DEPARTMENT OF TRANSPORTATION

Performance of Concrete Overlays over Full Depth Reclamation

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Intertek

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16. Abstract (Limit: 250 words) Concrete overlays of full depth reclamation (FDR) are a pavement rehabilitation option involving reclamation of existing pavement layers while providing a new concrete surface to the roadway. This report reviewed nine concrete overlays of FDR that have been constructed in three states since 2006. Design and construction details were collected for each project and performance was analyzed through automated pavement condition data. Additional distress surveys and testing, including falling weight deflectometer (FWD) testing, were performed at one project in Freeborn County, MN, to further characterize the pavement condition and structural properties. Overall, the projects reviewed in this study were performing well to date, and concrete overlays of FDR appear promising as cost-effective, sustainable tools for helping agencies maintain their pavement network.		
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EXECUTIVE SUMMARY

Agencies are confronting greater needs for cost-effective and sustainable methods to maintain and rehabilitate their pavement network. To meet these needs, concrete overlays of full depth reclamation (FDR) are a promising solution that makes use of existing pavement materials while providing a new, long-lasting concrete surface to the roadway. Although only a handful of projects have been built in the US to date, the concept is promising and can offer a number of unique use cases as well as advantages over other types of rehabilitation solutions.

Nine concrete overlays of FDR constructed in three states since 2006 were identified as case studies for analysis. These projects were built on a wide range of roadway facilities, from rural county highways to busy divided highways in urban areas to one section of rural interstate highway with heavy truck traffic.

Design and construction details were collected for each project. The performance of each project was evaluated using automated pavement condition data, consisting of pavement smoothness measured in terms of International Roughness Index (IRI) and average joint faulting. At one project, CSAH 46 in Freeborn County, MN, falling weight deflectometer (FWD) testing was also performed to characterize joint load transfer performance and backcalculate layer structural properties, alongside a visual survey, curling measurements, and analysis with an ultrasonic tomography device.

Some of the key findings of the analysis of these nine projects are as follows:

- The designs of each project were similar to each other and resembled those of conventional unbonded concrete overlays, consisting of relatively thick (7.5+ inch), doweled concrete overlays placed over the FDR layers.
- The FDR layers were between 6 and 12 inches thick and were unstabilized for most projects. Two projects added cement as a stabilizing agent at a dosage rate of 6% by weight.
- FWD backcalculation of the CSAH 46 project with an unstabilized FDR layer determined that the FDR had a substantially higher backcalculated elastic modulus than typical crushed stone subbase layers for concrete pavements. FWD testing was not able to be performed over a stabilized FDR layer.
- These projects were still relatively early in their service life, but demonstrated good smoothness performance overall. Most projects fell within the FHWA threshold for good performance at the time of the last data collection, with measured IRI less than 95 in/mi. IRI trends were mostly stable, with most projects showing a slight increase over time.
- Three projects in Colorado experienced a high degree of curling, which appeared to cause volatility in IRI measurements and high early age IRI readings at two projects. It was not clear whether the curling behavior was related to the design of an overlay placed on FDR.
- Average transverse joint faulting values were very low for all projects that were tested for faulting, less than 0.028 inches, and joint load transfer performance of the CSAH 46 project was excellent after 14 years of service. These findings make sense given that all projects were doweled.

While only a limited number of these types of overlays have been constructed to date, the projects in this study are performing well for the most part. As agencies consider further use of concrete overlays of FDR, it may also be worthwhile for them to explore the use of alternative design options such as greater use of stabilization for the FDR layers and thinner concrete overlay designs that make use of shorter joint spacings and fiber-reinforced concrete. Overall, concrete overlays of FDR appear to be a promising method to help agencies meet their pavement rehabilitation needs.

CHAPTER 1: BACKGROUND

1.1 PROJECT OVERVIEW

As pavements age, they become more difficult for public agencies and owners to maintain. As resources become scarcer and more limited, using materials already available at a site can make for a more economically and environmentally friendly rehabilitation choice. Full depth reclamation (FDR) is a pavement rehabilitation method that makes use of existing pavement surface layers, base/subbase layers, and sometimes the subgrade to produce a subbase layer for a new pavement surface. The use of FDR in pavement construction frequently meets an agency's economic and environmental criteria.

Concrete overlays are another rehabilitation method that can meet present-day needs of agencies and the traveling public. A concrete overlay is the construction of a new concrete surface over an existing pavement surface layer. These overlays can be adapted to a variety of existing pavement conditions and design situations, providing a new, long-lasting concrete pavement surface. Concrete overlays also serve as a sustainable and cost-effective solution. They require little maintenance, resulting in lower life-cycle costs and environmental impacts.

When you combine the benefits of FDR with concrete overlays, there is promise of more durable infrastructure with lower environmental impacts. This rehabilitation option has not been widely adopted in the U.S., therefore there was interest in understanding the performance of existing sections as a way to increase their adoption and improve their design. The objective of this study was to create a synthesis of concrete overlay on FDR performance based on case studies of existing sections across the U.S.

This synthesis report begins with an overview of the full depth reclamation process and concrete overlay options. This is followed by descriptions of the testing and data gathering necessary to create case studies of performance for nine projects in three states. The report concludes with observations and conclusions related to the general behavior and performance of concrete overlays on FDR examined in this study.

1.2 LITERATURE SEARCH

1.2.1 Pavement structures

Pavements generally have the structure shown in Figure 1.1.

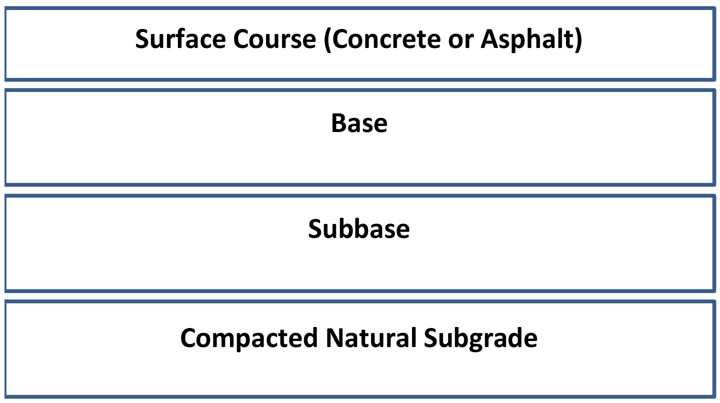

Figure 1.1 Typical pavement structure

The subgrade can be the natural soil or borrow material or embankment. The top surface of the soil is compacted to support and provide a platform for the pavement structure. The upper layer of this natural soil may be compacted or stabilized to improve its properties, including strength, stiffness, and stability. The subgrade serves as the foundation of the pavement structure (*1*).

Base and subbase layers are placed above the subgrade and underneath the pavement surface course. These layers act as a separation layer between the subgrade and the pavement surface, increasing the structural capacity and improving the foundation stiffness. Base or subbase layers may consist of crushed stone, gravel, recycled asphalt or concrete, or asphalt base courses, among other options. These materials may also be treated with stabilizing admixtures such as Portland cement, lime, or fly ash to increase their strength and stiffness. If there are distinct base and subbase layers, the base course is placed on top of the subbase and will generally consist of higher quality materials. Other advantages of base and subbase layers may include (*2*):

- Preventing erosion and migration of fine-grained subgrade soils
- Providing a working platform during construction
- Providing drainage
- Minimizing the effects of frost action by insulating from frost-susceptible soils and raising the height of the pavement surface above the groundwater table
- Allowing a reduction in thickness of the pavement surface course

The surface course consists of an asphalt or concrete layer. This layer carries and transfers loads to the layers below. Concrete is rigid and behaves as a plate and does not localize stress. This layer also accommodates traffic loads, provides skid resistance, prevents abrasion, and resists climate conditions.

Concrete pavements are typically constructed on a single subbase layer placed over the subgrade. The subbase layer serves to increase the effective stiffness of the foundation and provide uniform support over the cross section. The subbase may also be stabilized to improve its ability to perform these functions. Unstable, non-uniform foundation layers can result in early age distresses.

As pavements age, the serviceability declines, leaving agencies with several options for rehabilitation. These options include removal and replacement, structural overlay with a new pavement surface, or (for existing asphalt pavements) full depth reclamation and placement of a new surface course. A comparison of the advantages and disadvantages of these strategies is shown in Table 1.1.

Table 1.1 Comparison of different strategies for rehabilitation

	Reclamation	Structural Overlay	Removal and Replacement
Fast Construction	Yes	Yes	No
Traffic disruption lower	Yes	Yes	No
Minimal material hauling	Yes	Yes	No
Conserve resource	Yes	Yes	No
Maintain existing elevation	Yes	No	Yes
Low cost	Yes	Yes	No

The benefits of FDR are similar to those of a structural overlay, but sometimes the pavement structure degrades to the point where a typical structural overlay cannot be applied. Additionally, maintaining the existing elevation of a roadway can sometimes be important, offering further advantages to use of an FDR layer. If the existing elevation is not maintained in urban areas, the agencies may have to adjust curbs, driveways, intersections, and utility structures such as intakes and manholes. In rural areas, if the elevation is not maintained, fill material may be needed to preserve side slopes. FDR offers the ability to remove some material prior to reclamation to avoid raising the grade too much.

1.2.2 Full depth reclamation

Full depth reclamation involves the rehabilitation of an existing pavement by recycling portions of the surface layer and sometimes its base, subbase, and subgrade layers into a new subbase layer. A reclamation machine pulverizes the existing pavement surface and underlying layers. Blending, stabilizing, and compacting the recovered material creates a higher quality, uniform subbase.

The FDR construction process is shown in Figure 1.2. Depending on the desired geometry and level of the existing pavement surface, part of the existing surface course may be removed prior to reclamation to lower the pavement profile. Additionally, extra pulverized material can be used to build shoulders, turn lanes, etc. A stabilizing agent may also be added to the FDR material to increase stiffness.

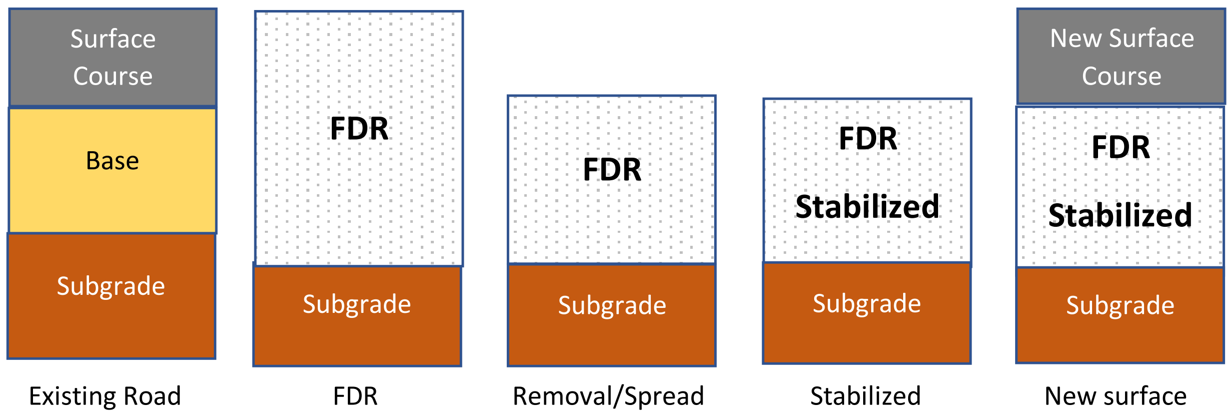

Figure 1.2 FDR construction process for asphalt pavement sections

The depth of an FDR treatment can vary depending on the depth of the existing pavement structure, but generally range between 4 to 12 inches and can be as thick as 24 inches. At greater thicknesses, however, FDR layers may need to be compacted in multiple lifts. If necessary, additional granular material or soil can be blended into the FDR layer. A schematic of the milling drum of a reclaimer pulverizing the existing pavement surface into the underlying base material is shown in Figure 1.3.

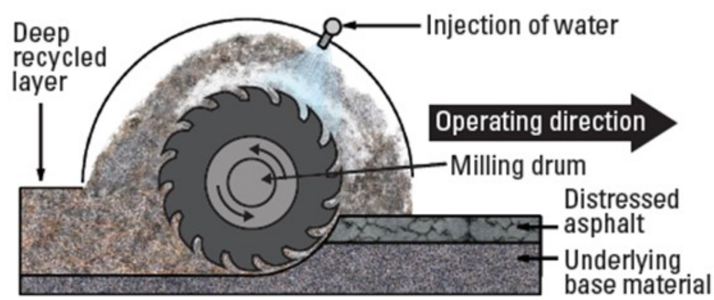

Figure 1.3 Schematic of a reclaimer (*3*)

As previously mentioned, stabilizing agents may also be added to the FDR layer. These materials, such as cement, fly ash, lime, or emulsified asphalt, can be added by the reclaimer or spread and mixed with the pulverized material. The stabilization process is illustrated in Figure 1.4, showing an example of adding foamed asphalt as the stabilizing agent during the reclamation process.

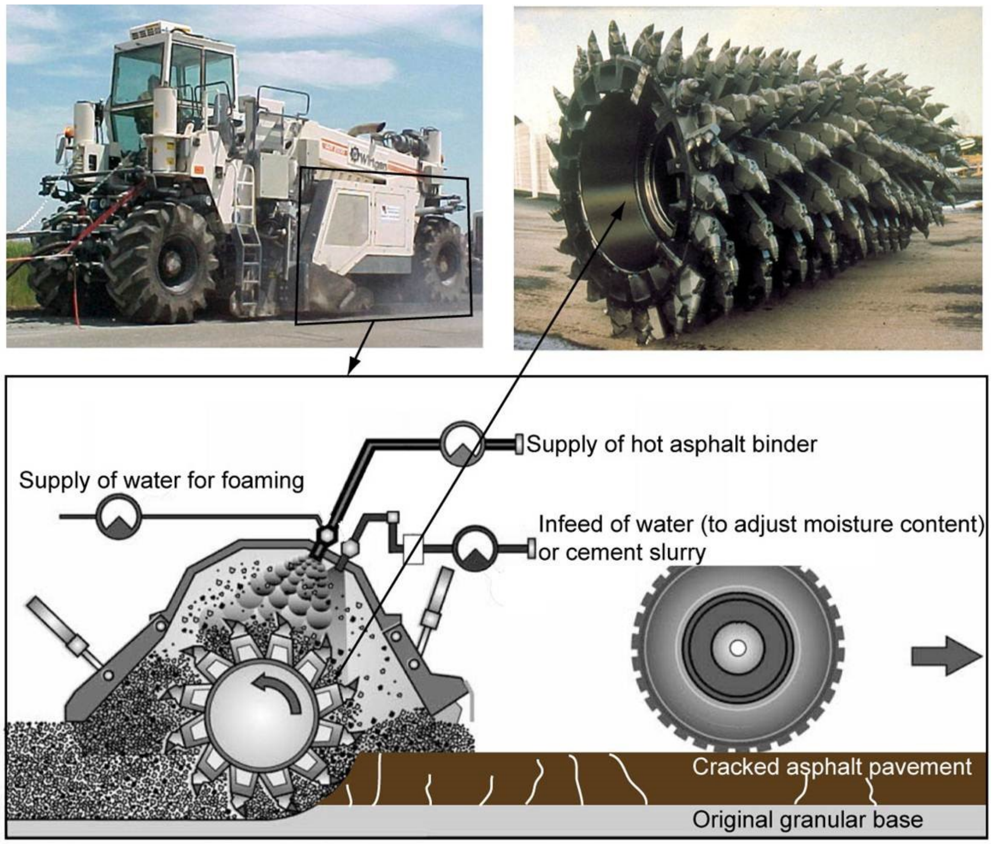

Figure 1.4 Reclamation with a stabilizing agent (*4*)

Figure 1.5 shows two examples of the pulverized base left behind the reclaimer. After pulverizing the FDR layer and (if specified) adding the stabilizing agent, the FDR layer is compacted as close as possible to the maximum dry density. An example of compaction of FDR is shown in Figure 1.6.

