Validation of Staged Construction Pavement Design with the Falling Weight Deflectometer

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In 2005, the Illinois Tollway staff rehabilitated a 33-mi-long segment of Interstate Highway 88 in Illinois by rubblizing deteriorated jointed plain concrete pavement (JPCP) and placing a hot-mix asphalt (HMA) overlay on the rubblized JPCP. The pavement design called for the ultimate pavement structure to be constructed in two stages: the first at the time of rubblization, when at least 6.0 in. of HMA would be placed, and the second 10 years later, when 2.0 in. of HMA would be cold milled and 6.0 in. of HMA placed back. The pavement design called on the tollway staff's experience with rubblization and research surrounding the fatigue endurance limit to develop a staged construction strategy. The strategy would reduce up-front construction costs by placing a minimum of 6.0 in. of HMA at initial construction. The tollway staff was confident that the strategy would work, but the pavement performance was monitored annually for verification. Annual monitoring included pavement distress condition surveys, deflection testing and deflection data analysis, and pavement coring. In particular, the tollway staff desired confirmation that the staged construction strategy would yield a perpetual asphalt pavement by controlling the amount of bottom-up fatigue cracking that would develop in the asphalt mat. After 9 years of observations and data analysis, the staff has confirmed the pavement is exhibiting the properties of a perpetual pavement and the staged construction strategy should ultimately prove successful.

In 2005, staff at the Illinois Tollway desired to rehabilitate a deteriorated 33-mi-long segment of Interstate Highway 88 between US-30 and Illinois SR-251. The deteriorated pavement consisted of 14 in. of jointed plain concrete pavement (JPCP) that had been placed in 1974 with a 3-in. asphalt overlay. The JPCP was constructed directly on the native subgrade. The silty clay soil conditions in this area were poor, with California bearing ratio values averaging 3%.

The Illinois Tollway Materials Engineering Department collaborated with Applied Research Associates, Inc. (ARA) to develop a pavement design solution that would minimize up-front construction costs, work within the confines of an extremely short construction schedule, and use as many of the in-place materials as possible (*1*). The department and ARA developed a solution that was expected to meet all of those criteria. The solution involved

- Cold milling the existing asphalt overlay (from the mainline lanes and shoulders),
- Rubblizing the existing JPCP in place with a multiple head breaker,
- Compacting the rubblized JPCP with a Z-foot roller,
- Placing a minimum of 6.0 in. of hot-mix asphalt (HMA) materials on top of the rubblized JPCP (Stage 1), and
- After 10 years, cold milling 2.0 in. of HMA and placing 6.0 in. of HMA (Stage 2).

This paper documents the Illinois Tollway staff's monitoring and observation process during the first 9 years of service to validate the original design assumptions and verify the Stage 2 pavement design strategy and overall pavement performance.

PAVEMENT DESIGN

The staff at the Illinois Tollway had experience with rubblization and HMA overlays on its tollway system. Specifically, on the same corridor, the tollway worked with ARA and Marshall Thompson to design two 2-mi test sections on which the existing 14-in. JPCP was rubblized and then 8.0 in. of HMA was placed on top of the rubblized JPCP. These sections were constructed in 2001 and were monitored annually to assess the performance of the HMA overlays. In particular, the tollway wished to see if the HMA overlays would develop bottom-up fatigue cracking. The pavement was designed as a perpetual, or long-lasting, pavement, and the development of bottom-up fatigue cracking would have violated the perpetual pavement concept. The tollway staff had monitored the perpetual pavement test sections annually after construction by performing deflection testing, pavement distress surveys, rut depth measurements, and pavement coring. Through 2005, no evidence of bottom-up fatigue cracking was found (*2*).

The design for this pavement relied heavily on laboratory research work conducted at the University of Illinois at Urbana–Champaign that described the fatigue endurance limit (*3*). Engineers at the tollway and ARA used finite element modeling to predict the strain levels at the bottom of the asphalt layer. Engineers used the hypothesis tested in the university's research that an allowable amount of bottom-up fatigue cracking could develop before additional asphalt was placed on top of the original mat thickness, thereby stopping the progression of bottom-up fatigue cracking and recovering the perpetual pavement. The pavement was designed as a 6-in. HMA with the plan that, over 10 years, the pavement would develop an acceptable amount of bottom-up fatigue cracking and then receive a second thickness of HMA (cold mill 2.0 in., place back 6.0 in.). The net thickness of HMA materials would rise to 10.0 in., which would reduce the strain at the bottom of the HMA thickness to a level too low for bottom-up fatigue cracking to begin and further, halt the migration of any

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bottom-up fatigue cracks that had already formed but had not fully propagated through the original 6-in. HMA mat. Construction of Stage 1 was completed in late 2005.

