



STRATEGIES FOR CONCRETE PAVEMENT PRESERVATION

Volume III: Case Studies

June 2026

NOTICE

This is volume III of a three-volume report. The other volumes in this series are *Volume I: Final Report* and *Volume II: Feasibility Assessment and Strategy Selection*.

STRATEGIES FOR CONCRETE PAVEMENT PRESERVATION

Volume I: Final Report

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.093	square meters	m ²
yd ²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi ²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft ³	cubic feet	0.028	cubic meters	m ³
yd ³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m ³	cubic meters	35.314	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in ²

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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CHAPTER 1. INTRODUCTION

Overview and Purpose

Long service life is a hallmark of concrete pavements; outstanding performance for 50 or more years is not uncommon. Such extended performance periods are easily achievable when concrete pavements employ durable materials and are properly designed and constructed. Throughout their service lives, these pavements will not need any significant rehabilitation other than the timely application of appropriate preservation treatments to maintain their functionality (e.g., smoothness, friction, quietness). When applied to the right concrete pavement at the right time, appropriate preservation strategies can be extremely successful at cost-effectively maintaining functionality and extending service life for 50 years or more (Van Dam et al. 2019).

It is common today to design concrete pavements with the expectation that they will develop structural distress (e.g., slab cracking and joint faulting) within a relatively short time frame (e.g., 20 years), triggering the need for full-depth repairs, diamond grinding, slab stabilization, and other treatments to maintain their serviceability. Thus, the application of maintenance treatments and minor rehabilitation are not used to “preserve” the pavement structure but instead are applied to concrete pavements that are already suffering structural distresses and other deficiencies to extend the service life for a relatively short amount of time. Although such treatments restore functionality in the short term, they often do not address the underlying causes of the problems, so the performance of the pavement continues a downward spiral toward inevitable major rehabilitation or reconstruction (Van Dam et al. 2019).

In this series of reports concrete pavement preservation refers to preserving the existing concrete pavement structure to extend its service life for as long as possible, by arresting, greatly diminishing, or avoiding the pavement deterioration process. This can be achieved through three fundamental approaches: (a) designing and constructing pavements that remain structurally adequate and relatively distress-free throughout their service lives (i.e., using long-life concrete pavement), (b) using asphalt or concrete overlays as preservation treatments to maintain the functional performance of the pavement, and (c) maintaining the serviceability of the pavement using concrete pavement restoration (CPR) treatments (Van Dam et al., 2019).

This report documents 11 concrete pavement projects around the U.S. that have successfully demonstrated the application of the three fundamental preservation approaches mentioned above. The information gleaned from these success stories have been used in the development of long-term concrete pavement preservation strategies (presented in Volume II of this report, *Strategies for Concrete Pavement Preservation. Volume II: Feasibility Assessment and Strategy Selection*).

Case Study Projects

Eleven concrete pavement projects around the U.S. that have successfully demonstrated the application of the three fundamental preservation approaches mentioned above have been documented in this report (see figure 1-1). Additional details on each of the 11 case studies are summarized in table 1-1.

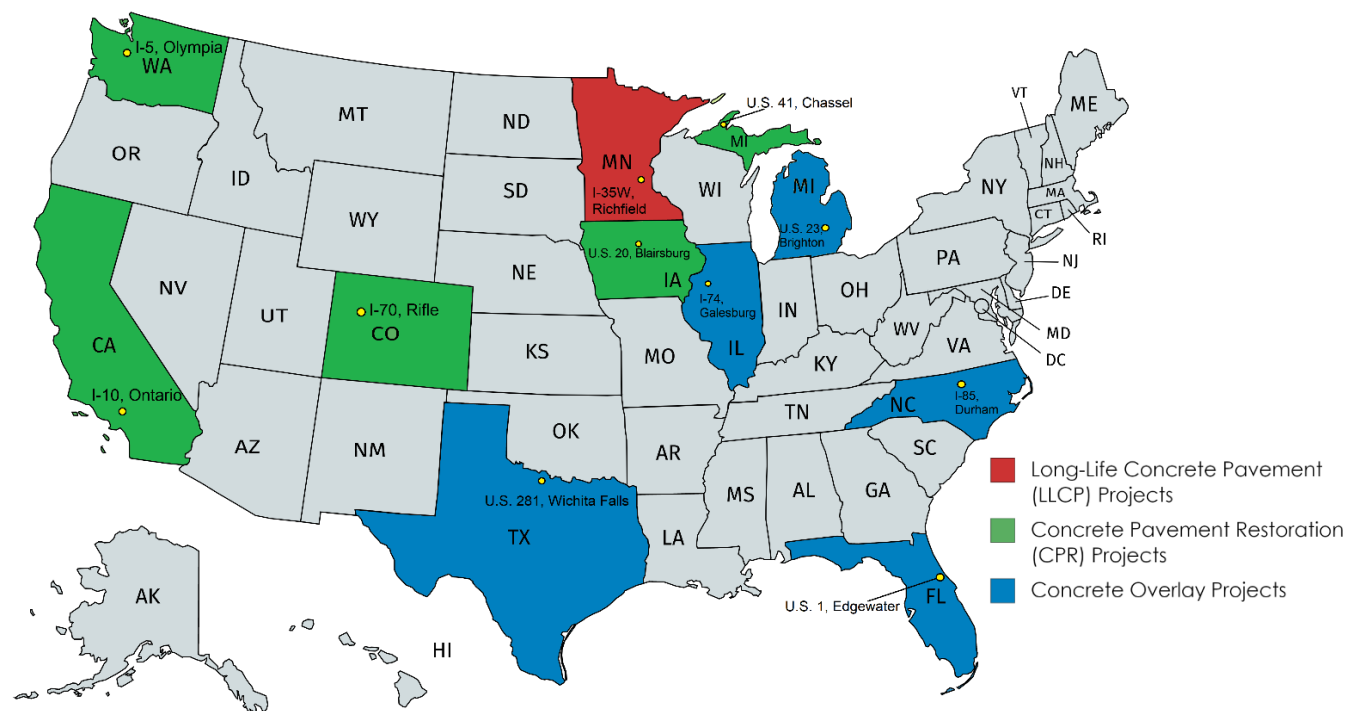


Figure 1-1. Field project locations.

Table 1-1. Details on case study projects.

State	Route	Project Type
Minnesota	I-35W	60-Year long-life concrete pavement design
Michigan	U.S. 23	Unbonded JPC overlay on JRCP
Illinois	I-74	Unbonded CRC overlay on CRCP
North Carolina	I-85	Unbonded JPC overlay on CRCP and CPR: diamond grinding, joint resealing, minor partial-depth repairs
Florida	U.S. 1	Unbonded JPC overlay on asphalt pavement
Texas	U.S. 281	Bonded CRC overlay on CRCP
Iowa	U.S. 20 EB	CPR: Dowel bar retrofit and diamond grinding
Michigan	U.S. 41	CPR: full-depth repairs, crack and joint resealing
Colorado	I-70	CPR: full-depth repairs and diamond grinding
California	I-10	CPR: Diamond grinding, grooving, and slab replacements
Washington	I-5	CPR: load transfer restoration, partial-depth repairs, diamond grinding, and joint resealing

Case Study Contents and Report Organization

Each case study report (presented in chapters 2 through 12) documents the following information:

- Generation project overview
- Original pavement design, materials, and construction details
- Traffic and service conditions
- Maintenance and rehabilitation history
- Present-day condition (based on 2018/2019 site visits)
- Economic analysis
- Key takeaways/lessons learned

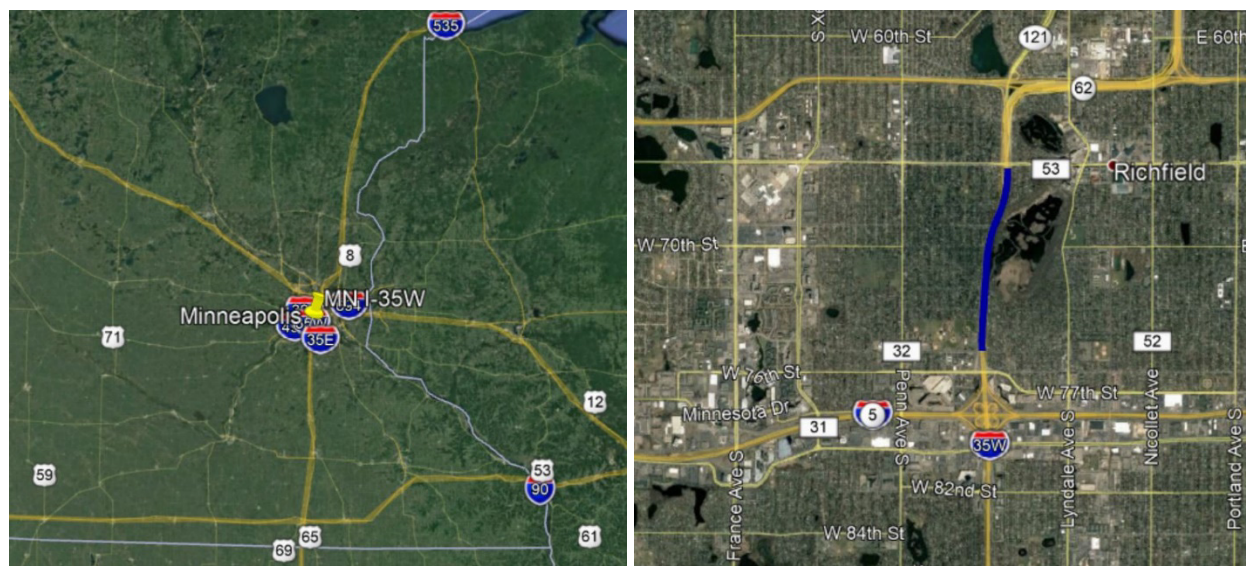
Chapter 13 presents an overall summary of the lessons learned and Appendix A summarizes the approach and procedures used for the economic analysis conducted for each of the 11 case studies.

CHAPTER 2. LONG-LIFE CONCRETE PAVEMENT ON I-35W IN MINNESOTA

Introduction

In 2000, the Minnesota Department of Transportation (MnDOT) constructed its first high performance concrete pavement (HPCP) project on a 1.15-mi (1.85-km) stretch of I-35W between I-494 and State Highway 62 in Richfield, approximately 7 miles south of Minneapolis (see figure 2-1). The project involved the reconstruction of an existing 4-lane divided highway to a 6-lane highway using a 13.5-inch (343 mm) jointed plain concrete pavement (JPCP) with a design life of 60 years. Based on an analysis conducted by MnDOT, a 35-year design resulted in slab thickness of 12.5-inch (318 mm); the additional thickness provided resulted in almost twice the projected service life. The design Equivalent Single Axle Load (ESAL) increased from 57 million [for a 12.5-inch (318 mm) JPCP] to 100 million [for the 13.5-inch (343 mm) JPCP].

Constructing an HPCP involved not only an increase in slab thickness, but also a more robust concrete mixture design. Some of the key mixture design considerations included: lower permeability, higher air content, and a lower water-to-cement ratio. Increased emphasis was placed on aggregate quality (specific gravity, absorption, and alkali-silica reactivity potential) and aggregate gradation. Ground granulated blast furnace slag (GGBFS) (used at a 35 percent replacement for portland cement) was employed to densify the concrete microstructure and reduce permeability. Also, instead of the commonly used epoxy-coated steel dowels, the project employed solid stainless steel and stainless steel-clad dowels for the mainline pavement.



(L) © 2018 Google Earth; Data: Google, Landsat/Copernicus, NOAA, SIO, NOAA, U.S. Navy, NGA, GEBCO. (Added pin for project limits).
(R) © 2018 Google Earth; Data: Google, Landsat/Copernicus, NOAA (Added line for project limits).

Figure 2-1. Project location.

Original Pavement Design, Materials, and Construction

Structural Design

As described above, the original pavement was constructed as a 13.5-inch (343 mm) JPCP on a 4-inch (102 mm) dense-graded granular base over a 12-inch (305 mm) select granular subbase layer. Three 12-ft (3.6 m) wide lanes were paved in each direction with 15-ft (4.6 m) transverse joint spacing. Table 2-1 provides general information and pavement design details.

Table 2-1. I-35W project details.

Key Information	Data
Project Location	I-35W NB/SB, Richfield, MN Milepost 9.127 to 10.276 6-lane divided freeway
Climatic Region	Wet-freeze
Highlighted Project Design Characteristic	Long-life concrete pavement (60-year design)
Initial Construction Year	2000
Rehabilitation Types, Years, and Details	No contracted maintenance or rehabilitation since construction. Minor CPR: Diamond grinding and joint repair is planned for 2020.
Traffic Data	Design Data: 1999: AWDT (Average Weekday Daily Traffic): 118,800 2019 AWDT: 156,240 (predicted) 2018 AADT: 106,939, 6.7% trucks
Slab Design Details	13.5-inch JPCP 15-ft transverse joint spacing Dowels used on mainline pavement: 1.5-inch diameter, 18-inch-long solid stainless steel 1.75-inch diameter, 18-inch-long stainless steel clad 1.5-inch diameter, 18-inch-long stainless steel clad Dowels used on ramp pavement: 1.5-inch diameter, 18-inch-long stainless steel clad 1.5-inch diameter, 18-inch-long epoxy-coated
Base/Subbase	4-inch (102 mm) dense-graded granular base 12-inch (305 mm) select granular subbase
Subgrade	Existing sandy base course, several feet thick
Subdrainage	No open-graded base or edge drains included in design
Shoulder Type and Design	13.5-inch JPCP 15-ft transverse joint spacing Variable width Dowels: 1.5-inch diameter, 18-inch long plastic-coated
Unique/Noteworthy Features	60-year long-life design

Mix Design

Table 2-2 presents a summary of the concrete mix design used for the I-35W project. The coarse aggregate was a low carbonate class C gravel that qualified for full aggregate quality bonus. “Safety Grit” was added by the contractor to the coarse rock and sand mix to meet the denser gradation specified (FHWA 2006).

Table 2-2. I-35W concrete mix design (Burnham, Izevbekhai, and Rangaraju 2006).

Material	Quantity (per cu. yd.)
Cement	384 lbs.
Slag	206 lbs.
Coarse Aggregate—1.5-inch (38 mm)	575 lbs.
Coarse Aggregate—0.75-inch (19 mm)	800 lbs.
Coarse Aggregate—0.50-inch to #4 (12.5 to 4.75 mm))	350 lbs.
Fine Aggregate—ASTM C 33	950 lbs.
Fine Aggregate—Safety Grit (2.36 to 9 mm)	440 lbs.
Water Reducer	30 oz.
Water	220 lbs.
w/cm Ratio	0.37

The concrete mixtures were produced in parallel from two batch places at the same site, using the same mix design. The target fresh concrete air content was 6.5 ± 1.5 percent. The measured plastic air contents exhibited high variability—ranging from less than 5 percent to over 10 percent. Price reductions were assessed for failing air contents after the project was completed. Hardened concrete air content was determined using ASTM C457 and the results ranged from 6.7 to 15.7 percent, with an average value of 10.6 percent. Around 60 percent of the hardened air content measurements (of a total of 26 measurements) were over 10 percent and only one measurement fell below the 7 percent minimum specified. Further analysis indicated the prevalence of small air voids with diameters less than 100 microns and few air voids in the 150 to 250 micron range. It was theorized that volumetric air meters used for fresh concrete air content measurements are not efficient in measuring small bubbles, which could potentially explain the differences in the fresh and hardened concrete air contents (FHWA 2006).

Construction

The original paving layout consisted of three 12-ft (3.6-m) wide lanes in each direction with transverse joints spaced at 15-ft (4.6-m). This included inside and outside shoulders that were constructed following the same design as the mainline since they were expected to carry traffic during construction. The original sand base was left in place for the new pavement and was capped with 12-inches (305-mm) of select granular material, which was then topped with 4-inches (100-mm) of Class 5 dense-graded granular base to provide a uniform platform for paving (FHWA 2006).

Based on historical experience in Minnesota, epoxy-coated dowel bars generally do not last beyond 25 years without exhibiting significant amount of corrosion (Rettner 1999). Therefore, MnDOT specified high-quality dowels for use on the I-35W HPCP project. The original plans called for the use of 1.5-inch (38-mm) solid stainless-steel and stainless-steel clad dowel bars, and 1.75-inch (44-mm) stainless-steel clad bars. The control section used the 1.5-inch (38-mm) stainless-steel clad dowels, which was less expensive when compared to the solid stainless-steel

option. To compare the performance between traditional epoxy-coated and stainless-steel clad dowels, the NW exit ramp to CSAH 53 planned to employ 1.5-inch (38-mm) epoxy coated dowels, while the rest of the ramps were to use the 1.5-inch (38-mm) stainless-steel clad bars. All the dowel bars specified were 18-inches (457-mm) long, spaced 12-inch (300-mm) center-to-center along the transverse joints. Table 2-3 summarizes the original dowel placement plan.

**Table 2-3. I-35W original dowel placement plan
(Burnham, Izevbekhai, and Rangaraju 2006).**

Dowel Type	Diameter	Installation Locations
Stainless Steel Clad	1.5-in (38-mm)	Majority of mainline, shoulders, and ramps
	1.75-in (44-mm)	35 Joints, mainline and shoulders
Solid Stainless Steel	1.5-in (38-mm)	35 Joints, mainline and shoulders
Epoxy-Coated	1.5-in (38-mm)	NW exit ramp to CSAH 53

Securing the supply of stainless-steel clad dowels became a challenge for the construction team, and by spring 2000, close to the time of initial construction, only 75 percent of the total required stainless-steel clad dowels was available (FHWA 2006). To avoid project delays, plastic-coated dowels were used in the shoulders, and the available stainless-steel clad bars were used in some parts of the northbound mainline lanes; while the southbound lanes were paved using solid-stainless steel dowels. An example of the dowel configuration during the original construction is presented in figure 2-2.



© Minnesota DOT

Figure 2-2. Dowel bar configuration used in the I-35W project.

Paver Vibration Monitoring

MnDOT specified the use of internal electronic vibrator monitors to prevent excessive vibration during the slipform paving process. Paver track speed and vibrator operating frequencies were required to be submitted on a daily basis (Burnham, Izevbekhai, and Rangaraju 2006).

Surface Texture

Due to the moratorium imposed in 1998 on transverse tining of concrete pavements due to excessive noise levels, MnDOT used an artificial turf drag texture for the I-35W project, which resulted in an average texture of 0.8 mm, measured immediately after construction using a volumetric sand patch test (ASTM E965) (Burnham, Izevbekhai, and Rangaraju 2006). Figure 2-3 shows a close-up of the surface texture.



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Figure 2-3. Close-up of surface texture for the I-35W project.

Curing

A poly-alpha-methylstyrene membrane curing compound was used on the project. Under MnDOT specifications, these curing compounds are required to exceed 42 percent total solids by mass and provide a minimum reflectance of 65 percent (measured by ASTM E1374) in 72 hours.

Opening to Traffic

The minimum flexural strength MnDOT specified on the project for opening to traffic was 350 psi (2.5 MPa).

Service Conditions

Annual traffic experienced a 0.3 percent average annual increase between 1998 and 2017, with the exception between 2006 and 2010 when overall traffic experienced a steep decline. In terms of heavy traffic, MnDOT Heavy Commercial Annual Average Daily Traffic (HCAADT) trend for I-35W shows the volume of heavy commercial vehicles stayed somewhat constant during the initial years of the project, but a significant drop was observed between 2013 and 2017. Traffic trends for the I-35W project are shown in figure 2-4.

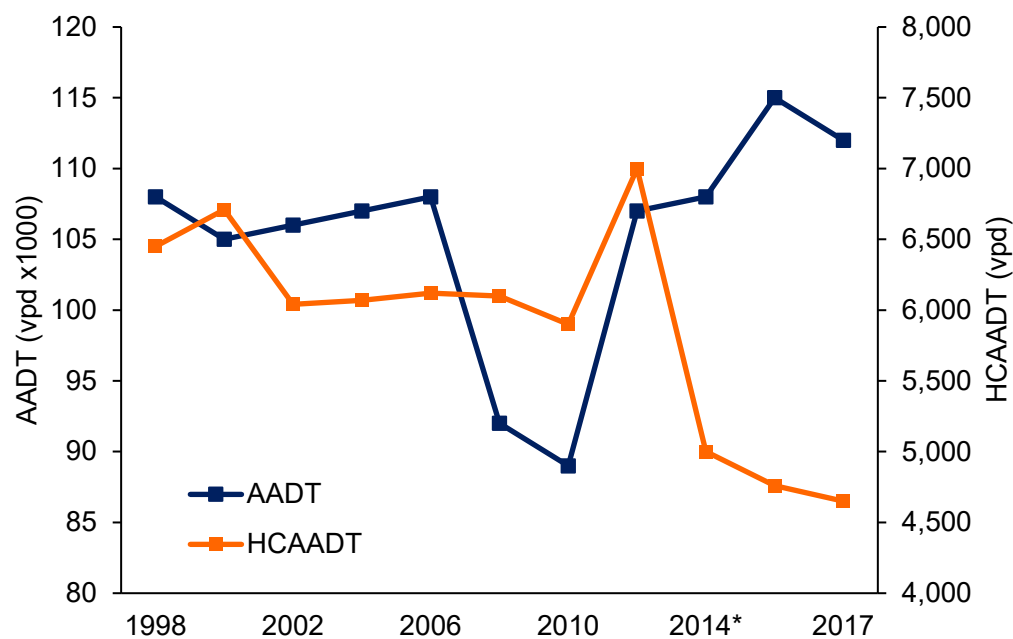


Figure 2-4. Traffic trends for I-35W project (Data source: MnDOT, Site: 10801).

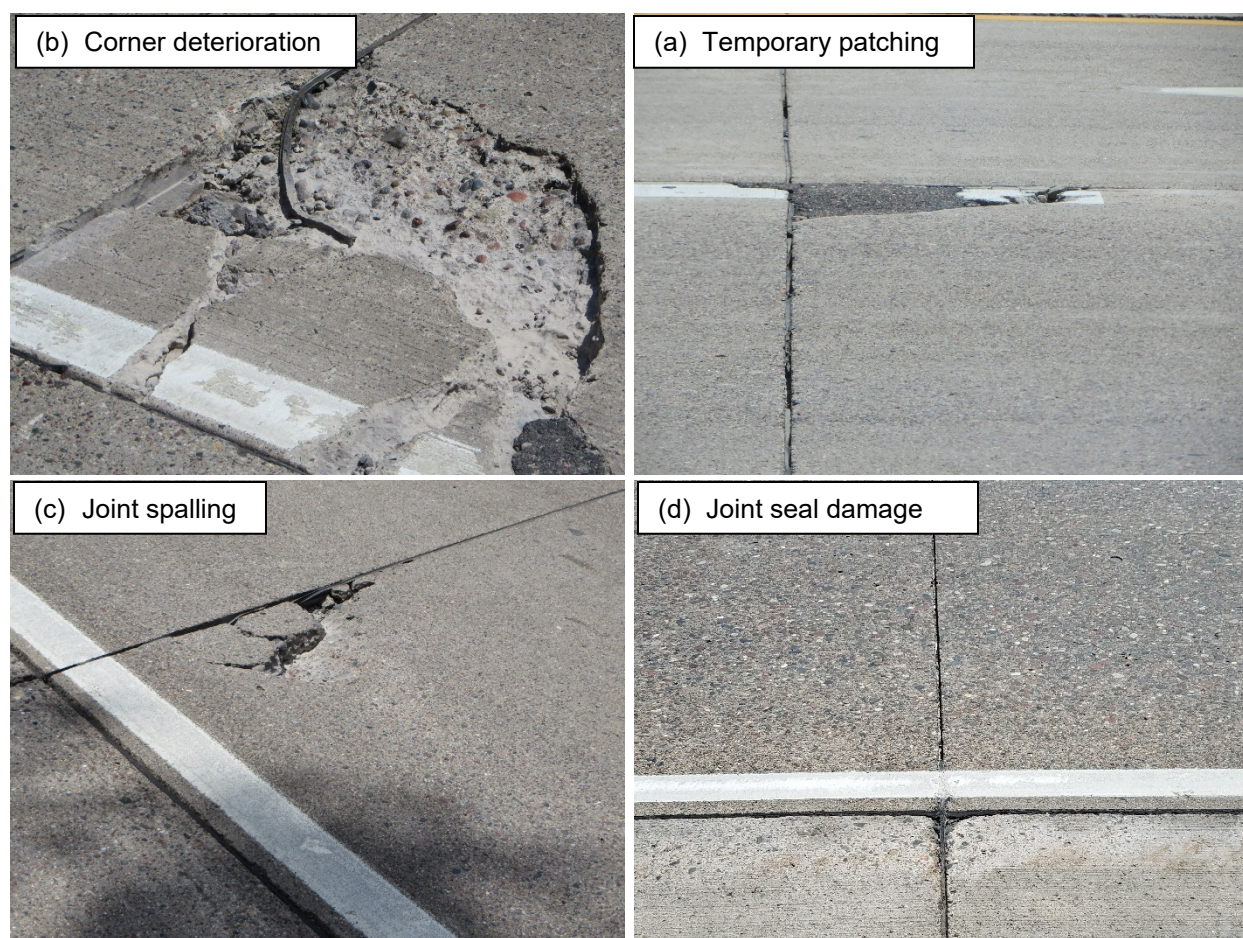
Project Characteristics

A field visit during the summer of 2018 revealed that the entire section was still in generally good condition, with sporadic presence of joint and corner deterioration, with temporary patching to address some of these distresses. The joint sealant appeared to be missing or damaged at a few locations. Figure 2-5 shows an overview of I-35W, and figure 2-6 presents some photographs of typical distresses observed.



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Figure 2-5. Overview of I-35W pavement.



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Figure 2-6. Some examples of distresses observed on I-35W.

Pavement Performance

MnDOT collects pavement condition data using digital inspection vehicles, which is used to determine three performance measures: Ride Quality Index (RQI), Surface Rating (SR), and Pavement Quality Index (PQI) and these indices are summarized in table 2-4.

Table 2-4. Pavement condition indices used by MnDOT (MnDOT 2015).

Index	Pavement Attribute Measured by Index	Rating Scale
Ride Quality Index (RQI)	Pavement Roughness	0.0 – 5.0
Surface Rating (SR)	Pavement Distress	0.0 – 4.0
Pavement Quality Index (PQI)	Overall Pavement Quality	0.0 – 4.5

RQI is obtained by correlating the International Roughness Index (IRI) value measured on the left wheel path to the ride quality experienced by the road user. SR is obtained by a weighted average of the distresses affecting the pavement, which are gathered by performing a detailed video inspection of the pavement to identify the type, quantity and severity of distresses that are present. PQI is the result of the square root of the product between RQI and SR (MnDOT 2015).

Figure 2-7 shows time-series IRI and RQI trends for the I-35W project. The IRI values have remained fairly constant since the original construction. Between 2001 and 2017, the change in IRI values for the northbound lanes was noted to be 12 inches/mile (189 mm/km) and for the southbound lanes, it was noted to be 7.8 inches/mile (123 mm/km).

Based national pavement performance measures reporting requirements ([HPMS](#)), the southbound lanes are rated to be in “Good” condition ($IRI < 95$ in/mi) and the northbound lanes are reported to be in “Fair” condition ($IRI < 95$ in/mi), based on the 2017 IRI values reported.

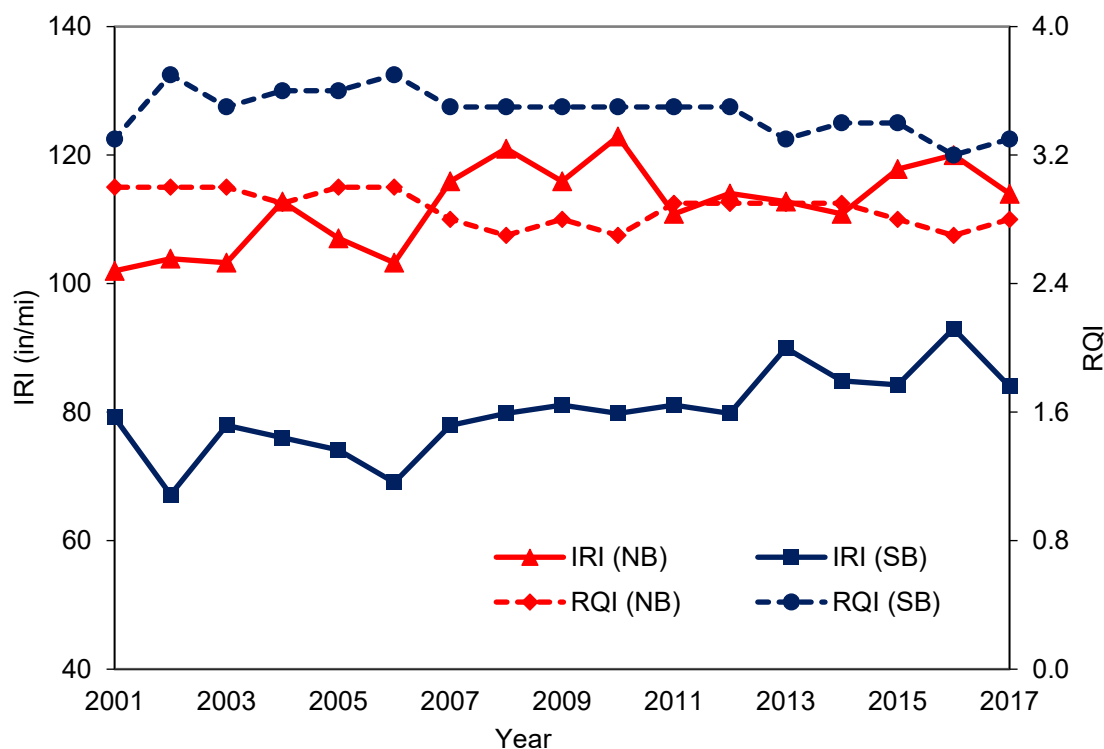


Figure 2-7. Surface roughness progression for I-35W (Data source: MnDOT).

The second component of MnDOT’s condition rating procedure involves the assessment of distresses observed on the pavement surface to calculate the SR. The SR for the southbound lanes of I-35W was calculated to be 4.0 between 2002 and 2016, with the first drop in 2017 when the rating was calculated to be 3.99. The SR for the northbound lanes experienced the first drop from a 4.0 rating 2014 when the SR value was calculated to be 3.8, and in 2014 it dropped to 3.7. However, in 2017, the value increased to 3.99, presumably due to some localized maintenance and repair actions performed.

The PQI values (see figure 2-8) for both the northbound and southbound lanes have remained fairly constant since 2002. The southbound lanes are in slightly better condition than the northbound lanes, primarily due to the higher smoothness levels (as noted in figure 2-7). A value of 3.0 is the MnDOT’s minimum PQI condition threshold for its principal arterial system.(MnDOT 2019). The I-35W HPCP has maintained satisfactory condition levels for almost 20 years with minimal maintenance. This highlights the potential of HPCP as a road construction alternative for high volume urban areas where periodic maintenance and

rehabilitation activities can potentially create significant traffic disruption and negatively impact the region's economic and natural environments.

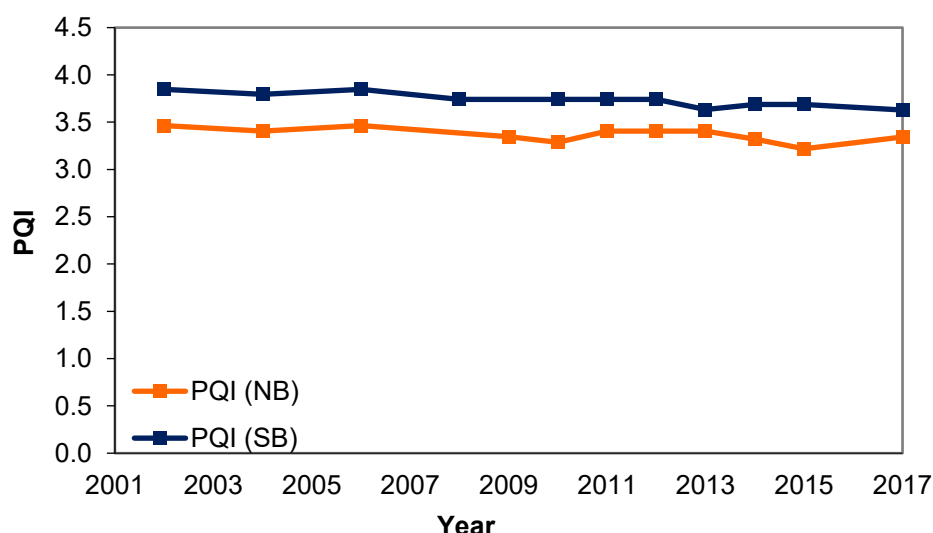


Figure 2-8. Historic PQI progression for I-35W (Data source: MnDOT).

Economic Analysis

Table 2-5 summarizes the agency costs for the initial construction and pavement rehabilitation activities that are proposed and assumed to be performed over the years on the stretch of I-35W documented in this case study. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 2-5. Activity and cost data for I-35W.

Year	Activity	Activity Detail	Cost
2000	HPCP Construction	13.5-in Concrete Pavement	\$3,690,000*
2020 ¹	Planned CPR	Minor CPR plus DG and resurfacing SPP	\$181,179*
2035 ²	CPR (assumed)	Minor CPR plus DG and resurfacing SPP	\$362,358**
2050 ²	CPR (assumed)	Minor CPR plus DG and resurfacing SPP	\$724,716**
2060	End-of-Life	Assumed Pavement End of Service	NA

*Cost shown in then-current dollars, from MnDOT contract documents

** 2035 costs assumed to be x2-times the 2020 CPR costs, 2050 costs assumed to be x4-times the 2020 CPR costs

¹Minnesota DOT proposed CPR activity

²Assumed CPR activity

A 60-year analysis period was used (2000-2060) and the impacts of four different discount rates (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 2-5 are summarized below:

- 2000 HPCP: 60 years (end-of-service: 2060)
- 2020 CPR: 15 years (end-of-service: 2035)
- 2035 CPR: 15 years (end-of-service: 2050)
- 2050 CPR: 10 years (end-of-service: 2060)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 2-6 shows the construction costs converted to 2000 dollars.

Table 2-6. Agency costs for the I-35 project converted to 2000 dollars.

Year	Activity	2000 Dollars
2000	HPCP Construction	\$3,690,000
2020	CPR	\$92,132
2035	CPR	\$184,264
2050	CPR	\$368,528
2060	Remaining Value (2000 HPCP Construction)	(\$0)
2060	Remaining Value (2050 CPR)	(\$0)

Figure 2-9 shows the expenditure stream diagram for the I-35 project using the four different discount rates used in the analysis

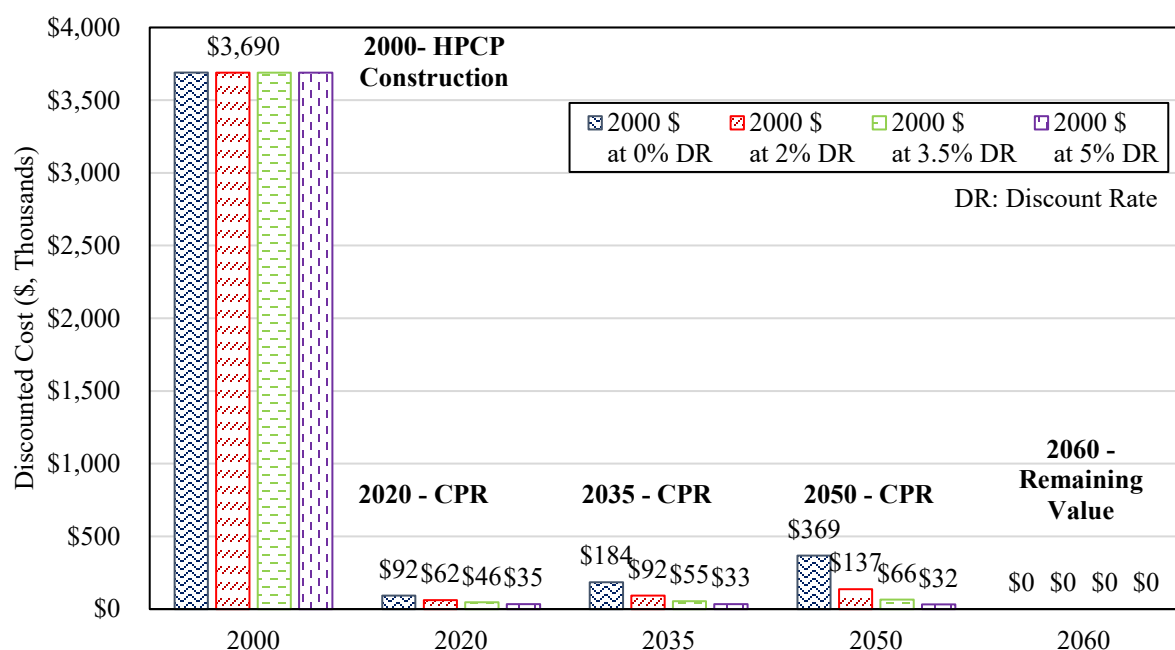


Figure 2-9. Expenditure stream diagram for I-35.

Table 2-7 summarizes the LCCA results for each discount rate considered. As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 12.6 percent, and the equivalent uniform annual cost (EUAC) increases by 177 percent. This is graphically represented in Figure 2-10.

Table 2-7. LCCA summary for I-35W.

0% Discount Rate				
LCCA Element	2000 Dollars		2019 Dollars	
2000 Original	PV =	\$3,690,000	PV =	\$6,582,422
2020 CPR	PV =	\$92,132	PV =	\$164,350
2035 CPR	PV =	\$184,264	PV =	\$328,701
2050 CPR	PV =	\$368,528	PV =	\$657,401
Remaining Value (2000 HPCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2050 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$4,334,924	NPV =	\$7,732,887
	EUAC =	\$72,249	EUAC =	\$128,881
2% Discount Rate				
LCCA Element	2000 Dollars		2019 Dollars	
2000 Original	PV =	\$3,690,000	PV =	\$6,582,435
2020 CPR	PV =	\$62,002	PV =	\$110,603
2035 CPR	PV =	\$92,137	PV =	\$164,359
2050 CPR	PV =	\$136,918	PV =	\$244,243
Remaining Value (2000 HPCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2050 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$3,981,058	NPV =	\$7,101,640
	EUAC =	\$114,527	EUAC =	\$204,300
3.5% Discount Rate				
LCCA Element	2000 Dollars		2019 Dollars	
2000 Original	PV =	\$3,690,000	PV =	\$6,582,435
2020 CPR	PV =	\$46,302	PV =	\$82,597
2035 CPR	PV =	\$55,275	PV =	\$98,603
2050 CPR	PV =	\$65,986	PV =	\$117,710
Remaining Value (2000 HPCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2050 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$3,857,563	NPV =	\$6,881,344
	EUAC =	\$154,644	EUAC =	\$275,864
5% Discount Rate				
LCCA Element	2000 Dollars		2019 Dollars	
2000 Original	PV =	\$3,690,000	PV =	\$6,582,435
2020 CPR	PV =	\$34,724	PV =	\$61,942
2035 CPR	PV =	\$33,405	PV =	\$59,590
2050 CPR	PV =	\$32,137	PV =	\$57,328
Remaining Value (2000 HPCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2050 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$3,790,266	NPV =	\$6,761,295
	EUAC =	\$200,233	EUAC =	\$357,187
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost				

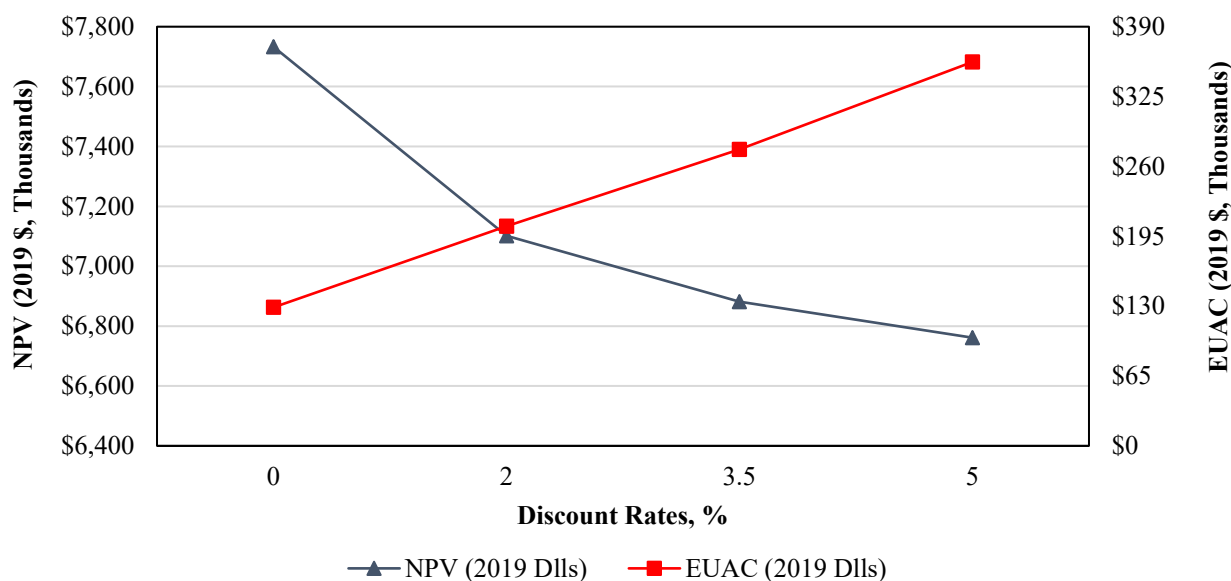


Figure 2-10. NPV and EUAC (2019 Dollars) vs. discount rate.

Lessons Learned

In 2000 the MnDOT constructed its first HPCP project on a 1.15-mi stretch of I-35W south of Minneapolis, MN. The goals were to minimize maintenance needs during the pavement life cycle and to provide better ride quality. Long-life concrete pavement designs involve much more than just increasing the slab thickness. It involves the use of high-quality materials, durable mix designs, more stringent specifications, and good foundation and drainage designs. After being subjected to high traffic volumes over 19 years with little to no maintenance, the HPCP shows acceptable levels of serviceability and smoothness. Lessons learned from this study are summarized below.

- Constructing an LLCP involves thicker slabs and more stringent specifications for the concrete mixture and dowel bars.** Some of the key mixture design considerations included: lower permeability, higher air content, and a lower water-to-cement ratio. Increased emphasis was placed on aggregate quality (specific gravity, absorption, and alkali-silica reactivity potential) and aggregate gradation. Ground granulated blast furnace slag (used at a 35 percent replacement for portland cement) was employed to densify the concrete microstructure and reduce permeability. Instead of the commonly used epoxy-coated steel dowels, the project employed solid stainless steel and stainless steel-clad dowels for the mainline pavement.
- LLCP significantly reduces future maintenance needs.** The construction of an HPCP on I-35W enabled MnDOT to offer acceptable levels of serviceability and smoothness, as measured by distress surveys and IRI measurements, without having to invest in maintenance projects for 19 years. As seen from the economic analysis conducted, the future maintenance costs for the I-35W long-life pavement are expected to be in the range of 3 to 15 percent of the initial construction cost, depending on the discount rate selected.

CHAPTER 3. UNBONDED PCC OVERLAYS ON U.S. 23 IN MICHIGAN

Introduction

In 1999 and 2001, the Michigan Department of Transportation (MDOT) constructed a concrete unbonded overlay (UBOL) on a severely deteriorated, 10.5-mile (16.9-km) stretch of U.S. 23 near Hartland, Michigan (see figure 3-1). The original jointed reinforced concrete pavement (JRCP), built in 1959, had deteriorated to the point where rideability suffered and several asphalt concrete overlays (ACOLs) had been placed on large sections of the corridor as stop-gap measures. At this location, U.S. 23 is a four-lane divided highway with two 12-ft (3.7-m) wide lanes in each direction.



Figure 3-1. U.S. 23 project location.

Original Pavement Design, Materials, and Construction

Structural Design

Based on available record drawings, the original pavement was constructed in 1959 as a 9-inch (229-mm) JRCP on a 4-inch (102-mm) aggregate base and a 10-inch (254-mm) sand subbase. The transverse joints were doweled and spaced at 100-ft (30.5-m) intervals. Table 3-1 provides general information and pavement design details for the original and rehabilitated concrete pavements.

Paving Materials

The 1957 Michigan State Highway Department concrete specification (MSHD 1957) required a 208-day compressive strength of 3,500 psi (24.1 MPa) and 5.5 percent entrained air. Other specific details of the concrete mixture proportions were not available.

Table 3-1. U.S. 23 project details.

Key Information	Data
Project Location	U.S. 23 NB/SB, Hartland, MI Milepost 67.25 to 77.90 (M-59 to Genesee County line) Rural 4-lane Highway
Climatic Region	Wet-freeze
Highlighted Project Design Characteristics	Unbonded Concrete Overlay (two overlays of similar design)
Initial Construction Year	1959 (underlying 9-inch JRCP)
Rehabilitation Types, Years and Details	1999: 8-in JPCP UBOL Paving with 1-in AC separation layer at north section 2001: 7-in JPCP UBOL Paving with 1-in AC separation layer at south section
Traffic Data	2017 AADT: 46,394 – 57,947 (3,896 – 4,405 commercial)
Slab Design Details (original 1959 construction)	9-inch JRCP (wire mesh, 0.16 % longitudinal steel) 99-ft transverse contraction joint spacing with weakened plane joints every 10 feet 1.25-inch diameter by 15-inch length dowel bars at 12-inch spacings
Slab Design Details (1999/2001 UBOL) [Data Source: MDOT (1998)]	8-inch JPCP (north section, 1999) 7-inch JPCP (south section, 2001) 13-ft transverse joint spacing 1.5-inch diameter epoxy-coated dowel bars placed on 11-inch centers (with first bar at 4 inches from slab edge).
Separator Layer (between UBOL & existing slab)	1-inch MDOT 13A asphalt concrete
Base/Subbase (original 1959 construction)	4-inch aggregate base 10-inch sand subbase
Subgrade	Unknown
Subdrainage	Edge drains added as part of the UBOL construction
Shoulder Type and Design	Original: 6-inch compacted aggregate With UBOL: Tied PCC
Unique/Noteworthy Features	1999 & 2001: Localized Reconstruction Beneath Bridge Overpasses • 11-inch JPCP (north section, 1999) • 9.5-inch JPCP (south section, 2001) • 13-ft joint spacing • 1.5-inch diameter, epoxy-coated dowel bars • 4-inch open graded drainage course (north section, 1999) • 4-inch aggregate base (south section, 2001) • Existing sand base/subbase • JPCP shoulders

Rehabilitation Pavement Design, Materials, and Construction

Structural Design

After 40 years of service, the original 1959 JRCP was rehabilitated with UBOLs in 1999 (north portion) and 2001 (south portion). The UBOLs were constructed as an 8-inch (203-mm) jointed plain concrete pavement (JPCP) in the north portion and a 7-inch (178-mm) JPCP in the south portion; both overlay designs were placed over a nominal 1-inch (25-mm) asphalt concrete separation layer. To maintain vertical clearances, localized areas of full-depth reconstruction were performed under bridge overpasses featuring an 11-inch (280-mm) JPCP on an open-graded drainage course (north portion, 1999) and 9.5-inch (241-mm) JPCP on an aggregate base (south portion, 2001). General design details for the UBOLs are summarized in table 3-1.