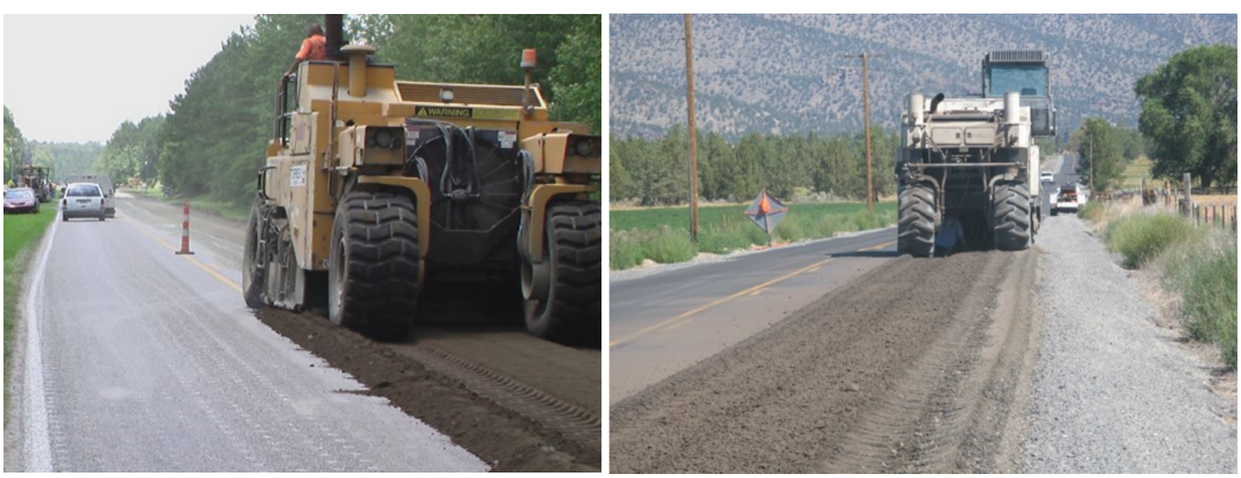

Figure 1.5 Reclaimed existing pavement (*4*)

Figure 1.6 Final compaction (*4*)

If the FDR layer is too wet or too dry, the maximum dry density may not be achieved. Before starting an FDR project, tests should be done to determine the optimum moisture content. Water is added during or after pulverization to reach the optimum moisture content to allow for compaction as close to the maximum dry density as possible. As density increases, the strength of FDR layers also increases, as demonstrated in Figure 1.7.

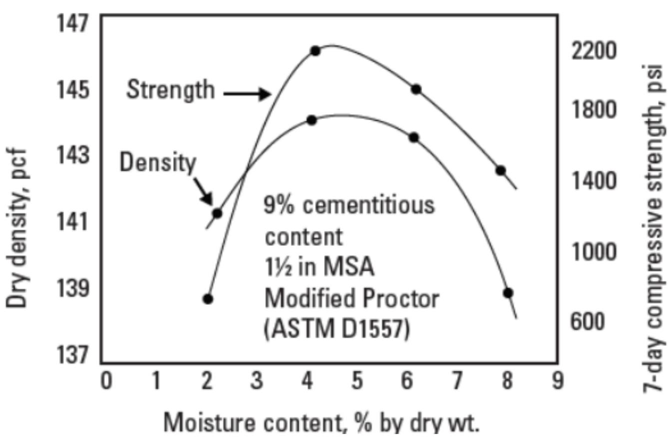

Figure 1.7 Relationship between dry density, moisture content, and compressive strength (*4*)

1.2.3 Types of FDR stabilization

Generally, stabilization of FDR layers refers to stabilization with an admixture. However, the addition of new aggregate material to an FDR layer can provide a degree of mechanical stabilization. This additional aggregate provides particle interlock to the pulverized mixture of existing pavement and subsurface layers.

Chemical stabilization methods achieve bond between particles within the FDR layer through the use of materials including Portland cement, fly ash, lime, lime kiln dust, calcium chloride, magnesium chloride, emulsified asphalt, foamed asphalt, or a proprietary material. A geosynthetic layer can also be placed between an FDR layer and the new pavement surface to provide lateral strength and increase stability.

The benefits of stabilizing an FDR layer include an increased structural capacity of the pavement, increased durability of the FDR layer, and a shorter construction schedule, which can allow earlier opening to traffic and reduce impacts on the community during construction. Figure 1.8 (*5*) contains guidance for the selection of a stabilizing agent that is most appropriate for different types of granular materials and soils. Climate and weather conditions may also play a role in choosing the most appropriate stabilization agent. Considerations for climate and weather effects are listed in Table 1.2 (*6*).

Stabilized full depth reclamation is regarded as a cost-effective method for pavement rehabilitation. Recommendations for mix design of stabilized FDR have been proposed based on field experience and experimental investigation. However, there is still much work to be done to understand how stabilization of FDR affects the long-term properties of the pavement, which is important for developing a method to determine desirable properties for a given application of a stabilized FDR layer (*7*).

Reclaimed Material Type	Well-Graded Gravel	Poorly Graded Gravel	Silty Gravel	Clayey Gravel	Well-Graded Sand	Poorly Graded Sand	Silty Sand	Clayey Sand
USCS[(1)] Classification	GW	GP	GM	GC	SW	SP	SM	SC
AASHTO[(2)] Classification	A-1-a	A-1-a	A-1-b	A-1-b A-2-6	A-1-b	A-3 A-1-b	A-2-4 A-2-5	A-2-6 A-2-7
Asphalt Emulsion SE[(3)]>30 or PI[(4)]<6 P200[(5)]<20%	Highly Recommended	Highly Recommended	Recommended	Recommended	Recommended	Recommended	Not recommended	Not recommended
Foamed Asphalt PI<10 5%<P200<20%	Not recommended	Not recommended	Highly Recommended	Highly Recommended	Not recommended	Not recommended	Recommended	Not recommended
Cement, CKD, and Fly-ash PI<20	Recommended	Recommended	Highly Recommended	Highly Recommended	Recommended	Recommended	Highly Recommended	Highly Recommended
Lime, LKD PI>20 P200<25%	Not recommended	Not recommended	Not recommended	Recommended	Not recommended	Not recommended	Not recommended	Highly Recommended

Not recommended   Recommended   Highly Recommended

(1) Unified Soil Classification System, ASTM D 2487
(2) American AASHTO Association State Highway Transportation Officials, AASHTO M 145
(3) Sand Equivalent (AASHTO T 176 or ASTM D 2419)
(4) Plasticity Index (AASHTO T 90 or ASTM D 4318)
(5) Percent passing No. 200 (0.075 mm) sieve

Figure 1.8 Stabilization additive recommendations for different types of materials (*5*)

Table 1.2 Climate and weather considerations for stabilization (*6*)

Type of Additive	Climatic Limitation for Construction
Lime, Fly Ash or Lime-Fly Ash	Do not perform work when reclaimed material could be frozen. Air temperature in the shade should be no less than 4ºC (39ºF) and rising. Complete stabilization at least one month before the first hard freeze. Two weeks minimum of warm to hot weather is desirable after completing the stabilization work.
Cement or Cement Fly-Ash	Do not perform work when reclaimed material could be frozen. Air temperature in shade should be no less than 4ºC (39ºF) and rising. Complete stabilization should be at least one month before the first hard freeze.
Asphalt Emulsion	Do not perform work when reclaimed material could be frozen. Air temperature in the shade should be no less than 15ºC (59ºF) and rising. Asphalt emulsion stabilization should not be performed if foggy or when other high humidity conditions (humidity >80%). Warm to hot dry weather is preferred for all types of asphalt stabilization involving cold mixtures because of improved binder dispersion and curing.
Calcium Chloride	Do not perform work when reclaimed material could be frozen. Air temperature in shade should be no less than 4ºC (39ºF) and rising. Complete stabilization should be at least one month before the first hard freeze.

1.2.4 Comparison of FDR with other soil cement applications

There are a number of other types of cement-stabilized soil and subbase layers that can be used in pavement structures. These layers include cement-modified soils, cement-stabilized subgrades, and cement-treated base (CTB) layers (*8,9*). Figure 1.9 contains a comparison of design details, properties, and benefits of these alternative soil cement/subbase applications and stabilized FDR layers.

Soil-Cement Type	Cement-Modified Soil (CMS)	Cement-Stabilized Subgrade (CSS)	Cement-Treated Base (CTB)	Full-Depth Reclamation (FDR)
Purpose	• Promotes soil drying • Provides a significant improvement to the working platform • Provides a permanent soil modification (does not leach)	• Provides all the benefits of CMS plus the following: - Potentially allows for a reduction in pavement thickness or increased pavement life - Increases the bearing capacity for building slabs, footings, and other structural elements	• Provides a strong, frost-resistant base layer for asphalt or concrete pavements	• Provides a strong, frost-resistant base layer for asphalt or concrete pavements
Materials	• Primarily fine-grained soils • 2%–4% cement	• Primarily fine-grained soils • 3%–6% cement	• Primarily coarse-grained manufactured materials • 3%–6% cement	• Pulverized asphalt blended with existing pavement base, subbase, and/or subgrade • 3%–6% cement
Material Properties	• Reduced moisture susceptibility	• 100–300 psi (0.7–2.1 MPa) seven-day compressive strength	• 300–600 psi (2.1–4.1 MPa) seven-day compressive strength	• 300–600 psi (2.1–4.1 MPa) seven-day compressive strength
Construction Practices	• Minimum 95% of maximum density • Mixed in place	• Minimum 95% of maximum density • Mixed in place	• Minimum 95%–98% of maximum density • Mixed in place or at a plant	• Minimum 95%–98% of maximum density • Typically mixed in place

Figure 1.9 Comparison between different soil cement/subbase applications and stabilized FDR (*9*)

1.2.5 Quality control of FDR

1.2.5.1 Sieve analysis

Different sieve analysis methods may be required for acceptance of FDR layers, such as ASTM D422 (soil particle size) or ASTM C136 (fine and coarse aggregates). Gradations for FDR with cement stabilization recommended by the Portland Cement Association (PCA) are shown in Table 1.3.

Table 1.3 PCA recommended gradation for cement-stabilized FDR (*10*)

Sieve	Minimum % Passing
3-inch	100
2-inch	95
#4	55

1.2.5.2 Moisture and density

As mentioned in section 1.2.2, regular testing should be performed to determine the optimum moisture content so that maximum dry density can be achieved during compaction. An example of an optimum moisture/maximum dry density curve is shown in Figure 1.10.

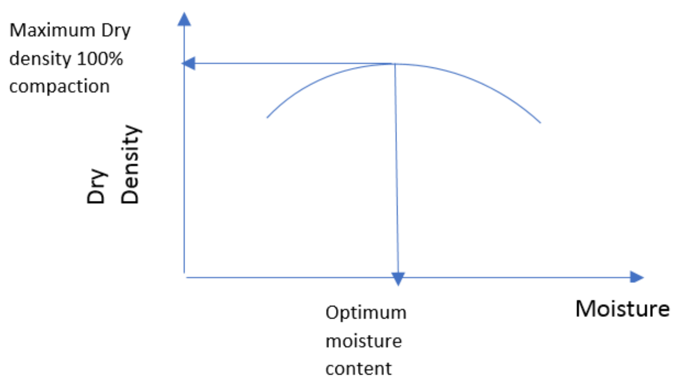

Figure 1.10 Optimum moisture/maximum dry density curve

1.2.5.3 Compressive strength (stabilized FDR)

The total structural capacity of an FDR base layer increases with thickness and strength. When a stabilized FDR layer is used, a 7-day compressive strength ranging from 300 psi to 600 psi is usually specified. However, it should be noted that with cement-stabilized FDR, layers with increased strength may be more susceptible to shrinkage and cracking.

1.2.6 Spring thaw effects on FDR

After an FDR layer is constructed, the subbase under the pavement becomes more homogenous. This can reduce the amount of faulting and cracking in a concrete overlay layer caused by differential settlement. FDR performed with a stabilizing agent can further reduce variability in the subbase material and reduce its susceptibility to spring thaw effects (*11*).

Little is known about the effects of spring thaw on stabilized FDR layers. Previous research has indicated that FDR with cement stabilization can positively benefit pavements that previously experienced frost heave during winter or loss of shear strength during spring thawing events (*4,11*). Moisture can infiltrate into unstabilized FDR layers more easily and cause softening of the subbase material that reduces strength and stiffness (*11*).

1.2.7 Concrete Overlays

A concrete overlay is a pavement rehabilitation technique consisting of the construction of a new concrete surface directly over an existing concrete, asphalt, or composite pavement (*12*). Concrete overlays are categorized based on the existing pavement surface and whether they are designed to bond to the underlying surface. Figure 1.11 provides an overview of the different types of concrete overlays.

