

EVALUATION OF THE IMPACT OF CONCRETE PAVEMENT RESTORATION TECHNIQUES ON PAVEMENT PERFORMANCE

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ABSTRACT

Concrete Pavement Restoration (CPR) techniques were applied on a 30 mile stretch of I-65 between SR-114 and US-231 at Crown Point, in Indiana, in 1985. At the time of rehabilitation, some typical functional and structural distresses were developed. The distresses included corner, diagonal, longitudinal and transverse cracking and faulted, spalled and broken joints. The deteriorated surface resulted in a rough ride and a low skid resistance with potential slippery problems. Three types of full depth patches (dowelled, inverted T, and inverted T with a middle dowelled joint) were constructed to replace sections of severe pavement breakups. Partial depth patches were placed to repair minor surface distresses. Cracks were routed and sealed, and joints were refaced to inhibit intrusion of water. Diamond grinding was employed to restore the smooth riding quality and to increase the skid resistance, as well as to remove faulting at joints and cracks.

This study evaluated the impact of the CPR techniques on pavement condition and performance. The three types of full depth patches were still functioning well after five years in service. However, the dowelled full depth patch had lower load transfer coefficients than the inverted T full depth patches. The middle contraction joint installed on inverted T patches did not function as desired, and therefore its use should be discontinued. The partial depth patches performed poorly and required continuous maintenance throughout the study period.

Diamond grinding greatly reduced the roughness and moderately increased the wet skid resistance of the concrete pavement. The quick reduction of the wet skid resistance after rehabilitation limited the service life to 5 to 6 years. The concrete pavement continued to deteriorate after rehabilitation and the amount of concrete defects have been increasing at a significant rate. However, most of the defects were in the low severity category and the pavement was still structurally sound by the end of this study.

A life cycle cost analysis indicated that CPR was more cost effective than an asphalt overlay for this project even though CPR gained a service life that is shorter than the typical service life of an asphalt overlay. This study indicated that the cost of CPR and the service life of a pavement after rehabilitation are determined not only by the success of CPR techniques, but also by the condition and remaining service life of the original pavement at the time of rehabilitation. Therefore, it is strongly recommended that adequate condition evaluation and cost analysis be performed before rehabilitation to determine the feasibility of CPR.

1 INTRODUCTION

This study encompasses a comprehensive concrete pavement restoration (CPR) of a jointed reinforced concrete pavement on I-65 between SR-114 and US-231 at Crown Point, Indiana. This 30 mile stretch of jointed reinforced concrete pavement on I-65 was rehabilitated utilizing a number of CPR techniques in the summer of 1985 under Contract R-15306. The original concrete pavement was opened to traffic between

1966 and 1968 under six different contracts. It consisted of a 10 inch thick reinforced concrete slab over an 8 inch thick subbase. Slab dimensions were 40 feet in length and 12 feet in width. At the time of rehabilitation, some typical functional and structural distresses were developed. The distresses included corner, diagonal, longitudinal and transverse cracking and faulted, spalled and broken joints. The deteriorated surface resulted in a rough ride with an average roughness number of 1214 and a corresponding Predicted Serviceability Index (PSI) of 3.0. Friction values of the pavement surface also decreased considerably to a low level with an average friction number of 30.

To restore this pavement, various CPR techniques, such as undersealing, full and partial depth patching, diamond grinding, sealing cracks and joints, and edge drain treatment, were utilized. The performance of the rehabilitated pavement was monitored and evaluated for a period of five years, and a life cycle cost analysis was performed to compare the cost effectiveness of CPR to that of a conventional asphalt overlay. This paper presents the study results of the impact of various CPR techniques on pavement condition and performance, and the life cycle cost analysis. Although undersealing and the installation of edge drains were part of the effort to rehabilitate the pavement on this contract, the evaluation of these two features was not included in the scope of this study.

2 CONSTRUCTION PROCEDURES

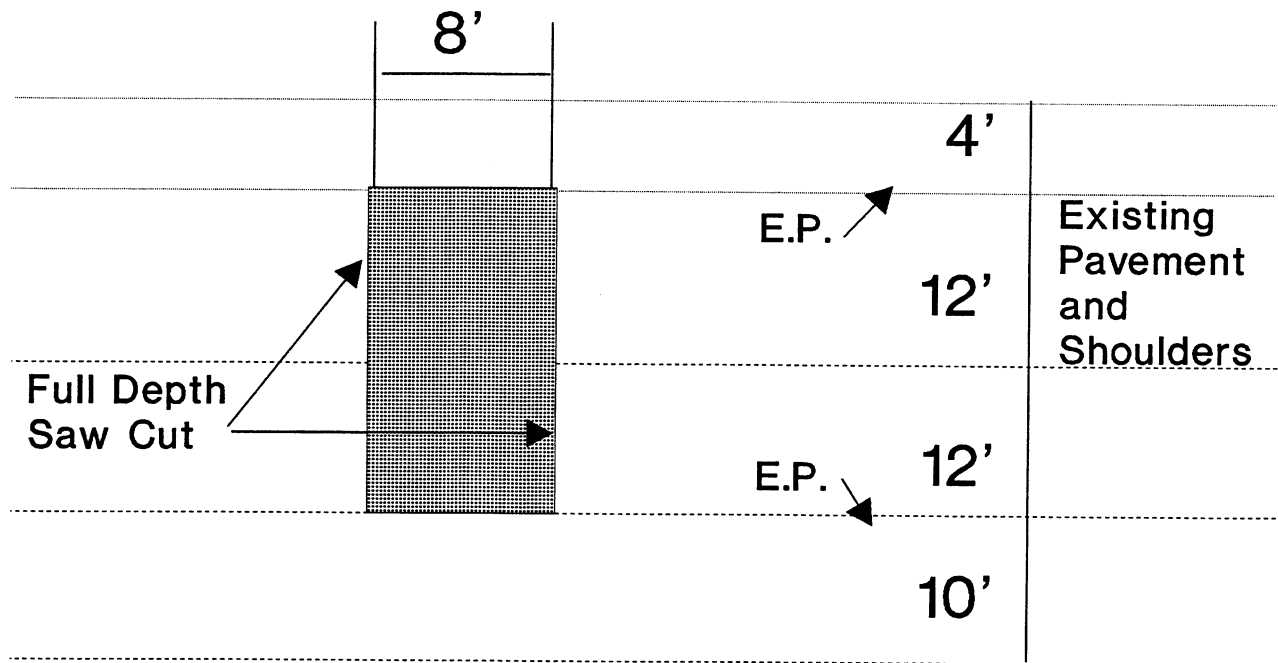
Concrete slabs were first undersealed in areas showing high deflections with oxidized asphalt to improve the strength and uniformity of the subbase support to concrete slabs. The undersealing locations were determined by Dynaflect measurements. Oxidized asphalt was pumped in these locations resulting in uniform pavement support and well seated slabs.

