer were used as seed moduli in the Deflection Basin Fit module, the average percent differential deflections for all geophones was one of the lowest. In some cases, some trials yielded lower average percent differential deflections; however, the average layer moduli were unreasonable and/or the standard deviation for the layer moduli was high. Therefore, it is wise to perform several extra runs with different seed moduli before selecting the final seed moduli. Once a final set of seed moduli are selected, the depth to bedrock is calculated and checked if it's shallow or deep. An example below better illustrates the aforementioned steps.

Sample Backcalculation Procedure at Section 1 on the Southbound Traveling Direction

The following example shows the backcalculation procedure for Section 1 SB. From Station 25+00 to Sta. 46+24 the test section consists of an HMA layer over rubblized concrete over soil-cement layer over subgrade. Only Section 1 SB is considered in this example.

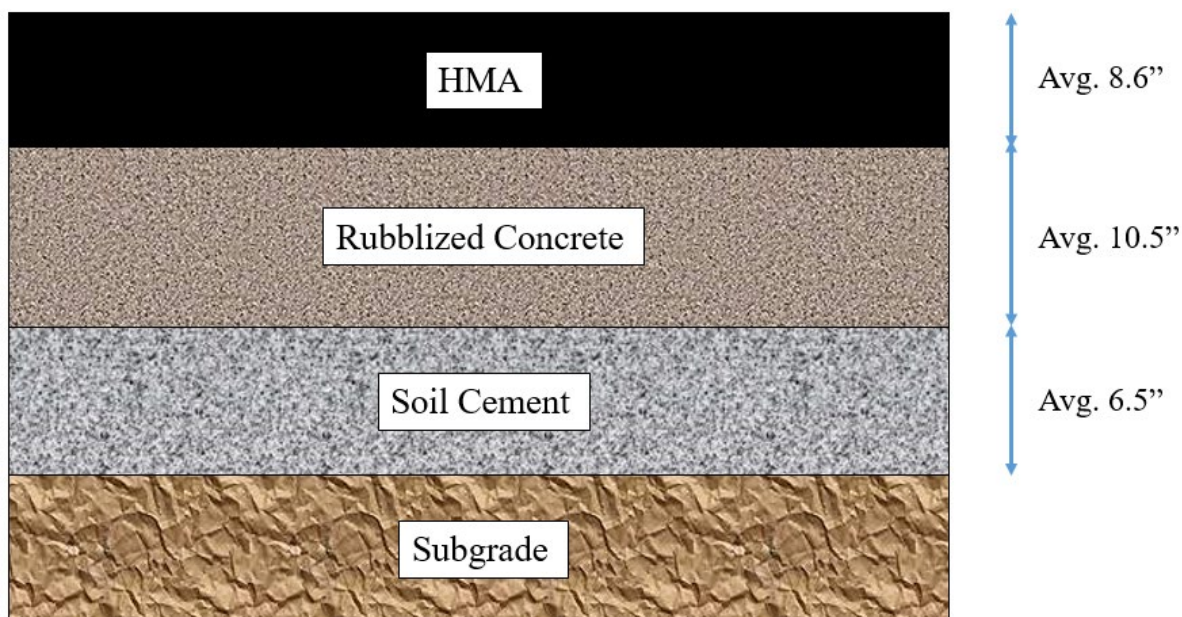


Figure 17. Cross Section of Section 1 SB

Screenshots of the FE module surface modulus and input screen are shown in Figure 18. The FEM button is selected, the non-linearity of the subgrade “N” is selected, and the FEM mesh is kept at 24 rows/columns. Once these buttons are selected, the “Fit and Save all Points” button is clicked, and the data will be saved in the database. The screenshot below presents an example of the settings of the FEM methodology.

Options Show Calibrate Units

Chainage from 2500.3 to 4624 From drop 2 to 2

Contact stress 0.082 Radius 5.9000

Use first 9 geophones

Fit point Save point

Fit and save all points

Run in background

Generate FEM mesh with

24 rows/ columns

Edit Mesh FEMplot

Close FEM/LET/MET

RMS abs % minimize %

Deflections FEM LET MET

Distance	FEM	LET	MET
0	5.48		
19768	4.45		
19828	4.16		
00679	3.73		
96561	3.20		
01358	2.41		
93121	1.82		
89301	1.38		
27161	1.08		

Figure 18. FEM Methodology ELMOD Input Screen

The Figure below shows an example of the FEM fitting between calculated (in yellow) and measured (in dark red) deflections basins as well as surface modulus at station 25+00 SB.

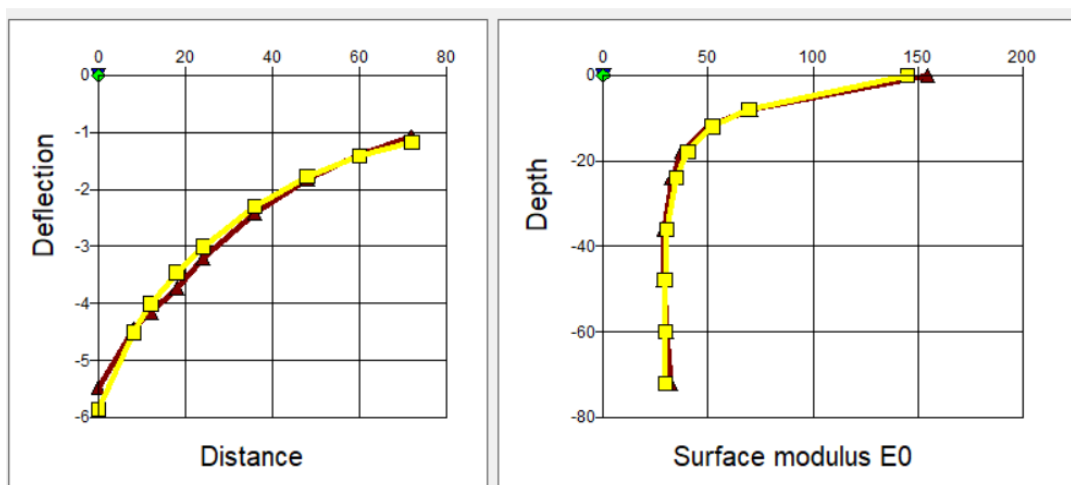


Figure 19. FEM Methodology Fitting Example at Station 25+00 SB

The Following Figure summarizes in a flowchart format the backcalculation procedure described above.

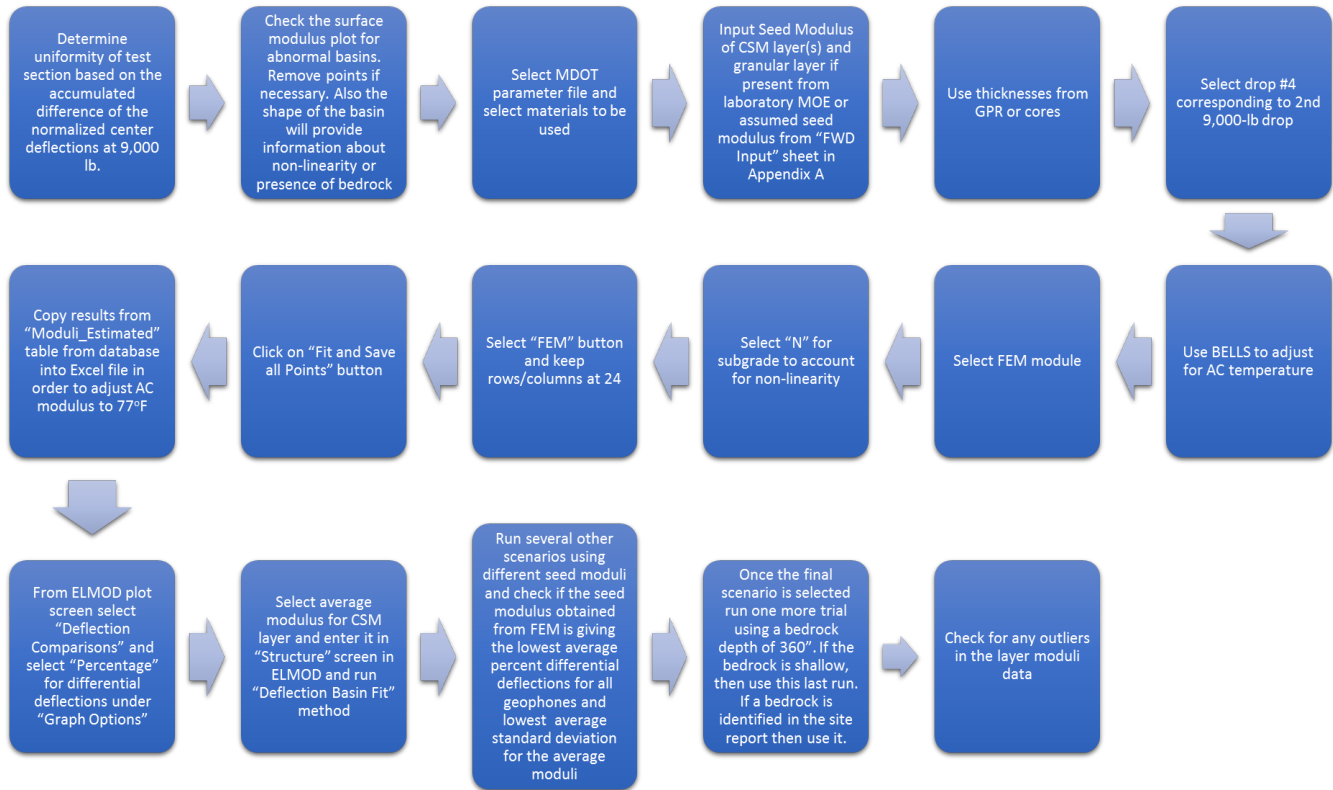


Figure 20. Flowchart of the Backcalculation Procedure

Backcalculation Results

The procedure summarized in Figure 20 was utilized and the backcalculated layer moduli for all sections were estimated and the average values for each section are shown in the tables below, while the detailed results are provided in a spreadsheet format in Appendix B.

Table 5. Northbound Average Backcalculation Results

Section	Stations	Layer Moduli (ksi)				
		Layer	N	Mean	Std.	84th
1	25+00 to 40+03	Asphalt	8	489.5	1.227	398.9
		Crushed Concrete	8	151.3	1.826	82.9
		Subgrade	8	37.4	1.698	22.0
2	42+50 to 57+52	Layer	N	Mean	Std.	84th
		Asphalt	7	505.5	1.519	332.8
		Crushed Concrete	7	142.2	2.951	48.2
		Subgrade	7	37.4	1.539	24.3
3	62+52 to 97+53	Layer	N	Mean	Std.	84th
		Asphalt	15	589.8	2.783	211.9
		Rubblized Concrete	15	117.2	1.729	67.8
		Subgrade	15	28.8	1.761	16.4

Section	Stations	Layer Moduli (ksi)				
		Layer	N	Mean	Std.	84th
4	100+02 to 145+05	Asphalt	19	474.1	1.241	382.0
		Rubblized Concrete	19	169.6	1.656	102.4
		Soil Cement	19	59.4	1.381	43.0
		Subgrade	19	31.7	1.319	24.0
		Layer	N	Mean	Std.	84th
5	147+52 to 202+51	Asphalt	23	582.7	2.145	271.7
		Rubblized Concrete	23	152.1	2.159	70.4
		Subgrade	23	38.6	1.543	25.0
		Layer	N	Mean	Std.	84th
6	205+02 to 245+01	Asphalt	17	917.2	1.147	799.7
		Rubblized Concrete	17	184.2	1.364	135.0
		Soil Cement	17	66.1	1.281	51.6
		Subgrade	17	25.6	1.209	21.2
		Layer	N	Mean	Std.	84th
7	247+53 to 312+52	Asphalt	27	556	1.589	349.9
		Rubblized Concrete	27	337.1	1.734	194.4
		Soil Cement	27	103.6	1.618	64.0
		Subgrade	27	34.2	1.622	21.1
		Layer	N	Mean	Std.	84th
8	315+03 to 335+05	Asphalt	9	512.7	1.289	397.8
		Crushed Concrete	9	190.3	2.872	66.3
		Subgrade	9	33.2	1.72	19.3
		Layer	N	Mean	Std.	84th
9	337+54 to 360+01	Asphalt	10	556.2	1.083	513.6
		Rubblized Concrete	10	192.7	1.848	104.3
		Soil Cement	10	64.3	1.619	39.7
		Subgrade	10	27.3	1.327	20.6
		Layer	N	Mean	Std.	84th
10	362+51 to 372+51	Asphalt	6	562.4	1.315	427.7
		Crushed Concrete	6	181.2	2.227	81.4
		Subgrade	6	33.7	1.206	27.9
		Layer	N	Mean	Std.	84th
11	375+04 to 387+52	Asphalt	6	562.4	1.315	427.7
		Crushed Concrete	6	181.2	2.227	81.4
		Subgrade	6	33.7	1.206	27.9
		Layer	N	Mean	Std.	84th
12	390+02 to 397+52	Asphalt	4	703.1	1.387	506.9
		Crushed Concrete	4	174.2	2.146	81.2
		Subgrade	4	42.3	1.993	21.2
		Layer	N	Mean	Std.	84th
13	400+01 to 410+02	Layer	N	Mean	Std.	84th

Section	Stations	Layer Moduli (ksi)				
14	415+03 to 424+72	Asphalt	6	692.5	1.273	544.0
		Crushed Concrete	6	148.7	1.63	91.2
		Subgrade	6	28.7	1.125	25.5
		Layer	N	Mean	Std.	84th
14	415+03 to 424+72	Asphalt	5	698	1.127	619.3
		Crushed Concrete	5	163	1.512	107.8
		Subgrade	5	35.3	1.6	22.1
		Layer	N	Mean	Std.	84th

Table 6. Southbound Average Backcalculation Results

Section	Stations	Layer Moduli (ksi)				
1	25+00 to 46+24	Layer	N	Mean	Std.	84th
		Asphalt	10	495.5	1.294	382.9
		Rubblized Concrete	10	123.2	1.904	64.7
		Soil Cement	10	56.9	1.687	33.7
		Subgrade	10	27.7	1.286	21.5
2	48+75 to 56+24	Layer	N	Mean	Std.	84th
		Asphalt	4	517.6	1.071	483.3
		Rubblized Concrete	4	183.3	1.609	113.9
		Soil Cement	4	59	1.133	52.1
		Subgrade	4	28.8	1.099	26.2
3	58+76 to 91+59	Layer	N	Mean	Std.	84th
		Asphalt	15	478	1.334	358.3
		Rubblized Concrete	15	137.3	1.94	70.8
		Gravel	15	44.6	1.376	32.4
		Subgrade	15	35.3	1.487	23.7
4	93+77 to 11+374	Layer	N	Mean	Std.	84th
		Asphalt	9	513.4	1.152	445.7
		Rubblized Concrete	9	213	1.877	113.5
		Gravel	9	54.3	1.739	31.2
		Subgrade	9	40.5	1.518	26.7
5	116+26 to 146+25	Layer	N	Mean	Std.	84th
		Asphalt	13	473.2	1.191	397.3
		Rubblized Concrete	13	153.4	1.541	99.5
		Gravel	13	42.1	1.257	33.5
		Subgrade	13	34	1.664	20.4
6	148+74 (one station)	Layer	N	Mean	Std.	84th
		Asphalt	1	436.3	N/A	N/A
		Rubblized Concrete	1	1650.1	N/A	N/A
		Gravel	1	434.3	N/A	N/A
		Subgrade	1	102.6	N/A	N/A
7	151+27 to 168+74	Layer	N	Mean	Std.	84th

Section	Stations	Layer Moduli (ksi)				
		Asphalt	8	538.7	1.18	456.5
		Rubblized Concrete	8	238.4	1.977	120.6
		Gravel	8	45.8	1.637	28.0
		Subgrade	8	41.1	2.066	19.9
8	171+24 to 213+76	Layer	N	Mean	Std.	84th
		Asphalt	19	390.5	1.187	329.0
		Rubblized Concrete	19	139.8	1.49	93.8
		Gravel	19	40.7	1.431	28.4
		Subgrade	19	25.5	1.519	16.8
9	216+24 to 233+74	Layer	N	Mean	Std.	84th
		Asphalt	8	386.7	1.246	310.4
		Crushed Concrete	8	119.2	1.34	89.0
		Gravel	8	43.3	1.254	34.5
		Subgrade	8	20.3	1.347	15.1
10	236+25 to 311+25	Layer	N	Mean	Std.	84th
		Asphalt	33	418.8	1.192	351.3
		Crushed Concrete	33	133.9	1.952	68.6
		Gravel	33	45.6	1.355	33.7
		Subgrade	33	31.4	1.502	20.9
11	313+77 to 323+75	Layer	N	Mean	Std.	84th
		Asphalt	5	555.6	1.159	479.4
		Rubblized Concrete	5	437.2	1.643	266.1
		Subgrade	5	72.2	1.679	43.0
12	326+24 to 358+74	Layer	N	Mean	Std.	84th
		Asphalt	15	478.4	1.152	415.3
		Rubblized Concrete	15	199.9	1.27	157.4
		Gravel	15	57.4	1.289	44.5
		Subgrade	15	25.4	1.345	18.9
13	361+25 to 383+74	Layer	N	Mean	Std.	84th
		Asphalt	10	601.6	1.282	469.3
		Rubblized Concrete	10	205.3	2.53	81.1
		Subgrade	10	40.7	1.478	27.5
14	386+25 to 401+24	Layer	N	Mean	Std.	84th
		Asphalt	7	756.4	1.19	635.6
		Rubblized Concrete	7	550.1	3.734	147.3
		Subgrade	7	54.6	2.001	27.3
15	403+76 to 424+46	Layer	N	Mean	Std.	84th
		Asphalt	11	682	1.136	600.4
		Crushed Concrete	11	142.4	2.678	53.2
		Subgrade	11	48.2	1.229	39.2

Investigating Variability

High variability in the backcalculation results was noticed. That can be noticed by observing the standard deviation factors presented in Tables 4 and 5. This variability was also noticed in the GPR data and in the coring report. This variability can be described as follows:

- Some cores concluded that the second layer is rubblized concrete, while other pavement cores indicated showed that the second layer corresponds to crushed concrete.
- In some cores, geotextile fibers were present at the subbase / subgrade interface, and they did not exist at other locations.
- The average backcalculated layer modulus for layer 2 varied from 117 ksi to 550 ksi with one section showed an average backcalculated layer modulus of 1,650 ksi.
- From Figure 12 and Figure 13, high variability in the pavement thickness can be observed. It can be also noticed that the third pavement layer was not continuous through the testing stretch.

It is uncommon to observe this high variability in a relatively short, 7.7-mile highway segment; therefore, further investigation was performed. The DOT provided two pavement design reports for the I-55 and another similar project to assist in this investigation. In both reports, it can be noticed that there were pavement changes when there are vertical clearance requirements and/or merging of acceleration or deceleration lanes.

<u>(Rubblization of Mainline Concrete):</u>	<u>(Removal of Mainline Concrete):</u>
MAINLINE 1.00" Open Graded Friction Course 1.50" SMA (9.5 mm Mixture) (1 @ 1.5) 2.00" SMA (12.5 mm Mixture) (1 @ 2.0) 3.50" ASPH PAVE, HT (19 mm Mixture) (1 @ 3.5) 10.00" Rubblized Concrete to remain in place on mainline 18.00" Structure Thickness	The following pavement design is proposed for the flexible pavement construction of mainline in the areas where the in-place concrete will be removed. (Bridge ends and overpasses) MAINLINE 1.00" Open Graded Friction Course 1.50" SMA, HT (9.5 mm Mixture) (1 @ 1.5) 2.00" SMA, HT (12.5 mm Mixture) (1 @ 2.0) 3.00" ASPH PAVE, HT (19 mm Mixture) (1 @ 3.0) 3.00" ASPH PAVE, ST (19mm Mixture) (1 @ 3.0) 10.00" Crushed Stone Base 20.50" Structure Thickness

Figure 21. Screenshots from the Provided Pavement Design Report

Looking at the project limits, 7 overpasses are present, in addition to 1 bridge, this may explain the variability that was observed in the pavement thicknesses throughout the evaluated pavement segment. The design report also showed that at these overpass location, the rubblized concrete layer was substituted with crushed concrete layer. This can explain the observed variability in the rubblized concrete presence through the extracted pavement cores. It also can be noticed that the design report recommended "crushed stone"; however, what have been observed in the field was "crushed concrete".



