Pavement Evaluation and Development of Maintenance and Rehabilitation Strategies for Illinois Tollway East-West Extension

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The procedures used in the field evaluation and development of maintenance and rehabilitation strategies for the Illinois Tollway Authority's 70-mi, four-lane East-West Extension are presented. The extension consists of 14-in. portland cement concrete (PCC) placed directly on the silty clay subgrade with random joint spacing varying between 12 and 17 ft. Although the pavement is only 17 years old, it is experiencing rapid decrease in serviceability. The field evaluation consisted of a visual condition survey, nondestructive deflection testing including void detection and transverse joint load transfer efficiencies, coring, soil evaluation, determination of slab and soil properties, and petrographic analysis of the PCC. After the pavement evaluation was completed, several maintenance and rehabilitation strategies were developed for different life expectancies varying from 2 to 20 years. All of the strategies were evaluated using EXPEAR (EXpert system for Pavement Evaluation And Rehabilitation).

The results of an extensive pavement engineering evaluation and rehabilitation analysis of the Illinois Tollway East-West Extension on I-88 are presented. The East-West Extension begins approximately 4 mi west of the Fox River near Aurora and extends in a westerly direction for 69 mi to its terminus 1 mi east of Rock Falls. It was constructed during the 1972, 1973, and 1974 construction seasons and opened for traffic in fall 1974.

On the basis of the structural design and the traffic level, the East-West Extension was expected to provide up to 30 years of satisfactory performance with only moderate rehabilitation. However, it is experiencing a rapid decrease in the level of service, and a rehabilitation program will need to be implemented soon.

Interesting and useful information, which will help reduce future portland cement concrete (PCC) pavement design failures, was gained from this study.

OBJECTIVE

The objective of this investigation was to develop a cost-effective pavement rehabilitation program. The objective was achieved through a comprehensive engineering evaluation that answered the following critical questions about the current condition of the East-West Extension pavement:

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- What are the extent and severity of the deterioration?
- What are the causes of the deterioration?
- Is the load-carrying capacity adequate for future operation or is structural improvement needed?
- What are the feasible rehabilitation alternatives?
- What are the expected life cycle costs of each feasible alternative?

BACKGROUND

The original design plans called for the typical pavement section to be built as a 10-in. concrete slab on a 4-in. cement-stabilized base on the subgrade. However, during construction, the typical section was modified to 14-in. concrete slab placed directly on the subgrade. The typical pavement sections consist of a nonreinforced concrete slab lying directly on the subgrade and randomly skewed joints with joint spacing ranging from 12 to 17 ft. The concrete shoulders are tied to the mainline slabs and decrease in thickness from 8 to 6 in. as they extend out from the traffic lanes. Portions of the East-West Extension were paved as one wide monolithic slab including the shoulders, whereas other portions were paved with the traffic lanes separate from the shoulders. The centerline joints and the longitudinal joints in the monolithic sections were formed by using a polyethylene tape embedded in the concrete.

DATA COLLECTION

The data collection efforts started with a visual survey of the pavement surface. After the survey, the cause of deterioration was not clearly defined, and it was determined that a complete PCC evaluation program must be implemented to clearly define the problem. The program included traffic information, nondestructive deflection testing (NDT) using the falling weight deflectometer (FWD), pavement coring and soil boring, and laboratory testing of the soil and PCC.

Traffic Information

The current average daily traffic (ADT) is 10,000 and the average daily truck traffic (ADTT) is 1,125 in two directions

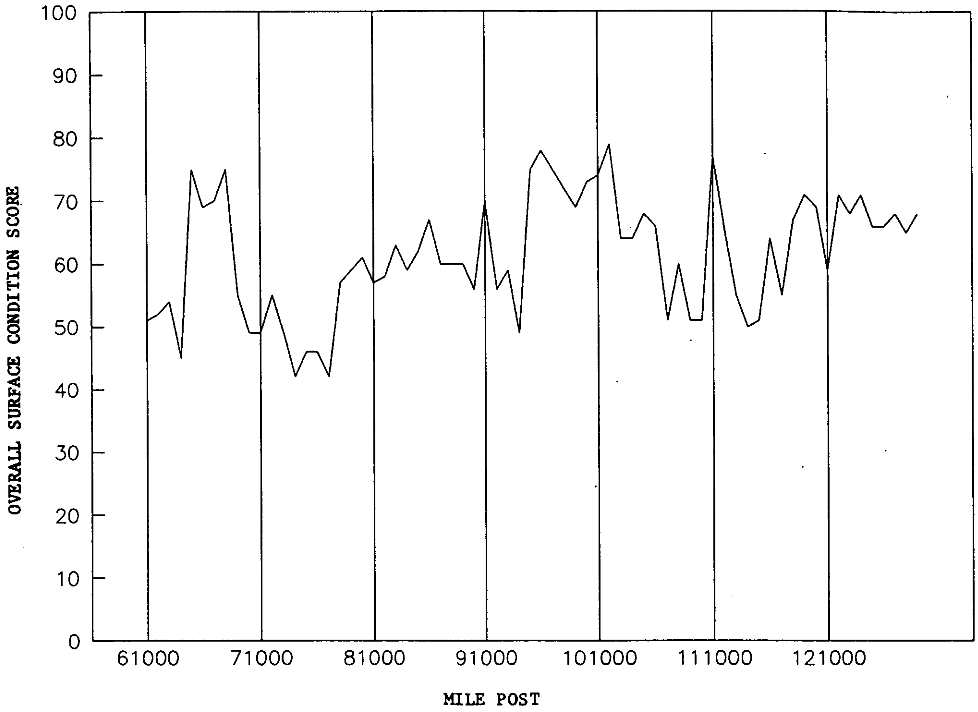

FIGURE 1 Eastbound visual rating.