Full depth patches were used to replace sections of severe pavement breakups at either midslab or joint areas. Three types of full depth patches as shown in Figures 1, 2 and 3 were employed:

1. Inverted T patch (midslab repairs on the northern 28 miles);
2. Dowelled patch (midslab repairs on the southern 2 miles);
3. Inverted T patch with a dowelled contraction joint (at all replaced joints).

The inverted T patch was constructed by cutting up and lifting broken old slab pieces and removing the existing subbase beneath the deteriorated slabs and 6 inches underneath the edge of the adjacent slabs, and then replacing the excavated area with new concrete. No dowel bars were used in the inverted T patch. The dowelled patch did not require removal of the subbase as with the inverted T patch. Six 1.25" diameter dowel bars were used on each side of a dowelled full depth patch for load transfer between the patch and its adjacent existing concrete slabs. Inverted T patches with dowel basket assemblies in the center were used to replace damaged joints. This type of patch was constructed by adding the contraction joint to the inverted T patch. No problems were identified during construction of these three types of full depth patches.

Plan View



Cross Section

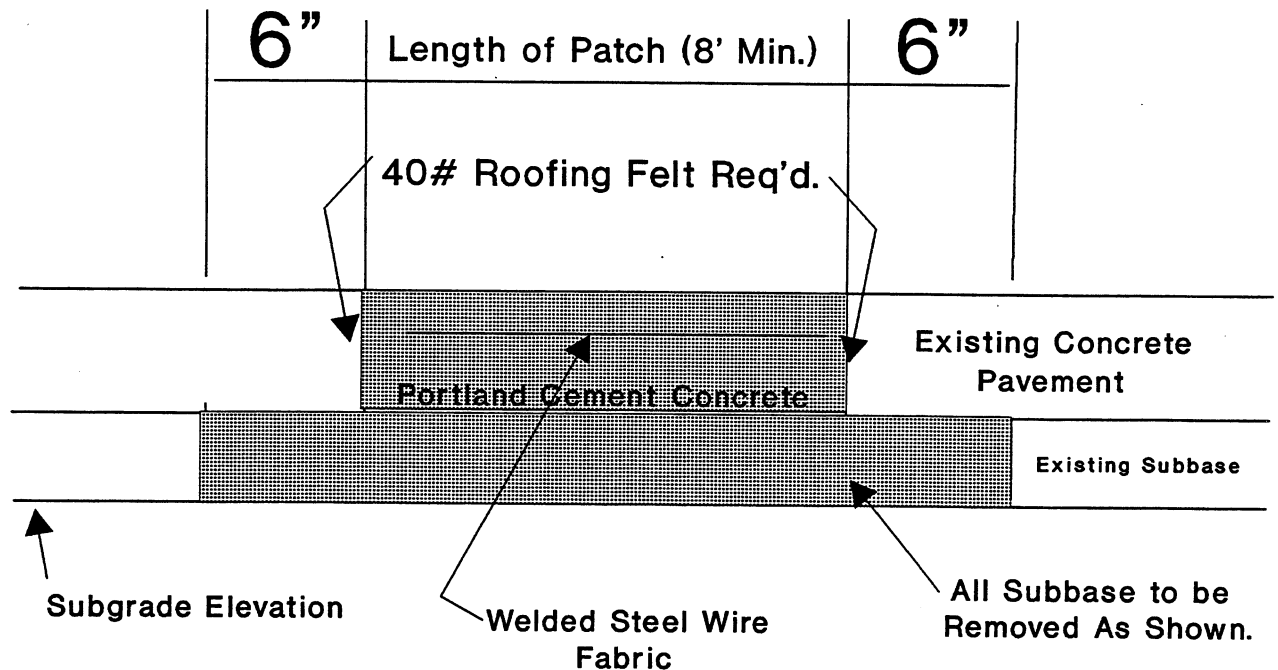
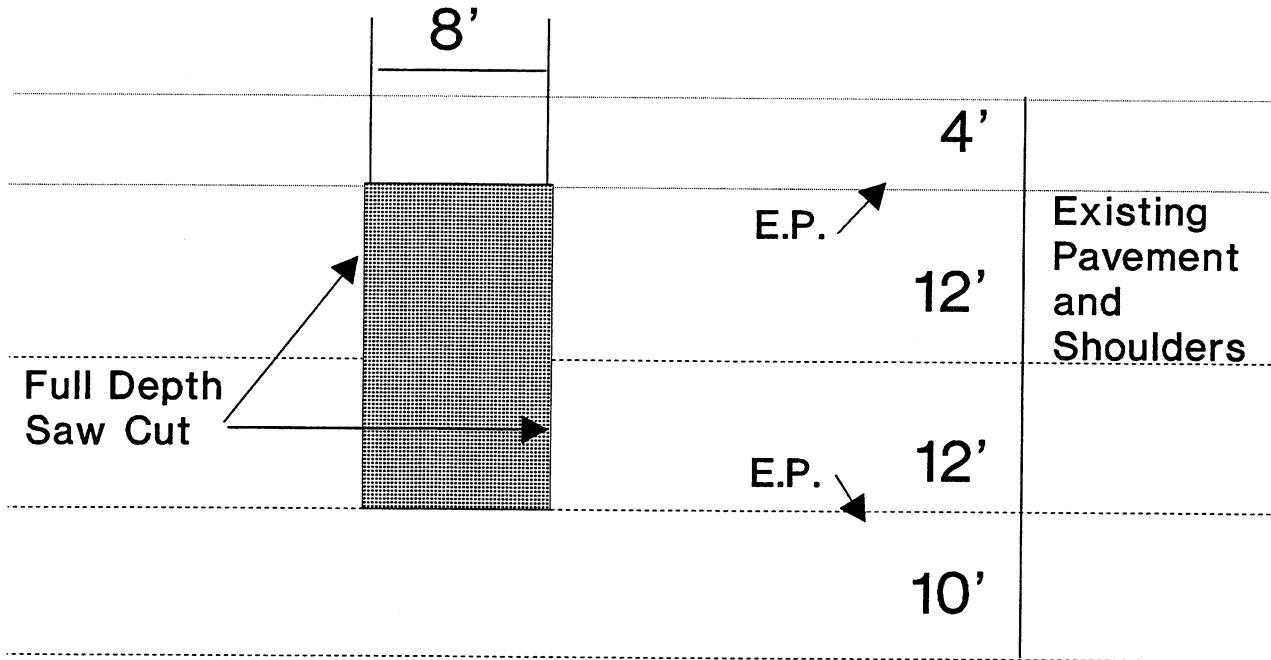
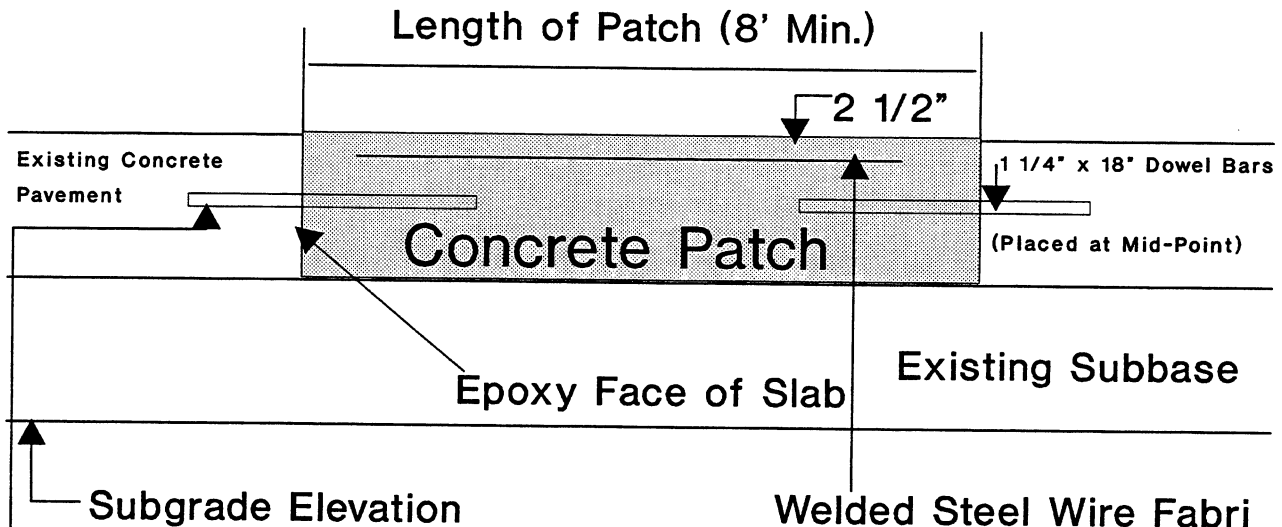