APPEDIX A: CORING LOCATIONS

- I-55 Pike County Summary.xlsx
- I-55 Pike County_Cores.kml

APPEDIX B: BACKCALCULATION RESULTS

- I-55_Backcalculation Results.xlsx

Appendix C

I-55 Tate County Non-Destructive Tests

MDOT State Study 312 — Recycling PCC Pavements using Rubblization

Final Report

STUDY 308 FWD TESTING AND BACKCALCULATION OF RUBBLIZED PCC PAVEMENT MODULI FOR MDOT I-55 TATE COUNTY

Introduction

ARA has performed nondestructive testing (NDT) on approximately 1.5 centerline miles of I-55 southbound in Tate County, spanning from approximately project station 477+20 (north) to project station 398+43 (south). The section's boundaries extend from south of Rifle Range Rd. to north of Highway 4. The project scope includes Falling Weight Deflectometer (FWD) testing, Ground Penetrating Radar (GPR) testing, and backcalculation analysis of the pavement layer moduli before rubblization of the Portland Cement Concrete (PCC) layer and on top of the Aggregate Base (AB) layer after rubblization. Figure 1 illustrates the overall project location.

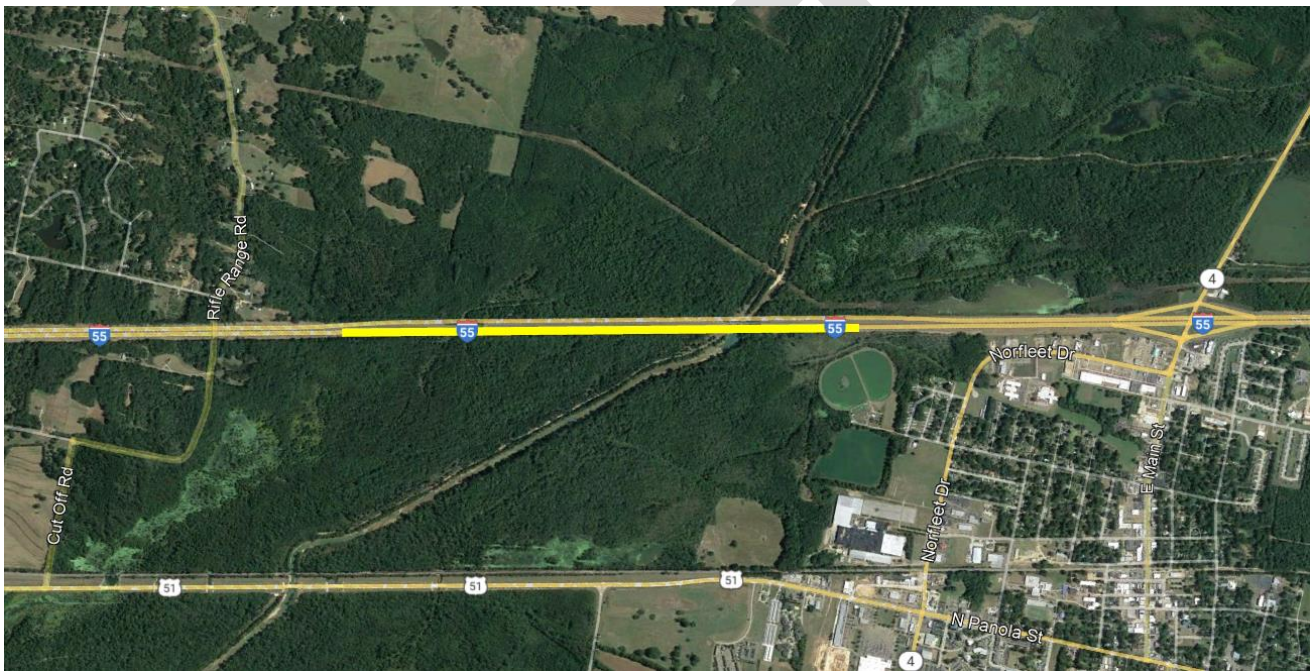


Figure 1. Overall Project Location

Pavement Layers from As-Built

Only as-built data was available for the tested section, and no cores were obtained for analysis. The structure before rubblization consisted of a PCC layer on top of a Cementitiously Stabilized Material (CSM) layer, which was further supported by a clay gravel layer over a second CSM layer on top of the subgrade.

After rubblization, the structure consisted of an Asphalt Concrete (AC) layer on top of a variable crushed stone layer labeled as AB in this report. Beneath this AB layer, there was a CSM layer, followed by a variable fill soil layer, a PCC layer, another CSM layer, a clay gravel layer, and finally, a third CSM layer on the subgrade.

It should be noted that the FWD testing after rubblization was performed on top of the AB layer before the AC layer was placed. The presence of multiple layers posed challenges during the FWD and GPR data analysis. To overcome this, certain layers were combined to estimate the backcalculated layer moduli. All the layers below the PCC layer were grouped together and referred to as the “support” layer.

Figure 2 provides visual representations of the pavement structure based on the as-built data before and after the rubblization process.

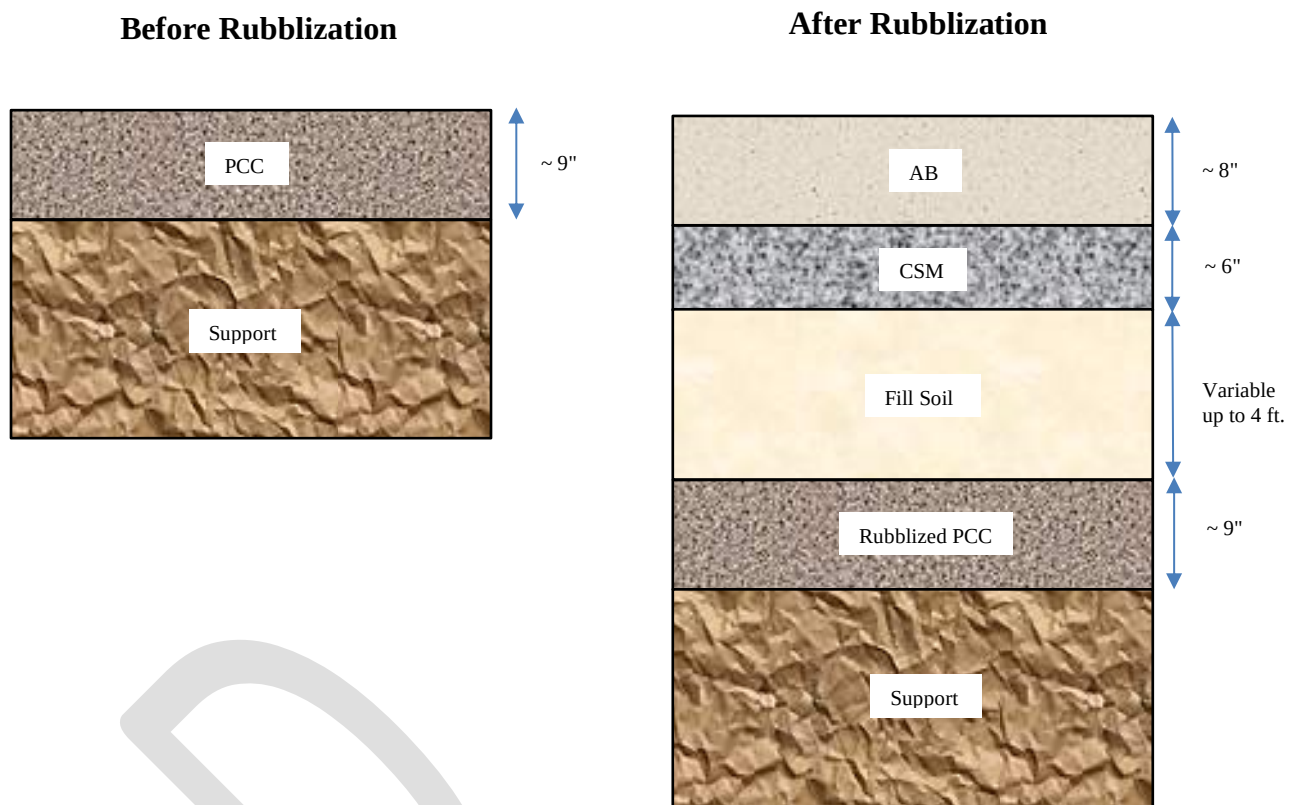


Figure 2. Pavement Structure from As-Built

Falling Weight Deflectometer Testing

FWD testing was conducted on the inside lane prior to rubblization of the PCC and on both the inside and outside lanes on the AB layer after rubblization, in the southbound direction of I-55 in Tate County, Mississippi. The FWD testing on the intact PCC took place on February 23rd, 2022, while the testing on the AB layer after rubblization occurred on July 22nd, 2022.

During the FWD testing on intact PCC, measurements were taken in the fast lane of the southbound direction, spanning from approximately project station 466+68 to project station 398+43. On the other hand, the FWD testing on the AB layer after rubblization was performed in both the fast and slow lanes of the southbound direction, covering the area from approximately project station 477+20 to project

station 433+98. As a result, only a portion of the tested section overlapped between the intact PCC and the rubblized PCC.

FWD testing was conducted at an interval ranging from 25 to 100 feet while the GPR was conducted continuously. A total of 73 FWD stations were tested on the intact PCC and 184 FWD stations were tested on the AB surface layer after rubblization. Four load levels were applied at each tested station: 6,000 lbs., 9,000 lbs., 12,000 lbs., and 16,000 lbs. Table 1 provides a statistical summary of the load-normalized center deflection data corresponding to an FWD load level of 9,000 lbs., while Figure 3 displays the load-normalized center deflection corresponding to the same FWD load level on the southbound traveling direction.

Both Table 1 and Figure 3 demonstrate an increase in deflection after rubblization, with greater variability in deflections observed after rubblization. This increased variability could be attributed to the variable thickness of the fill soil placed on top of the rubblized PCC and/or the extent to which the PCC was rubblized.

Table 1. Center Deflection Statistical Summary

Phase	Highway	Direction	Lane	Station (ft)		Normalized Center Deflections (mils)		
				From	To	Average	Standard Deviation	84th Percentile
Before Rubblization testing on Intact PCC	I-55 Tate County	SB	Fast	398+43	466+68	3.21	0.71	3.92
After Rubblization testing on AB Layer	I-55 Tate County	SB	Slow	433+91	477+20	27.92	20.33	48.25
		SB	Fast	433+91	477+20	22.48	11.77	34.25

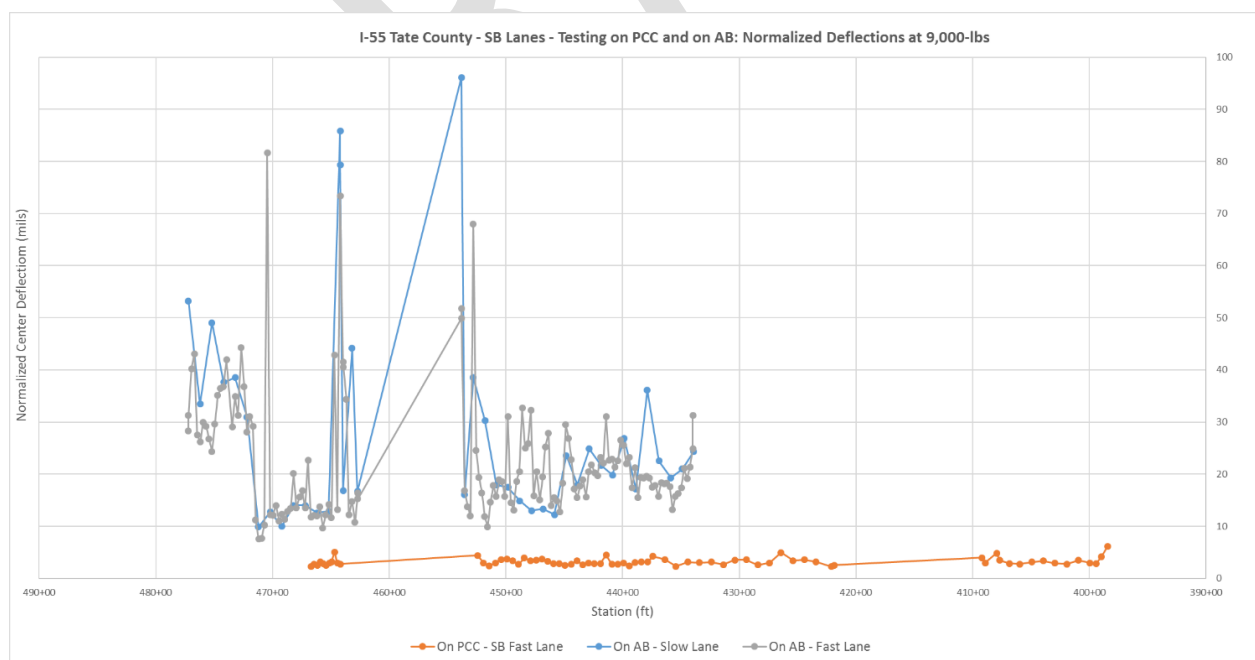


Figure 3. I-55 SB Before and After Rubblization Normalized Deflections at 9,000 lbs.

The surface modulus refers to a property that characterizes the stiffness or resistance of the entire pavement to deformation under applied loads. The surface modulus under the load is calculated from the surface deflection under the load, regardless of the layers type and thickness. The surface modulus provides an indication of the bearing capacity of all layers under the load. Table 2 presents the statistical summary of the surface modulus under the load before and after rubblization, while

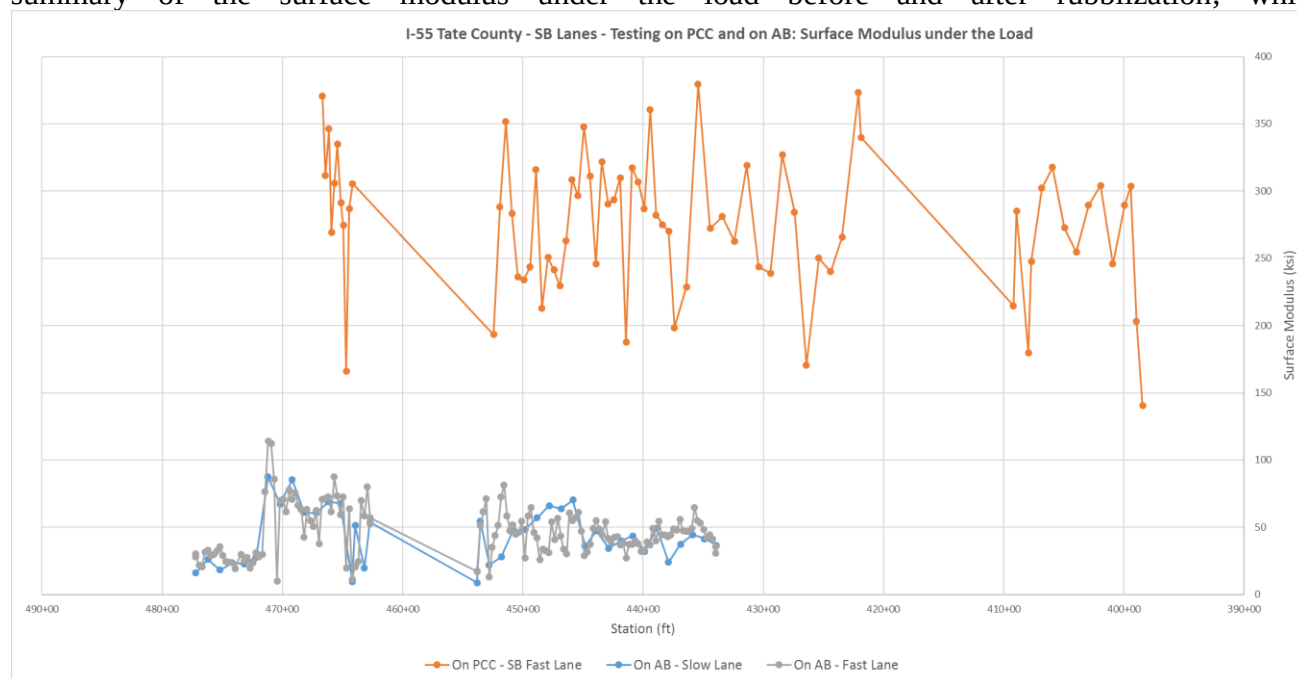


Figure 4 displays the surface modulus under the load at each test point before and after rubblization. Additionally, Figure 5 shows the average surface modulus under the load for the entire test section before and after rubblization.

The data in the table and plots clearly demonstrate that the bearing capacity was significantly reduced after rubblization, despite several layers being added on top of the rubblized PCC. This confirms that the modulus of the PCC was significantly reduced after rubblization.

Table 2. Surface Modulus Under the Load Statistical Summary

Phase	Highway	Direction	Lane	Station (ft)		Surface Modulus Under the Load (ksi)		
				From	To	Average	Standard Deviation Factor	84th Percentile
Before Rubblization on Intact PCC	I-55 Tate County	SB	Fast	398+43	466+68	270.9	1.221	222.0
After Rubblization on AB Layer	I-55 Tate County	SB	Slow	433+91	477+20	37.2	1.789	20.8
		SB	Fast	433+91	477+20	42.4	1.546	27.4

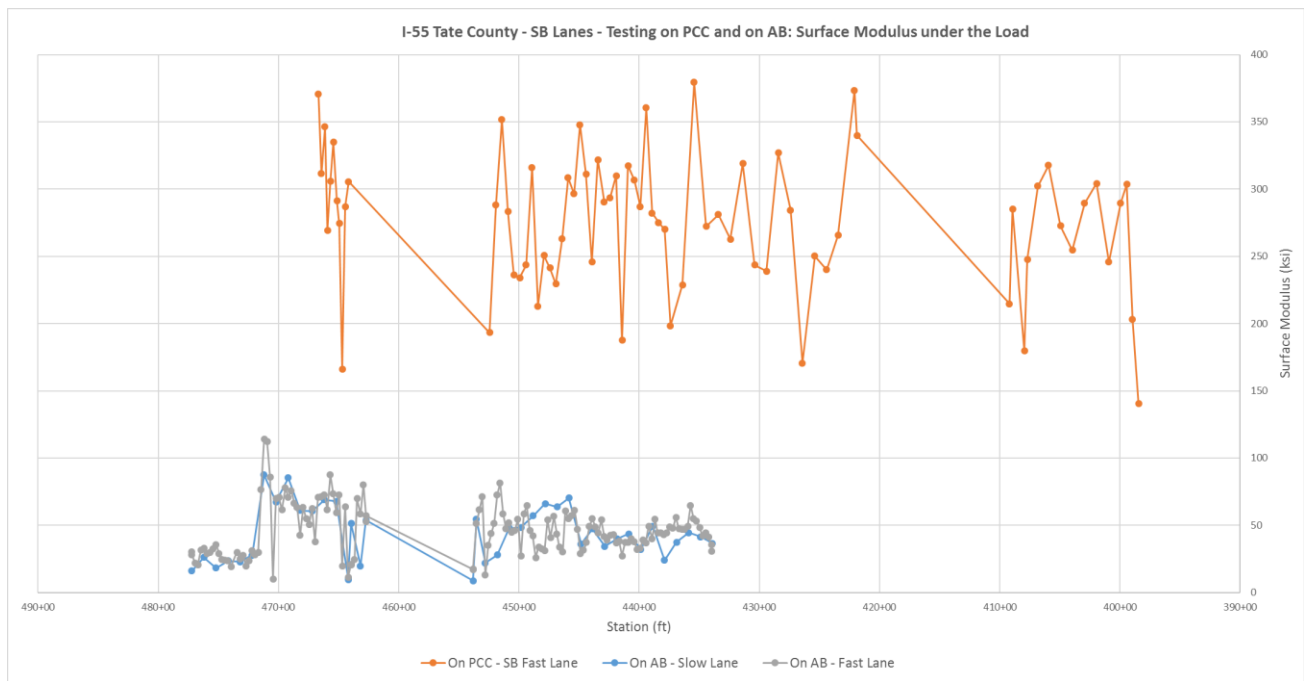


Figure 4. I-55 SB Before and After Rubblization Surface Modulus Under the Load

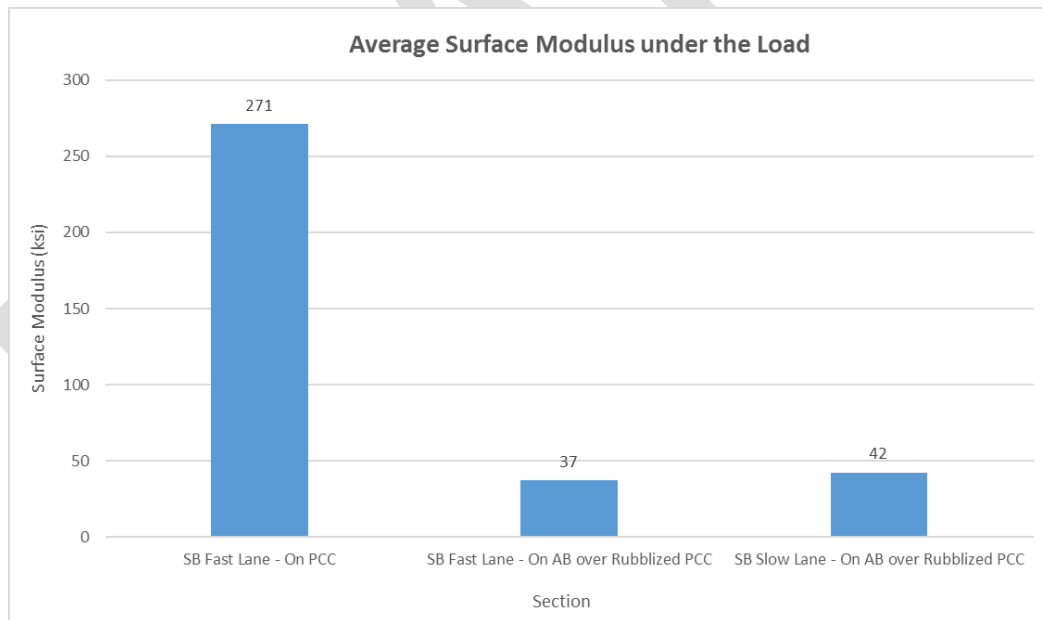


Figure 5. I-55 SB Before and After Rubblization Average Surface Modulus Under the Load

Ground Penetrating Radar Testing

Continuous GPR testing was conducted on the pavement section on I-55 before and after rubblization. The GPR data was then processed to estimate the thickness of the pavement layers after calibration with thickness information obtained from as-built data, as pavement cores were not available. The testing was performed using the GSSI SIR-30 GPR. Two antennas were utilized in the data collection: an air-horn antenna with a frequency of 2.0 GHz and a ground-coupled antenna with a frequency of

900 MHz. The Air-horn antenna provides high accuracy measurements of the surface layer thickness, while the ground-coupled antenna provides measurements at up to 2-ft deep. Figure 6 displays the GPR equipment used in this project, while Figure 7 and Figure 8 showcase screenshots of the raw GPR data on I-55 southbound before and after rubblization, respectively.