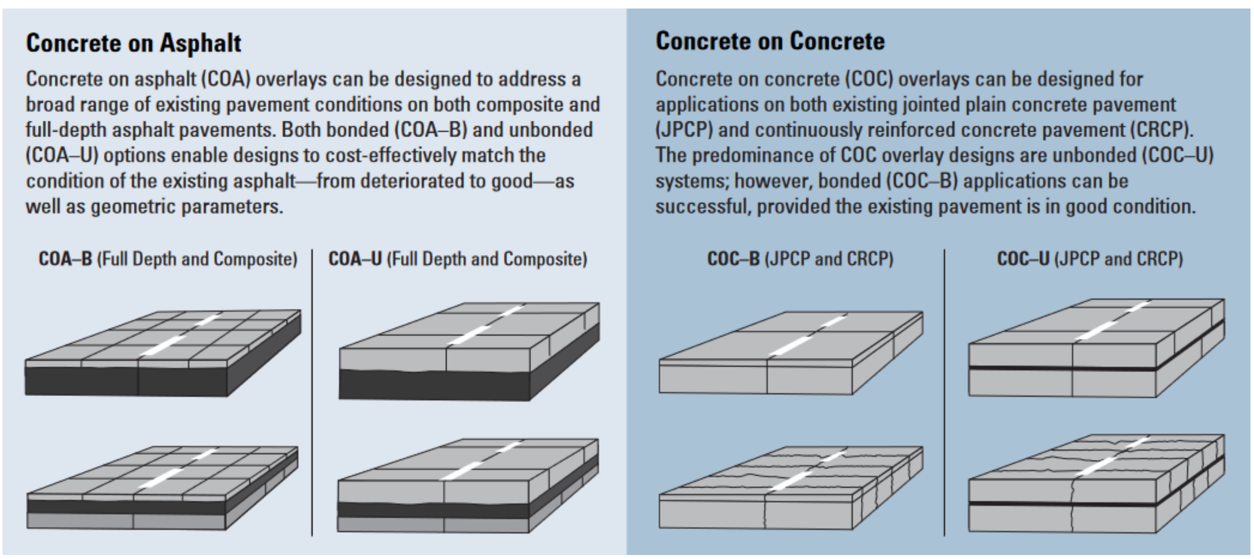

Figure 1.11 Overview of concrete overlays (*12*)

With the large assortment of design options available for concrete overlays, they can be adapted to a variety of existing pavement conditions and classes of roadways, allowing concrete overlays to fulfill many different types of performance and service life goals for owners and agencies. Design details (including bonding) vary depending on the existing pavement condition, traffic loading, geometric constraints, and desired design life.

The amount of surface preparation required prior to building a concrete overlay depends on the type of concrete overlay and the condition of the existing pavement. The existing surface layer must still be in relatively good condition to be able to support a bonded concrete overlay. Milling may be used to remove distresses within the first few inches of the surface, as well for profiling and/or grade corrections to accommodate the geometric constraints of the roadway. More significant distresses in the existing pavement surface may be addressed through techniques such as full- or partial-depth patching prior to overlay (*12*).

1.2.8 Concrete overlays of FDR

The significant majority of pavements which receive an FDR treatment are surfaced with a new layer of asphalt. However, a number of concrete overlays of FDR have been constructed around the US in recent years, where a new concrete surface is placed directly over an FDR layer after reclamation of an existing asphalt pavement. This combination of the concept of a concrete overlay with full depth reclamation offers the ability to meet agency needs for sustainable, long-lasting pavement rehabilitation methods.

While reclaiming existing pavement layers represents additional effort compared to placing a concrete overlay directly over the surface, the combination of these techniques may offer several advantages compared to a concrete overlay or a conventional FDR with an asphalt surface. First, existing asphalt pavements may deteriorate to a point where they are not suitable for a concrete overlay. For example, a concrete overlay may not be possible in cases of structural instability or material degradation such as stripping within asphalt layers, or the overlay could be very susceptible to reflective cracking when placed over an existing asphalt pavement with wide, dominant cracks. In these types of cases, reclaiming and remixing the pavement offers the opportunity to still make use of the existing pavement materials underneath a new concrete overlay surface.

Concrete overlays of FDR also offer the ability to build a concrete overlay without raising the surface profile. This can sometimes be accomplished simply by milling an existing asphalt surface down to the depth of the concrete overlay. However, this can only be done when there would be sufficient thickness of remaining asphalt (minimum three inches) in sound condition to support the concrete overlay. If that were not the case, the FDR process could allow for removal of the surface material while the remaining asphalt is mixed with subbase and subgrade layers and (if desired) stabilized with an additive, allowing for placement of a concrete overlay within the existing height of the pavement. Since grade constraints are a common obstacle to construction of concrete overlays, concrete overlays of FDR could be used in situations where an overlay could not normally be built.

Another potential use for concrete overlays of FDR would be when considering a new concrete overlay to replace an existing concrete on asphalt (COA) overlay. While it is possible to build a concrete on concrete (COC) overlay over an existing COA overlay, grade constraints can also limit the use of this type of overlay. The existing COA overlay could be removed and replaced with a new COA overlay, but the underlying asphalt pavement may have deteriorated over time to the point where a new concrete overlay would no longer be appropriate. In this case, the existing asphalt pavement could be reclaimed to allow for the placement of a new concrete overlay.

Concrete overlays of FDR may also be able to fulfill longer-term service life expectations than conventional FDR approaches. While specific design guidance does not yet exist for concrete overlays of FDR, the FDR layer can be treated like a subbase layer in the design process. From there, conventional concrete pavement design methods, construction materials, and placement techniques may be used to build a new concrete pavement surface capable of fulfilling a long-term design life.

The use of a stabilized FDR layer may also offer the possibility to design concrete overlays of FDR similarly to a thin concrete on asphalt–bonded (COA–B) overlay. In these cases, the stiffness of the reclaimed layer and design elements such as shorter joint spacing or fiber-reinforced concrete could be used to make a thinner concrete overlay design feasible over FDR.

As previously mentioned, the cross section of a concrete overlay of FDR may resemble a conventional full-depth concrete pavement placed over a subbase layer. That said, since the combination of these methods still provides an alternative to complete removal and replacement by making use of the existing pavement materials while placing a new concrete surface on top, this type of pavement will be considered a distinct type of concrete overlay for the purposes of this study.

CHAPTER 2: EVALUATION OF CONCRETE OVERLAYS OVER FULL DEPTH RECLAMATION

2.1 INTRODUCTION

Concrete overlays of FDR are not a common pavement type in the US, with perhaps only a few dozen projects constructed in a handful of states in recent years. That said, these existing sections provide an opportunity to evaluate their design and performance to determine elements of successful projects, their suitability for different roadway applications, and the value that agencies may be able to realize from wider adoption of this technique.

In this study, nine projects were identified in three states as case studies for which design, construction and performance data were able to be collected for analysis. These projects are listed in Table 2.1, and include concrete overlays of FDR on roadways including rural county and state highways, busy urban thoroughfares, and one interstate highway.

Note that all of these projects are not necessarily classified or specified as concrete overlays by the agencies that construct them. For example, certain agencies used terminology such as "full-depth concrete reconstruction" and "concrete surfacing" in the plans and referred to specifications for standard concrete pavements rather than concrete overlays. That said, each of these projects share a common structure of a concrete pavement surface placed directly over a full depth reclamation layer, so they fit into the working definition of a concrete overlay of FDR established in this study.

Table 2.1 Concrete overlay of FDR projects included in evaluation

Route	Location	Construction Year	Concrete Thickness (in)	FDR Type	FDR Layer Thickness (in)
CSAH 46	Freeborn County, MN	2006	7.5	Unstabilized	8
US 75	Brown County, KS	2012	8	Stabilized with cement	7
K-68	Franklin County, KS	2014	9	Unstabilized	6
US 81	Republic County, KS	2017	9.5	Unstabilized	6
I-70	Gove County, KS	2019	12	Stabilized with cement	12
US 385	Yuma County, CO	2010	7.5	Unstabilized	8-12
US 34	Weld County, CO	2012	9	Unstabilized	8
US 24	El Paso County, CO	2017	8.25	Unstabilized	12
SH 45	Pueblo, CO	2017	8.25	Unstabilized	8

2.2 METHODOLOGY

2.2.1 Project design and construction details

To begin the evaluation of each concrete overlay of FDR project identified in Table 2.1, project information was compiled from state DOT sources to document relevant aspects of the pavement design and construction process. These details include the year of construction, project location and length, average annual daily traffic (AADT) and average annual daily truck traffic (AADTT), concrete overlay thickness, joint spacing, whether dowel bars were included, and finally, the type (stabilized vs. unstabilized) and thickness of the FDR layer.

Relevant details from the FDR construction process and specifications are included for each project. Availability of plan sets varied between projects, but relevant information from the existing pavement section and typical cross sections for the FDR and concrete overlay are provided when possible.

2.2.2 Pavement performance data

The primary method for evaluating the performance of the concrete overlays of FDR was analysis of automated pavement condition data. For all but the Freeborn County, MN project, these data were the only performance information available for analysis.

The International Roughness Index (IRI), a property of the pavement surface profile and the most-widely used measure of pavement smoothness (*13*), was collected for all projects. In addition, the average faulting measured across each transverse joint was measured for the pavements in Minnesota and Kansas. Both types of measurements were taken by pavement management vans. Faulting information was not available for the Colorado projects, so only IRI values were analyzed.

These condition data were collected at intervals ranging from every year to every five years depending on the project. The most recent data collection year for all projects in the study was 2020. IRI and average faulting were analyzed as a function of pavement age. None of the projects were more than 14 years old at the time of the last data collection, and many projects were less than 5 years old, so the ability to project long-term performance was limited, but the data were still useful for characterizing their early in-service performance.

2.2.3 Visual survey

At the Freeborn County, MN project, a visit was made to perform a visual survey to document typical field conditions of the CSAH 46 project by taking photos and noting distresses such as transverse and longitudinal cracking. Slab curling measurements were also performed using a portable device developed by Ceylan et al. (*14*). The device consists of a string connected by hollow steel columns that are placed at the edges or corners of the slab, allowing for measurements of the displacement between the string and the surface of the pavement. The curling behavior can be characterized by looking at the difference between the displacement at the interior of the slab compared to the edges of the slab. This device is pictured in Figure 2.1.

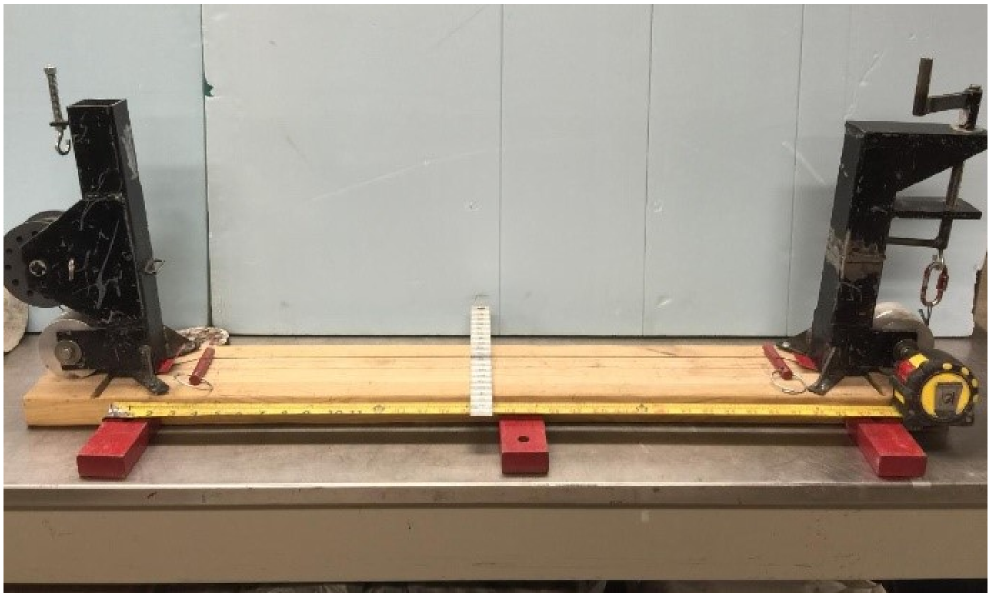

Figure 2.1 Portable device for measuring curling and warping (*14*)

2.2.4 FWD testing and analysis

2.2.4.1 FWD test plan

Falling weight deflectometer (FWD) testing was performed by MnDOT at the CSAH 46 overlay in Freeborn County, MN. The FWD involves dropping a steel load plate on the pavement at a specified target load and measuring the deflection of the pavement surface at varying distances from the load plate. Depending on the distance from the load plate, deflections are influenced to varying degrees by the response of the different pavement layers, including the surface, base/subbase layers, and the subgrade. A schematic demonstrating the concept of FWD testing is shown in Figure 2.2 (*15*).

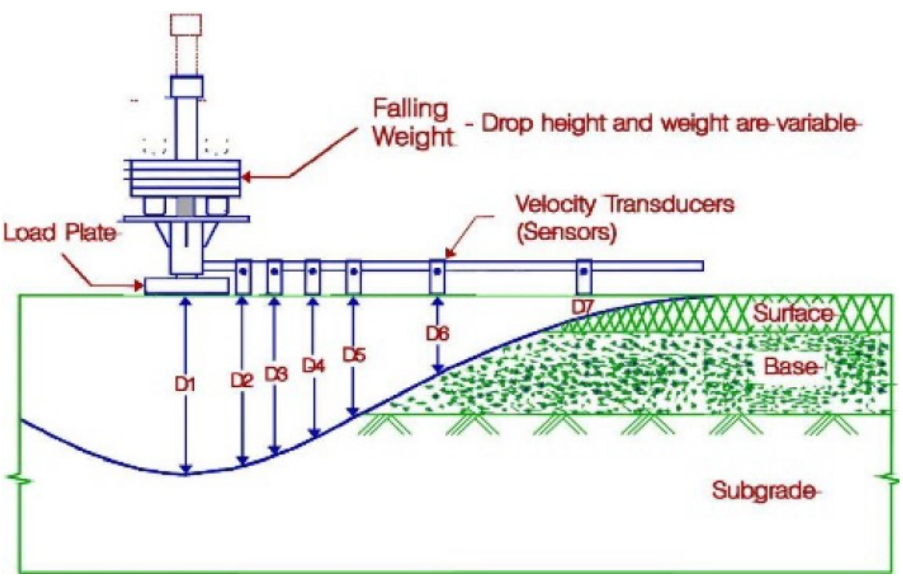

Figure 2.2 FWD testing (*15*)

FWD tests were performed on eight different slabs at five locations on each slab according to the test diagram in Figure 2.3, which is based on the testing plan developed during the Long-Term Pavement Performance (LTPP) program. Ultimately, only data from testing at locations J_1, J_4, and J_5 were used for analysis, which will be described in more detail later in this section.