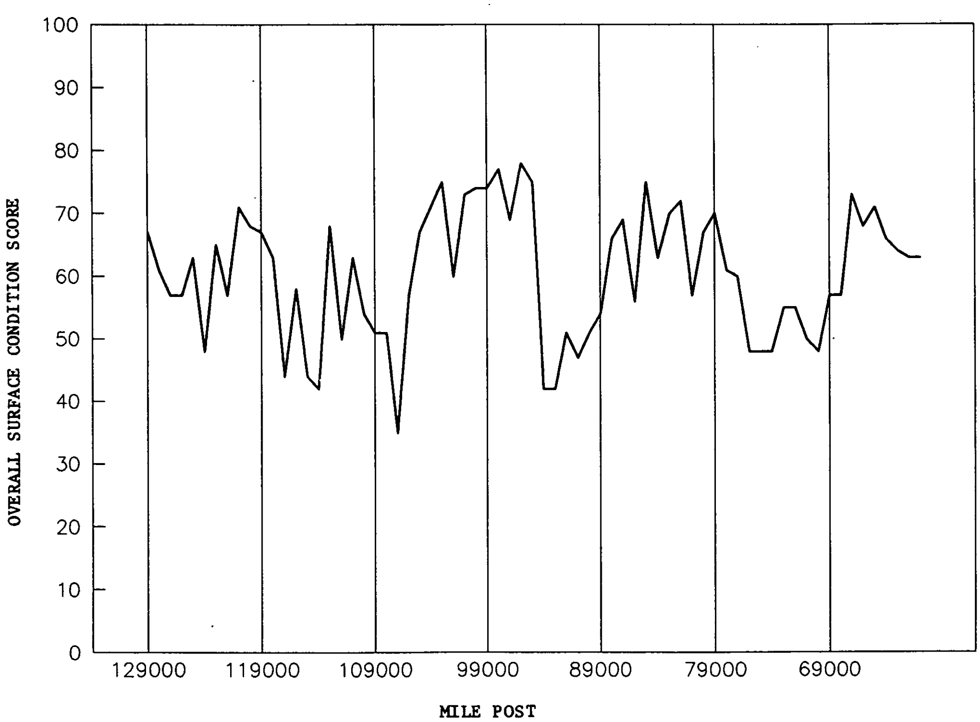

FIGURE 2 Westbound visual rating.

along most of the route (trucks include vehicles of 6 tons or greater). The total number of truck loadings on the East-West Extension was estimated using traffic data from 1988, 1989, and 1990. Assuming a yearly growth since 1974 of 7 percent, the average yearly transaction growth on the East-West Extension over the last 6 years, it is estimated that the East-West Extension has had approximately 3,895,000 truck applications or 5,452,441 equivalent single-axle loads (ESALs) in both directions (85 percent of the traffic in the driving lane and 15 percent in the passing lane).

Visual Condition Survey

The visual survey consisted of a detailed survey of the first 500 ft beyond the even mileposts in the direction of travel from Milepost 61 through Milepost 128. Information outside the 500 ft was gathered by driving at low speed on the shoulder. Conditions such as heave cracking, patched areas, broken slabs, and areas of notable in-slope erosion were visually inspected.

Several types of distress were present with the extent and severity varying significantly throughout the entire area. The predominant types of distress present were spalling of the transverse joints and spalling and cracking of the longitudinal joints in the driving lanes and on the shoulders. The overall condition scores for each direction are shown in Figures 1 and 2. A score of 100 means the pavement is in excellent condition, a score of 50 means the pavement is in fair condition, and a score of 0 means the pavement is failed.

The distress severity associated with the longitudinal joints is one of the most severe because both concrete durability spalling and longitudinal cracking exist. In several locations, the severity level of the centerline joint is so severe that it can interfere with vehicles going back and forth across the joint during lane changes.

Another major problem with the pavement was the spalling of the transverse joints. The severity varied from low to high, and in some locations the spalling was as wide as 12 in. on either side of the joint and as deep as 8 in. Figures 3 and 4 show the combined condition of the longitudinal and transverse joints throughout the project.

The entire pavement structure exhibits a nearly total failure of the transverse and longitudinal joints sealant.

The average faulting that was measured in the field was 0.123 in., and in some locations the measurement was as high as 0.27 in. A faulting of less than 0.25 in. is considered low severity.

NDT

NDT was performed using an FWD. The FWD is an impulse device that exerts a force similar in magnitude and duration to a moving vehicle tire load. By varying the weight and height from which it is dropped, the magnitude of the load can be changed. The resulting pavement deflection is measured by seven seismic deflection transducers, one of which is at the loading plate and the others at preset intervals from the loading plate.