However, the presence of a relatively thick fill soil layer, up to 4 feet, on top of the rubblized PCC posed challenges in identifying the interfaces between the fill soil and rubblized PCC, as well as between the rubblized PCC and the support layer. As a result, the GPR data on top of the AB layer after rubblization was only analyzed where the reinforcement in the rubblized PCC was visible. This approach ensures that the visible layer in the GPR data correspond specifically to the rubblized PCC layer and not any other layer. Only a portion of the tested section met this criterion, allowing for clear tracing of the AB, CSM, and rubblized PCC layers from approximately Station 471+55 to Station 468+88. In this section, the fill soil appears to be absent. Figure 9 illustrates the limits of this section as two black lines. This test section is comprised of an AB layer over CSM over rubblized PCC over support.



Figure 6. GPR Unit with Double Antennas

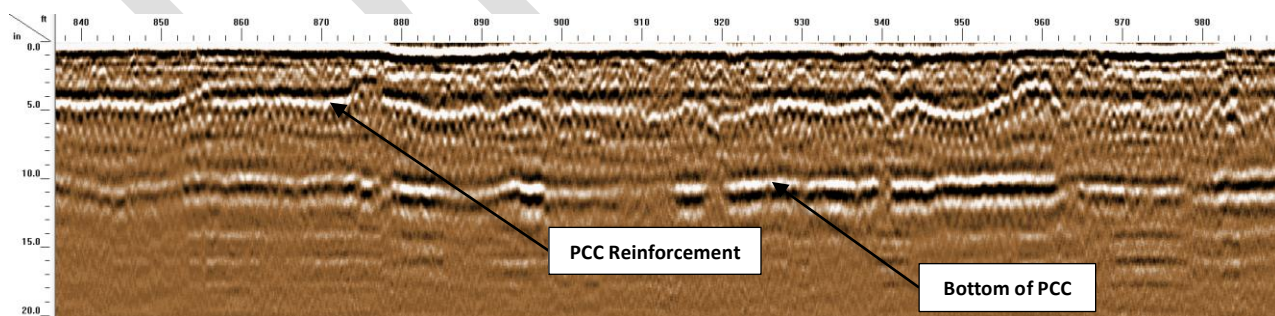


Figure 7. Raw GPR Data Example from I-55 Southbound Before Rubblization

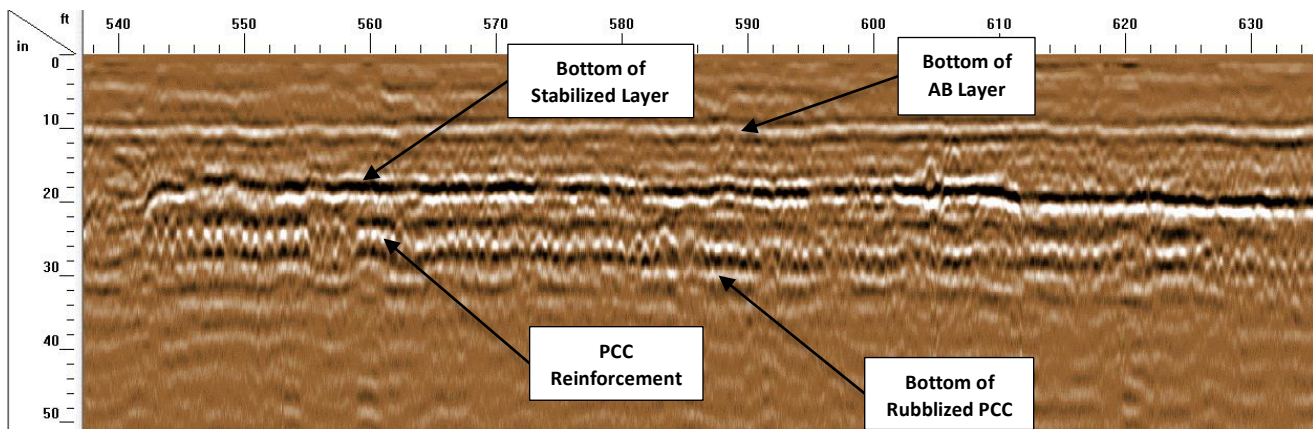


Figure 8. Raw GPR Data Example from I-55 Southbound After Rubblization

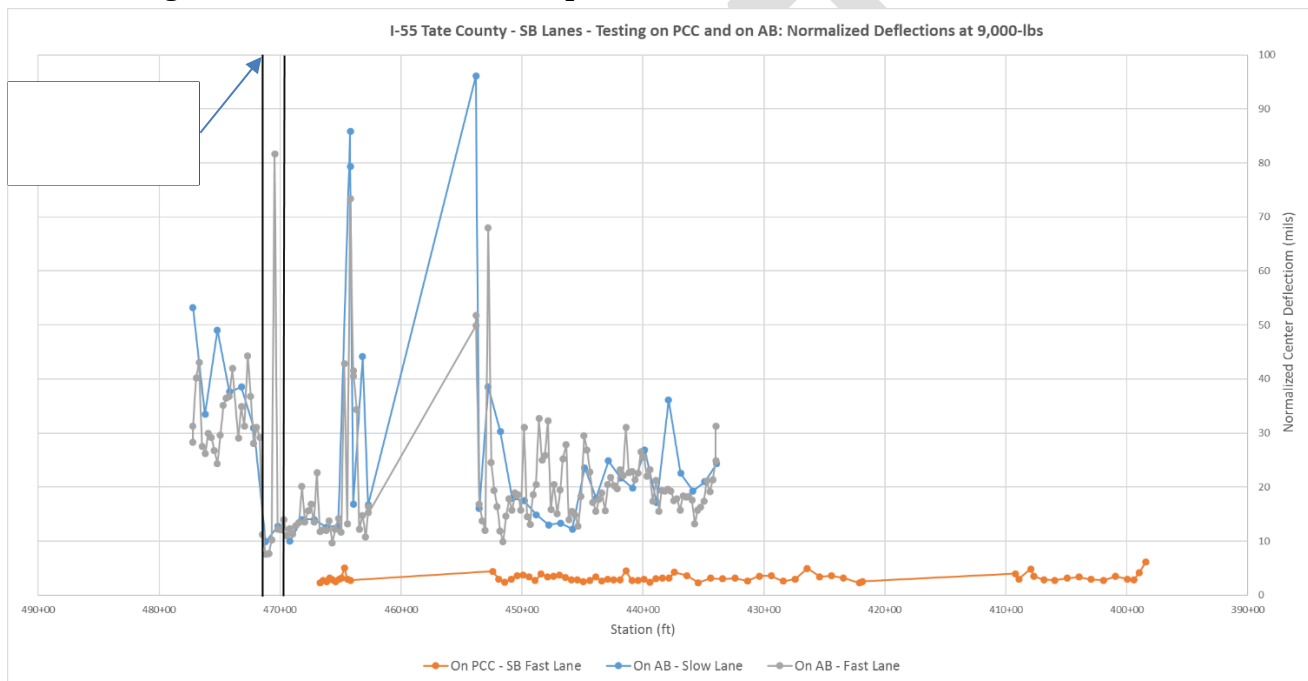


Figure 9. Location where the Rubblized PCC is Visible

Right of Way Images

Right-of-way (ROW) images were collected every 25 feet simultaneously with the FWD and GPR testing. These images assist in making engineering judgments on the backcalculation results, including the condition of layer surface. An example of the ROW image during testing on the intact PCC before rubblization is presented in Figure 10, while Figure 11 displays an example of the ROW image during testing on AB layer after rubblization.



Figure 10. Example ROW Image of PCC Before Rubblization



Figure 11. Example ROW Image of AB After Rubblization

Backcalculation Procedure

The following steps were performed in order to conduct the backcalculation analysis using ELMOD 6.0 software:

Step 1: ARA processed the GPR data and calibrated it using thickness information from as-built data, as no cores were obtained for this section. The GPR data on top of the intact PCC was analyzed, and the thickness data were plotted as illustrated in Figure 12. As mentioned earlier, the layers underneath the PCC layer were combined and treated as a support layer.

The presence of a relatively thick fill soil layer, up to 4 feet, on top of the rubblized PCC posed challenges in identifying the interfaces between the fill soil and rubblized PCC, as well as between the rubblized PCC and the support layer. As a result, the GPR data on top of the AB layer after rubblization was only analyzed where the reinforcement in the rubblized PCC was visible. This approach ensures that the visible layer in the GPR data correspond specifically to the rubblized PCC layer and not any

other layer. Only a portion of the tested section met this criterion, allowing for clear tracing of the AB, CSM, and rubblized PCC layers from approximately Station 471+55 to Station 468+88. In this section, the fill soil appears to be absent. The average PCC thickness prior to rubblization was used to calibrate the thickness of the rubblized PCC layer. The as-built thicknesses were used to calibrate the AB and CSM layers. The thickness data for the southbound fast and slow lanes are shown in Figure 13 and Figure 14, respectively. Then, the data was averaged at 10-ft intervals and were utilized in the backcalculation process.

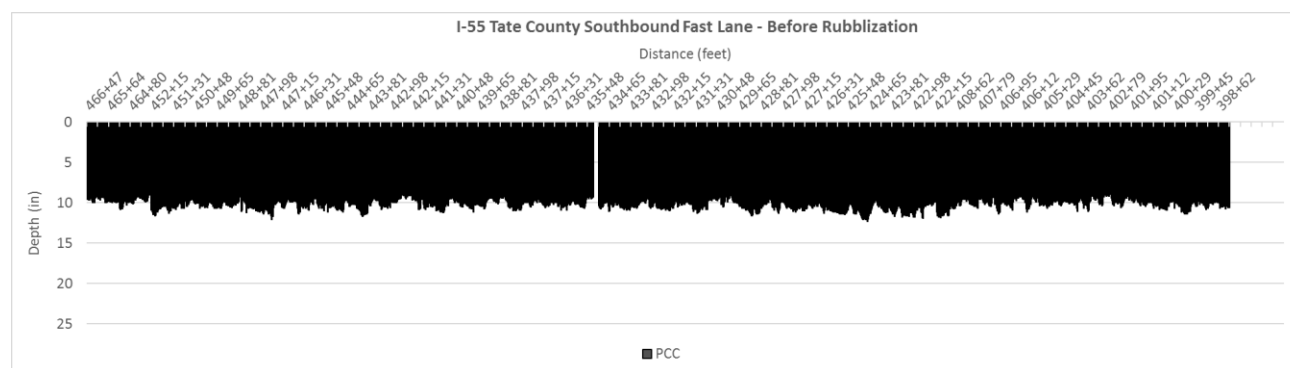


Figure 12. Processed GPR Results Southbound Fast Lane - Before Rubblization

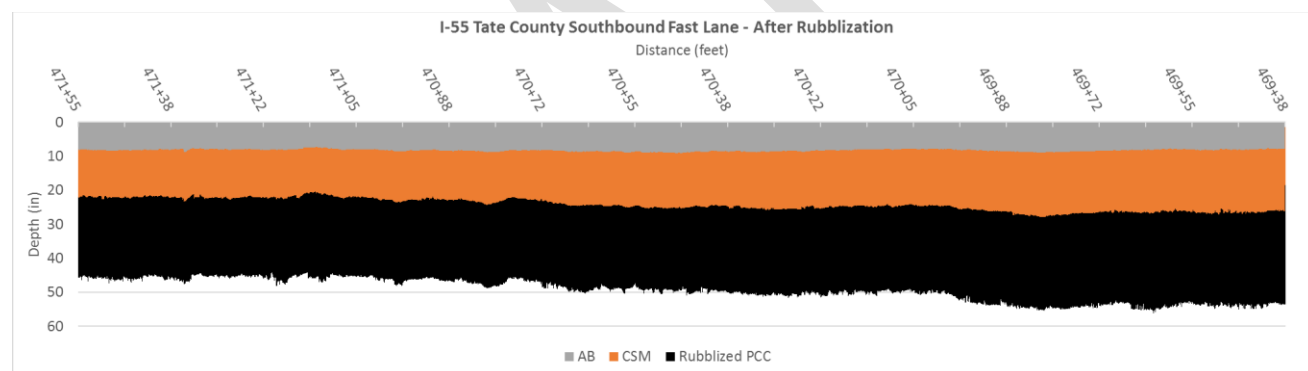


Figure 13. Processed GPR Results Southbound Fast Lane - After Rubblization

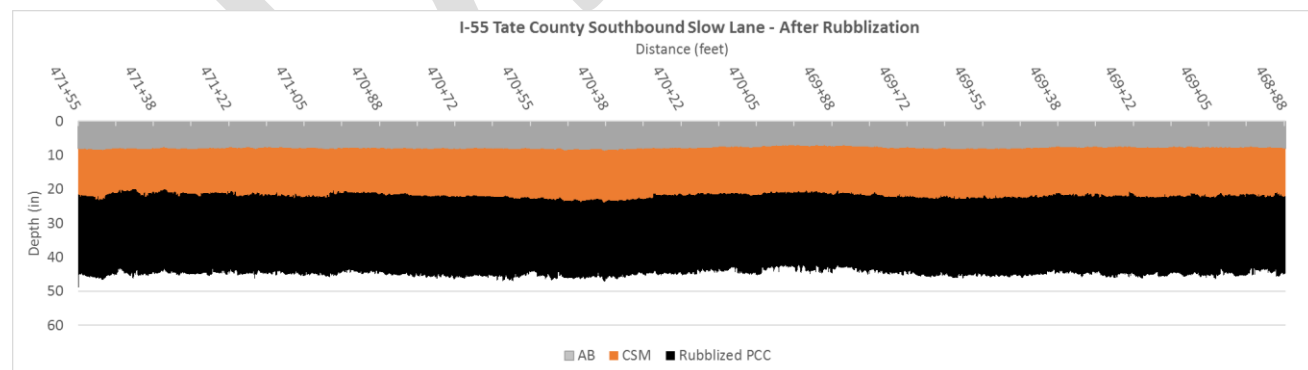


Figure 14. Processed GPR Results Southbound Slow Lane - After Rubblization

Step 2: The surface modulus plot for each test point was examined for abnormalities. Typically, the surface modulus basin should decrease with an increase in distance from load and then slightly increase again if the subgrade is non-linear or if a bedrock is present. Abnormal points can be excluded from the analysis, or the entire station can be excluded if most of the geophone readings need to be removed. Examples of normal and abnormal surface modulus basins are illustrated in Figure 15 and Figure 16, respectively.

For the tests conducted on the PCC, no points were excluded. However, in the case of tests conducted on the AB layer, most surface modulus plots were abnormal. Despite this, no points or stations were deleted, as similar plots are common when testing on uneven surfaces or unbound layers.

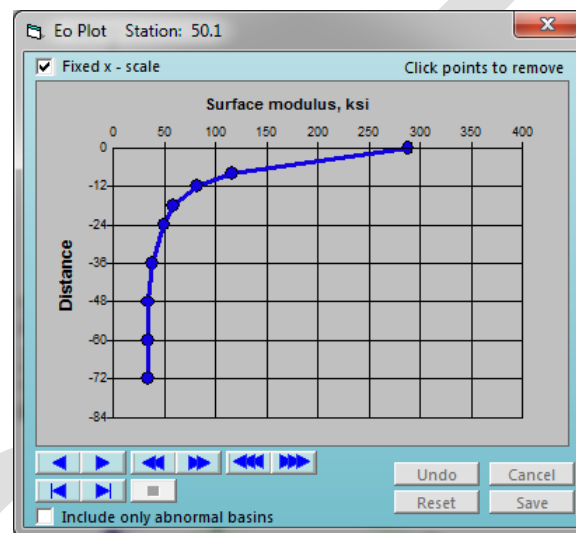


Figure 15. Example of a Normal Surface Modulus Plot when Testing on PCC

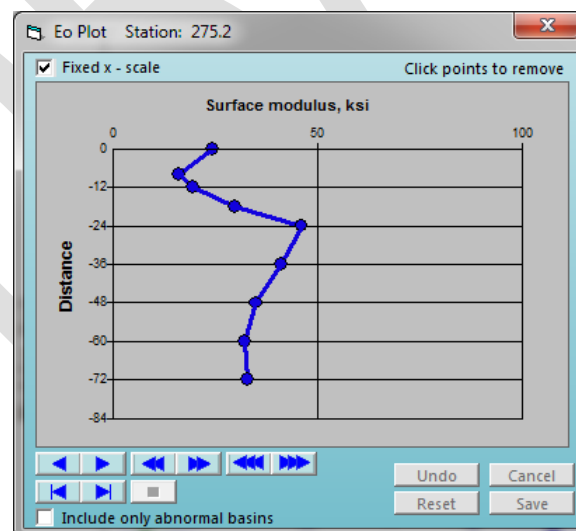


Figure 16. An Abnormal Surface Modulus Plot when Testing on AB

Step 3: A parameter file was set up in ELMOD with different layer types before performing the backcalculation. The layer thickness information obtained from GPR was averaged at 10-ft intervals and used in the backcalculation process. The FWD drop corresponding to the second 9,000-lb load level was used in the analysis. Multiple trials were conducted for each section using the Finite Element (FE) module and the Deflection Basin Fit module in ELMOD for every dataset. A mesh consisting of 24 columns and rows was selected for the FE analysis, and non-linearity of the subgrade was taken into consideration. The average percent differential deflections for all geophones were utilized to compare the various runs and select the trial with the best deflection basin fit.

Previous test sections in Mississippi revealed that using provided seed moduli from laboratory tests in the FE module, and then using the average moduli results from the FE analysis as seed moduli for each layer beneath the HMA layer and above the subgrade layer, resulted in one of the lowest average percent Root Mean Square (RMS) value based on differential deflections for all geophones. In some cases, certain trials produced lower RMS; however, the average layer moduli were found to be unreasonable and/or the standard deviation for the layer moduli was high. Therefore, it is recommended to perform several additional runs with different seed moduli before selecting the final seed moduli, especially when no laboratory modulus of elasticity is provided for use as seed values in the backcalculation process. Once the final set of seed moduli is selected, the depth to bedrock is calculated and checked to determine whether it is shallow or deep. An illustrative example below provides a clearer demonstration of the aforementioned steps.