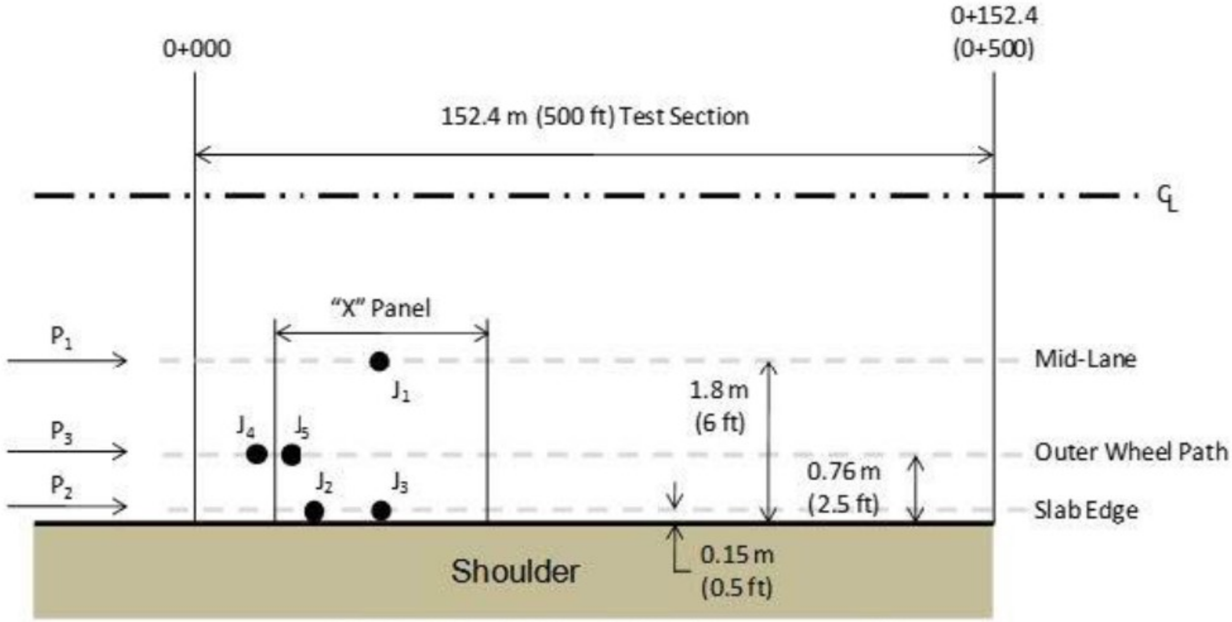

Figure 2.3 FWD testing plan (*15*)

The configuration of the sensors on the FWD device for calculating deflection is listed in Table 2.2. The loading sequence that was used for testing at each slab location (J_1 through J_5), which was based on the LTPP program, is listed in Table 2.3.

Table 2.2 FWD sensor spacing from the center of the load plate

Sensor #	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Distance (in)	0	8	12	18	24	36	48	60	72	-12

Table 2.3 FWD loading sequence (*15*)

Target Load (lbf)	**# of Drops**
6,000	4
9,000	4
12,000	4
16,000	4

2.2.4.2 Analysis of FWD data

FWD data were analyzed to gain two key insights into the performance of the concrete overlay of FDR on CSAH 46. These items of interest were calculation of the load transfer efficiency at the transverse joints, and backcalculation of the modulus of subgrade reaction and elastic modulus of the concrete and FDR layers.

Joint load transfer performance was measured based on the FWD drops performed across the transverse joints at locations J_4 and J_5 in Figure 2.3. From the deflection values obtained on the loaded (δ_L) and unloaded (δ_U) sides of the joint, the joint load transfer efficiency (LTE) was calculated according to Equation 2.1. The joint LTE characterizes the ability of the joints to effectively transmit the load from one side of the joint to the next (*15*).

$$LTE = \frac{\delta_U}{\delta_L} \times 100\% \qquad (2.1)$$

The center slab FWD drops (location J_1) were used to perform a closed-form backcalculation of the dynamic modulus of subgrade reaction, or dynamic k-value, as well as the modulus of elasticities of concrete and of the FDR layer. These calculations were performed according to the AREA method as outlined by Pierce et al. (*15*).

First, a normalized deflection basin (*AREA*) was calculated from the relative deflections at increasing distance from the load plate according to Equation 2.2, where δ_x represents the deflection at a distance *x* inches from the load plate.

$$AREA = 4 + 6\frac{\delta_8}{\delta_0} + 5\frac{\delta_{12}}{\delta_0} + 6\frac{\delta_{18}}{\delta_0} + 9\frac{\delta_{24}}{\delta_0} + 18\frac{\delta_{36}}{\delta_0} + 12\frac{\delta_{48}}{\delta_0} \qquad (2.2)$$

AREA values obtained for each center slab load drop were then used to calculate estimates for the radius of relative stiffness, ℓ, according to Equation 2.3 for an infinite slab. After calculating ℓ, estimates for the dynamic modulus of subgrade reaction, k, were obtained using Equation 2.3, where *P* is the applied load and δ_0 is the deflection underneath the load plate.

$$k = \frac{P \cdot 0.1245 e^{-0.14707 e^{-0.07565\ell}}}{\delta_0 \cdot \ell} \qquad (2.3)$$

From there, adjustment factors for were used to re-calculate the dynamic k-values based on the finite dimensions of the slabs (15 x 13.5 ft) (*16*). Next, these k-values were used to calculate the effective elastic modulus of the combined concrete overlay and FDR layer (E_e) according to the Equation 2.4, where *v* is the Poisson's ratio of concrete (0.15) and *h* is concrete overlay thickness.

$$E_e = \frac{h^3}{12\ell^4(1-v^2)k} \qquad (2.4)$$

From the effective combined $\underline{E_e}$ value, the separate elastic modulus values for the concrete and FDR layers were resolved according to a procedure outlined by Khazanovich et al. (*16*). These modulus values were calculated two ways based on different assumptions for the condition at the interface between the two layers. First, it was assumed that the two layers acted separately as two unbonded plates. Under this condition, E_{PCC} and E_{FDR} were calculated using Equations 2.5 and 2.6, where h_1 and h_2 are the thicknesses of the PCC and FDR layers, respectively, and β is the ratio of E_{FDR} to E_{PCC}.

$$E_{PCC} = \frac{h_1^3}{h_1^3 + \beta h_2^3} E_e \qquad (2.5)$$

$$E_{FDR} = \frac{\beta h_1^3}{h_1^3+\beta h_2^3} E_e \qquad (2.6)$$

Second, a bond condition was assumed at the interface between the two layers. Note that this assumption does not necessarily mean that the two layers are physically bonded. Rather, it assumes that the slab and subbase layer are in full contact with each other underneath the FWD load plate with substantial friction (*17*). Under this condition, E_{PCC} and E_{FDR} were calculated using Equations 2.7 through 2.9.

$$E_{PCC} = \frac{h_1^3}{h_1^3+\beta h_2^3+12h_1(x-\frac{h_1}{2})^2+12\beta h_2(h_1-x+\frac{h_2}{2})^2} E_e \qquad (2.7)$$

$$E_{FDR} = \frac{\beta h_1^3}{h_1^3+\beta h_2^3+12h_1(x-\frac{h_1}{2})^2+12\beta h_2(h_1-x+\frac{h_2}{2})^2} E_e \qquad (2.8)$$

$$\text{where } x = \frac{\frac{h_1^2}{2}+\beta h_2(h_1+\frac{h_2}{2})}{h_1+\beta h_2} \qquad (2.9)$$

In Equations 2.5 through 2.9, a value for *β* must be selected before the calculations are performed. A ratio of 1/100 was used for this calculation, which was the suggested value for both bitumen-treated subbase layers and recycled concrete subbase layers for use in backcalculation in the LTPP program (*16*).

2.2.5 MIRA testing and analysis

A MIRA ultrasonic tomography device was used to perform testing at the overlay on CSAH 46. This device uses an array of dry point contact transducers applied to a concrete surface to send and receive sound waves through a concrete element. The signals received back at the device are used to reconstruct a two-dimensional cross-section image. Based on the intensity of the signal reflected back to the MIRA as a function of depth, the user can determine the location of interfaces between different materials, which can be used to locate reinforcing bars, voids, and determine element thickness (*18*).

Two common applications of MIRA for concrete pavements include determination of layer thickness (*18*) and evaluation of bond between two layers (*19*). The MIRA device was used in this study to estimate the thickness of the concrete overlay. In the case of this project, the different natures of the concrete and FDR layers were such that it was likely not possible to use MIRA to infer whether they might be physically bonded to each other. Testing of a concrete pavement surface with the MIRA device is pictured in Figure 2.4.

Figure 2.4 Testing of a concrete pavement surface with MIRA

2.3 PROJECT EVALUATION

2.3.1 CSAH 46, Freeborn County, Minnesota

2.3.1.1 Project design and construction details

The concrete overlay of FDR on Freeborn County State Aid Highway (CSAH) 46 was constructed in 2006. This section is a two-lane rural county highway located between the intersection with County Highway 38 (780th Avenue) and the intersection with 840th Avenue just east of Albert Lea, MN. Design and construction details are included in Table 2.4 and the project location is indicated in Figure 2.5.

Table 2.4 CSAH 46 project details

Construction Year	2006
Project Length (miles)	5.96
AADT (vpd)	3,503
AADTT (vpd)	n/a
PCC Thickness (in)	7.5
Joint Spacing, Transverse x Longitudinal (ft)	15 x 13.5
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	8

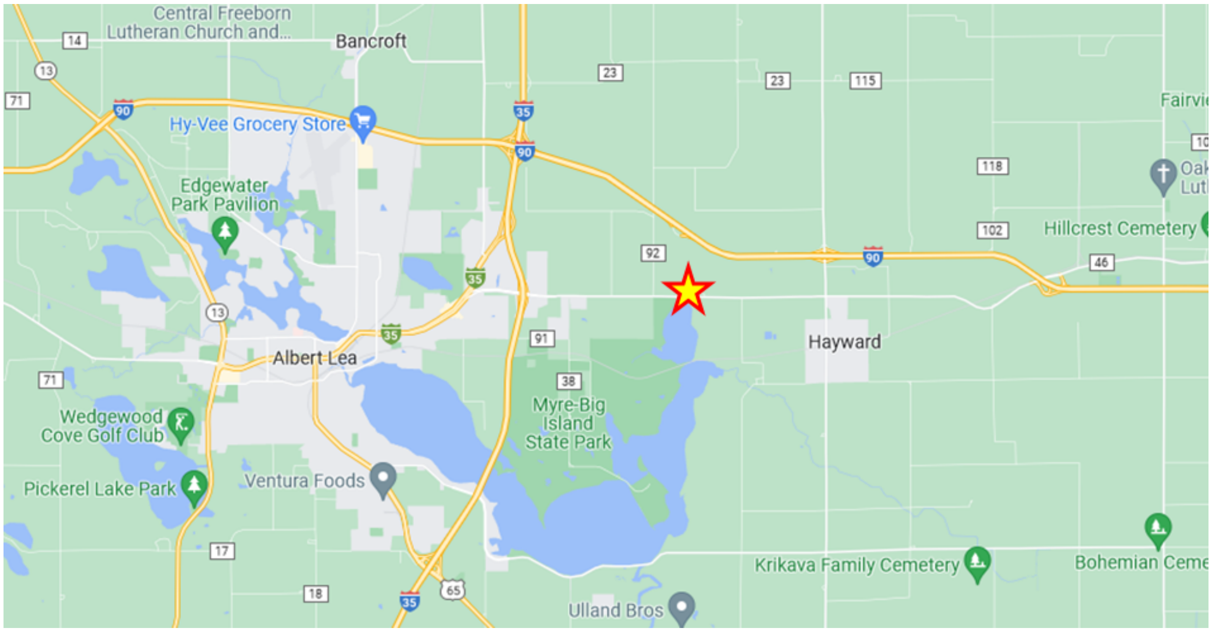

Figure 2.5 CSAH 46 project location in Freeborn County, MN

The existing pavement was 9.5 inches of bituminous surface over a 2-inch Class 5 aggregate base. For the FDR, 6.5 inches of the bituminous surface was milled off. After milling, 3 inches of new Class 5 base was added to the top of the remaining 3 inches of the bituminous layer. Then, reclamation was carried out on the combined 8 inches of new and existing class 5 material and the bituminous material. The reclamation was performed according to section 2215 of the MnDOT Standard Specifications (*20*), with the materials pulverized and blended in-place to produce a uniform aggregate base. The FDR was not stabilized with any type of bituminous or cementitious material. After compaction of the FDR layer, the 7.5-inch thick concrete overlay was paved on top. The dowel bars consisted of assemblies of 3 bars that were placed only in the outer wheel path in both lanes, with no dowels in the inner wheel path. Figure 2.6 contains the typical cross section from the plans.

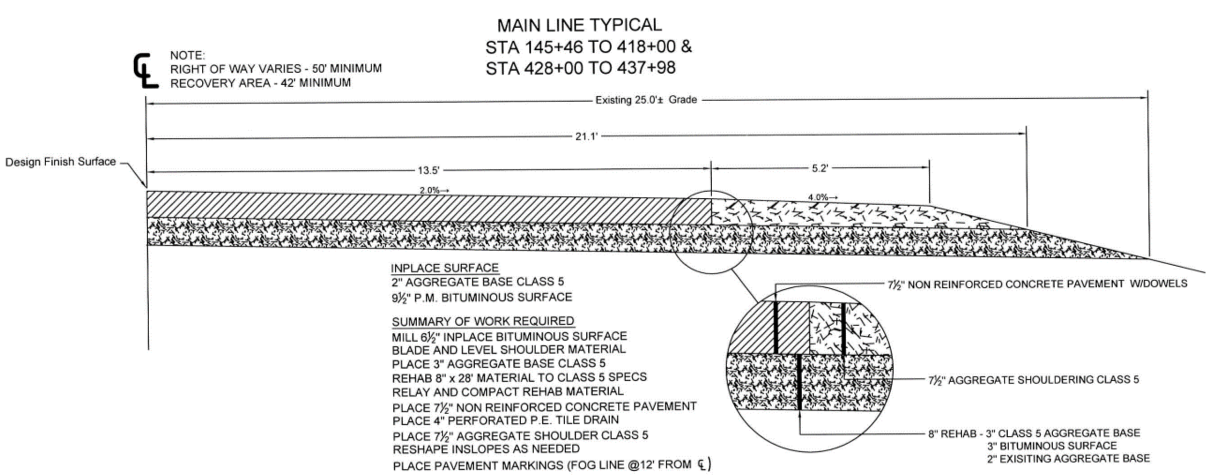

Figure 2.6 Typical cross section for CSAH 46

2.3.1.2 Pavement performance data

Automated pavement condition data collected for CSAH 46 between 2010 and 2020 are listed in Table 2.5. Measurements include IRI data collected in 2010, 2014, 2018, and 2020, and average faulting data collected in 2014 and 2018. Both sets of data were collected in both directions, and IRI values are an average of measurements taken in the left and right wheel paths.