Repair of Existing Concrete

Prior to the placement of the UBOLs, intermittent repair of concrete joints, cracks, and patches was performed to provide a suitable paving platform. No details on the specific types or quantities of repair were available from the project records. Figure 3-2 illustrates the typical concrete condition prior to the placement of the UBOLs.

Construction

The UBOLs were constructed on a nominal 1-inch (25-mm) asphalt concrete separator layer. In sections under overpass bridges, a 4-inch (102-mm) open-graded drainage course (north portion, 1999) and 4-inch (102-mm) aggregate base (south portion) were installed over existing sand subbase and served as a base layer for new JPCP. Edge drains were installed at the edge of the existing concrete slab and consisted of a vertical column of MDOT 34R, open-graded aggregate and horizontal drain pipe that was daylighted to the ditch. A 2011 forensic investigation revealed that the trench drains were capped with a dense-graded interlayer in some locations (Hansen and Liu 2013).

Traffic and Service Conditions

In addition to typical AASHTO Class 9 trucks, U.S. 23 experiences heavy loading year-round from the construction and material handling industry. Michigan has one of the nation's highest allowable legal truck load limits (up to 164,000 lb [74,400 kg] on 11 axles). Figure 3-3 shows examples of typical haul trucks and axle configuration observed on U.S. 23.



© 1999 MDOT Video Log

Figure 3-2. Typical condition of JRCP prior to UBOL.

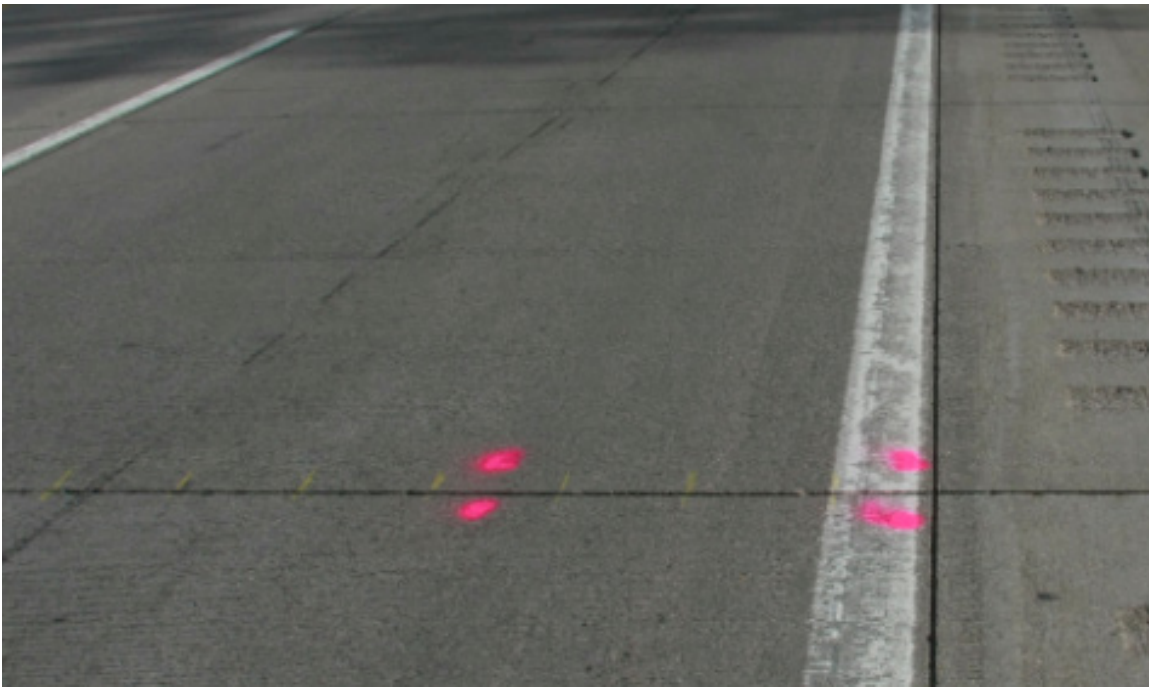


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Figure 3-3. Examples of haul trucks on U.S. 23.

Field Visit

A 2018 site visit revealed the following distresses in the overlay sections: a few low- and medium-severity transverse cracks, minor surface wear, joint seal deterioration, and isolated spalling. In comparison, the reconstructed sections passing under bridges exhibited more advanced distresses. Figures 3-4 and 3-5 present examples of the pavement condition in 2011 while figures 3-6 and 3-7 show the condition in 2018.



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Figure 3-4. UBOL concrete surface in 2011.



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Figure 3-5. Low- to medium-severity transverse cracking in 2011.



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Figure 3-6. View of the existing UBOL in 2018.



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Figure 3-7. Example of medium-severity, mid-panel crack (left image) and corner break (right image) in 2018.

Note: These distresses were isolated to a few areas where poor drainage was documented.

Pavement Performance and Maintenance History

MDOT uses a distress index (DI), defined as the total accumulated distress point value, normalized to 0.1-mi sections, to characterize pavement distress. The scale for DI is as follows (MDOT 2017):

- Good = 0 – 25
- Fair = 26 – 49
- Poor $\geq$ 50

Further, MDOT characterizes ride quality using the IRI, applying the following scale (MDOT 2017):

- Good = 0 – 94 inches/mi (0 – 1484 mm/km)
- Fair = 94 – 170 inches/mi (1484 – 2683 mm/km)
- Poor > 170 inches/mi (>2683 mm/km)

The first available IRI and distress index (DI) data for this section of U.S. 23 are from 1993 (MDOT records), 33 years after construction of the original JRPC. MDOT video log shows that asphalt concrete overlays were placed throughout much of the project (as a stop-gap measure) between 1997 and 1999. No other maintenance records were available.

North Section

Figure 3-8 depicts time-series IRI and DI data (north section) from 1993 (pavement age of 33 years) until 2017 (original pavement age of 57 years, UBOL age of 18 years), while table 3-2 summarizes performance through 2017.

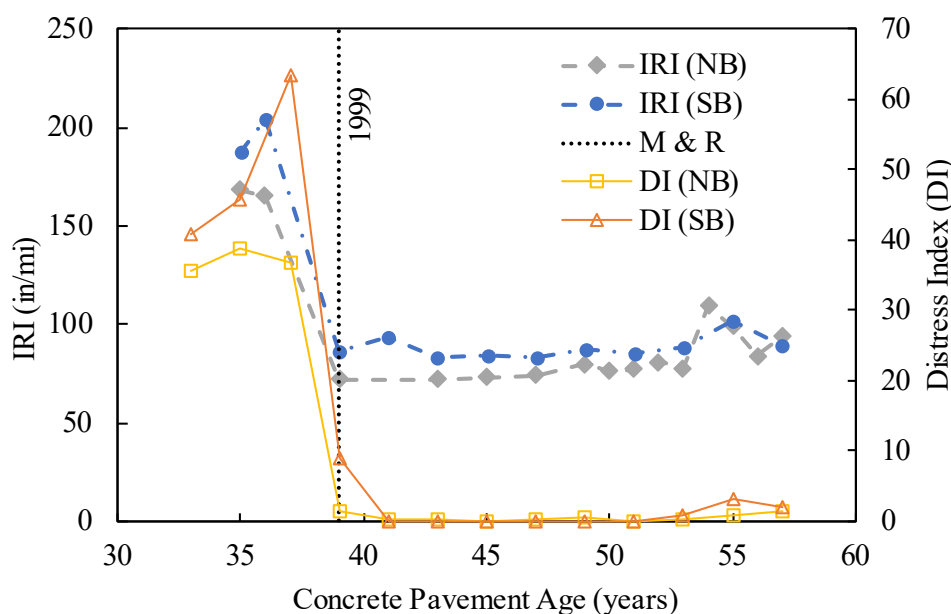


Figure 3-8. Time-series IRI and distress index for north section (data from MDOT records).

Table 3-2. Change in IRI and DI for the north section (data from MDOT records).

Performance Measure	Direction	Prior to PCC Overlay ¹	1999/2001	2017 Data
IRI (in/mi)	Northbound	165	72	94
	Southbound	204	86	89
DI	Northbound	37	0	2
	Southbound	64	0	2

¹ Includes temporary asphalt concrete overlay areas.

South Section

Figures 3-9 and 3-10 present time-series IRI and DI data (south section) from 1993 (pavement age of 33 years) until 2017 (pavement age of 57 years) and table 3-3 summarizes performance through 2017. MDOT provided data for two sub-sections within the south section of the project, separated by Crouse Road. The reduction in DI, prior to the UBOL, appeared to be related to the placement of a stop-gap asphalt concrete overlay. The MDOT video log from 1999 showed asphalt concrete overlay on most of the southbound lanes between Crouse Road and M-59.

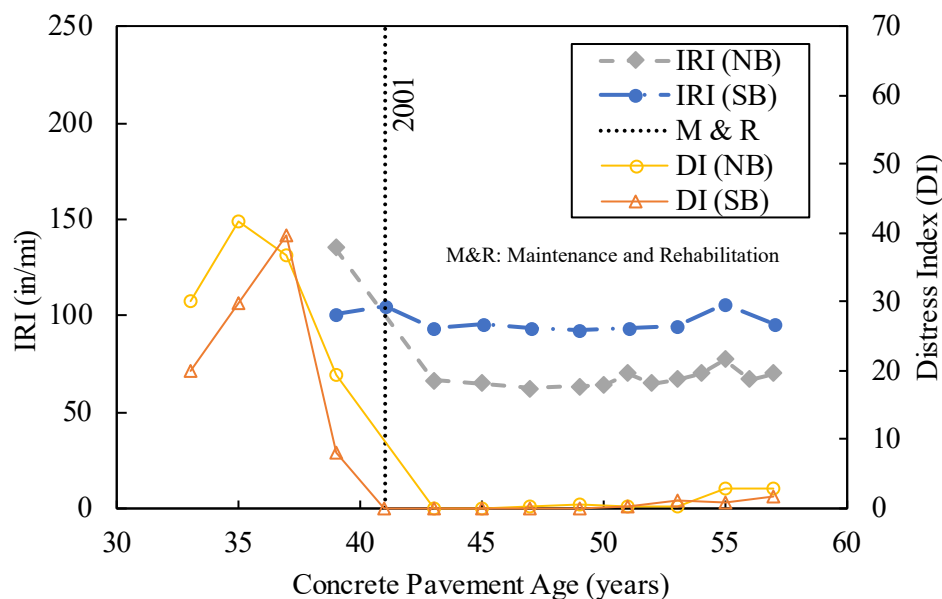


Figure 3-9. Time-series IRI and distress index for south section, M-59 to Crouse Road (data from MDOT records).

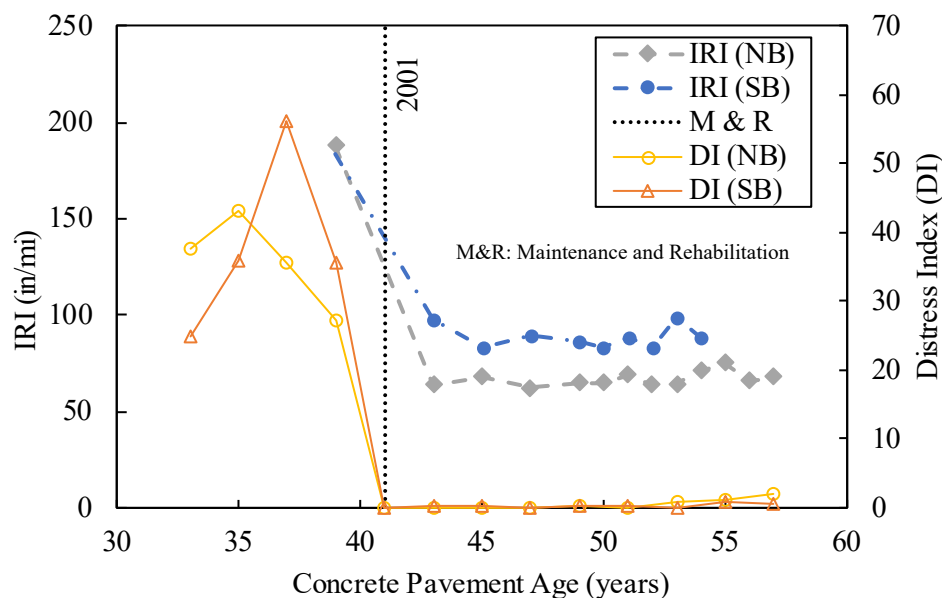


Figure 3-10. Time-series IRI and distress index for south section, Crouse Road to Fausett Road (data from MDOT records).

Table 3-3. Change in IRI and DI for the south section (data from MDOT records).

Performance Measure	South Section	Direction	Prior to PCC Overlay ¹	1999/2001	2017 Data
IRI (in/mi)	M-59 to Crouse Rd	Northbound	135	66	70
		Southbound	101	93	96
	Crouse Rd to Fausett Rd	Northbound	183	64	68
		Southbound	183	97	88
DI	M-59 to Crouse Rd	Northbound	19	0	3
		Southbound	8	0	2
	Crouse Rd to Fausett Rd	Northbound	27	0	1
		Southbound	36	0	2

¹ Includes temporary asphalt concrete overlay areas.

As expected, IRI and DI dropped significantly after the construction of the unbonded overlay. More importantly, these data support the 2018 visual observations that the overlay is still performing very well after 17 – 19 years of service. In fact, average IRI is still below 100 in/mile (1578 mm/km) over the entire corridor despite large differences in pre-construction IRI (101 – 204 in/mile [1594 – 3220 mm/km]). DI values are all below 3, an indication that very minimal distresses have developed over the service life.

2011 Forensic Investigation (Hansen and Liu 2013)

The UBOs on U.S. 23 were one of eight Michigan case studies highlighted in a 2013 research report prepared by Hansen and Liu and were one of three projects that exhibited good performance, noting that no slabs in the northbound direction were cracked as of 2011. Forensic investigation showed an asphalt concrete cap in some sections that blocked the path to the underdrains (see figure 3-11). Records note periodic outlet cleaning but no pavement maintenance.

One of the conclusions from the study was that good drainage was essential to a long-lasting unbonded concrete overlay. This included planning for drainage during design and proper construction. Joint faulting was measured in the outer wheel-path and transverse joint edge and was found to be insignificant over a 500-foot (152 m) test section. It was concluded that pumping was minimal on this project despite the asphalt cap found over the edge drain (Hansen and Liu 2013).

As a note, drainage outlets were inspected as part of the 2018 site visit. In general, it was observed that the outlets were not being maintained, and some were clogged by vegetation as shown in figure 3-12. In a few isolated locations, the lack of drainage was linked to evidence of pumping and slab cracking as shown in figure 3-13.



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Figure 3-11. Example of asphalt concrete cap over edge drain.



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Figure 3-12. Drainage outlet blocked by vegetation after being exposed with a shovel.



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Figure 3-13. Isolated slab cracking linked to poor drainage.

Economic Analysis

Table 3-4 summarizes the agency costs for the overlay construction and assumed 2024 CPR activity on the stretch of U.S. 23 documented in this case study. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 3-4. Activity and cost data for U.S. 23.

Year	Activity	Activity Detail	Cost
1999	8-inch unbonded JRCP Overlay	8-in JRCP Paving with 1-in AC separation layer (UBOL: North portion)	\$7,367,678*
2024	CPR (estimated based on observed performance trends)	Full-depth repairs or deteriorated joints and transverse cracks, reseal joints	\$231,988**
2039	End-of-Life	Assumed Pavement End of Service	NA

*Cost in 1999 dollars (from DOT Contract)

** Cost in 1999 dollars, estimated based on other construction projects in Michigan that were similar in nature

A 40-year analysis period was used (1999-2039) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 3-4 are summarized below:

- 1999 JRCP UBOL: 40 years (end-of-service: 2039)
- 2024 CPR: 15 years (end-of-service: 2039)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 3-5 shows the construction costs converted to 1999 dollars.

Table 3-5. Agency costs for U.S. 23 converted to 1999 dollars.

Year	Activity	1999 Dollars
1999	Concrete Overlay Paving	\$7,367,678
2024	CPR	\$231,988
2039	Remaining Value (1999 JRCP UBOL)	(\$0)
2039	Remaining Value (2024 CPR)	(\$0)

Figure 3-14 shows the expenditure stream diagram for the U.S. 23 project using the four different discount rates used in the analysis.

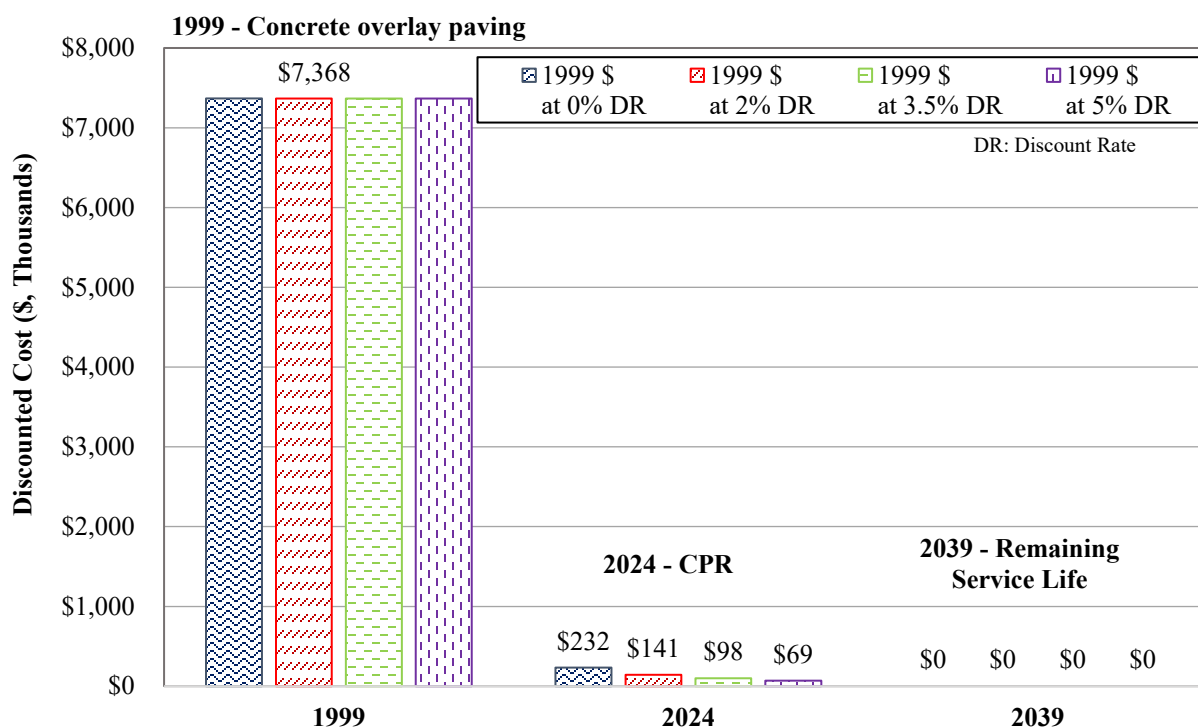


Figure 3-14. Expenditure stream diagram for U.S. 23.

Table 3-6 summarizes the LCCA results for each discount rate considered. As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 2 percent and the equivalent uniform annual cost (EUAC) increases by 128 percent. This is represented in figure 3-15.

Table 3-6. LCCA summary for U.S. 23.

0% Discount Rate				
LCCA Element	1999 Dollars		2019 Dollars	
1999 Concrete Paving	PV =	\$7,367,678	PV =	\$14,019,081
2024 CPR	PV =	\$231,988	PV =	\$441,422
Remaining Value (1999 Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2024 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$7,599,666	NPV =	\$14,460,504
	EUAC=	\$189,992	EUAC=	\$361,513
2% Discount Rate				
LCCA Element	1999 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$7,367,678	PV =	\$14,019,081
1991 Drainage Maintenance	PV =	\$141,404	PV =	\$269,061
Remaining Value (1968 PCC)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2014 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$7,509,082	NPV =	\$14,288,142
	EUAC=	\$274,500	EUAC=	\$522,314
3.5% Discount Rate				
LCCA Element	1999 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$7,367,678	PV =	\$14,019,081
1991 Drainage Maintenance	PV =	\$98,165	PV =	\$186,787
Remaining Value (1968 PCC)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2014 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$7,465,843	NPV =	\$14,205,868
	EUAC=	\$349,605	EUAC=	\$665,222
5% Discount Rate				
LCCA Element	1999 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$7,367,678	PV =	\$14,019,081
1991 Drainage Maintenance	PV =	\$68,507	PV =	\$130,353
Remaining Value (1968 PCC)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2014 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$7,436,185	NPV =	\$14,149,435
	EUAC=	\$433,367	EUAC=	\$824,603

PV: Present Value

NPV: Net Present Value

EUAC: Equivalent Uniform Annual Cost

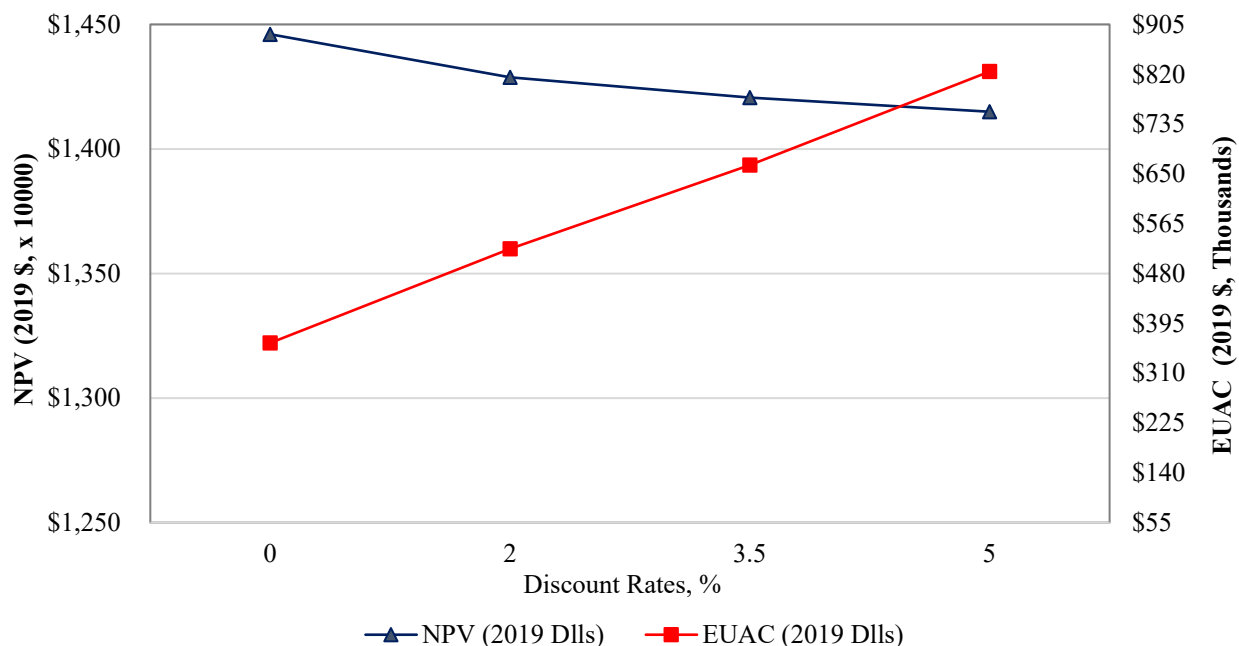


Figure 3-15. NPV and EUAC (2019 Dollars) vs discount rate.

Lessons Learned

The UBOLs on a 10.5-mi (16.9-km) section of U.S. 23 near Hartland in Michigan’s Lower Peninsula are excellent examples of long-life concrete overlay solutions that are performing well after nearly 20 years of service with minimal maintenance. Based on the LCCA conducted, future maintenance costs are less than 5 percent of the UBOL construction cost. As of 2017, average IRI was less than 100 inches/mi (1578 mm/km), and minimal distress was observed despite the location in a wet-freeze climate and carrying very heavy trucks. The lessons learned from these pavement sections include:

- **A good drainage system is essential to a well-performing UBOL.** Even though a 2011 forensic evaluation found an asphalt concrete cap blocking the path to the edge drains, at that time pumping and joint faulting were deemed insignificant (Hansen and Liu 2013). Yet by 2018, some isolated areas of slab cracking and pumping were linked to poor drainage.
- **Repair of existing concrete pavement (prior to overlay) was likely a factor in the good performance of the unbonded overlay.** However, repair quantities were not available in MDOT records, and this observation is purely anecdotal.
- **Use of a separator layer improves UBOL performance.** Use of a nominal 1-inch (25 mm) asphalt concrete separator layer prior to the construction of the concrete overlay appeared to be an effective method to minimize or eliminate reflective cracking. MDOT’s distress index values remained very low over the life of the overlay.

CHAPTER 4. UNBONDED PCC OVERLAY ON I-74 IN ILLINOIS

Introduction

In 1995, the Illinois Department of Transportation (IDOT) constructed a 9-inch (229 mm) continuously reinforced concrete (CRC) unbonded overlay (UBOL) on a 7.9-mi (12.7 km) stretch of the westbound lanes of I-74 near Galesburg. The project boundaries are between the Spoon river bridge and the intersection with IL-97 and U.S.150, just east of Galesburg (see figure 4-1). The original underlying pavement consisted of a 7-inch (178 mm) continuously reinforced concrete pavement (CRCP) constructed in 1969 that had received two asphalt overlays (1983 and 1994) prior to the placement of the UBOL. A pavement distress survey on the overlaid pavement prior to the construction of the UBOL documented significant longitudinal and transverse cracking as well as punchouts, D-cracking, and patching in the original concrete pavement (McNeal 1996).

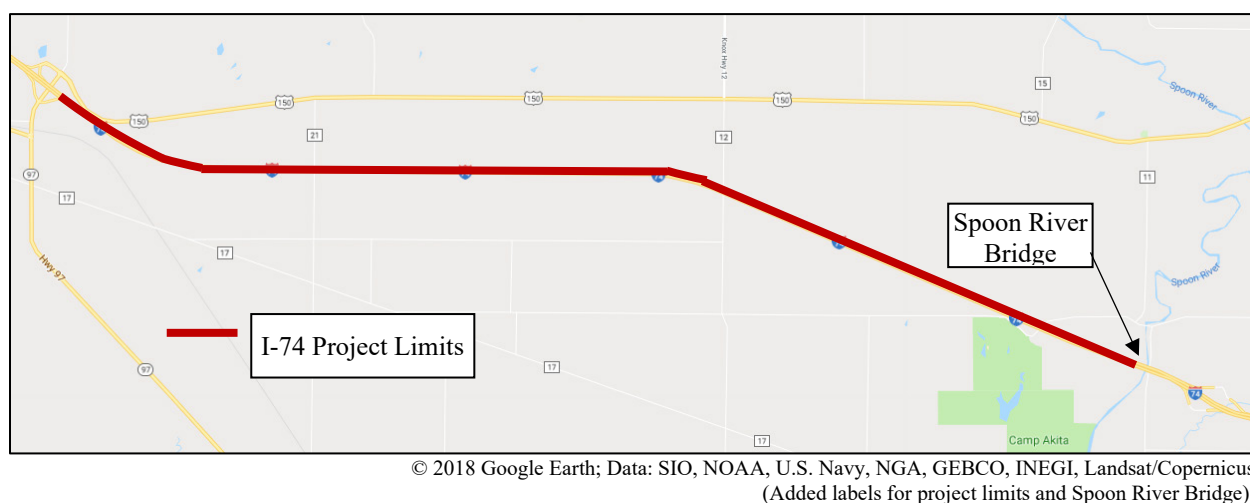


Figure 4-1. Location of UBOL on I-74 near Galesburg.

Overlay Design, Materials, and Construction

Structural Design

IDOT considered three design approaches for the CRCP overlay: the U.S. Army Corps of Engineers (Hutchinson 1982), the 1993 AASHTO Design Guide (AASHTO 1993), and the then-current IDOT pavement design procedure. The design thicknesses were calculated using a 20-year design life and a design traffic of 24 million equivalent single-axle load (ESAL) applications. As summarized in table 4-1, the three design approaches resulted in overlay thicknesses ranging between 8.75 and 9.50 inches (220 to 240 mm), with a final design thickness of 9 inches (230 mm) ultimately selected (McNeal 1996).

Table 4-1. Design thicknesses for UBOL on I-74

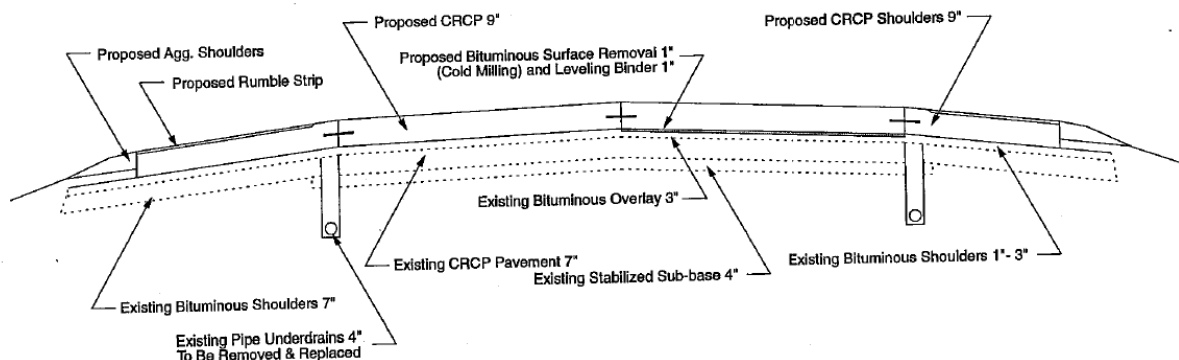
Design Method	CRCP Overlay Thickness (inch)
U.S. Army Corps of Engineers	8.75-in
AASHTO 1993	9.50-in
IDOT Method	9.25-in
Final Thickness Selected	9.00-in

Table 4-2 provides general information and pavement design details for the overlay design.

Table 4-2. I-74 project details.

Key Information	Data
Project Location/Characteristics	I-74 WB only Milepost 53.79 to 61.73 4-lane divided freeway
Climatic Region	Wet-freeze
Highlighted Project Design Characteristics	Unbonded CRC Overlay (on CRCP)
Initial Construction Year	1969 (for the underlying 7-inch CRCP)
Rehab Types, Years, and Detail	1995: 9-inch unbonded CRC overlay
Separator Layer (between UBOL and original CRCP)	3-inch asphalt interlayer
Traffic Data	1992 ADT: 11,500 (72% passenger vehicles, 4% single unit trucks, and 24% multi-unit trucks) 20-Year Design ESAL: 24 million 2017 AADT: 15,500 (4,800 Trucks)
Base/Subbase	4-inch bituminous stabilized base beneath existing 7-inch CRCP
Subgrade	Unknown
Subdrainage	Edge drains added as a part of the 1995 UBOL construction
Shoulder Type and Design	9-inch tied CRC shoulders
Unique/Noteworthy Features	Experimental piston dowel joint system used at the expansion joint at the east end of the project

Figure 4-2 shows the proposed cross section of the pavement section. The existing 3-inch (76 mm) asphalt overlay was to serve as the interlayer. Tied PCC shoulders were specified, and existing underdrains were to be removed and replaced as part of the overlay construction (McNeal 1996).



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Figure 4-2. Proposed pavement cross section (McNeal 1996).

Overlay Tie-In

A portion of the existing CRC pavement was removed and replaced at both ends of the project. On the west end, a lug system was used to tie the overlay into the existing pavement while an experimental piston dowel joint system was installed on the east end (McNeal 1996). The joint design is similar to a normal expansion joint but employs special pistons instead of dowels (see figure 4-3).

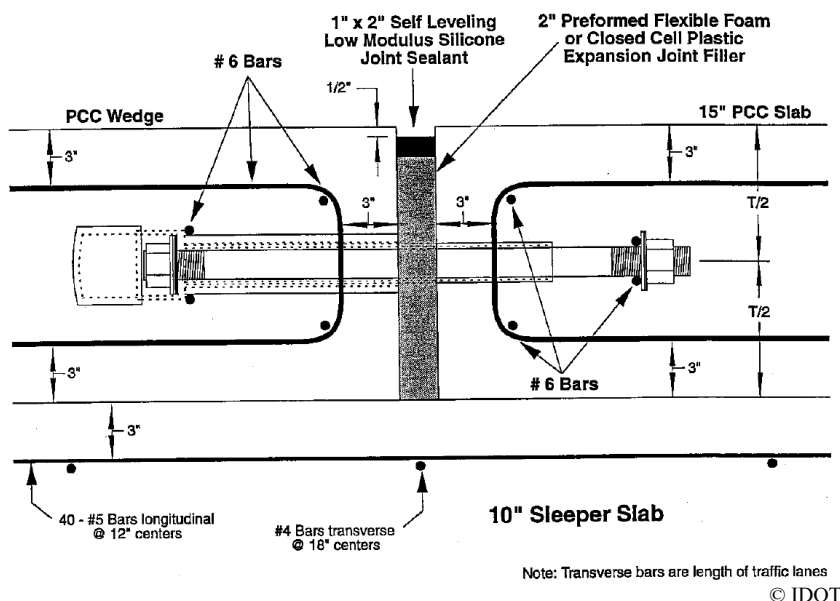


Figure 4-3. Detail of piston dowel joint terminal treatment.

Six piston dowel systems were placed per lane. As the pavement expands, the joint is designed to close and if additional expansion is needed, the adjacent expansion joint can begin to close. During contraction, the piston dowel joint opens, held in position by the piston head. The primary reason for this unique expansion joint design was to control the expansion of the pavement slabs and prevent them from pushing the bridge abutment (McNeal 1996).

Preparation of the Existing Pavement

Preparation of the existing pavement prior to the CRC overlay consisted of partial-depth repairs and cold milling followed by the placement of a leveling binder. All existing concrete patches on the existing pavement were milled to the level of the existing concrete pavement and replaced with full-depth asphalt patches to provide a uniform asphalt interlayer cross section for the UBOL. In areas where rutting in the overlay exceeded 0.5 inches (13 mm), the pavement was cold milled to a depth of 1 inch (25 mm), cleaned, primed, and a leveling binder was then placed to match the thickness of the interlayer (McNeal 1996).

Concrete overlay construction work was performed between June 15 and September 15, 1995 and involved a “pavement whitewash” over new asphalt surfaces. The dark color of the new asphalt surface was expected to be excessively hot during the summer months. This could increase the rate of hydration at the bottom of the concrete overlay, which can result in the development of thermal stresses that could cause premature cracking. The whitewashing was not specified on old asphalt layers since the surface was oxidized enough to reflect the heat from the sun.

Materials

A typical IDOT concrete paving mixture was used for the overlay. Details on the mixture designs are not available. Concrete mixtures were produced at an on-site concrete batch plant.

Overlay Construction

Overlay paving operations commenced on May 22, 1995. Reinforcing bars were placed on chairs on the prepared pavement surface prior to the overlay paving. The overlay paving operations were the same as those for a new concrete pavement. The surface was burlap dragged, finished using a mechanical float, and then hand finished. The pavement was transverse tined. Curing compound was sprayed over the pavement after the water sheen disappeared from the surface. Centerline sawing was performed either late in the evening or the following day (McNeal 1996).

As the paving operations extended into mid-June, the cold-milled areas had not yet been overlaid. Since the reinforcing bars had already been placed, the “pavement whitewash” could not be performed without impacting the rebars. Hence, water was sprayed on the asphalt layer to reduce the temperature (McNeal 1996).

Service Conditions

An analysis of IDOT traffic data over the last 10 years shows that annual traffic experienced a significant drop by 2011 but has continued to steadily increase since then to an AADT of 15,500 vehicles (31 percent trucks) per day as of 2017 (see figure 4-4).

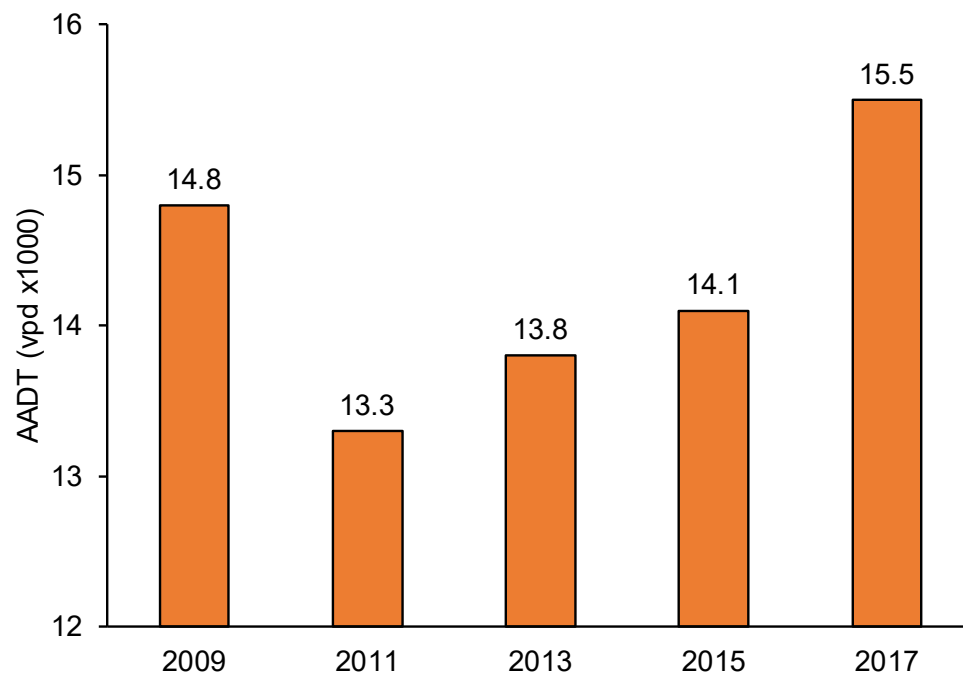


Figure 4-4. Historic AADT for I-74 WB project (Data source: IDOT).

Figure 4-5 shows typical truck traffic loadings on this segment of I-74. As noted earlier in table 3, the design traffic factor for the project was 24 million ESALs. As of 2017, the cumulative ESALs were determined to be 25.3 million ESALS (5 percent over the 20-year design traffic loadings).



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Figure 4-5. Typical truck traffic on I-74 WB project.

Project Characteristics

A 2019 site visit revealed that the CRC overlay was performing well (see figure 4-6 for an overview of the westbound lanes). Distresses observed included spalling along transverse cracks, punchouts, isolated full-depth repairs, and longitudinal cracking (see figures 4-7 through 4-9).



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Figure 4-6. Overview of I-74 WB project in 2018.



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Figure 4-7. Spalling along transverse crack (left) and punchout (right).



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Figure 4-8. Center of lane longitudinal crack (left) and deteriorated full-depth repair (right).



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Figure 4-9. Punchouts between shoulder and driving lane addressed with temporary patching.

Pavement Performance

IDOT uses the Condition Rating Survey (CRS) methodology to assess the condition of its road network to assign a numerical rating between 1 (*Failed*) and 9 (*New*) to the pavement section. Table 4-3 shows the CRS history data with distress and IRI information over the years. As of 2016 (and after 21 years of service), the CRS value was 7.7, indicating a pavement in *Excellent* condition. The CRS value is expected to fall below 4.5 (which is the threshold for *Poor* condition) by 2040 based on pavement performance prediction models used by IDOT (Heckel and Wienrank 2018).

Table 4-3. CRS, distress, and IRI data for I-74 WB project (Heckel 2002; Heckel and Wienrank 2018).

Year	CRS	Distresses	IRI, inches/mile
1995	9.0	-	-
1996	8.3	-	58
1998	*	*	68
2000	8.3	-	70
2002	7.9	Centerline Deterioration	72
2004	7.9	Centerline Deterioration	80
2006	7.8	Centerline Deterioration	78
2008	8.1	Transverse Cracking, Popouts	68
2010	8.2	Transverse Cracking, Popouts	65
2012	8.1	Transverse Cracking, Popouts	73
2014	7.5	Longitudinal and Transverse Cracking, Popouts	68
2016	7.7	Longitudinal Cracking, Edge Punchouts, Popouts, Permanent Patch Deterioration	71

* Invalid CRS data.

CRS Condition Categories: Excellent: 9.0 to 7.6; Good: 7.5 to 6.1; Fair: 6.0 to 4.6; Poor: 4.5 to 1.0

CRC pavements are characterized by a regular pattern of tight, low-severity transverse cracks, with a typical average spacing between cracks of 3 to 8 ft (0.9 to 2.4 m). IDOT performed inspections to count the number of transverse cracks over four 500-ft (152 m) segments, one at each mile post, and averaging them to obtain a final number. IDOT charted the progression of the average low-severity transverse crack spacing for the I-74 WB project from 3.1 in 1997 to 3.0 in 1998, 2.8 in 1999, and 2.3 in 2000 and 2001 (Heckel and Wienrank 2018).

IDOT monitors the roughness of its road network using the International Roughness Index (IRI). In 2000, the IRI for the project site on WB I-74 was recorded at 70 inches/mi (1,105 mm/km). In 2016, the average IRI of 71 inches/mi (1,121 mm/km) was recorded with infrequent low severity longitudinal cracking and popouts, 1 to 3 punchouts per mile, and less than 6 percent of the area patched. The pavement was still rated as being Excellent condition. A survey in 2019 (by the project team using its automated data collection equipment) revealed an average IRI of 73 inches/mi (1,152 mm/km) for the driving lane of the section. Figure 4-10 presents the 2019 IRI values over the length of the project reported on 0.1-mi (0.16 km) intervals.

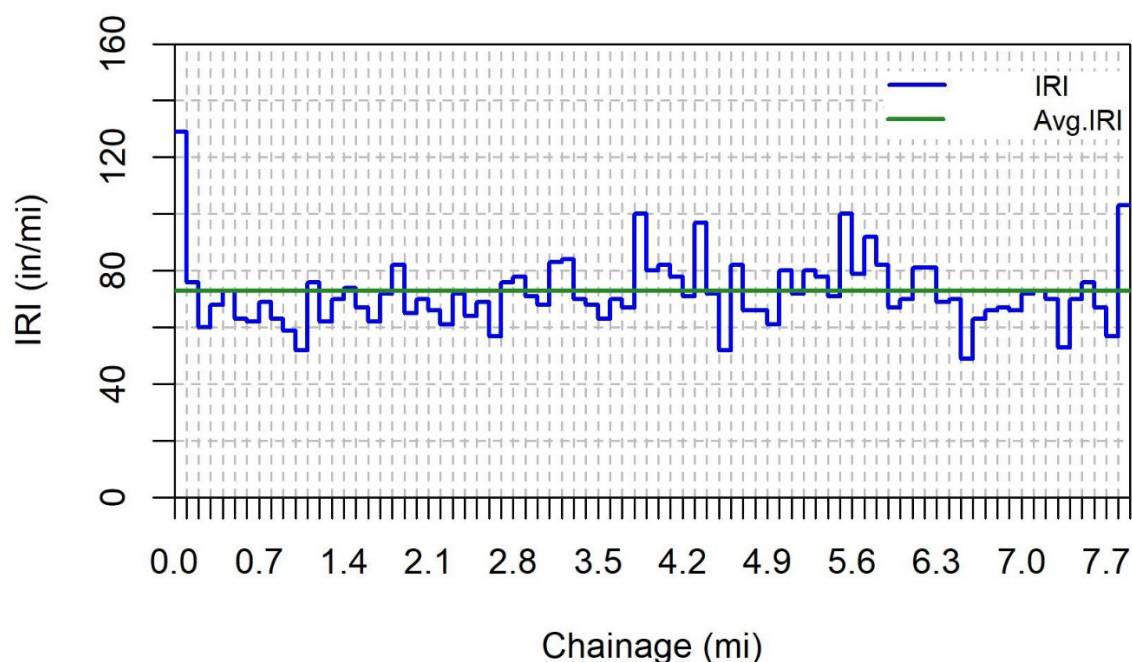


Figure 4-10. IRI profile for the driving lane of I-74 WB project.

IDOT also monitored the structural condition of the pavement using falling weight deflectometer (FWD) testing. Table 4-4 summarizes time-series deflection data for the I-74 project, including the deflection under the load (D_0), the deflection basin AREA [based on deflections measured under the load, and at 12 inches (305 mm), 24 inches (610 mm), and 36- inches (914 mm) from the load], the dynamic subgrade resilient modulus (E_{RI}), and the deflection load transfer efficiency (LTE) across transverse cracks.

**Table 4-4. Deflection data summary for I-74 WB project
(Heckel 2002; Heckel and Wienrank 2018).**

Date	Temp. (°F)	D_0 (mils)	Deflection Basin AREA (inches)	Dynamic E_{RI} (lb/in ²)	LTE (%)
7/95	90	2.0	31.0	17,400	-
8/96	77	2.2	31.6	15,100	-
7/97	85	2.4	31.4	14,300	92.6
4/99	56	2.1	31.0	17,800	92.6
7/01	94	2.3	30.8	16,300	91.6
9/16	N/A	1.8	31.7	17,100	91.9

The average deflection under the FWD load has remained fairly constant at approximately 2.1 mils (0.05 mm). The lower the deflection, the stiffer the pavement system. Deflections under 6 mils (0.15 mm) are generally indicative of a structural sound concrete pavement (Heckel and Wienrank 2018). The deflection basin AREA represents the ability of the pavement to spread the load to the underlying layers; the higher the basin area the greater the load-spreading capacity of the pavement. The historic trend shows that the deflection basin AREA parameter has remained fairly constant over the years.

E_{RI} represents the subgrade structural support (dynamic modulus), with higher values signaling greater subgrade stability. E_{RI} values show a decreasing trend over the first three measurements and the values then increase to the original measurement of around 17,000 lb/in² (117 MPa). All measurements are well over 10,000 lb/in² (69 MPa), which is considered good support for concrete pavement (Heckel and Wienrank 2018).

LTE measures the ability of the pavement to transfer loading between adjacent cracks or joints. LTE has decreased over time, but as of 2016 the average value is still over 90 percent.

Since the structural parameters recorded in 2016 are not much different from the values measured between 1995 and 1997, the structural integrity of the pavement does not appear to have been significantly diminished since the original construction of the UBOL.

Economic Analysis

Table 4-5 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of I-74 documented in this case study. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 4-5. Activity and cost data for I-74.

Year	Activity	Activity Detail	Cost
1969	Initial Construction	7-inch CRCP	Not used in the analysis
1995	Concrete Overlay	9-inch CRCP Paving (UBOL)	\$5,117,028 ¹
2014	Minor CPR	1% Class A Type III Patch 100% Joint Sealing (long)	\$573,401 ²
2026	CPR (assumed to be performed in the future)	10% Class A Type III Patch 25% Diamond Grinding	\$2,245,844 ²
2040	End-of-Life	Assumed Pavement End of Service	NA

¹Cost in then-current dollars; data source: IDOT project scoping document

²Cost based on average unit prices reported in IDOT bid tabulations (2017 dollars)

A 45-year analysis period was used (1995-2040) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 4-5 are summarized below:

- 1995 Concrete Overlay Paving (UBOL): 45 years (end-of-service: 2040)
- 2014 CPR: 12 years (end-of-service: 2026)
- 2026 CPR: 14 years (end-of-service: 2040)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 4-6 shows the construction costs converted to 1995 dollars.

Table 4-6. Agency costs for I-74 project converted to 1995 dollars.

Year	Activity	1995 Dollars
1995	Concrete Overlay Paving	\$5,117,028
2014	CPR	\$281,298
2026	CPR	\$1,106,467
2040	Remaining Value (1995 UBOL Construction)	(\$0)
2040	Remaining Value (2026 CPR)	(\$0)

Figure 4-11 shows the expenditure stream diagram for the I-74 project using the four different discount rates used in the analysis.