Sample Backcalculation Procedure for the Section of I-55 Southbound After Rubblization

The following example illustrates the backcalculation procedure for the section on the southbound fast and slow lanes, spanning approximately from Station 471+55 to Station 468+88. This test section is composed of an AB layer over CSM over rubblized PCC over the support layer as shown in Figure 17. It is worth noting that no fill soil was present within this section. Therefore, it was possible to identify the AB, CSM, and rubblized PCC layers from the GPR data.

However, it should be emphasized that the sections before and after the above mentioned stations were not included in the report due to the backcalculated moduli being deemed unreasonable. This unreasonableness resulted from the lack of accurate thickness information from cores and the correct identification of layer types in those sections.

After Rubblization

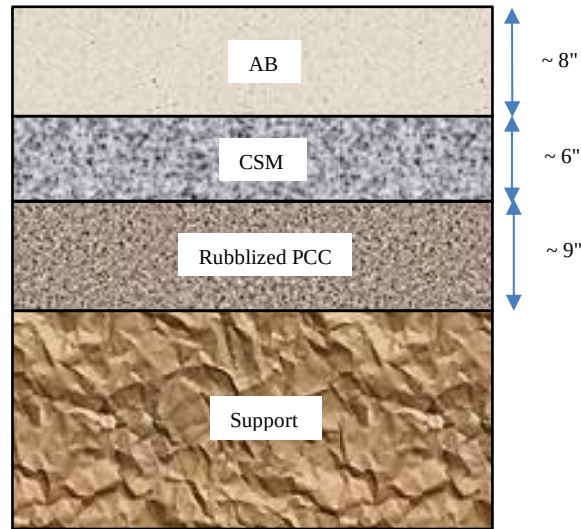


Figure 17. Cross Section of I-55 After Rubblization

Screenshots of the FE module surface modulus and input screen are shown in Figure 18. The FEM button is clicked, the non-linearity of the subgrade “N” is selected, and the FEM mesh is kept at 24 rows/columns. After selecting these options, the “Fit and Save all Points” button is clicked, and the data is then saved in the database. The screenshot below provides an example of the settings used for the FEM methodology.

As no laboratory Modulus of Elasticity (MOE) was provided, approximate seed moduli of 40 ksi, 200 ksi and 200 ksi were initially utilized for the AB, CSM, and rubblized PCC layers, respectively. Different runs were performed using a seed modulus ranging from 100 ksi to 400 ksi for each of the CSM and rubblized PCC layers.

Options Show Calibrate Units

Chainage from 576.1 to 780 From drop 4 to 4

Contact stress 0.080 Radius 5.90000

Use first 9 geophones

Thickness E ny N Emin Emax

8.18	40	0.35	0	1.5	14504.9
6.35	200	0.35	0	1.5	14504.9
8.94	200	0.35	0	1.5	14504.9
	28.2	0.35	-.04	1.5	14504.9

☐ Rock ☐

Save seed values Select point

Fit point Save point

Fit and save all points

☐ Run in background

Generate FEM mesh with

24 rows/ columns

Edit Mesh FEMplot

Close FEM/LET/MET

RMS abs % ☐ minimize %

Deflections FEM LET MET

Distance	FEM	LET	MET
0	7.33		
8	3.88		
63257	2.15		
18	1.77		
26514	1.57		
36	1.48		
53027	1.27		
60	1.07		
72	0.85		

Figure 18. FEM Methodology ELMOD Input Screen

Figure 19 shows an example of the FEM fitting between calculated (in yellow) and measured (in dark red) deflections basins as well as surface modulus.

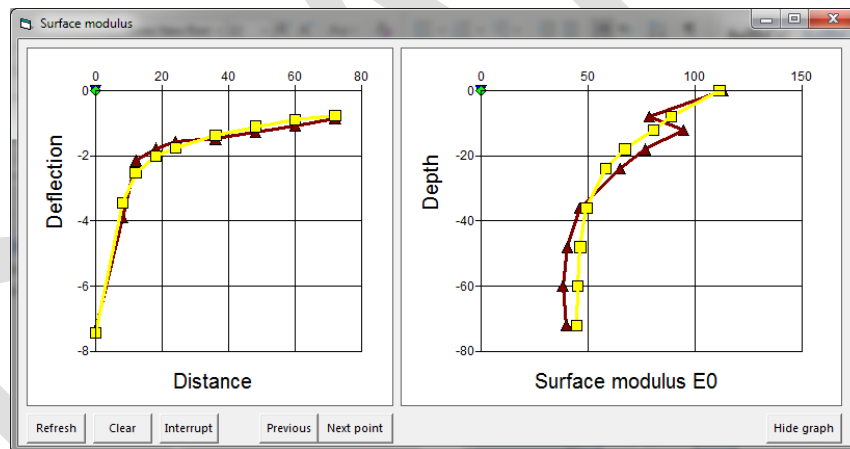


Figure 19. FEM Methodology Fitting Example

Figure 20 summarizes the backcalculation procedure described above in a flowchart format.

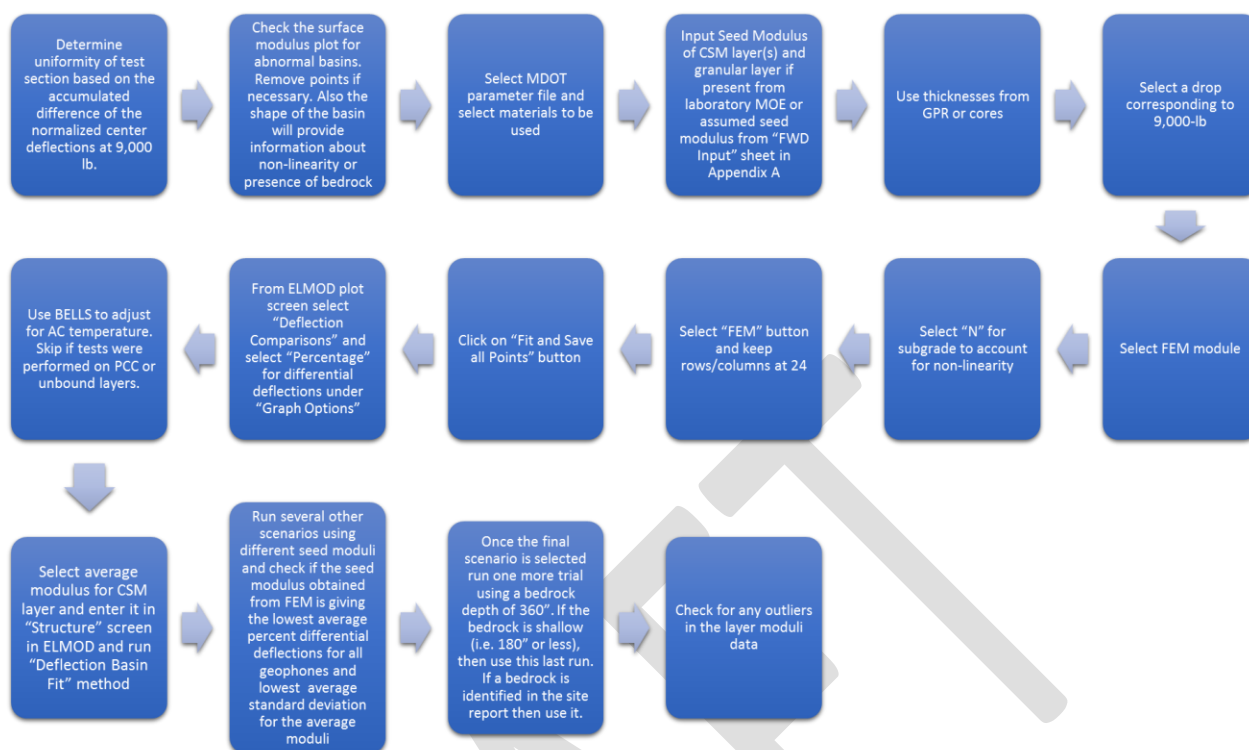


Figure 20. Flowchart of the Backcalculation Procedure

Backcalculation Results

The backcalculation of the data before rubblization was straightforward as there were only two layers, making seeding unnecessary. For the data after rubblization, the procedure outlined in Figure 20 was utilized to estimate the backcalculated layer moduli for each layer. The AB modulus for the different scenarios that were considered ranged on average from 63 ksi to 66 ksi. The CSM modulus for the different scenarios that were considered ranged on average from 54 ksi to 96 ksi. The rubblized PCC modulus for the different scenarios that were considered ranged on average from 234 ksi to 412 ksi. The support modulus for the different scenarios that were considered ranged on average from 57 ksi to 62 ksi.

Taking into account the average percent RMS, the estimated modulus of the rubblized PCC from I-55 Pike County, the literature review performed on rubblized pavements, and the reasonableness of results, a seed modulus of 200 ksi was selected for both CSM and rubblized PCC layers. The selected average moduli results are presented in Table 3 and Table 4 for before and after rubblization, respectively. The standard deviation factor for the AB, CSM, and rubblized PCC was relatively high. Additionally, the support modulus increased significantly after rubblization, which could be attributed to the compaction that occurred during the rubblization process. It should be noted that the number of points that were available for analysis after rubblization was limited. More detailed results are provided in a spreadsheet format in Appendix A.

Table 3. Southbound Fast Lane Average Backcalculation Results – Before Rubblization

Highway	Direction	Lane	Stations	Layer Moduli (ksi)				
I-55	SB	Fast	398+43 to 466+68	Layer	No. of Points	Average	Standard Deviation Factor	84 th Percentile
				PCC	73	3,588	1.938	1,851
				Support	73	26.9	1.480	18.2

Table 4. Southbound Fast & Slow Lanes Average Backcalculation Results – After Rubblization

Highway	Direction	Lane	Stations	Layer Moduli (ksi)				
I-55	SB	Fast & Slow	468+88 to 471+55	Layer	No. of Points	Average	Standard Deviation Factor	84 th Percentile
				AB	13	64.8	2.214	29.3
				CSM	13	74.3	2.483	29.9
				Rubblized PCC	13	233.9	2.402	97.4
				Support	13	62.2	1.263	49.2

APPEDIX A: BACKCALCULATION RESULTS

- I-55 Tate County Backcalculation Results.xlsx

DRAFT

Appendix D

Reporting 16th versus 84th Percentile

MDOT State Study 312 — Recycling PCC Pavements using Rubblization

Final Report

From: [Alvaro Ulloa](#)
To: asaeed@saeedconsult.com
Subject: 16 percentile write up
Date: Friday, October 18, 2024 3:34:52 PM
Attachments: [image001.png](#)

Hi Athar,

Here you go:

For certain measures such as layer moduli, it is prudent to be conservative and to report the 84th percentile as the average – 1 standard deviation (since lower moduli are considered more conservative). Theoretically, this would be the 16th percentile, but for practical reasons, the result is presented as the 84th percentile. Pavement layer moduli may exhibit a very large range in values (even for homogenous sections), in which case the standard deviation may at times be larger than the arithmetic mean, resulting in a negative value for the 84th percentile. In such cases, the average and standard deviation factor are computed based on a log-normal distribution, with the 84th percentile calculated as the ratio of the average to the standard deviation factor.

In essence, the 84th percentile is conservative and practical percentile measure that indicates the percentage of either the normalized deflections or backcalculated layer moduli below which 84 percent of the results may be found. The statistical analysis results are presented to provide a better understanding of the overall pavement layer stiffness. The 84th percentile is used to account for the inherent variability of the material properties, pavement thickness and pavement responses, which in turn is useful to be able to compare the pavement structure between lanes.

Have a great weekend.

Best,

Alvaro Ulloa, PhD, P.E.
Principal Engineer
Geosciences Group Leader



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Appendix E

I-59 Forrest/Jones Counties Non-Destructive Tests

MDOT State Study 312 — Recycling PCC Pavements using Rubblization

Final Report



I-59 Pavement Rehabilitation

FWD Testing and Backcalculation of Rubblized PCC Pavement Moduli

Forrest and Jones Counties, Mississippi

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Introduction

Applied Research Associates (ARA) performed nondestructive testing (NDT) on approximately 9.7 centerline miles of I-59 in Forrest and Jones Counties, Mississippi as shown in Figure 1. The project involved Falling Weight Deflectometer (FWD) testing and backcalculation analysis of the pavement layer moduli to evaluate the structural condition of the rubblized Portland cement concrete (RPCC) pavement before, during, and after rehabilitation.

The rehabilitation treatment consisted of rubblization of the existing Portland cement concrete (PCC) pavement followed by placement of a hot mix asphalt (HMA) overlay. FWD testing was conducted over five rounds spanning from February 2022 to September 2025, capturing the pavement structural response at each stage of construction: intact PCC (pre-rubblization), rubblized PCC, first HMA lift, and final HMA overlay.

A total of 1,240 measurements points were collected across the five testing rounds. This report presents the FWD deflection data, the backcalculation procedure, and the resulting layer moduli for all rounds.

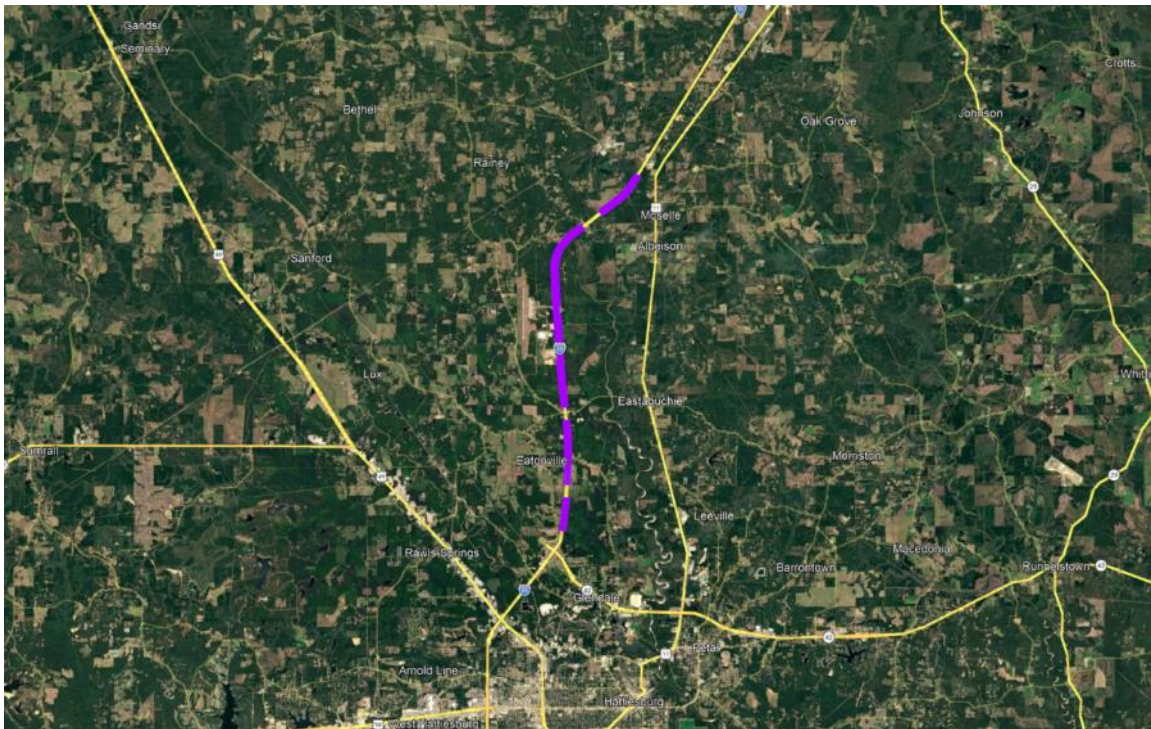


Figure 1. Project Location



Testing Program Overview

FWD testing was performed on the outer lane in both the northbound (NB) and southbound (SB) traveling directions on I-59. The five testing rounds are summarized in Table 1 below and shown graphically in Figure 2 through Figure 8. Rounds 1 and 5 were conducted on stable, intact pavement structures (intact PCC and final HMA overlay, respectively) and produced reliable deflection and backcalculation data. Rounds 2 through 4 were conducted on rubblized PCC or a thin first HMA lift and should be treated as reference data only due to the limitations described in the notes below.

Table 1. Testing Rounds Overview

Round	Date	Surface Condition	Direction	Length (miles)	# Points	Avg ND1 at 9 kips (mils)	Backcalculation Reliability
Round 1	Feb 15 and 16, 2022	Intact PCC	NB and SB	16.2	295	3.6	Reliable
Round 2a	Jun 13, 2022	Rubblized PCC	NB	0.23	62	39.4	Unreliable
Round 2b	Jun 13, 2022	1st HMA Lift	SB	3.2	75	15.0	Unreliable
Round 3	Apr 18, 2023	Rubblized PCC	NB	0.85	189	29.3	Unreliable
Round 4a	Apr 20, 2023	1st HMA Lift	NB	0.85	183	20.1	Unreliable
Round 4b	Apr 20, 2023	Rubblized PCC	NB	0.7	144	39.4	Unreliable
Round 5	Sep 9 and 10, 2025	Final HMA	NB and SB	16.2	292	5.3	Reliable

It should be noted that Rounds 2, 3, and 4 were conducted on rubblized PCC or a thin first HMA lift and should be treated as reference only. Rubblized PCC produces a highly irregular, non-homogeneous surface: the broken slab pieces are uneven and not fully compacted, which causes erratic sensor deflection readings and violates the uniform layer assumption required by layered-elastic backcalculation theory. Additionally, the first HMA lift placed over the rubblized base was thin (typically 2 to 3 in.), introducing two further issues: (1) backcalculation algorithms have difficulty isolating a stiff thin layer from the underlying material, often producing unrealistic moduli (hence the HMA modulus was fixed at 200 ksi); and (2) the thin lift provides limited lateral confinement to the rubblized PCC beneath it. Furthermore, the HMA was tested shortly after placement and had not yet undergone the aging and stiffening that occur over time. Taken together, these factors undermine the fundamental assumptions of layered-elastic backcalculation, making Rounds 2 to 4 data unreliable.

Rounds 1 and 5 represent the full project limits and constitute the longest tested sections, covering approximately 8.1 lane-miles in each direction (approximately 16.2 lane-miles combined per round). The remaining rounds covered shorter segments corresponding to specific construction stages.

Round 1 (Figure 2) shows the original intact PCC over cement stabilized base (CSB) prior to rubblization, tested in February 2022 over the full project extents (16.2 lane-miles). Round 2 is represented by two figures: Figure 3 shows the RPCC with the exposed surface and no HMA overlay, while Figure 4 shows the RPCC with the 1st lift of HMA in place. Round 2 testing (June 2022) covered 3.43 lane-miles within the main project alignment. Rounds 3 and 4 were conducted on a separate segment located south of the main Round 1/Round 5 alignment, outside the primary project limits. Round 3 (Figure 5) tested the RPCC with the exposed surface (0.85 lane-miles, NB only), while Round 4 is split into two figures: Figure 6 shows the RPCC with the 1st lift HMA overlay and Figure 7 shows the RPCC with the exposed surface (1.55 lane-miles total, NB only). Both Rounds 3 and 4 were conducted in April 2023. Round 5 (Figure 8) shows the final completed pavement structure with the full HMA overlay over RPCC and CSB, tested in September 2025 over the full project extents (16.2 lane-miles), matching the Round 1 limits.