Table 2.5 Pavement condition data for CSAH 46

Year	Pavement Age (years)	IRI (in/mi)	Average Faulting (in)
2010	4	71	n/a
2014	8	81	0.027
2018	12	84	0.027
2020	14	93	n/a

As seen in Table 2.5, pavement roughness increased steadily over time, although as of 2020 the IRI was still under the FHWA threshold defining good ride quality (less than 95/mi) (*21*). Average faulting held steady between 2014 and 2018 at 0.027 inches.

2.3.1.3 Visual survey

On November 5th, 2020, the research team and MnDOT personnel visited this project to perform a visual survey, FWD testing, and to collect MIRA measurements. Figures 2.7 through 2.10 contain photos showing the typical condition of the pavement at the time of the survey. While no detailed distress survey was carried out to categorize and quantify the prevalence of different distress types, etc., a number of instances of longitudinal cracking were noted throughout the project, as shown in Figures 2.8 through 2.10.

Figure 2.7 CSAH 46 typical project conditions

Figure 2.8 CSAH 46 typical project conditions showing longitudinal cracking

Figure 2.9 Close-up view of longitudinal crack filled with sealant

Figure 2.10 Longitudinal crack with spalling at intersection with transverse joint near location of dowel bars

The longitudinal cracks occurred both at mid-panel locations as well as closer to the edge of the slab under the outer wheel path. Cracks tended to run continuously through several slabs at a time, continuing across the transverse joints for lengths ranging from 45 to 150 ft. Cracks sometimes terminated at transverse joints, like in Figure 2.9, while other times they turned out and terminated at the pavement edge or developed branches that ran to the edge, which can be seen in Figures 2.8 and 2.10.

The majority of cracks had been filled with a hot pour filler material to protect against spalling. In general, the cracks appeared to be in stable condition, with no observations of significant spalling or faulting across the cracks. In some cases, like in Figure 2.10, spalling occurred at the intersection of a longitudinal crack and transverse joint. A small number of transverse cracks were also observed, with an example pictured in Figure 2.11. The transverse cracks were in a similar condition to the longitudinal cracks.

During the visual survey, curling measurements were performed on four slabs using the portable device developed by Ceylan et al. (*14*). The measurements were performed between 10:00 am and 2:30 pm. Curling was measured longitudinally along the outside edge of the slab, longitudinally down the middle of the slab, and diagonally across the slab. Each slab was only measured once, so these measurements only provided an indication of slab curling in the middle of a late fall day without insight into how the profile might change at different times or seasons. Figure 2.12 shows a diagonal curling measurement performed at the slab corner.

Figure 2.11 Transverse crack on CSAH 46

Figure 2.12 Curling measurement performed at slab corner on CSAH 46

Each slab was curled upward. The maximum changes in elevation between the interior of the slab and the edges/corners of the slab ranged from 0.05 to 0.10 inches when measured longitudinally, to 0.10 to 0.15 inches when measured diagonally. While this test method does not have an established scale to correlate the readings to the degree or severity of curling, these measurements were consistent with typical measurements for full-depth concrete pavements included in the study by Ceylan et al. (*14*). In the context of the pavement smoothness and joint load transfer measurements (which are discussed in the next section), it did not appear that the extent of curling at this project was notably small or large.

2.3.1.4 FWD testing and analysis

FWD testing was performed at 8 different slabs throughout the project, with an average distance of about 3,600 ft between each slab that was tested. Joint LTE values obtained from the drops performed before and after the transverse joint at each slab location are listed in Table 2.6. As seen in the table, each slab had a joint LTE above 90%, which is considered excellent joint performance (*22*), and the average across all joints was 92.8%.

Table 2.6 Joint LTE at each slab location

Slab Location	Joint LTE (%)
1	94.4
2	94.1
3	94.4
4	92.0
5	92.7
6	90.8
7	93.4
8	90.6
Average	**92.8**

Table 2.7 contains backcalculated values obtained from the center slab FWD drop locations for the dynamic modulus of subgrade reaction (dynamic k-value) and the elastic modulus (E) of the concrete and FDR layers. Two sets of modulus values are listed for each slab, one assuming no bond at the interface of the slab and FDR layer, and one assuming a bonded interface.

As seen from the results in Table 2.7, assuming a bonded condition at the interface between the slab and FDR layer produced a much more realistic average backcalculated value for the elastic modulus of concrete (6,780,257 psi) compared to assuming no bond at the interface (10,031,970 psi). This finding is consistent with analysis of LTPP data, where assuming a bonded condition between slab and subbase has produced better results for a significant majority of concrete LTPP sections around the US (*16,17*).

Table 2.7 Backcalculated elastic modulus at each slab location

Slab Location	Dynamic k-value (psi/in)	Assuming No Bond at Interface		Assuming Bonded Interface	
		E_{PCC} (psi)	E_{FDR} (psi)	E_{PCC} (psi)	E_{FDR} (psi)
1	199	9,446,892	94,469	6,384,823	63,848
2	225	11,193,020	111,930	7,564,970	75,650
3	294	12,419,426	124,194	8,393,855	83,939
4	307	8,732,866	87,329	5,902,237	59,022
5	233	10,073,676	100,737	6,808,444	68,084
6	265	8,864,431	88,644	5,991,158	59,912
7	162	12,503,301	125,033	8,450,543	84,505
8	216	7,022,148	70,221	4,746,024	47,460
Average	**238**	**10,031,970**	**100,320**	**6,780,257**	**67,803**

The average backcalculated elastic modulus of the unstabilized FDR layer assuming a bonded interface was 67,803 psi. This value is greater than typical backcalculated values for crushed stone bases, which typically vary between about 20,000 and 40,000 psi (*16,17*). However, this value is substantially lower than the modulus of a cement-stabilized soil/aggregate mixture, which is commonly found to be around 500,000 psi (*16*).

Based on guidelines in the Mechanistic-Empirical Pavement Design guide (*22*), the average backcalculated elastic modulus of the FDR layer (assuming a bonded interface) of 67,803 psi would produce a design resilient modulus value of approximately 89,500 psi, assuming a multiplier of 1.32 recommended for use for an unstabilized FDR layer.

2.3.1.5 MIRA testing and analysis

MIRA was used to take measurements at 16 different slab locations on the CSAH 46 project. Two-dimensional cross-sectional b-scan images were obtained for each measurement, with one example pictured in Figure 2.13. The strong red reflection in the b-scan image in Figure 2.13 occurs at a depth of approximately 200 mm (7.8 inches), which corresponds closely to the design concrete thickness (7.5 inches), indicating that it represents a back-wall reflection from the bottom of the concrete layer. Another, weaker reflection in the b-scan image occurs at a depth of approximately 400 mm (15.7 inches), which likely represents a second reflection from the bottom of the concrete layer. The two reflections in yellow at a depth of approximately 75 to 90 mm (3 to 3.5 inches) likely correspond to reinforcing bars.

Average thickness values were not calculated from the data since they would only represent rough approximations from the b-scan images, but in general the first reflection in each b-scan occurred at depths between 7.5 and 8.0 inches. It would not be surprising if the overlay depth slightly exceeded 7.5 inches in many locations, as it is not uncommon for in-place concrete pavement thickness to exceed design thickness depending on construction penalties or disincentives for inadequate thickness.

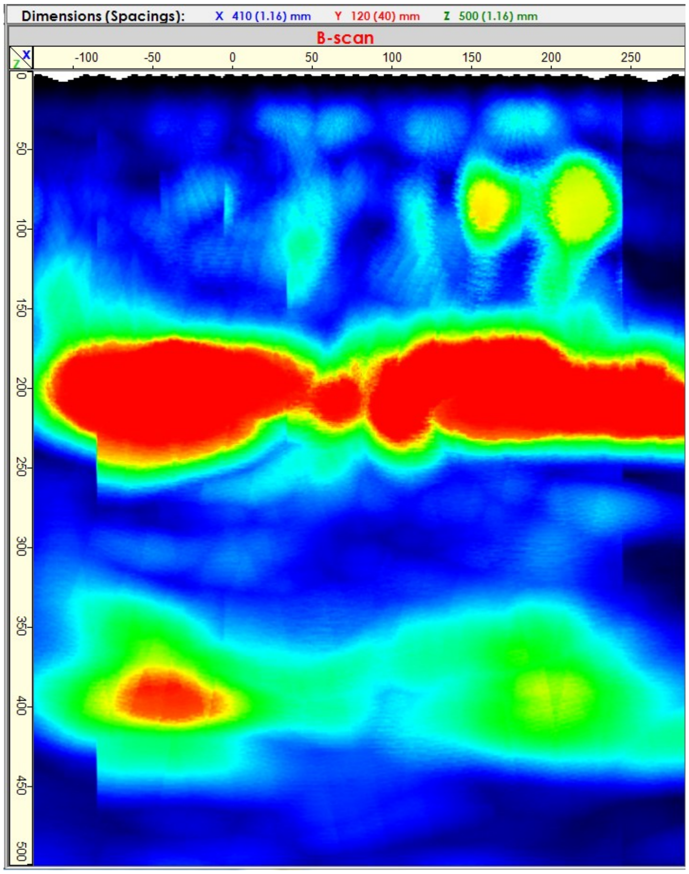

Figure 2.13 B-scan image obtained from MIRA testing

It should be noted that the backcalculation procedure is sensitive to slab thickness, so if the concrete was greater than 7.5 inches in practice, it might change the backcalculated elastic modulus values for both the concrete and FDR layers. If the concrete overlay was assumed to be 8 inches thick, the average elastic modulus of concrete (assuming bonded interface condition) would be 5,760,889 psi, while the average modulus of the FDR layer would be 57,609 psi, holding all other inputs and assumptions equal.

2.3.2 US 75, Brown County, Kansas

2.3.2.1 Project design and construction details

The concrete overlay of FDR on US 75 in Brown County, KS was constructed in 2012. This section is a two-lane rural US highway running north along the Brown-Nemaha County line between the town of Sabetha to the Nebraska state line. Design and construction details are included in Table 2.8 and the project location is indicated in Figure 2.14.

This project was originally designed as a 4-inch concrete on asphalt–bonded (COA–B) overlay. The existing 7-inch asphalt pavement was planned to be milled down to 3 inches prior to overlay. However, at the beginning of the milling process, extensive deterioration was observed in the existing asphalt and it was decided not to proceed with the original COA–B overlay design out of concern for its future performance.

Table 2.8 US 75 project details

Construction Year	2012
Project Length (miles)	4.50
AADT (vpd)	2,150
AADTT (vpd)	793
PCC Thickness (in)	8
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Stabilized with cement
FDR Layer Thickness (in)	7

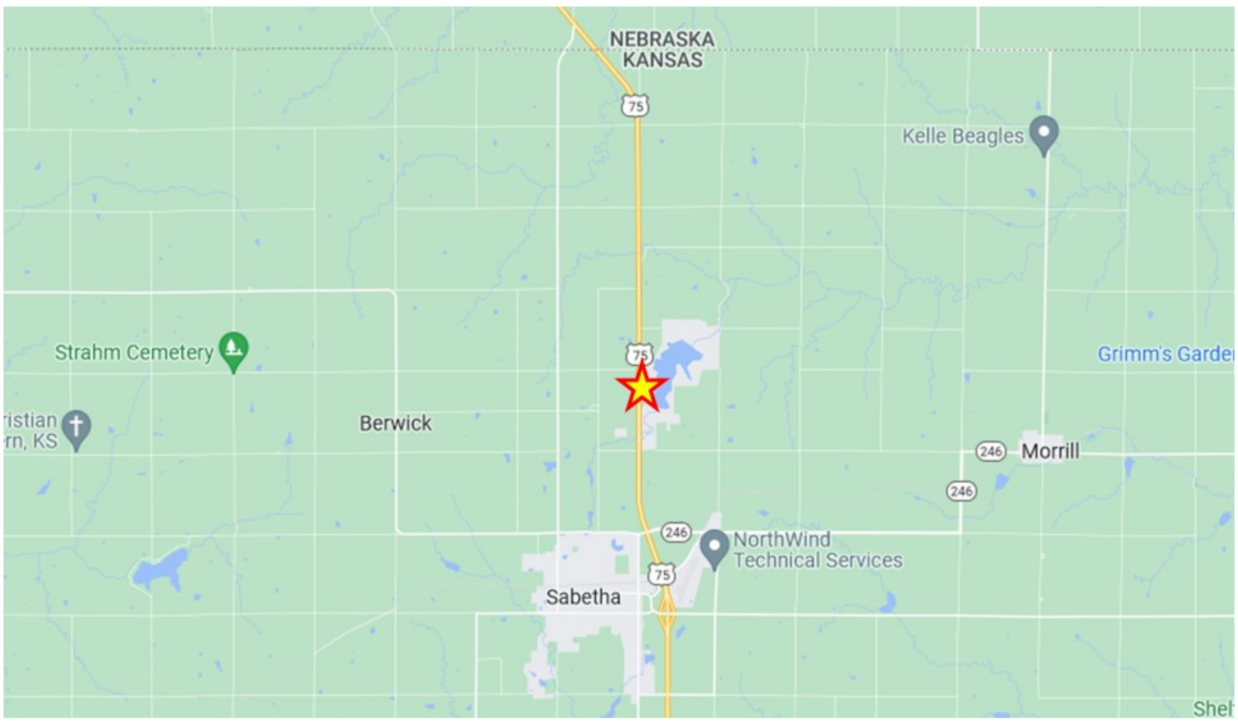

Figure 2.14 US 75 project location in Brown County, KS

The design was changed to an 8-inch concrete overlay of FDR. After milling, the existing pavement was reclaimed to a depth of 7 inches according to a special provision for subgrade modification with in-place material (existing pavement). The FDR layer was stabilized with cement at a dosage rate of 6%. The cement was spread over the existing material, which was then pulverized and mixed in place until homogenous and friable. The overlay also included 6 to 8-inch thick integral paved concrete shoulders placed over FDR of the existing shoulder material. After paving, the entire project was diamond ground.

2.3.2.2 Pavement performance data

Automated pavement condition data collected for US 75 in 2015 and 2020 are listed in Table 2.9. Measurements include IRI and average faulting. As seen in the table, pavement roughness has increased over time, but the ride was very smooth from an early age thanks to the initial diamond grinding and has remained smooth with low average faulting values. As of 2022, no significant distresses have been reported for this pavement, and a current Google street view image is included in Figure 2.15.