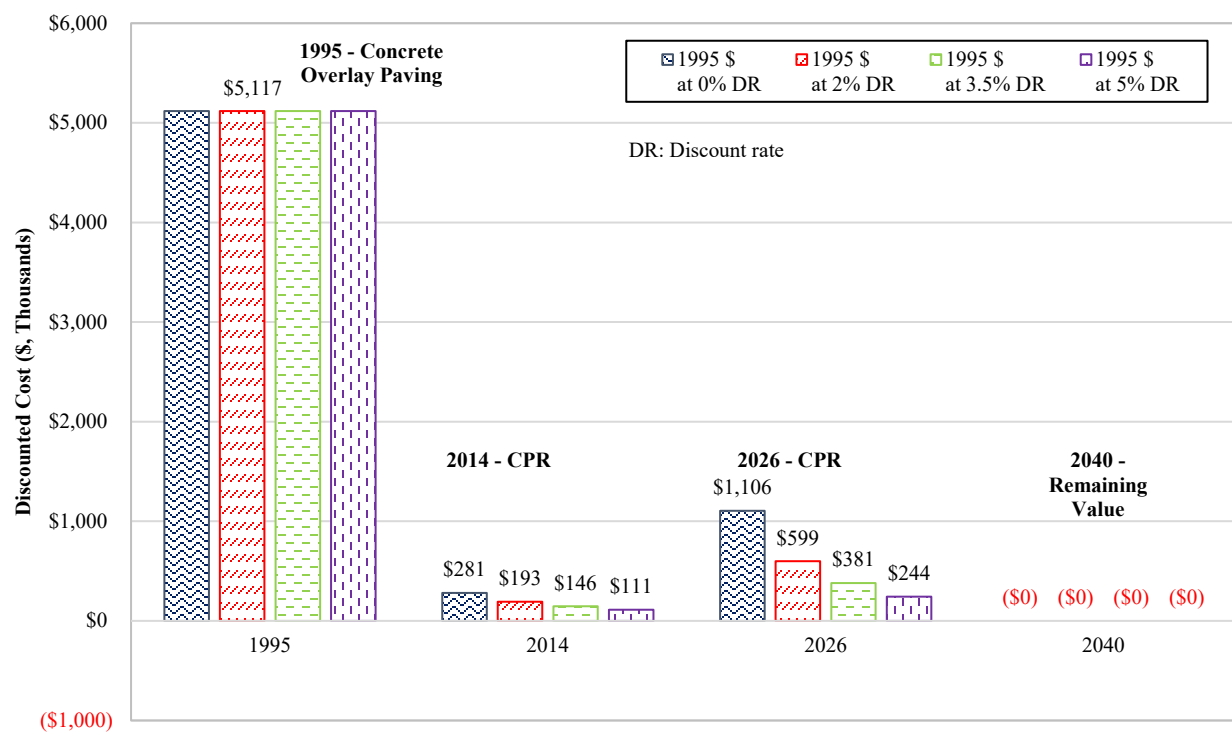
**Figure 4-11. Expenditure stream diagram for I-74 project (in 1995 dollars).**

Table 4-7 summarizes the LCCA results for each discount rate considered, expressed in both 1995 and 2019 dollars.

Table 4-7. LCCA summary for I-74 project.

0% Discount Rate				
LCCA Element	1995 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$0	PV =	\$0
1995 Concrete Overlay Paving	PV =	\$5,117,028	PV =	\$10,902,740
2014 CPR	PV =	\$281,298	PV =	\$599,356
2026 CPR	PV =	\$1,106,467	PV =	\$2,357,526
Remaining Value (1995 Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2026 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$6,504,793	NPV =	\$13,859,622
	EUAC =	\$144,551	EUAC =	\$307,992
2% Discount Rate				
LCCA Element	1995 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$0	PV =	\$0
1995 Concrete Overlay Paving	PV =	\$5,117,028	PV =	\$10,902,740
2014 CPR	PV =	\$193,092	PV =	\$411,416
2026 CPR	PV =	\$598,871	PV =	\$1,276,001
Remaining Value (1995 Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2026 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$5,908,991	NPV =	\$12,590,158
	EUAC =	\$200,372	EUAC =	\$426,927
3.5% Discount Rate				
LCCA Element	1995 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$0	PV =	\$0
1995 Concrete Overlay Paving	PV =	\$5,117,028	PV =	\$10,902,740
2014 CPR	PV =	\$146,319	PV =	\$311,758
2026 CPR	PV =	\$380,880	PV =	\$811,532
Remaining Value (1995 Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2026 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$5,644,226	NPV =	\$12,026,030
	EUAC =	\$250,905	EUAC =	\$534,598
5% Discount Rate				
LCCA Element	1995 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$0	PV =	\$0
1995 Concrete Overlay Paving	PV =	\$5,117,028	PV =	\$10,902,740
2014 CPR	PV =	\$111,319	PV =	\$237,186
2026 CPR	PV =	\$243,821	PV =	\$519,503
Remaining Value (1995 Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2026 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$5,472,168	NPV =	\$11,659,429
	EUAC =	\$307,874	EUAC =	\$655,980
PV: Present Value				
NPV: Net Present Value				
EUAC: Equivalent Uniform Annual Cost				

As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 16 percent and the equivalent uniform annual cost (EUAC) increases by 113 percent (see figure 4-12).

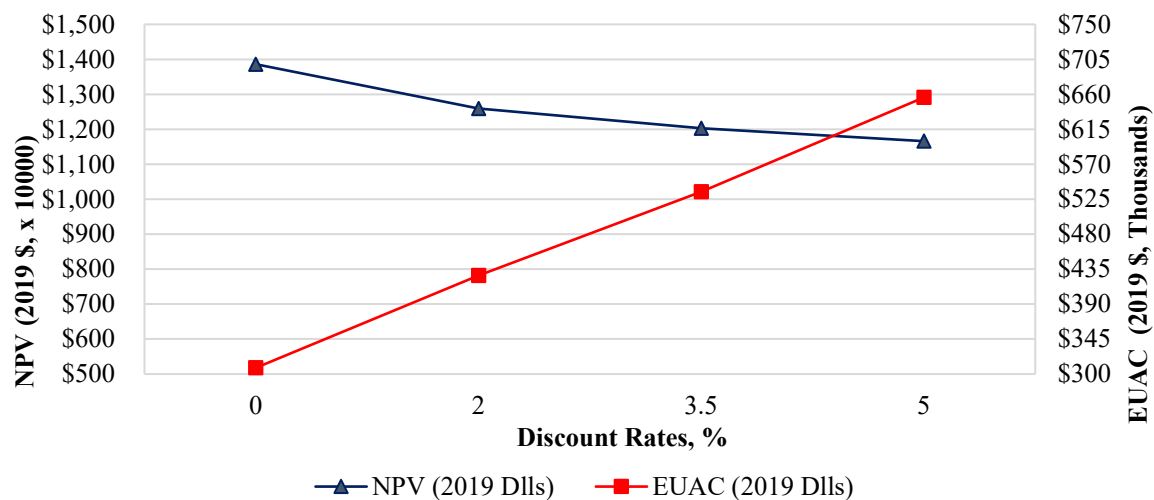


Figure 4-12. NPV and EUAC (2019 Dollars) vs. discount rate.

Lessons Learned

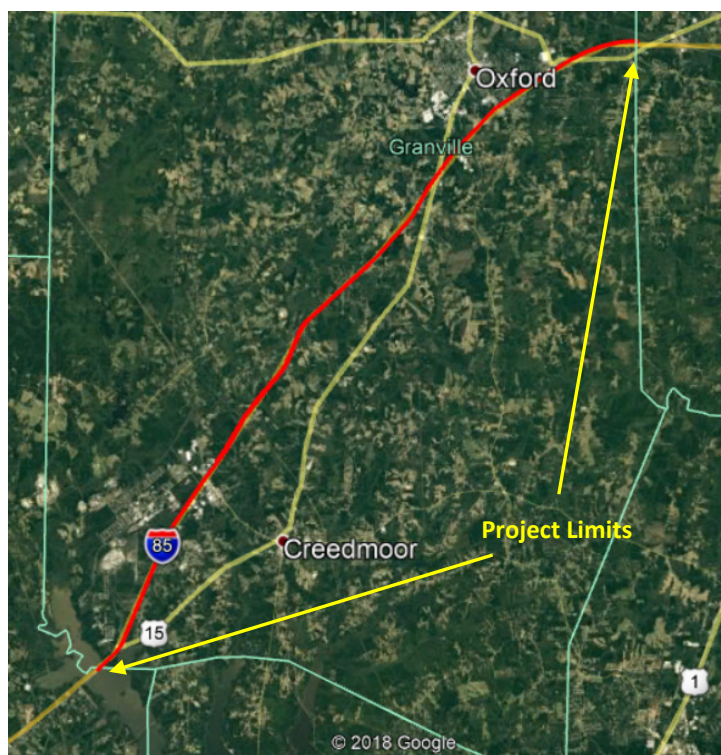
In 1995, IDOT rehabilitated a 7.9-mile (12.7 km) stretch of I-74 WB near Galesburg, IL, by constructing a CRC UBOL over the original CRC built in 1969. The primary motivation for this project was to evaluate the performance of a concrete overlay as an effective rehabilitation alternative to complete pavement reconstruction. After more than 20 years in service, the UBOL is still in good condition. Some lessons learned are summarized below. Based on the LCCA conducted, future maintenance costs vary between 6 and 21 percent of the UBOL construction cost, depending on the discount rate used.

- **Unbonded CRC overlay on existing CRC is a viable rehabilitation option, especially when the underlying concrete pavement is severely deteriorated.** The structural capacity of the overlay is expected to be similar to that of new concrete pavement construction and the cost of constructing an overlay is expected to be lower than the cost of complete removal and reconstruction.
- **The UBOL has demonstrated outstanding performance for more than 20 years, as it has exhibited minimal distress development and maintained an exceptional level of serviceability.**
 - As of 2017, the cumulative ESALs on the pavement section was estimated to be approximately 5 percent over the 20-year design traffic loadings of 24 million ESALs. Structural evaluation (using FWD) shows that even after more than 20 years, the structural capacity of the pavement has not been impacted significantly.
 - Except for some isolated spot locations, the average IRI value has not degraded significantly since the original construction. An IRI value of 73 inches/mile (1152 mm/km) after more than 20 years of service shows that the pavement is still in excellent functional condition.

CHAPTER 5. UNBONDED CONCRETE OVERLAY ON I-85 IN NORTH CAROLINA

Introduction

In 1997, the North Carolina Department of Transportation (NCDOT) rehabilitated a 10.2-mi [16.4-km] stretch of I-85 (Northbound lanes) in Granville County, approximately 40 miles [64 km] north of Raleigh, North Carolina (see figure 5-1) by constructing a 10-in [25-cm] unbonded Concrete (JPC) overlay (UBOL). The project boundaries just south of the rest area near the U.S.-15 and I-85 interchange (southern boundary) to just north of the Vance County line (northern boundary). This pavement section was originally built in 1970 as an 8-inch [20-cm] continuously reinforced concrete pavement (CRCP), with 12-ft [3.7-m] wide lanes. Since the 1997 UBOL, diamond grinding is the only major rehabilitation activity that has been performed.



© 2018 Google Earth; Data: SIO, NOAA, U.S. Navy, NGA, GEBCO, INEGI, Landsat/Copernicus (Added labels added to indicate project limits).

Figure 5-1. NC I-85 unbonded overlay project location and limits.

Original Pavement Design and Materials

Structural Design

The original pavement was constructed in 1971 as an 8-in (20-cm) CRCP over a 4-in [10-cm] unbound aggregate base course over sandy loam and other mostly loamy soils. CRCP reinforcing details are not available. Project plans indicate the presence of longitudinal shoulder drains, but the original drainage design details are not available.

Paving Materials

Information concerning the CRCP concrete mixture proportions, concrete aggregate type, and aggregate base properties is not available.

Unbonded Overlay Design and Construction

By 1997 (after 26 years of service), the original CRCP was exhibiting serious cracking and had been repaired in several locations (see Figure 5-2). A 10-inch [250-mm] unbonded jointed plain concrete overlay was placed over a 2-inch [50-mm] permeable asphalt interlayer on the CRCP. Full-depth repairs were performed prior to overlay placement. Project bid tabulations (Oman Systems, Inc. 2019a) suggest that these repairs were undoweled and totaled approximately 12,975 s.y. [10,849 m²].



Figure 5-2. Typical CRCP condition prior to overlay in 1997 (source: Morrison 2017).

The concrete overlay was placed with 27-ft [8.2-m] panels and doweled (1.5-in [38-mm] diameter), perpendicular transverse joints. Asphalt shoulders were constructed with a total of 12 inches [30 cm] total thickness, matching the total thickness added to the top of the CRCP. The concrete pavement surface was transversely tined.

Service Conditions

I-85 is a major north-south travel corridor in North Carolina that experiences high levels of traffic year-round. Design traffic volumes for the original CRCP are unknown, but the following design traffic data is available from the as-constructed plans for the 1997 overlay (NCDOT, undated):

ADT 1995 = 25,800
ADT 2020 = 55,000
DHV = 10 percent
D = 60 percent (5 percent Duals)
T = 17 percent TTST

Annual average daily traffic (AADT) data are available for 2002-2017 and are presented in figure 5-3. The data show that traffic has increased steadily from about 28,300 vehicles per day (VPD) in 2003 to approximately 35,800 in 2017 – an increase of 26.5 percent over 15 years.

Heavy truck traffic is currently reported to comprise about 8.4 percent of all vehicles, although the truck volume seemed significantly higher during the 2018 site visit (see Figure 5-4).

The I-85 corridor in NC is located in a wet-nonfreeze climate, which results in a relatively narrow range of temperatures and humidity conditions that may reduce the severity of temperature and moisture gradients, and therefore, curling and warping stresses in concrete pavement. The absence of deep freeze conditions eliminates the likelihood of durability cracking, frost heave and many other temperature-related distresses in paving materials and foundations.

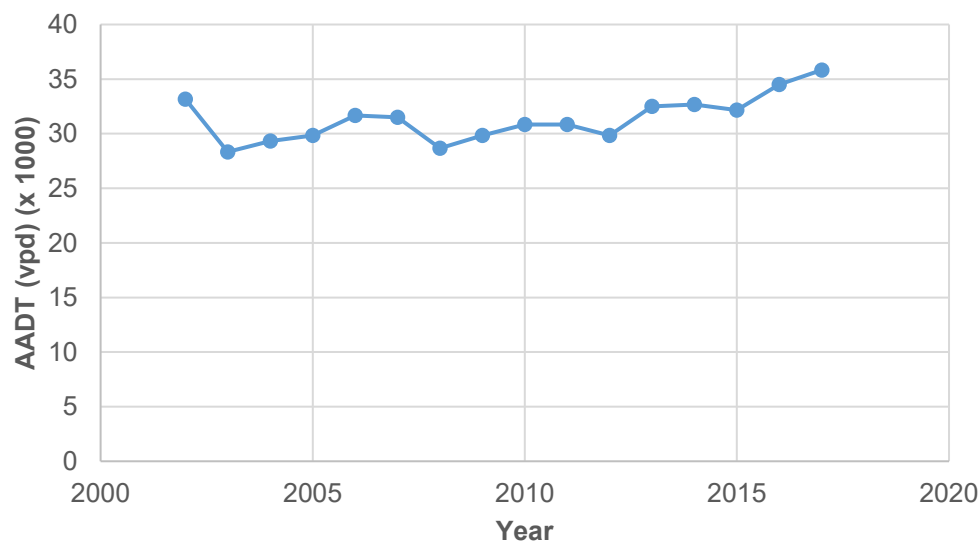


Figure 5-3. Historic AADT for I-85 NB (Data source: NCDOT 2019).



Figure 5-4. Photo of I-85 late morning traffic volume and composition during December 2018 site visit.

Design, Construction and Rehabilitation Summary

Table 5-1 presents a summary of the project design, construction, and rehabilitation activities.

Table 5-1. Summary of NC I-85 project design, construction and rehabilitation information.

Project Location	Northbound Interstate 85, Granville County, NC From: Station 753+00.00 in Granville County (~Milepost 198.2) To: Station 36+50 in Vance County line (~Milepost 208.5) Project length: 10.171 centerline-miles [16.37 centerline-km] Four-lane divided freeway Wet-nonfreeze climate
Highlighted Project Design Characteristics	Unbonded concrete overlay (UBOL) of continuously reinforced concrete pavement (CRCP)
Original Construction Overview	1971 construction; four-lane divided freeway • Pavement: 8-inch [200 mm] CRCP • Subbase: 4 inch [100 mm] unbound aggregate • Subgrade: Predominantly (>60%) sandy loam, with loam, clay loam and other soils in lesser quantities • Subdrainage: Longitudinal shoulder drains were included, likely 4-in [100-mm] diameter pipe.
Rehabilitation	1997 Unbonded Concrete Overlay • 10-inch [250-mm] jointed concrete overlay on 2-in [50-mm] permeable asphalt separation layer • 27-ft [8.2-m] panel length, rectangular panels • 1.5-inch [38-mm] diameter dowels on 12-inch [300-mm] centers • Shoulders: 2.5-inch [62-m] asphalt surface, 3.5-inch [90-mm] asphalt binder course, 4-inch [100-mm] asphalt base, 2-inch [50-mm] permeable asphalt drainage layer. • Installation of shoulder edge drains and associated drainage features 2010 CPR • Blanket diamond grinding • Joint sealing • Scattered small partial-depth repairs Additional Work (assumed 2010 – 2018) • Full-depth overlay repairs near southern project limit.
Traffic Data	Original Design: unknown Overlay Project (Design): • ADTT (1995): 25,800 vpd • ADTT (2020): 55,000 vpd • DHV: 10% • D: 60% (5% Duals) • T: 17% TTST 2017 AADT (reported): ~35,800 (11.1% trucks) [Data source: NCDOT 2019]
Shoulder Type and Design	Asphalt shoulders
Unique/Noteworthy Features	10-inch [250-mm] unbonded JPCP overlay with 27-ft [8.2-m] long panels over CRCP.

Maintenance and Rehabilitation Activities

Concrete pavement restoration (CPR) work was undertaken in 2010 (after 13 years of overlay service) to restore the condition of the unbonded concrete overlay. The main features of this work included a blanket diamond grind of the entire project and resealing of all transverse and longitudinal joints (including the two lane-shoulder joints. Some partial-depth repairs (442 s.f. [41 m²]) were also performed (using what appeared to be polymer concrete) at slab corners and

along some of the transverse cracks that had developed in the 27-ft [8.2-m] panels. The repairs were not part of the original contract and do not appear in the 2010 bid tabulations (Oman Systems, Inc. 2019b), but were performed under a supplemental agreement to the contract (Personal Communication, Wiley Jones, NCDOT Asst. State Construction Engineer).

The 2018 site visit revealed that a few full-depth overlay repairs have been performed near the rest area at the south end of the project. This work is also not documented in the 2010 bid tabulations (Oman Systems, Inc. 2019b) and these repairs did not exhibit the same grinding pattern as the rest of the pavement during the 2018 site visit, so it is assumed that they were constructed by NCDOT personnel sometime between 2010 and 2018.

No other significant maintenance or rehabilitation work has been performed on this project since the overlay construction in 1997.

Pavement Performance History

The unbonded concrete overlay on this section of I-85 has performed very well for more than 20 years. Some (but not many) of the panels have developed transverse cracks, which is likely due to the exceptionally long panel length used (27 ft [8.2m]) and the resulting high combined stresses of curling/warping and traffic loads. These cracks have generally not deteriorated and have been either sealed or repaired using what appears to be a polymer concrete repair material, and they have not resulted in unusual loss of pavement ride quality, as shown in Figure 5-5. Figure 5-5 also shows the effectiveness of the 2010 diamond grinding on significantly reducing (by about 50 percent) the International Roughness Index (IRI). It is reported that the diamond grinding contractor earned a ride quality incentive of approximately \$6,400 on this project.

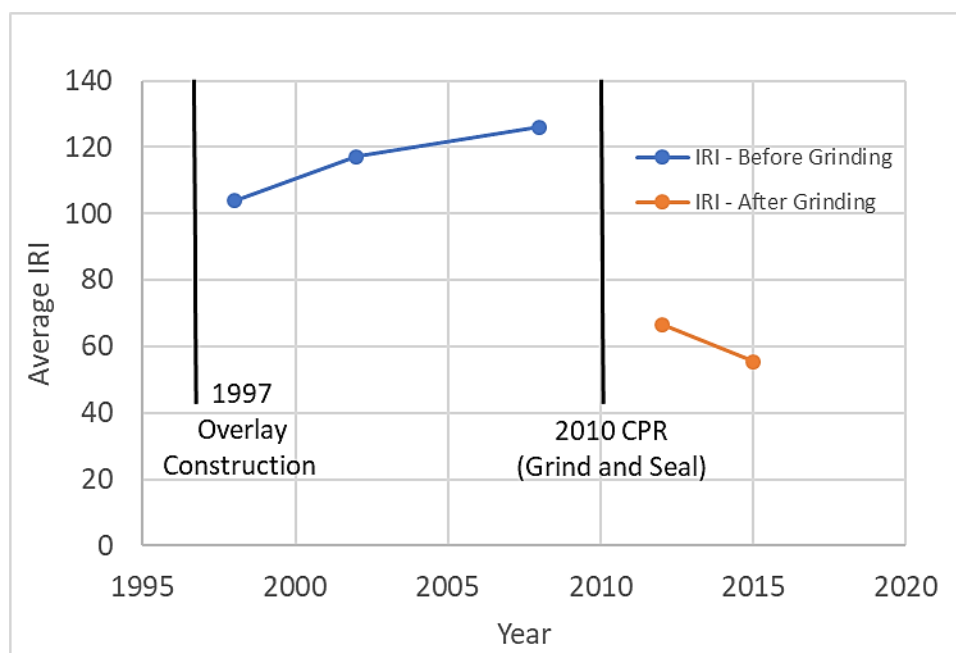


Figure 5-5. IRI data for NC I-85 unbonded overlay, 1998 – 2015 (made from data Morrison 2017).

NCDOT performs periodic pavement condition surveys that are used to develop distress indices ranging from 0 – 100, where 100 corresponds to “no notable distress present.” Indices are computed for surface wear, corner breaks, joint seal condition, joint faulting, and overall pavement condition. Index data are available for 2013 and 2015, as shown in Table 5-2. The data show no significant changes in pavement performance over the reported period; variability is likely due to variations in slab sampling and measurement practices between years.

Table 5-2. Summary of NCDOT distress Index data for Northbound I-85 (MP 198.2 – 208.5) (Morrison 2017).

Distress	Year	
	2013 (based on 4377 slabs)	2015 (based on 4477 slabs)
Surface Wear	90	86
Corner Break	100	92
Joint Seal	82	100
Faulting	99	97
Overall	89	93

Conditions Observed During 2018 Site Visit

A field visit during November 2018 verified that the pavement is still in very good condition after 21 years of service. Typical pavement conditions (no cracking or other distresses, excellent ride quality) are shown in Figure 5-6.



Figure 5-6. Typical 2018 pavement conditions on NC I-85 overlay project.

Occasional mid-panel cracking was observed, generally in the outer (driving) lane. In some cases, these cracks were only sealed with a liquid crack sealing material (see Figure 5-7) while in other cases partial-depth spall repairs had been performed over some or all of the crack length (see Figure 5-8). The repair material appeared to be a proprietary polymerized concrete.



Figure 5-7. Typical 2018 mid-panel transverse cracking with only joint sealant applied (source: Google Earth, December 2018).



Figure 5-8. Typical 2018 mid-panel transverse cracking with different amounts of partial-depth spall repair.

Figure 5-9 shows typical small partial-depth spall repairs that were applied at slab corners and along transverse cracks and joints in 2010. Figure 5-10 shows examples of full depth repairs that were observed in one area near the south end of the project (adjacent to the rest area); these are atypical and not representative of the overall project. As noted previously, these repairs appear to have been placed after the 2010 CPR because they exhibit a different grinding pattern than that of the surrounding concrete (see Figure 5-10).



Figure 5-9. Typical partial-depth joint spall repairs on NC I-85 overlay project.



Figure 5-10. Examples of full-depth repairs near rest area (~MP 198.5) on NC I-85 overlay project.

Economic Analysis

Table 5-3 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of I-85 documented in this case study. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 5-3. Activity and cost data for I-85.

Year	Activity	Activity Detail	Cost ¹
1971	Initial Construction ²	8-inch [200-mm] thick CRC over 4-in [100-mm] aggregate base	N/A
1997	Unbonded JPCP Overlay	10-in [250-mm] thick JPC overlay with 2-in [50-mm] asphalt interlayer, including pre-overlay repairs	\$14,201,695
2010	Concrete Pavement Rehabilitation (CPR) [data source: Morrison and Jones 2018]	Diamond grinding (143,208 s.y. [119,740 m ²] @ \$2.20/s.y. [\$2.63/m ²])	\$315,058
		Joint resealing (47,735 lin ft of transverse joints [14,550 m] @ \$0.78/lin ft [\$2.56/m] and 161,109 lin ft of longitudinal joints [49,106 m] @ \$0.96/lin ft [\$3.15/m])	\$191,898
		Partial-depth spall repairs (est. 442.42 s.f [41.1 m ²] @ \$149.64/s.f. [\$1610/m ²])	\$66,204
2025	Concrete Pavement Rehabilitation (CPR) [assumed]	Diamond grinding, joint resealing, partial depth repair	\$573,160 ³
2037	Assumed Pavement End of Service		

¹Costs shown in then-current dollars²Not considered in analysis³Cost based on 2010 CPR; in 2010 dollars

A 40-year analysis period was used (1997-2037) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 5-3 are summarized below:

- 1997 UBOL: 40 years (end-of-service: 2037)
- 2010 CPR: 15 years (end-of-service: 2025)
- 2025 CPR: 12 year (end-of-service: 2037)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 5-4 shows the construction costs converted to 1997 dollars.

Table 5-4. Agency costs for the I-85 project

Year	Activity	1997 Dollars
1970	Original Construction	-
1997	Concrete Overlay Paving	\$14,201,695
2010	CPR	\$351,997
2025	CPR	\$351,997
2037	Remaining Value (1997 UBOL)	\$0
2037	Remaining Value (2010 CPR)	\$0

Figure 5-11 shows the expenditure stream diagram for the I-85 project using the four different discount rates used in the analysis.

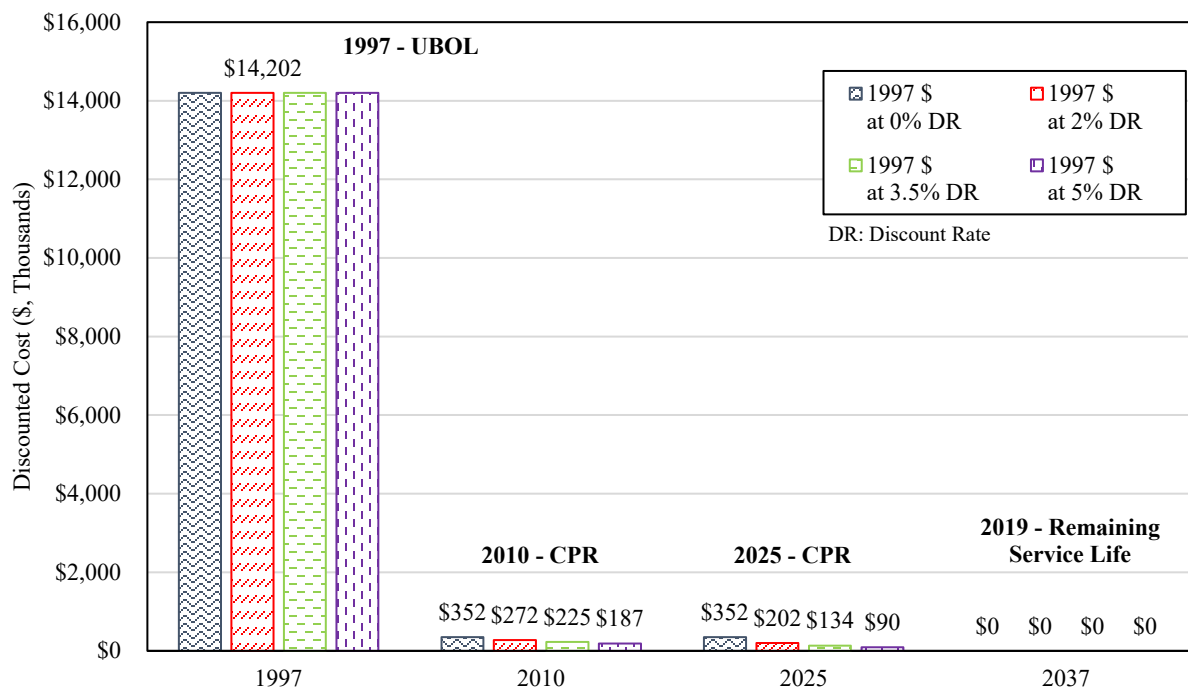


Figure 5-11. Expenditure stream diagram for I-85.

Table 5-5 summarizes the LCCA results for each discount rate considered. As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 3 percent and the equivalent uniform annual cost (EUAC) increases by 126 percent. This is graphically represented in figure 5-12.

Table 5-5. LCCA summary for the I-85 project.

0% Discount Rate				
LCCA Element	1997 Dollars		2019 Dollars	
1997 Concrete Overlay Paving	PV =	\$14,201,695	PV =	\$28,243,504
2010 CPR	PV =	\$351,997	PV =	\$700,030
2025 CPR	PV =	\$351,997	PV =	\$700,030
Remaining Value (1997 Concrete Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2010 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$14,905,688	NPV =	\$29,643,565
	EUAC =	\$372,642	EUAC =	\$741,089
2% Discount Rate				
LCCA Element	1997 Dollars		2019 Dollars	
1997 Concrete Overlay Paving	PV =	\$14,201,695	PV =	\$28,243,504
2010 CPR	PV =	\$272,105	PV =	\$541,146
2025 CPR	PV =	\$202,178	PV =	\$402,080
Remaining Value (1997 Concrete Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2025 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$14,675,978	NPV =	\$29,186,730
	EUAC =	\$536,491	EUAC =	\$1,066,943
3.5% Discount Rate				
LCCA Element	1997 Dollars		2019 Dollars	
1997 Concrete Overlay Paving	PV =	\$14,201,695	PV =	\$28,243,504
2010 CPR	PV =	\$225,068	PV =	\$447,602
2025 CPR	PV =	\$134,341	PV =	\$267,170
Remaining Value (1997 Concrete Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2025 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$14,561,104	NPV =	\$28,958,276
	EUAC =	\$681,857	EUAC =	\$1,356,037
5% Discount Rate				
LCCA Element	1997 Dollars		2019 Dollars	
1997 Concrete Overlay Paving	PV =	\$14,201,695	PV =	\$28,243,504
2010 CPR	PV =	\$186,671	PV =	\$371,241
2025 CPR	PV =	\$89,792	PV =	\$178,573
Remaining Value (1997 Concrete Overlay)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2025 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$14,478,158	NPV =	\$28,793,318
	EUAC =	\$843,760	EUAC =	\$1,678,022
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost				

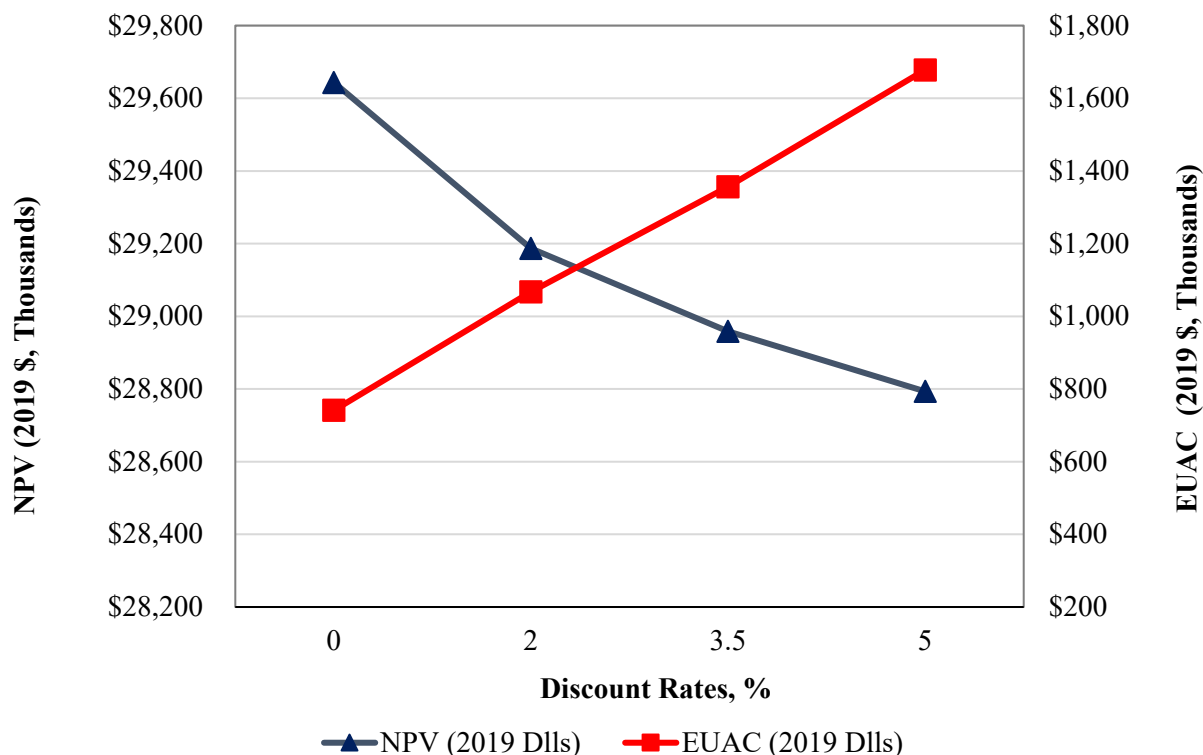


Figure 5-12. NPV and EUAC (2019 Dollars) vs. discount rate.

Lessons Learned

The 10.33-mile [16.64-km] unbonded, jointed plain concrete overlay of continuously reinforced concrete pavement on northbound I-85 in Granville County, North Carolina is an example of the excellent performance that can be achieved with unbonded concrete overlays, particularly with timely maintenance and rehabilitation. The subject pavement is more than 20 years old, is in very good condition (high distress and overall serviceability indices and very low IRI values) and has needed only routine diamond grinding and joint sealing work (with minor spall repair work). Based on the LCCA conducted, future maintenance costs are estimated to be less than 5 percent of the UBOL construction costs. The lessons learned from this pavement section include:

- Unbonded concrete overlays can provide excellent service and long life, even when constructed over badly distressed pavement.** The effective structural thickness of the overlaid pavement system is likely 12 – 13 inches [300 – 325 mm]. Given the high effective thickness, the relatively mild climate, and the excellent condition of this pavement after more than 20 years of service, it is not difficult to imagine that 40 years or more of service life will be attained with the use of timely and effective pavement preservation techniques. NCDOT recognizes the potential for long service life offered by unbonded concrete overlays and has used overlays on several other projects along the I-85 corridor and in other parts of the State.
- Timely pavement preservation activities prevent rapid pavement deterioration and help to ensure a long service life.** The 2010 CPR work was highly effective in extending pavement life by addressing key potential distress mechanisms. For example, the diamond grinding improved ride quality and reduced the potential for heavy vehicle

impact damage, while the joint resealing reduced the potential for surface water infiltration that might otherwise cause stripping of the asphalt interlayer stripping and softening and pumping of the pavement system foundation. A repetition of these activities (along with additional partial-depth spall repairs, as needed) may be needed to achieve 40 or more years of service from the overlay.

- **The use of shorter concrete overlay panels might have further improved pavement performance.** The 27-ft [8.2-m] panels used on this project are unusually long for unreinforced panels, resulting in a panel aspect ratio of 2.25 – substantially higher than the 1.25 – 1.50 limit typically allowed by most agencies. The mild climate and compliant asphalt interlayer may have contributed to the generally successful use of long panels on this project, but some mid-panel transverse cracks have developed and others may in the future. The use of shorter panels may avoid this problem.

CHAPTER 6. UNBONDED PCC OVERLAY ON U.S. 1 IN FLORIDA

Introduction

In 1988, the Florida Department of Transportation (FDOT) constructed an unbonded concrete overlay (UBOL) on a deteriorated 1.86-mi (3-km) stretch of asphalt pavement on U.S. 1/State Route (SR) 5 in Edgewater, about 20 miles (32 km) south of Daytona Beach (see figure 6-1). Constructed as an experimental project, the pavement is located in the southbound lanes of U.S. 1 between the intersections with Omni Drive on the north end and River Park Boulevard on the south. The original asphalt pavement consisted of 5 inches (127 mm) of asphalt concrete placed over 8.5-inches (216 mm) of limerock base and granular subgrade. The primary motivation behind the decision to construct the UBOL was the economic concern related to the volatility in asphalt prices.



L: © 2018 Google Earth; Data: SIO, NOAA, U.S. Navy, NGA, GEBCO, INEGI, Landsat/Copernicus. (Added pin for project location). R: © 2018 Google Earth; Data: Google, TerraMetrics, Airbus, Data SIO, NOAA, U.S. Navy, NGA, GEBCO. (Added line for project limits).

Figure 6-1. U.S. 1 project location.

Overlay Design, Materials, and Construction

Structural Design

A standard jointed plain concrete pavement (JPCP) was designed and constructed at the south end of the project to serve as a control for the UBOL test segments. That control pavement [7-inch (177 mm) JPCP over 8.5-inch (216 mm) limerock base; 14-ft (4.3-m) joint spacing] was placed on the existing limerock base after the existing asphalt had been removed and was designed for a 10-year life using the 1986 *AASHTO Guide for the Design of Pavement Structures*. The primary design variables considered in the design of the control pavement were traffic loading, flexural strength, concrete elastic modulus, and the modulus of subgrade reaction.

For the rest of the project, approximately 4 inches (102 mm) of the existing asphalt pavement was milled off and a 1-inch (25-mm) leveling surface was applied prior to the placement of the JPCP UBOL. A total of 18 test sections were constructed, investigating the following primary design features (Greene et al. 2018):

- Slab thickness—6-, 7-, and 8-inch (152-, 177-, and 203-mm) thick slabs.
- Joint spacing—12-, 14-, 16-, 18-, and 20-ft (3.7, 4.3, 4.9-, 5.5-, 6.1-m) spacings. Each slab thickness included three joint spacing, beginning with a spacing in feet that was double the slab thickness, and then including joint spacings that were 2 and 4 ft (0.6 and 1.2 m) longer (for example, the 6-inch [152-mm] slabs included 12-, 14-, and 16-ft [3.7-, 4.3- and 4.9-m] joint spacings).

- Dowel configuration—two dowel configurations were used, one a “standard” configuration in which the dowels are spaced at 12-inch (305-mm) intervals across the joint, and one a “special” configuration with a set of three dowels clustered in each wheelpath and spaced 12 inches (305 mm) apart. The dowels were 18 inches (457 mm) long and of diameter 0.75 inches (19 mm) for the 6-inch (152-mm) slabs and 1-inch (25 mm) for the 7- and 8-inch (177- and 203-mm) slabs.

Table 6-1 provides general information and pavement design details for the overlay design.

Table 6-1. U.S. 1 project details.

Key Information	Data
Project Location	U.S. 1 SB, Edgewater, FL Milepost 9.598 to 11.459 4-lane divided Highway
Climatic Region	Wet – Non-freeze
Highlighted Project Design Characteristics	Unbonded Concrete Overlay (on asphalt pavement)
Original Asphalt Pavement Thickness	5 inches
Initial Construction Year (original asphalt pavement)	Original Construction: ~1955 First Resurfacing ~ 1965
Rehab Types and Years	UBOL: 1988 (southbound lanes)
Traffic Data	2016 AADT: 6,620 (4.6% Trucks)
Primary UBOL Design Inputs	Cumulative Traffic Loading (ESALs): 1,160,000 PCC Flexural Strength: 500 lb/in ² PCC Elastic Modulus: 4,000,000 lb/in ² Modulus of subgrade reaction: 240 lb/in ² /in
Slab Design Details	Slab thickness: Variable (6, 7, and 8 inches) Joint spacing: Variable (12, 14, 16, 18, and 20 feet) Load transfer configuration: “standard” layout with 12 dowels per lane and “special” layout with 3 dowels clustered in each wheelpath Load transfer devices: 18-inch long dowel bars, 0.75-inch diameter for the 6-inch slabs and 1-inch diameter for the 7- and 8-inch slabs Joint Sealant: Silicone
Surface Preparation of Existing Asphalt Layer	4 inches of underlying asphalt layer milled; 1-inch asphalt levelling course added. Surface sprayed with lime slurry prior to UBOL placement
Base/Subbase (original construction)	8.5-inch limerock base
Subgrade	Granular subgrade
Subdrainage	4-inch diameter edge drains installed along outside shoulder as part of UBOL construction
Shoulder Type and Design	Asphalt paved shoulder
Unique/Noteworthy Features	Experimental UBOL with varying slab thicknesses, joint spacing, and load transfer configurations.

Preparation of Existing Asphalt Pavement

Prior to the UBOL construction, the original 5-inch asphalt pavement exhibited significant amounts of cracking and some rutting. The original asphalt was milled to a depth of 4 inches (102 mm), leaving on average a nominal asphalt thickness of 1 inch (25 mm). On top of that remaining material, a 1-inch (25 mm) leveling course was applied with the objective of improving the uniformity of surface condition along the length of the test section.

Construction

Immediately before the placement of the UBOL, the asphalt surface was sprayed with a lime slurry to reduce the temperature of the asphalt surface. A control section was included in this study, for which the entire thickness of the asphalt was milled, and the UBOL was constructed directly on top of the original base (in other words, the HMA leveling course was not provided on the control section).

As previously described, the load transfer system employed consisted of 18-inch (457 mm) long dowel bars installed in two configurations:

1. Standard configuration (ST): 12-inch (305 mm) spacing along entire length of the transverse joints, for a total of 12 dowels in each lane.
2. Special configuration (SP): three dowels along each wheel patch spaced at 12 inches (305 mm).

Dowels of 0.75-inch (19 mm) diameter were used for the 6-inch (152 mm) slabs, while dowels of 1-inch (25 mm) diameter were used for the 7 and 8-inch (178 and 203 mm) slabs.

FDOT considered a series of different slab layouts, thicknesses, and dowel bars configurations as project variables that were believed to potentially impact pavement performance. Three slab thicknesses, three different joint spacing per thickness, and two types of dowel configuration were considered to result in a total of eighteen test sections, six per each slab thickness, in addition to the control section (Section 0). Figure 6-2 shows the layout of the various test sections within the project limits.

1.86 miles

Section No.	Joint Spacing	Section Length	Dowel Configuration	
0	14 ft.	504 ft.	ST	7 in. JPCP Overlay
98 ft. TRANSITION				
1	12 ft.	504 ft.	SP	6 in. JPCP Overlay
2	14 ft.	504 ft.	SP	
3	16 ft.	496 ft.	SP	
4	12 ft.	504 ft.	ST	
5	14 ft.	504 ft.	ST	
6	16 ft.	496 ft.	ST	
90 ft. TRANSITION				
7	14 ft.	504 ft.	SP	7 in. JPCP Overlay
8	16 ft.	496 ft.	SP	
9	18 ft.	504 ft.	SP	
10	14 ft.	504 ft.	ST	
11	16 ft.	496 ft.	ST	
12	18 ft.	504 ft.	ST	
102 ft. TRANSITION				
13	16 ft.	496 ft.	SP	8 in. JPCP Overlay
14	18 ft.	504 ft.	SP	
15	20 ft.	500 ft.	SP	
16	16 ft.	496 ft.	ST	
17	18 ft.	504 ft.	ST	
18	20 ft.	500 ft.	ST	

1.86 miles

Figure 6-2. Section layout for concrete overlay on U.S. 1 (based on Greene et al. 2018).

Service Conditions

Annual traffic has increased considerably since the construction of the UBOL in 1988 (see Table 6-2). The total cumulative truck traffic since the construction of the UBOL is estimated to be over 2.3 million.

Table 6-2. Traffic data trend for SB U.S. 1 (based on Greene et al. 2018).

Year	Southbound Annual Trucks (x1000)	Southbound Cumulative Trucks (x1000)
1989	110	110
1990	63	173
1991	65	238
1992	67	305
1993	62	367
1994	70	437
1995	72	509
1996	74	583
1997	76	659
1998	78	737
1999	75	812
2000	62	874
2001	69	943
2002	90	1033
2003	74	1107
2004	91	1198
2005	115	1313
2006	112	1425
2007	109	1534
2008	92	1626
2009	85	1711
2010	84	1795
2011	84	1879
2012	83	1962
2013	87	2049
2014	89	2138
2015	98	2236
2016	111	2347

Project Characteristics

A 2018 field visit revealed that the overall project was in fair to good condition, but some of the slabs exhibited transverse cracking, corner breaks, and light to moderate joint faulting. Corner breaks were observed only on the thinner (6- and -7-inch [152- and 177-mm]) slabs, and the test section with the 20-ft (6.1-m) joint spacing exhibited the highest amount of cracking (Greene et al. 2018). Low- to moderate-joint seal damage was also observed throughout the section. Figure 6-3 shows an overview of site, and figures 6-4 through 6-6 show the typical site conditions.



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Figure 6-3. Overview of concrete overlay, SB U.S. 1 in 2018.



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Figure 6-4. Example transverse cracking on 20-ft panel observed in 2018.



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Figure 6-5. Minor faulting on 7-inch slab with special dowel configuration in 2018.



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Figure 6-6. Corner break on control section in 2018.

Pavement Performance

In 2015, FDOT performed a study to evaluate the performance of the UBOl using the performance indicators originally measured in 1988 (Greene et al. 2018). Deflection, roughness, structural capacity, load transfer efficiency (LTE), and distress data were analyzed. Additionally, coring was conducted to assess the interface bonding condition between the concrete slabs and the underlying asphalt layer.

For the bonding evaluation, a total of 19 cores were extracted in 2015, of which 10 were found to be completely debonded. For those cores that were retrieved in a bonded state, the average shear bond strength was 160 lb/in² (1.1 MPa), as measured using a shear test similar to Iowa Test Method 406C. No apparent pattern of debonding was attributed to slab thickness or dowel configuration, but cores extracted from the 20-ft (6.1-m) panels were more often found to be debonded as were cores taken along the slab corners (Greene et al. 2018).

Corner deflection data were studied to evaluate the effectiveness of the two different dowel configurations. While the thicker 8-inch (203 mm) slabs with the standard dowel configuration exhibited the lowest levels of deflection, the 6-in slabs showed the highest increase in corner deflection between 1988 and 2015. Table 6-3 summarizes the average corner deflection readings for all slab and dowel configurations between 1988 and 2015.

**Table 6-3. Corner deflection performance (based on Greene et al. 2018).
(SP=special configuration and ST= standard configuration).**

Configuration	Corner Deflection (mils)	
	1998	2015
SP-6in	18	25
SP-7in	20	20
SP-8in	20	22
ST-6in	17	23
ST-7in	14	17
ST-8in	12	11

SP = special configuration

ST = standard configuration

FDOT also investigated the load transfer efficiencies and differential deflections of the two configurations (see table 6-4). Deflection testing was performed at night along the outside wheel path, and it was found that the special configuration had consistently lower LTE than the standard arrangement. Differential deflection measurements were consistently higher for the slabs with the special dowel configuration.