Whereas the elastic modulus of asphaltic mixtures is well documented, there is not as much consensus on the expected structural performance of rubblized concrete. Part of the problem rests in the discontinuity and variability of the breakage process: rubblization attempts to reduce slabs to particles larger than 3-in. across at the surface and smaller than 9-in. across at the bottom of the slab. The AASHTO *Guide for Design of Pavement Structures* indicates the backcalculated modulus of rubblized portland cement concrete is between 100,000 pounds per square inch (psi) and several hundred thousand pounds per square inch and within project coefficient of variation as much as 40% (*4*). The AASHTO guide suggests structural layer coefficients for rubblized portland cement concrete to come from a range of 0.14 to 0.30. The American Concrete Pavement Association recommends a range of structural layer coefficients for new asphalt concrete between 0.20 and 0.44 and a structural layer coefficient for rubblized concrete between 0.14 and 0.25 (*5*).

NOTES FROM CONSTRUCTION

The Illinois Tollway staff experienced some challenges during Stage 1 of the rubblized pavement construction. The pavement shoulders were difficult to rubblize with uniformity. The outside shoulders were made from JPCP, but they began at the outside edge at 6-in. thick and transitioned to 9-in. thick over the 10-ft width. It was observed during rubblization that when sufficient breakage was achieved in the 9-in. section, the rubblization was pulverizing the JPCP in the 6-in. section and forcing the pieces into the soft subgrade soil. Correcting this action left particles in the 9-in. section that were larger than desired. Although the original design called for the installation of underdrains for the lane-to-shoulder joints for the inside and outside shoulders, the inside shoulder longitudinal underdrain was eliminated for cost and time savings. At horizontal curves, this step placed the lone longitudinal underdrain on the high side of the superelevation, significantly reducing its effectiveness. The tollway had observed over the years many instances of frost heave, and the removal of the underdrain contributed to continued heave of the shoulders, especially in horizontal curves. In 2012, the tollway performed a retrofit installation of the missing underdrains in horizontal curves. Finally, the HMA materials were placed quickly in 2005 to meet the aggressive schedule. Several quality characteristics of the HMA materials (density, asphalt content, aggregate gradation) were found to deviate from the tollway's specifications after the HMA was placed. However, the tollway agreed to keep the HMA materials in place because the road needed to be opened to traffic. Although the quality characteristics were not ideal, the tollway did receive additional HMA materials; the plan thickness was designed to be at least 6.0 in., but pavement cores cut from the HMA mat during subsequent years found at least 6.5 in. and sometimes as much as 7.5 in.

ANNUAL MONITORING OF PAVEMENT PERFORMANCE

Since 2002, the tollway staff has performed, as part of its pavement management system update, a network-level structural evaluation. The goal of the evaluation is to provide an annual structural condition assessment of in-place pavements by evaluating the modulus values of the various pavement materials present on each route, the load transfer efficiency across joints and cracks, and the potential for voids beneath concrete pavements. The evaluation consists of systematic deflection testing along each route on the tollway's system. Deflection tests are performed with falling weight deflectometers (FWDs), and tests have been performed at roughly 1-mi intervals each year. The deflection tests are staggered to slowly shrink the overall deflection testing interval. Deflection tests are performed at center slab locations and across joints or cracks. Pavement cores are cut each year to inspect the thicknesses of pavement material layers at each location and to inspect the physical condition of the pavement materials beneath the surface. Over the years, cores have been cut at both center slab locations and across joints and cracks. In the case of the rubblization project, pavement cores cut after Stage 1 construction were cut through the HMA layers.

Beginning in 2006, the first year after Stage 1 construction was completed, several FWDs were used to perform deflection testing. The models used for each year of data collection on the staged rubblization pavement are presented in Table 1. The FWDs were calibrated annually at calibration centers according to ASTM specifications, and relative calibrations were performed frequently each year to ensure geophones were measuring properly and within expected tolerances.

Deflection testing was performed each year at target load levels of 9, 12, and 15 kips. Testing was performed at approximately 1-mi intervals, staggered from year to year to get more coverage distribution. Therefore, each year of data represents approximately 66 test locations distributed along the 33-mi project length. Surface and air temperatures were recorded at each test. The deflections were normalized to a 9,000-lb load and temperature of 68°F. Deflections were analyzed with ELMOD software (*6*), using an average HMA thickness of 6.5 in. and the plain JPCP thickness (now rubblized) of 14.0 in. Table 2 presents the average modulus values obtained during backcalculation.