Round 1 - Intact PCC

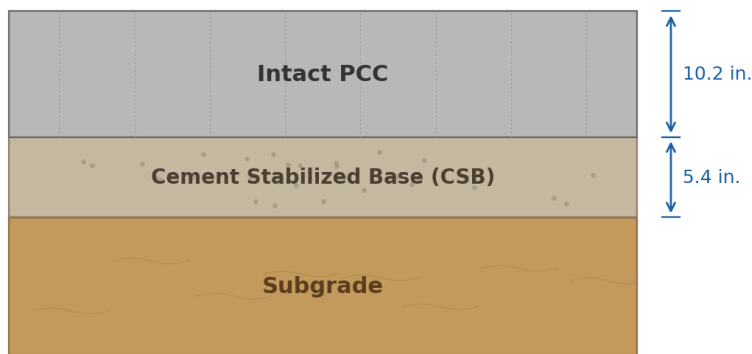


Figure 2. Round 1, Intact PCC Cross-Section (Feb 2022)

Round 2a - Rubblized PCC

Exposed surface (no HMA overlay)

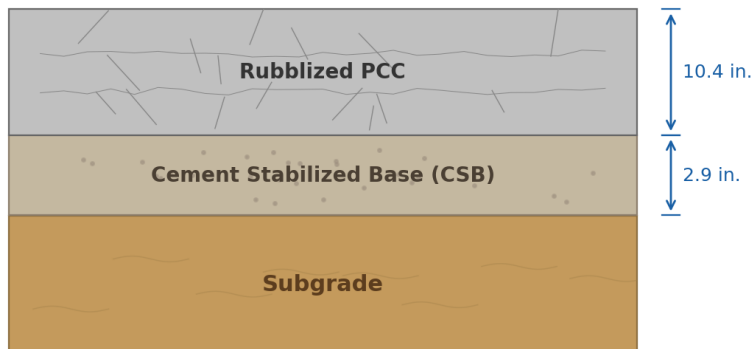


Figure 3. Round 2a, Rubblized PCC Cross-Section (Jun 2022)

Round 2b - 1st Lift HMA over RPCC

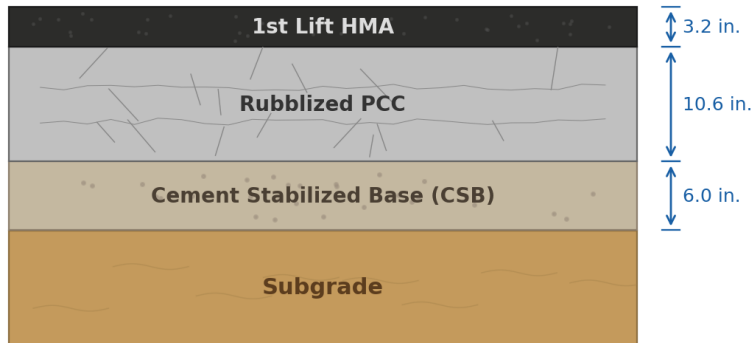


Figure 4. Round 2b, 1st Lift HMA over RPCC Cross-Section (Jun 2022)

Round 3 - Rubblized PCC

Exposed surface (no HMA overlay)

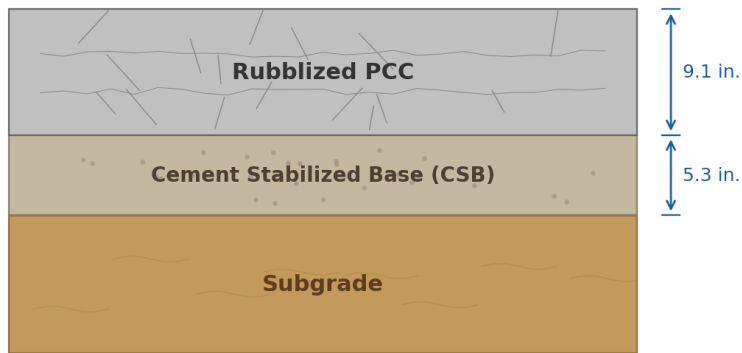


Figure 5. Round 3, Rubblized PCC Cross-Section (Apr 2023)

Round 4a - 1st Lift HMA over RPCC

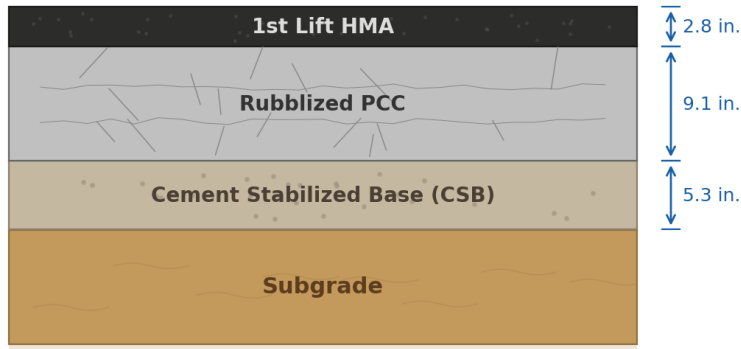


Figure 6. Round 4a, 1st Lift HMA over RPCC Cross-Section (Apr 2023)

Round 4b - Rubblized PCC

Exposed surface (no HMA overlay)

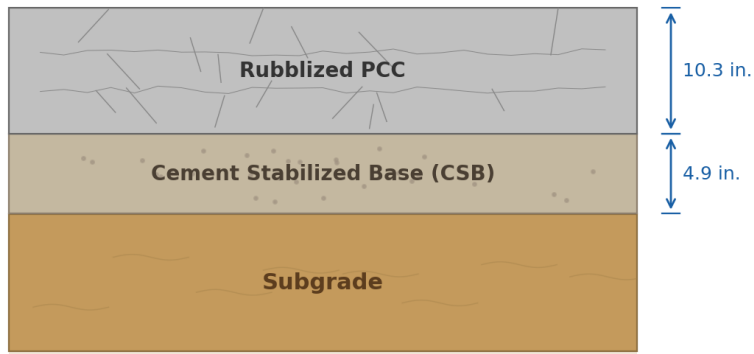


Figure 7. Round 4b, Rubblized PCC Cross-Section (Apr 2023)

Round 5 - Final HMA Overlay

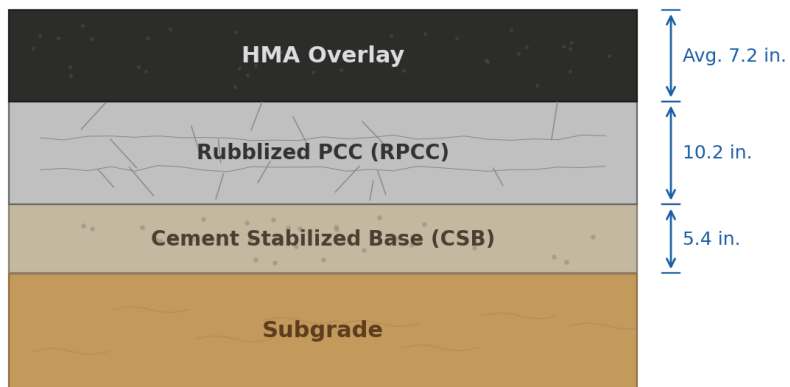


Figure 8. Round 5, Final HMA Overlay Cross-Section (Sep 2025)



Falling Weight Deflectometer Testing

Round 1: Intact PCC (February 2022)

Round 1 FWD testing was performed on February 15 and 16, 2022, on the intact PCC pavement prior to rubblization. A total of 295 FWD stations were tested across both the northbound and southbound lanes. The normalized center deflection at 9,000 lbs load level averaged 3.60 mils across the project, which is within the expected range for intact jointed PCC pavement.

The testing stretch was divided into 13 uniform sections per direction based on the accumulated difference method applied to the normalized center deflections. Table 2 and Table 3 present the deflection statistical summaries for the northbound and southbound lanes, respectively. The deflection profiles for both directions are shown in Figure 9 and Figure 10.

Table 2. Round 1 - NB Center Deflection Summary (Intact PCC)

Section	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
1	0	6,018	2.69	0.45	3.14	25
2	7,787	10,791	3.61	0.92	4.53	13
3	11,045	17,766	3.74	0.59	4.32	28
4	18,022	19,751	3.71	0.81	4.52	8
5	21,539	24,284	3.59	0.38	3.97	12
6	24,543	29,275	4.03	0.55	4.58	20
7	29,530	31,773	3.53	0.48	4.01	10
8	32,023	32,792	3.68	0.65	4.33	4
9	35,772	37,761	3.50	0.47	3.98	9
10	38,014	38,782	3.88	0.47	4.36	5
11	39,041	41,023	4.85	1.14	5.99	9
12	41,330	42,293	4.16	0.39	4.55	5
13	42,551	43,005	3.74	0.32	4.06	3

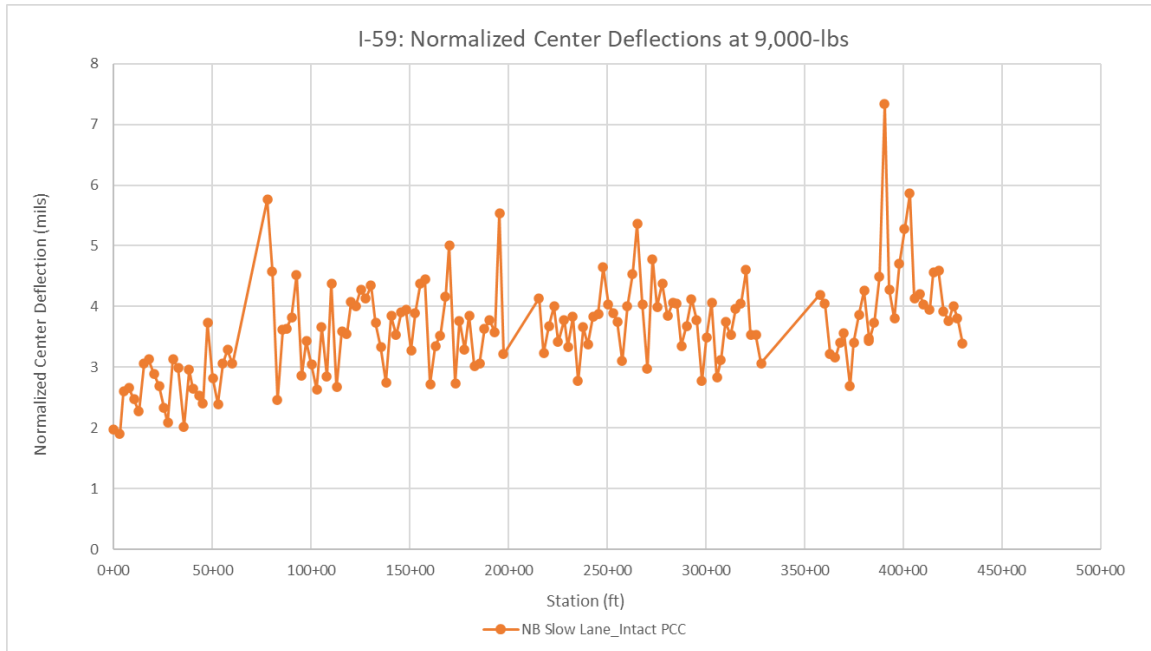


Figure 9. I-59 NB Normalized Center Deflections at 9,000 lbs (Round 1)

Table 3. Round 1 - SB Center Deflection Summary (Intact PCC)

Section	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
1	5	5,759	3.77	0.50	4.27	24
2	7,739	8,719	3.93	0.89	4.82	5
3	8,973	11,468	3.71	0.63	4.34	11
4	11,718	13,703	3.78	0.26	4.04	9
5	14,985	17,478	3.57	0.51	4.08	11
6	17,733	19,259	3.86	0.44	4.30	7
7	19,518	22,011	3.88	0.24	4.12	4
8	22,211	25,216	3.24	0.53	3.77	13
9	25,474	29,516	3.30	0.49	3.80	17
10	29,772	31,760	3.03	0.61	3.64	9
11	31,961	32,984	3.00	0.27	3.27	5
12	35,963	40,510	3.53	0.68	4.21	19
13	40,760	43,008	3.99	1.96	5.95	10

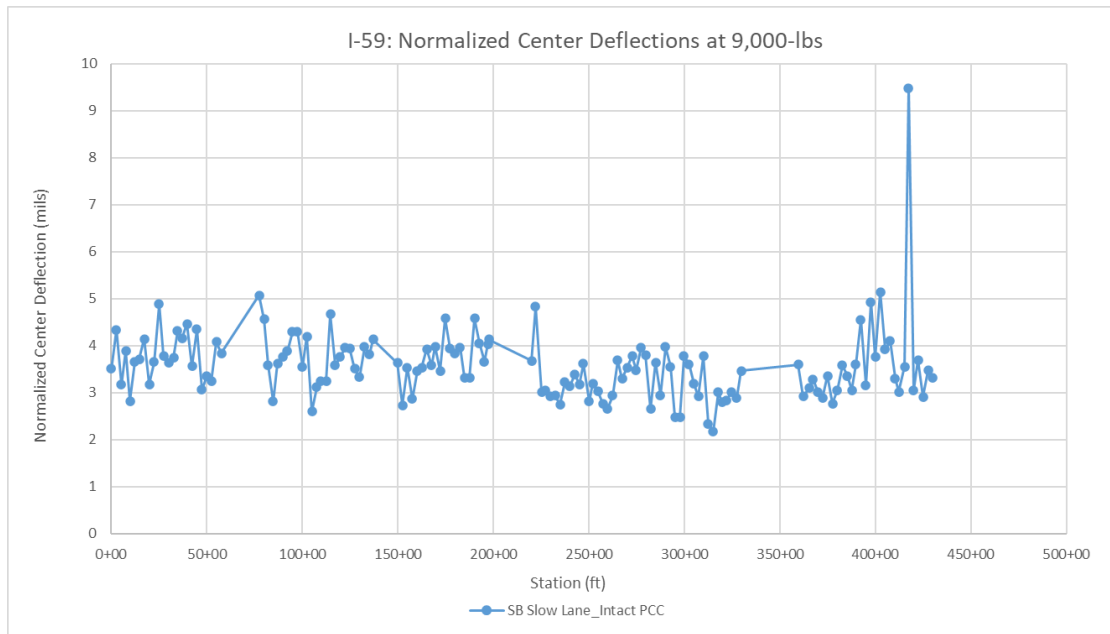


Figure 10. I-59 SB Normalized Center Deflections at 9,000 lbs (Round 1)

Round 2: Rubblized PCC / 1st Lift HMA (June 2022)

Round 2 was conducted on June 13, 2022. At this stage, a short section of the northbound lane corresponding to Sections 12 and 13 had been rubblized but not yet overlaid, while a section of the southbound lane corresponding to Sections 8 to 13 had received the first HMA lift. A total of 137 FWD stations were tested. The average normalized center deflection was 39.4 and 15.0 mils, respectively, which is much higher than the intact PCC values, as expected for rubblized PCC and thin first-lift HMA. These deflection levels reflect the reduced structural stiffness of the rubblized material and the inability of a thin HMA layer to distribute load effectively. Table 4 presents the deflection statistical summaries for the northbound and southbound sections. The deflection profiles for both directions are shown in Figure 11 and Figure 12.

Table 4. Round 2 - Center Deflection Summary (Rubblized PCC / 1st Lift HMA)

Section	Surface Condition	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
12	Rubblized PCC	41,755	42,257	31.10	22.41	53.52	23
13	Rubblized PCC	42,282	42,995	44.27	33.00	77.26	39
8	1st HMA Lift	22,435	25,126	16.81	2.25	19.05	12
9	1st HMA Lift	25,372	29,361	15.15	3.15	18.29	17
10	1st HMA Lift	29,612	31,857	13.85	2.73	16.58	10
11	1st HMA Lift	32,105	33,076	16.00	2.50	18.50	5
12	1st HMA Lift	35,890	40,575	14.12	3.54	17.66	21
13	1st HMA Lift	40,823	42,938	15.06	6.99	22.05	10

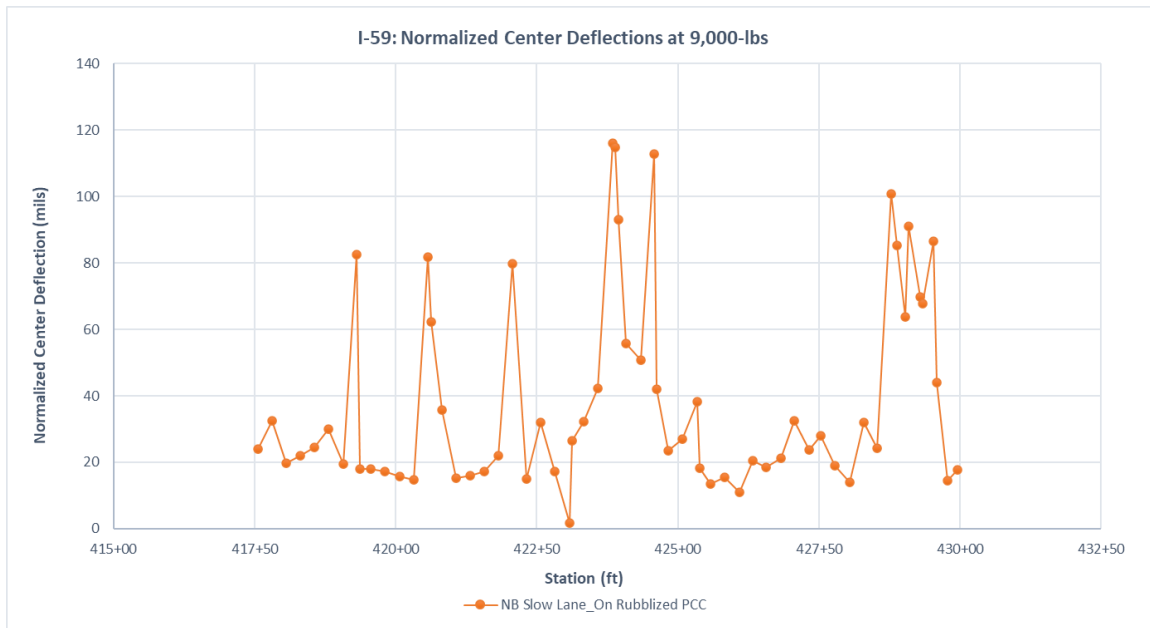


Figure 11. I-59 NB Normalized Center Deflections at 9,000 lbs (Round 2a)

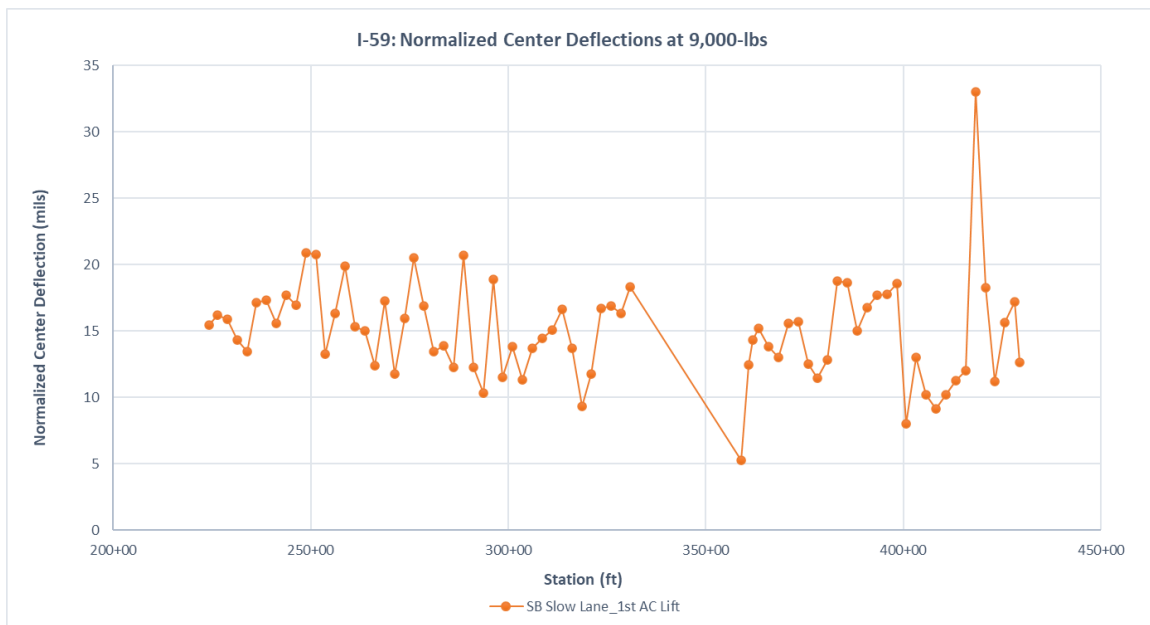


Figure 12. I-59 SB Normalized Center Deflections at 9,000 lbs (Round 2b)

Round 3: Rubblized PCC (April 2023)

Round 3 was conducted on April 18, 2023, on the northbound lane, which consisted of rubblized PCC without any HMA overlay. This section is located south of the main Round 1/Round 5 alignment and falls outside the primary project limits. A total of 189 FWD points were tested over a single section from 0 to 4,500 ft. The average normalized center deflection



was 29.3 mils with a coefficient of variation of 38.5%, reflecting the inherent irregularity of the rubblized PCC surface. Table 5 presents the deflection statistical summary and Figure 13 shows the deflection profile for this section.