Table 6-4. LTE testing results (based on Greene et al. 2018).

Configuration	LTE (percent)	Differential Deflection (mils)
SP-6in	48	6.7
SP-7in	46	7.5
SP-8in	43	6.1
ST-6in	72	3.4
ST-7in	78	2.2
ST-8in	86	0.8
Control (7-in)	64	3.0

SP = special configuration

ST = standard configuration

Detailed distress investigations and surface roughness surveys were also performed in 2015. Table 6-5 shows a summary of longitudinal, transverse, and corner cracks count from the 2015 distress survey, while table 6-6 presents the average IRI and faulting results for all sections.

Only 5 percent of the total number of slabs within the project limits exhibited cracks, with section 18 (8-inch [203-mm] slab with 20-ft [6.1-m] joints) being the most heavily distressed. The roughness survey assessed faulting and IRI. Sections with the special dowel configuration showed the steepest jumps in IRI values between 1988 and 2015, with a similar trend observed for faulting. The control section also experienced a significant increase in IRI, higher than any increase from other sections constructed with a standard dowel configuration. This may be attributable to the erodibility of the underlying limerock base and potentially other factors related to the foundation design.

Table 6-5. Crack survey summary for U.S. 1 (based on Greene et al. 2018).

Section ID	Percent Cracked Slabs (%)		
	Corner Cracks	Longitudinal Cracks	Transverse Cracks
0	-	6	-
1	10	-	-
2	2	-	-
3	-	7	3
4	5	2	5
5	-	-	-
6	14	-	-
7	2	-	-
8	-	-	-
9	-	-	-
10	2	-	-
11	7	7	-
12	3	-	-
13	-	-	-
14	-	-	-
15	-	-	4
16	-	-	-
17	-	-	3
18	4	8	8

Table 6-6. IRI and faulting for U.S. 1 (based on Greene et al. 2018).

Section ID	IRI (in/mi)		Fault Depth (inch)
	1988 Smoothness	2015 Smoothness	
0	75	126	0.049
1	73	90	0.059
2	76	150	0.055
3	74	105	0.038
4	73	95	0.04
5	73	70	0.034
6	73	80	0.049
7	76	145	0.067
8	77	140	0.062
9	75	85	0.045
10	74	74	0.035
11	75	87	0.04
12	73	73	0.036
13	80	160	0.058
14	82	162	0.057
15	73	90	0.042
16	73	78	0.034
17	74	74	0.037
18	72	75	0.04

Economic Analysis

Table 6-7 summarizes the agency costs for the unbonded concrete overlay that had been performed on the stretch on of U.S. 1 documented in this case study. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 6-7. Life-cycle costs for U.S. 1.

Year	Activity	Activity Detail	Cost (from DOT Contract)*
1955	Initial Construction	AC pavement	Not considered in analysis
1988	Concrete Paving	8-in UBOL	\$186,400
2028	End-of-Service	Assumed Pavement End of Service	NA

*Costs shown in then-current dollars

A 31-year analysis period was used (1988 to 2019) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service interval for the 1988 unbonded overlay was assumed to be 40 years.:

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 6-8 shows the construction costs converted to 1988 dollars.

Table 6-8. Agency costs for U.S. 1 converted to 1988 dollars.

Year	Activity	1988 Dollars
1988	8-inch UBOL	\$186,400
2019	Remaining Value (1988 UBOL)	(\$41,940)

Figure 6-7 shows the expenditure stream diagram for the U.S. 1 project using the four different discount rates used in the analysis.

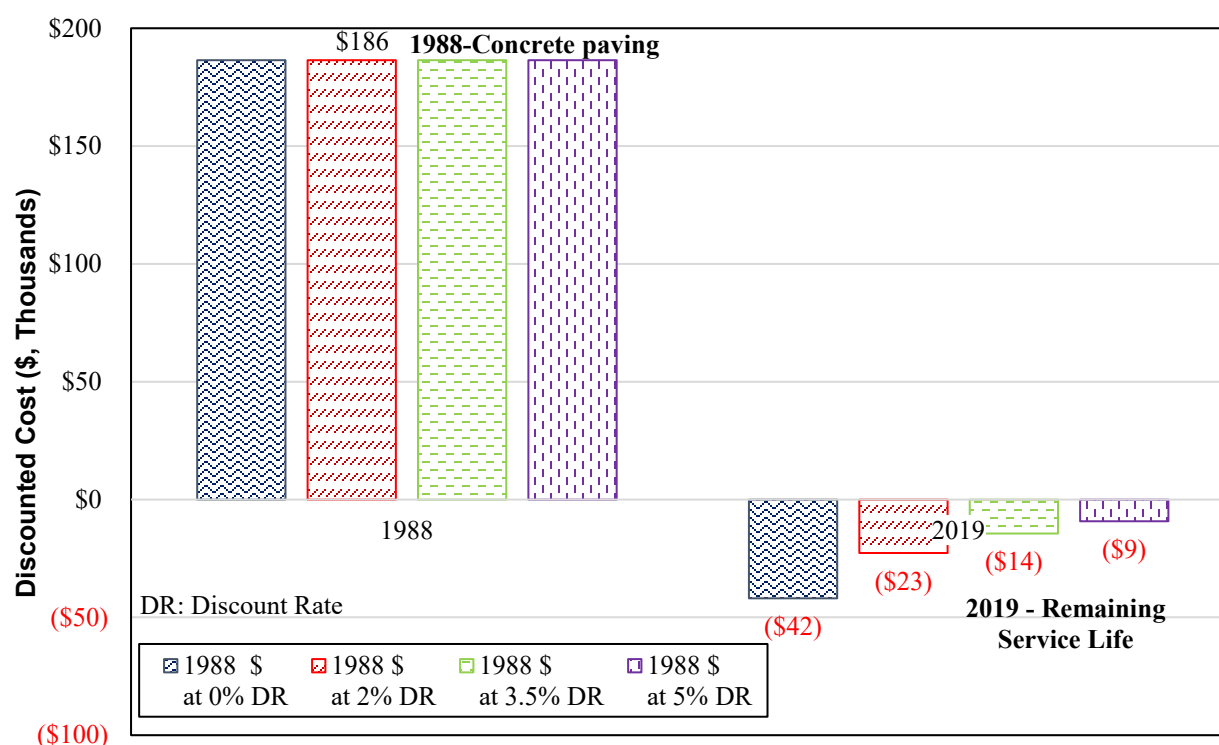


Figure 6-7. Expenditure stream diagram for U.S. 1.

Table 6-9 summarizes the LCCA results for each discount rate considered. As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) and equivalent uniform annual cost (EUAC) increases by 23 percent and 144 percent, respectively. This is graphically represented in Figure 6-8.

Table 6-9. LCCA summary for U.S. 1.

0% Discount Rate				
LCCA Element	1988 Dollars		2019 Dollars	
1988 UBOL	PV =	\$186,400	PV =	\$454,161
Remaining Value (1988 UBOL)	PV =	(\$41,940)	PV =	(\$102,186)
Totals	NPV =	\$144,460	NPV =	\$351,975
	EUAC =	\$4,660	EUAC =	\$11,354
2% Discount Rate				
LCCA Element	1988 Dollars		2019 Dollars	
1988 UBOL	PV =	\$186,400	PV =	\$454,161
Remaining Value (1988 UBOL)	PV =	(\$22,700)	PV =	(\$55,308)
Totals	NPV =	\$163,700	NPV =	\$398,854
	EUAC =	\$7,137	EUAC =	\$17,389
3.5% Discount Rate				
LCCA Element	1988 Dollars		2019 Dollars	
1988 UBOL	PV =	\$186,400	PV =	\$454,161
Remaining Value (1988 UBOL)	PV =	(\$14,437)	PV =	(\$35,176)
Totals	NPV =	\$171,963	NPV =	\$418,986
	EUAC =	\$9,178	EUAC =	\$22,362
5% Discount Rate				
LCCA Element	1988 Dollars		2019 Dollars	
1988 UBOL	PV =	\$186,400	PV =	\$454,161
Remaining Value (1988 UBOL)	PV =	(\$9,242)	PV =	(\$22,518)
Totals	NPV =	\$177,158	NPV =	\$431,644
	EUAC =	\$11,362	EUAC =	\$27,682
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost				

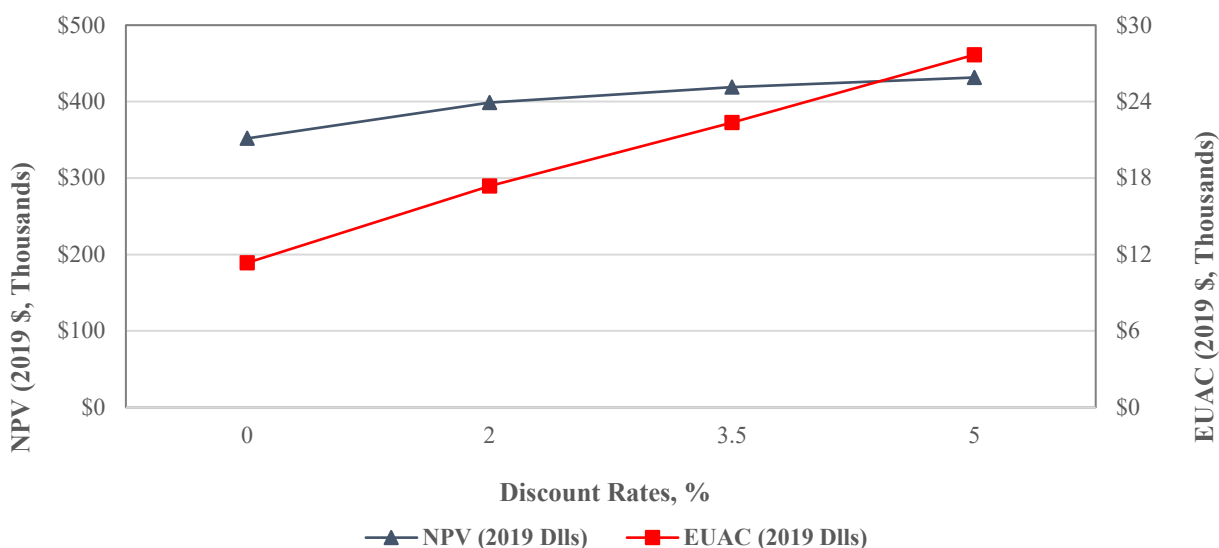


Figure 6-8. NPV and EUAC (2019 Dollars) vs discount rate.

Lessons Learned

In 1988 the FDOT rehabilitated a 1.86-mi (3-km) long asphalt pavement section on U.S. 1 in Edgewater, FL, by constructing an unbonded concrete overlay. As part of the study, FDOT constructed the test section with multiple slab thicknesses, joint spacings, and dowel configurations with the purpose of studying the effect of these variables on pavement performance. After 30 years in service, the test section shows acceptable levels of distresses and good rideability, particularly for those sections constructed with the standard dowel configurations. Lessons learned from this study are summarized below.

- **If properly designed and constructed, unbonded concrete overlays of existing asphalt pavements can provide long service lives.** After 28 years of service, only a small fraction of the slabs (5 percent; 33 out of the total 612 slabs) in the experimental section on U.S. 1 exhibited some form of cracking in the in 2015. Section 18, an 8-inch (203-mm) slab with 20-ft (6.1-m) joint spacing, exhibited the highest level of cracking (20 percent; 5 slabs out of 25). No correlations could be discerned between the experimental variables and the distress density measured in 2015.
- **Dowel configuration substantially influences the load transfer efficiency and deflections.** In general, corner deflections were higher on sections constructed with the special dowel configuration when compared to those with the standard configuration.
- **Base layer type and dowel configuration have a significant impact on pavement smoothness.** While the sections with the special dowel configuration exhibited more than a 60 percent increase in IRI values compared to 1988 values, the sections with the standard dowel configurations showed only about a 15 percent increase. On the control section where the UBOL was placed directly on the limerock base layer, the IRI values increased by around 75 percent compared to the 1988 values.

CHAPTER 7. CONCRETE PAVEMENT REHABILITATION AND BONDED CONCRETE OVERLAY OF U.S.-281 IN TEXAS

Introduction

In 2002, the Texas Department of Transportation (TX DOT) rehabilitated a 3.693-mi (5.94-km) section of U.S. 281 in Wichita County from near the Wichita-Archer county line to the end of Holliday Creek bridge (see figure 7-1). Originally built in 1969 as an 8-inch (203 mm) continuously reinforced concrete pavement (CRCP) with a 20-year design life, the section received no significant maintenance or rehabilitation until 2002 (after 33 years of service), when some full-depth repairs were performed on the existing CRCP prior to placement of a 4-inch (100-mm) latex-modified, continuously reinforced bonded concrete overlay (BCO). A few repairs to the overlay have been performed since that time. Reasons for choosing to use a bonded concrete overlay on this project included the following:

- It was determined that either major rehabilitation or reconstruction was needed.
- Falling weight deflectometer (FWD) data indicated the subgrade was in good condition.
- TxDOT decision-makers were actively introducing new technologies, including the use of the first latex-modified concrete overlay of bridge deck on this same project (source: 9/18/19 e-mail from TxDOT project staff).

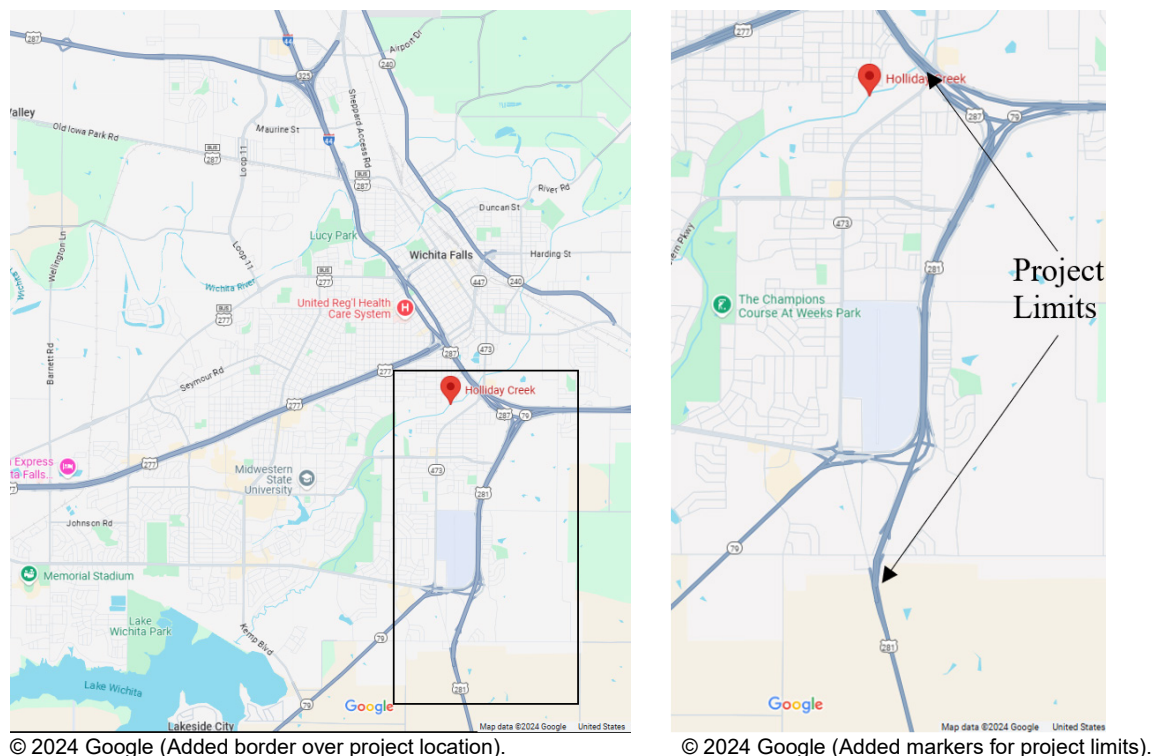


Figure 7-1. U.S.-281 project location and limits.

Service Conditions

The project is located on the southeast side of Wichita Falls; the weather in the area is relatively dry (<30 inches [76 cm] precipitation annually) with freeze-thaw cycling during the coldest months.

Traffic was estimated at 20,800 vehicles/day (two-way) in 2002 and was projected to increase to 28,000 by 2022, but 2017 traffic was reported as 31,178 vehicles/day with 10.7 percent trucks.

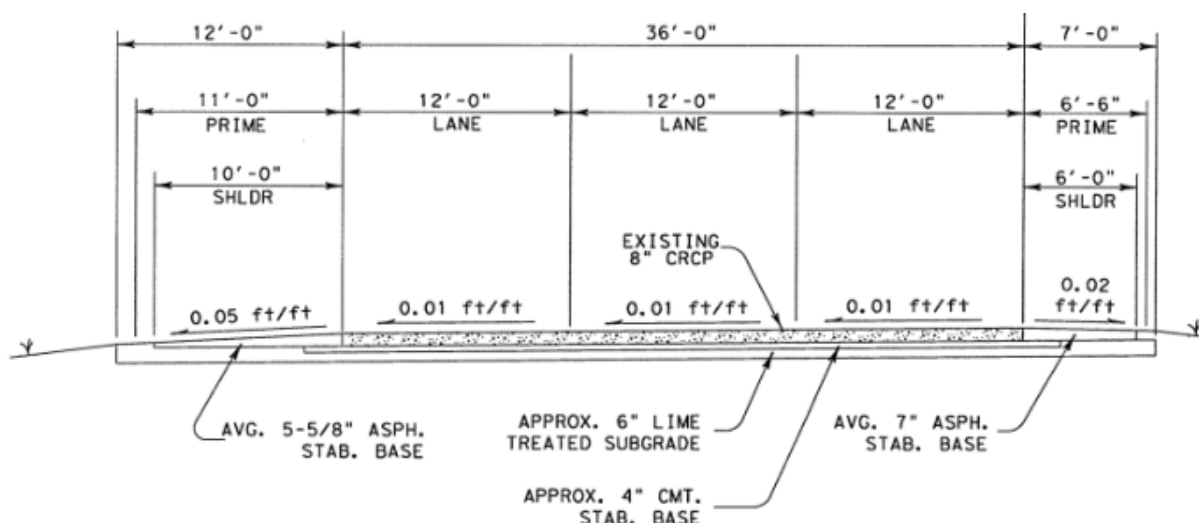
Original Pavement Design and Materials

Structural Design

The original pavement was constructed in 1969 as an 8-inch (200-mm) CRCP on a 4-inch (100-mm) cement stabilized-base, 6-inch (150-mm) lime-treated subgrade and natural subgrade (sandy loam and clay loam). Information concerning the pavement reinforcing quantity and details is not available. Table 7-1 summarizes general project design, location, and traffic information for the original and rehabilitated concrete pavement. Figure 7-2 shows a typical 3-lane section for the original pavement (2-lane sections were similar).

Table 7-1. Summary of U.S.-281 project information.

Key Information	Data
Project Location	U.S.-281 NB and SB, Station 28+00 to 223+00 (3.7 centerline miles) Wichita County, TX (wet-freeze climate) Project limits: North: Holliday Creek bridge South: Wichita-Archer county line
Highlighted Project Characteristics	Full-depth repair of CRCP; Bonded CRCP overlay of CRCP
Original Construction Overview	1969 construction; 4- and 6-lane divided freeway • Pavement: 8-in CRCP (limestone coarse aggregate, 0.6% longitudinal steel); 6-in CRCP in ramp areas • Base/Subbase: 4-in cement-stabilized base over 6-in lime-treated subgrade • Subgrade: sandy loam and clay loam • Subdrainage: None • Shoulders: HMAC (sealed lane-shoulder joints)
Rehabilitation	2002 construction: • Full-depth CRCP repair • 4-in latex-modified bonded CRCP overlay of CRCP (0.61% longitudinal steel) • Tapered thickness HMAC shoulder overlay
Traffic Data	Two-way AADT: est. 20,800 (2002); 31,178 (2017) – 10.7% trucks
Unique/Noteworthy Features	Pre-overlay surface preparation: shot-blasting, no bonding agent Latex-modified concrete overlay Continuously reinforced concrete overlay Lane-at-a-time construction



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Figure 7-2. Typical 3-lane typical section for U.S.-281 original (1969) construction.

Mix Design

No information is available on the mix design used in the original 1969 construction except that the coarse concrete aggregate was limestone.

Rehabilitation and Bonded Concrete Overlay Design, Materials and Construction

The following sections describe pavement rehabilitation and overlay work that was performed during the summer of 2002. No other significant maintenance or rehabilitation work is believed to have been performed on this section of U.S. 281 prior to the work described herein.

Work was performed one lane at a time, first in the southbound lanes, then the northbound lanes. The work zone was kept relatively small; work on any area of the pavement was to be completed within 24 hours. Thus, pre-overlay repairs and overlay preparation work (i.e., shot-blasting, placement of reinforcing steel, etc.) were done between 6 a.m. and 7 p.m. daily over an area no greater than what could be paved that same night. Bonded concrete overlay placement was done between 7 p.m. and 6 a.m. daily.

Pre-Overlay Repairs

Full-depth repairs (9721 s.y.) of the existing concrete pavement were completed prior to the placement of the BCO. Repairs were performed according to Items 360 ("Concrete Pavement"), 361 ("Repair of Concrete Pavement"), 421 ("Hydraulic Cement Concrete") and 440 ("Reinforcement for Concrete") of the Texas DOT Standard Specifications for Construction (TxDOT 2004) and applicable Departmental Materials Specifications. The specified minimum allowable repair dimension was 12 ft by 15 ft (3.7m by 4.6m), but some smaller repairs may have been placed.

Full-depth saw cuts were made around along the perimeter of each area to be repaired, including cuts made perpendicular to the pavement edge and centerline, forming rectangular repair areas. Tie bar holes (to accommodate #6 [19-mm diameter] tie bars) were drilled to a minimum depth of 12 inches [300 mm] on 12-inch [300-mm] centers at approximately mid-depth along transverse repair boundaries. Tie bars were grouted into the drilled holes and extended at least 25

inches [630 mm] into the repair area. Longitudinal repair steel also #6 [19-mm diameter] deformed bars spaced at 12 inches [300 mm] and tied to the tie bars. Transverse repair steel was #5 [16-mm diameter] deformed bar placed on top of and tied to the longitudinal steel in a single layer. Reinforcing bars lengths were nominally 2 inches [50mm] less than the repair length or width that the bar was to span. The repairs were constructed using materials and practices conforming to TxDOT standard specifications.

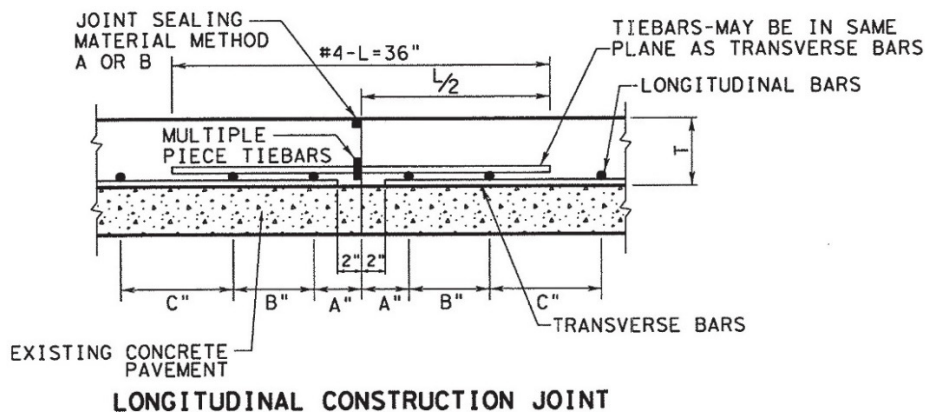
Concrete Overlay

The surface of the repaired original pavement was cleaned and textured using a shot-blast machine (see figure 7-3). The surface was air-blasted again and moistened (targeting a saturated, surface-dry condition) just prior to concrete placement. No bonding agent was used.



Figure 7-3. Shotblasting operation (left) and resulting surface texture (right) (Won 2016).

Transverse deformed #4 [13-mm diameter] reinforcing bars were placed on 18-inch [460-mm] centers directly on the shot-blasted surface; chairs were not allowed. Longitudinal deformed #4 [13-mm] reinforcing bars were placed on no less than 8-inch [200-mm] centers directly on top of the transverse bars, with the distance to the first bar from longitudinal edges or joints being set at 3-to-4 inches [75 – 100 mm]. Multi-piece tie bars were used at longitudinal construction joints between travel lanes (see figure 7-4). Additional longitudinal reinforcing was used at transverse construction joints.



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Figure 7-4. Longitudinal construction joint detail showing overlay steel placement and multi-piece tie bars (TxDOT 2002).

Latex-modified concrete was selected for use for many reasons, including relative impermeability, high tensile bond strength, and reduced shrinkage characteristics. Steel fiber reinforcing was used in the 2-inch [50-mm] bridge deck overlays to provide additional tensile strength and resistance to early-age cracking (early opening to traffic) and improved durability. The concrete mixture proportions (per cubic yard [per cubic meter]) were reported as (Beer 2002):

- 658 lbs [391 kg] cement
- 1777 lbs [1057 kg] ½-inch [13-mm] nominal top size coarse aggregate (limestone)
- 1184 lbs [704 kg] #8 [2.36-mm] sieve fine aggregate (granite wash)
- 40 lbs [24 kg] steel fibers (bridge deck overlay only)
- Air content: 2.0 – 3.0 percent

Concrete was mixed, delivered, and placed by transit-mixer trucks. Slump and mixture temperature at placement were 7-8 inches [180 - 200 mm] and 85 - 94°F [29 - 34°C], respectively. Paving was performed using a dual string line-controlled slip-form paving machine (except on the bridge decks, which were hand placements). Finishing was done using a 16-ft [4.9-meter] straightedge, with final finish performed using a full float and trowels at the edge. Surface texture was created using an inverted turf drag (longitudinal) and transverse tining. The surface was cured for 24 hours with wet burlap and clear polyethylene sheeting.

All project work (including 3.452 centerline miles of roadway plus 12 bridge structures) was completed in 50 days. Figure 7-5 presents a typical section for the overlaid pavement in areas with three travel lanes.

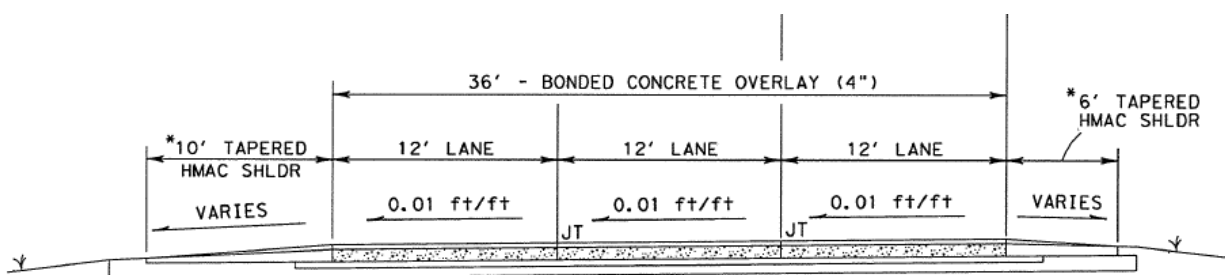


Figure 7-5. Three-lane typical section with bonded overlay for U.S.-281.

Pavement Performance and Maintenance History

2005 Study

The performance of the BCO was investigated in 2005 as part of a larger study of thin concrete overlay performance in Texas performed by researchers from Texas Tech University (Trevino, Medina-Chavez, and Won 2005). Fieldwork for this study took place in June 2005 and included a visual survey, sounding (delamination testing), falling weight deflectometer (FWD) testing and extraction of core samples. The evaluation area was approximately one mile (1.6 km) in length and was located in the inside lane of the southbound direction.

The 2005 evaluation found that this pavement section was generally in very good structural and functional condition, with no major distress. All FWD deflections were low (averaging 2.80 mils

[71 μm]) except for one test (6.2 mils [160 μm] performed in an area with more deteriorated cracking). Eight of nine cores retrieved were in good condition (see figure 7-6). One core was taken from a small area near the outside lane-shoulder joint where sounding tests indicated possible delamination; the core revealed voids and deterioration in the original pavement at the level of the reinforcing steel and delamination between the overlay and original pavement, although the overlay concrete was in good condition (see figure 7-6).



Figure 7-6. Photos of typical U.S.-281 core in good condition (left) and deteriorated and delaminated core (right). (Trevino, Medina-Chavez, and Won 2005)

Strength tests performed on the cored material produced compressive strengths ranging from 5,364 to 6,426 psi [37.0 to 44.3 MPa] and elastic modulus values averaging 4,818 ksi [33.2 GPa] (outlier core excluded). Coefficient of thermal expansion (CTE) testing indicated a value of $4.23\mu\text{ε}/^{\circ}\text{F}$ [$7.6\mu\text{ε}/^{\circ}\text{C}$]. Petrographic examination of cores found no evidence of ASR or other chemical reactions. (Trevino, Medina-Chavez, and Won 2005).

2016 Study

Researchers from Texas Tech University visited the project site again in March of 2016 and performed visual surveys along with deflection and multi-transducer ultrasonic scanning along a 0.1-mile [0.16-km] section of the southbound project lanes between reference markers 194+0.3 and 194+0.4 (Choi et al. 2018). The performance of the then-14-year-old overlay was rated as “Excellent” with no distresses observed except at some areas of transverse construction joints (TCJ) in the BCO.

Figure 7-7 shows the FWD deflections at TCJs were much larger than deflections in other areas (Choi et al. 2018). Ultrasonic scanning detected delamination between the overlay and original pavement and also within the existing CRCP (near mid-depth, along the plane of the reinforcing steel – see figure 7-8). The delamination and mid-depth cracking disappeared a short distance away from the TCJ (see figure 7-9).

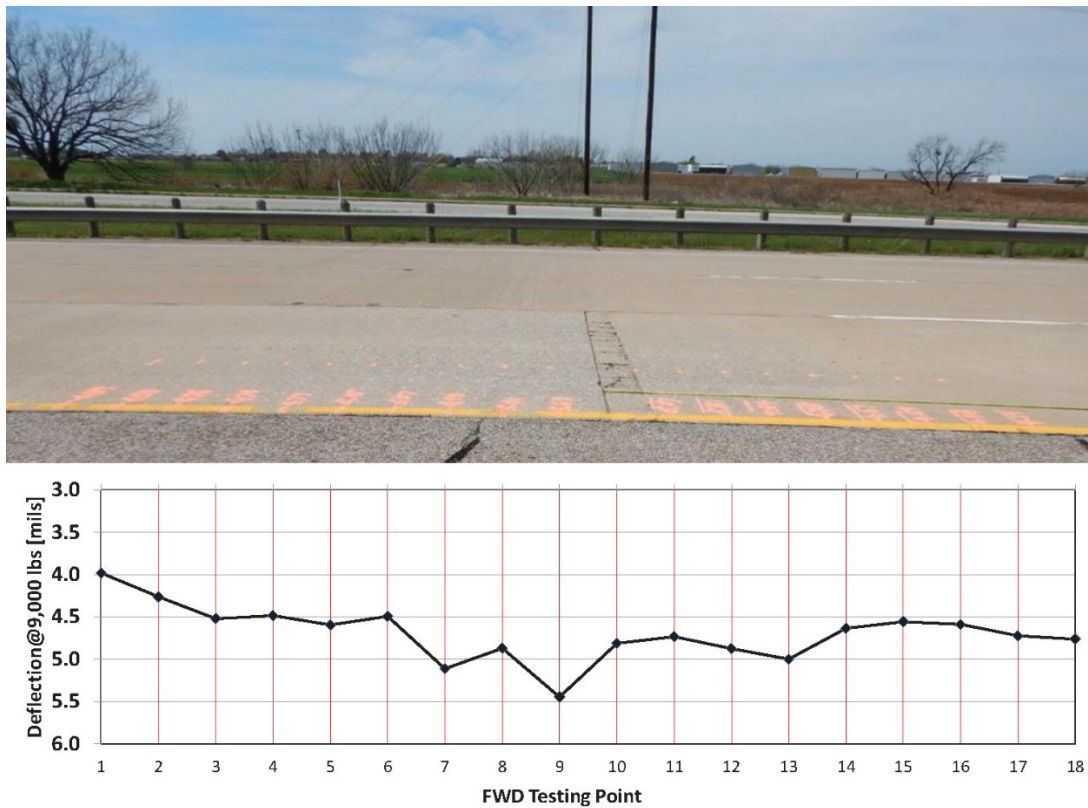


Figure 7-7. Photo of deterioration near BCO transverse construction joint and associated FWD deflection test results (Choi et al. 2018).

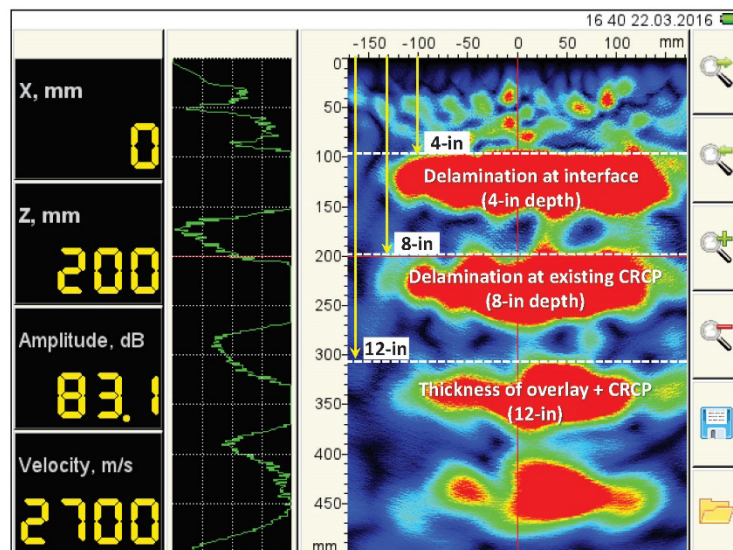


Figure 7-8. Ultrasonic test image near BCO transverse construction joint (Choi, et al., 2018).

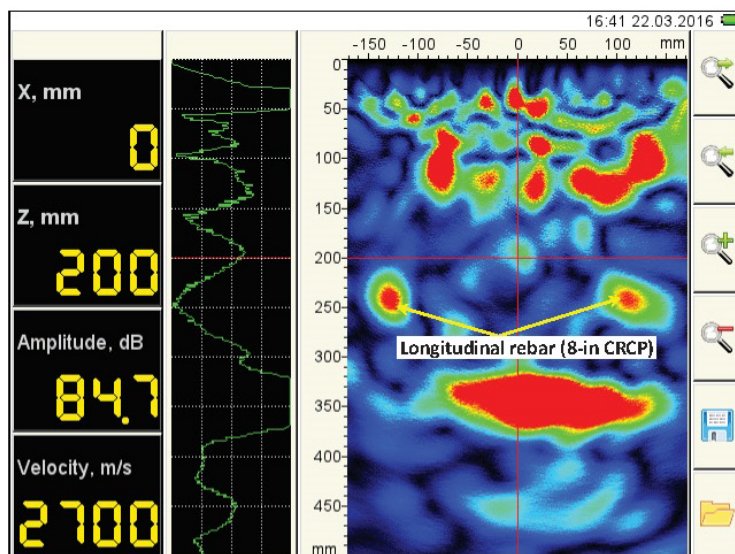


Figure 7-9. Ultrasonic test image away from BCO transverse construction joint (Choi et al. 2018).

Cores were retrieved from various locations to evaluate the strength of the bond between the overlay and original pavement. Bond strength testing was performed in accordance with ASTM C1583 (Pull-Off Method). Bond strength ranged from 126 to 254 psi [0.869 to 1.75 MPa], with failures developing within either the BCO or the original pavement in all cases; failure did not develop at the BCO-pavement interface in any test (Choi et al. 2018).

Slab support conditions were evaluated through drilled core holes and were found to be somewhat lower than those observed at other BCO projects in Texas. The researchers hypothesize that the satisfactory performance of this section, in spite of the reduced slab support conditions, may be due to the relatively low truck traffic on this section (Choi et al. 2018).

2018 Site Visit

A project team member visited the project site with TxDOT staff in December 2018. The project was mostly in excellent condition after 16 years of service. Figure 7-10 presents representative photos of typical project conditions.

A few areas of localized repair or distress were observed:

- A few wide transverse cracks (mainly in the outer lane of 2-lane sections and the middle lane of 3-lane sections), as shown in figure 7-11.
- Maintenance (patching) performed in 2016 at some bridge approach transitions (see figure 7-12).
- Two patches placed in 2018 on the southbound ramp to Hatton Road (see figure 7-13).
- Occasional concrete deterioration around transverse construction joints (see figure 7-14).
- A few small areas of concrete deterioration (possible loss of bond and breakup of overlay - see figure 7-15).



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Figure 7-10. Typical 2018 conditions on U.S. 281 BCO project.



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Figure 7-11. Typical wide (deteriorating) transverse cracks.



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Figure 7-12. Typical 2016 patching at bridge approach transitions.



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Figure 7-13. Patch at the southbound exit to Hatton Road.



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Figure 7-14. Examples of overlay deterioration around transverse construction joints.



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Figure 7-15. Areas of overlay deterioration, possibly indicating localized loss of bond.

TxDOT's Pavement Management Information System (PMIS) provides a summary of visual distress and ride quality information that is collected annually on 0.5-mile [0.8-km] segments of the pavement network using automated equipment operating at highway speeds. Ride quality data is collected from both wheel paths and is based on the International Roughness Index (IRI) but is converted into a Serviceability Index (SI) that ranges from 0 (worst) to 5 (best). Distress counts in each survey segment are used to compute a segment Distress Score that ranges from 0 (worst) to 100 (best – no distress). Finally, a Condition Score ranging from 0 (worst) to 100 (best) is computed based on the Distress Score and the Utility Factor for Ride Quality, a value ranging from 0 to 1 and based on SI and segment traffic level (Goehl 2013). Plots of Serviceability Index, Distress Score and Condition Score for four representative sections of U.S. 281 are presented in figures 7-16, 7-17, and 7-18, respectively.

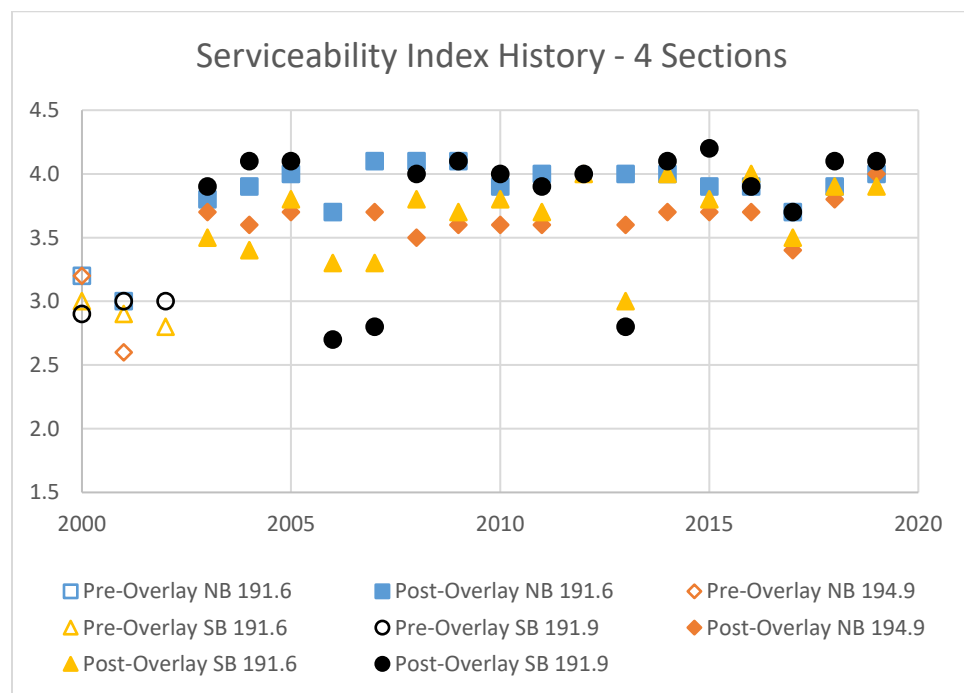


Figure 7-16. Serviceability index history (2000 – 2019) for four representative sections of U.S. 289 (data source: TxDOT PMIS).

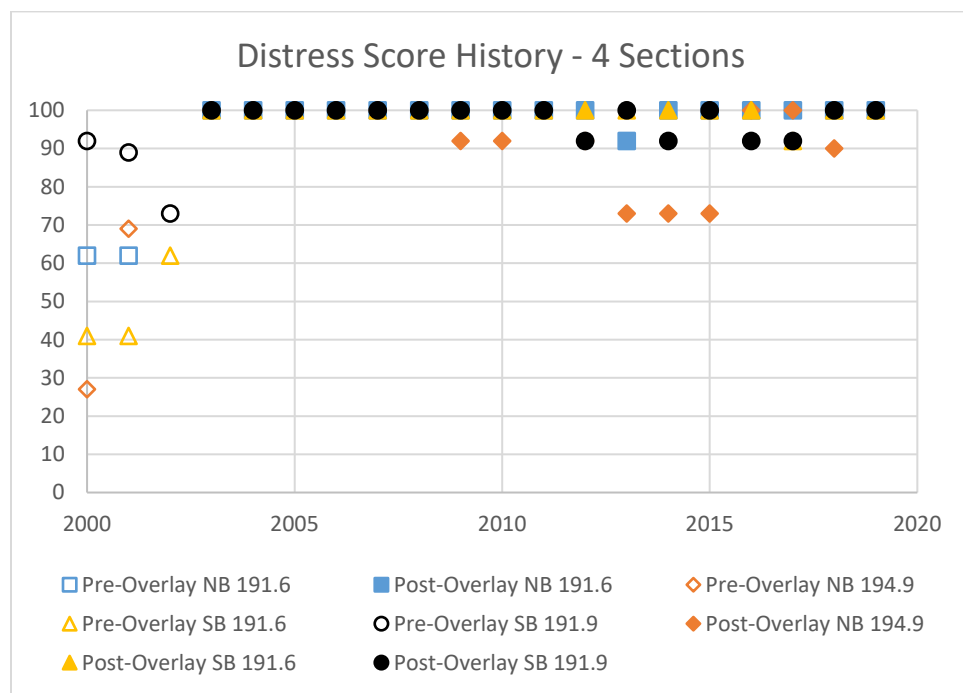


Figure 7-17. Distress score history (2000 – 2019) for four representative sections of U.S. 289 (data source: TxDOT PMIS).

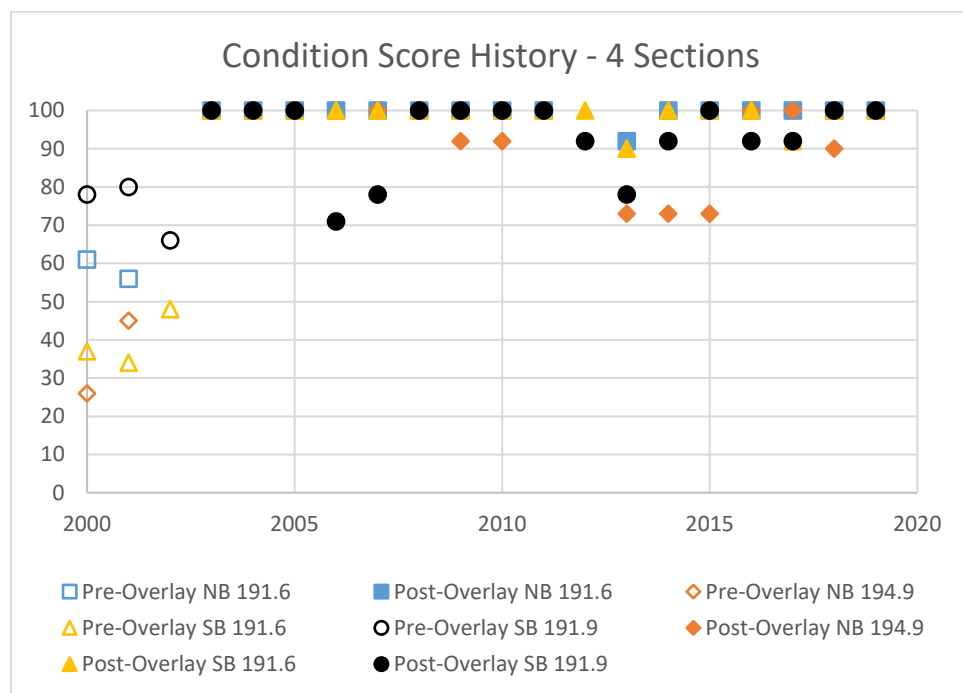


Figure 7-18. Condition score history (2000 – 2019) for four representative sections of U.S. 289 (data source: TxDOT PMIS).

Figures 7-16 through 7-18 show the distinct improvement in pavement ride index, distress score and condition score following the 2002 bonded overlay construction. Slight improvements in all three measures are also apparent after the 2016 CPR project. What is most evident is the consistently high level of service and low levels of distress present on this project since the 2002 overlay construction; very little deterioration in pavement condition or ride quality has been observed.

Economic Analysis

Table 7-2 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of U.S.-281 documented in this case study. The costs corresponding to the rehabilitation activities were obtained from TXDOT's Wichita Falls District Office. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 7-2. Project cost data for U.S.-281.

Year	Activity	Activity Detail	Cost (Contract)
1969	CRCP Construction	8-in PCC Paving	\$4,215,617 ^{1,3}
2002	Full-depth repairs	Full-depth CRCP repairs 9721 s.y. [8128 m ²]	\$707,689 ³ (\$72.80/s.y. [87.21/m ²])
2002	Overlay paving	4-inch fiber-reinforced, latex-modified, bonded CRCP overlay 121,335 s.y. [101,281 m ²]	\$2,838,026 ³ (\$23.39/s.y. [\$28.02/m ²])
2002	Bridge deck paving**	Surface prep and 2-inch LMC overlay 13,206 s.y. [11,041 m ²]	\$620,020 ³ (\$46.95/s.y. [\$56.25/m ²])
2002	Other bid items	Including 72,869 l.f. [22,211 meters] joint and crack sealing, 8299 tons [7545 tonnes] HMAC, drill seeding and watering, detour construction, traffic control, raised and reflective pavement markers, pavement marking, guardrail, etc.	\$1,495,155 ³
2016	Full-depth repairs	Full-depth repairs	\$100,000 ^{2,3}
2022	Full-depth repairs	Full-depth repairs	\$29,684 ⁴
2032	End-of-Life	Assumed Pavement End of Service	\$0

¹Original construction cost estimated using an average 8-in CRCP paving unit cost and adjusted to then-current dollars using the National Highway Construction Cost Index (NHCCI). The paving quantity was assumed to be the same as the bonded concrete overlay, and the total paving cost to represent 14 percent of the total project.

²Cost estimated based on input from TxDOT.

³Cost in then-current dollars.

⁴Cost in 1969 dollars.

**Excluded from LCCA

A 63-year analysis period was used (1969-2032) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 7-2 are summarized below:

- 2002 CPR: 14 years (end-of-service: 2016)
- 2002 Concrete Overlay Paving: 30 years (end-of-service: 2032)
- 2002 Other Bid items: 14 years (end-of-service: 2016)
- 2016 CPR: 16 years (end-of-service: 2032)
- 2022 CPR: 10 years (end-of-service: 2032)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 7-3 shows the construction costs converted to 1969 dollars.

Table 7-3. Agency costs for the U.S.-281 project converted to 1969 dollars.