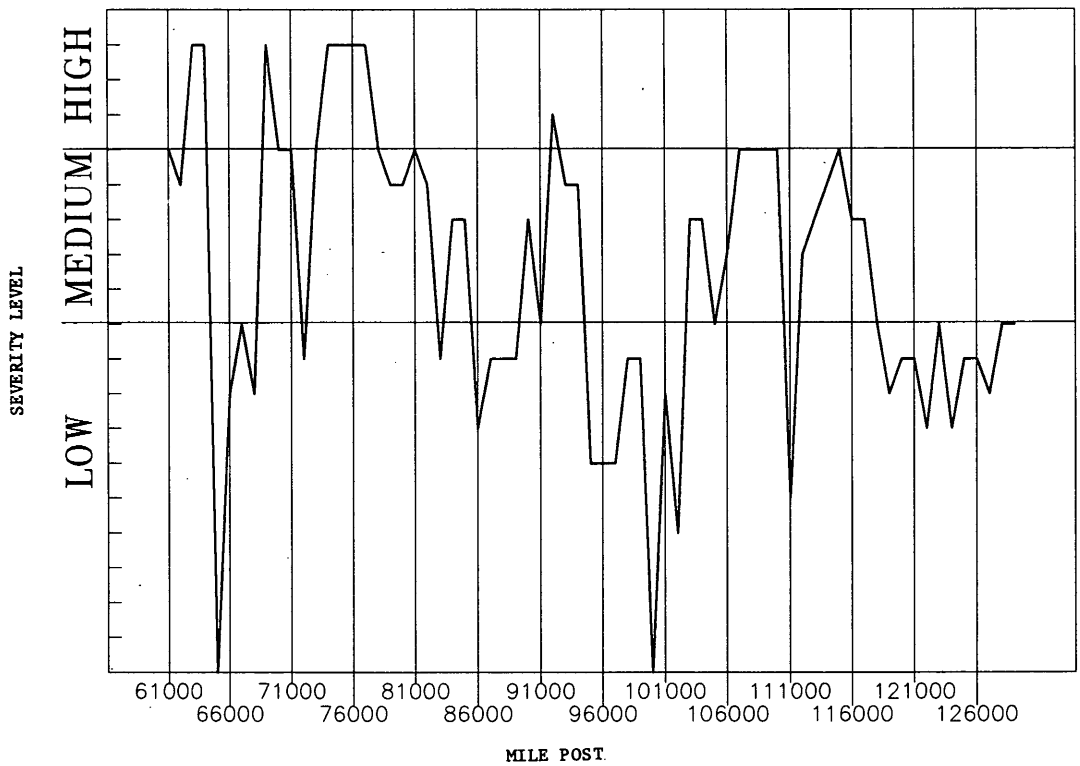

FIGURE 3 Eastbound visual rating of joints.

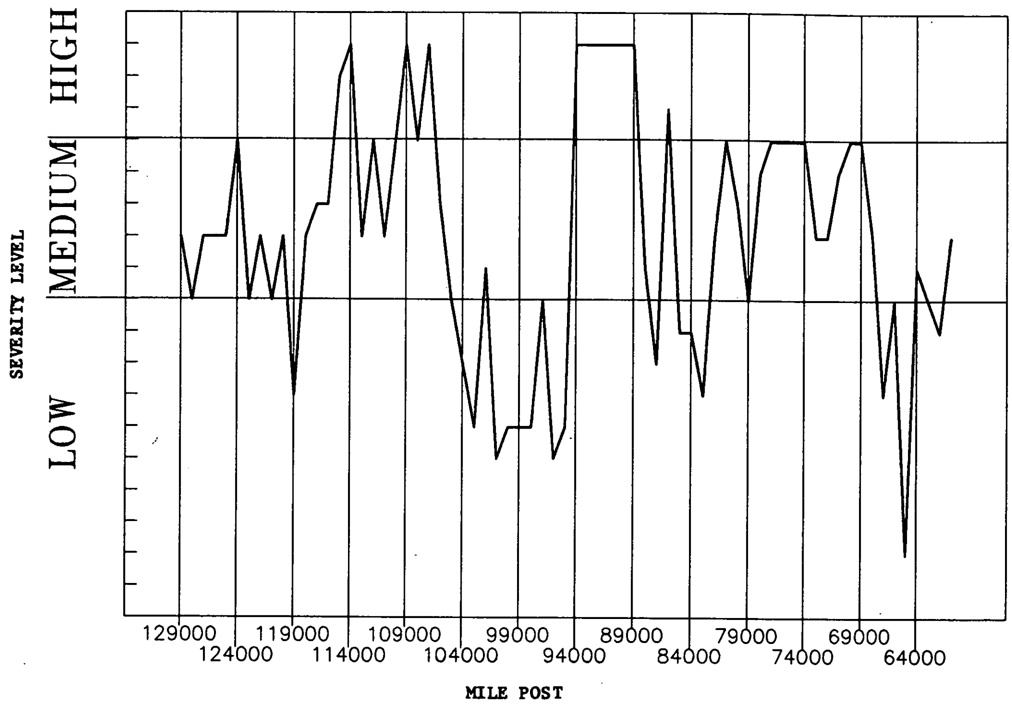

FIGURE 4 Westbound visual rating of joints.

The initial pattern called for each test site to have five center slab tests, three lead corner tests, and three lag corner tests as shown in Figure 5. However, after 3 days of testing with very little variability between test readings, the pattern was changed to four center slab, two lead corner, and two lag corner tests.