Table 2.9 Pavement condition data for US 75

Year	Pavement Age (years)	IRI (in/mi)	Average Faulting (in)
2015	2	30	0.016
2020	7	42	0.023

Figure 2.15 Current condition of US 75

2.3.3 K-68, Franklin County, Kansas

2.3.3.1 Project design and construction details

The concrete overlay of FDR on K-68 in Franklin County, KS was constructed in 2014. This section is a two-lane rural state highway that runs east for approximately 1.25 miles from an interchange with I-35 outside of Ottawa, KS. Design and construction details are included in Table 2.10, and the project location is indicated in Figure 2.16. Although truck traffic data were not available for this project, it was adjacent to a Walmart distribution facility, so the percentage of trucks is likely high.

The existing pavement on K-68 was a 9 inch unbonded concrete on asphalt (COA–U) overlay. The concrete overlay was completely removed, and the remaining underlying asphalt was used for the FDR layer. After milling the asphalt layer down by 1 to 1.5 inches, it was reclaimed according to section 301 of the Kansas DOT Standard Specifications (*23*) to a depth of 6 inches. The FDR layer was not stabilized.

Prior to construction of the new concrete overlay, a geotextile fabric was placed over the FDR layer. The overlay also included integral paved concrete shoulders over the FDR layer. Figure 2.17 contains the typical cross section from the plans.

Table 2.10 K-68 project details

Construction Year	2014
Project Length (miles)	1.25
AADT (vpd)	5,750
AADTT (vpd)	n/a
PCC Thickness (in)	9
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	6

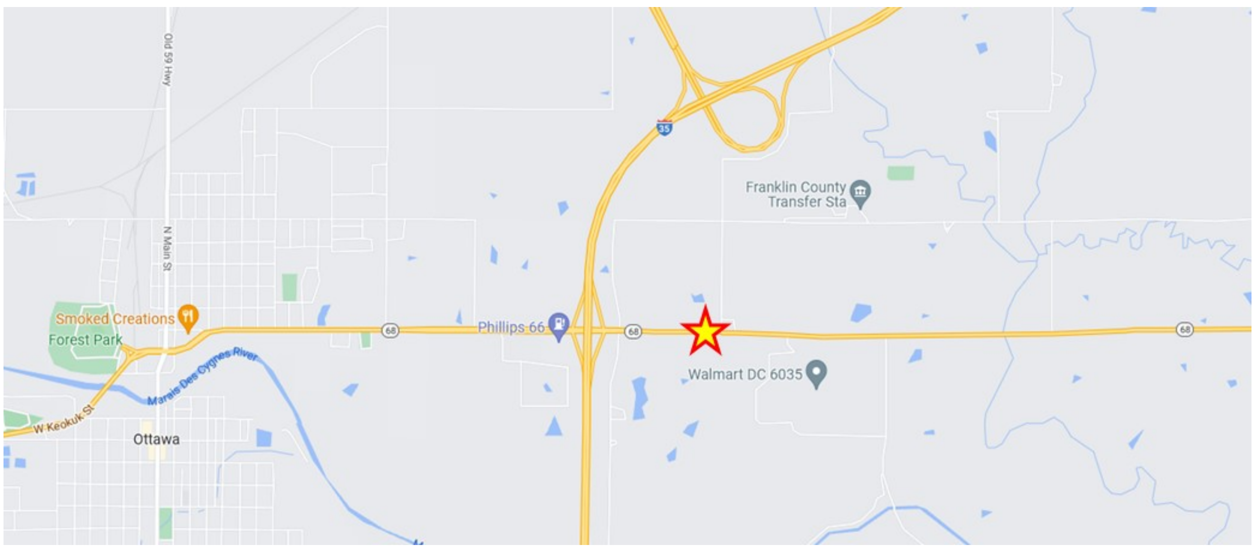

Figure 2.16 K-68 project location in Franklin County, KS

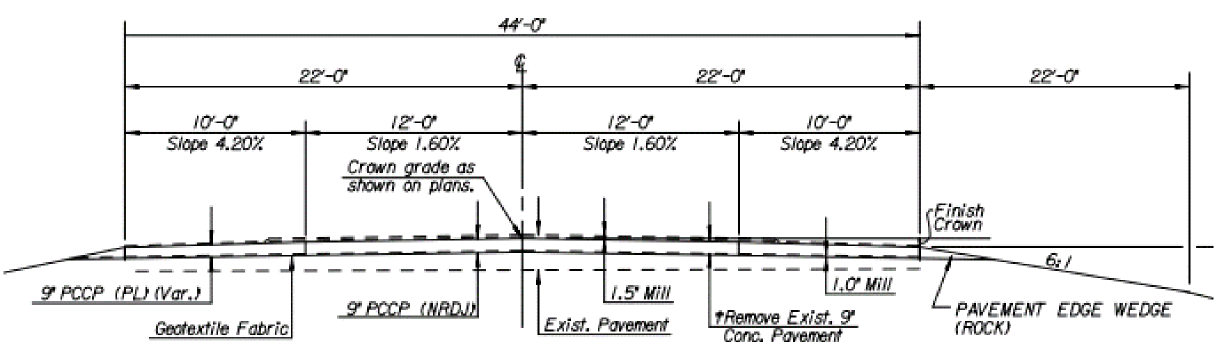

Figure 2.17 Typical cross section for K-68

2.3.3.2 Pavement performance data

Automated pavement condition data collected for K-68 in 2015 and 2020 are listed in Table 2.11. Measurements include IRI and average faulting. As seen in the table, the pavement has exhibited good ride quality and IRI and faulting has remained steady with time. As of 2022, no significant distresses have been reported for this pavement. A current Google street view image is included in Figure 2.18.

Table 2.11 Pavement condition data for K-68

Year	Pavement Age (years)	IRI (in/mi)	Average Faulting (in)
2015	1	53	0.019
2020	6	60	0.017

Figure 2.18 Current condition of K-68

2.3.4 US 81, Republic County, Kansas

2.3.4.1 Project design and construction details

The concrete overlay of FDR on US 81 in Republic County, KS was constructed in 2017. This section is a divided highway running north-south on US 81 for about 2.23 miles south of an interchange with US 36 near Belleville, KS. Design and construction details are included in Table 2.12 and the project location is indicated in Figure 2.19.

The existing asphalt pavement section was milled to a depth of 9.5 inches, and then reclaimed according to section 301 of the Kansas DOT Standard Specifications (*23*) to a further depth of 6 inches. The FDR layer was not stabilized, although the subgrade underneath the existing pavement had been previously modified with lime. From there, the 9.5-inch concrete overlay was paved over the FDR layer, matching the level of the existing pavement surface.

Table 2.12 US 81 project details

Construction Year	2017
Project Length (miles)	2.23
AADT (vpd)	5,778
AADTT (vpd)	n/a
PCC Thickness (in)	9.5
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	6

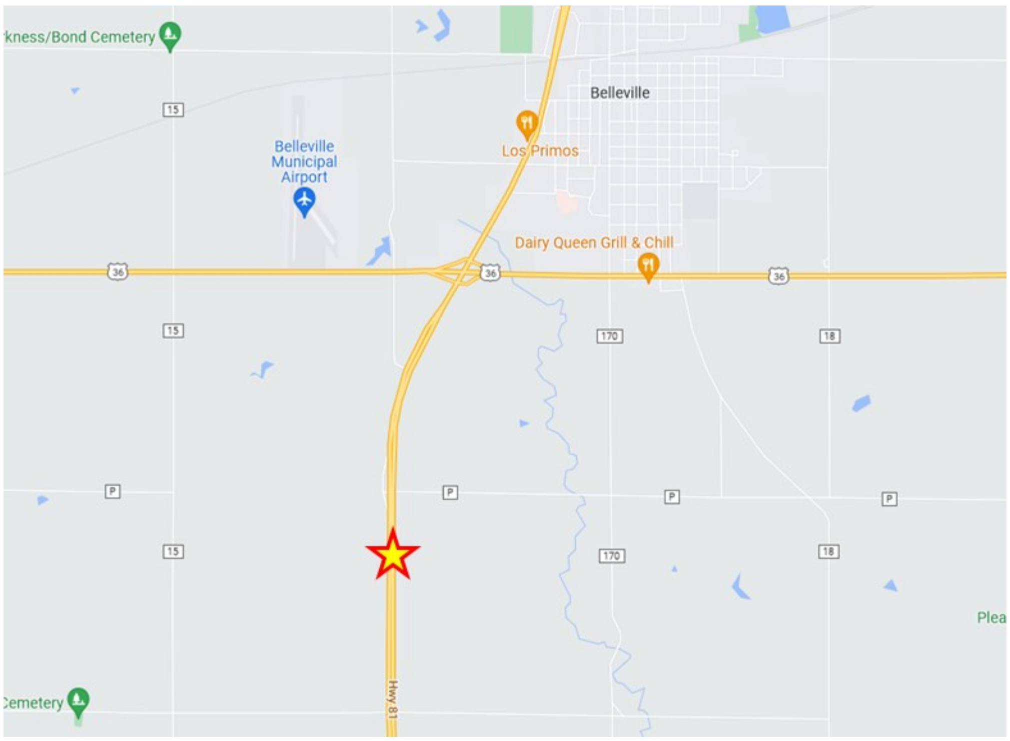

Figure 2.19 US 81 project location in Republic County, KS

2.3.4.2 Pavement performance data

Automated pavement condition data collected for US 81 in 2018 and 2020 are included in Table 2.13. Data include IRI and average faulting, which were averaged across both the NB and SB lanes of the divided highway. As seen in the table, both IRI and faulting have remained steady since construction. As of 2022, the only distresses that have been reported for this pavement are some joint deterioration and cracking in pavement placed at a center turn lane area. No distresses have been reported in the mainline pavement, which is pictured in Figure 2.20.

Table 2.13 Pavement condition data for US 81

Year	Pavement Age (years)	IRI (in/mi)	Average Faulting (in)
2018	1	78	0.016
2020	3	78	0.020

Figure 2.20 Current condition of US 81

2.3.5 I-70, Gove County, Kansas

2.3.5.1 Project design and construction details

The concrete overlay of FDR on I-70 was constructed in 2019. This section is a divided interstate highway located approximately between mileposts 94 and 103 between the towns of Grainfield and Quinter in Kansas. Design and construction details are included in Table 2.14, and the project location is indicated in Figure 2.21.

Table 2.14 I-70 project details

Construction Year	2019
Project Length (miles)	8.99
AADT (vpd)	12,000
AADTT (vpd)	4,200
PCC Thickness (in)	12
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Stabilized with cement
FDR Layer Thickness (in)	12

Figure 2.21 I-70 project location in Gove County, KS

The existing cross section consisted of 24.5 inches of asphalt pavement. The top 12 inches of the surface was milled off, and then reclaimed according to section 301 of the Kansas DOT Standard Specifications (*23*) to a depth of 12 inches. The FDR layer was stabilized with cement at a dosage rate of 6%, and an additional 1.5 inches of soil was added to fill in the voids and increase the density. The overlay also included integral paved shoulders over the FDR layer. The typical cross section for the EB lanes is shown in Figure 2.22. (The typical cross sections were the same in the EB and WB lanes.)

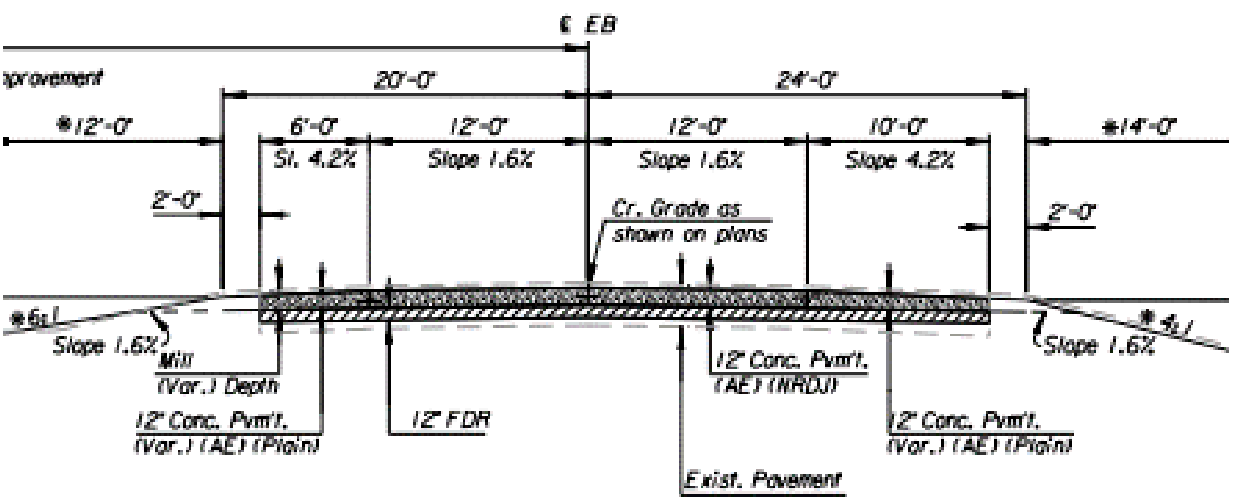

Figure 2.22 Typical I-70 cross section

2.3.5.2 Pavement performance data

Automated pavement condition data collected for I-70 collected in 2019 and 2020 are included in Table 2.15. Measurements included IRI and average faulting. As expected for a pavement that was only one year old at the time of the last survey, the condition has not changed much, with good ride quality and little faulting. No distresses have been reported as of 2022, and a current Google street view image is included in Figure 2.23.

Table 2.15 Pavement condition data for I-70

Year	Pavement Age (years)	IRI (in/mi)	Average Faulting (in)
2019	0	73	0.023
2020	1	71	0.028

Figure 2.23 Current condition of I-70

2.3.6 US 385, Yuma County, Colorado

2.3.6.1 Project design and construction details

The concrete overlay of FDR on US 385 was constructed in 2010. This section is a two-lane rural county highway running north for 6.7 miles from an intersection with US 36 about 10 miles west of the Kansas border. Design and construction details are included in Table 2.16 and the project location is indicated in Figure 2.24.

The existing asphalt pavement at this section was reclaimed to a depth of 8 inches throughout most of the project, with reclamation extending to a depth of 12 inches at some locations based on variation in pavement thickness. The reclamation was performed according to section 310 of the Colorado DOT construction specifications (*24*). The existing asphalt was pulverized and mixed with subbase and subgrade material until homogeneous, the FDR layer was compacted, and the concrete overlay was placed directly on top.