Year	Activity	1969 Dollars
1969	CRCP Construction	\$4,215,617
2002	Full-depth repairs	\$148,418
2002	Overlay paving	\$595,196
2002	Other bid items	\$313,567
2016	Full-depth repairs	\$12,642
2022	Full-depth repairs	\$29,684
2032	Remaining Value (1969 CRCP Construction)	(\$0)
2032	Remaining Value (2016 Full-depth repairs)	(\$0)

Figure 7-19 shows the expenditure stream diagram for the U.S.-281 project using the four different discount rates used in the analysis.

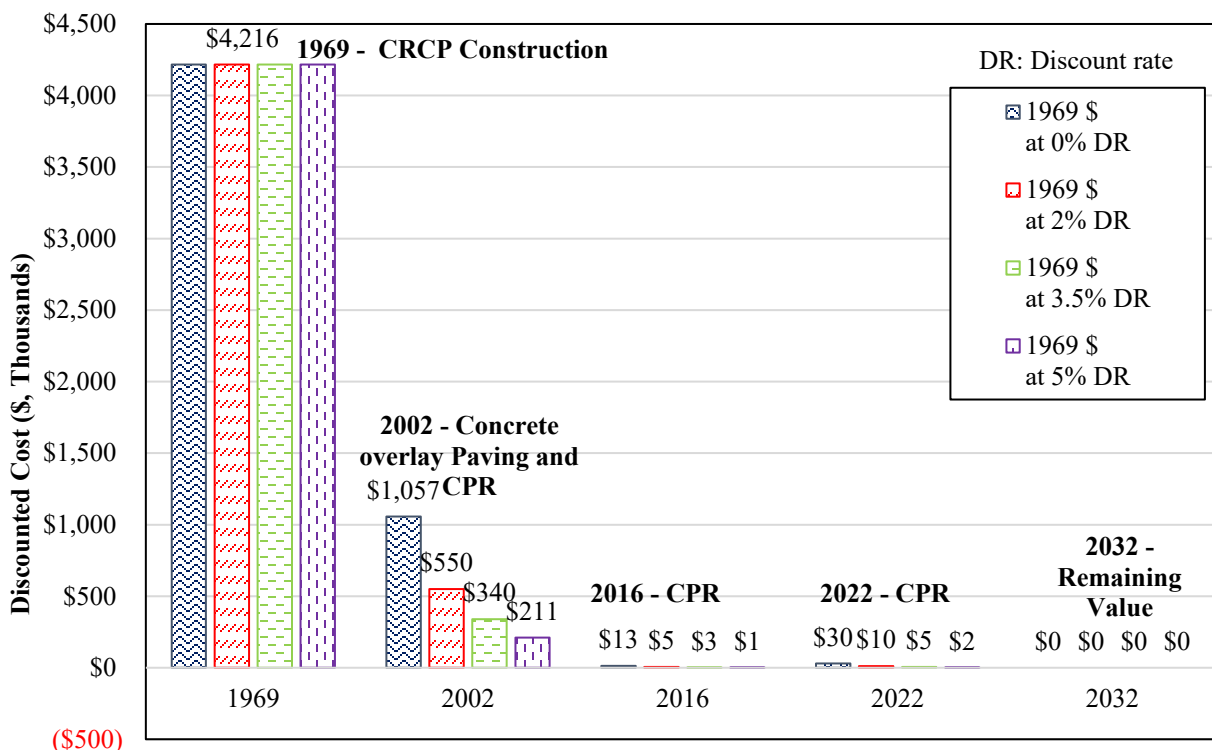


Figure 7-19. Expenditure stream diagram for U.S.-281.

Table 7-4 summarizes the LCCA results for each discount rate considered

Table 7-4. LCCA summary for U.S.-281.

0% Discount Rate				
LCCA Element	1969 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$4,215,617	PV =	\$35,299,695
2002 CPR	PV =	\$148,418	PV =	\$1,242,784
2002 Concrete overlay paving	PV =	\$595,196	PV =	\$4,983,905
2002 Other bid items	PV =	\$313,567	PV =	\$2,625,667
2016 CPR	PV =	\$12,642	PV =	\$105,855
Remaining Value (1969 CRCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2016 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$5,315,122	NPV =	\$44,506,462
	EUAC =	\$84,367	EUAC =	\$706,452
2% Discount Rate				
LCCA Element	1969 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$4,215,617	PV =	\$35,299,695
2002 CPR	PV =	\$77,211	PV =	\$646,532
2002 Concrete overlay paving	PV =	\$309,638	PV =	\$2,592,770
2002 Other bid items	PV =	\$163,126	PV =	\$1,365,947
2016 CPR	PV =	\$4,984	PV =	\$41,735
Remaining Value (1969 CRCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2016 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$4,780,969	NPV =	\$40,033,700
	EUAC =	\$134,147	EUAC =	\$1,123,285
3.5% Discount Rate				
LCCA Element	1969 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$4,215,617	PV =	\$35,299,695
2002 CPR	PV =	\$47,693	PV =	\$399,360
2002 Concrete overlay paving	PV =	\$191,262	PV =	\$1,601,541
2002 Other bid items	PV =	\$100,762	PV =	\$843,739
2016 CPR	PV =	\$2,510	PV =	\$21,014
Remaining Value (1969 CRCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2016 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$4,562,637	NPV =	\$38,205,490
	EUAC =	\$180,339	EUAC =	\$1,510,077
5% Discount Rate				
LCCA Element	1969 Dollars		2019 Dollars	
1969 Original Construction	PV =	\$4,215,617	PV =	\$35,299,695
2002 CPR	PV =	\$29,665	PV =	\$248,398
2002 Concrete overlay paving	PV =	\$118,963	PV =	\$996,146
2002 Other bid items	PV =	\$62,673	PV =	\$524,799
2016 CPR	PV =	\$1,276	PV =	\$10,686
Remaining Value (1969 CRCP)	PV =	(\$0)	PV =	(\$0)
Remaining Value (2016 CPR)	PV =	(\$0)	PV =	(\$0)
Totals	NPV =	\$4,430,430	NPV =	\$37,098,448
	EUAC =	\$232,263	EUAC =	\$1,944,865
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost				

As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 17 percent, and the equivalent uniform annual cost (EUAC) increases by 175 percent (see figure 7-20). These results demonstrate the importance of selecting an appropriate discount rate.

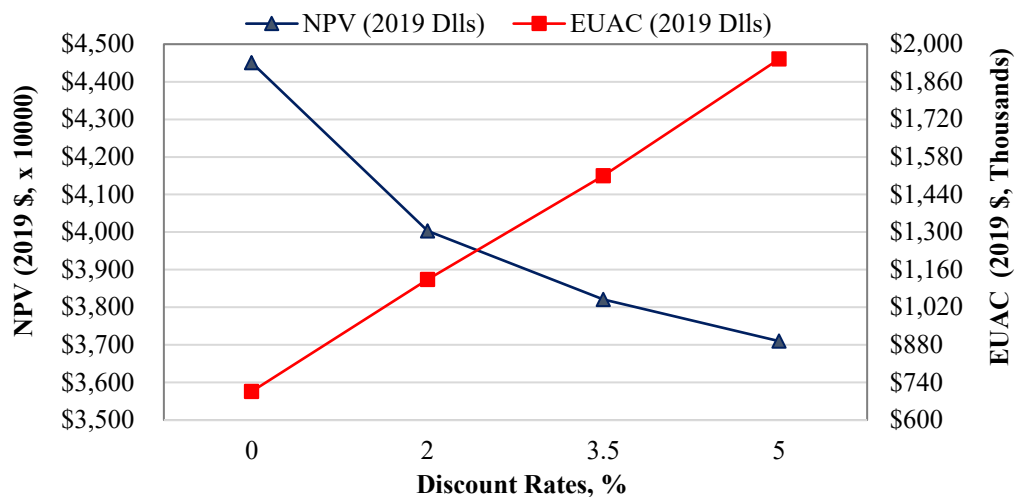


Figure 7-20. NPV and EUAC (2019 Dollars) vs discount rate

Summary and Lessons Learned

U.S.-281 between the Wichita-Archer county line and Holliday Creek Bridge is a 1969 CRCP that needed little or no maintenance prior to a 2002 rehab and overlay with bonded, latex-modified CRCP. The original pavement far exceeded its 20-year design service life and the bonded overlay has provided overall excellent performance for more than 16 years with no significant maintenance or rehabilitation until recently. Recent repairs have been primarily at paving discontinuities (i.e., bridge approaches, transverse construction joints, etc.). Based on the LCCA conducted, future maintenance costs are estimated to be between 5 and 21 percent of the original CRCP construction cost, based on the discount rate used.

- **The original CRCP provided exceptional performance with little, if any, maintenance.** Factors in this success obviously include the relatively low traffic levels and mild climate, but also include key design features, such as the use of a jointless pavement system, low concrete coefficient of thermal expansion, and a good pavement foundation (2 layers of stabilized material over a uniform loamy subgrade).
- **The bonded concrete overlay (with pre-overlay repairs) greatly extended the life of the original pavement, providing excellent ride quality with minimal maintenance.** Bonded concrete overlays of concrete pavement offer potential for tremendous increases in pavement structural capacity for relatively little cost. If bond between the overlay and original pavement is lost, the overlay can deteriorate quickly, as was observed in a few small, localized areas of this project.
- **This project demonstrated that significant extensions in concrete pavement life can be achieved using bonded concrete overlays at relatively low unit costs.** The 24-hour rolling, lane-at-a-time closure approach used on this project also demonstrated that concrete overlays can be constructed quickly and with relatively little impact to the traveling public.

CHAPTER 8. CONCRETE PAVEMENT REHABILITATION ON U.S. 20 IN IOWA

Introduction

In 2014, the Iowa Department of Transportation (Iowa DOT) rehabilitated a 3.8-mi (6.1 km) stretch of U.S. 20 (eastbound lanes) in Hamilton County, approximately 60 miles (96 km) north of Des Moines, Iowa (see figure 8-1). The project boundaries are between the intersections with U.S. 69 and I-35. Originally built in 1968 as a 10-inch (254-mm) undoweled jointed plain concrete pavement (JPCP) with 20-ft (6.2-m) transverse joint spacing and 12-ft (3.7 m) wide lanes, the section received its first rehabilitation in 2003 in the form of a 3.5-inch (89 mm) hot-mix asphalt (HMA) overlay to address surface roughness issues. However, the development of severe joint reflection cracking on the HMA layer motivated the Iowa DOT to consider another rehabilitation in 2014. At that time, the HMA layer was completely removed to expose the original concrete pavement followed by a dowel bar retrofit (DBR) installation to restore transverse joint load transfer. The entire surface was then diamond ground to produce a smooth riding surface.

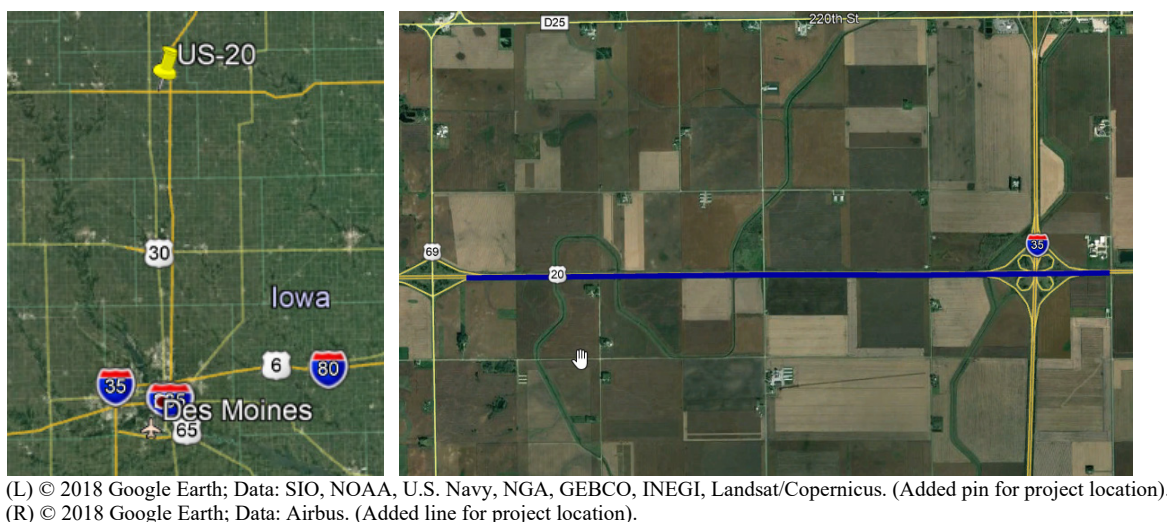


Figure 8-1. U.S. 20 project location.

Original Pavement Design

Structural Design

As described above, the original pavement was constructed in 1968 as a 10-inch (254 mm) undoweled JPCP on a 4-inch (102 mm) uniform granular aggregate base course. The transverse joints were spaced at 20-ft (6.1 m) intervals. Table 8-1 provides general information and pavement design details for the original and rehabilitated concrete pavements.

Mix Design

Table 8-2 provides information on the basic mix design used on the project. The water-cement ratio of the mix was approximately 0.43. The average fresh concrete air content based on 70 tests was reported to be 6 percent.

Table 8-1. U.S. 20 project details.

Key Information	Data
Project Location	U.S. 20 EB/WB, Hamilton County, IA* Milepost 149.50 to 153.47 4-lane divided freeway
Climatic Region	Wet-freeze
Highlighted Project Design Characteristics	Concrete Pavement Rehabilitation: Dowel bar retrofit and diamond grinding
Initial Construction Year	1968
Rehabilitation Types, Years, and Details	1991: 4-inch diameter longitudinal subdrains with 6-inch diameter outlets at 500-ft spacings 2003: 3.5-inch HMA overlay (EB and WB), with “Type B” granular shoulder 2014: HMA overlay removal, dowel bar retrofit, and diamond grinding (EB only) Dowel Bar Retrofit (DBR): Two sets of three 1.5-inch diameter epoxy-coated dowel bars clustered in the wheelpath on 12-inch spacings. Diamond Grinding: Land between grooves: 0.090 – 0.110 inches Texture depth: 0.125 inches (range: 0.0625 – 0.1875 inches)
Traffic Data	Original Design: 1968 AADT: 5,088, 12% Trucks 1988: AADT: 7,668 2015 AADT: 8,800 vehicles, 2,120 commercial
Slab Design Details (original 1968 construction)	10-inch undoweled JPCP 20-ft transverse joint spacing Longitudinal joints tied with 0.5-inch diameter, 36-inch-long dowels on 30-inch spacings.
Base/Subbase	4-inch uniform granular subbase
Subgrade	Unknown
Subdrainage	4-inch longitudinal subdrain (1991).
Shoulder Type and Design	1968: 8-inch asphalt- treated base shoulders over granular subbase 2003: “Type B”: granular shoulders to match thickness of HMA resurfacing 2014: Top portion of granular should excavated, 4-ft wide, 6-inch thick HMA shoulders added
Unique/Noteworthy Features	None
*Eastbound lanes are the focus of this case study	

Table 8-2. U.S. 20 concrete mix design.

Material	Absolute Volume	Weight (lbs./yd ³)
Cement	0.114	603
Water	0.154	260
Air	0.060	N/A
Coarse Aggregate	0.370	1,596
Fine Aggregate	0.302	1,364

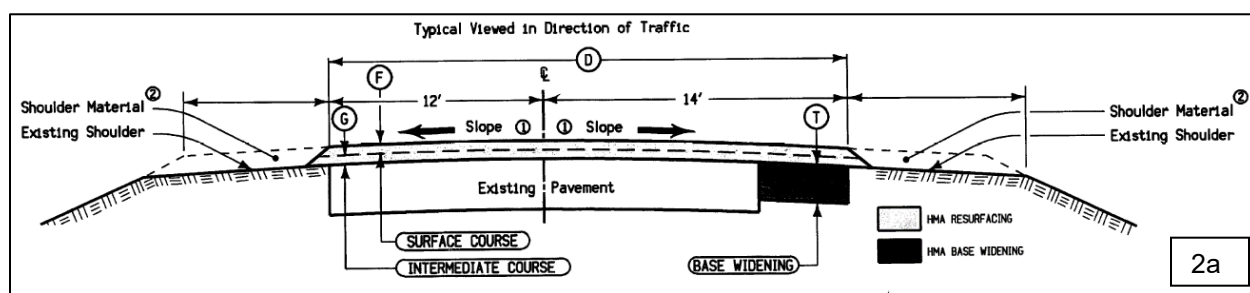
Rehabilitation Pavement Design, Materials and Construction

Subdrainage

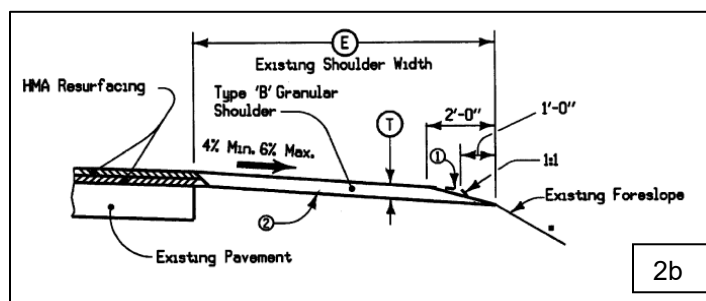
In 1991, 4-inch (102-mm) diameter longitudinal subdrains with 6-inch (152-mm) diameter outlets spaced at 500-ft (152-m) intervals were added on U.S. 20 from MP 135.85 to MP 152.72 (Iowa DOT 1991). To facilitate the placement of the subdrains, the existing asphalt shoulder was cut along the entire length, and the removed material was stockpiled and later used for the embankments. The trench was finished with a minimum of 6-inch (152) of asphalt-treated base material using mix designs approved by the resident engineer.

HMA Overlay

In 2003, the original concrete pavement (both EB and WB directions) was resurfaced with a 3.5-in (89 mm) HMA overlay. Figure 8-2a shows the typical pavement detail for the HMA overlay (Iowa DOT 2003). The shoulder material specified was a “Type B” granular surfaced shoulder, the typical detail for which is shown in figure 8-2b (Iowa DOT 2003). The existing shoulder surface was shaped to a uniform cross slope prior to placement of the granular shoulder material. A granular shoulder was not needed on the outside of the eastbound lanes due to extra material from the pavement scarification and base widening operations.



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Figure 8-2. Pavement detail for HMA resurfacing and widening.

Repair of Existing Concrete

Periodic maintenance activities (patching and joint resealing) were performed on the concrete pavement by local maintenance forces between 1968 and 2003.

Overlay Removal, Dowel Bar Retrofit, Diamond Grinding

The HMA overlay placed in 2003 developed severe joint reflection cracking that contributed to increased roughness levels. Consequently, in 2014, the HMA overlay in the two traffic lanes eastbound direction were completely removed to expose the original concrete pavement. Retrofit dowel bars were installed at all transverse joints (as well as at selected cracks), and this was followed by a diamond grinding operation to produce a smooth riding surface.

Figure 8-3 shows the typical pavement detail for the removal of the asphalt overlay in the eastbound direction, while figure 8-4 shows an overview of the pavement surface after the removal of the asphalt overlay but before the dowel bar retrofit and diamond grinding operations were performed (Iowa DOT 2003).

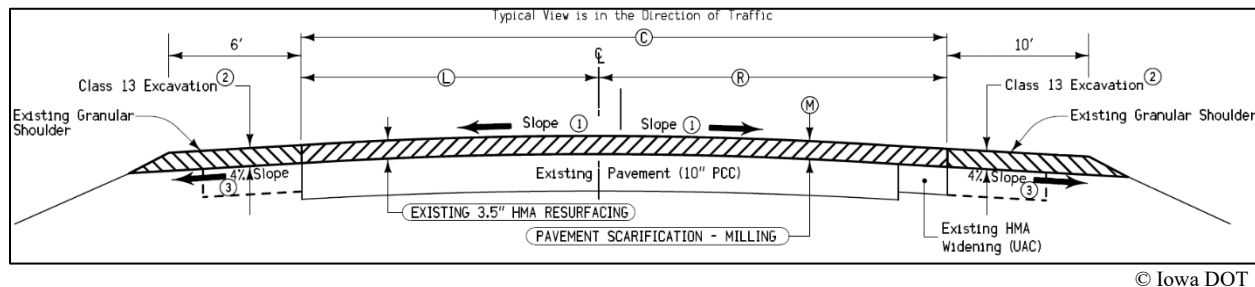
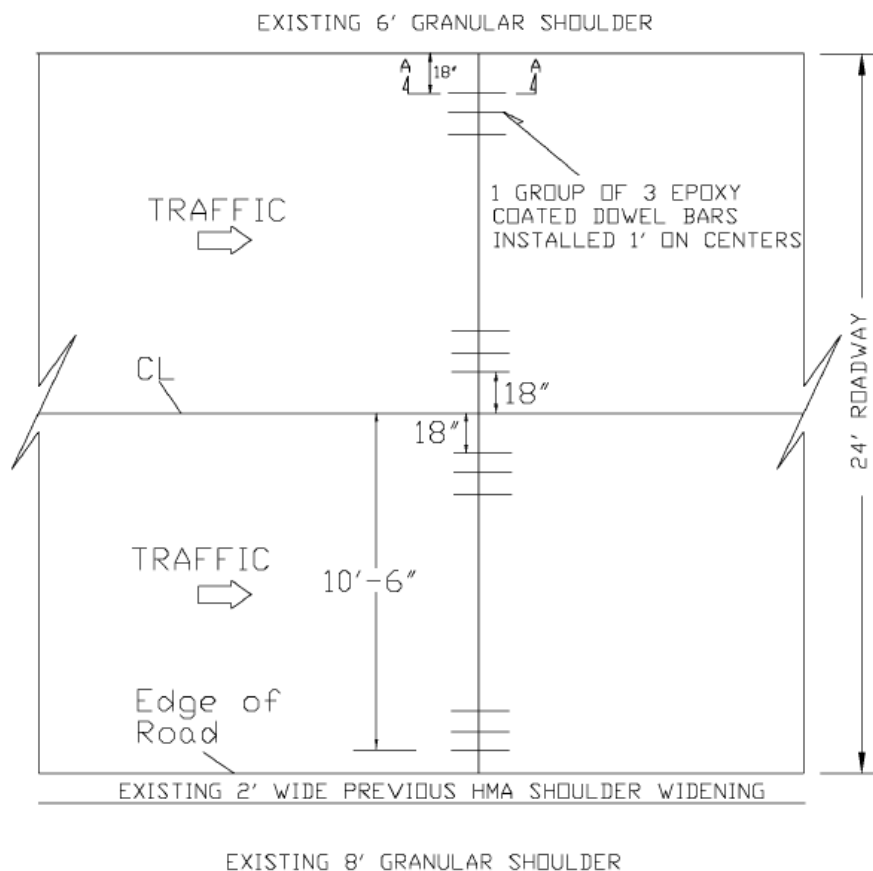


Figure 8-3. Pavement detail for removal of asphalt overlay in eastbound direction of U.S. 20.



Figure 8-4. Exposed concrete (left), close-up to pre-existing cracking in eastbound direction of U.S. 20 (right).

For the DBR operation, the Iowa DOT specification required the installation of two sets of three dowels clustered in the wheelpaths. The first dowel of each set was installed 18-inches (457 mm) from the closest longitudinal joint. The spacing between each dowel bar was 12 inches (305 mm) center-to-center. Figure 8-5 presents an overview of the dowel bar retrofit layout at each transverse joint (Iowa DOT 2013).

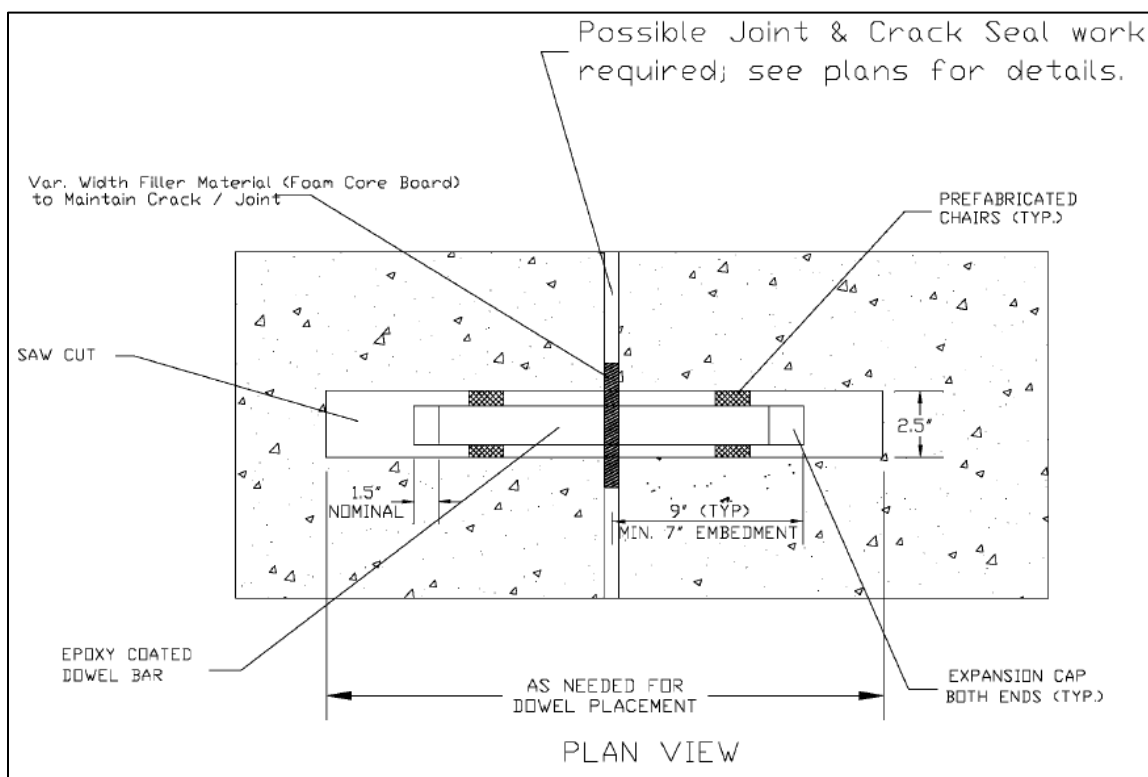


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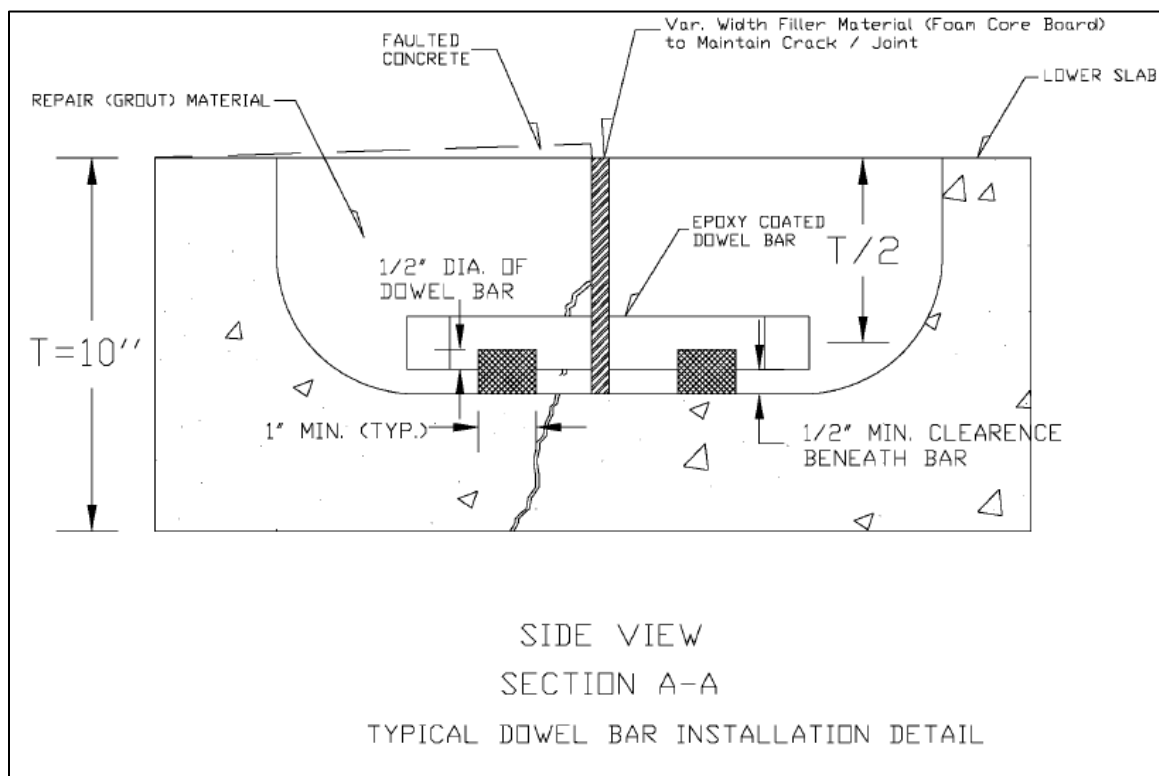
Figure 8-5. Dowel bar retrofit typical layout.

For the DBR, the 2012 Iowa DOT specification required the use of 1.5-inch (38 mm) diameter, 15-inch (381 mm) long epoxy-coated dowels with expansion caps on both ends (Iowa DOT 2012). To accommodate the dowel bars, 2.5-inch (64 mm) wide slots were sawed into the slabs to a depth such that the dowels would be located at mid-depth of the existing concrete slab and with a minimum clearance of 0.5 inches (13 mm) between the bottom of the slot and the bottom of the dowel. To this end, the dowels were seated on prefabricated chairs and each dowel was required to be installed with a foam core board placed at its mid-length to maintain the alignment of the dowel with the joint. Figure 8-6 shows the detail of the slot cutting requirements, while figure 8-7 shows the alignment requirements for each dowel (Iowa DOT 2013).

After the dowels were properly aligned, the slots were filled with grout material that consisted of a proprietary rapid-setting concrete backfill material mixed with a 0.38-inch (9.5-mm) nominal maximum size pea gravel. A handheld vibrator was used to ensure that the grout material properly covered the chairs and encapsulated the bottom of the dowel. After the backfill material had set up, the transverse joints were reestablished by saw cutting, typically within 8 hours of the grout placement.



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Figure 8-6. Dowel bar slot cutting detail.

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Figure 8-7. Dowel bar installation detail.

After all the dowels were placed, the concrete surface was subjected to a diamond grinding treatment to correct the surface irregularities and produce a smooth riding surface. For this project, the grinding head tolerances were set for limestone materials following Iowa Standard Specification 2532 (Iowa DOT 2014). The land between grooves was required to be between 0.090 and 0.110 inches (2.3 and 2.8 mm), with a target mean texture depth of 0.125 inches (3.2 mm).

Service Conditions

U.S. 20 is major arterial road that experiences year-round heavy loading from the agriculture and livestock industries. Traffic counts from the Iowa DOT show that this stretch of U.S. 20 has experienced a constant increase in traffic over the last 20 years (see figure 8-8). Figure 8-9 shows examples typical heavy traffic truck traffic on U.S. 20.

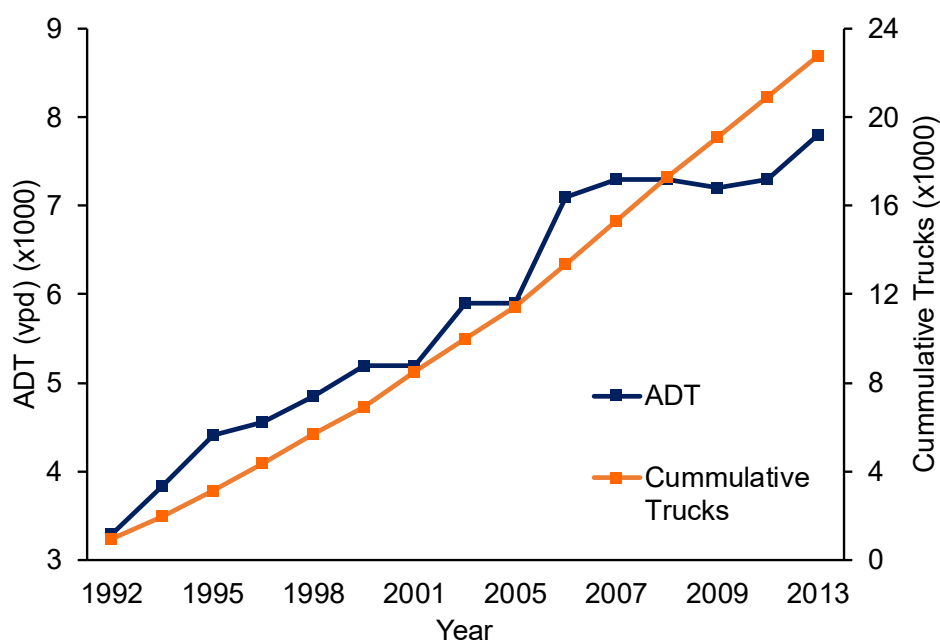


Figure 8-8. Historic traffic data for U.S. 20 (Data source: IDOT).



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Figure 8-9. Examples of truck traffic on U.S. 20 in 2018.

Project Characteristics

A 2018 site visit revealed that the westbound lanes, still overlaid with HMA, exhibited medium- to high-severity joint reflection cracking along the entire stretch of the section. Maintenance efforts using crack seal application were evident. Figure 8-10 shows an overview of the westbound lanes of U.S. 20 while figure 8-11 presents a close-up of the typical transverse joint reflective cracking.



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Figure 8-10. Overview of U.S. 20 WB with HMA overlay in 2018.



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Figure 8-11. Typical transverse joint reflective cracking on U.S. 20 WB in 2018.

On the eastbound lanes that had received DBR and diamond grinding, joint seal deterioration, minor to severe spalling on some joints, and some medium- to high-severity mid-panel transverse cracks were noted. Figure 8-12 shows an overview of the eastbound lanes of U.S. 20, and figures 8-13 and 8-14 present examples of joint seal damage, spalling, and mid-panel transverse cracking observed on some slabs.



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Figure 8-12. Overview of U.S. 20 EB with DBR and diamond grinding in 2018.



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Figure 8-13. Example of spalling and joint seal damage on U.S. 20 EB in 2018.



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Figure 8-14. Example of mid-panel transverse cracking on U.S. 20 EB in 2018.

Pavement Performance and Maintenance History

Iowa DOT monitors the performance of its road network using several performance measures. Pavement surface characteristics monitored include roughness (in terms of IRI) and surface friction. Iowa DOT uses the Pavement Condition Index (PCI) method to report the overall condition of its road network. The PCI values range between 0 and 100, with 0 representing a failed condition and 100 representing excellent condition. Figure 8-15 shows time-series IRI and PCI data for the eastbound lanes.

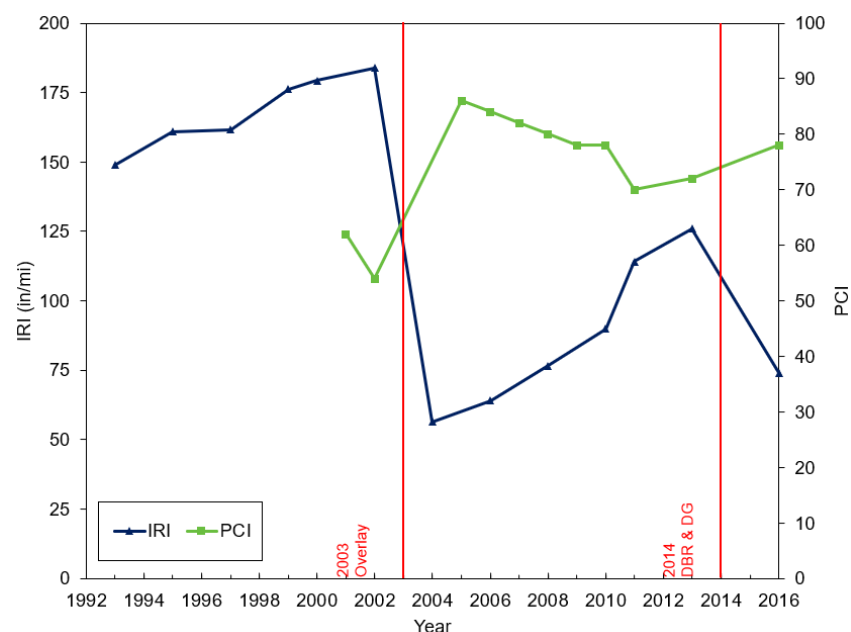


Figure 8-15. Time-series for IRI and Friction, EB U.S. 20 EB (Data source: Iowa DOT).

The effects of the 2003 AC overlay and the 2014 concrete rehabilitation on surface smoothness are clearly evident, with a decrease in IRI after each treatment. In terms of overall condition, after the 2014 rehabilitation the PCI increased to same level as it was in 2010.

The U.S. 20 project shows that it is possible to achieve a long-life concrete pavement with the proper and timely application of rehabilitation treatments. Even after 50 years since the original construction, adequate levels of serviceability are attainable without the need for a major reconstruction effort.

Economic Analysis

Table 8-3 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of U.S. 20 documented in this case study. All costs were obtained from the Iowa DOT contract documents. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 8-3. Activity and cost data for U.S. 20.

Year	Activity	Activity Detail	Cost (from DOT Contract)*
1968	JPCP Construction	10-inch PCC Paving	\$584,290
1991	Drainage Maintenance	4-inch longitudinal subdrain with 6-inch outlets	\$772,986
2003	HMA Resurfacing	3.5-inch Overlay HMA Intermediate Course HMA Surface Course Asphalt Binder, 64-22	\$809,602
2014	HMA Removal and CPR	HMA Milling, Dowel Bar Retrofit and Diamond Grinding	\$2,803,720
2024	End-of-Service	Assumed Pavement End of Service	NA

*Costs shown in then-current dollars

A 51-year analysis period was used (1969-2019) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 8-3 are summarized below:

- 1968 JPCP: 56 years (end-of-service: 2024)
- 1991 Drainage Maintenance: 33 years (end-of-service: 2024)
- 2003 HMA Resurfacing: 11 years (end-of-service: 2014)
- 2014 CPR: 10 years (end-of-service: 2024)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 8-4 shows the construction costs converted to 1968 dollars.

Table 8-4. Agency costs for U.S. 20 project (eastbound lanes).

Year	Activity	1968 Dollars
1968	JPCP Construction	\$584,290
1991	Drainage Maintenance	\$204,127
2003	HMA Resurfacing	\$153,426
2014	HMA Removal and CPR	\$320,315
2019	Remaining Value (1968 JPCP Construction)	(\$52,169)
2019	Remaining Value (2014 CPR)	(\$160,158)

Figure 8-16 shows the expenditure stream diagram for the U.S. 20 project using the four different discount rates used in the analysis.

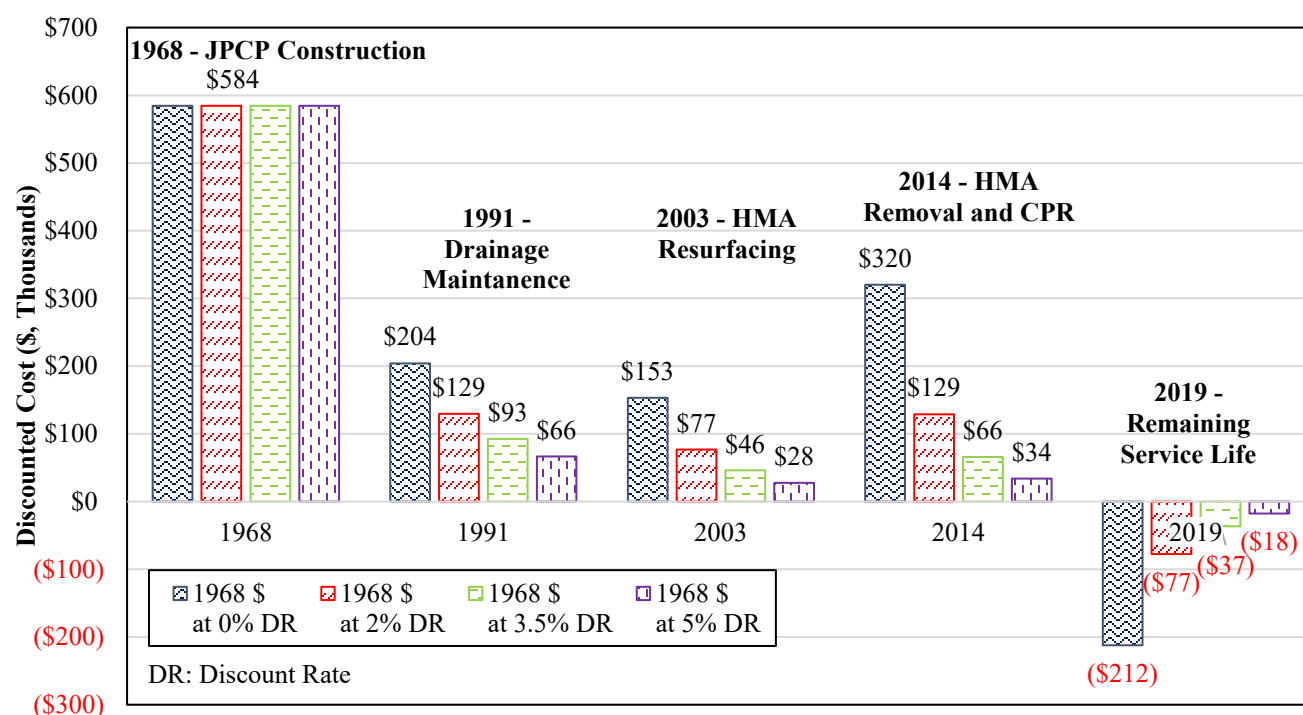
**Figure 8-16. Expenditure stream diagram for U.S. 20 (eastbound lanes).**

Table 8-5 summarizes the LCCA results for each discount rate considered. As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 34 percent and the equivalent uniform annual cost (EUAC) increases by 84 percent. This is graphically represented in Figure 8-17.

Table 8-5. LCCA summary for U.S. 20.

0% Discount Rate				
LCCA Element	1968 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$584,290	PV =	\$5,345,790
1991 Drainage Maintenance	PV =	\$204,127	PV =	\$1,867,603
2003 HMA Overlay	PV =	\$153,426	PV =	\$1,403,724
2014 CPR (DBR, DG)	PV =	\$320,315	PV =	\$2,930,631
Remaining Value (1968 PCC)	PV =	(\$52,169)	PV =	(\$477,303)
Remaining Value (2014 CPR)	PV =	(\$160,158)	PV =	(\$1,465,315)
Totals	NPV =	\$1,049,832	NPV =	\$9,605,129
	EUAC =	\$20,585	EUAC =	\$188,336
2% Discount Rate				
LCCA Element	1968 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$584,290	PV =	\$5,345,790
1991 Drainage Maintenance	PV =	\$129,448	PV =	\$1,184,351
2003 HMA Overlay	PV =	\$76,717	PV =	\$701,901
2014 CPR (DBR, DG)	PV =	\$128,816	PV =	\$1,178,564
Remaining Value (1968 PCC)	PV =	(\$52,169)	PV =	(\$477,303)
Remaining Value (2014 CPR)	PV =	(\$64,408)	PV =	(\$589,282)
Totals	NPV =	\$802,695	NPV =	\$7,344,021
	EUAC =	\$25,252	EUAC =	\$231,032
3.5% Discount Rate				
LCCA Element	1968 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$584,290	PV =	\$5,345,790
1991 Drainage Maintenance	PV =	\$92,528	PV =	\$846,557
2003 HMA Overlay	PV =	\$46,024	PV =	\$421,085
2014 CPR (DBR, DG)	PV =	\$65,814	PV =	\$602,150
Remaining Value (1968 PCC)	PV =	(\$52,169)	PV =	(\$477,303)
Remaining Value (2014 CPR)	PV =	(\$32,907)	PV =	(\$301,075)
Totals	NPV =	\$703,580	NPV =	\$6,437,204
	EUAC =	\$29,777	EUAC =	\$272,433
5% Discount Rate				
LCCA Element	1968 Dollars		2019 Dollars	
1968 Original Construction	PV =	\$584,290	PV =	\$5,345,790
1991 Drainage Maintenance	PV =	\$66,458	PV =	\$608,038
2003 HMA Overlay	PV =	\$27,815	PV =	\$254,481
2014 CPR (DBR, DG)	PV =	\$33,952	PV =	\$310,637
Remaining Value (1968 PCC)	PV =	(\$52,169)	PV =	(\$477,303)
Remaining Value (2014 CPR)	PV =	(\$16,976)	PV =	(\$155,319)
Totals	NPV =	\$643,370	NPV =	\$5,886,325
	EUAC =	\$35,082	EUAC =	\$320,973
PV: Present Value				
NPV: Net Present Value				
EUAC: Equivalent Uniform Annual Cost				

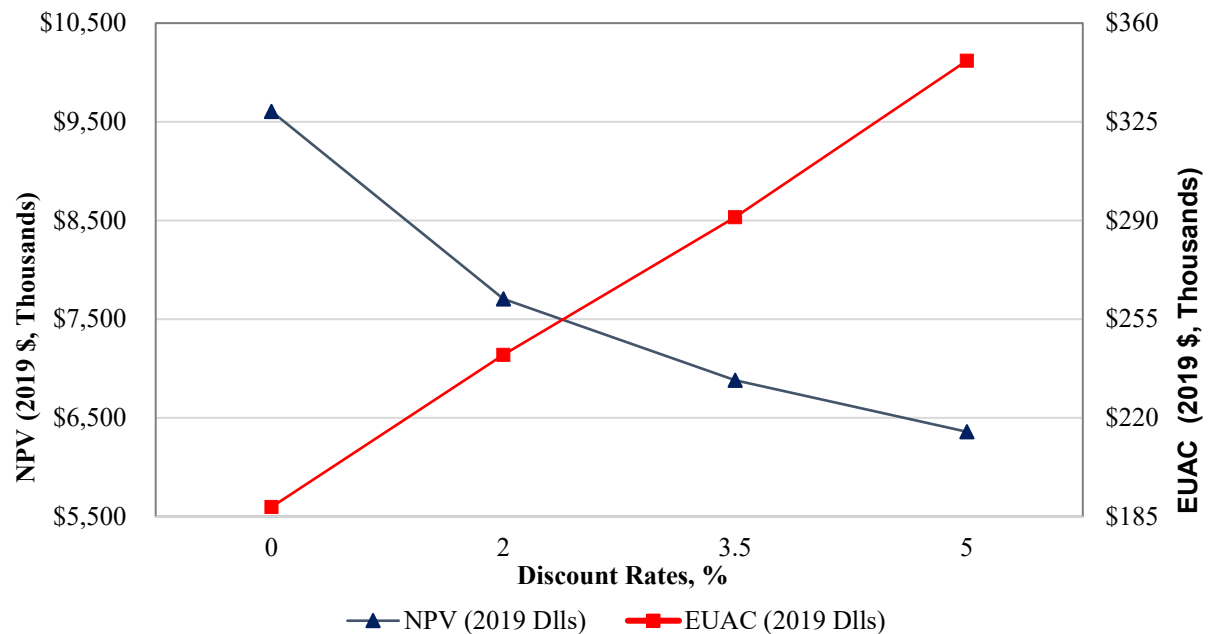


Figure 8-17. NPV and EUAC (2019 Dollars) vs discount rate.