For the East-West Extension testing program, the weights were dropped from three heights to produce loads of approximately 8,000, 11,000, and 14,000 lbf. The deflections under the loading plate, 12 in. behind the plate, and at 12, 24, 36, 48, and 60 in. in front of the plate were recorded during testing.

Pavement load-deflection data were used to estimate the PCC slab modulus of elasticity, PCC modulus of rupture, transverse joint load transfer, loss of support underneath the slab, and foundation support (effective K-value beneath the PCC slab).

Coring

Concrete coring was performed to determine the extent of the joint deterioration, properties and thickness of the PCC

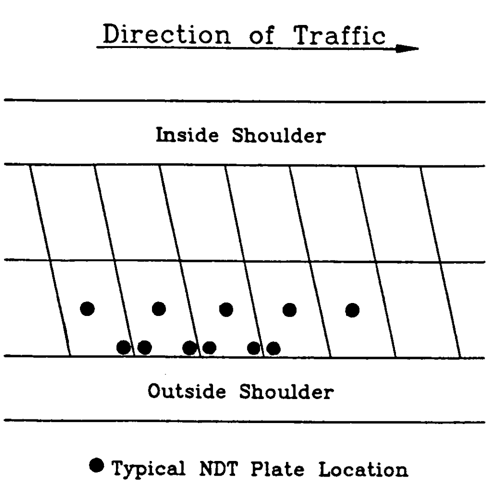

FIGURE 5 FWD testing pattern.

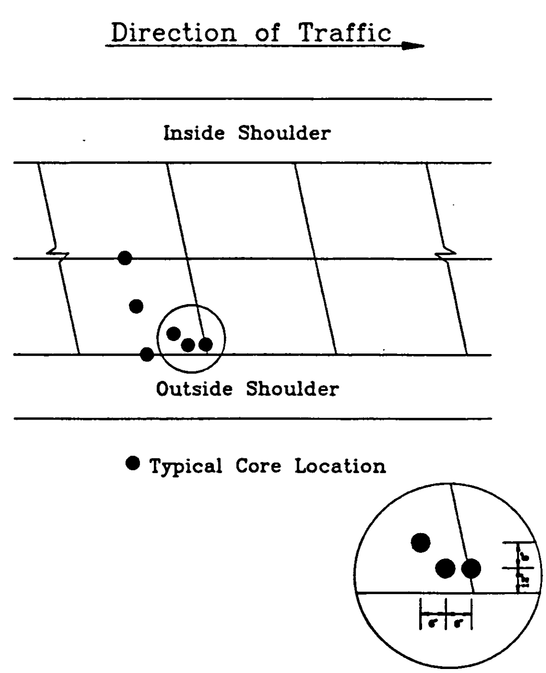

FIGURE 6 Concrete coring locations.

TABLE 1 PCC Thickness and Aggregate Type

MILE POST	THICKNESS, in.	AGGREGATE TYPE
67.5 EB	14.5	Buff and white crushed dolomite
76 EB	15.5	Buff and white crushed dolomite
92 WB	16	Gravel
98.5 WB	15	White dolomite
100 EB	14	White dolomite
112 EB	14.5	Buff and white crushed dolomite
116 WB	14	Buff and white crushed dolomite
125 WB	14	Gravel

slabs, air voids system analysis, and petrographic examinations. A total of 53 concrete cores were taken, in groups of 4 to 8, at eight different milepost locations, four eastbound and four westbound. The sites for testing were chosen to get representative samples from each soil type, pavement distress level, and construction zone on the East-West Extension.

The coring pattern called for a core to be taken at the longitudinal, transverse, and lane/shoulder joints, center slab, and corner slab. Additional cores were taken at approximately 6 in. from the longitudinal joint, on the shoulders, and in frost heave locations. Figure 6 shows the location of the cores within a site, and Table 1 gives the thickness of the PCC and the aggregate type at specific mileposts.

After coring, the drilling water remained in the holes (no drainage), showing a very low permeability of the subgrade. The water was manually removed, and the holes were patched with Set 45 concrete.

Soil Sampling

At 14 of the 53 cores taken, borings were made using a truck-mounted drill rig with the bore holes being advanced by continuous auger flight methods. Samples were taken according to the ASTM D 1586 procedure for split-spoon sampling of soils. Representative portions of the split-spoon samples were placed in glass containers with screw-type lids and taken to the laboratory for examination and testing. Laboratory work consisted of water content determinations for most of the samples, with unconfined compression strength tests being performed on representative samples. Approximate measurements of unconfined compression strengths were made for some of the samples using a calibrated pocket penetrometer. The pocket penetrometer is an indirect method for evaluating the compressive strength of a clay soil. All of the slab sections sampled rested directly on a fine-grained soil subgrade.

FWD ANALYSIS

FWD analysis was performed using finite element techniques and procedures developed by the University of Illinois and the Corps of Engineers.

Modulus of Elasticity and Modulus of Subgrade Reaction

The PCC slab modulus of elasticity (E) and the effective dynamic modulus of subgrade reaction (K-value) (at the bottom of the PCC slab) were backcalculated from deflection basin measurements. A closed-form backcalculation procedure was used, which is based on a theoretically rigorous approach using the principles of dimensional analysis as well as the concept of deflection basin area. The backcalculation method was developed and automated by Ioannides through the computer program ILLI-BACK. The approach models the pavement system as an elastic medium-thick plate resting on a dense liquid foundation.