Table 2.16 US 385 project details

Construction Year	2010
Project Length (miles)	6.70
AADT (vpd)	770
AADTT (vpd)	256
PCC Thickness (in)	7.5
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	8-12

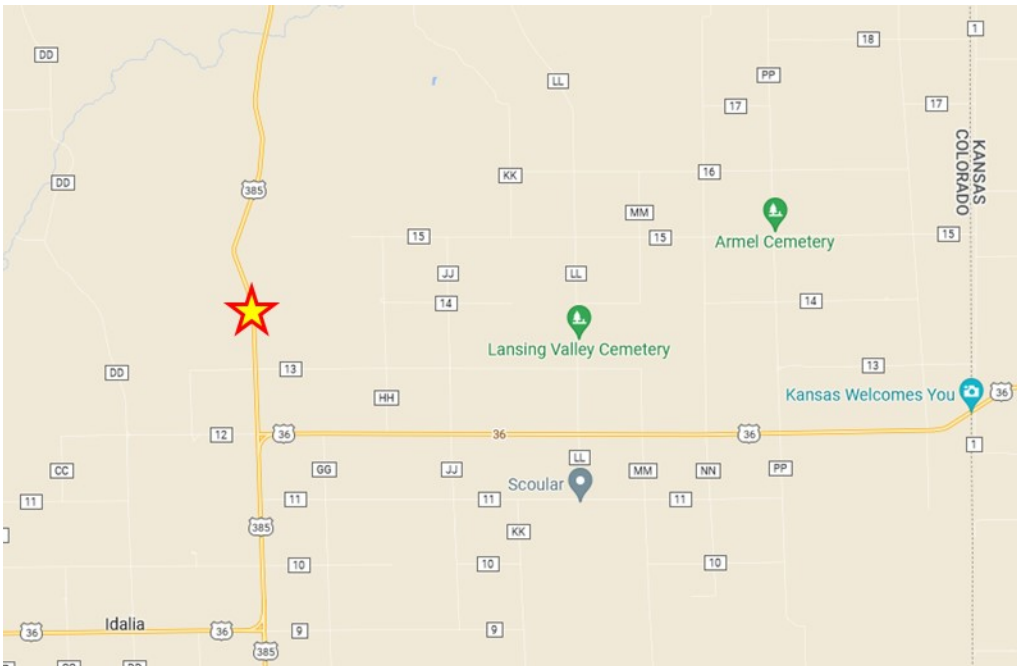

Figure 2.24 US 385 project location in Yuma County, CO

2.3.6.2 Pavement performance data

Pavement condition data collected for US 385 between 2011 and 2010 are listed in Table 2.17. Only IRI data were available for this section, with data collected on an annual basis. It was notable from the table that IRI appeared to decline from values ranging between 88 and 104 in/mi to IRI values prior to 2018 to values of 74 to 83 in/mi afterward. It is possible that this unusual finding resulted from diurnal or seasonal curling behavior that can cause significant changes to the pavement profile as a function of time of day and temperature. Although the dates and times of the smoothness measurements were not recorded to be able to confirm this hypothesis, this possibility is discussed further in section 2.3.7.2.

This pavement has also appeared to experience some longitudinal cracking, although the severity and extent of this cracking is not clear. A Google street view image of the current condition of the pavement is shown in Figure 2.25, which shows that the longitudinal cracks and transverse joints have been filled with sealant.

Table 2.17 Pavement condition data for US 385

Year	Pavement Age (years)	IRI (in/mi)
2011	1	99
2012	2	90
2013	3	92
2014	4	88
2015	5	97
2016	6	104
2017	7	99
2018	8	74
2019	9	82
2020	10	83

Figure 2.25 Current condition of US 385

2.3.7 US 34, Weld County, Colorado

2.3.7.1 Project design and construction details

The concrete overlay of FDR on US 34 was constructed in 2012. This section is a divided highway on the outskirts of Greeley, CO, running east for 2.12 miles from the interchange with US 85. Design and construction details are included in Table 2.18 and the project location is indicated in Figure 2.26.

The existing asphalt pavement ranged from 6 to 8 inches thick, and was reclaimed to a depth of 8 inches, including up to 2 inches of the underlying aggregate subbase. The reclamation was performed according to section 310 of the Colorado DOT construction specifications (*24*). The existing asphalt was pulverized and mixed with the subbase material until homogeneous, the FDR layer was compacted, and the concrete overlay was placed directly on top.

Table 2.18 US 34 project details

Construction Year	2012
Project Length (miles)	2.12
AADT (vpd)	17,000
AADTT (vpd)	1,496
PCC Thickness (in)	9
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	8

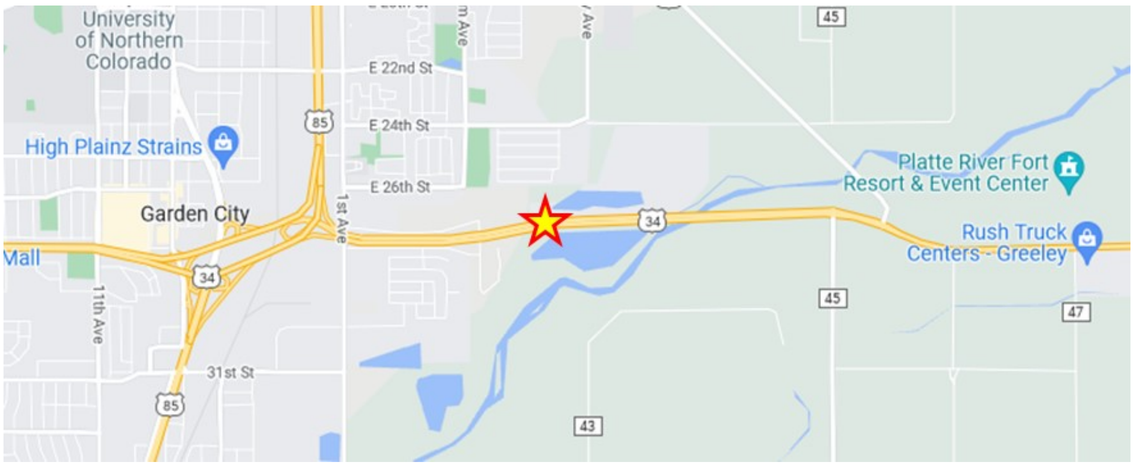

Figure 2.26 US 34 project location in Weld County, CO

2.3.7.2 Pavement performance data

Pavement condition data collected for US 34 between 2013 and 2020 are listed in Table 2.19. Only IRI data were available for this section, with data collected on an annual basis (except for 2015). As with US 385, the IRI behavior over time appeared to be somewhat unusual. In this case, a large spike occurred in 2016, before IRI values returned to a stable trendline consistent with values collected prior to 2016.

Table 2.19 Pavement condition data for US 34

Year	Pavement Age (years)	IRI (in/mi)
2013	1	68
2014	2	70
2016	4	89
2017	5	70
2018	6	66
2019	7	66
2020	8	73

Shortly after this project was constructed in 2012, certain sections of the pavement exhibited notable slab curling and undesirable roughness observed at certain times of day. As a result, Merritt et al. (*25*) performed an extensive analysis of daily and seasonal changes to curling behavior and pavement smoothness on this project. The study found differences in IRI as large as 20 to 30 in/mi between measurements taken in the early morning and early afternoon on the same sections of pavement on the same day in the summer.

The findings of the curling study suggest that the unusual spike in IRI observed in 2016 recorded in Table 2.19 could easily result from the measurement being taken at a different time of day or different season than in other years. Although the factors that might contribute to such significant slab curling are outside of the scope of this study, if similar climate conditions were present and similar materials were used on other Colorado concrete overlays of FDR such as the US 385 project, it is possible that this type of behavior could be responsible for some of the unusual trends in IRI over time.

As of 2022, no other distresses have been reported for this pavement. A Google street view image of its current condition is shown in Figure 2.27.

Figure 2.27 Current condition of US 34

2.3.8 US 24, El Paso County, Colorado

2.3.8.1 Project design and construction details

The concrete overlay of FDR on US 24 was constructed in 2017. This section is a divided highway on the outskirts of Colorado Springs, and the concrete overlay was constructed in the westbound lanes only running southwest from Falcon to Cimarron Hills. Design and construction details are included in Table 2.20 and the project location is indicated in Figure 2.28.

The existing asphalt pavement at this section was reclaimed to a depth of 12 inches. The reclamation was performed according to section 310 of the Colorado DOT construction specifications (*24*). The existing asphalt was pulverized and mixed with subbase and subgrade material until homogeneous, the FDR layer was compacted, and the concrete overlay was placed directly on top.

Table 2.20 US 24 project details

Construction Year	2017
Project Length (miles)	3.00
AADT (vpd)	19,000
AADTT (vpd)	1,444
PCC Thickness (in)	8.25
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	12

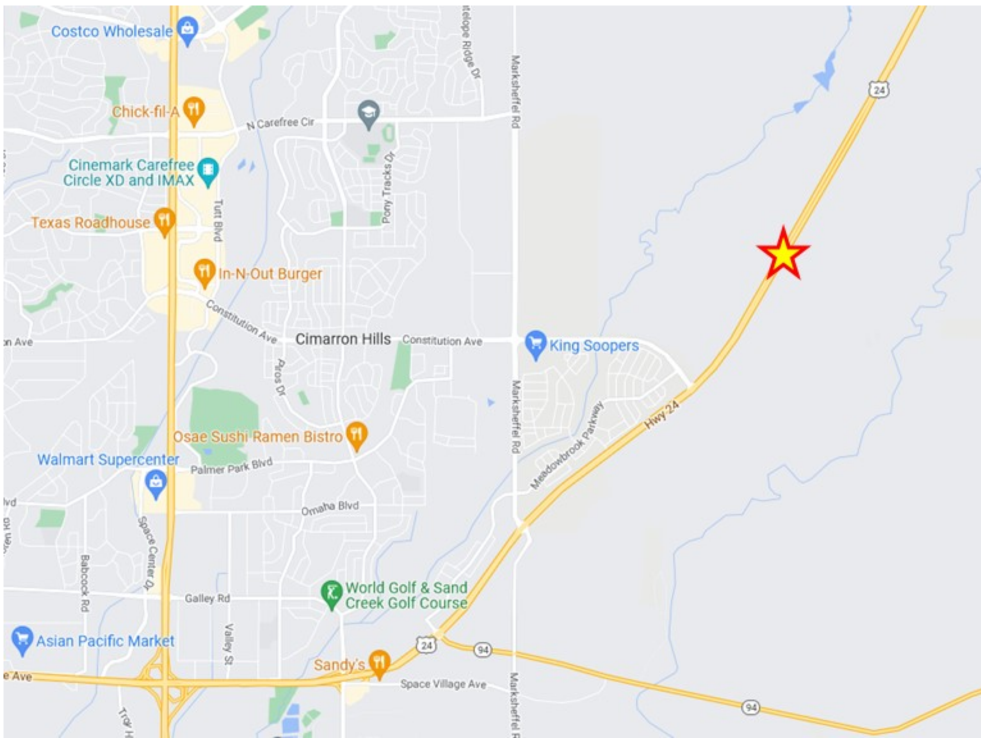

Figure 2.28 US 24 project location in El Paso County, CO

2.3.8.2 Pavement performance data

Pavement condition data for the US 24 project collected between 2018 and 2020 are included in Table 2.21. Only IRI data were available this project, with measurements taken on an annual basis. As seen in the table, this project demonstrated dips/spikes in IRI in its first 3 years in service. During construction, the contractor noted significant curling behavior shortly after placement similar to the US 34 project, including significant variations in the profile between the morning and afternoon. No distresses have been reported for this pavement as of 2022, and a Google street view images of its current condition is shown in Figure 2.29.

Table 2.21 Pavement condition data for US 24

Year	Pavement Age (years)	IRI (in/mi)
2018	1	95
2019	2	80
2020	3	99

Figure 2.29 Current condition of US 24

2.3.9 SH 45 (Pueblo Boulevard), Pueblo, Colorado

2.3.9.1 Project design and construction details

The concrete overlay of FDR on state highway (SH) 45 was constructed in 2017. This section is a boulevard section with a paved median in Pueblo, CO located between mileposts 4.9 and 8.7. Design and construction details are included in Table 2.22, and the project location is indicated in Figure 2.30.

Table 2.22 SH 45 project details

Construction Year	2017
Project Length (miles)	3.80
AADT (vpd)	20,000
AADTT (vpd)	1,000
PCC Thickness (in)	8.25
Joint Spacing, Transverse x Longitudinal (ft)	15 x 12
Dowels	Yes
FDR Type	Unstabilized
FDR Layer Thickness (in)	8

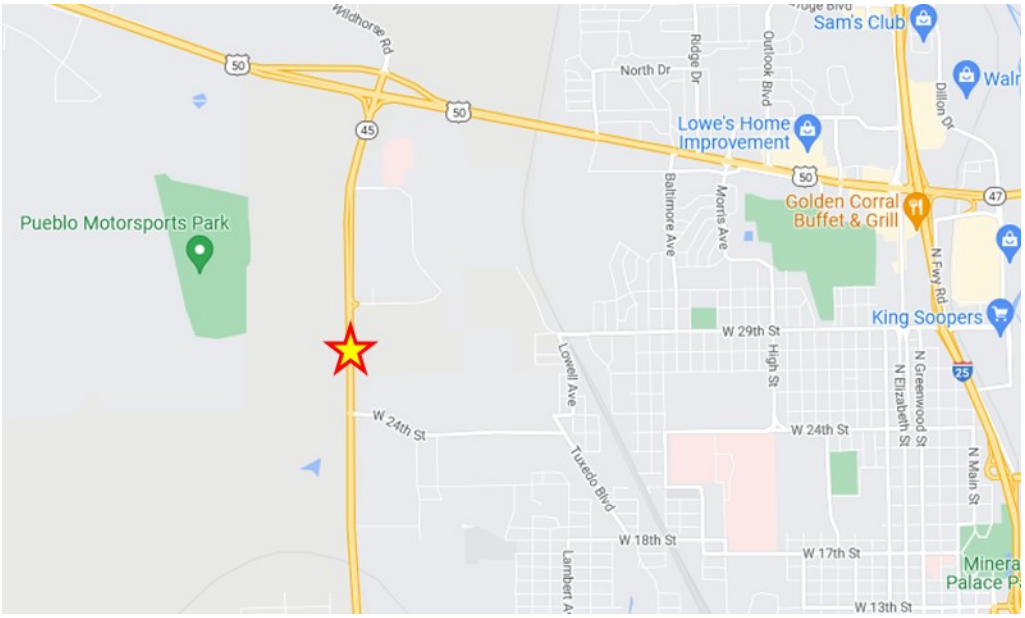

Figure 2.30 SH 45 project location in Pueblo, CO

The existing asphalt pavement at this section was reclaimed to a depth of 8 inches. The reclamation was performed according to section 310 of the Colorado DOT construction specifications (*24*). The existing asphalt was pulverized and mixed with subbase and subgrade material until homogeneous, the FDR layer was compacted, and the concrete overlay was placed directly on top.