Lessons Learned

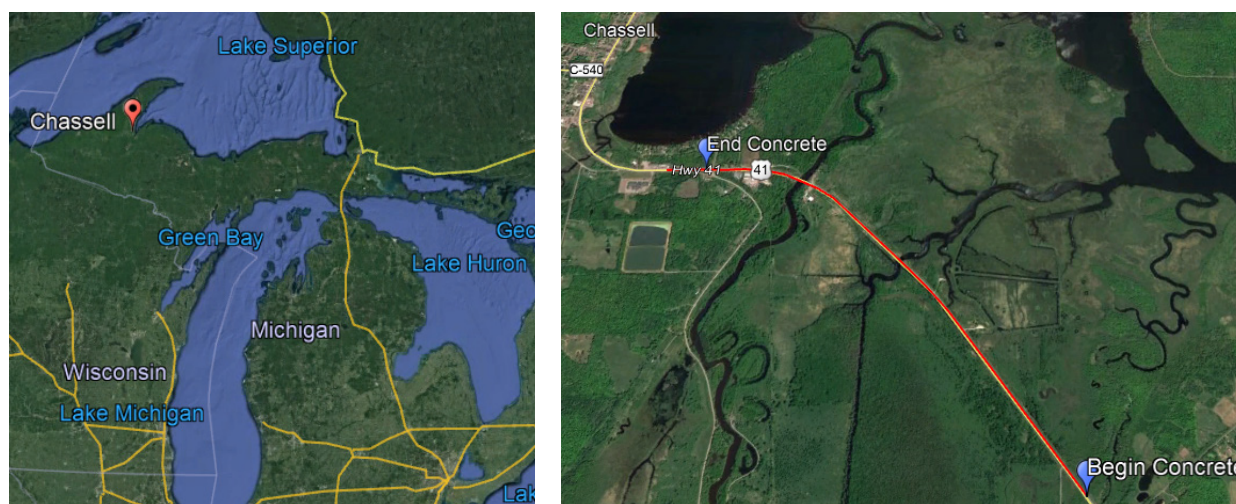
Originally constructed in 1968, a 3.8-mi (6.1 km) segment of U.S. 20 between U.S. 69 and I-35 in central Iowa has shown that it is possible to achieve a long-life pavement structure through the application of timely rehabilitation treatments. The original pavement was designed for a 20-year service life, but after several intermediate rehabilitation treatments, the pavement remains serviceable even after 50 years. Based on the LCCA conducted, future maintenance costs are estimated to be between 17 and 49 percent of the original JPCP construction cost, depending on the discount rate used. Some key lessons learned from this project are:

- **HMA overlay on undoweled JPCP is not a long-term solution.** An asphalt overlay over JPCP, can improve the serviceability and pavement surface characteristics (friction and IRI). However, in the long run, reflective cracking becomes a major distress affecting the rideability of the pavement.
- **Proper and timely CPR activities can extend the service life of concrete pavement significantly.** By restoring the load transfer capacity of the concrete pavement and correcting the surface roughness, adequate levels of serviceability can be achieved and maintained after 50 years of service.

CHAPTER 9. CONCRETE PAVEMENT REHABILITATION ON U.S. 41 IN MICHIGAN

Introduction

In 1960, the Michigan State Highway Department (forerunner to the Michigan Department of Transportation) constructed a 9-inch (229-mm) jointed reinforced concrete pavement (JRCP) on U.S. 41, southeast of Chassell (see figure 9-1). The concrete section was approximately 2.35 mi (3.78 km) long and was constructed through a low-lying, wetland known as the Sturgeon River Sloughs. The pavement consisted of two 12-ft (3.7-m) wide traffic lanes and included doweled transverse joints spaced at 100-ft (30.5-m) intervals. General project information is summarized in table 9-1.



(L) © 2018 Google Earth; Data: SIO, NOAA, U.S. Navy, NGA, GEBCO, INEGI, Landsat/Copernicus. (R) © 2018 Google Earth; Data: Airbus, TerraMetrics, NOAA. (Added line for project limits).

Figure 9-1. U.S. 41 project location.

Original Pavement Design, Materials, and Construction

Structural Design

The 2.35 mi (3.78 km) section of concrete pavement (33,050 yd²) was bid in 1958 with a winning bid of \$514,089.80. Construction occurred in 1960. The project raised a portion of the existing U.S. 41 alignment through the Sturgeon Rivers Sloughs area of Houghton County and re-aligned a section of U.S. 41 with a new bridge over the Sturgeon River. Most of the area along the alignment is low-lying wetlands. Drawings show that the original roadway was a 20-ft (6.1 m) wide concrete section (thickness and construction date not identified). Concrete was placed in September and October of 1960 with ambient paving temperatures between 37 °F (3 °C) and 68 °F (20 °C) (average of 55 °F (13 °C) per test records). Table 9-1 provides general location and design details for the project.

Table 9-1. U.S. 41 project details.

Key Information	Data
Project Location	U.S. 41 NB/SB, Chassell, MI Milepost 4.46 – 6.81 (county) Rural 2-lane Highway
Climatic Region	Wet-freeze
Highlighted Project Design Characteristics	Concrete Pavement Restoration: Full depth repairs and crack and joint resealing.
Initial Construction Year	1960
Rehabilitation Types, Years and Details	2000: Full depth repair with transverse crack and joint resealing. 2007: Full depth repair with transverse crack and joint resealing. 2017: Full depth repair with transverse crack and joint resealing.
Traffic Data	1957 ADT: 2,000 1997 ADT: 3,896 2017 AADT: 3,991
Slab Design Details (Original 1960 construction)	9-inch JRCP (wire mesh, 0.16% longitudinal steel) 100-ft transverse joint spacing 1.25-inch diameter dowel bars Centerline tie bars (hooked) ¹ with keyway 12-ft traffic lanes
Base/Subbase	Stamp sand (variable up to 5 ft), unclassified core fill (up to 7-ft) (likely stamp sand), or 18 inches of unclassified pit materials.
Subgrade	Unknown, but constructed through a wetland.
Subdrainage	None in stamp sand and 18-inch subbase section. Porous weep drains from core fill through waste material used to construct outer slopes.
Shoulder Type and Design	9-inch compacted gravel
Unique/Noteworthy Features	Reported that explosives were used to remove peat and impart deep compaction in wetland areas.

¹ Not shown on plans but detailed in an adjacent construction project designed the same year.

Paving Materials

The 1957 MDOT concrete specification required a 28-day compressive strength of 3,500 psi (24.13 MPa) and 5.5 percent entrained air. The MDOT Grade A concrete mixture used had a cement factor of 5.5 sacks [517 lbs. (234.5 kg)] per cubic yard with a slump of around 2.5 inches (64 mm). The cement source was Penn-Dixie Cement Company in Petoskey, Michigan and Superior Sand & Gravel in Hancock, Michigan provided the coarse aggregate (Class 4A and 10A) with a 2-inch (51-mm) maximum size aggregate. Other specific details of mixture proportions were not available. The concrete was membrane cured and beams for strength testing were buried in damp sand for curing. Table 9-2 provides a summary of the material test results found in project records.

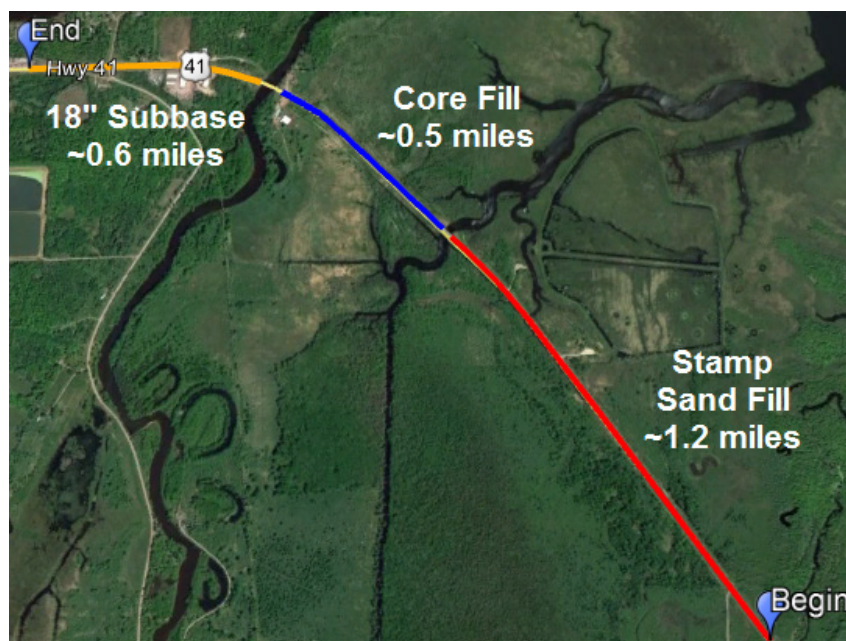
Table 9-2. Concrete test results from project records.

Descriptive Statistic	Thickness (in)	Modulus of Rupture, 7-day (psi)	Modulus of Rupture, 14-day (psi)	Compressive Strength, 60+ day (psi)
Average	9.2	584	688	5,240
Stand. Dev.	0.2	32	41	450
N	52	19	14	52
Max	9.7	643	791	6,270
Min	8.8	533	625	4,295

Construction

The project was constructed on three distinct areas of fill material as highlighted in figure 9-2. The southeastern portion of the project to the Sturgeon River Sloughs [~ 1.2 mi (1.9 km)] was constructed on up to 5 ft (1.5 m) of Isle Royale stamp sand, a coarse, angular fine aggregate that is produced when metal-rich ore is crushed in stamp mills. The middle segment [~ 0.5 mi (0.8 km)], between the Sturgeon River Sloughs and the Sturgeon River, was constructed on a 7-ft (2.13 m) thick fill material (not classified, but likely also stamp sand) and the northwestern portion [~ 0.6 mi (0.97 km)] was constructed on an 18-inch (457.2 mm) subbase of pit material (not classified).

A phone interview was conducted with Pat Thornton (worked for 40+ years with Superior Sand & Gravel) whose grandfather worked on the original U.S. 41 project as the owner of Thornton Construction. He recalled a story passed down from his uncle where the grade through wetland area sunk during construction. Holes were drilled along the alignment and explosives were used to “blow the muck out to the sides of the roadbed” and collapse the problem soils.



© 2018 Google Earth; Data: Airbus, NOAA, TerraMetrics. (Added lines for project limits and labeled details of fill sections).

Figure 9-2. General breakdown of project fill sections.

Traffic and Service Conditions

In addition to typical AASHTO Class 9 trucks, U.S. 41 experiences heavy loading year-round from the local logging industry. Michigan has one of the nation's highest allowable legal truck load limits (up to 164,000 lb [74,400 kg] on 11 axles). Figure 9-3 shows an example of a typical loaded log truck and axle configuration observed on U.S. 41.



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Figure 9-3. Example of typical logging truck on U.S. 41.

Project Maintenance and Rehabilitation

This section of U.S. 41 remained in service with little repair for over 40 years (although evidence suggests that some intermittent repairs were conducted, primarily at joints, likely by MDOT regional maintenance crews). The first significant rehabilitation took place in 2000 in the form of concrete pavement restoration (CPR), consisting of full-depth repairs at deteriorated joints and transverse cracks, joint resealing, and complete removal of approximately 900 ft (274.3 m) of concrete at the westernmost project limits (this is the segment constructed on 18 inches (457.2 mm) of subbase composed of unclassified pit materials) and replacement with an asphalt concrete section. The concrete used in the full-depth repairs placed in 2000 was proportioned to achieve high-early strength through the use of a 1.5 to 2.0 percent calcium chloride accelerator added on site (note that the Michigan DOT has since discontinued the use of calcium chloride accelerator for its pavement repair materials). Some of these repairs exhibited premature failure attributed to materials-related distress of the repair material.

CPR was again performed in 2007 to replace some of the repairs constructed in 2000 and to address on-going deterioration at joints and cracks. The most recent CPR occurred in 2017, and today there are 228 full-depth repairs in the northbound direction and 235 in the southbound directions.

Field Visit

A site visit to the project was performed in 2018. Distresses observed during that site visit included a few low- and medium-severity transverse cracks, joint sealant deterioration, isolated

spalling (particularly at the longitudinal joint) that has been patched with asphalt material, one area of slab settlement, and some general surface wear. Figures 9-4 through 9-7 present examples of the 2018 pavement condition.



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Figure 9-4. View of the project showing full-depth repairs and worn concrete surface.



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Figure 9-5. Low-severity mid-panel crack.



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Figure 9-6. Medium-severity mid-panel crack with temporary asphalt “repair.”



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Figure 9-7. Typical full-depth joint repair.

The color differences are indicative of patches placed at different times; the darker repair was placed in 2007, and the lighter repair was placed in 2017.

Pavement Performance and Maintenance History

MDOT uses a distress index (DI), defined as the total accumulated distress point value that is normalized to 0.1-mi (0.16 km) segments, to characterize pavement distress. The scale for DI is as follows (MDOT 2017):

- Good = 0 – 25
- Fair = 26 – 49
- Poor $\geq$ 50

In addition to distress, MDOT considers ride quality through the use of the International Roughness Index (IRI) in accordance with the following scale (MDOT 2017):

- Good = 0 – 94 inches/mi
- Fair = 95 – 170 inches/mi
- Poor $>$ 170 inches/mi

Maintenance repairs on the concrete section are evident in MDOT video records, collected in or prior to 1998. Figure 8 shows examples of concrete and asphalt repairs that are seen in the 1998 MDOT video log within the project limits. A review of the 1998 video log identified approximately 20 to 25 concrete repairs and 8 to 10 asphalt repairs in each direction (northbound and southbound).



©1998 MDOT Video Log

Figure 9-8. Example of repairs prior to 1998.

The first documented major rehabilitation in 2000 included full-depth repairs and joint resealing on the southeastern sections and removal of approximately 900 ft (274 m) of concrete at the northwestern project limits. Documents did not specifically state why the 900 ft (274 m) section of concrete was removed; however, the 1998 MDOT video log showed the section to be more severely deteriorated (figure 9-9) than the remainder of the project with significantly more cracking and joint deterioration. A potential explanation for this poorer performance is that this northwestern section, which was constructed on 18 inches of unclassified pit materials, may not have had adequate support compared to the southeastern sections that were constructed on 5 to 7 ft (1.5 to 2.1 m) of stamp sands.



©1998 MDOT Video Log

Figure 9-9. Examples of 1998 concrete condition prior to removal in 2000.

Subsequent rehabilitations featuring full-depth concrete repairs and joint resealing took place in 2007 and again in 2017 to repair additional deteriorated joints and cracks and replace full-depth repairs that had themselves failed. In particular, the concrete material used in the 2000 repairs appeared to be susceptible to materials-related distress (MRD), potentially due to the high cementitious content and use of calcium chloride accelerator, and many of the repairs had to be reconstructed in the 2007 project.

The first available IRI and distress index (DI) data for this section of U.S. 41 are from 1992 (MDOT records). Figure 9-10 shows average IRI and DI data from 1992 (pavement age of 33 years) until 2017 (pavement age of 58 years) for the entire section. Overall, the average IRI dropped 27 in/mi (0.43 m/km) to 104 inches/mile (1.64 m/km) after the first major rehabilitation but increased to 123 in/mi (1.94 m/km) within 2 years after the treatment. IRI did not significantly change from age 43 to 58 years despite two additional rehabilitations. Thus, the timing of the 2007 and 2017 rehabilitations appears to be related to the increase in distress index as shown in figure 9-10.

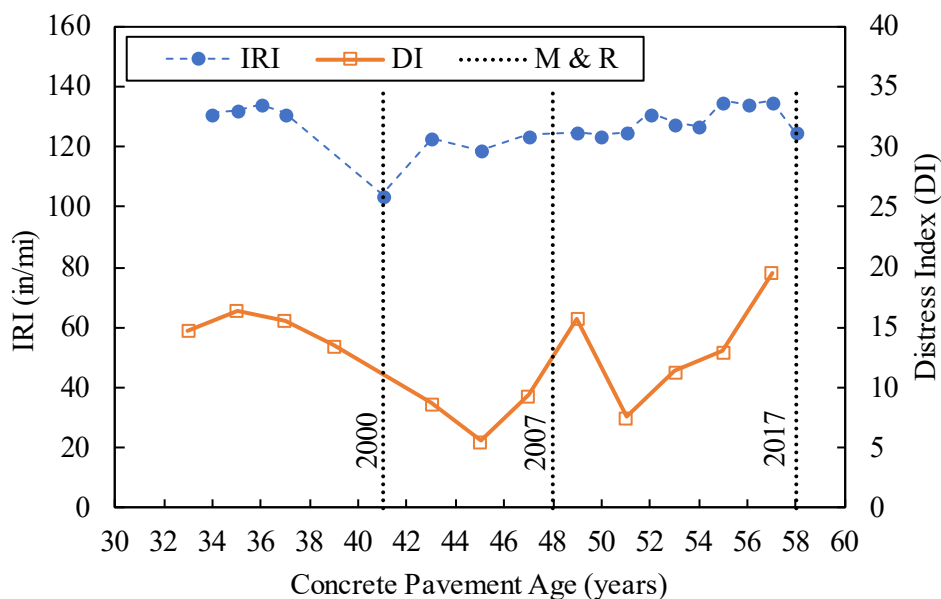


Figure 9-10. Time-series IRI and distress index (created from MDOT data).

Table 9-3 lists a summary of the approximate number of full-depth repairs and an estimate of the number of previous repairs that were replaced in a subsequent rehabilitation. The number of repairs and percent replacement was consistent in both directions and the number of replaced repairs was approximately two times greater in 2017 than 2007. It was not possible to determine if the repairs constructed in 2017 replaced those constructed in 2000 or 2007.

Table 9-3. Summary of full-depth repairs.

Northbound/ Southbound	Image Year	Approximate Number of Repairs ¹	Estimate of Previous Repairs Replaced (%) ¹
Northbound	2001	108	-
Northbound	2007	202	18
Northbound	2017	228	43
Southbound	2000	101	-
Southbound	2007	190	17
Southbound	2017	235	32

¹ Based on MDOT video log and Google Earth imagery.

The 2018 visual assessment identified MRD in many of the remaining repairs that were placed in 2000. The MRD took the form of map cracking and corner spalling; an example is shown in figure 9-11. Some patches appeared to deteriorate rather quickly and were replaced in 2007, while others were replaced in 2017. In other instances, the original concrete adjacent to the repair deteriorated resulting in this material as well as the repair being removed and replaced with a new, larger repair. This experience emphasizes the need to construct full-depth repairs carefully with the goal of having a service life consistent with the remaining life of the pavement.



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Figure 9-11. Example of 2000 repair condition in 2018 showing map cracking and corner spalling.

The durability of the repair materials is heavily influenced by the materials and proportions selected. This stretch of U.S. 41 is heavily trafficked for the area and provides a critical link between Houghton-Hancock and Marquette, and there is no possibility of a reasonable detour. MDOT chose to use concrete mixtures that have an opening time of 8 hours or less for the repair material as summarized below:

- The full-depth repairs placed in 2000 used an MDOT Type P-FS 9-sack mixture with 18 to 45 lb/yd³ (11 to 27 kg/m³) of 77 percent grade flaked calcium chloride. Calcium chloride content depended on the ambient temperature at the time of placement as was added on site to the back of the truck. Additional requirements included a slump of 1 to 3 inches (25 to 76 mm) and 5.5 ± 1.5 percent entrained air. This mix was expected to achieve a minimum flexural strength of 300 psi (2.1 MPa) in 8 hours.
- The full-depth repairs placed in 2007 used a MDOT Type P-MS 9-sack mixture (with a 0 to 18 percent calcium chloride accelerator used when the ambient temperature was below 60 °F (16 °C). Additional requirements included a slump of 1 to 3 inches (25 to 76 mm), 5.5 ± 1.5 percent entrained air, and a full 8-hour cure period prior to opening to traffic. Repairs were doweled and a minimum of 4 ft (1.2 m) long.
- The full-depth repairs placed in 2017 used a MDOT Type P-NC 8-sack mixture with non-chloride accelerator, specified as the minimum amount of accelerator to achieve at least 300 psi (2.1 MPa) flexural strength at the required open-to-traffic time. Repairs were doweled and a minimum of 4 ft (1.2 m) long. A full 8-hour cure period was specified.

As indicated, over time MDOT has reduced the cement contents in these high-early-strength mixtures and switched from calcium chloride to non-chloride accelerator, largely to improve the long-term durability of the repairs.

Economic Analysis

Table 9-4 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of U.S. 41 documented in this case study. All costs were obtained from Michigan DOT contract documents. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 9-4. Activity and cost data for U.S. 41.

Year	Activity	Activity Detail	Cost (from DOT Contract)*
1960	JRCP Construction	10-inch PCC Paving	\$181,900
1990	CPR	Full-depth repairs at joints	\$58,300 ¹
2000	CPR	Full-depth repairs or deteriorated joints and transverse cracks, reseal joints	\$104,700 ¹
2007	CPR	Full-depth repairs or deteriorated joints and transverse cracks, reseal joints	\$396,810 ¹
2017	CPR	Full-depth repairs or deteriorated joints and transverse cracks, reseal joints	\$420,511
2027	End-of-Life	Assumed Pavement End of Service	NA

*Costs shown in then-current dollars

¹Costs estimated using the 2017 unit costs and converted to then-current dollars using the National Highway Construction Cost Index (NHCCI). CPR quantities were estimated based on the extent of repair.

A 60-year analysis period was used (1960-2019) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 9-4 are summarized below:

- 1960 JRCP: 67 years (end-of-service: 2027)
- 1990 CPR: 10 years (end-of-service: 2000)
- 2000 CPR: 7 years (end-of-service: 2007)
- 2007 CPR: 10 years (end-of-service: 2017)
- 2017 CPR: 10 years (end-of-service: 2027)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 9-5 shows the construction costs converted to 1960 dollars.

Table 9-5. Agency costs for U.S. 41 project.

Year	Activity	1960 Dollars
1960	JRCP Construction	\$181,900
1990	CPR	\$12,462
2000	CPR	\$16,677
2007	CPR	\$40,334
2017	CPR	\$39,415
2019	Remaining Value (1960 JRCP Construction)	(\$21,719)
2019	Remaining Value (2017 CPR)	(\$31,532)

Figure 9-12 shows the expenditure stream diagram for the U.S. 41 project using the four different discount rates used in the analysis.

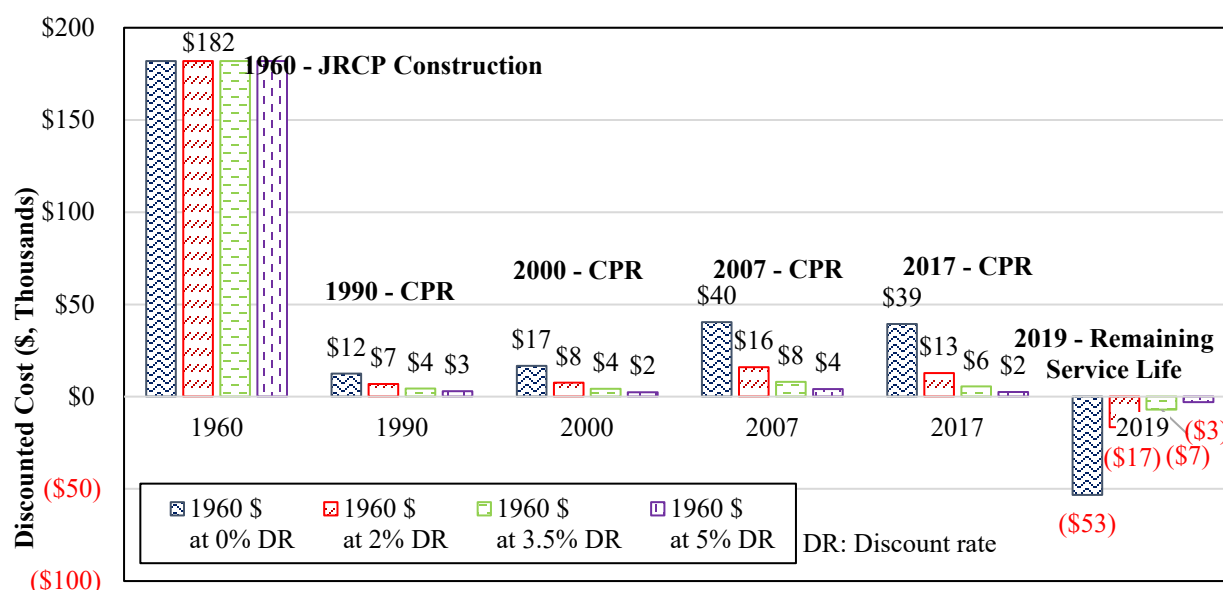


Figure 9-12. Expenditure stream diagram for U.S. 41.

Table 9-6 summarizes the LCCA results for each discount rate considered.

Table 9-6. LCCA summary for U.S. 41.

0% Discount Rate				
LCCA Element	1960 Dollars		2019 Dollars	
1960 Original Construction	PV =	\$181,900	PV =	\$2,037,143
1990 CPR	PV =	\$39,415	PV =	\$441,422
2000 CPR	PV =	\$39,415	PV =	\$441,422
2007 CPR	PV =	\$39,415	PV =	\$441,422
2017 CPR	PV =	\$39,415	PV =	\$441,422
Remaining Value (1960 PCC)	PV =	(\$21,719)	PV =	(\$243,241)
Remaining Value (2017 CPR)	PV =	(\$31,532)	PV =	(\$353,138)
Totals	NPV =	\$286,310	NPV =	\$3,206,453
	EUAC=	\$4,853	EUAC=	\$54,347
2% Discount Rate				
LCCA Element	1960 Dollars		2019 Dollars	
1960 Original Construction	PV =	\$181,900	PV =	\$2,037,143
1990 CPR	PV =	\$21,760	PV =	\$243,696
2000 CPR	PV =	\$17,851	PV =	\$199,916
2007 CPR	PV =	\$15,540	PV =	\$174,039
2017 CPR	PV =	\$12,748	PV =	\$142,772
Remaining Value (1960 PCC)	PV =	(\$21,719)	PV =	(\$243,241)
Remaining Value (2017 CPR)	PV =	(\$10,199)	PV =	(\$114,218)
Totals	NPV =	\$217,881	NPV =	\$2,440,108
	EUAC=	\$6,323	EUAC=	\$70,818
3.5% Discount Rate				
LCCA Element	1960 Dollars		2019 Dollars	
1960 Original Construction	PV =	\$181,900	PV =	\$2,037,143
1990 CPR	PV =	\$14,043	PV =	\$157,269
2000 CPR	PV =	\$9,955	PV =	\$111,491
2007 CPR	PV =	\$7,825	PV =	\$87,631
2017 CPR	PV =	\$5,547	PV =	\$62,123
Remaining Value (1960 PCC)	PV =	(\$21,719)	PV =	(\$243,241)
Remaining Value (2017 CPR)	PV =	(\$4,438)	PV =	(\$49,699)
Totals	NPV =	\$193,113	NPV =	\$2,162,718
	EUAC=	\$7,781	EUAC=	\$87,144
5% Discount Rate				
LCCA Element	1960 Dollars		2019 Dollars	
1960 Original Construction	PV =	\$181,900	PV =	\$2,037,143
1990 CPR	PV =	\$9,120	PV =	\$102,135
2000 CPR	PV =	\$5,599	PV =	\$62,702
2007 CPR	PV =	\$3,979	PV =	\$44,561
2017 CPR	PV =	\$2,443	PV =	\$27,357
Remaining Value (1960 PCC)	PV =	(\$21,719)	PV =	(\$243,241)
Remaining Value (2017 CPR)	PV =	(\$1,954)	PV =	(\$21,885)
Totals	NPV =	\$179,367	NPV =	\$2,008,772
	EUAC=	\$9,502	EUAC=	\$106,421
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost				

As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 20 percent and the equivalent uniform annual cost (EUAC) increases by 151 percent. This is graphically represented in Figure 9-13.

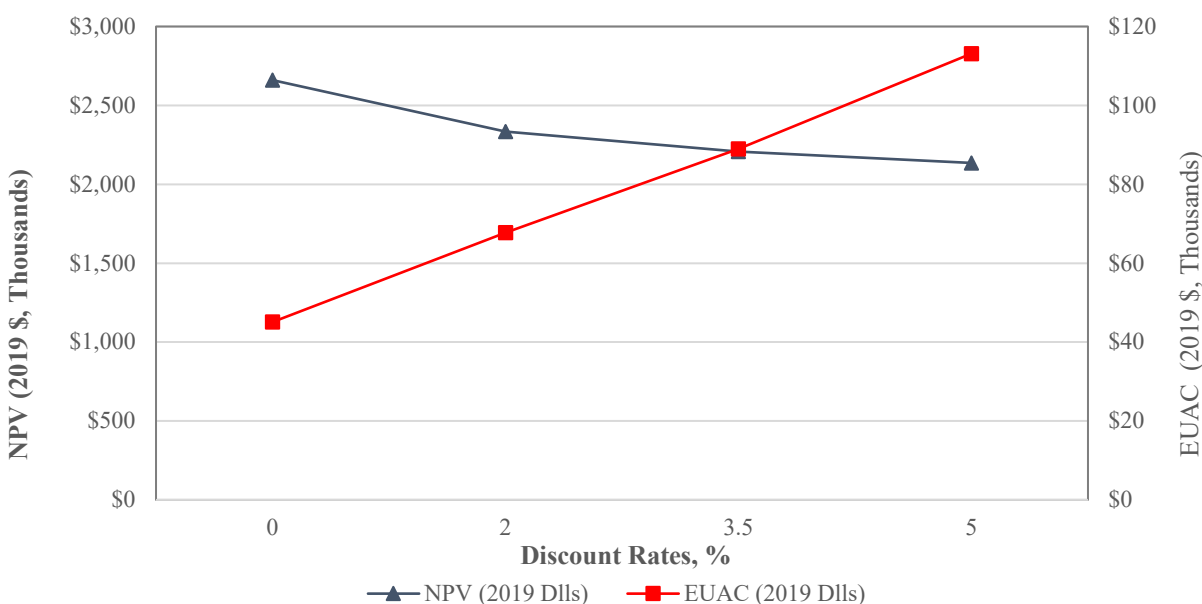


Figure 9-13. NPV and EUAC (2019 Dollar) vs discount rate.

Lessons Learned

The JRCF segment of U.S. 41 just south of Chassell in Michigan's Upper Peninsula is an excellent example of a long-life concrete pavement that has been kept in serviceable condition for almost 60 years through the timely application of CPR (full-depth concrete repairs and joint resealing). Not only is this section located in a harsh environment that experiences annual average snowfalls in excess of 200 inches (5.1 m), it is predominately located through a wetland. The lessons learned from this pavement section include:

- **A good foundation is essential.** The long-lived sections of concrete were constructed on 5 to 7 ft (1.5 to 2.1 m) of stamp sands, a highly angular manufactured fine aggregate that is a residual of crushing metal-rich ore in a stamp plant. The thickness and nature of this material provides an excellent level of support through an extremely wet area. An identical 900 ft (274 m) concrete section constructed at the same time was supported on 18 inches (457 mm) of unclassified pit material. It was removed and replaced with an asphalt pavement in 2000 due to poor performance.
- **Durable concrete mix design is critical for long-life.** The concrete mixture had a relatively low cementitious content of 517 lb/yd³ (307 kg/m³) and featured a 2-inch (51-mm) maximum aggregate size from a gravel source and 5.5 percent entrained air. This mixture would be expected to have low shrinkage and good aggregate interlock across intermediate cracks that formed in the JRCF and has demonstrated excellent durability over nearly six decades of service in harsh conditions and heavy traffic loading.

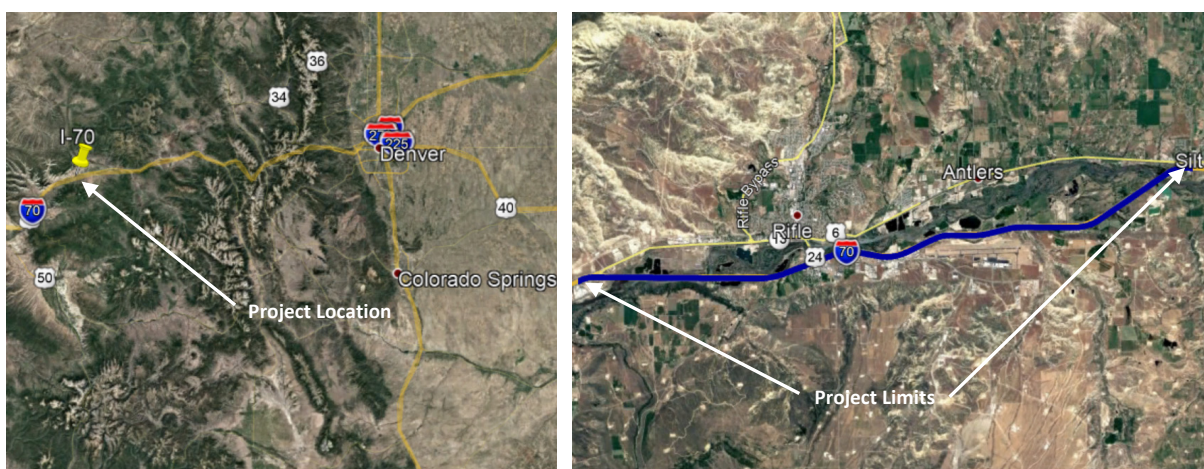
- **Full-depth repairs should be designed and constructed for a service life consistent with the anticipated remaining service life of the concrete pavement.** A number of failures have occurred in or adjacent to a previously constructed full-depth repair. Ensuring the longevity of the repair would ensure improved long-term performance of the pavement section.

CHAPTER 10. CONCRETE PAVEMENT REHABILITATION ON I-70 IN COLORADO

Introduction

The Colorado Department of Transportation (CDOT) has been preserving a 10.5-mi [16.9-km] stretch of jointed plain concrete pavement (JPCP) on I-70 between milepost 86.5 (west of Rifle, CO) and milepost 97 (near Silt, CO) in Garfield County using selected concrete pavement rehabilitation (CPR) techniques – full-depth repairs, diamond grinding and joint resealing. The original pavement construction for this project, which comprised 8 inches [200 mm] of undoweled JPCP with skewed joints on 4 inches [100 mm] of lean concrete base and 6 inches [150 mm] of aggregate subbase, was completed in 1976 (CDOT 1976). Bridge approach panel repairs were performed in 2003, followed by extensive full-depth pavement repairs in 2004 and a few additional repairs and blanket diamond grinding in 2005 (CDOT 2004, CDOT 2005a). Further pavement repairs and grinding were performed in 2014 (along with bridge deck overlays) [CDOT 2014].

Figure 10-1 shows the general location of the project within Colorado (left) and the approximate project limits (right). This section of pavement is the only concrete pavement on I-70 in Colorado between the border with Utah and the Denver metropolitan area.



(L): © 2018 Google Earth; Data: SIO, NOAA, U.S. Navy, NGA, GEBCO, INEGI, Landsat/Copernicus. (Added pin and label for project location). (R): 2018 Google Earth; Data: Airbus. (Added line and label for project limits).

Figure 10-1. I-70 project location and project limits.

Original Pavement Design and Materials

Structural Design

The original pavement consisted of 8 inches [200 mm] of jointed concrete pavement placed over 4 inches [100 mm] of lean concrete base and 6 inches [150mm] of CDOT Class 1 aggregate base course. Subgrade soils are generally alluvial deposits that range from poorly drained clayey loam (in areas close to the Colorado River) to loams and sandy loams (at slightly greater distances from the river) [USDA 2019]. Figure 10-2 is a typical pavement section taken from the as-built plans.

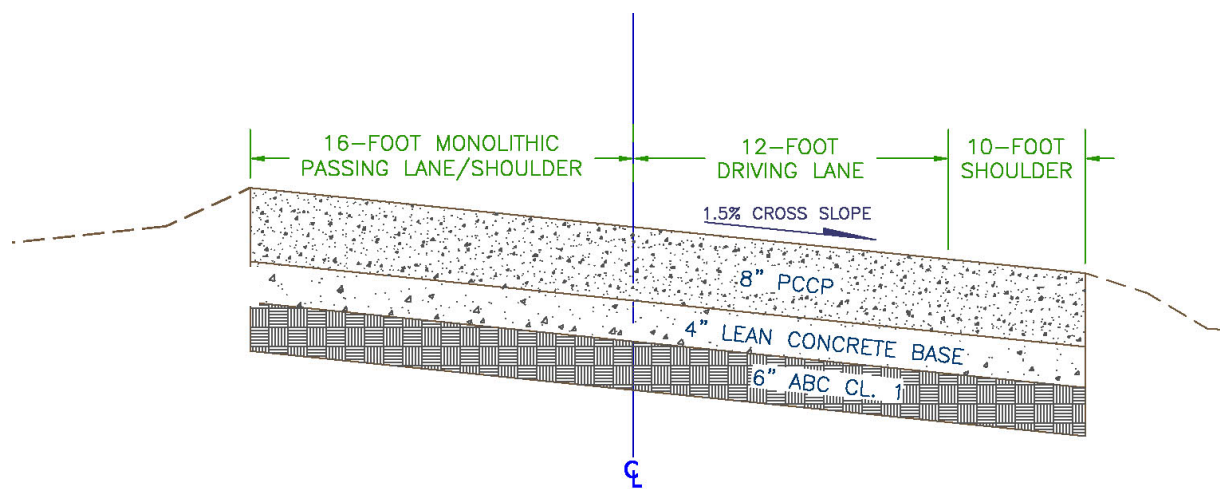


Figure 10-2. CO I-70 typical construction cross-section (Necessary and Lemesany 2006).

The original transverse joints were undoweled, skewed (right ahead, 2:12) and nonuniformly spaced using a pattern of 12-18-19-13 ft [3.66-5.49-5.79-3.96 m]. The longitudinal contraction joint between 12-ft [3.66-m] travel lanes was sawed to a depth of $T/4 + \frac{1}{4}$ inch [6 mm], minimum, and was tied with #4 [13-mm] deformed bars that were 30 inches [760 mm] long and spaced 36 inches [910 mm] apart. The 10-ft [3.05 m] outside shoulder was similarly tied. The 4-ft [1.22-m] inside shoulder, however, was cast integrally with the two travel lanes and was delineated with striping but no contraction joint, resulting in an effective panel width of 16 ft [4.88m].

Paving Materials

No information is available concerning the concrete paving mixture design or proportions used in the 1975-1976 construction.

Traffic

Interstate 70 serves as the primary through-corridor from Denver to points west. It carries a great deal of heavy truck traffic, and both total traffic and heavy truck traffic have increased rapidly since its completion. The 1975 project construction plans noted that the assumed 1995 design traffic level was 11,800 vehicles per day (vpd), with 3960 heavy trucks per day (33.6 percent). Actual traffic data from a continuous count measurement site near Silt, CO (just beyond the east end of the project) is available for 2002 to present and is presented graphically in Figure 10-3, which shows that current total traffic is approximately 23,000 vpd; 10.6 percent trucks are reported, with less than 20 percent of the trucks being single-unit trucks (CDOT Online Transportation Information System Data for Station 103011).

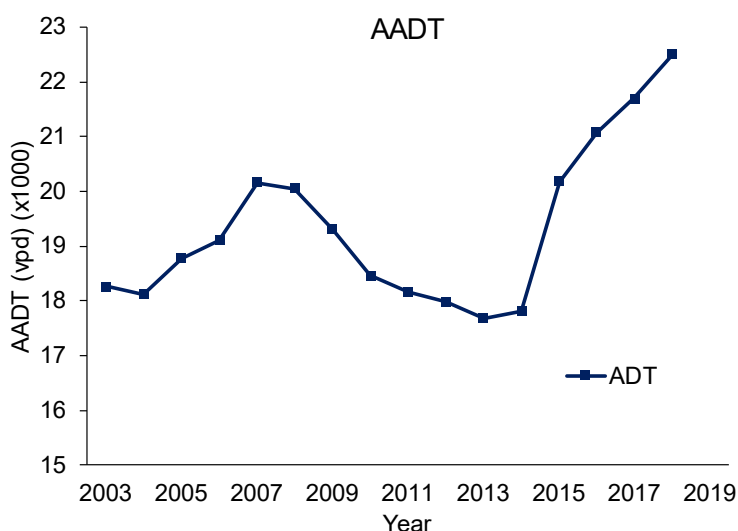


Figure 10-3. AADT data for I-70 traffic at MP 97 (Data source: CDOT).

Service Conditions

This portion of Interstate 70 is located on a plateau in western Colorado at an average elevation of approximately 5400 ft [1645 m]. The area is considered to be in a “dry-freeze” climate but is frequently exposed to snowy conditions during winter months. Studded tires are permitted in Colorado and the state puts “chain laws” into effect during certain weather conditions; these factors have resulted in unusually high levels of pavement surface wear on some roadways.

Maintenance and Rehabilitation Activities

This project has been in service for nearly 45 years with the help of rehabilitation activities that were carried out in 2003-2005 (after 28-29 years of service) and 2014 (after another 10 years of service), as described below. Very little maintenance work has been performed.

2003 Bridge Approach Panel Replacements

Selected bridge approach panels were replaced in 2003. Work performed included 600 s.y. [502 m²] of pavement removal and replacement (including the placement and compaction of 50 c.y. [38 m³] of aggregate base course), placement of 7 c.y. [5.5 m³] of miscellaneous concrete patching material, and the replacement of 80 lineal feet [24.4 m] of bridge compression joint sealant. The total cost of this work was \$68,970 (including \$22,050 for traffic control).

2004 Concrete Pavement Restoration

Distressed pavement panels were replaced in both directions in 2004. In most areas, the repair was done across both lanes and both shoulders over several panels of length, but some repairs were confined to one or both travel lanes and short distances. Plans called for joints in the newly placed slabs to be placed in the same locations as previous joints with skews to match existing joints in adjacent slabs. Unlike the original transverse joints, the repair joints were constructed with doweled joints (5 dowels per wheel path on 12-inch [300mm] centers with the outermost dowels located 6 inches [150 mm] from the nearest longitudinal joint).

Work performed included 10,580 s.y. [8,846 m²] of pavement removal and replacement, placement of 7.5 c.y. [5.7 m³] of miscellaneous concrete patching material, excavation of 691 c.y. [528 m³] of muck and placement of 800 c.y. [612 m³] of flowable fill, replacement of pavement stripes, and project traffic control. The total cost of this work was \$1,239,469 (including \$246,263 for traffic control).

2005 Concrete Pavement Restoration

This work was primarily a blanket diamond grind of the entire project (329,912 s.y. [274,914 m²] of grinding), but it also included a small amount (1199 s.y. [1003 m²]) of additional concrete pavement patching, rumble strip grinding (22,532 lineal feet [6868 m]) and weather sensor installation. The purpose of the grinding was to re-establish a uniform textured surface, but also to remove the significant surface “rutting” caused by tire chains and studded tires (see Figure 10-4). Photos of the 2005 repair and grinding work are presented in Figures 10-5 and 10-6.

The total cost of this work was \$1,855,548 (including \$235,915 for traffic control).



Figure 10-4. Project wheel path rutting due to tire chains and studded tires (Necessary and Lemasany 2006).



Figure 10-5. 2005 full-depth repair construction (Necessary and Lemasany 2006).



Figure 10-6. 2005 diamond grinding work (Necessary and Lemasany 2006).

2014 Concrete Pavement Restoration

This work consisted mainly of extensive concrete pavement repairs (23,760 s.y. [19,866 m²]) and diamond grinding of the repaired areas, along with the placement of thin, bonded, polyester concrete bridge deck overlays. As in 2004, repair joints were doweled and located/skewed to match original joints and adjacent panels.

The total cost of concrete pavement repairs and grinding for this project are estimated as \$3,132,306. Bridge deck overlay work costs totaled approximately \$1,644,695. Traffic control, mobilization and other miscellaneous project costs were approximately \$432,599 and are due to both the pavement repair and grinding as well as the bridge deck overlays.

Table 10-1 summarizes general design, construction, service conditions, and rehabilitation information for this project.

Table 10-1. Summary of CO I-70 project design, construction and rehabilitation features.

Key Information	Data
Project Location	I-70 EB/WB, Garfield County, CO Milepost 86.5 to 97.0 4-lane divided freeway
Project Location	Interstate 70 EB and WB in Garfield, County, CO MP 86.5 – 97.0, including: • 10.177 centerline-miles [16.38 centerline-km] of roadway • 0.31 centerline-miles [0.50 centerline-km] of structures Dry-freeze climate
Highlighted Project Characteristics	Long service life through Concrete Pavement Restoration (CPR)
Original Construction Overview	1975-1976 construction; 4-lane divided freeway • Pavement: 8-inch [200mm] JPCP, 12-18-19-13 ft [3.66-5.49-5.79-3.96 m] panel length pattern with skewed, undoweled joints • Base/Subbase: 4-inch [100mm] lean concrete base, 6-inch [150mm] aggregate subbase • Subgrade: Variable, from poorly drained clayey loam to moderately well-drained sandy loam • Subdrainage: none • Shoulders: 10-ft [3.05m] tied concrete outside, 4-ft [1.22m] integral inside shoulder
Rehabilitation Details	2003 - Bridge approach panel repairs 2004 - Panel replacement/patching 2005 – Minor panel replacement/patching, blanket grinding 2014 – Extensive panel repairs, grinding (only replaced panels), bridge deck overlays
Traffic Data	1975 Original design estimates for 1995: • AADT: 11,800 • ADTT: 3,960 (33.6% trucks) 2018 actual AADT: 23,000 with 10.6% (2438) trucks
Unique/Noteworthy Features	Integral inside shoulder results in 16-ft [4.88m] inside panel width

Pavement Performance History

The generally good historical performance of this pavement section can be inferred from the nearly 30 years of maintenance-free service provided since initial construction in 1975-1976 until the 2003-2005 repair and grinding activities. The impact of the 2005 grinding on eastbound ride quality can be seen in Figure 10-7, which shows an average reduction in CDOT's ride index (a 0 – 100 index based on IRI) of 78 percent in the driving lane. Similar improvements were observed in the westbound lanes (IGGA 2008). The grinding was also highly effective in removing the wheel path ruts that had developed due to years of studded tire and tire chain wear.