Figure 1: Inverted T Full Depth Patch

Plan View



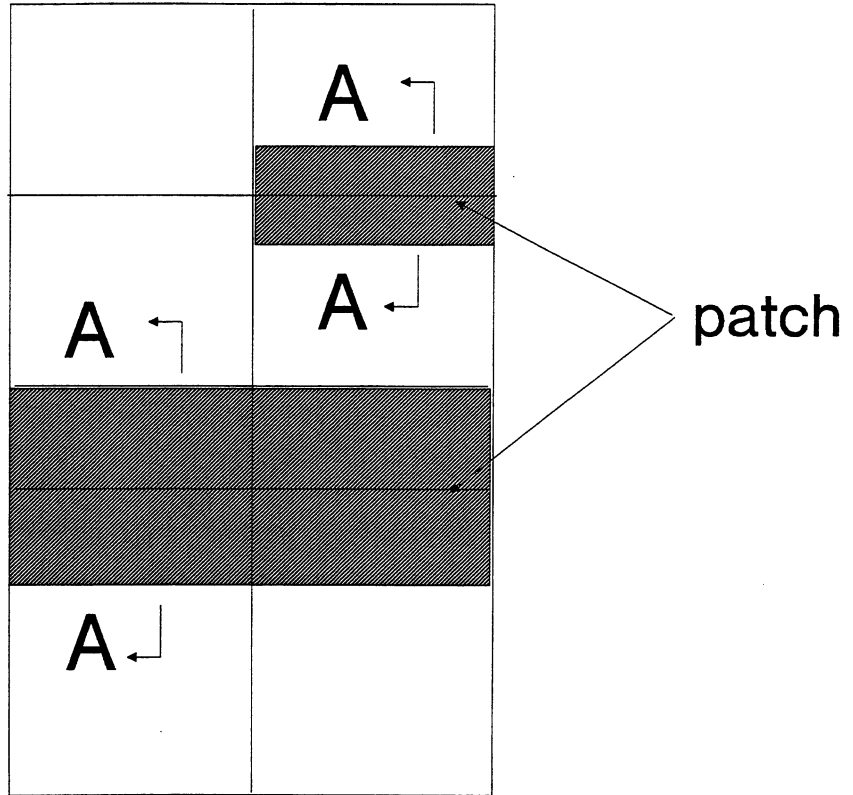
Cross Section



Dowel bars are spaced 1, 2, 3, 8, 9, and 10 feet from the outside pavement edge.