The backcalculated mean values and ranges were as follows: E = 4,235 ksi (3,300 to 6,000 ksi) and K = 276 psi/in. (120 to 440 psi/in.).

The static K-value is estimated (*1,2*) to be approximately 276/2 = 138 psi/in., which is a typical value for this type of soil.

PCC Modulus of Rupture

The PCC modulus of rupture is most accurately determined by sawing standard-sized beams (6 by 6 by 30 in.) from several slabs and subjecting them to third-point loading tests. This is expensive and time-consuming, however. The PCC modulus of rupture can be estimated fairly well by using the indirect tensile strength of recovered 6-in.-diameter cores or even from the compressive strength of the cores.

The PCC flexural strength or modulus of rupture (MR) can also be estimated approximately from the PCC modulus of elasticity backcalculated from the FWD test results. The following relationship was developed at the University of Illinois to obtain an approximate flexural strength of pavement slabs nondestructively (*1*):

$$MR = 43.5(E/10^6) + 488.5$$

where MR is the modulus of rupture of the PCC and E is the modulus of elasticity of the PCC.

The calculated mean third-point modulus of rupture value is 673 psi.

Transverse Joint Load Transfer

The deflection load transfer across the transverse joints was measured by FWD testing. The load transfer is computed as the ratio of the unloaded slab's deflection to that of the loaded slab's deflection. A small correction is applied to this load transfer efficiency to allow for natural slab bending, which would occur even though no joint existed between the first and second sensor. The following equation was used to calculate the percent load transfer:

$$\text{Load transfer efficiency (\%)} = \frac{D_L}{D_A} \times 100$$

where D_A is the deflection in the approach slab and D_L is the deflection in the leave slab.

The FWD testing was conducted during cool temperatures when the joints were not excessively tight. The mean load transfer of transverse joints is 95.6 percent, which is very high, indicating good aggregate interlock across the joints. The high

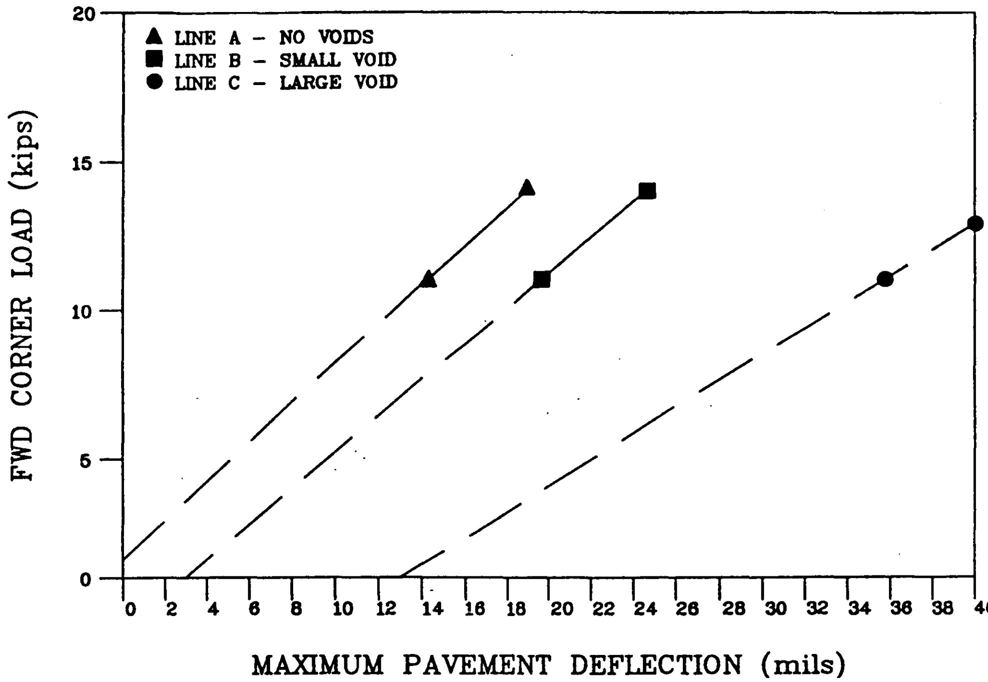

FIGURE 7 Load versus deflection void detection plot.

load transfer explains the low severity faulting of the transverse joints even though the joints are not doweled.

Corner Void Detection

A void can be described as an area of loss of support beneath the slab corners. This occurs because of heavy loads deflecting the slab corners, causing some pumping of fines with impulse water pressure. Any loss of support, even less than 0.10 in. thickness beneath the slab, will result in greatly increased slab stresses, causing corner breaks or diagonal cracks. The most common location for these voids is slab corners along the lane/shoulder joints. The loss of support can be determined using a procedure developed at the University of Illinois for the FWD (*3*). An examination of the load versus deflection at a slab corner can provide a rapid and simple indication of the existence of a void beneath the slab corner. The corner of each slab tested was loaded at three load levels and the corresponding deflections measured. Corners that have a load versus deflection plot that crosses the deflection axis near the origin of zero deflection, such as Line A in Figure 7, do not have voids beneath the slab. However, a line that passes significantly to the right of the origin, such as Lines B and C in Figure 7, is indicative of a void or loss of support beneath the slab corner. Figure 8 shows actual data from the East-West Extension.