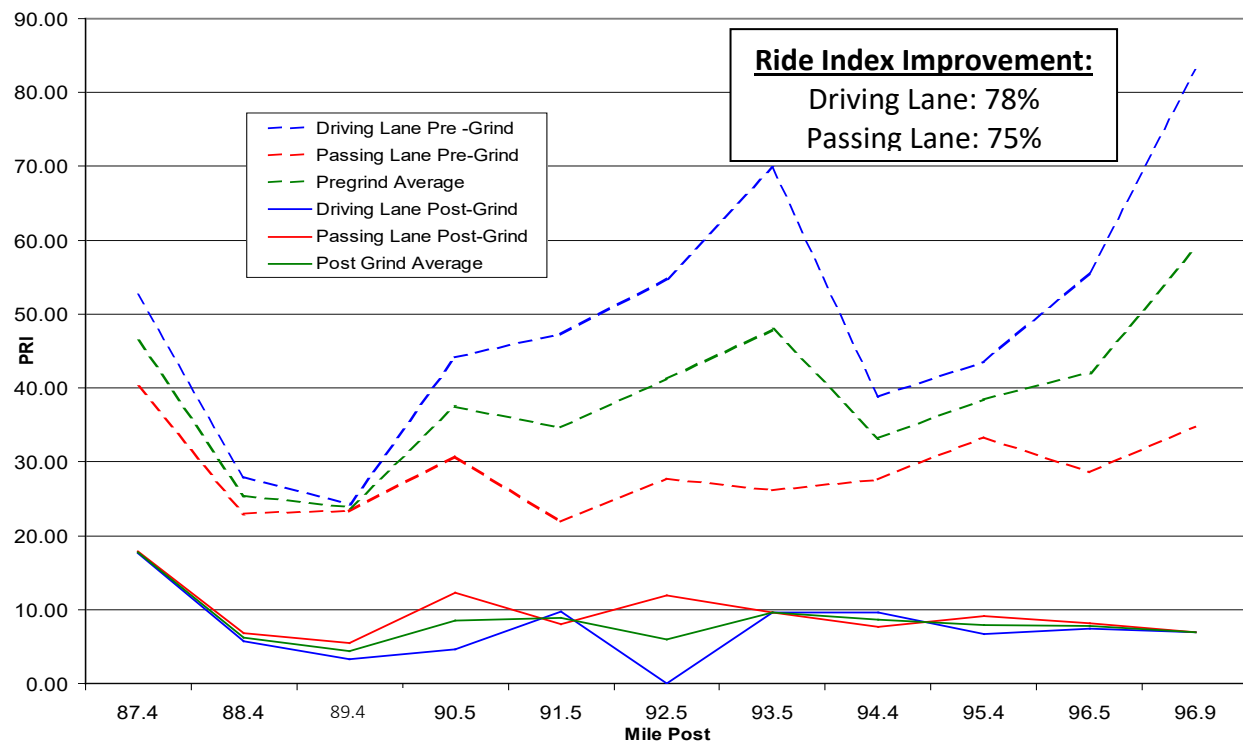


Figure 10-7. Impact of 2005 diamond grinding on eastbound ride index along project length (Necessary and Lemasany, 2006).

More recent performance trends are presented in table 10-2, which shows various CDOT performance index values for four included pavement subsections before and after the 2014 CPR work. The performance index values used by CDOT to quantify the ride quality or number and severity of distresses over 1/10-mile increments using a standard 0 – 100 scale (where 100 represents perfect ride or no distresses and 0 represents the worst possible ride or distress condition (CDOT 2005b). Table 10-2 also summarizes trends in “Driveability Life,” which is a CDOT performance metric intended to correlate with driver experience and represents the number of remaining years that the road is expected to remain acceptable for driving.

Table 10-2. Summary of CDOT performance indices for 2013 – 2017 (CDOT 2018)

Segment	IRI Index			Rut Index			T Crack Index			L Crack Index			Driveability Life (yrs)			Condition		
	2013	2016	2017	2013	2016	2017	2013	2016	2017	2013	2016	2017	2013	2016	2017	2013	2016	2017
EB MP 87-91.5	83	83	82	95	100	98	76	53	51	82	85	86	8	1	0	Mod.	Low	Low
EB MP 92-96.5	85	84	83	91	100	93	80	75	69	84	93	93	9	8	7	Mod.	Mod.	Mod.
WB MP 87-91.5	86	84	83	94	100	96	84	68	65	84	86	86	10	6	5	Mod.	Mod.	Mod.
WB MP 92-96.5	84	82	81	88	100	89	82	73	57	91	92	89	9	8	3	Mod.	Mod.	Low

This table shows IRI and rutting indexes remaining approximately level throughout the analysis period, while the longitudinal cracking index increases in 2016 (presumably because many longitudinally cracked panels were replaced in the 2014 rehabilitation) and remains approximately level in 2017. It is noteworthy, however, that the transverse cracking index exhibits a consistent decreasing trend throughout the analysis period (in spite of the fact that some cracked panels were probably replaced in 2014), and this increased transverse cracking appears to be the major contributor to a greatly reduced driveability life over the analysis period.

Conditions Observed during 2018 Site Visit

A site visit was conducted with CDOT staff in September 2018. The visit included office data collection, a high-speed pass of the full project in both directions, and a slow-speed shoulder pass of the full project in both directions (with frequent stops for closer inspection and photos taken from the shoulder). The overall ride quality was good, although there were localized areas of roughness and settlement.

Typical conditions consisted of original and/or replaced panels in generally good overall condition (see Figure 10-8) but often with longitudinal cracking (especially in the passing lane because of the very wide panels created by not sawing a longitudinal contraction joint with the inside shoulder – see Figure 10-9). There was also widespread shallow surface cracking that could be related to drying shrinkage or some type of material-related distress (e.g., ASR), as shown in Figure 10-10; sampling and testing would better define the nature of this cracking and its potential impact on future pavement performance. It can be seen in many of the photographs that the transverse joints are generally tight (with extruded joint sealant), possible evidence the panel expansion that often accompanies ASR.



Figure 10-8. Examples of typical good overall pavement condition.



Figure 10-9. Examples of typical longitudinal cracking in the inside lane (left) and outside lane (right).

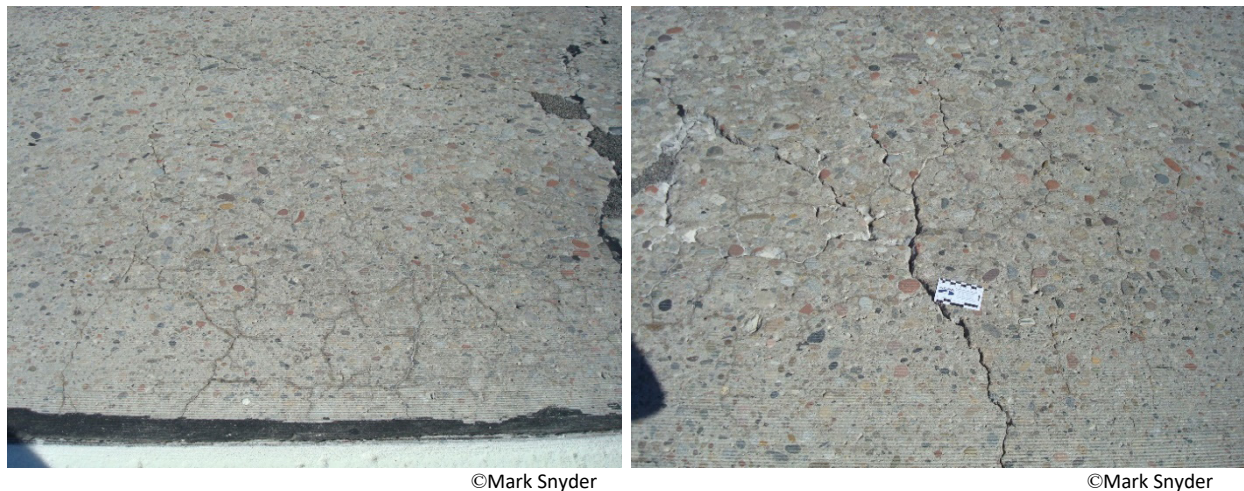


Figure 10-10. Examples of typical surface cracking (left) and deteriorating surface cracking (right).

There was very little surface texture remaining from the 2005 blanket diamond grinding, but more texture was observed in the panels that were apparently placed and diamond ground in 2014 (see Figure 10-11). Original panels were easily differentiated from 2004-2005 and 2014 repair panels by slight differences in color and significant differences in the particle sizes of the concrete aggregate (much larger stones in the original concrete), as shown in Figures 10-11 and 10-12.

Some areas exhibited high levels of distress, including shattered slabs (panels with several cracks, generally both longitudinal and transverse cracks), settled panels, and deterioration around and between cracks (see Figure 10-13). The previously described surface cracking was more severe in some areas as well, and there were a few areas where thin surface patching materials had been placed (apparently by CDOT maintenance forces, because this type of work was not described in the plans and bid documents for the 2003-2005 and 2014 rehabilitation activities), as shown in Figure 10-14.



Figure 10-11. Examples of typical surface texture in original pavement (left) and replaced panels (right).



Figure 10-12. Examples of visible concrete color and aggregate size differences between original pavement and full-depth repairs.



Figure 10-13. Examples of shattered slabs.



Figure 10-14. Examples of use of thin, partial-depth material along settled panel lane-shoulder joints.

An unusual observation was the presence of deformed reinforcing bars (probably tie bars) that were frequently visible on the surface of the westbound driving lanes (typically near the outer wheel path), especially near the east end of the project (see Figure 10-15). It appears that these bars were supposed to be in the longitudinal joint between travel lanes, but magnetic, ultrasonic, or other detection devices would be necessary to confirm.



Figure 10-15. Examples of displaced rebar at pavement surface in outer wheel path of westbound driving lane.

In summary, this section of pavement is still generally in serviceable condition more than 40 years after original construction, but it appears to be deteriorating quickly, and portions seem to need immediate attention. CDOT reports that either CPR or asphalt overlay is being planned for 2023 (October 3, 2019 e-mail communication with CDOT Region 3 Pavement Manager).

Economic Analysis

Table 10-3 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of I-70 documented in this case study. All costs were obtained from Michigan DOT contract documents. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 10-3. Activity and cost data for I-70.

Year	Activity	Activity Detail	Cost*
1976	Initial construction	453,759 s.y. [379,397m ²] of 8-inch [200mm] JPCP (two 12-ft [3.66m] travel lanes, 10-ft [3.05m] outside shoulder and 4-ft [1.22m] inside shoulder in each direction, over 10.177 centerline-miles [16.38 centerline-km])	\$3,975,175
2003	Bridge Approach Panel Repairs**	Bridge Approach Panel Repairs	--
2004	CPR	Panel replacement/patching (10,580 s.y. [8846 m ²])	\$1,239,469
2005	CPR	Slab replacement (1199 s.y. [1003 m ²]), blanket diamond grinding (329,912 s.y. [275,850 m ²])	\$1,855,548
2014	CPR	Slab replacement and diamond grinding (23,760 s.y. [19,866 m ²] of each)	\$3,132,306
2023	CPR (Estimated, based on input form CDOT)***	Slab replacement and diamond grinding (23,760 s.y. [19,866 m ²] of each)***	\$3,132,306***
2033	End-of-Life	Assumed Pavement End of Service	NA
*Pavement construction and repair costs only, shown in then-current dollars (Oman Systems, Inc. 2019c) **Not considered in analysis ***Quantities and costs assumed to be same as 2014 CPR (cost in 2014 dollars)			

A 57-year analysis period was used (1976-2033) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation and the assumed service intervals for the various activities shown in table 10-3 are summarized below:

- 1976 JPCP: 57 years (end-of-service: 2033)
- 2004 CPR: 10 years (end-of-service: 2014)
- 2005 CPR: 9 years (end-of-service: 2014)
- 2014 CPR: 9 years (end-of-service: 2023)
- 2023 CPR: 10 years (end-of-service: 2033)

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 10-4 shows the construction costs converted to 1976 dollars.

Table 10-4. Agency costs for I-70 project.

Year	Activity	1976 Dollars
1976	Initial construction	\$3,975,175
2004	CPR	\$425,549
2005	CPR	\$532,923
2014	CPR	\$709,704
2023	CPR	\$709,704
2033	Remaining Value (1976 Initial construction)	\$0
2033	Remaining Value (2014 CPR)	\$0

Figure 10-16 shows the expenditure stream diagram for the I-70 project using the four different discount rates used in the analysis.

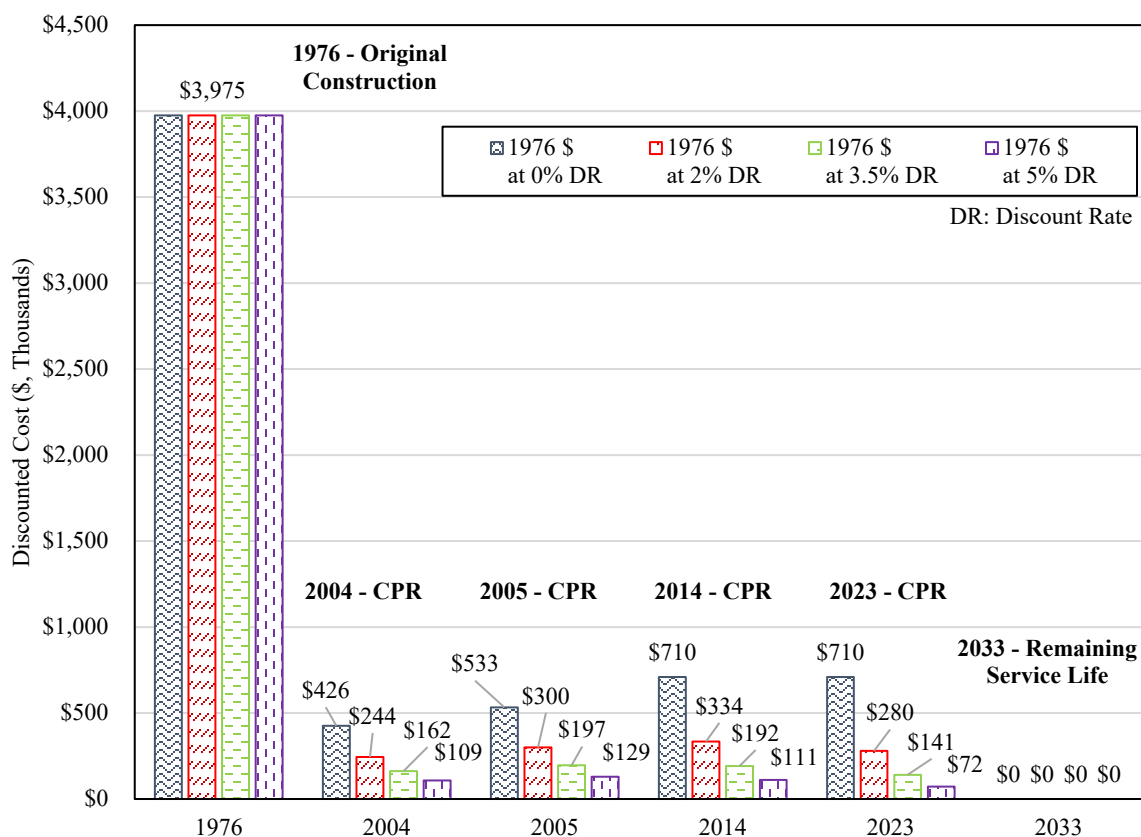
**Figure 10-16. Expenditure stream diagram for I-70.**

Table 10-5 summarizes the LCCA results for each discount rate considered.

Table 10-5. LCCA summary for I-70.

0% Discount Rate				
LCCA Element	1976 Dollars		2019 Dollars	
1976 Original Construction	PV =	\$3,975,175	PV =	\$18,338,737
2004 CPR	PV =	\$425,549	PV =	\$1,963,192
2005 CPR	PV =	\$532,923	PV =	\$2,458,543
2014 CPR	PV =	\$709,704	PV =	\$3,274,090
2023 CPR	PV =	\$709,704		\$3,274,090
Remaining Value (1976 PCC)	PV =	\$0	PV =	\$0
Remaining Value (2023 CPR)	PV =	\$0	PV =	\$0
Totals	NPV =	\$6,353,056	NPV =	\$29,308,653
	EUAC=	\$111,457	EUAC=	\$514,187
2% Discount Rate				
LCCA Element	1976 Dollars		2019 Dollars	
1976 Original Construction	PV =	\$3,975,175	PV =	\$18,338,737
2004 CPR	PV =	\$244,425	PV =	\$1,127,608
2005 CPR	PV =	\$300,096	PV =	\$1,384,436
2014 CPR	PV =	\$334,404	PV =	\$1,542,709
2023 CPR	PV =	\$279,814	PV =	\$1,290,870
Remaining Value (1976 PCC)	PV =	\$0	PV =	\$0
Remaining Value (2023 CPR)	PV =	\$0	PV =	\$0
Totals	NPV =	\$5,133,913	NPV =	\$23,684,360
	EUAC=	\$151,765	EUAC=	\$700,138
3.5% Discount Rate				
LCCA Element	1976 Dollars		2019 Dollars	
1976 Original Construction	PV =	\$3,975,175	PV =	\$18,338,737
2004 CPR	PV =	\$162,413	PV =	\$749,261
2005 CPR	PV =	\$196,514	PV =	\$906,583
2014 CPR	PV =	\$192,019	PV =	\$885,844
2023 CPR	PV =	\$140,890	PV =	\$649,971
Remaining Value (1976 PCC)	PV =	\$0	PV =	\$0
Remaining Value (2023 CPR)	PV =	\$0	PV =	\$0
Totals	NPV =	\$4,667,011	NPV =	\$21,530,397
	EUAC=	\$190,099	EUAC=	\$876,986
5% Discount Rate				
LCCA Element	1976 Dollars		2019 Dollars	
1976 Original Construction	PV =	\$3,975,175	PV =	\$18,338,737
2004 CPR	PV =	\$108,555	PV =	\$500,798
2005 CPR	PV =	\$129,472	PV =	\$597,294
2014 CPR	PV =	\$111,144	PV =	\$512,740
2023 CPR	PV =	\$71,644	PV =	\$330,517
Remaining Value (1976 PCC)	PV =	\$0	PV =	\$0
Remaining Value (2023 CPR)	PV =	\$0	PV =	\$0
Totals	NPV =	\$4,395,989	NPV =	\$20,280,086
	EUAC=	\$234,321	EUAC=	\$1,080,998
PV: Present Value				
NPV: Net Present Value				
EUAC: Equivalent Uniform Annual Cost				

As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 31 percent and the equivalent uniform annual cost (EUAC) increases by 111 percent, respectively. This is graphically represented in Figure 10-17.

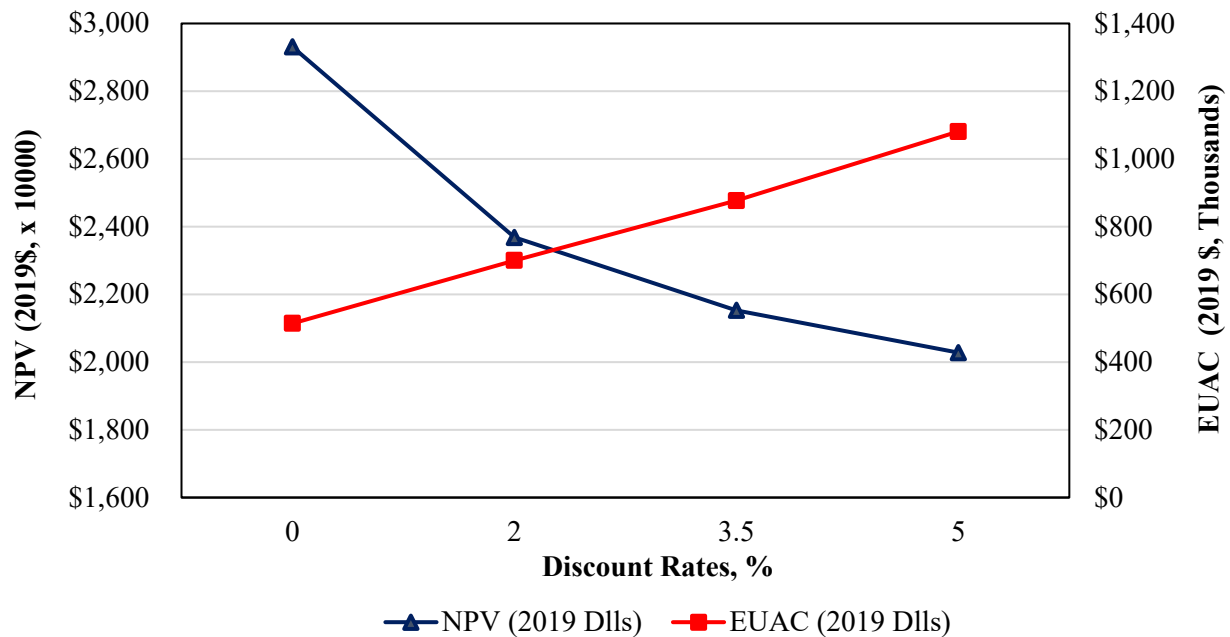


Figure 10-17. NPV and EUAC (2019 Dollars) vs discount rate.

Lessons Learned

The JPCP segment of I-70 between Rifle and Silt is an example of a 20-year concrete pavement design that has far exceeded original expectations to provide more than 40 years of service. This success has been due, at least in part, to the application of timely rehabilitation strategies and some effective design concepts. Based on the LCCA conducted, future maintenance costs are estimated to be between 10 and 37 percent of the original JPCP construction cost. The lessons learned from this pavement section include:

- **A good stable base contributed to the long service life.** The pavement thickness was only 8 inches [200 mm], which is relatively thin for interstate highway pavement. That thickness was likely significantly decreased by the diamond grinding (which removed studded tire and chain damage) in 2005. Further, the original joints were undoweled and the pavement was undrained. However, the pavement was constructed on a lean concrete base and aggregate subbase, all on top of alluvial subgrade material that is likely to have provided relatively uniform support.
- **CPR treatments helped to maintain pavement serviceability and extend pavement service life.** Two major rehabilitation efforts were undertaken to repair and replace cracked panels (and to improve panel support in those areas) after approximately 30 and 40 years of service. Blanket diamond grinding in 2005 has certainly contributed to pavement serviceability, and repair grinding in 2014 has helped to maintain good IRI levels.
- **Pavement performance would have been further improved with current jointing practices.** Much of the longitudinal cracking observed on this project could have been

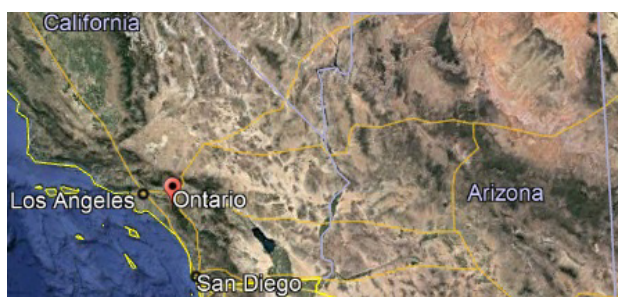
avoided by providing a contraction joint between the inside travel lane and inside shoulder. Much of the transverse cracking observed could probably have been avoided with the use of shorter panels (i.e., 15 ft [4.6 m] or less), perpendicular joints, and doweled joints. This would have reduced slab curl stresses (especially on the lean concrete base) and delayed the onset of panel cracking.

- **Current material selection and/or curing practices may have further improved pavement performance.** Widespread pattern cracking on the concrete pavement surface suggests either drying shrinkage or a slow-developing alkali-silica reaction problem. The progression and deterioration of this cracking in some areas, together with signs of slab expansion (tight joints, extruded joint sealant, etc.) also point to a possible ASR issue. Avoiding such issues through current material selection practices and curing practices may have further improved the performance of this pavement section.

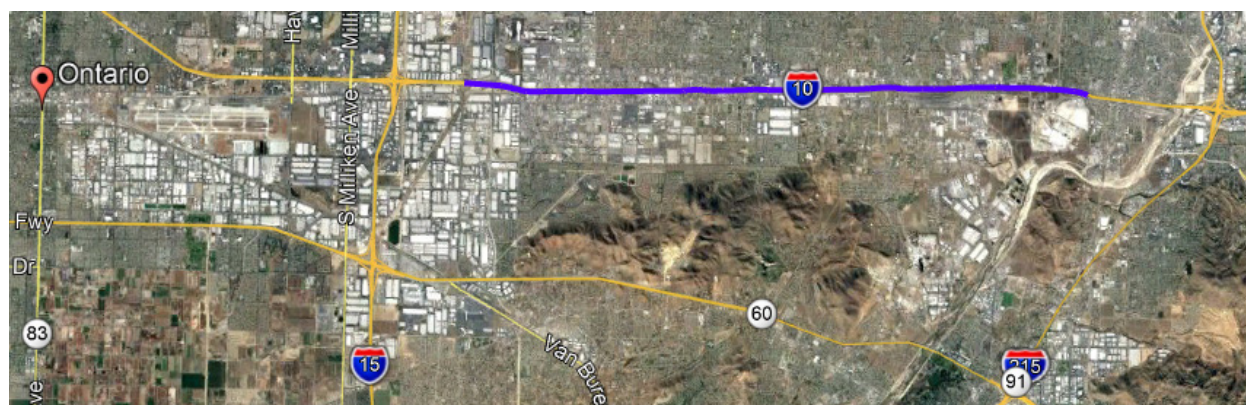
CHAPTER 11. CONCRETE PAVEMENT RESTORATION ON I-10 IN CALIFORNIA

Introduction

In 1947, the California Department of Highways (currently Caltrans) completed the construction of an 8-inch (200 mm) thick, un-doweled, jointed plain concrete pavement (JPCP) in San Bernardino County, east of Ontario, CA (figure 11-1). This 4-lane, divided highway, was originally designated as U.S. 70-99, and would eventually become Interstate 10 (I-10). The original concrete section was approximately 10 mi. (16 km) in length, with two 12-foot (3.66 m) lanes in each direction with asphalt concrete shoulders (8-foot [2.4 m] wide outer, and 3-foot [0.9 m] wide inner). Concrete joint spacing (weakened plane) was 15 feet (4.5 m) with tie bolt contact joints every 60 feet (18 m). This project was constructed through agricultural and some uncultivated land.



© 2018 Google Earth; Data: SIO, NOAA, U.S. Navy, NGA, GEBCO, INEGI, Landsat/Copernicus.



© 2018 Google Earth; Data: Google, Airbus. (Added line to indicate project location).

Figure 11-1. I-10 project location.

In 1966, approximately 7.5 mi. (12 km) of this section of I-10 was widened (2 inner lanes added in each direction) and converted into an 8-lane divided interstate highway. A majority of the original 1947 pavement remains in-service today under heavy traffic conditions, a 70+ year service life. General project information is summarized in table 11-1.

Table 11-1. I-10 project details.

Key Information	Data
Project Location	I-10 WB/EB, east of Ontario, Ca PM 11.0 – R21.0 (San Bernardino) Original: Rural 4-lane divided freeway Current: Urban 8-lane divided interstate
Climatic Region	Dry, non-freeze
Highlighted Project Design Characteristics	Concrete pavement restoration
Year Construction Completed	1947
Rehabilitation Types, Years¹, and Details	Diamond grinding: 1966 & 1967 Grooving: 1976 & 1982 Slab replacement & diamond grinding: 1991 Slab replacement: 1993 Slab replacement & grinding 1998 Slab replacement: 2001 Diamond grinding: 2006 & 2012 Slab replacement and grinding: 2016
Traffic Data	1941 ADT: 7,328 (2,085 trucks) 1960 Design ADT: 34,000 (1,700 trucks) 1975 Design ADT: 113,000 (3,390 trucks) 2016 AADT: 203,000 (20,300 trucks) ²
Slab Design Details (Original 1947 construction)	Slab thickness: 8 inch (200 mm) Weakened plane joint spacing: 15 ft. (4.5 m) Load transfer: none, tie bolts at longitudinal and transverse construction joints Tie bars: 5/8-inch (16-mm) diameter x 30 inch (760 mm) tie bolt
Base/Subbase	3 inch (75 mm) bituminous-treated, granular base with 3/4 inch (19 mm) max aggregate size)
Subgrade	Selected material, unknown cement treated subgrade
Subdrainage	None
Shoulder Type and Design	Asphalt concrete
Unique/Noteworthy Features	None

¹Not all CPR treatments were done on the entire segment.

²Data from Caltrans Division of Traffic Operations (2016).

Original Pavement Design, Materials and Construction

Structural Design

The JPCP was constructed for a total cost of \$1,387,000. Construction of the approximately 10-mile (16 km) project began in January 1946 and the work was completed in April 1947. This project included approximately 62,000 yd³ (47,400 m³) of Class B concrete at \$9.00/yd³ (\$11.78/m³) for a total cost of \$558,000.

The pavement structure consisted of an 8-inch (200 mm) JPCP on 3 inches (75 mm) of bituminous-treated base material. From the special provisions, the pavement did not include dowels (by saying omitted it sounds as if dowels were a standard practice at the time and it was not included in these projects) from the weakened plane joints (every 15 ft [4.5 m]) and instead tie bolt assemblies were used for longitudinal and transverse contact joints (every 4th transverse joint). Tie bolts were installed at 24-inch (600 mm) spacing across transverse joints with a minimum distance of 5 inches (125 mm) from the slab edge.

Paving Materials

A Caltrans Class B portland cement concrete was used to pave the road section. The average 28-day compressive strength from 89 samples was reported to be 3,503 psi (24 MPa) (California Division of Highways 1947). The 1945 Caltrans specifications for Class B concrete required the mixture to be held constant at 470 lbs of cement per cubic yard (280 kg/m³) of concrete. Per the Caltrans standard specifications, the cement was either Type I or Type II for a Class B concrete that was supplied by California Portland Cement Company. According to the specifications, the amount of water used in the mix was determined by the slump tests AASHTO T119-42. For concrete pavements, a maximum 2-inch (50 mm) slump was required for a Class B concrete and not more than 60 lb. (27 kg) of water per sack could be used. All aggregate for subgrade stabilization was crushed to a maximum size of ¾ in (19 mm) and mixed with RC3 and RC4 liquid asphalt at 4.5 to 5.5 percent by weight of the aggregate.

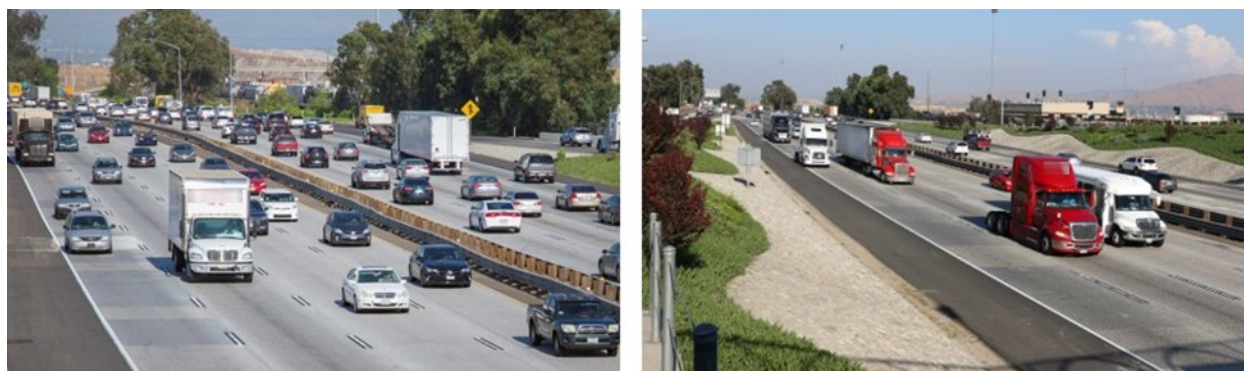
Construction

Bituminous-treated base material was selected to reduce chances of pumping subgrade fines (Kane 1946). This material was placed on a 24 ft. (7.0 m) wide prepared grade and spread and compacted in two lifts: 1) the first lift was 2-inches thick and was spread by a motor grader and compacted with loaded trucks, and 2) the second lift was 1-inch thick and was spread by an asphalt concrete finisher and compacted with a 12-ton, three-wheel roller and 8-ton tandem roller.

The concrete mixture was produced in an automatic batching plant. Aggregate and bulk cement were delivered directly to the plant. A dual-drum mixer was used to produce the concrete in 1-1/3 yd³ (1 m³) batches. The 12-ft (3.6 m) wide lanes were hand-formed and paved with a mechanical reversible, screw type concrete spreader with vibrators at each end to spread the concrete onto the prepared grade. This operation was followed by two automatic finishing, floating and tamping machines. The final floating was done manually, and a broom finish was applied. Finally, the pavement was cured using a liquid membrane system.

Traffic and Service Conditions

The pavement was designed for an “*estimated equivalent 5000 lb. wheel loads in 10 years of 30,950,000 [2-way].*” The average daily traffic in 1941 was 7,328 vehicles (2,085 trucks). In 2016, this section of I-10 in California served an average daily traffic of 203,000 vehicles with about 10 percent trucks (California Division of Traffic Operations 2016). Figure 11-2 shows a recent picture of typical traffic.



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Figure 11-2. Typical present-day traffic.

Project Maintenance and Rehabilitation

After 70+ years of service, the original concrete lanes of I-10 remain in service and have received several diamond grinding treatments along with intermittent slab replacements. This was the first diamond grinding treatment to occur and was constructed in 1966 (19 years after construction) and was the first performed in California and perhaps the first in the United States. The corridor began to experience roughness from upward slab curling and faulting. The roughness measurements performed with the California standard profilograph for some sections were 25 in/mi (1 m/km) (Caltrans allowable value was 7 in/mi [0.3 m/km]). Diamond grinding was selected because an experimental diamond grinding test section, constructed early in the 1960s, on I-10 demonstrated the economic feasibility of the strategy to improve ride quality as roughness was reduced to 1 in/mi (0.04 m/km) after the diamond grind. Due to the success of this practice, Caltrans and many other agencies have adopted diamond grinding as a preservation practice. The most recent diamond grinding on all eight lanes occurred in 2016.

Field Visit

A site visit to the project was performed in 2018. Distresses observed during the site visit include a few low and medium severity mid-panel cracks, surface wear, joint sealant deterioration, isolated spalling (particularly at the longitudinal joint) that has been patched with asphalt material, and one area of slab settlement. Figures 11-3 and 11-4 present examples of the 2018 pavement condition.



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Figure 11-3. View (towards oncoming traffic) of the existing concrete pavement in 2018. Left two lanes are original concrete pavement.



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Figure 11-4. Example view of existing concrete pavement in 2017 showing replaced slabs. Left two lanes (in photo) are original concrete pavement.

Pavement Performance and Maintenance History

As discussed, the first major diamond grinding treatments took place in 1966 and 1967 to reduce roughness caused by slab curling and faulting. Historical information shows average faulting in the outside travel lane of 0.1 inch (2.5 mm) in 1957 and an increase to 0.15 inch (3.75 mm) in 1961. This grind removed 0.10 – 0.15 inches (2.5 – 3.75 mm) near the transverse joints and “skimmed” the surface of the remainder of the slab. Cost of diamond grinding was between \$1.22 and \$1.50 per square yard (\$1.46 and \$1.79 per square meter). The amount of grinding was approximately 1,143,000 yd² (955,700 m²) Prior to 1967, 14 percent of the original pavement was replaced.

The ride of the concrete, prior to the 1966 diamond grind, was described by H.C. Prentice, Assistant District Engineer, Dist. 08, Calif. State Div. of Highways as follows, “*When the roadway is traveled, a vibration is set up in the vehicle. Empty trucks will bounce to the point that the tires leave the pavement surface. The four-rear wheels of a six-wheel unladen truck have been observed to vibrate vertically with an amplitude of as much as 6 (Prentice 1966).*”

Table 11-2 highlights CPR work performed within the limits of the original 1947 concrete pavement. Some CPR work listed in table 11-2 was done only on a portion of the original project limits. Major diamond grinding treatments occurred in 1966/1967, 1998 and again in 2006. Caltrans records indicate that approximately 25 to 29 percent of the original 1947 concrete pavement has been replaced after 70+ years of service.

Table 11-2. Summary of work performed on I-10 segment.

Year Completed	CPR Treatment	Approximate Quantity		
		Replace Slab (yd ³)	Grinding (yd ²)	Grooving (yd ²)
1966	Diamond grind outer lane		96,900	
1967	Diamond grind spot locations		47,000	
1976	Grooving all lanes, various locations			47,200
1976	Grooving in westbound lanes (1.3-mile segment)			15,300
1982	Grooving in westbound lanes (0.2-mile segment)			25,300
1991	Diamond grinding outside two lanes and minor slab replacement (1-mile segment)	23	28,700	
1993	Minor slab replacement	33		
1998	Diamond grinding (all lanes) and slab replacement	140	253,000	
2001	Slab replacement	199		
2006	Diamond grinding (all lanes)		253,000	
2012	Diamond grinding (all lanes, eastern edge of project limits)		28,160	
2016	Diamond grinding (all lanes) and slab replacement	11,100	253,000	

1 yd²=0.84 m²; 1 yd³=0.76 m³

Figure 11-5 shows time series IRI data from 2015 through 2018 for lanes 3 and 4, in both directions. Available Caltrans data showed that the average IRI for both outer lanes (3 and 4) in the eastbound and westbound directions, prior to the last CPR, were 182.1 in/mi (2.8 m/km) and 162.6 in/mi (2.5 m/km), respectively. Two years after CPR these IRI values in the eastbound and westbound directions remained below 80 in/mi (1.3 m/km) at 73.4 in/mi (1.1 m/km) and 76.6 in/mi (1.2 m/km), respectively. Caltrans IRI threshold for a poor ride is set at 170 in/mi (2.6 m/km) (Caltrans 2018), this shows the efficacy of diamond grinding.

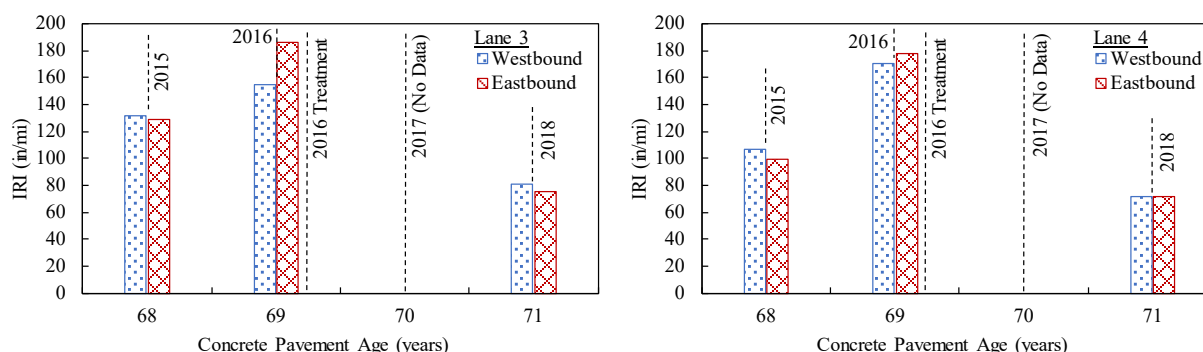


Figure 11-5. Time-series IRI performance data (created from Caltrans data).

Economic Analysis

Table 11-3 summarizes the agency costs for the initial construction and pavement rehabilitation activities that have been performed on the stretch of I-10 documented in this case study. All costs were obtained from the Caltrans contract documents. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 11-3. Activity and cost data for the I-10 project.

Year	Activity	Activity Detail	Cost*
1947	Original Construction	8-inch JRCP	\$1,387,000
1966	CPR	Grind outside lane PM 11.0 to 12.5 and Grind #4 lane PM 11.0 to 19.6	\$48,091
1967	CPR	Grind #4 lane PM 17.3 to 21	\$24,193
1976	CPR	Grooving various locations PM 11 to 20.3 and Grooving PM 15.7 to 17.0	\$70,320
1982	CPR	Grooving PM 11 to 11.2	\$44,826
1991	CPR	Slab Replacement PM 20 to 21 and Grind outside lanes PM 20 to 21	\$63,169
1993	CPR	Slab Replacement PM 13.0 to 18.8	\$7,656
1998	CPR	Slab Replacement PM 11 to 20 and Grinding outside (1946 pavement) PM 11 to 20	\$1,050,995
2001	CPR	Slab Replacement PM 11.8 to 19.8	\$37,810
2006	CPR	Grind (full 8-lanes) PM 11 to 21	\$794,420
2012	CPR	Grind (full 8 lanes) PM 20 to 21	\$125,919
2016	CPR	Grind PM 11 to 20 and Slab Replacement entire area PM 11 to 21	\$4,995,694
2020	End-of-Life	Assumed Pavement End of Service	NA

*Costs shown in then-current dollars

A 72-year analysis period was used (1947-2019) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation.

The nominal dollars were converted to constant base-year dollars using the using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 11-4 shows the construction costs converted to 1947 dollars.

Table 11-4. Agency costs for I-10 converted to 1947 dollars.

Year	Activity	1947 Dollars
1947	Initial Construction	\$1,387,000
1966	CPR	\$36,371
1967	CPR	\$17,515
1976	CPR	\$24,810
1982	CPR	\$10,061
1991	CPR	\$11,672
1993	CPR	\$1,404
1998	CPR	\$164,515
2001	CPR	\$5,187
2006	CPR	\$69,563
2012	CPR	\$10,569
2016	CPR	\$404,436
2019	Remaining Value (1947 Initial Construction)	(\$19,000)
2019	Remaining Value (2016 CPR)	(\$101,109)

Figure 11-6 shows the expenditure stream diagram for the I-10 project using the four different discount rates used in the analysis and table 11-5 summarizes the LCCA results.



Figure 11-6. Expenditure stream diagram for I-10 project.

Table 11-5. LCCA summary for I-10.

0% Discount Rate				
LCCA Element	1947 Dollars		2019 Dollars	
1947 Initial Construction	PV =	\$1,387,000	PV =	\$18,135,626
1966 CPR	PV =	\$36,371	PV =	\$475,564
1967 CPR	PV =	\$17,515	PV =	\$229,017
1976 CPR	PV =	\$24,810	PV =	\$324,408
1982 CPR	PV =	\$10,061	PV =	\$131,554
1991 CPR	PV =	\$11,672	PV =	\$152,622
1993 CPR	PV =	\$1,404	PV =	\$18,361
1998 CPR	PV =	\$164,515	PV =	\$2,151,100
2001 CPR	PV =	\$5,187	PV =	\$67,820
2006 CPR	PV =	\$69,563	PV =	\$909,564
2012 CPR	PV =	\$10,569	PV =	\$138,195
2016 CPR	PV =	\$404,436	PV =	\$5,288,176
Remaining Value (1947 Initial Construction)	PV =	(\$19,000)	PV =	(\$248,433)
Remaining Value (2016 CPR)	PV =	(\$101,109)	PV =	(\$1,322,044)
Totals	NPV =	\$2,022,995	NPV =	\$26,451,531
	EUAC =	\$28,097	EUAC =	\$367,382
2% Discount Rate				
LCCA Element	1947 Dollars		2019 Dollars	
1947 Initial Construction	PV =	\$1,387,000	PV =	\$18,135,626
1966 CPR	PV =	\$24,966	PV =	\$326,442
1967 CPR	PV =	\$11,787	PV =	\$154,122
1976 CPR	PV =	\$13,971	PV =	\$182,678
1982 CPR	PV =	\$5,031	PV =	\$65,781
1991 CPR	PV =	\$4,884	PV =	\$63,857
1993 CPR	PV =	\$565	PV =	\$7,384
1998 CPR	PV =	\$59,923	PV =	\$783,523
2001 CPR	PV =	\$1,780	PV =	\$23,278
2006 CPR	PV =	\$21,626	PV =	\$282,763
2012 CPR	PV =	\$2,918	PV =	\$38,149
2016 CPR	PV =	\$103,143	PV =	\$1,348,634
Remaining Value (1947 Initial Construction)	PV =	(\$4,566)	PV =	(\$59,703)
Remaining Value (2016 CPR)	PV =	(\$24,298)	PV =	(\$317,712)
Totals	NPV =	\$1,608,729	NPV =	\$21,034,822
	EUAC =	\$42,353	EUAC =	\$553,780

Table 11-5. LCCA summary for I-10 (continued).

3.5% Discount Rate				
LCCA Element	1947 Dollars		2019 Dollars	
1947 Initial Construction	PV =	\$1,387,000	PV =	\$18,135,626
1966 CPR	PV =	\$18,918	PV =	\$247,367
1967 CPR	PV =	\$8,802	PV =	\$115,096
1976 CPR	PV =	\$9,149	PV =	\$119,625
1982 CPR	PV =	\$3,018	PV =	\$39,463
1991 CPR	PV =	\$2,569	PV =	\$33,592
1993 CPR	PV =	\$289	PV =	\$3,773
1998 CPR	PV =	\$28,461	PV =	\$372,137
2001 CPR	PV =	\$809	PV =	\$10,582
2006 CPR	PV =	\$9,139	PV =	\$119,496
2012 CPR	PV =	\$1,130	PV =	\$14,770
2016 CPR	PV =	\$37,667	PV =	\$492,518
Remaining Value (1947 Initial Construction)	PV =	(\$1,596)	PV =	(\$20,869)
Remaining Value (2016 CPR)	PV =	(\$8,493)	PV =	(\$111,056)
Totals	NPV =	\$1,496,862	NPV =	\$19,572,120
	EUAC =	\$57,195	EUAC =	\$747,845
5% Discount Rate				
LCCA Element	1947 Dollars		2019 Dollars	
1947 Initial Construction	PV =	\$1,387,000	PV =	\$18,135,626
1966 CPR	PV =	\$14,393	PV =	\$188,197
1967 CPR	PV =	\$6,601	PV =	\$86,314
1976 CPR	PV =	\$6,028	PV =	\$78,814
1982 CPR	PV =	\$1,824	PV =	\$23,850
1991 CPR	PV =	\$1,364	PV =	\$17,836
1993 CPR	PV =	\$149	PV =	\$1,946
1998 CPR	PV =	\$13,663	PV =	\$178,651
2001 CPR	PV =	\$372	PV =	\$4,866
2006 CPR	PV =	\$3,910	PV =	\$51,129
2012 CPR	PV =	\$443	PV =	\$5,797
2016 CPR	PV =	\$13,957	PV =	\$182,492
Remaining Value (1997 Concrete Overlay)	PV =	(\$566)	PV =	(\$7,406)
Remaining Value (2016 CPR)	PV =	(\$3,014)	PV =	(\$39,411)
Totals	NPV =	\$1,446,124	NPV =	\$18,908,700
	EUAC =	\$74,528	EUAC =	\$974,485
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost				

As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 29 percent and the equivalent uniform annual cost (EUAC) increases by 165 percent, respectively. This is graphically represented in Figure 11-7.

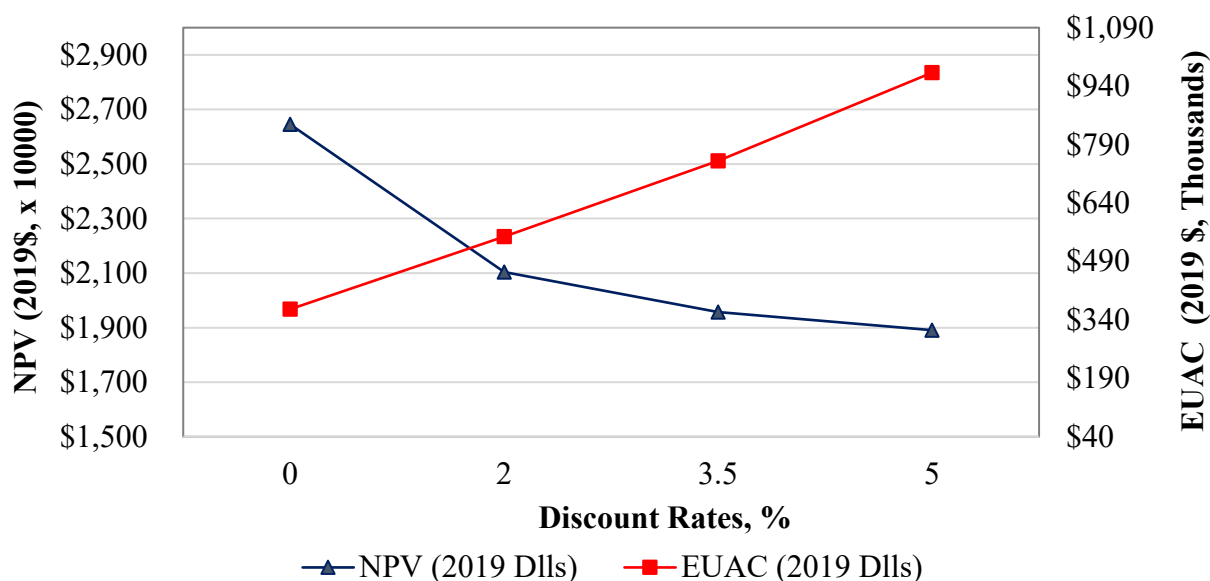


Figure 11-7. NPV and EUAC (2019 Dollars) vs discount rate.