Figure 2: Dowelled Full Depth Patch

Plan View



Section AA

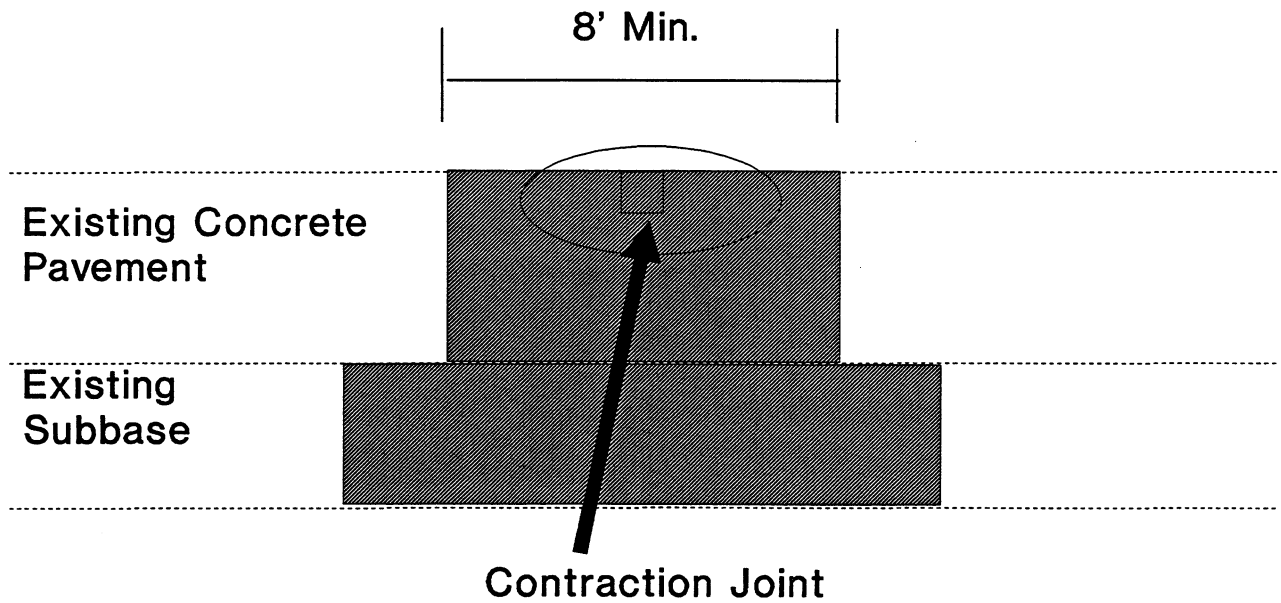


Figure 3: Inverted T Full Depth Patch with a Contraction Joint

Partial depth patches were used to repair shallow defects. No problems were encountered during construction of the partial depth patches.

After all the patches were completed, the slabs were smoothed using the diamond grinding machine. The grinding head was 3 feet wide and required four passes for each 12 feet lane. This machine had no vertical control, so in many spots the ground surfaces of adjacent passes did not line up. Consequently, longitudinal misalignments as great as 1/8 inch were produced. To keep misalignments between adjacent passes from occurring, lateral control to adjacent ground should be established. This problem could also be minimized by using wider grinding heads requiring fewer passes.

Finally, cracks were routed and sealed, joints were refaced to inhibit intrusion of water, and open graded base edge drains were installed between the edge of pavement and the existing trench drains at locations needing improved drainage.

3 EVALUATION OF CPR TECHNIQUES

In order to evaluate the CPR techniques applied in this project, necessary pavement tests and inspections were conducted annually for five years, in addition to the data collection immediately before and after rehabilitation. The condition improvement by CPR and the performance of the rehabilitated pavement were analyzed based on the data of immediately before and after rehabilitation tests and the data of annual tests between 1985 and 1990.

3.1 Condition Improvement by CPR

3.1.1 Visual Inspection

Neither faulting nor spalling were observed at the joints after rehabilitation. Many of the routed and sealed cracks showed smaller crescent shaped cracks around. These smaller cracks existed at the time of rehabilitation and were not repaired by CPR. The visual inspection showed that after rehabilitation, the pavement was visually in a much "smoother and better" condition.

3.1.2 Improvement in Roughness

The entire section of the project was tested for roughness with a PCA roadmeter before and after rehabilitation in 1985. The following are the average roughness numbers (RN) for the northbound (NB) and southbound (SB) travelling lanes and the corresponding predicted serviceability indices (PSI):

Test	NB	SB	Avg. RN	PSI
Before CPR (1985)	1155	1272	1214	3.0
After CPR (1985)	215	247	231	5.0

PSI values were obtained based on the regression equation [1]:

$$PSI = 11.73 - 2.83364 \log(RN).$$

PSI are used to represent the level of service, which range from 0.0 to 5.0 with 5.0 being the best PSI value. A PSI of 2.5 or less is considered unsatisfactory on interstate pavements, which corresponds to a RN of 1800 for concrete pavement. As indicated by the before and after values of RN and PSI, the CPR was very successful in improving riding comfort and level of service. It should be noted that the PSI value was increased to 5.0, which corresponds to the level of service of a new pavement.

3.1.3 Improvement in Skid Resistance

The standard towed trailer with ribbed, locked wheel torque measuring friction testing system, (ASTM E-274), was used to measure pavement wet skid resistance in terms of friction number. The friction numbers of before and after rehabilitation for the northbound and southbound travelling lanes are given below for comparison:

Test	NB	SB	Avg. FN
Before CPR (1985)	31	29	30
After CPR (1985)	40	44	42

A pavement with a friction number of 30 or less is considered to be in a condition with potential slippery problems. Therefore, before rehabilitation the pavement was at such a condition that the occurrence of vehicle skidding was a concern. The CPR operation, or diamond grinding, adequately improved the skid resistance of the pavement to a relatively safe condition from the condition with potential slippery problems.

3.1.4 Effect on Pavement Strength

The following are the average pavement deflections (in 10^{-5} inch) at midslabs measured by the Dynaflect for travelling lanes before and after rehabilitation:

Test	Sensor No. 1			Sensor No. 5		
	NB	SB	Avg	NB	SB	Avg
Before CPR (1985)	28	31	30	18	18	18
After CPR (1985)	30	34	32	20	20	20

Sensor No. 1 is located between the two loading wheels of the Dynaflect, and sensors 2, 3, 4 and 5 are spaced at one foot increments from sensor No. 1 with sensor No. 5 being the farthest (4 feet) from sensor No. 1. In practice, sensor No. 1 is used as an indicator of slab strength and sensor No. 5 as an indicator of base and subbase support strength. Readings of 50×10^{-5} inch or less by sensor No. 1 and 30×10^{-5} or less by sensor No. 5 are indications of adequate concrete slab strength and good pavement support, respectively [2]. The data show that the pavement was still structurally sound before rehabilitation. The deflection values for both sensor No. 1 and sensor No. 5 increased slightly after rehabilitation, that is, the strength of the pavement structure has been weakened by the CPR operation. This is attributed to the fact that all the CPR techniques used were only to repair the functional failures, except that undersealing helped strengthen the subbase support at only the undersealed spots. In fact the grinding process may have reduced the slab strength slightly by removal of a thin layer of concrete. However, since the differences between the deflection values before and after CPR were not significant, the grinding process did not practically cause any problem of pavement strength. It is therefore believed that for pavements with adequate strength the effect of the diamond grinding operation on pavement strength should not be a concern.