2.3.9.2 Pavement performance data

Pavement condition data for SH 45 collected between 2018 and 2020 are included in Table 2.23. Only IRI data were available for this project, with measurements taken on an annual basis. This project demonstrated a rapid increase in IRI in its first 3 years in service. The contractor also noted significant curling behavior on this project shortly after placement, so it is difficult to distinguish whether pavement roughness might be increasing with time or whether significant curling is causing large variations in data collected year-to-year. In either case, IRI values over 100 in/mi after 2 to 3 years in service are unusually high. No other distresses have been reported for this pavement as of 2022, and a Google street view image of its current condition is shown in Figure 2.31.

Table 2.23 Pavement condition data for SH 45

Year	Pavement Age (years)	IRI (in/mi)
2018	1	72
2019	2	100
2020	3	108

Figure 2.31 Current condition of SH 45

2.4 ANALYSIS OF FINDINGS

2.4.1 Overlay structural design

The set of nine projects included in this report is a small sample, but there are still a number of insights they can provide into the performance of concrete overlays of FDR. While these projects were constructed on a variety of roadway types, from rural county roads to interstate highways, all of the cross-section designs were relatively similar. The thickest overlay section (12 inches) was designed on the pavement (I-70) which had the highest traffic levels, while the thinnest overlays (7.5 inches each) were constructed on the lowest volume roadways (CSAH 46 and US 385). All projects were constructed with doweled transverse joints.

With a minimum overlay thickness of 7.5 inches, each overlay was relatively thick when compared to bonded concrete overlays, which are usually 6 inches or less (*12*). In general, the design of these concrete overlays appeared to be most similar to thicker unbonded concrete overlay, or even a conventional concrete pavement over a subbase layer. It is notable that the US 75 project was originally designed as a thin, 4-inch COA–B overlay with a short joint spacing design, but when the design was changed to a concrete overlay of FDR, the overlay thickness was increased to 8 inches with dowels and a more conventional 15 x 12 ft joint spacing.

FDR layers at each project were constructed between 6 and 12 inches thick. At the CSAH 46 project, the backcalculated elastic modulus of the unstabilized FDR layer indicated that it was significantly stiffer than is typically found for a conventional crushed stone base. Elastic modulus values of the stabilized

FDR layers on the US 75 and I-70 projects in Kansas were not able to be determined for comparison to unstabilized FDR.

2.4.2 Ride quality performance

Ride quality in terms of IRI was the primary metric used for evaluating the performance of the concrete overlays of FDR in this study. IRI values obtained at each project for every year of data collection are plotted together in Figure 2.32. IRI is plotted separately for each project, and a linear trendline has been fit to the data for each project.

The linear trendlines in Figure 2.32 were not meant to represent a smoothness projection for future performance. Given the small sample size and the fact that most projects were still early in their life, with a maximum age of 14 years and most projects less than 10 years old, it would be difficult to use this data to project smoothness performance for these types of overlays. That said, the linear trendlines can still be helpful in characterizing and comparing project performance.

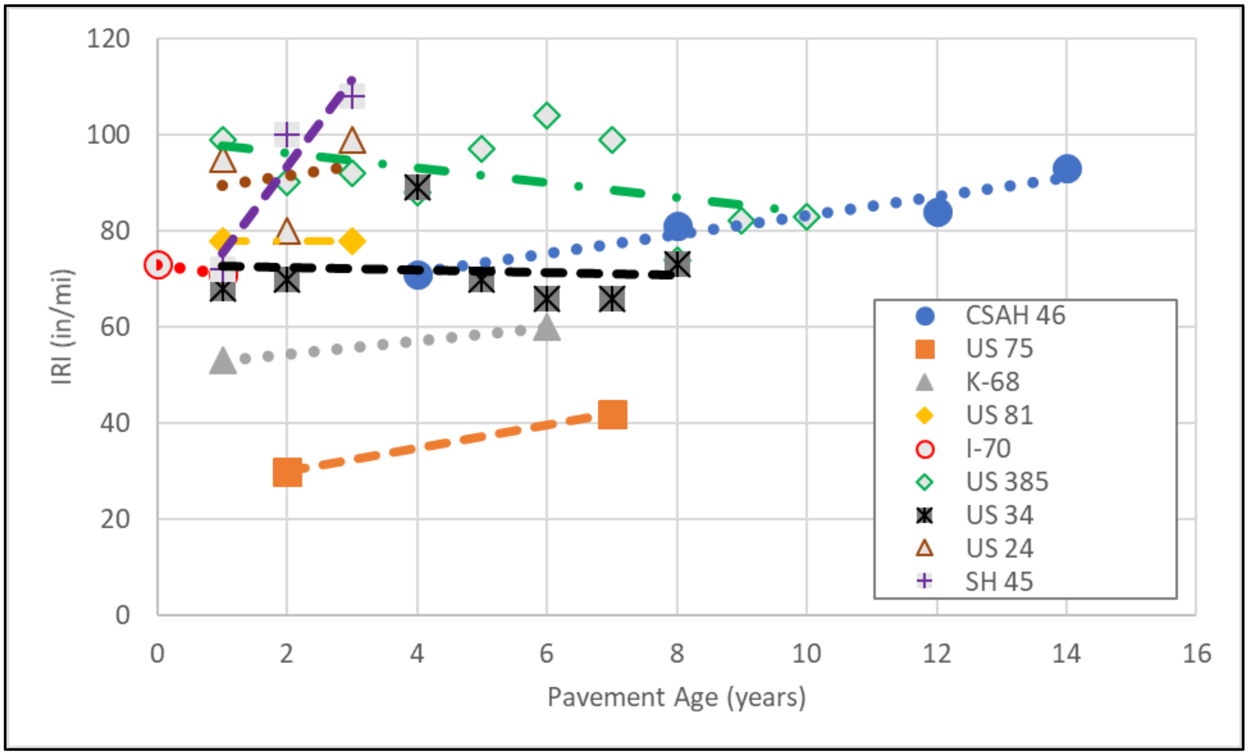

Figure 2.32 Ride quality performance trends

Overall, the ride quality performance of these sections has been good to date. While still at relatively early ages, most projects (with the exceptions of US 24 and SH 45) fell within FHWA's definition of good pavement smoothness (< 95 in/mi) (*21*) as of the last time of data collection. Trends in performance for most projects were relatively stable over time, demonstrating either slight linear increases or approximately stable behavior in terms of IRI over time. The Colorado projects were an exception, demonstrating more volatility in IRI, which is discussed further in section 2.4.4.

Since most of these projects have been built relatively recently and had similar thickness and joint spacing designs, no significant correlations between pavement smoothness and design factors were observed. For the most part, differences in smoothness between projects appeared to be a function of their smoothness after construction based on the first measurements taken at 1-2 years of service.

2.4.3 Faulting

Transverse joint faulting measurements were collected for the projects in Minnesota and Kansas. The average faulting values were very low overall, with no projects exhibiting an average faulting value greater than 0.028 inches. The CSAH 46 project also exhibited excellent joint load transfer performance at 14 years of service life. These findings make sense, as each of these pavements were constructed with dowel bars, which should provide good load transfer performance over the life of the overlay.

2.4.4 Curling behavior

As noted in sections 2.3.6 through 2.3.9, three of the projects built in Colorado exhibited a high degree of curling very soon after construction, and they were highly susceptible to diurnal temperature changes. Thus, diurnal temperature curling may be a cause of the volatility in year-to-year IRI values obtained at these projects. While this volatility makes it somewhat difficult to analyze smoothness trends for these projects, the overall IRI values at the US 24 and SH 45 projects have been notably high at times through their first three years of life, indicating negative impacts of built-in curling on smoothness. The overall smoothness trends of the two oldest Colorado projects, US 385 and US 36, appeared to be fairly stable over time when considering the slope of the linear trendlines in Figure 2.32, but they also exhibited volatility and inconsistency between readings.

Outside of Colorado, the projects in Minnesota and Kansas did not exhibit any signs of curling behaviors and demonstrated stable trends in pavement smoothness. The low curling measurements obtained at the CSAH 46 project in conjunction with the good load transfer performance confirm that curling in the longitudinal direction was not an issue there.

It is not clear whether the FDR layer might be a contributing factor to the curling behavior for the Colorado projects. There are a large number of climatic-, materials-, and construction-related factors that can cause built-in curling. The curling behavior of these projects would need to be compared to those of other concrete pavements and concrete overlays in Colorado to determine whether there might be any unique issue with the concrete overlay of FDR system.

2.4.5 Longitudinal cracking

A cause was not determined for the longitudinal cracking at CSAH 46 or US 385. The cracking on the overlay of FDR on CSAH 46 actually closely resembled longitudinal cracking that also occurred on a conventional COA overlay that was 6 inches thick on a different section of CSAH 46 in Freeborn County west of Albert Lea. An example of the longitudinal cracking on the COA overlay section is pictured in Figure 2.33.

Figure 2.33 Longitudinal cracking on a COA overlay on a different section of CSAH 46 in Freeborn County, MN

While they were two different types of overlays, the projects on CSAH 46 shared two similarities in that they were both constructed in colder temperatures late in the year, and that they both had dowel basket assemblies consisting of three dowel bars placed only in the outer wheel path. With concentrated groups of dowel bars isolated near the edges of the pavement, one possible cause of the cracking could be that stresses developed around those assemblies and propagated over time.

Longitudinal cracking can also develop due to non-uniformity in subbase layers or in pavements with high amounts of curling. That said, there was no evidence of non-uniformity on either project, particularly the overlay of FDR, which involved mixing the underlying materials. Additionally, curling measurements taken on CSAH 46 in fall 2020 were low. Given the amount of curling observed at other projects in Colorado, it is possible that curling was a factor in longitudinal cracking on US 385.

The longitudinal cracking in the concrete overlay of FDR projects on CSAH 46 and US 385 did not appear to be a major concern or to have too much of an effect on ride quality. That said, further investigation may be warranted, and both sections should continue to be monitored over time to ensure the cracking remains stable. The cracking in the 6-inch COA section on CSAH 46 was more severe than in the overlay of FDR, and portions had been cross-stitched as of 2022.

2.4.6 Geotextile separation layer

One project, K-68 in Franklin County, KS, employed the use of a geotextile separation layer between the FDR layer and concrete overlay, similar to a concrete on concrete–unbonded (COC–U) design. While this project has performed well to date, a structural evaluation of this section would be needed to characterize the contributions of the geotextile layer and evaluate how they impact overlay performance.

2.5 FUTURE OPPORTUNITIES AND AREAS OF STUDY

While only a limited number of concrete overlays of FDR have been constructed to date, most of the projects identified in this study appear to be performing well, demonstrating the potential usefulness of this rehabilitation method. As previously mentioned, all of the projects in this study consisted of relatively thick, doweled concrete overlays, analogous to an unbonded concrete overlay design. Most projects were also constructed using unstabilized FDR layers. The two projects that were stabilized (US 75 and I-70 in Kansas) both used cement stabilization.

Given the benefits to design and construction offered by stabilized FDR layers, construction of more concrete overlays of FDR with stabilization would be a good opportunity for future study and analysis. Backcalculation methods could be employed to determine the response of the pavement layers in these types of designs and evaluate how they compare to unstabilized FDR layers. Projects stabilized with different materials (i.e. cement, fly ash, asphalt emulsion) could also be compared to determine the relative performance of different stabilizing agents. Performance of stabilized FDR layers could also be compared to other types of stabilized subbase layers for concrete pavements, such as CTB (cement treated base).

As discussed in Section 1.2.8, a stabilized FDR layer could also be combined with design concepts used for COA–B overlays like short joint spacing and fiber-reinforced concrete to produce thinner concrete overlays of FDR. These designs might not be totally analogous to a COA–B overlay, as the structural design of those projects depends on the bond between the concrete and underlying asphalt, and it is not clear how much a concrete overlay might bond to an underlying FDR layer. That said, thin overlay designs of FDR would be a good subject for future study, and it would be interesting to observe how their performance compares to the thicker designs for these types of overlays that currently prevail in practice.

CHAPTER 3: CONCLUSIONS

While only a small number of concrete overlays of FDR have been constructed on roadways in the US to date, they offer promise as an economical, sustainable, long-lasting pavement rehabilitation solution. Nine projects constructed in three states since 2006 were identified and analyzed to study their performance to date. Some of the key findings of the analysis are as follows:

- The designs of each project were similar to each other and resembled those of conventional unbonded concrete overlays, consisting of relatively thick (7.5+ inch), doweled concrete overlays placed over the FDR layers.
- The FDR layers were between 6 and 12 inches thick and were unstabilized for most projects. Two projects added cement as a stabilizing agent at a dosage rate of 6% by weight.
- FWD backcalculation of the CSAH 46 project with an unstabilized FDR layer determined that the FDR had a substantially higher backcalculated elastic modulus than typical crushed stone subbase layers for concrete pavements. FWD testing was not able to be performed on an overlay placed over a stabilized FDR layer.
- These projects were still relatively early in their service life, but demonstrated good smoothness performance overall. Most projects fell within the FHWA threshold for good performance at the time of last data collection, with measured IRI less than 95 in/mi. IRI trends were mostly stable, with most projects showing a slight increase over time.
- Three projects in Colorado experienced a high degree of curling, which appeared to cause volatility in IRI measurements and high early age IRI readings at two projects. It was not clear whether the curling behavior was related to the design of an overlay placed on FDR.
- Average transverse joint faulting values were very low for all projects that were tested for faulting, less than 0.028 inches, and joint load transfer performance of the CSAH 46 project was excellent after 14 years of service. These findings make sense given that all projects contained doweled joints.

While only a limited number of these types of overlays have been constructed to date, the projects in this study are performing well for the most part. As agencies consider further use of concrete overlays of FDR, it may also be worthwhile for them to explore the use of alternative design options such as greater use a of stabilization for the FDR layers and thinner concrete overlay designs that make use of shorter joint spacings and fiber-reinforced concrete. Overall, concrete overlays of FDR appear to be a promising method to help agencies meet their pavement rehabilitation needs.

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