Lessons Learned

A 10-mi (16-km) section of I-10 east of Ontario, California is an excellent example of how CPR techniques were used to extend a pavement service life to more than 70 years. As of 2018, average IRI was less than 80 inches/mi (1.3 m/km) and the pavement remains in good condition despite heavy traffic and pavement age. Based on the LCCA conducted, future maintenance costs are estimated to be between 4 and 32 percent of the original construction cost, depending on the discount rate used. The lessons learned from this pavement section include:

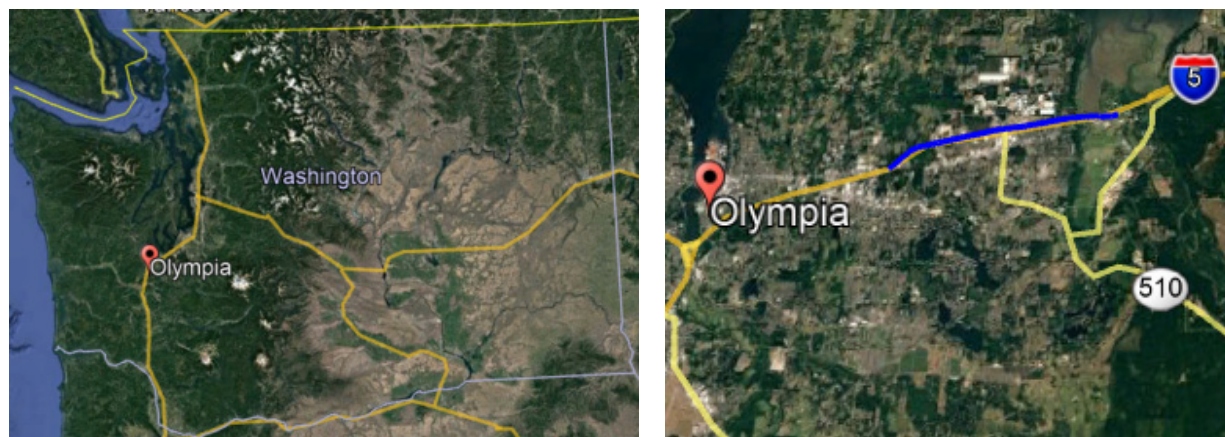
- **A good stable base contributed to the long service life of this concrete pavement.** The 3-inch bituminous-treated base provided good support for the pavement, even though the most heavily trafficked lanes largely remain undoweled (every 4th joint had tie bolts). The base continues to provide good service to this day.
- **CPR treatments effectively maintained a high level of serviceability.** These included a combination of diamond grinding, grooving (for skid resistance), and slab replacements that extended the pavement service life to 70+ years of service.
- **Diamond grinding is a very effective CPR treatment to maintain smoothness.** After 70+ years of service and through four major diamond grinding projects, the pavement has been maintained with good ride quality proving the efficacy of this method.

CHAPTER 12. CONCRETE PAVEMENT REHABILITATION ON I-5 IN WASHINGTON

Introduction

In 1969, the Washington State Highway Commission Department of Highways (forerunner to the Washington State Department of Transportation) constructed an 8.5-inch (216-mm) jointed plain concrete pavement (JPCP) on I-5 east of Olympia (see figure 12-1). Transverse joints are not doweled and are spaced at 15-ft (4.6-m) intervals on a 1:6 skew. The original concrete section was 8.2 mi (13.2 km) long, of which 5.3 mi (8.6 km) remain in service. The roadway consists of three 12-ft (3.6 m) lanes in each direction.

Because of joint faulting and rideability issues, WSDOT performed major concrete pavement restoration (CPR) on the outer (truck) lane of this project in 1996 (northbound direction) and 1997 (southbound direction). This work included the installation of subdrains, dowel bar retrofit of the undoweled transverse joints, cross stitching of cracks, intermittent slab replacements and partial-depth repairs, diamond grinding, and joint resealing. A similar CPR project consisting of DBR and diamond grinding was done in 2010 in the center lane (both directions).



(L) © 2018 Google Earth; Data: SIO, NOAA, U.S. Navy, NGA, GEBCO, INEGI, Landsat/Copernicus. (R) © 2018 Google Earth; Data: Google, Airbus. (Added line to show project limits).

Figure 12-1. Project location.

Original Pavement Design, Materials and Construction

Structural Design

Project construction began in January 1968 and the work was completed in May 1969. Based on the available records, the original 8.5-inch (216-mm) JPCP pavement was placed on a 3- or 4.2-inch (76 or 107-mm) asphalt-treated base layer. Table 12-1 provides general information and pavement design details for the original and rehabilitated concrete pavements.

Paving Materials

The construction drawings show that mainline concrete was paved with a 14-day, 6-sack mix (cement factor of 564 lb/yd³ [335 kg/m³]). The 14-day compressive and flexural strengths were specified as “...not less than ninety (90) percent of that developed by concrete made from the same cement and washed sand and gravel from Steilacoom [Washington] of the same grading, and mixed in the same proportions, and to the same consistency (WSHC 1963).” No other concrete mixture details were available.

Table 12-1. I-5 project details.

Key Information	Data
Project Location	I-5 NB/SB, Olympia, WA Milepost 109.21 – 114.54 (state) Rural/urban 6-lane divided interstate
Climatic Region	Wet, non-freeze
Highlighted Project Design Characteristics	Long-Life Concrete Pavement Restoration
Initial Construction Year	1969
Rehab Types and Years	1996/1997 (Lane 1) (installation of subdrains, dowel bar retrofit, slab replacements, cross-stitching cracks, partial-depth repair, diamond grind, and joint resealing) 2010 (Lane 2) (same as 1996/1997 without crack cross-stitching) 2014 (minor partial-depth repair by state forces)
Traffic Data	1962 (design) ADT: 20,800 (4% trucks) 1985 (design) ADT: 52,500 (4% trucks) 2017 AADT: 120,000 (~10.6% trucks) 1.9 million ESALs/year (2016 data)
Pavement Design Details	8.5-inch JPCP 15-foot transverse joint spacing (skewed) No dowels #4 centerline tie bars 2 ft. long @ 3 ft centers 12-foot traffic lanes
Base/Subbase	0.25 or 0.35-ft asphalt-treated base
Subgrade	Unknown
Subdrainage	None originally, edge drains installed in 1996/1997
Shoulder Type and Design	10 ft HMA outside shoulder and 6 ft HMA inside shoulder placed on asphalt-treated base
Unique/Noteworthy Features	None

Construction

The construction of embankments and additional grading to lower the grade to meet the minimum clearance at existing underpasses started on January 6, 1968 and paving began June 6, 1968. It consisted of concrete and hot-mix asphalt (HMA) paving on an asphalt-treated base. The concrete pavement was placed with a slipform paver, a finishing machine, and curing spray rigs. The average concrete paving rate was 4,000 linear feet (1219 m) of 36-ft (10.9-m) wide roadway per day.

A final project summary document stated that the contractor struggled to achieve the desired 4% air content in the concrete throughout the project. In addition, two instances were noted where random cracking developed in newly constructed concrete; the first a result of the contractor's inability to supply water, and the second a combination of hot weather and when "joints were hand sawed with only two saws." The contractor repaired the cracked concrete. Notes also stated that two additional sacks of cement were added for approximately one-mile of 24 ft wide paving to expedite opening-to-traffic. Overall, the project work was considered to be minimal standard or substandard (WSHC 1969).

Traffic and Service Conditions

Interstate 5 serves as the primary north-south route in Washington State carrying a significant amount of freight traffic servicing the West Coast. Truck traffic is generated by the ports of Seattle and Tacoma as well as industries in the area (e.g., aerospace, agriculture, technology, and software). According to the 2016 Annual Traffic Report, the estimated AADT in 2016 was 108,000 (1.9 million ESALs) with about 10 percent truck traffic (WSDOT 2016). Most of the truck traffic on I-5 is typical AASHTO double-unit, Class 8, 9, and 10 trucks (see figure 12-2). Studded tires are legal during the winter months on this section of I-5, which has resulted in some concrete surface wear.



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Figure 12-2. Typical traffic on this section of I-5.

Project Maintenance and Rehabilitation

This concrete section has been in service for nearly 50 years. As previously indicated, the first major CPR was performed in the outer (truck) lane in 1996 (northbound direction) and 1997 (southbound direction). The work included the installation of subdrains, dowel bar retrofit of the original undoweled joints, cross-stitching of cracks, intermittent slab replacements, partial-depth repair, diamond grinding, and joint resealing. A similar CPR project—consisting of DBR and diamond grinding—was performed in 2010 in the center lane in both directions. Minor partial-depth repair was performed in 2014 by state maintenance crews.

Field Visit

A project site visit was performed in 2018. Distresses observed during that visit included low- and medium-severity mid-panel cracks, surface wear, isolated joint and crack spalling, and severe wheel path wear (from studded tires) in some panels replaced during the previous CPR projects. Figures 12-3 through 12-7 present examples of the 2018 pavement condition.



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Figure 12-3. View of outer lane and adjacent shoulder (northbound direction).



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Figure 12-4. View of the northbound project showing dowel bar retrofitting and texture produced by diamond grinding in outer (truck) lane) constructed in 1996.



© APTech

Figure 12-5. Slab replacement in outer (truck) lane and medium-severity transverse crack in center and inner lanes.



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Figure 12-6. Replaced slabs showing signs of severe surface wear from studded tires.



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Figure 12-7. Dowel bar retrofit across transverse joint and crack.

Pavement Performance and Maintenance History

WSDOT uses a pavement rating system based on three different metrics: 1) pavement structural condition (PSC), 2) pavement rutting condition (PRC), and 3) pavement profile condition (PPC). PSC is derived from observed pavement distress and ranges from a value of 100 (for a pavement with no distress) to a value of 0 (for a pavement with extensive and severe distresses). PRC is a measure of rut depth, being converted to a 0 to 100 scale in which 100 indicates no rutting and 0

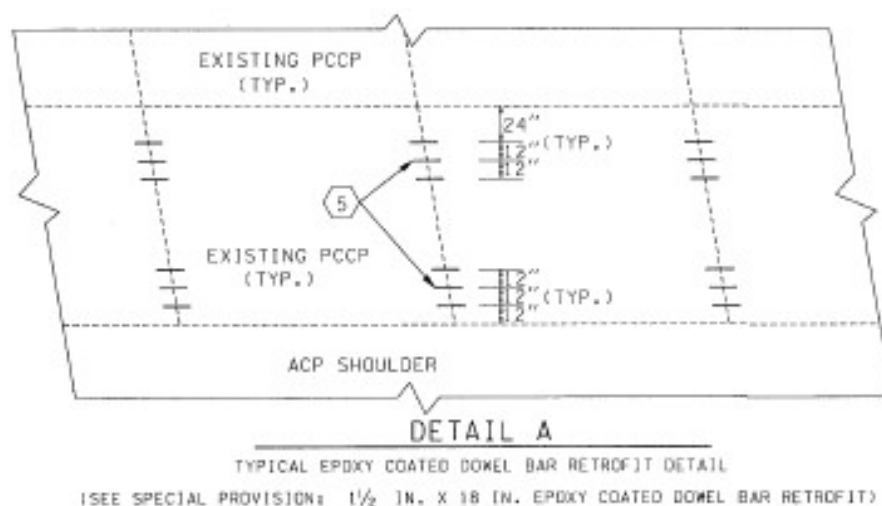
indicates rutting of 0.70 inches (18 mm) or more. PPC is based on pavement roughness as assessed by IRI using the following scale (WSDOT 2010):

- Very Good: ≤ 95 inches/mi (1499 mm/km).
- Good: $>95 - 170$ inches/mi (1499 – 2683 mm/km).
- Fair: $>170 - 220$ inches/mi (2683 mm/km – 3472 mm/km).
- Poor: $>220 - 320$ inches/mi (3472 – 5051 mm/km) (*considered unacceptable*).
- Very Poor: >320 inches/mi (5051 mm/km) (*considered unacceptable*).

Typically, a rehabilitation treatment is triggered once the PRC or PSC reaches a value of 50. The PPC is considered as a “lagging” indicator, meaning that roughness is the result of distresses such as cracking and faulting, which are captured in the PRC and PSC indices. The first available PRC, PSC, and PPC (IRI) data for this section of I-5 are from 1990, or 21 years after the pavement was originally constructed.

The first major CPR was performed in the outer (truck) lane only, first in the northbound direction in 1996 and then in the southbound direction in 1997. The combined CPR projects consisted of installing edge drains, retrofitting 21,937 dowel bars at transverse joints and cracks, performing approximately 25 slab replacements, cross-stitching of cracks, spall repair, diamond grinding 91,065 yd² (76,142 m²) of outer lane pavement in each direction, and resealing joints and random cracks. A rapid-setting concrete was used that contained 4.5 to 6.5 percent air and exhibited a strength of 3000 lb/in² (20.7 MPa) at 4 hours. The 1996 project used calcium sulfoaluminate cement manufactured by CTS Cement Manufacturing Corp for slab repairs while the 1997 project used a Type I & II cement from Lafarge Canada, Inc. and a non-chloride accelerator.

The dowel bar retrofit plan called for the placement of 18-inch (457 mm) long, 1.5-inch (38-mm) diameter epoxy-coated smooth dowel bars. The tie bars used in the cross stitching were required to be #5 deformed bars. Construction details for the dowel bar retrofit and cross-stitching are shown in figures 12-8 and 12-9, respectively.



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Figure 12-8. Dowel bar retrofit details.

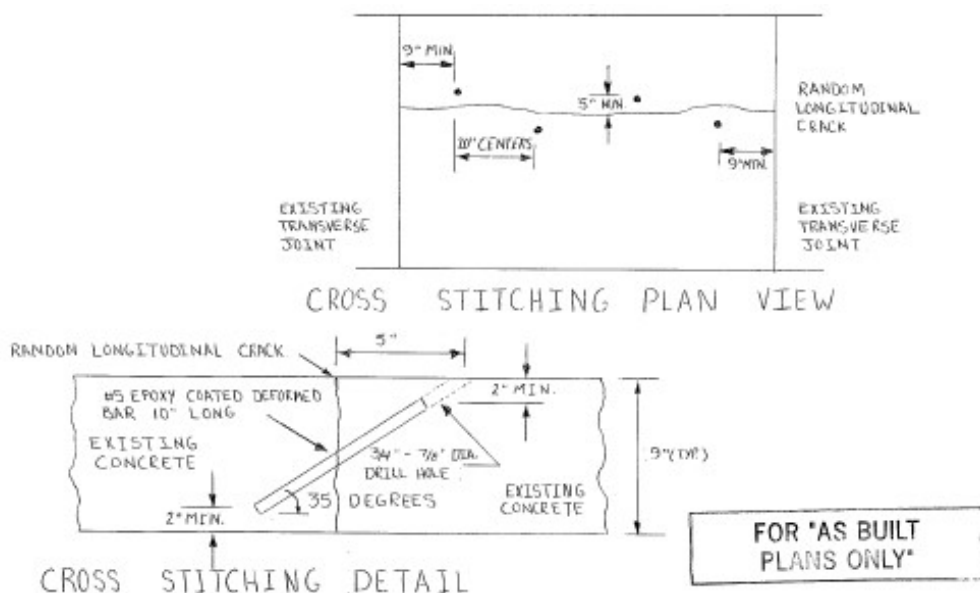
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Figure 12-9. Cross-stitching details for longitudinal cracks.

The next major CPR occurred in 2010 and consisted of retrofitting 13,051 dowel bars and grinding of 51,826 yd² (43,333 m²) of center lane pavement (both directions), along with 109 slab replacements in all lanes. The 2008 WSDOT specifications for panel replacements required a concrete mix with a minimum of 5.5 percent air and a minimum compressive strength of 3,000 lb/in² (20.7 MPa) at 3 hours and 5,000 lb/in² (34.5 MPa) at 24 hours. Transverse repair joints were to be doweled with 1.5-inch (38-mm) diameter, 18-inch (458-mm) epoxy-coated dowel bars. Test reports listed a portland cement factor of 795 lb/yd³ (472 kg/m³) with use of air entraining and water reducing admixtures. This mixture achieved 14-day compressive strengths of approximately 10,000 lb/in² (68.9 MPa).

State maintenance crews also performed minor spall repairs in 2014 (quantities not available).

Figure 12-10 shows time-series IRI and PRC data for the outer lane of the project from 1990 until 2017. After the first major rehabilitation in 1996 and 1997, the IRI dropped from a value of about 160 inches/mi (2525 mm/km) to an IRI value of approximately 60 inches/mi (947 mm/km). After the 2010 CPR, the IRI dropped slightly (approximately 16 inches/mi [253 mm/km]), likely due to the slab replacements that were performed in the outer lane. According to WSDOT, the reduction in IRI after 2010 may be due to data collection problems encountered with new equipment acquired by the State in 2008. Regardless, the data demonstrates that the CPR conducted in 1996/1997 was highly effective in not only dramatically improving ride quality, but in sustaining it for the next 20 years.

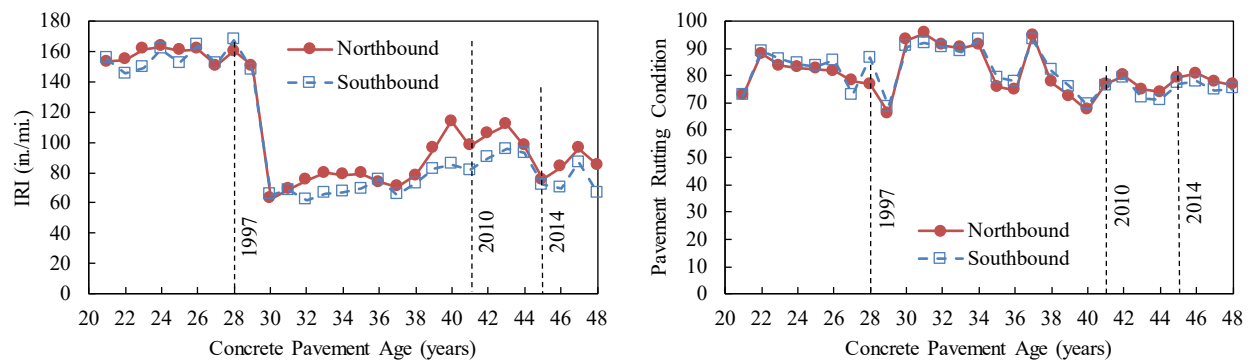


Figure 12-10. Time-series performance data (created from WSDOT data).

Economic Analysis

Table 12-2 summarizes the agency costs for the initial construction and subsequent pavement rehabilitation activities that have been performed on the stretch of I-5 documented in this case study. Details on the approach and procedures used for the economic analysis are available in Appendix A.

Table 12-2. Activity and cost data for the I-5 project.

Year	Activity	Activity Detail	Cost (from DOT Contract)*
1969	JPCP Construction	8.5-in PCC Paving	\$2,402,584
1996	CPR	Northbound: installation of subdrain, dowel bar retrofit, slab replacements, cross-stitching cracks, partial-depth repair, diamond grind, and reseal joints (lane 1: right)	\$909,354
1997	CPR	Southbound: installation of subdrain, dowel bar retrofit, slab replacements, cross-stitching cracks, partial-depth repair, diamond grind, and reseal joints (lane 1: right)	\$871,968
2010	CPR	Dowel bar retrofit, slab replacements, partial-depth repair, diamond grind, and reseal joints (NB and SB)	\$1,888,895
2025	End-of-Life	Assumed Pavement End of Service	NA

*Costs shown in then-current dollars

A 50-year analysis period was used (1969-2019) and the impacts of four different discount rates: (0 percent, 2 percent, 3.5 percent, and 5 percent) were examined. The remaining service life value was determined using a straight-line depreciation.

The nominal dollars were converted to constant base-year dollars using the National Highway Construction Cost Index (NHCCI) as explained in Appendix A. Table 12-3 shows the construction costs converted to 1969 dollars.

Table 12-3. Agency costs for I-5 converted to 1969 dollars.

Year	Activity	1969 Dollars
1969	JPCP Construction	\$2,402,584
1996	CPR	\$234,661
1997	CPR	\$207,095
2010	CPR	\$275,511
2019	Remaining Value (1969 Original Construction)	(\$257,420)
2019	Remaining Value (2010 CPR)	(\$110,204)

Figure 12-11 shows the expenditure stream diagram for the I-5 project using the four different discount rates used in the analysis and table 12-4 summarizes the LCCA results.

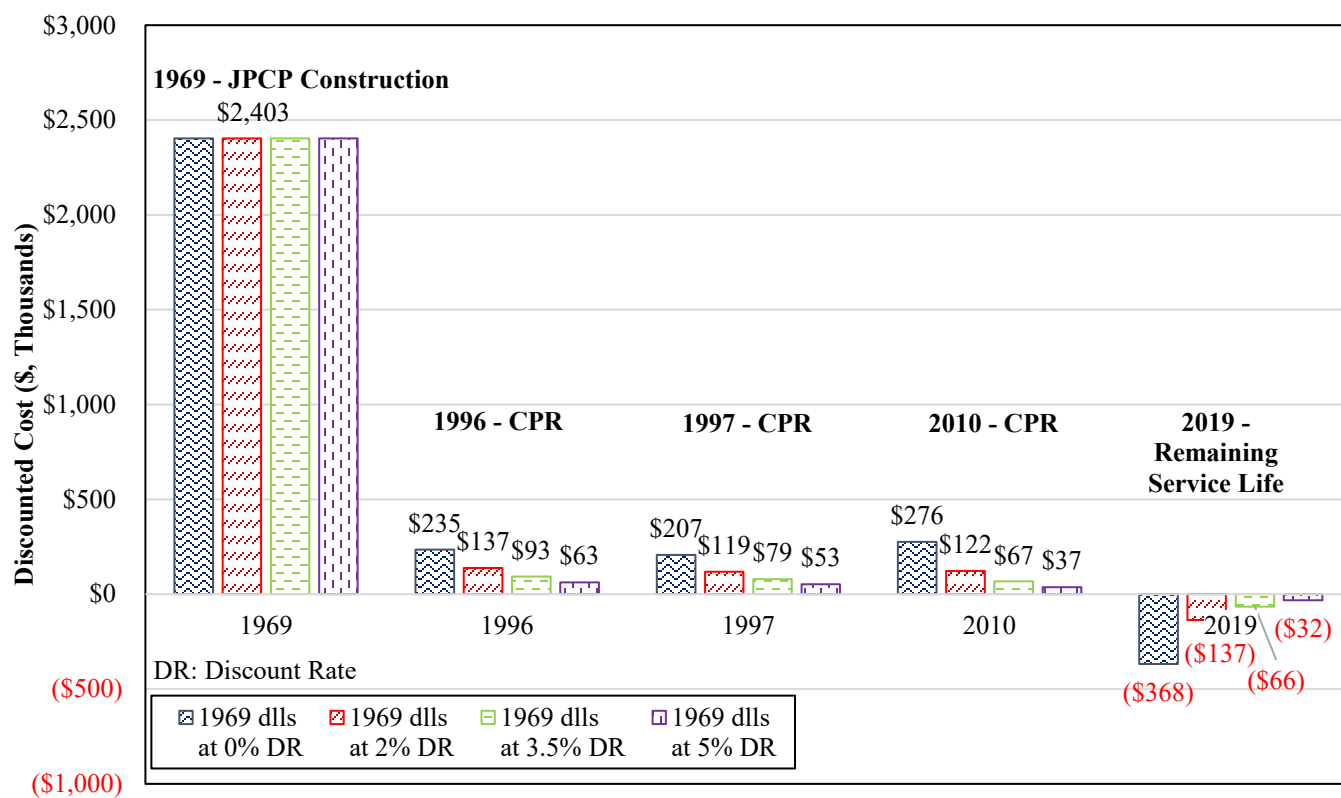
**Figure 12-11. Expenditure stream diagram for I-5.**

Table 12-4. LCCA summary for I-5.

0% Discount Rate			
LCCA Element	1969 Dollars		2019 Dollars
1969 Original Construction	PV =	\$2,402,584	PV = \$20,118,167
1996 CPR (DBR,DG)	PV =	\$234,661	PV = \$1,964,944
1997 CPR (DBR,DG)	PV =	\$207,095	PV = \$1,734,119
2010 CPR (DBR,DG)	PV =	\$275,511	PV = \$2,307,006
Remaining Value (1969 PCC)	PV =	(\$257,420)	PV = (\$2,155,518)
Remaining Value (2010 CPR)	PV =	(\$110,204)	PV = (\$922,802)
Totals	NPV =	\$2,752,226	NPV = \$23,045,916
	EUAC =	\$55,045	EUAC = \$460,918
2% Discount Rate			
LCCA Element	1969 Dollars		2019 Dollars
1969 Original Construction	PV =	\$2,402,584	PV = \$20,118,167
1996 CPR (DBR,DG)	PV =	\$137,479	PV = \$1,151,186
1997 CPR (DBR,DG)	PV =	\$118,950	PV = \$996,034
2010 CPR (DBR,DG)	PV =	\$122,330	PV = \$1,024,334
Remaining Value (1969 PCC)	PV =	(\$95,639)	PV = (\$800,835)
Remaining Value (2010 CPR)	PV =	(\$40,944)	PV = (\$342,847)
Totals	NPV =	\$2,644,760	NPV = \$22,146,039
	EUAC =	\$84,165	EUAC = \$704,758
3.5% Discount Rate			
LCCA Element	1969 Dollars		2019 Dollars
1969 Original Construction	PV =	\$2,402,584	PV = \$20,118,167
1996 CPR (DBR,DG)	PV =	\$92,694	PV = \$776,177
1997 CPR (DBR,DG)	PV =	\$79,039	PV = \$661,834
2010 CPR (DBR,DG)	PV =	\$67,233	PV = \$562,982
Remaining Value (1969 PCC)	PV =	(\$46,092)	PV = (\$385,953)
Remaining Value (2010 CPR)	PV =	(\$19,732)	PV = (\$165,231)
Totals	NPV =	\$2,575,725	NPV = \$21,567,976
	EUAC =	\$109,813	EUAC = \$919,523
5% Discount Rate			
LCCA Element	1969 Dollars		2019 Dollars
1969 Original Construction	PV =	\$2,402,584	PV = \$20,118,167
1996 CPR (DBR,DG)	PV =	\$62,853	PV = \$526,307
1997 CPR (DBR,DG)	PV =	\$52,829	PV = \$442,363
2010 CPR (DBR,DG)	PV =	\$37,272	PV = \$312,095
Remaining Value (1969 PCC)	PV =	(\$22,448)	PV = (\$187,969)
Remaining Value (2010 CPR)	PV =	(\$9,610)	PV = (\$80,472)
Totals	NPV =	\$2,523,479	NPV = \$21,130,491
	EUAC =	\$138,228	EUAC = \$1,157,459
PV: Present Value NPV: Net Present Value EUAC: Equivalent Uniform Annual Cost			

As the discount rate is increased from 0 percent to 5 percent, the net present value (NPV) decreases by 8 percent and the equivalent uniform annual cost (EUAC) increases by 151 percent. This is illustrated in figure 12-12.

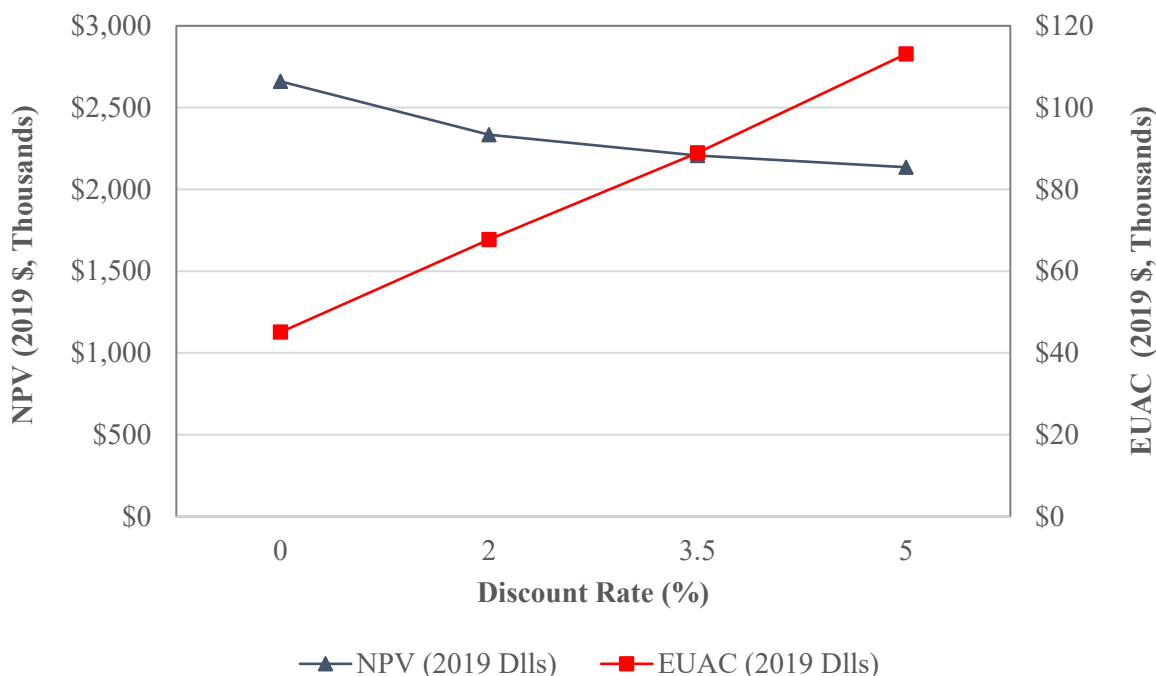


Figure 12-12. NPV and EUAC (2019 dollars) vs discount rate.

Lessons Learned

The JPCP segment of I-5 east of Olympia is an excellent example of a long-life concrete pavement that has been kept in serviceable condition for almost 50 years through the timely application of CPR treatments, including retrofitting of subdrainage, retrofitting dowel bars, cross-stitching, slab replacement, and diamond grinding. Based on the LCCA conducted, future maintenance costs are estimated to be between 6 and 22 percent of the original construction cost. The lessons learned from this pavement section include:

- **A good stable base contributed to the long service life.** The 0.25- or 0.35-ft thick asphalt-treated base provided good support for the pavement, even though the most heavily trafficked lane was undoweled and without subdrainage for nearly 30 years until it underwent CPR in 1996/1997. The base continues to provide good service to this day, as the ride quality has consistently remained high for the 20 years following the 1996/1997 CPR.
- **A durable concrete mixture contributed to the long service life.** The concrete mixture has demonstrated good durability over 5 decades of service under heavy-loading and exposure to studded tires.

- **CPR treatments effectively maintained a high level of serviceability.** Treatments including retrofitting subdrains, dowel bar retrofitting, cross-stitching longitudinal cracks, intermittent slab replacement, diamond grinding, and joint resealing worked together to effectively restore the overall serviceability of the pavement and maintain it for an extended period.
- **Retrofitting dowel bars and diamond grinding are key for long service life.** The original concrete pavement was undoweled and exhibited fair ride-quality (i.e., IRI of approximately 160 inches/mi [2525 mm/km]) after about 30 years in service. Since retrofitting dowel bars and diamond grinding, the ride quality has remained in the very good category with an IRI of approximately 80 inches/mi (1263 mm/km) after 20 years of additional service.
- **Rutting from studded tires remains a concern.** Surface wear as the result of studded tires was observed to be severe in some of the slab replacements reinforcing the idea that concrete used for repairs should have similar or better wear resistance and durability (based on location and/or service conditions) than the concrete being replaced.

CHAPTER 13. SUMMARY AND LESSONS LEARNED

Summary

Three broad preservation strategies, working in concert, can be used to help ensure the longevity of concrete pavements. The first is to design and construct concrete pavements for long-life, which involves the selection of volumetrically stable and durable materials, the selection of design features that reduce load- and environmentally induced stresses, and the construction of a high-quality pavement. The design and construction of long-life concrete pavements do not involve any special or exceptional measures but instead rely upon applying common techniques with diligence and attentiveness. For long-life concrete pavements, the slab thickness is typically increased slightly to enhance structural load-carrying capacity and to accommodate future grinding. Greater attention is paid to the supporting layers, ensuring that they provide uniform support, are non-erodible, and facilitate good drainage. Other design features are implemented to reduce stress, including the use of doweled joints at transverse joints, relatively short joint spacing, and widened slabs or tied shoulders to provide edge support. Concrete mixtures are proportioned to be durable, should exhibit good volume stability under temperature and moisture variations, and possess reduced permeability.

The second strategy involves the use of concrete overlays as a way to extend pavement life. With regard to unbonded concrete overlays of concrete, most have been used to restore structural capacity to highly distressed concrete pavements, but there is no reason that they could not be used as a preservation strategy on a concrete pavement that is currently in good condition. The use of thin bonded concrete overlays of concrete have also demonstrated good success in correcting surface or structural deficiencies on many projects but there have also been some notable failures when applied to pavements in deteriorated condition or when effective bond was not achieved between the overlay and the underlying concrete pavement.

The third strategy involves the application of CPR techniques to maintain a high level of serviceability. CPR can be the most economical solution for long-term concrete pavement preservation if the existing pavement is in sound structural condition and free of materials-related distresses (MRD), such as D-cracking or ASR. If this is the case, non-invasive CPR preservation strategies can often be employed with minimal disruption to traffic to maintain pavement functionality for 50 years or more from the time of original construction.

Lessons Learned from the Long-Life Concrete Pavement Design Project

Lessons learned from the I-35W LLCP project are summarized below.

- **Constructing an LLCP involves thicker slabs and more stringent specifications for the concrete mixture and dowel bars.** Some of the key mixture design considerations included: lower permeability, higher air content, and a lower water-to-cement ratio. Increased emphasis was placed on aggregate quality (specific gravity, absorption, and alkali-silica reactivity potential) and aggregate gradation. Ground granulated blast furnace slag (used at a 35 percent replacement for portland cement) was employed to densify the concrete microstructure and reduce permeability. Instead of the commonly used epoxy-coated steel dowels, the project employed solid stainless steel and stainless steel-clad dowels for the mainline pavement.

- **LLCP significantly reduces future maintenance needs.** The construction of an HPCP on I-35W enabled MnDOT to offer acceptable levels of serviceability and smoothness, as measured by distress surveys and IRI measurements, without having to invest in maintenance projects for 19 years. A simple economic analysis showed that over a 60-year period, the future maintenance costs are expected to be less than 15 percent of the initial construction costs.

Lessons Learned from Concrete Overlay Projects

Lessons learned from the concrete overlay projects documented in this report are summarized below.

- **Unbonded concrete overlays can provide excellent service and long life, even when constructed over badly distressed pavement.** As seen from the Michigan, North Carolina, and Illinois UBOL projects, it is not difficult to imagine that 40 years or more of service life can be attained through the use of UBOLs in conjunction with the use of timely and effective pavement preservation techniques. The structural capacity of the overlay is expected to be similar to that of new concrete pavement construction and the cost of constructing an overlay is expected to be lower than the cost of complete removal and reconstruction.
- **A good drainage system is essential to a well-performing UBOL.** On Michigan U.S. 23 overlay project, some isolated areas of slab cracking and pumping were linked to poor drainage.
- **Use of a separator layer improves UBOL performance.** The use of an asphalt concrete separator layer (≥ 25 mm thickness) prior to the construction of the UBOL appears to be an effective method to minimize reflective cracking.
- **If properly designed and constructed, unbonded concrete overlays of existing asphalt pavements can provide long service lives.** After 28 years of service, only a small fraction of the slabs (5 percent; 33 out of the total 612 slabs) in the experimental section on U.S. 1 exhibited some form of cracking in 2015.
- **Bonded concrete overlay (with pre-overlay repairs) can successfully and greatly extend the life of the original pavement, providing excellent ride quality with minimal maintenance.** Bonded concrete overlays of concrete pavement offer potential for tremendous increases in pavement structural capacity for relatively little cost. The U.S. 281 project in Texas demonstrated that significant extensions in concrete pavement life can be achieved using bonded concrete overlays at relatively low unit costs. The 24-hour rolling, lane-at-a-time closure approach used on this project also demonstrated that concrete overlays can be constructed quickly and with relatively little impact to the traveling public.
- **Loss of bond can result in rapid deterioration of a bonded concrete overlay.** As observed in a few small, localized areas of the U.S. 281 project in Texas, if bond between the overlay and original pavement is lost, the overlay can deteriorate quickly.

Lessons Learned from CPR Projects

Lessons learned from the CPR projects documented in this report are summarized below.

- **A good foundation contributes to long service life.** The long-lived sections of the concrete pavement in Michigan (U.S.-41) were constructed on 0.5 to 2.1 m of stamp sands, a highly angular manufactured fine aggregate that is a residual of crushing metal-rich ore in a stamp plant. The thickness and nature of this material provides an excellent level of support through an extremely wet area. An identical 274 mm concrete section constructed at the same time was supported on 457 mm of unclassified pit material. It was removed and replaced with an asphalt pavement in 2000 due to poor performance.

The pavement thickness was only 200 mm for the Colorado I-70 project, which is relatively thin for interstate highway pavement. That thickness was likely significantly decreased by the diamond grinding in 2005. Further, the original joints were undoweled and the pavement was undrained. However, the pavement was constructed on a lean concrete base and aggregate subbase, all on top of alluvial subgrade material that is likely to have provided relatively uniform support.

The 75-mm bituminous-treated base used on the I-10 project in California provided good support for the pavement, even though the most heavily trafficked lanes largely remain undoweled (every 4th joint had tie bolts). The base continues to provide good service to this day.

The thick asphalt-treated base used on the I-5 project in Washington provided good support for the pavement, even though the most heavily trafficked lane was undoweled and without subdrainage for nearly 30 years until it underwent CPR in 1996/1997. The base continues to provide good service to this day, as the ride quality has consistently remained high for the 20 years following the 1996/1997 CPR.

- **Durable concrete mix design is critical for long-life.** A concrete mixture that has low cementitious materials content, high-quality aggregates, and a good air-void system is expected to be highly durable. In the U.S.-41 project, the concrete mixture had a relatively low cementitious content of 307 kg/m³ and featured a 51-mm maximum aggregate size from a gravel source and 5.5 percent entrained air. This mixture would be expected to have low shrinkage and good aggregate interlock across intermediate cracks that formed in the JRCF and has demonstrated excellent durability over nearly six decades of service in harsh conditions and heavy traffic loading. The concrete mixture used on the I-5 project in Washington has demonstrated good durability over five decades of service under heavy-loading and exposure to studded tires.
- **Proper and timely CPR activities can extend the service life of concrete pavement significantly at fraction of the original construction cost.** By restoring the load transfer capacity of the concrete pavement and correcting the surface roughness, adequate levels of serviceability can be achieved and maintained after 50 or more years of service. A simple economic analysis showed that by spending approximately 20 to 35 percent of the original construction costs for future maintenance and rehabilitation activities, the service life of the original concrete pavement can be extended by 30 years or more.

- **Retrofitting dowel bars and diamond grinding are key for long service life.** By restoring the load transfer capacity of the concrete pavement and correcting the surface roughness, adequate levels of serviceability can be achieved and maintained after 50 years of service as seen from the U.S. 20 project in Iowa and the I-70 project in Colorado.
- **Diamond Grinding is a very effective way to maintain pavement smoothness.** For the Colorado I-70 project, blanket diamond grinding in 2005 has contributed to pavement serviceability, and repair grinding in 2014 has helped to maintain good IRI levels. After 70+ years of service and through four major diamond grinding projects on the I-10 section in California, the pavement has been maintained with good ride quality proving the efficacy of this method.
- **HMA overlay on undoweled JPCP is not a long-term solution.** An asphalt overlay over JPCP can improve the serviceability and pavement surface characteristics (friction and IRI). However, in the long run, reflective cracking can become a major distress affecting the rideability of the pavement as evident from the westbound lanes of the U.S. 20 section in Iowa.

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APPENDIX A. ECONOMIC ANALYSIS APPROACH AND PROCEDURES

Economic Analysis

Introduction

Using the actual (and in some cases estimated/assumed) historical construction and rehabilitation cost data, a life-cycle cost analysis (LCCA) was performed to document the costs incurred over the service life of each project documented in this report. Much of the cost information was obtained from original contract documents supplied by each state highway agency featured in this report. For some projects, the American Concrete Pavement Association (ACPA) helped the project team in extracting the cost data extracted from the Oman database (obtained through ACPA). In cases where no cost information was available, assumptions were made based on information provided by the state highway agency and/or expert judgement of the project team. For most of the case study projects featured in this report, some localized patching and other maintenance activities have apparently been performed by maintenance forces, but these costs are not included as they commonly have a very small impact on the LCCA results.

Life-Cycle Cost Analysis

Traditional LCCA methods described by Walls and Smith (1998) were adopted for the economic analysis conducted for each project featured in this report. There are, however, some unique aspects to be considered when performing an LCCA for which actual treatment timings and costs are known (as opposed to most traditional LCCA when forecasts of treatment timings and costs are made). Since the costs shown in the expenditure stream diagram are actual, they are expressed in terms of nominal (then-current) dollars and not in terms of constant (real) dollars (as is typically done in LCCA in which future costs are forecasted). User costs were not included in the analyses. The LCCA process involved the following steps:

Step 0: Establish LCCA parameters

- Establish Analysis Period: This number varied from project to project as documented for each case study.
- Discount rate: The impacts of four differed real discount rates were examined: 0 percent, 2 percent, 3.5 percent, and 5 percent.
- Remaining Service Life Value: This was determined using a straight-line depreciation based on estimated activity service life guidance from the American Concrete Pavement Association (ACPA 2012, see table A-1).

**Table A-1. Expected service lives of various preservation treatments
(reported by ACPA 2012).**

Treatment	Expected Service Life (years)
Unbonded concrete over concrete	15 – 40
Bonded concrete over concrete	15 – 30
CPR without overlay	5 – 15
Diamond grinding of concrete surface	8 – 20

The procedure used for calculating the remaining value is illustrated in figure A-1.

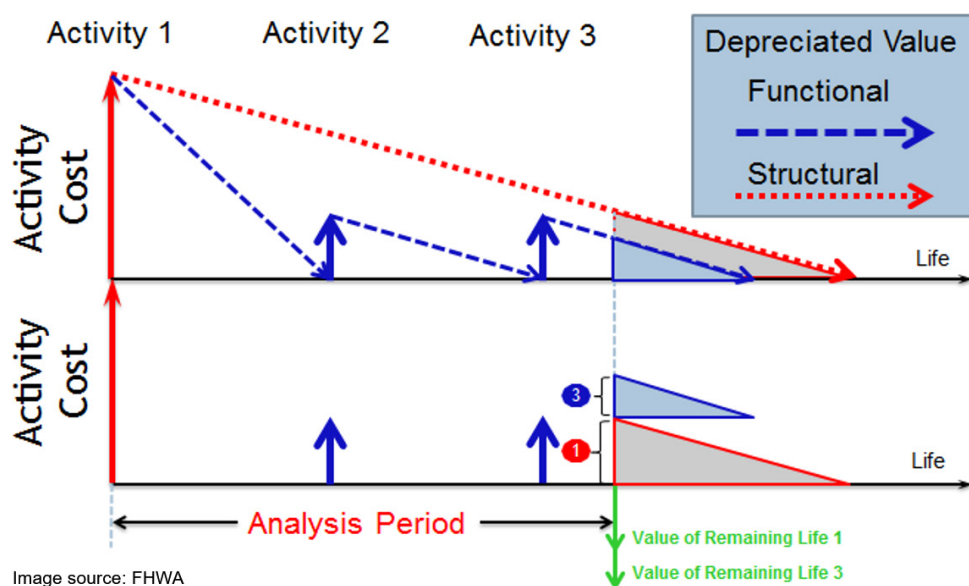


Figure A-1. Remaining service life value calculation.

Figure A-1 shows two activities (Activities #2 and #3) being performed after the initial activity (Activity #1) at year 0. Activity #1 has a structural life that extends beyond the analysis period and Activity #3 has a functional life that extends beyond the analysis period. The depreciated values of Activities #1 and #3 beyond the analysis period represents the remaining service life value.

Step 1: Summarize Activity Timings and Cost

Activity timings and agency costs were obtained from the DOT contract documents. The contract costs typically included activities beyond just the costs for paving materials and construction activities. Costs also included items such as traffic control, pavement marking, and other on-site activities related to the construction. The costs outside project limits were estimated and excluded from the LCCA. Also, any shoulder work that did not relate to supporting activities on the mainline pavement were excluded from the LCCA.

Step 2: Convert Nominal Costs to Base Year Dollars

The nominal (then-current) dollars were converted to constant dollars using the [National Highway Construction Cost Index \(NHCCI\)](#), an index used to track price changes associated with highway construction costs, and to convert current dollar expenditures on highway construction to real or constant dollars. NHCCI was developed to replace the Bid Price Index (BPI) and NHCCI data are available from 2003. NHCCI values prior to 2003 were estimated using the year-by-year growth rate of the Bid Price Index (BPI), which was published by FHWA from 1922 to 2006. An example illustrating how the NHCCI is used is presented below.

Table A-2. Example illustrating use of NHCCI to convert costs to base-year dollars.

Year	Activity	Nominal Cost	NHCCI	Cost in Base Year dollars (1968\$)	Discounted Costs (1968\$)			
					0% Discount Rate	2% Discount Rate	3.5% Discount Rate	5% Discount Rate
1968	JPCP Construction	\$584,290	0.1921	\$584,290	\$584,290	\$584,290	\$584,290	\$584,290
1991	Drainage Maintenance	\$772,986	0.7275	\$204,127	\$204,127	\$129,448	\$92,528	\$66,458

Since the 1968 JPCP construction activity is already expressed in the base-year dollars (1968\$), the discount rates do not have any impact. To convert the 1991 drainage maintenance cost to 1969 dollars, the following approach was used:

$$\text{Drainage Cost (1968 dollars)} = \frac{0.1921}{0.7275} \times \$772,986 = \$204,127$$

After converting the cost to the base-year dollars, the discounted cost (for each discount rate used in the analysis) was calculated using the following equation:

$$\text{Discounted Cost} = \text{Future Cost} \times \frac{1}{(1 + \text{Discount Rate})^n}$$

Where, n = number of years in the future where the cost will be incurred

For the 1991 drainage maintenance activity, using a 2 percent discount rate, the discounted cost in 1968 dollars is calculated as follows:

$$\text{Discounted Cost @ 2 percent discount rate} = \$204,127 \times \frac{1}{(1+0.02)^{23}} = \$129,448$$

Step 3: Compute Life-Cycle Cost

Life-cycle costs were calculated and expressed using two metrics: (a) net-present value (NPV), and (b) equivalent uniform annual cost (EUAC). The NHCCI (described above) was used to convert all LCCA results to 2019 dollars in order to have a common reporting unit for all the case study projects.

NPV is a calculation that converts all future costs into base-year dollars. NPV is calculated using the following equation:

$$NPV = \text{Initial Cost} + \sum \text{Future Cost} \times \left(\frac{1}{(1+r)^n} \right)$$

where:

r = discount rate

n = number of years in the future where the cost will be incurred

EUAC is a calculation that converts initial and future costs and benefits into annualized present value dollars over the life cycle EUAC is calculated using the following equation:

$$EUAC = NPV \left[\frac{r(1+r)^N}{(1+r)^N - 1} \right]$$

where:

r = discount rate

N = analysis period