3.2 Pavement Performance Evaluation

In order to evaluate the pavement performance after CPR, ten samples were randomly selected for each of the following six features: dowelled full depth patches, inverted T full depth patches, inverted T full depth patches with dowel basket assemblies in the center, midslabs, original joints, and midslab cracks. These sixty samples were tested annually for deflections with Dynaflect. Visual inspections of cracking and faulting of the same features were made at the times of deflection tests. The roughness and skid resistance tests as well as visual inspections for the entire project were also conducted annually after rehabilitation. Based on the six year data, the pavement performance is evaluated as follows.

3.2.1 Deterioration of Pavement Condition

Before rehabilitation in 1985, 10.7% of southbound slabs and 6.5% of northbound slabs had defects. A visual inspection conducted in August, 1990 showed that the percentages of defective slabs have increased to 42.5% and 36.7% respectively in the period of five years after rehabilitation. The major types of distress were transverse cracks (50.37 each/mile) and corner breaks (23.49 each/mile). Some blowups (2.21 each/mile) were also found during the inspection in 1990. However, most of these distresses were reported to be in the category of "low severity". A few of them were identified as "medium severity" distresses. Hence, immediate repairs for these distresses may not be required.

The inspection results indicated that during the five year period after CPR, the concrete pavement condition has deteriorated at a significant rate. It is apparent that CPR techniques did not prevent the pavement from continuing deterioration. This is because the CPR techniques repaired only the severely defected slabs by patching, but the majority of the concrete

slabs were not strengthened. As a result of heavy truck traffic (20,000-24,000 ADT with 30-40% trucks) and growing pavement age, the trend of pavement deterioration continued. These low severity distresses may eventually become high severity as the pavement gets old. Therefore, the service life of this roadway after rehabilitation is determined not only by the success of CPR techniques, but also by the condition and remaining service life of the original pavement at the time of rehabilitation.

3.2.2 Evaluation of Diamond Grinding

As discussed earlier, the grinding operation improved the pavement roughness significantly, and improved the skid resistance moderately. The combined average roughness numbers of travelling lanes for the NB and SB directions before employing CPR (1985) and after employing CPR (1985 through 1990) are shown below.

Year	NB	SB	Avg. RN	PSI
1985 (Before CPR)	1155	1272	1214	3.0
1985 (after CPR)	215	247	231	5.0
1986	401	299	400	4.4
1987	426	408	417	4.3
1988	510	498	504	4.1
1989	576	636	606	3.9
1990	604	560	582	3.9

PSI values are also included to show the level of service for each year of the five year evaluation period. The roughness number decreased between 1989 and 1990, probably due to routine maintenance or minor repair during the period. As indicated by the RN and PSI values, CPR was very successful in improving riding comfort and level of service. The PSI value of 3.9 in 1990 indicates that the improvement has been sustained for a period of 5 years and the level of service is still satisfactorily high.

The average friction numbers of travelling lanes in each year for the evaluation period were also recorded:

Year	NB	SB	Avg. FN
1985 (Before CPR)	31	29	30
1985 (After CPR)	40	44	42
1986	49	45	47
1987	40	42	41
1988	34	34	34
1989	34	33	34
1990	33	33	33

The reason for the increase of friction number in 1986 might be that in the first year after rehabilitation the concrete texture of the ground pavement was being worn away by traffic. As can be seen, after 5 years the average friction numbers have been decreased to 33 from 42. Since friction numbers less than 30 are considered unacceptable, the friction number of 33 indicates that the pavement is very near the terminal friction life. Figure 4 shows the friction numbers of different sections of the pavement in 1990 along the travelling lanes.

It indicates that on several sections the friction numbers were already below 30, or the coefficients of wet pavement friction were below 0.30. That is, 5 years after rehabilitation, some portions of the pavement were once again in a potentially slippery condition, and an immediate repair is recommended to restore the pavement skid resistance on these sections.

To estimate the future average friction number, a linear regression analysis [3] was performed and the following relationship between the average friction number FN and time T (in years) was obtained with a coefficient of correlation $r^2 = 0.7325$:

$$FN = 45.0 - 2.6 \times T.$$

It was predicted that the average FN will reach 30 when T equals 6. That is, if nothing is done now, then by the end of 1991 as much as half of the 30 mile pavement will have a friction number less than 30 and require an effort to improve the skid resistance. It should be noted that this

prediction was based on the assumption of linear relationship which may not be true in reality. Nevertheless, since most of the pavement sections had friction numbers close to 30, it can be anticipated that the friction numbers of many more sections of the pavement will drop below 30 in the very near future.

3.2.3 Pavement Strength and Concrete Condition

The following are the average deflection values under sensor No. 1 (S1) and sensor No. 5 (S5) of travelling lanes in the beginning and end of the evaluation period:

Time	Midslab		Joint		Crack	
	S1	S5	S1	S5	S1	S5
1985 (Before CPR)	30	18	39	20	39	19
1985 (After CPR)	32	20	40	20	39	20
1990	39	23	48	21	50	23

The travelling lanes are considered to be more critical because they carry heavy truck traffic more frequently. After five years in service, the average deflection values increased for both sensor No. 1 and sensor No. 5 readings. That is, both the slab strength and subbase support have been weakened due primarily to the cumulative effect of heavy traffic repetitions on the pavement during the study period. Since the average deflection value at midslabs was below 50×10^{-5} under sensor No. 1, and below 30×10^{-5} under sensor No. 5, the concrete slabs and subbase were still structurally sound.

One of the major causes of concrete pavement deterioration in northern region of Indiana is the effect of freezing and thawing. Pop-outs and D-cracking are the two widespread forms of freeze-thaw damage in concrete pavement [4]. To evaluate the concrete condition, the pavement was examined for these two types of deterioration. No pop-outs and D-cracking were observed on the pavement surface of this 30 mile project during the last visual inspection. However, since D-cracking usually starts in the lower parts of concrete slabs adjacent to joints, where moisture accumulates, and eventually progresses through the entire slab, no D-cracking on the surface does not necessarily mean that the bottom of the concrete slab was also free of freeze-thaw damage. To inspect the concrete condition, eight 4" diameter cylinder concrete cores were taken in 1990 from the pavement at randomly selected locations adjacent to joints and cracks. The cores showed no sign of D-cracking or any other damages.