After reviewing all of the load versus deflection plots, it was concluded that no voids or loss of support existed beneath the slabs tested except at Milepost 62.0153 eastbound. This was probably caused by the full-depth asphalt patch of the shoulder across the joint. Most likely the subgrade soil was disturbed during patching.

In addition to the NDT, an epoxy/core test procedure developed at Purdue University was used to detect voids at selected locations. The test was performed on several locations where voids were thought to exist on the basis of visual observation. No evidence of voids was detected at any of the locations tested.

LOAD-CARRYING CAPACITY

The load carrying capacity was evaluated using ILLISLAB, a computer-based finite element model, to determine the critical stresses in the PCC slab created by the truck loads. The modulus of elasticity of the PCC and the effective K-value

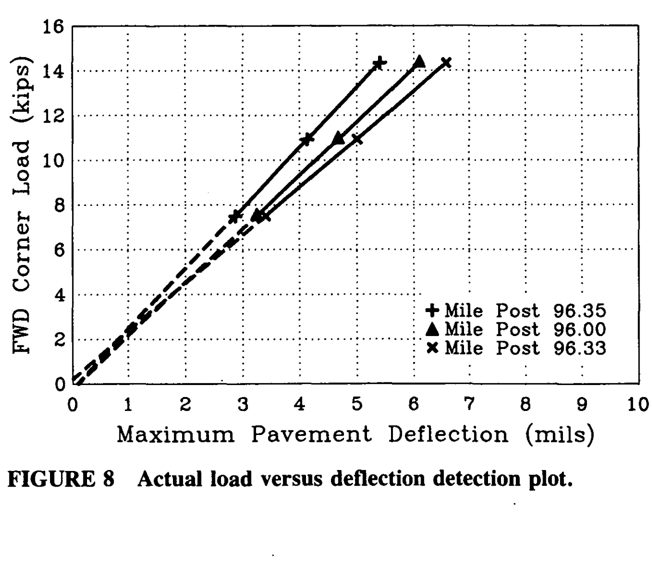

FIGURE 8 Actual load versus deflection detection plot.

were used along with the traffic characterization and weight to determine the critical stresses. The axles were positioned so that the wheel was placed directly on the slab edge along the lane/shoulder joint, the most critical location for stresses and crack development.

The number of edge loads allowable to failure (defined as the point at which 50 percent of the slabs suffer cracking) is calculated on the basis of the strength of the slabs and the critical stress produced by the traffic loading. The following equation form developed using Corps of Engineers field data was used:

$$\log_{10}(\text{axles}) = a\left(\frac{\text{MR}}{\text{stress}}\right)^{b}$$

where

MR = modulus of rupture of PCC slabs (psi),
stress = edge stress under traffic load (psi),
a, b = empirically derived constants developed from Corps of Engineers field data, and
axles = number of edge loading axles to failure (defined as 50 percent slab cracking).

This analysis showed that the number of edge loads to failure for an 18-kip axle was 5.45×10^{18} load applications. This indicates a very structurally sound PCC pavement, and no transverse cracking from traffic loadings is expected.

LABORATORY STUDIES

A total of 51 cores were received. Several of those cores were used for petrographic examinations and air void parameter determinations. The petrographic studies were done according to ASTM C 856, and the air content studies were done according to ASTM C 457. The modified point count method was used.

Detailed petrographic studies were conducted on 1 core from each section of roadway, plus 2 additional cores from selected area, making a total of 10 cores studied. Air void studies were also done on one core from each section plus two from selected areas. Cursory petrographic examinations were conducted on the remainder of the cores.

Three distinct coarse aggregate types were used in the concretes. They consist of a buff and white crushed dolomite, a white limestone/dolomite, and a gravel consisting of dolomite, granite, chert, and basalt. All cores contained natural sand, which did not contribute to the deterioration.

The deterioration observed in the cores fell generally into two distinct categories: (*a*) freeze-thaw damage to the portland cement binder and (*b*) D-cracking of the coarse aggregates.

Freeze-thaw damage occurs in concrete that is sufficiently saturated and does not contain an adequate air void system. D-cracking results from a combination of a specific aggregate type, the presence of moisture, and freeze-thaw cycling.

Soil Results

All of the soils were fine-grained (sand, silt, and clay) ranging in AASHTO classification from A-2-6 to A-7-6; most were A-6. The liquid limits (LLs) ranged from 20 to 51 and the plasticity indices (PIs) ranged from 10 to 30. The cohesive soils varied from hard to very tough, whereas all of the cohesionless soils were firm. A thin layer of organic topsoil was occasionally encountered, with characteristically high moisture contents and low dry densities. Occasional lenses of fill directly beneath the pavement were well compacted, with moderate moisture contents.

Analysis of Air Void Systems

Properly air-entrained PCC is protected from the increase of hydraulic pressure during freeze-thaw cycles. The air void bubbles would act as pressure relief valves or "safety valves" that allow the excess water to escape to them and freeze without damaging the PCC (*4*).