Table 5. Round 3 - Center Deflection Summary (Rubblized PCC)

Section	Surface Condition	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
1	Rubblized PCC	0	4,500	29.27	11.28	40.55	189

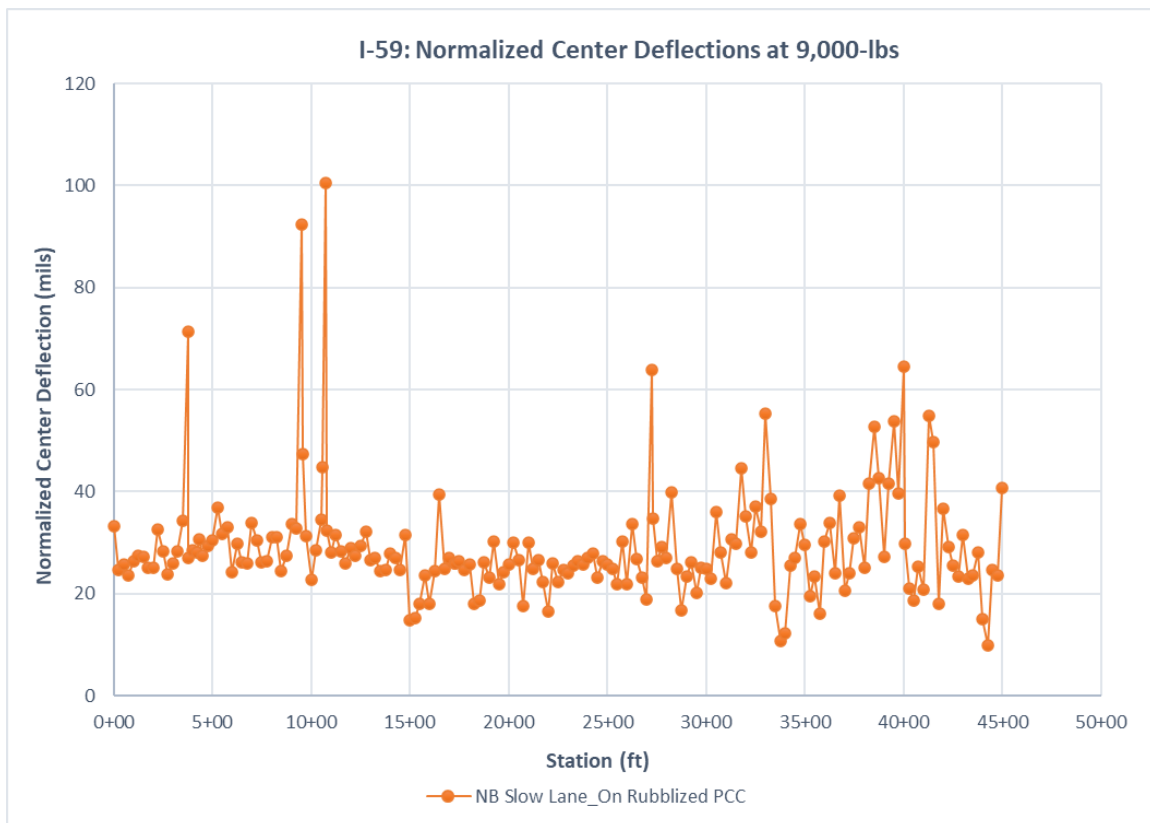


Figure 13. I-59 NB Normalized Center Deflections at 9,000 lbs (Round 3)

Round 4: 1st Lift HMA / Rubblized PCC (April 2023)

Round 4 was conducted on April 20, 2023, on the northbound lane only, south of the main project limits. Section 1 (0 to 4,512 ft) received the first HMA lift (Round 4a), while Section 2 (6,080 to 9,701 ft) remained on rubblized PCC (Round 4b). A total of 327 FWD stations were tested. The 1st lift HMA section (Round 4a) averaged 20.1 mils (COV 13.9%), while the rubblized PCC section (Round 4b) averaged 39.4 mils (COV 35.4%), consistent with the much weaker rubblized PCC surface compared to the 1st lift section. Table 6 presents the deflection statistical summary and Figure 14 shows the deflection profile for both sections.

Table 6. Round 4 - Center Deflection Summary (1st Lift HMA/ Rubblized PCC)

Section	Surface Condition	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
1	1st HMA Lift	0	4,512	20.08	2.79	22.86	183
2	Rubblized PCC	6,080	9,701	39.42	13.97	53.39	144

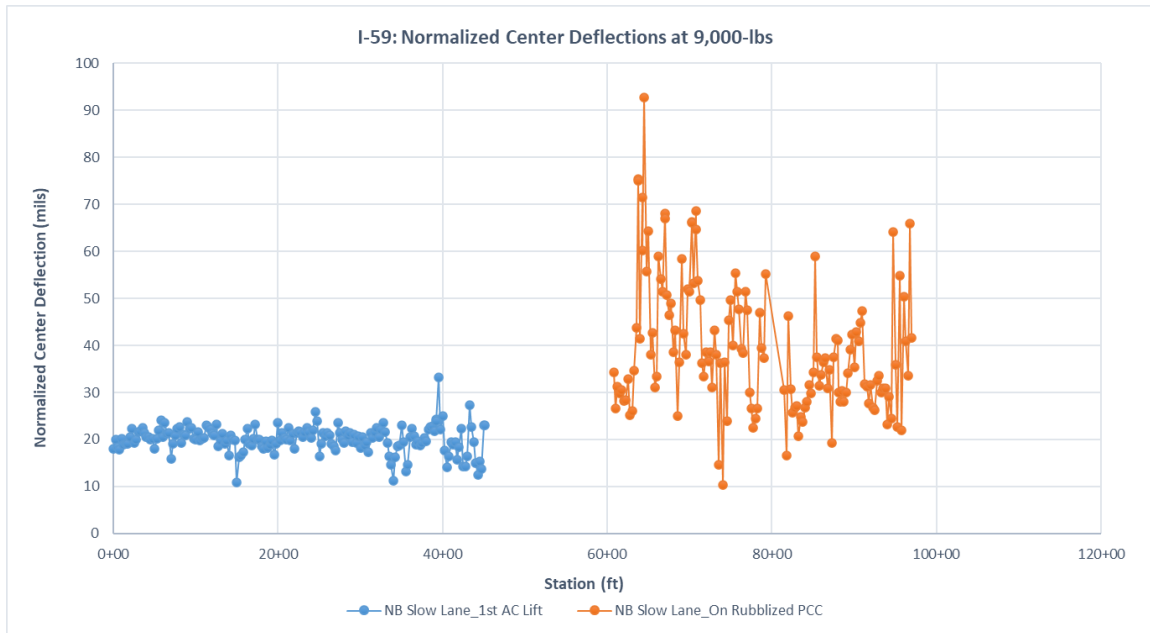


Figure 14. I-59 NB Normalized Center Deflections at 9,000 lbs (Rounds 4a and 4b)

Round 5: Final HMA Overlay (September 2025)

Round 5 was conducted on September 9 and 10, 2025, after the full HMA overlay had been placed and had been in service. A total of 292 FWD stations were tested across both directions. The average normalized center deflection was 5.26 mils, which is approximately 46% higher than the Round 1 intact PCC average of 3.60 mils. Table 7 and Table 8 present the deflection statistical summaries for the northbound and southbound lanes, respectively. The deflection profiles for both directions are shown in Figure 15 and Figure 16.

Table 7. Round 5 - NB Center Deflection Summary (Final HMA)

Section	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
1	22	6,049	5.05	0.96	6.01	25
2	7,816	10,835	4.42	1.08	5.50	13
3	11,089	17,615	4.95	0.75	5.70	27
4	17,866	19,873	6.90	1.19	8.10	9
5	21,383	24,400	4.54	0.72	5.26	13
6	24,657	29,431	6.35	1.70	8.05	20
7	29,677	31,943	4.49	0.31	4.80	10
8	32,204	32,698	6.06	0.29	6.35	3
9	35,968	37,976	5.12	0.46	5.59	9
10	38,231	38,984	6.32	0.38	6.70	4
11	39,239	41,248	3.90	0.61	4.51	9
12	41,496	42,257	6.77	1.46	8.23	4
13	42,503	43,001	4.25	0.32	4.58	3

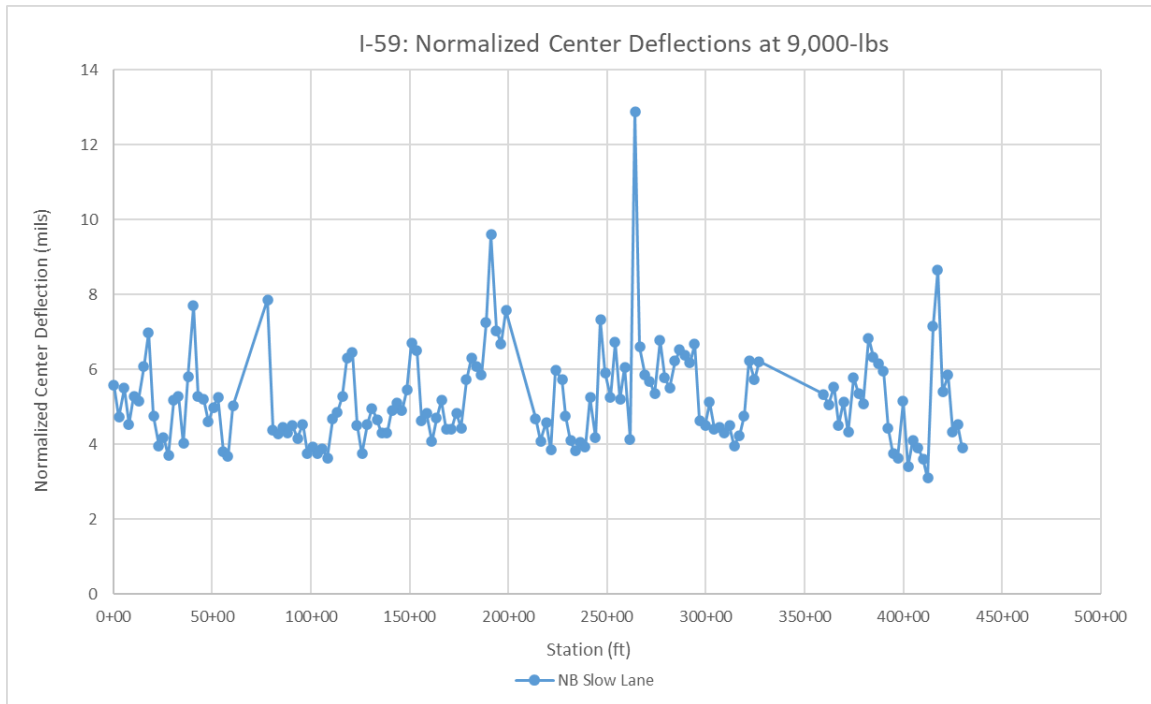


Figure 15. I-59 NB Normalized Center Deflections at 9,000 lbs (Round 5)

Table 8. Round 5 - SB Center Deflection Summary (Final HMA)

Section	From (ft)	To (ft)	Avg ND1 at 9 kips (mils)	Std Dev	84th %ile	N
1	69	5,553	5.85	1.09	6.94	23
2	7,616	8,617	7.56	1.41	8.98	5
3	8,874	11,389	4.59	0.73	5.33	11
4	11,643	13,899	5.80	1.28	7.08	10
5	15,158	17,421	5.48	1.00	6.47	11
6	17,678	19,181	4.32	0.55	4.87	7
7	19,437	21,951	8.30	0.76	9.06	3
8	22,201	25,222	5.39	0.87	6.25	13
9	25,475	29,494	5.24	1.16	6.39	17
10	29,746	31,757	3.95	0.45	4.40	9
11	32,006	33,015	5.83	1.04	6.88	5
12	36,030	40,559	5.25	0.70	5.95	19
13	40,807	42,944	4.26	0.77	5.03	10

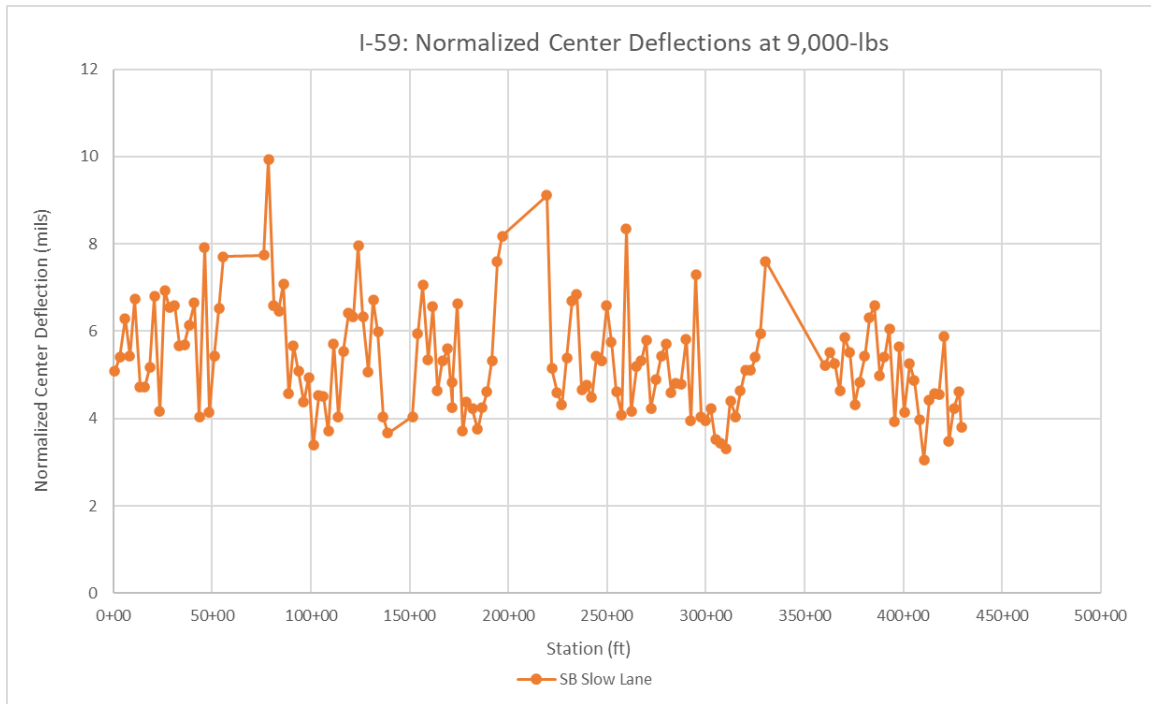


Figure 16. I-59 SB Normalized Center Deflections at 9,000 lbs (Round 5)



Field Coring

Pavement coring was performed by Burns Cooley Dennis, Inc. (BCD) over four days: September 30 and October 1, 2025 for the northbound lane, and October 7 and 8, 2025 for the southbound lane. A total of 27 cores were extracted across both directions: 14 cores in the northbound lane (Cores 1 through 14) and 13 cores in the southbound lane (Cores 15 through 27). The core locations were selected by ARA to capture representative conditions within each of the uniform sections identified from the deflection data. Some core locations had to be adjusted from their original positions due to proximity to on- and off-ramps or because they fell within sections that were not rubblized. At each core location, BCD also collected GPR scans consisting of one transverse scan and two longitudinal scans for calibration purposes.

The core logs identified four primary pavement layers: HMA overlay, rubblized PCC (or intact concrete/limestone at select locations), cement stabilized base, and cement-treated sand subgrade. The majority of the cores (21 of 27) encountered rubblized concrete beneath the HMA. The remaining six locations (Cores 3, 10, 15, 17, 21, and 27) showed intact concrete or limestone, indicating sections of the original pavement that were not rubblized. Measured HMA thicknesses ranged from 5.0 to 11.25 in., rubblized PCC thicknesses ranged from 9.5 to 10.5 in., and cement stabilized base thicknesses ranged from 3.0 to 7.0 in. At several locations, the stabilized base sample could not be recovered due to material degradation.

The measured layer thicknesses from the cores were input into RADAN software to calibrate the GPR traces along the full project length.

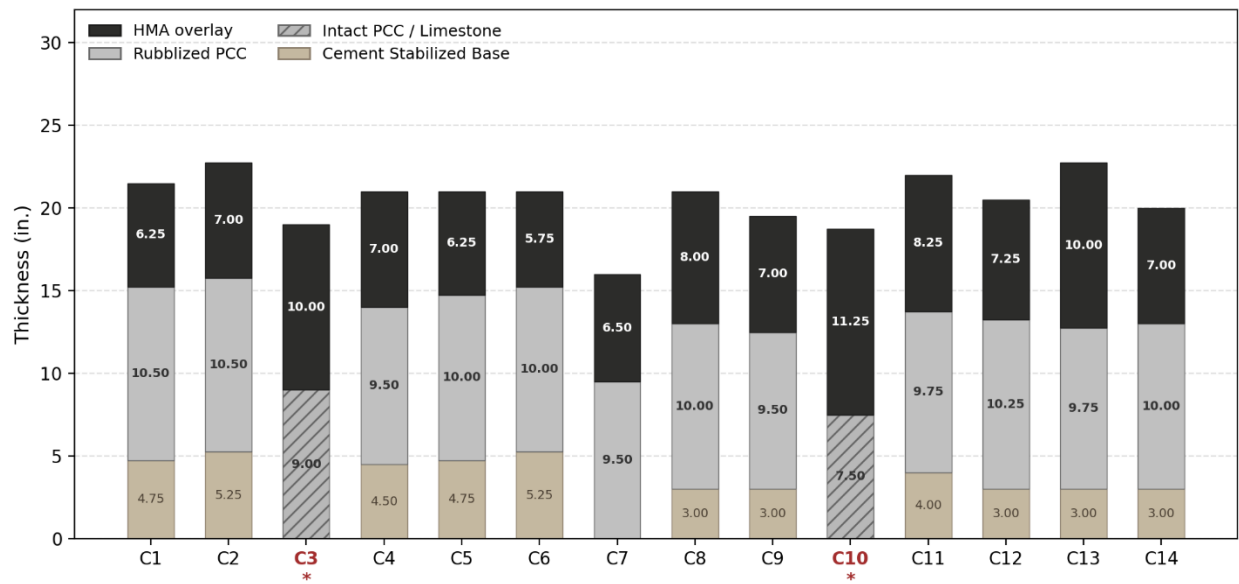
The measured layer thicknesses at each core location are summarized in Table 9 and illustrated graphically in Figure 17 and Figure 18 for the northbound and southbound lanes, respectively. Cores highlighted in pink represent locations where the original PCC was not rubblized (either intact concrete or limestone sections).