To obtain further information on the concrete condition, compressive strength tests were performed on the concrete cores. The compressive strength values range from 5790 psi to 7290 psi with an average value of 6627 psi. The INDOT Standard Specifications require that a new portland cement concrete pavement shall have a modulus of rupture of at least 550 psi before it is open to traffic. Although a minimum compressive strength is not specified, the obtained compressive strengths can be used to estimate the modulus of rupture of the pavement. In general, the ratio of flexural to compressive strength ranges from 0.11 to 0.23 [4]. Using ratio of 0.11, conservative values of flexural strengths can be estimated. The estimated flexural strengths range from 637 psi to 802 psi, with an average value of 729 psi. Therefore, even based on the conservative estimation, the flexural strength of the pavement was still considerably higher than the required strength of 550 psi.

The results of the Dynaflect test, core inspection and compressive strength test indicated that the pavement was structurally sound. However, the visual inspection showed that the pavement condition had deteriorated at a significant rate. This is because concrete distresses are caused not only by load, but also by such effects as thermal and moisture gradient stresses, drying shrinkage stresses, etc. Load related distresses occur when either the concrete stress exceeds the ultimate strength or the number of repeated applications of loads exceeds the fatigue limit. Since the fatigue strength of concrete is only about 50% of the static strength [4], the concrete pavement can be damaged by load repetition in combination with other effects even though the concrete strength is adequate for a single load. Diamond grinding improved ride comfort and skid resistance, and pavement patching repaired the severely damaged spots, however, CPR

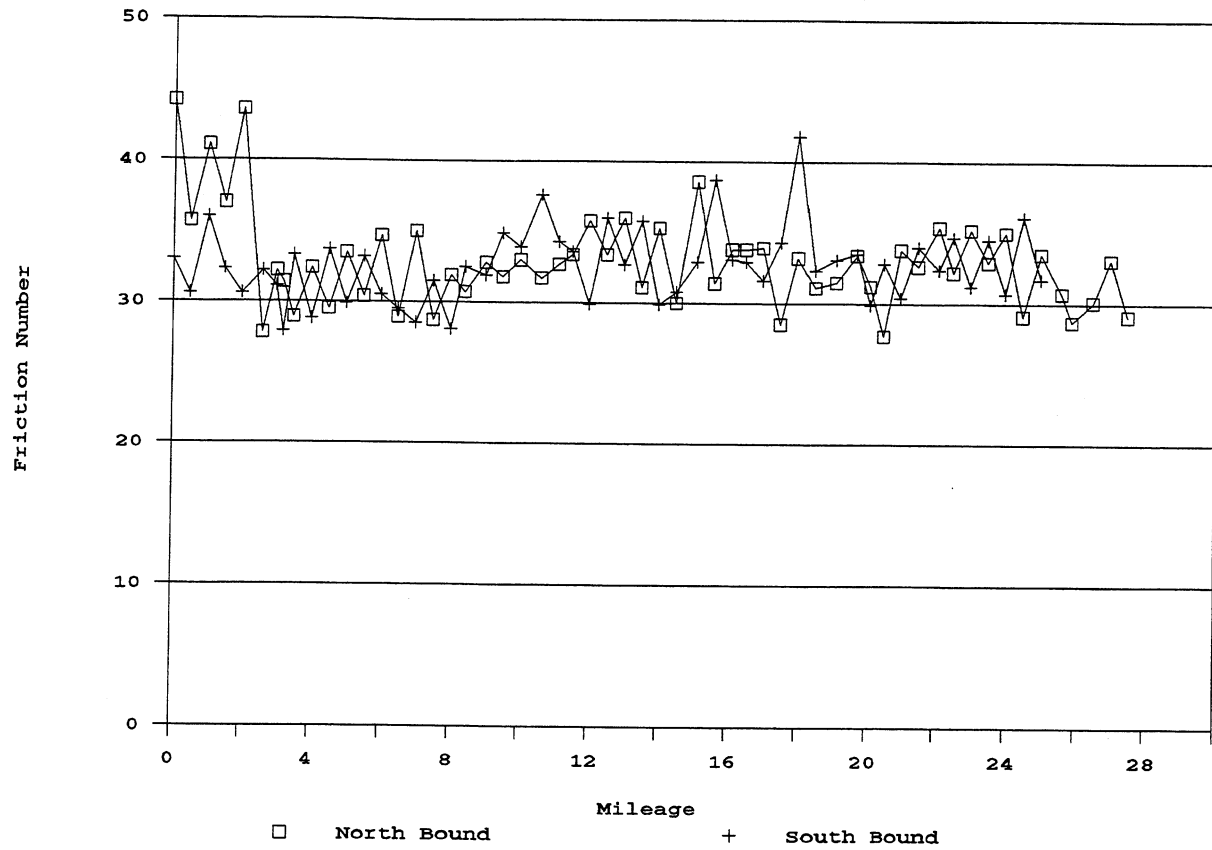


Figure 4: Friction Numbers in 1990

techniques did not provide any additional strength to the original pavement. Consequently, the fatigue-related pavement deterioration was not alleviated by CPR but continued and even accelerated as the pavement became old.

3.2.4 Full Depth Patches, Refaced Joints and Routed Cracks

No cracks, breakups and other defects were found on the full depth patches after 5 years in service. Slight faults (1/16 to 1/8 inch) were observed one year after rehabilitation at the last joints of full depth patches and at some of the routed and resealed cracks. These faults, however, remained almost the same by the end of the 5 year period. Some of the crescent hairline cracks around routed and resealed cracks eventually spalled out under the effect of heavy traffic.

Adequate load transfer at the transverse joints and cracks is very critical to a concrete pavement. The load transfer coefficient (LTC) was used to evaluate the load transfer efficiency of full depth patches and resealed cracks and routed cracks. The load transfer coefficient is defined as the ratio of the Dynaflect reading of sensor No. 2 to that of sensor No. 1, while the deflection is measured by placing sensor No. 1 at a point 0.5 foot before a joint or crack and sensor No. 2 at a point 0.5 foot after the joint or crack. The load transfer coefficients range from 0.0 to 1.0. If the load transfer is good, both readings should be nearly the same, and the load transfer coefficient, Sensor No.2 / Sensor No. 1, should be near 1.0. If the load transfer is poor, the reading of sensor No. 2 on the unloaded side would be small compared to the reading of sensor No. 1 on the loaded side. Load transfer coefficients of 0.75 or above are considered as good load transfer, values of 0.5 to 0.75 as fair load transfer, and below 0.5 as poor load transfer. The load transfer coefficients for the 60 selected features measured in 1990 are given below.