To evaluate the air void parameters of the project, 10 cores were selected for testing. The results are given in Table 2. ACI Committee 345 has recommended that the number of voids per lineal inch be significantly greater than the percentage of air, that the specific surface (**a**) be greater than 600 in.²/in.³ and that the spacing factor (L) be less than 0.008 in. For the ¾-to 1-in. maximum aggregate size in the I-88 cores, these requirements should be met with an air content of about 4½ to 7½ percent. However, although essentially all 10 of the cores tested had total air contents within this range, only one (125-4) met the requirements listed. The causes for this are not determinable. But the undesirable air void parameters measured are certainly sufficient to explain the cyclic freezing damage observed in most of the cores subjected to petrographic examination.

In general, the entrained air voids of the entire project are not well distributed. They tend to form in clusters and around coarse aggregate particles; thus, the air void systems are not well developed. In addition, the air in many of the cores occurs as large entrapped voids, which furnish a minimum protection from cyclic freezing damage. This explains the premature failures of the transverse and longitudinal joints.

SUMMARY OF EVALUATION RESULTS

This section presents the findings and engineering analyses on which maintenance and rehabilitation strategies were developed. The results of the evaluation study are as follows:

1. The concrete slab is 14 in. thick or more. Most pavement cores were greater than 14 in., and those that were not were within ¼ in.

TABLE 2 Results of Air Void Analysis

Mile Post	Total % Air	Voids/in	Specific Surface (a) in²/in³	Spacing Factor (L) in
67.5 EB	7.0	3.9	224	0.014
76 EB	6.2	3.3	210	0.017
92 WB	5.0	3.8	303	0.012
98.5 WB	5.5	4.3	313	0.015
98.5 WB	4.4	4.1	367	0.014
100 EB	5.6	4.8	342	0.009
112 EB	5.3	3.6	271	0.014
116 WB	6.7	4.4	262	0.016
125 WB	5.1	7.8	621	0.006
125 WB	4.9	3.8	308	0.014

2. Estimated total air contents range from 4.4 percent at Milepost 98.5 WB to 7.0 percent at Milepost 67.5 EB, and much of this was entrapped air. In general, the entrained air voids are not well distributed. The voids tend to form in clusters and around coarse aggregate particles; thus, the air void systems are not well developed. In addition, the air in many of the cores and in spalled pieces of PCC observed in the field occurs as large entrapped voids, which furnish a minimum of protection from cyclic freezing damage. This is believed to be the major cause of the spalling.

3. Some of the aggregate is susceptible to D-cracking damage. This has resulted in some of the spalling in limited specific sections along I-88.

4. The average dynamic modulus of subgrade reaction is 276 psi/in. (static value of about 276/2, which is typical of this type of soil).

5. The average modulus of elasticity of the PCC interim slabs is 4,200,000 psi, which is considered sound concrete.

6. The average modulus of rupture of the PCC estimated by NDT testing was 673 psi, which is considered good.

7. The average load transfer across the transverse joints is 95 percent. Any value 70 percent or more is considered good.

8. The average transverse joint faulting is 0.123 in. In several locations, the faulting was as high as 0.27 in.

9. The PCC slab is structurally sound and can carry a large number of repetitions before load fatigue cracking. Existing cracking is caused from other causes such as frost heaving or inadequate longitudinal joint construction.

10. There were no indications of any loss of support beneath any of the slabs tested.

11. The soils found on the extension were generally AASHTO Classfication A-6.

12. All of the joints throughout the entire project are experiencing deterioration caused by damage from cyclic freezing, D-cracking of the coarse aggregate particles, or a combination of the two.

13. The pavement has become very rough from the severe joint spalling.

MAINTENANCE AND REHABILITATION STRATEGIES

Several maintenance and rehabilitation strategies were considered for the East-West Extension project. All of the strategies were evaluated using EXPEAR (EXpert system for Pavement Evaluation And Rehabilitation) (*5–7*). EXPEAR is a practical and comprehensive computerized system to evaluate concrete highway pavements, develop rehabilitation alternatives, and predict the performance and cost-effectiveness of the alternatives. EXPEAR was originally developed for FHWA by the University of Illinois and it was recently modified by ERES Consultants, Inc.

On the basis of the recommendation of the Illinois Tollway Authority, strategies with different life expectancies were developed. These strategies were grouped into three different groups (short-term, intermediate-term, and long-term performance) so that the Illinois Tollway Authority can base its choice of strategy on budget constraints and performance period. The strategies were evaluated by life cycle cost analysis, which makes it possible to compare alternatives with different initial construction costs on the basis of equivalent annual cost. The unit costs information was obtained from several local areas and is included in Table 3. Table 4 gives all of the strategies, life expectancy, total initial costs, and equivalent annual costs.

The overlay thicknesses in Strategies 4, 5, 6, 7, and 9 are only approximate and must be verified through an engineering design analysis.

1. Spray patch (1): This strategy includes removing loose materials from the center longitudinal and transverse joints and spray patching the joints with emulsion and aggregate.

2. Spray patch (2): This strategy includes cleaning all of the joints except the shoulder joints and spray patching them with emulsion and aggregate.

3. Repair spalls with PCC: This strategy includes milling out only the center longitudinal and transverse joints 2 ft wide by 6 in. deep, repairing them with PCC, and then sawing and sealing the joints.