Table 9. Core Thickness Measurements on I-59

1	NB	6.25	R/Conc	10.50	4.75	Rubblized
5	NB	6.25	R/Conc	10.00	4.75	Rubblized
7	NB	6.50	R/Conc	9.50	---	Rubblized
9	NB	7.00	R/Conc	9.50	3.00	Rubblized
10						
11	NB	8.25	R/Conc	9.75	4.00	Rubblized
13	NB	10.00	R/Conc	9.75	3.00	Rubblized
15						
19	SB	8.75	R/Conc	10.00	4.25	Rubblized
20						
23	SB	5.00	R/Conc	10.00	5.00	Rubblized
25	SB	6.25	R/Conc	10.00	6.50	Rubblized
26	SB	6.50	R/Conc	10.50	6.50	Rubblized
27	SB	9.25	Limestone	11.00	---	Not rubblized

- 1) AC = Asphalt Concrete
- 2) R/Conc = Rubblized Concrete, C/Conc = Crushed Concrete
- 3) CSB = Cement Stabilized Base

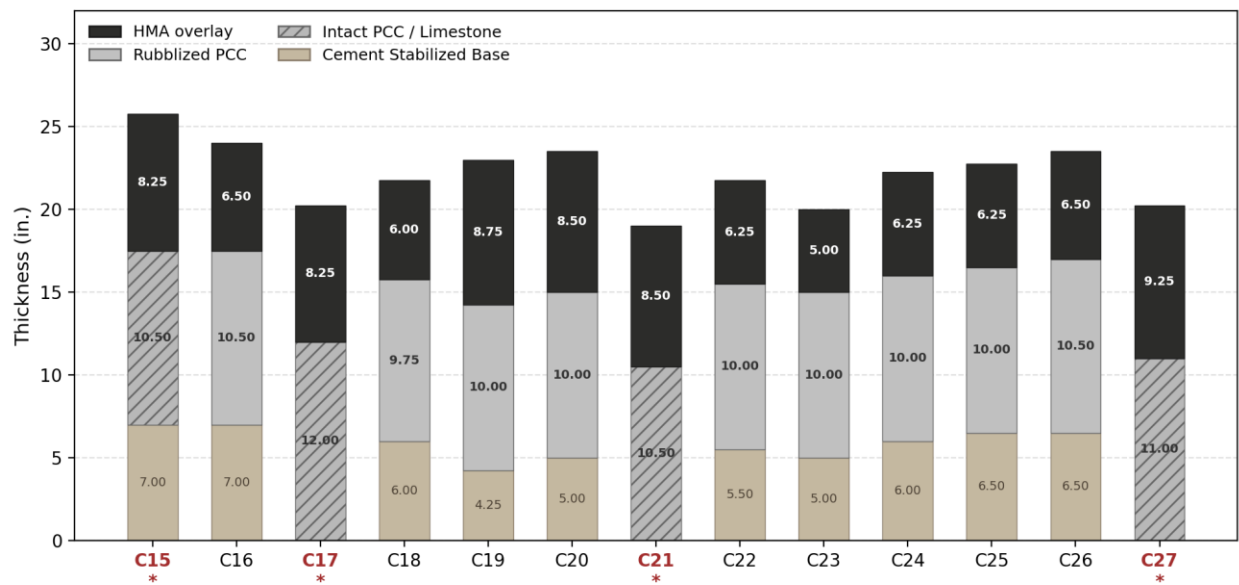
I-59 Core Thickness Summary - Northbound Lane



* Core location where PCC was not rubblized (original intact concrete or limestone)

Figure 17. Northbound Thickness Summary

I-59 Core Thickness Summary - Southbound Lane



* Core location where PCC was not rubblized (original intact concrete or limestone)

Figure 18. Southbound Thickness Summary



Ground Penetrating Radar Testing

Continuous ground penetrating radar (GPR) testing was performed along the same pavement sections of I-59 to estimate the in-place thicknesses of each pavement layer. The GPR data were subsequently processed and calibrated against ground-truth measurements obtained from the pavement cores, which provided the reference layer thicknesses used to convert radar travel times into depth.

The testing was conducted using a GSSI SIR-30 data acquisition system with two antennas operated simultaneously to capture both shallow and deeper pavement features. An air-launched antenna with a center frequency of 2.0 GHz was used to resolve the surface HMA layer with high accuracy, taking advantage of its fine vertical resolution and non-contact operation, which allows testing at highway speeds. A ground-coupled antenna with a center frequency of 900 MHz was used in parallel to penetrate deeper into the pavement structure (up to approximately 2 ft), allowing identification of the underlying rubblized PCC, cement stabilized base, and support interfaces. Figure 19 shows the GPR equipment used on this project, and Figure 20 presents a sample of the raw GPR data on the I-59 northbound lane after placement of the final HMA overlay.

The processed GPR data were used to generate continuous pavement layer profiles along the project length. Figure 21 presents the interpreted layer thickness profile for the northbound lane after placement of the final HMA overlay, showing the HMA surface, the underlying RPCC, the CSB, and the sections where intact limestone was encountered in place of RPCC. The station-by-station layer thicknesses extracted from this profile were used as direct input to the structural analysis and backcalculation of the pavement moduli presented in the following sections.



Figure 19. GPR Unit with Double Antennas

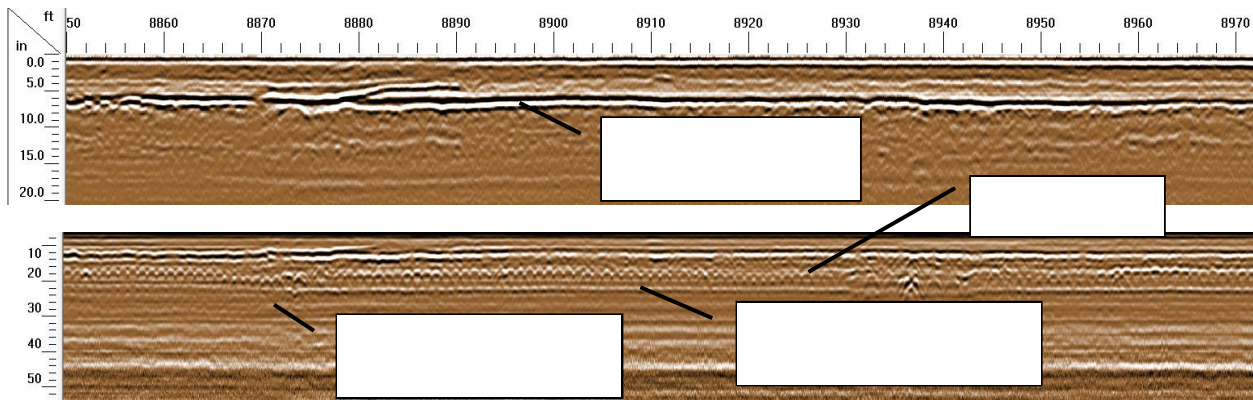


Figure 20. Raw GPR Data Example, I-59 NB Lane After Placement of Final HMA Overlay

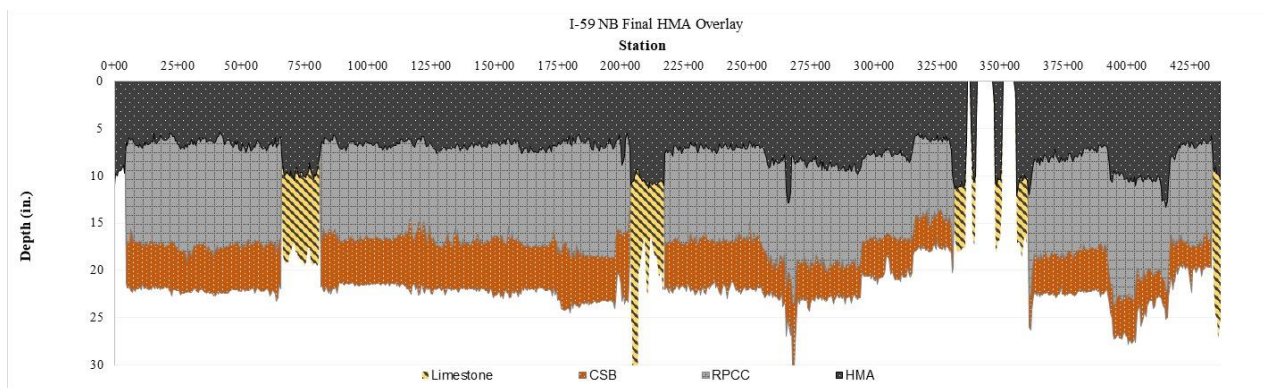


Figure 21. GPR Layer Thickness Profile, I-59 NB After Placement of Final HMA Overlay

Backcalculation Procedure

The load-deflection data generated by the FWD were analyzed using the ELMOD 6 computer program, following the backcalculation procedure developed by ARA under MDOT State Study 304. The analysis for this highway was conducted after the completion of State Study 304, allowing the procedure to incorporate refinements informed by the findings of that study (whereas the analyses for the other two highways were performed during the earlier stages of State Study 304). The overall methodology remains consistent with that established under State Study 304. ELMOD is a software tool used to backcalculate the mechanistic material properties of an axisymmetric, semi-infinite pavement system, and it accounts for the nonlinear behavior of subgrade soils, which is critical for producing reliable backcalculated moduli. The procedure described below applies to the reliable rounds (Rounds 1 and 5), while Rounds 2 through 4 were processed with additional constraints as noted at the end of this section.



As a result of the data screening described in Steps 5, 9, and 10, the number of FWD points included in the final backcalculation results is, in some sections, lower than the total number of points tested in the field. The deflection summary tables (Table 2 through Table 8) report all points tested, whereas the backcalculation results tables (Table 10 through Table 18) report only the points retained after screening.

The following steps summarize the backcalculation procedure applied to the I-59 FWD data:

Step 1: Pavement Sections and Data Review: The pavement cross-section for the I-59 project was defined based on the available design information, GPR data, and pavement cores. For Round 1 (intact PCC), the backcalculation cross-section consisted of three layers: Intact PCC / CSB / Support. For Round 5 (final HMA overlay), the cross-section consisted of four layers: HMA / RPCC / CSB / Support. All FWD data was reviewed to ensure completeness and only data corresponding to the outer lane between wheel paths was used in the analysis.

Step 2: Checking FWD Data for Uniformity and Section Subdivision: The FWD data for each testing round was thoroughly examined for uniformity using the accumulated difference method of the normalized center deflections at 9,000 lb. This method identifies variations within the deflection data by calculating the cumulative sum of the deviations from the mean deflection value for the entire section. Subsection boundaries were identified at locations where significant changes in the slope of the accumulated difference occurred. GPR and coring data were also utilized in the segmentation process. Following this analysis, each traveling direction was divided into 13 uniform sections.

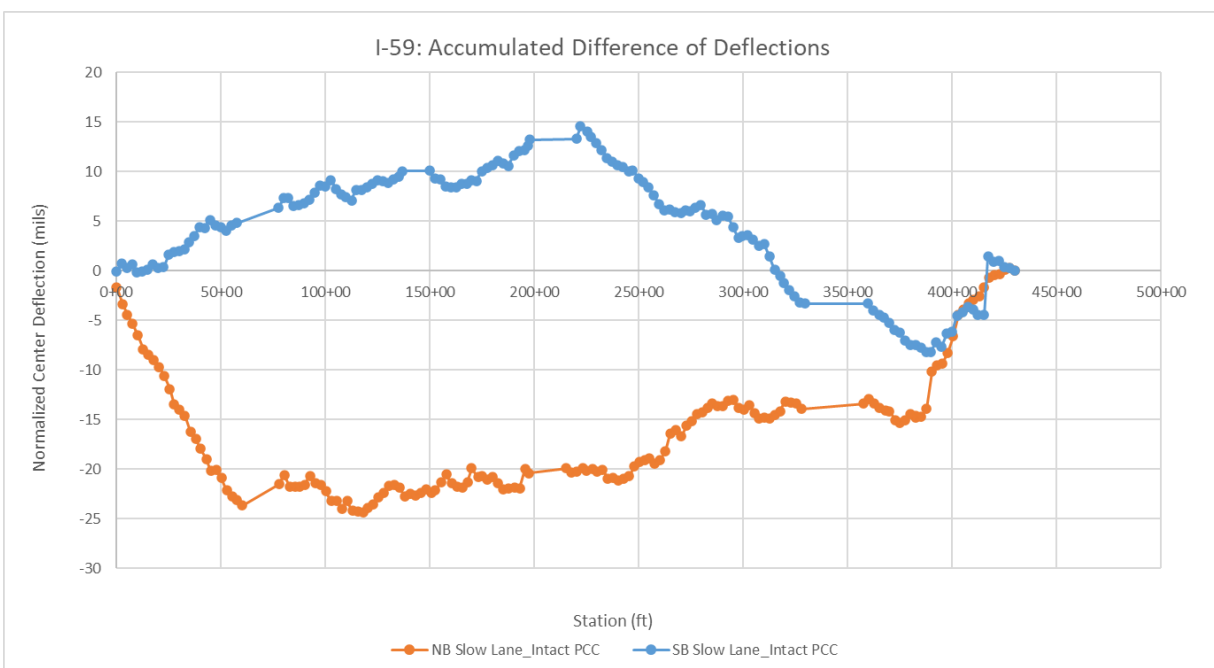


Figure 22. Accumulated Difference of Deflections NB and SB on Intact PCC



Step 3: Layer Thickness Information: The layer thickness information was incorporated into the backcalculation process. HMA thickness data derived from GPR measurements was provided at regular intervals and entered into the GPR data table in ELMOD, ensuring that each FWD station was associated with its corresponding thickness. Layer thickness was determined from pavement cores and GPR calibration.

Step 4: Setting up the Parameter File in ELMOD: Before performing the backcalculation, a parameter file was created in ELMOD to define the various layer types within the pavement structure. This setup is critical for temperature-dependent materials such as HMA, as it ensures that the backcalculated moduli are adjusted to a reference temperature.

Step 5: Surface Modulus Plot Evaluation and Handling Abnormal Data: The surface modulus plot for each FWD test point was carefully reviewed to identify abnormalities. Typically, the surface modulus profile decreases as the distance from the load increases, with a possible slight increase if a non-linear subgrade or a stiff underlying layer is present. If an abnormality involved one of the first three deflections or if most deflections at a test point were irregular, the entire FWD test point was eliminated from the analysis. ELMOD does not permit the removal of any of the first three deflections closest to the load, as these are critical to the backcalculation process. Removal of specific sensors past the third sensor was deferred until after the final backcalculation method was selected to ensure consistency when comparing RMS values across trials.

Step 6: Selection of Load Drop for Analysis: The load drop corresponding to approximately 9,000 lbs was selected for all backcalculation analyses. The 9,000-pound drop simulates the effect of an 18,000-pound single-axle load and is the standard load level used in backcalculation.

Step 7: Temperature Normalization for HMA Layer Moduli: The mid-depth temperature of the HMA layers was determined using the BELLS equation. The BELLS equation requires the average air temperature of the 'previous day' (one day prior to FWD data collection). Temperature data was obtained from www.wunderground.com website using the closest weather stations to each pavement test site. The primary objective of this step was to normalize the resulting HMA layer moduli to a reference temperature of 77°F.

Step 8: Selection of Seed Moduli and Backcalculation: The backcalculation procedure outlined in SS #304 was followed for this project, as illustrated in Figure 23. It should be noted, however, that certain steps within that procedure were not used because the corresponding laboratory data was not available. The selection of seed moduli for each layer is a critical step that significantly influences the backcalculation results. Multiple scenarios were tested for each



section, and the scenario producing the lowest RMS error, reasonable standard deviation, and moduli values consistent with laboratory and estimated values was selected. For the CSB layers, one of the trial scenarios used a seed modulus of 206 ksi, the average value for cement stabilized base materials reported in SS #304, as project-specific laboratory data for this layer was not available. For the I-59 project, the Deflection Basin Fit (DBF) module in ELMOD was the primary backcalculation method. The DBF module employs the Method of Equivalent Thicknesses (MET) and initiates with estimated moduli, calculates the theoretical deflection bowl, and iteratively adjusts the moduli until a minimum error between calculated and measured deflection basins is achieved. Subgrade non-linearity was accounted for in all analyses. No stiff layer was introduced in the backcalculation cross-section as the calculated depth to stiff layer exceeded 100 inches for all sections.



Figure 23. Backcalculation Procedure from SS #304

Step 9: Final Check and Sensor Removal: Once the final backcalculation method was selected, the surface modulus plot for each test point was reviewed again. Sensors exhibiting significant deviations from expected behavior were flagged for exclusion. Individual abnormal sensors were removed and the analysis was rerun to produce the final results.

Step 10: Outlier Detection: Chauvenet's criterion was applied to identify and remove statistical outliers from the backcalculated moduli dataset. For each FWD test point, only the specific layer modulus value identified as an outlier was removed from further analysis; the entire FWD test point was not removed unless all layer moduli at that point were flagged as outliers.



Step 11: Evaluation of Depth to Stiff Layer: A second set of backcalculations was run with a maximum depth to rigid layer of 9,999 inches entered in ELMOD to check for the presence of a shallow stiff layer. For all sections on I-59, the calculated depth to stiff layer was greater than 100 inches, indicating that no shallow bedrock or apparent stiff layer was present. Therefore, the subgrade non-linearity approach (without a fixed stiff layer) was used for all final backcalculation results.

Notes on Rounds 2, 3 and 4

For the 1st Lift HMA rounds (Rounds 2 SB and 4 NB Section 1), the HMA modulus was fixed at 200 ksi because the thin layer (typically 2–3 inches) could not be reliably isolated by the backcalculation algorithm. Backcalculation algorithms have difficulty separating a stiff thin layer from the underlying material, often producing unrealistic moduli.

For the rubblized PCC rounds (Rounds 2 NB, 3 NB, and 4 NB Section 2), the rubblized surface produces a highly irregular, non-homogeneous surface that violates the uniform layer assumption required by layered-elastic backcalculation theory. The broken slab pieces are uneven and not fully compacted, causing erratic sensor deflection readings. These rounds exhibited highly variable and generally unreliable moduli.

All modulus statistics in this report are based on log-normal distributions, with the standard deviation expressed as a multiplicative Stdev Factor and the 16th percentile calculated as the geometric mean divided by the Stdev Factor. This approach is consistent with the statistical methodology described in the ELMOD software documentation and the MDOT SS 304 backcalculation procedure.

Backcalculation Results

The backcalculated layer moduli are presented in this section. The pavement structure consists of three layers for Round 1 (Intact PCC / CSB / Support) and four layers for Round 5 (HMA / Rubblized PCC / CSB / Support). For Rounds 2, 3, and 4, the pavement structure depends on the stage of construction at the time of testing. Round 2 includes two conditions: Round 2a (RPCC / CSB / Support, three-layer) for the section tested on the exposed rubblized PCC surface, and Round 2b (1st Lift HMA / RPCC / CSB / Support, four-layer) for the section where the first HMA lift had already been placed. Round 3 is analyzed as a three-layer structure (RPCC / CSB / Support), since the tested section was on the exposed rubblized PCC surface. Round 4 follows the same split as Round 2, with Round 4a representing the 1st Lift HMA condition (four-layer) and Round 4b representing the RPCC-only condition (three-layer).



Project and section averages are reported as geometric means of the individual test point moduli; consequently, a project average is not the arithmetic mean of the section averages.

All Rounds Combined (Project Averages)

Table 10 presents the project-wide average backcalculated moduli for all rounds. This provides a high-level comparison of the structural condition at each stage of the rehabilitation.

Table 10. All Rounds Combined - Project Average Backcalculated Moduli

Round / Condition	PCC/HMA (ksi) ³⁾			RPCC (ksi) ³⁾			CSB (ksi) ³⁾			Support (ksi) ³⁾			Reliability
	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	
R1 Intact PCC	2,959	1.424	2,078	---	---	---	161	1.227	131	27	1.339	21	Reliable
R2a NB RPCC	---	---	---	86	1.969	44	27	1.427	19	6	3.159	2	Unreliable
R2b SB 1st Lift	200 ¹⁾	1.0	200	130	1.655	78	30	2.122	14	20	1.673	12	Unreliable
R3 NB RPCC	---	---	---	60	1.632	37	26	3.653	7	11	2.079	5	Unreliable
R4a NB 1st Lift	200 ¹⁾	1.0	200	76	1.288	59	15	1.215	13	10	1.481	7	Unreliable
R4b NB RPCC	---	---	---	47	1.553	30	30	1.840	16	5	2.435	2	Unreliable
R5 Final HMA	1,048 ²⁾	1.327	790	180	1.733	104	104	1.574	66	28	1.363	20	Reliable

1) HMA modulus fixed at 200 ksi for 1st Lift rounds due to thin-layer limitation in backcalculation

2) HMA modulus normalized to a reference temperature of 77°F

3) Log-Normal Statistics

4) Round 5 RPCC and CSB project averages exclude layer values at stations identified as incompletely rubblized

Round 1: Intact PCC Results by Section

Table 11 and Table 12 present the backcalculated moduli by section for Round 1 on the northbound and southbound lanes, respectively. The intact PCC moduli averaged approximately 2,959 ksi across both directions, with CSB moduli averaging 161 ksi and support moduli averaging 27 ksi.