Feature	Mean	Std. Dev.	# Tests
Midslab	0.93	0.017	10
Joint	0.84	0.118	10
Crack	0.85	0.184	10
1st Joint of Dowelled Patch	0.65	0.295	10
2nd Joint of Dowelled Patch	0.72	0.209	10
1st Joint of Inverted T Patch	0.88	0.060	10
2nd Joint of Inverted T Patch	0.92	0.114	10
1st Joint of Joint Replacement Inverted T Patch	0.85	0.037	10
2nd Joint of Joint Replacement Inverted T Patch	0.94	0.025	10
3rd Joint of Joint Replacement Inverted T Patch	0.92	0.069	10

The resealed dowelled joints and routed cracks had the almost same load transfer coefficients (0.84 and 0.85), but the values for cracks had greater variability and ranged from 0.382 to 0.981 (0.617 to 0.971 for joints). In general, the resealed joints and routed cracks had good load transfer and were still in satisfactory conditions.

Both types of inverted T patches (with or without center contraction joint) had better load transfer than the dowelled patches. Apparently, the added concrete under the adjacent existing slab acts like a "sleeper slab" and improves load transfer between slab and patch. The load transfer coefficients at the last joint on each of the inverted T patches were relatively high even though the sawn face provided very little grain interlock at the joint. It was also noted that in the direction of traffic, the first joint for each of the three types of patches had lower load transfer coefficient than the last joint of these patches. Load transfer is achieved by mechanical devices, such as dowel bars and sleeper slabs, and/or grain interlock between adjacent slabs. However, the above observations may imply that some other factors may also affect the values of load transfer coefficients. In an attempt to explain the observations, it was found that load transfer was also affected by the strength of the pavement which the load was applied to. The values of deflections (sensor No. 1 reading) and load transfer coefficients for each type of patches are given below:

Joint	Dowelled Patch	
	Avg. LTC	Avg. Deflection
First	0.65	51
Last	0.72	43
Joint	Inverted T Patch	
	Avg. LTC	Avg. Deflection
First	0.88	34
Last	0.92	27
Joint	Joint Replacement Inverted T Patch	
	Avg. LTC	Avg. Deflection
First	0.85	33
Last	0.92	28

As can be seen, for each type of the patches, the last joint had a higher LTC but a lower deflection value than the first joint. At the last joint, the load was applied to the patch rather than the original pavement as at the first joint. Since lower deflection values correspond to higher pavement strength, the data not only indicate that the patches had better strength than the original pavement, but also imply that the higher strength of patches helped improve load transfer at the last joint.

To study the effect of pavement strength on load transfer, the deflection values were divided into four levels for the purpose of statistical analysis. The following data show that as deflection value increases, the load transfer coefficient decreases:

Level	Deflection Range	Avg. LTC
1	20-30	0.900
2	30-40	0.854
3	40-50	0.764
4	> 50	0.619

The SAS statistical procedure of General Linear Models [5] was used to study the effect of deflection levels on load transfer efficiency. This analysis showed that the effect of deflection levels was significant at $\alpha = 0.05$. Thus, it was concluded that the strength of pavement was a factor affecting load transfer efficiency.

Comparing the deflection values of inverted T and dowelled patches, it also can be seen that the inverted T had lower deflection and higher load transfer values at both the first and the last joints. The inverted T patch increased its strength by the thickened slab (the thickness of original slab plus that of subbase). The concrete filled in the undercut area below the adjacent slabs provided additional support to the existing slabs at both the first and the last joint, which may consequently increase the load transfer efficiency.

The low load transfer coefficients for dowelled patches were probably attributed to:

1. inadequate number of dowels (six) along the joint on this type of patch,
2. relatively lower strength of the concrete slab compared to thicker inverted T patch, and
3. disturbance to the subbase cause by patching construction.

Consequently, to improve the load transfer efficiency, more dowel bars should be used for this type of patch. When constructing the dowelled patch, the removal of existing slabs by sawing them and lifting them out would minimize the disturbance to the subbase, and therefore retain the strength of the pavement.

For the joint replacement inverted T patch, the middle contraction joint had load transfer coefficient of 0.94, which corresponds to solid concrete. This means that the middle joint did not perform as a dowelled joint as desired. Since the patches without a middle joint performed as well as those with a middle joint, the installation of middle joints on inverted T patches was not necessary. Use of dowelled contraction joints in inverted T patches should be discontinued to save the extra cost of installing the middle joint. A dowelled patch or a inverted T patch should be used instead to replace the joint.

3.2.5 Partial Depth Patches

Partial depth patches placed adjacent to cracks and joints were spalled out or broken up. Repairs were required just one year after rehabilitation. These distresses were frequently noted throughout the five year study period. This was probably caused by poor load transfer and differential slab movement across the crack and through the partial depth patch. All partial depth patches that were placed over a crack had cracked. Furthermore, many new midslab cracks might have been caused by partial depth patches. This could be attributed to the weakness and stress concentration caused by the discontinuity of the concrete slab in the partially patched area.

It is recommended to limit the use of partial patches as much as possible and replace them with full depth patches, or to accept the fact that they have to be rehabilitated on an annual basis.

4 COST ANALYSIS

The cost of this project was \$190,775/mile. Also in 1985, another section of I-65 in similar condition was resurfaced with 4.75" asphalt overlay at a cost of \$359,614/mile. If the two projects had the same service lives, the CPR would be the better alternative because its cost was only half as much as that of asphalt overlay. However, the service life of the CPR project was estimated as only 5 to 6 years, while 10 years of service life is a reasonable estimation for an asphalt overlay. A life cycle cost analysis was conducted based on an interest rate of 7% and the assumption of a 10 year service life for asphalt overlay and 6 year service life for CPR. The first alternative was to use CPR techniques first and then to resurface the pavement 6 years later. The other alternative was to resurface the pavement now and to resurface it again 10 years later. Since the service lives were different, the initial investments were converted to uniform annual costs [6] as follows.

Known: Service life of CPR = 6 years;
Service life of overlay = 10 years;
Cost of CPR = \$190,775/mile;
Cost of overlay = \$359,614/mile;
Interest rate = 0.07.