4. AC overlay (1): This strategy includes repairing the joints with AC and applying a 5-in. AC overlay.

5. AC overlay (2): This strategy includes repairing the joints with PCC and applying a 5-in.-minimum AC overlay.

6. Crack and seat: This strategy includes breaking all of the PCC slabs of Lanes 1 and 2 into small pieces (1 to 2.5 ft), seating the pavement, and applying a 5-in. AC overlay.

7. Rubblizing: This strategy includes breaking all of the PCC slabs into rubble (less than 12 in. in size), compacting the rubble, and applying a 7-in. AC overlay.

TABLE 3 Unit Costs Information

DESCRIPTION	UNIT COST(dollars)	UNIT
Unbonded PCC Overlay	3.05 per inch	SY
AC Overlay	33.22	TON
Crack and Seat	1.00	SY
Rubblizing	1.65	SY
AC Partial Patch (Spray Patch)	1.31	LINEAR FT
	11.79	SY
PCC Partial Patch	15.00	LINEAR FT
	68.00	SY
PCC Recycling + Base	39.06	SY
Longitudinal Subdrains	2.46	LINEAR FT
Reconstruct Heaves	50.00	SY

TABLE 4 Summary of the Rehabilitation Strategies

Performance Period	Strategy	Life Expectancy (Years)	Total Initial Cost (U.S. Dollars)	Total Annual Cost (U.S. Dollars)
Short-Term	1. Spray Patching (1)	2	4,115,160	2,087,940
	2. Spray Patching (2)	2	6,024,252	3,056,700
	3. Patching with PCC	5	29,158,986	6,181,572
Intermediate-Term	4. AC Overlay (1)	7	33,938,064	5,288,574
	5. AC Overlay (2)	10	58,933,728	6,707,628
	6. Crack & Seat/ AC OL	10	34,919,658	3,974,400
	7. Rubblizing/ AC OL	10	52,246,662	5,946,558
Long-Term	8. Reconstruct Outer Lane & Restore Inner Lane	15	45,315,198	3,685,290
	9. Unbonded PCC Overlay	20	84,640,230	5,523,450
	10. Reconstruction	20	107,454,390	6,985,284

8. Recycling and restoring: This strategy includes recycling Lane 1 into a 4-in. stabilized base with a 10-in. PCC pavement and the joint spacings matching those of Lane 2. Also, all of the joints in Lane 2 would be repaired with PCC.

9. Unbonded PCC overlay: This strategy includes applying a 1-in. AC separation layer over Lanes 1 and 2 and then placing a 9-in. jointed plain PCC overlay with 15-ft joint spacing (*8*). The shoulders will be overlayed with 9 in. AC.

10. Reconstruction: This strategy includes removing the PCC from the mainlines and shoulders and rebuilding the pavement with 4-in. base, 12-in. PCC surface with 15-ft joint spacing, longitudinal edge drain, and 7.5-in. PCC shoulders.

CONCLUSIONS AND RECOMMENDATIONS

The engineering analysis conducted to determine an effective rehabilitation plan for the East-West Extension had many associated complexities. A major effort was made to determine the primary cause of the current surface distress affecting the user's satisfaction and riding comfort. After the pavement engineering investigations and analysis were completed, it was concluded that the main problem was the spalling of the transverse and longitudinal joints. The spalling of the joints was caused primarily by damage from cyclic freezing of the cement paste and D-cracking of the coarse aggregate particles. Even though the air content in the concrete was within specifications, the size and distribution of the voids were not within specifications to protect the concrete from freeze-thaw cycles, which caused the premature failure of the joints.

The pavement itself is structurally sound and has a remaining life left in it as would be typically expected. The primary concern is rehabilitation of the transverse and longitudinal joints and restructuring the pavement's resistance to infiltration of surface moisture. By accomplishing this, any chances of further damage from freeze-thaw will be reduced, and the service life of the pavement will substantially increase.

The preferred rehabilitation for the East-West Extension is primarily composed of three components. These are recommended on the basis of effectiveness, performance, and user satisfaction. Annual budget constraints have also been recognized along with the cost-efficiency of interim maintenance operations.

Low-Severity Sections

The low-severity areas should be addressed as soon as possible to prevent further disintegration of the pavement surface. The recommendation is to clean and reseal all the longitudinal and transverse joints. There will also be a need to do limited slab replacements and some joint repairs in isolated areas. There are locations within the low-severity areas with few broken slabs due to frost action in the soils. Those could be replaced and the soil removed and replaced. However, this recommended treatment will not address the profile of the pavement.

Medium- and High-Severity Sections

Before recommendations are made for repair in these areas, it must be recognized that everything cannot be done immediately from budget, project development, and overall tollway system planning aspects. Interim alternatives to address the current effort required to maintain the surface profile and provide user comfort should be performed. It was recommended to the Tollway Authority that the transverse and longitudinal joints be repaired temporarily using the spray patch procedure until a permanent rehabilitation strategy is implemented.

The overall rehabilitation of the project will be based on the service life the Tollway Authority wishes to obtain, budget constraints, and construction time and lane closures. The patching alternatives are not favorable due to insufficient life. The other alternatives pose a trade-off between longer service life and lower initial construction cost and increased future rehabilitation costs. For example, if the Tollway Authority favors a long-term (20 or more years) strategy, an unbonded PCC overlay is recommended. If the Tollway Authority favors a 10-year strategy, an AC overlay after rubblization or crack and seat is recommended.

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