Table 11. Round 1 - NB Backcalculated Moduli (Intact PCC)

Sec	Limits (ft)	N	Intact PCC Modulus (ksi)			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
1	0 – 6,018	25	3,515	1.28	2,747	---	---	---	160	1.32	122	34	1.34	25
2	7,787 – 10,791	13	2,845	1.47	1,933	---	---	---	168	1.15	145	30	1.45	20
3	11,045 – 17,766	28	2,625	1.33	1,968	---	---	---	155	1.21	128	28	1.29	21
4	18,022 – 19,751	8	2,404	1.26	1,913	---	---	---	150	1.16	130	23	1.36	17
5	21,539 – 24,284	12	3,123	1.33	2,343	---	---	---	170	1.12	152	30	1.12	27
6	24,543 – 29,275	20	2,634	1.39	1,894	---	---	---	171	1.23	139	26	1.16	22
7	29,530 – 31,773	10	4,598	1.34	3,439	---	---	---	205	1.14	179	31	1.16	27
8	32,023 – 32,792	4	6,269	1.34	4,678	---	---	---	194	1.05	184	30	1.15	26
9	35,772 – 37,761	9	2,835	1.27	2,228	---	---	---	161	1.08	149	32	1.17	27
10	38,014 – 38,782	5	3,302	1.34	2,459	---	---	---	175	1.21	145	22	1.15	20
11	39,041 – 41,023	9	1,918	1.56	1,231	---	---	---	176	1.18	149	19	1.35	14
12	41,330 – 42,293	5	3,326	1.11	2,992	---	---	---	159	1.15	137	26	1.12	23
13	42,551 – 43,005	3	2,437	1.18	2,063	---	---	---	145	1.02	142	32	1.05	31

1) Log-Normal Statistics

Table 12. Round 1 - SB Backcalculated Moduli (Intact PCC)

Sec	Limits (ft)	N	Intact PCC Modulus (ksi)			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
1	5 – 5,759	24	2,752	1.36	2,023	---	---	---	157	1.26	125	19	1.27	15
2	7,739 – 8,719	5	3,378	1.44	2,341	---	---	---	163	1.35	121	25	1.47	17
3	8,973 – 11,468	11	2,853	1.19	2,394	---	---	---	135	1.20	112	30	1.30	23
4	11,718 – 13,703	9	2,860	1.08	2,636	---	---	---	154	1.13	136	29	1.09	27
5	14,985 – 17,478	11	3,666	1.46	2,505	---	---	---	171	1.12	152	29	1.16	25
6	17,733 – 19,259	7	2,712	1.25	2,165	---	---	---	159	1.14	140	25	1.14	22
7	19,518 – 22,011	4	2,775	1.13	2,459	---	---	---	170	1.21	141	23	1.47	16
8	22,211 – 25,216	13	3,298	1.25	2,640	---	---	---	168	1.10	153	30	1.22	25
9	25,474 – 29,516	17	3,290	1.32	2,495	---	---	---	165	1.18	140	31	1.15	27
10	29,772 – 31,760	9	3,160	1.39	2,275	---	---	---	163	1.31	125	36	1.33	27
11	31,961 – 32,984	5	3,734	1.21	3,083	---	---	---	166	1.08	154	34	1.22	28
12	35,963 – 40,510	19	2,542	1.49	1,703	---	---	---	144	1.34	107	26	1.31	20
13	40,760 – 43,008	10	2,281	1.82	1,253	---	---	---	134	1.20	111	23	1.70	13

1) Log-Normal Statistics



Round 2: Rubblized PCC and 1st Lift HMA Results by Section

Table 13 and Table 14 present the backcalculated moduli by section for Round 2. The NB Lane (Sections 12 and 13) was tested on rubblized PCC, while the SB lane (Sections 8 to 13) had the first HMA lift. RPCC moduli for the rubblized sections ranged 78 to 106 ksi. On the SB lane, the RPCC moduli ranged from 97 to 162 ksi. As noted earlier, these results should be treated as reference data only.

Table 13. Round 2a - NB Backcalculated Moduli (Rubblized PCC)

Sec	Limits (ft)	N	HMA Modulus (ksi)			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
12	41,806 – 42,257	15	---	---	---	106	1.44	74	22	1.45	15	16	1.42	11
13	42,282 – 42,995	32	---	---	---	78	2.16	36	30	1.34	23	4	3.11	1

1) Log-Normal Statistics

2) N reflects the number of FWD points retained after outlier and abnormal-data screening

Table 14. Round 2b - SB Backcalculated Moduli (1st Lift HMA)

Sec	Limits (ft)	N	HMA Modulus (ksi)			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
8	22,435 – 25,126	12	200*	1.0	200	115	1.40	82	21	1.94	11	18	2.01	9
9	25,372 – 29,361	17	200*	1.0	200	162	1.64	98	38	2.17	17	17	1.57	11
10	29,612 – 31,857	10	200*	1.0	200	137	1.61	85	42	1.98	21	24	1.62	15
11	32,105 – 32,850	4	200*	1.0	200	97	1.46	67	31	2.47	12	29	1.16	25
12	36,088 – 40,575	20	200*	1.0	200	126	1.70	74	29	1.72	17	20	1.64	12
13	40,823 – 42,938	10	200*	1.0	200	115	1.91	60	26	2.92	9	23	1.58	15

1) HMA modulus fixed at 200 ksi for 1st Lift rounds due to thin-layer limitation in backcalculation

2) HMA modulus normalized to a reference temperature of 77°F

3) Log-Normal Statistics

4) N reflects the number of FWD points retained after outlier and abnormal-data screening



Round 3: Rubblized PCC Results by Section

Table 15 presents the backcalculated moduli for Round 3. Only NB Section 1 was tested, on rubblized PCC. The very high CSB Stdev Factor (3.65) and low support modulus (11 ksi) reflect the lack of confinement from a missing HMA overlay. These low values are consistent with testing on an unconfined rubblized surface.

Table 15. Round 3 - NB Backcalculated Moduli (Rubblized PCC)

Sec	Limits (ft)	N	HMA Modulus (ksi)			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
1	0 – 4,475	173	---	---	---	60	1.63	37	26	3.65	7	11	2.08	5

1) Log-Normal Statistics

2) N reflects the number of FWD points retained after outlier and abnormal-data screening

Round 4: 1st Lift HMA and Rubblized PCC Results by Section

Table 16 presents the backcalculated moduli for Rounds 4a and 4b. NB Section 1 received the first HMA lift (HMA modulus fixed at 200 ksi), while NB Section 2 was on rubblized PCC. NB Section 1 showed RPCC of 76 ksi and CSB of 15 ksi. NB Section 2 showed RPCC of 47 ksi and a very low support of 5 ksi, likely reflecting poor confinement and fresh rubblization.

Table 16. Round 4a and 4b - NB Backcalculated Moduli (1st Lift HMA and Rubblized PCC)

Sec	Limits (ft)	N	HMA Modulus (ksi)			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
1	0 – 4,512	183	200*	1.0	200	76	1.29	59	15	1.21	13	10	1.48	7
2	6,080 – 9,701	125	---	---	---	47	1.55	30	30	1.84	16	5	2.44	2

1) HMA modulus fixed at 200 ksi for 1st Lift rounds due to thin-layer limitation in backcalculation

2) HMA modulus normalized to a reference temperature of 77°F

3) Log-Normal Statistics

4) N reflects the number of FWD points retained after outlier and abnormal-data screening

Round 5: Final HMA Results by Section

Table 17 and Table 18 present the backcalculated moduli by section for Round 5 on the northbound and southbound lanes, respectively. The HMA moduli averaged approximately 1,048 ksi (normalized to 77°F), the RPCC moduli averaged 180 ksi, CSB averaged 104 ksi, and support averaged 28 ksi.

The backcalculated rubblized PCC moduli for Round 5 were reviewed to identify locations of incomplete rubblization. At most stations, the rubblized PCC modulus fell within the expected range for a well-rubblized layer (approximately 100 to 300 ksi). At a limited number of stations, the rubblized PCC modulus remained substantially elevated (approximately 800 to 1,000 ksi),



indicating that the original PCC slab was not fully fractured at those locations. At these stations, the rubblized PCC and CSB modulus values were excluded from the Round 5 results so that they would not skew the reported layer moduli, while the HMA and support values at the same points were retained. The affected stations were located in NB Section 7 and at several points toward the northern end of the NB lane.

Table 17. Round 5 - NB Backcalculated Moduli (Final HMA)

Sec	Limits (ft)	N	HMA Modulus (ksi) ¹⁾			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
1	22 – 6,049	25	840	1.17	716	133	1.45	92	85	1.25	68	31	1.44	22
2	7,816 – 10,835	13	1,108	1.43	777	173	1.44	120	103	1.45	71	34	1.33	25
3	11,089 – 17,615	27	1,116	1.24	903	155	1.40	111	102	1.41	73	28	1.23	23
4	17,866 – 19,873	9	1,117	1.21	922	84	1.52	55	73	1.47	50	19	1.27	15
5	21,383 – 24,400	13	1,232	1.23	1,006	316	1.67	189	159	1.53	104	30	1.26	23
6	24,657 – 29,431	20	791	1.27	623	228	1.57	145	144	1.44	100	21	1.43	15
7	29,677 – 31,943	10	968	1.28	759	---	---	---	---	---	---	33	1.22	27
8	32,204 – 32,698	3	1,092	1.28	851	295	1.10	269	165	1.09	152	24	1.08	22
9	35,968 – 37,976	9	685	1.16	591	337	1.49	226	180	1.23	147	32	1.14	28
10	38,231 – 38,984	4	823	1.28	641	242	1.30	186	148	1.20	124	25	1.14	22
11	39,239 – 41,248	9	839	1.08	776	377	1.36	277	186	1.55	119	43	1.19	36
12	41,496 – 42,257	4	966	1.34	719	179	1.90	95	107	1.47	73	25	1.19	21
13	42,503 – 43,001	3	1,081	1.40	772	394	1.35	292	173	1.28	135	40	1.09	37

1) HMA modulus normalized to a reference temperature of 77°F

2) Log-Normal Statistics

3) At NB Section 7, the rubblized PCC and CSB modulus values were excluded due to elevated rubblized PCC moduli indicating incomplete rubblization

Table 18. Round 5 - SB Backcalculated Moduli (Final HMA)

Sec	Limits (ft)	N	HMA Modulus (ksi) ¹⁾			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
1	69 – 5,553	23	1,217	1.22	994	132	1.55	86	92	1.37	68	26	1.33	19
2	7,616 – 8,617	5	1,270	1.20	1,060	86	1.28	67	64	1.36	47	19	1.32	14
3	8,874 – 11,389	11	1,269	1.19	1,062	206	1.68	123	130	1.40	93	30	1.24	24
4	11,643 – 13,899	10	1,168	1.26	924	161	1.65	98	102	1.60	64	25	1.27	20
5	15,158 – 17,421	11	1,344	1.25	1,077	210	1.44	146	92	1.48	62	27	1.30	21
6	17,678 – 19,181	7	1,248	1.43	873	340	1.21	282	137	1.58	87	29	1.34	22
7	19,437 – 21,951	3	1,117	1.13	992	80	1.34	60	31	1.44	21	16	1.26	12
8	22,201 – 25,222	13	1,290	1.28	1,009	151	1.52	99	80	1.47	54	25	1.21	21

Sec	Limits (ft)	N	HMA Modulus (ksi) ¹⁾			RPCC Modulus (ksi)			CSB Modulus (ksi)			Support Modulus (ksi)		
			Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th	Ave	SF	16 th
9	25,475 – 29,494	17	856	1.16	737	186	1.94	96	94	1.66	57	25	1.33	19
10	29,746 – 31,757	9	1,038	1.25	833	278	1.31	213	134	1.56	86	39	1.33	29
11	32,006 – 33,015	5	1,281	1.21	1,059	210	1.54	136	72	1.29	56	24	1.26	19
12	36,030 – 40,559	19	1,132	1.37	825	145	1.56	93	84	1.43	59	26	1.16	23
13	40,807 – 42,944	10	1,083	1.28	849	247	1.82	136	98	1.44	68	30	1.28	24

1) HMA modulus normalized to a reference temperature of 77°F

2) Log-Normal Statistics

Key Findings and Discussion

- Deflection Trends:** Round 1 (intact PCC) produced an average normalized deflection of 3.60 mils across the project. Round 5 (final HMA) averaged 5.26 mils, representing a 46% increase. This increase is expected because the rubblized PCC, while providing structural support, is inherently more flexible than intact PCC slabs. Rounds 2 through 4 exhibited deflections 5 to 30 times higher than Round 1, which is consistent with testing on unconfined rubblized material and thin first-lift HMA.
- PCC and HMA Layer Modulus:** The intact PCC modulus in Round 1 averaged 2,959 ksi. Following rubblization and placement of the overlay, the Round 5 final HMA modulus averaged 1,048 ksi (normalized to 77°F), which is within the expected range for a well-performing HMA overlay. Figure 24 presents the project average PCC and HMA moduli across all rounds, showing the progression from the intact PCC condition through the intermediate construction stages to the final HMA overlay.

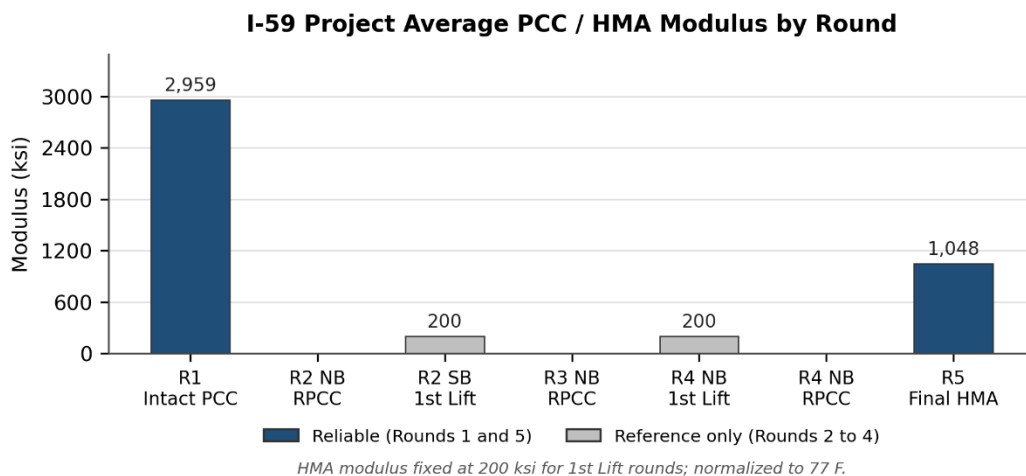


Figure 24. Project Average PCC / HMA Modulus by Round

- RPCC Modulus:** As shown in Figure 25, the RPCC modulus in Round 5 averaged 180 ksi, which represents a significant improvement over the intermediate rounds (47 to 130 ksi). The higher Round 5 modulus reflects the confinement provided by the full-depth HMA overlay and the time-dependent stiffening of the rubblized material under traffic loading. The lower RPCC moduli in Rounds 2 through 4 are attributed to a combination of factors: the rubblized layer was freshly fractured and, at the time of testing, had not yet been fully confined or compacted, and the irregular rubblized surface reduces the reliability of the backcalculation, as described in the Notes on Rounds 2, 3 and 4. The intermediate-round RPCC values should therefore be regarded as reference data only.

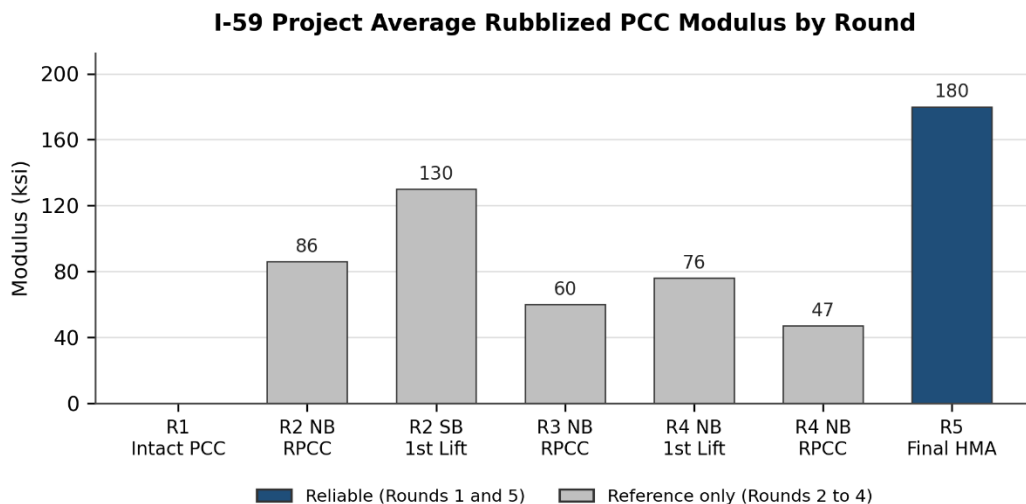


Figure 25. Project Average Rubblized PCC Modulus by Round

- CSB Modulus:** The CSB modulus decreased from an average of 161 ksi in Round 1 to 104 ksi in Round 5, as shown in Figure 26. This reduction may be attributed to the rubblization process, which could have introduced micro-cracking or disturbance in the underlying stabilized base layer. The substantially lower CSB moduli in Rounds 2 through 4 (15 to 30 ksi) are consistent with the intermediate construction conditions, when the sections were tested with no overlay or only a thin HMA lift, and, as described in the Notes on Rounds 2, 3 and 4, should be regarded as reference data only.

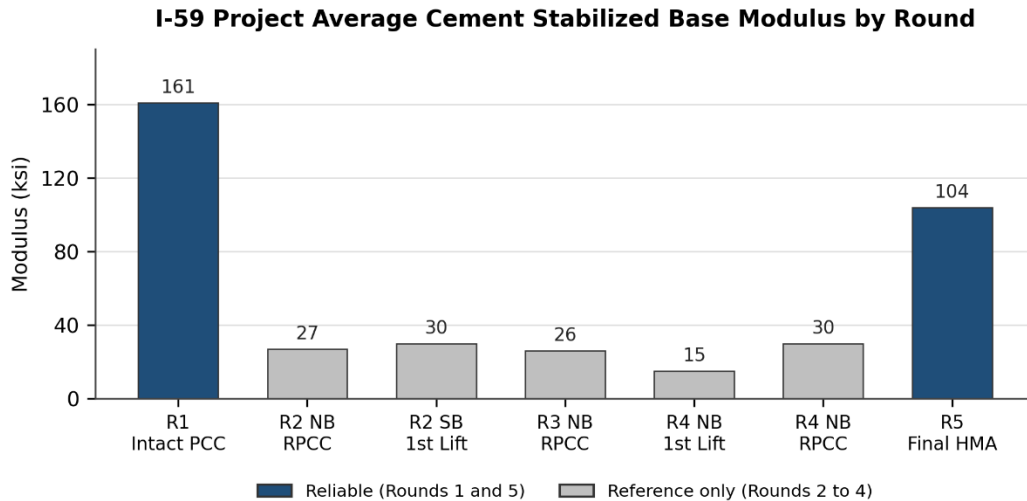


Figure 26. Project Average Cement Stabilized Base Modulus by Round

- Support Modulus:** The support modulus remained consistent between Round 1 (27 ksi) and Round 5 (28 ksi), as shown in Figure 27, indicating that the rubblization and overlay process did not adversely affect the underlying support. The intermediate rounds show lower apparent support moduli, consistent with the limitations of testing on unconfined or thinly surfaced sections.

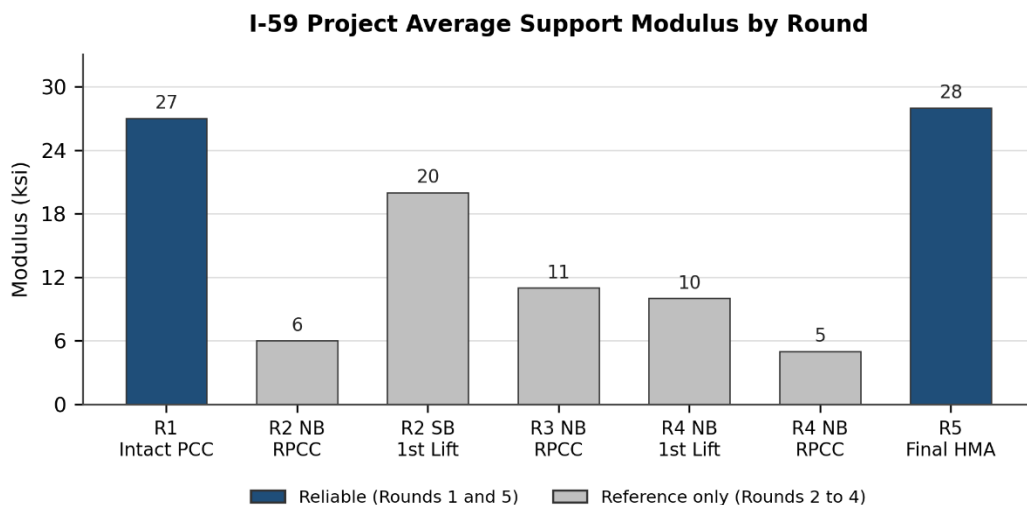


Figure 27. Project Average Support Modulus by Round

- Reliability of Intermediate Rounds:** Rounds 2 through 4 were conducted under conditions that violate the fundamental assumptions of layered-elastic backcalculation theory. The highly irregular rubblized surface, thin HMA lift geometry, lack of confinement, and freshly placed HMA all contribute to unreliable backcalculation



results. These rounds are included for completeness and as reference data, but the moduli from these rounds should not be used for structural design or pavement management decisions.