Uniform annual costs:

Option 1 — CPR-and-overlay:

$$A_1 = [190,775 + 359,614 \times \frac{1}{1+0.07} \times \frac{0.07(1+0.07)^{16}}{(1+0.07)^{16}-1}] \times \frac{0.07(1+0.07)^{16}}{(1+0.07)^{16}-1} = \$45,561$$

Option 2 — Overlay-and-overlay:

$$A_2 = [359,614 + 359,614 \times \frac{1}{1+0.07} \times \frac{0.07(1+0.07)^{20}}{(1+0.07)^{20}-1}] \times \frac{0.07(1+0.07)^{20}}{(1+0.07)^{20}-1} = \$51,198$$

The uniform annual cost was \$45,561 per interstate mile for option 1, and \$51,198 per interstate mile for option 2. Therefore, the first option (CPR-and-overlay) is more cost effective than the overlay-and-overlay option, and the saving is \$5,637 per interstate mile per year.

5 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations obtained from this study are summarized as follows:

5.1 Full Depth Patching

5.1.1 Performance

All three types of full depth patches functioned well after five years in service. Dowelled full depth patches had slight faulting at construction joints one year after construction. However, the faults did not further develop during the remaining four year period. The load transfer

coefficients of dowelled full depth patches were relatively lower than those of inverted T full depth patches.

The inverted T patches showed very good load transfer. Slight faulting was found one year after CPR. The slight faults also remained the same by the end of the five year period.

The dowel bar assembly in the inverted T patch did not function as a contraction joint and added 15% to the cost of the patch.

5.1.2 Recommendations

Dowelled Full Depth Patching: Six 1.25" diameter dowel bars were used on each side of a dowelled full depth patch for load transfer between the patch and its adjacent existing concrete slabs. However, an FHWA study [7] recommends to use ten 1 1/2" diameter dowel bars for the patching on heavy traffic roads, and the INDOT Standard Pavement Joints Sheet A requires ten 1 1/4" diameter dowel bars for load transfer. The relatively low load transfer coefficients of this type of patches in this project indicated that the number of dowel bars was not sufficient. Therefore, it is recommended that for dowelled full depth patching ten 1.25" diameter dowel bars be used as specified in Sheet A of the INDOT Standard Pavement Joints. Since it was found that the strength of the patches affected the load transfer, attempts should be made to increase the slab strength of dowelled patches. Where practical, the removal of existing slabs by sawing them and lifting them out would minimize the disturbance to the subbase when constructing the dowelled patch. The dowelled full depth patch with more dowel bars can be used as an acceptable alternative to the inverted T patch, if the existing slabs have adequate strength to retain the dowels without their spalling out.

Inverted T Full Depth Patching: The INDOT Standard Pavement Joints Sheet A specifies ten 1.25" diameter dowel bars for this type of patches. However, the inverted T full depth patches in this project did not have dowel bars and performed very well. Based on the results of this study, it is therefore recommended that the inverted T patching without dowel bars be applied on Indiana highways to save the extra labor and material costs of dowel bar installation. It should be emphasized that the concrete in the underpinned areas must be well consolidated to minimize faulting and improve load transfer efficiency.

Inverted T Patching with a Middle Dowelled Joint: Use of the dowel basket assembly in the inverted T patch at joints should be discontinued by INDOT. A dowelled patch or inverted T patch should be used in its place.

5.2 Partial Depth Patching

Partial depth patches performed poorly. They required frequent maintenance during the study period and have caused damage to the adjacent sound concrete. It is recommended to limit the use of partial depth patches as much as possible.

5.3 Diamond Grinding

5.3.1 Performance

Diamond grinding greatly reduced the roughness of the existing jointed concrete pavement from a RN around 1200 to about 200. It also increased the skid resistance from FN's around 30 to the low 40's. However, the average friction number decreased back to 33 and the friction numbers on several sections were already below the concern value of 30 after five years in service. The quick reduction of the skid resistance limited service life of the rehabilitated pavement to about six years. Grinding the pavement surface in 3 feet wide passes resulted in poor alignment between adjacent passes in some areas.

5.3.2 Recommendations

Diamond grinding can be used to reprofile jointed concrete pavements which have developed any of the following conditions [7]:

1. A rough ride due to faulting conditions.
2. Wheel ruts caused by studded tires.
3. Inadequate transverse slope for drainage.
4. Low skid resistance due to excessive polishing of the pavement surface.

For diamond grinding operations used to improve pavement ride comfort and/or skid resistance, it is recommended to set specifications based on resulting improvement in ride comfort (measured by roughness roadmeter) and skid resistance. Payment penalties should be enforced for failure to meet these specifications. Lateral control to adjacent ground surfaces should be established to keep misalignments between adjacent passes from occurring. This problem can be minimized by using wider grinding heads requiring fewer passes.

5.4 Cost Consideration

The life cycle cost analysis showed that use of CPR techniques to delay the first asphalt overlay was more cost effective than the overlay-and-overlay strategy. However, it should be pointed out that this is not a general conclusion, but is the result only for this particular project. The total cost of CPR largely depends on the condition of the existing pavement. The worse a pavement condition is, the higher is the CPR cost because it needs more patching and other remedial efforts and has less remaining service life. For pavements with certain severity of deterioration, the life cycle cost analysis may give results favoring overlay-and-overlay alternative. A previous study [8] has concluded that when 5% or more of the right lane required full depth replacement, the project was probably not a suitable CPR candidate. Therefore, it is recommended that adequate condition evaluation and cost analysis be performed before rehabilitation to determine the feasibility of CPR.

5.5 General Remarks

In general, most of the CPR techniques employed in this project were effective. The pavement is still structurally sound and has a fairly high level of ride comfort. On the other hand, continuous deterioration of the original concrete slabs and the low skid resistance may call for a new rehabilitation in the very near future. This study indicates that CPR techniques can provide as much improvement in pavement roughness and skid resistance

at less cost as an asphalt overlay can. However, comparing to asphalt overlay, pavement rehabilitated by CPR may not retain the improvement long and may need more maintenance efforts during service life since CPR techniques do not improve the pavement strength. The effectiveness of CPR techniques depends on the severity of the pavement deterioration at the time of rehabilitation. The cost of CPR will be higher and the condition deterioration will continue and even accelerate after CPR if the original pavement is already severely deteriorated. Therefore, it is critical to perform preliminary condition evaluation and life cycle cost analysis in determining a cost effective rehabilitation strategy.

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