Rubblized JPCP Modulus Analysis

Figure 1 shows the rubblized JPCP modulus during the years since construction. The modulus value began low in 2006 and then gained considerable stiffness. There is a lot of variability in the modulus results of the rubblized JPCP, likely owing to the inconsistent particle size and packing inherent in rubblized concrete. However, after 2006, only one backcalculated modulus value was found to be below 300,000 psi (on the eastbound direction of the highway in 2010). These modulus values are much higher than a typical granular base material and are within the accepted range of typical modulus values for HMA materials. These values, although highly variable, support the AASHTO guide recommended range for the structural layer

TABLE 1 FWDs Used During Each Year of Evaluation

Year	Falling Weight Deflectometer
2006	Dynatest FWD 068
2007	Dynatest FWD 068
2008	Dynatest FWD 068
2009	Dynatest HWD 015
2010	Dynatest FWD 068
2011	JILS 20T FWD
2012	JILS 20T FWD
2013	JILS 20T FWD
2014	JILS 20T FWD

TABLE 2 Average Modulus Values Obtained with ELMOD Backcalculation Analysis

Direction of Travel	Year	HMA Modulus (mean, psi)	HMA Modulus (SD, psi)	Rubblized JPCP Modulus (mean, psi)	Rubblized JPCP Modulus (SD, psi)
EB	2006	547,585	154,507	135,666	68,648
EB	2007	354,212	117,338	301,649	106,123
EB	2008	491,890	184,539	203,679	182,744
EB	2010	566,669	215,525	261,264	166,048
EB	2011	461,614	124,446	359,654	172,881
EB	2012	506,665	121,304	275,416	133,160
EB	2013	594,199	146,921	312,234	173,678
EB	2014	426,218	137,786	326,580	168,597
WB	2006	458,126	160,355	140,412	73,337
WB	2007	564,250	198,974	234,592	87,430
WB	2008	438,257	200,490	249,974	133,486
WB	2010	468,932	204,241	260,072	129,684
WB	2011	418,519	147,983	427,853	196,731
WB	2012	483,535	222,729	337,813	129,892
WB	2013	539,406	158,374	402,644	167,499
WB	2014	444,426	230,479	400,154	163,824

NOTE: EB = eastbound; WB = westbound. Data from 2009 were removed from the study after backcalculation results produced root mean square error (RMSE) values around 10%. All other data met the industry-accepted RMSE criteria of less than 2%, indicating good basin fit.

coefficient of rubblized concrete, especially the higher side of the published range.

HMA Modulus Analysis

Figure 2 shows the backcalculated modulus values for the HMA materials. The backcalculated modulus results came after normalizing the deflections for temperature and load. The HMA materials are exhibiting a much more consistent modulus value, averaging around 500,000 psi in each direction. These values indicate HMA materials that are performing well.

Layered Elastic Analysis

One of the critical factors in the success of the rubblization was the horizontal strain at the bottom of the asphalt mat. Strains at this location in the pavement structure can instigate bottom-up fatigue cracking in asphalt; such cracking would invalidate the perpetual pavement concept. The pavement structure and deflection values were analyzed with WinJULEA software to predict the strains at the bottom of the HMA mat (*7*). Table 3 presents the predicted strains from the WinJULEA analysis.

Although the fatigue endurance limit research went a long way to dispel the assumption that horizontal strain levels greater than 70 microstrain at the bottom of the asphalt would instigate bottom-up fatigue cracking, the results of the strain predictions indicate the strain levels are well below 70 microstrain anyway. The strain levels do vary from year to year, and the strains are higher in the eastbound direction than the westbound direction. Nonetheless, after 2006, all but one of the average strain values was found to be below 50 microstrain. This indicates the pavement is expected to perform as a perpetual pavement.

Focused Pavement Coring Evaluation in 2011

In 2011, the surface of the pavement was observed to be deteriorating. Specifically, several forms of cracking were observed on the pavement

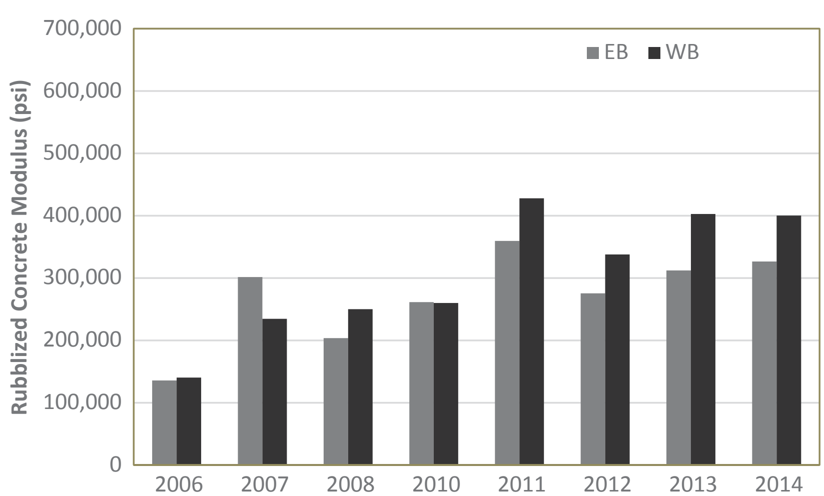

FIGURE 1 Backcalculated modulus values of rubblized JPCP.

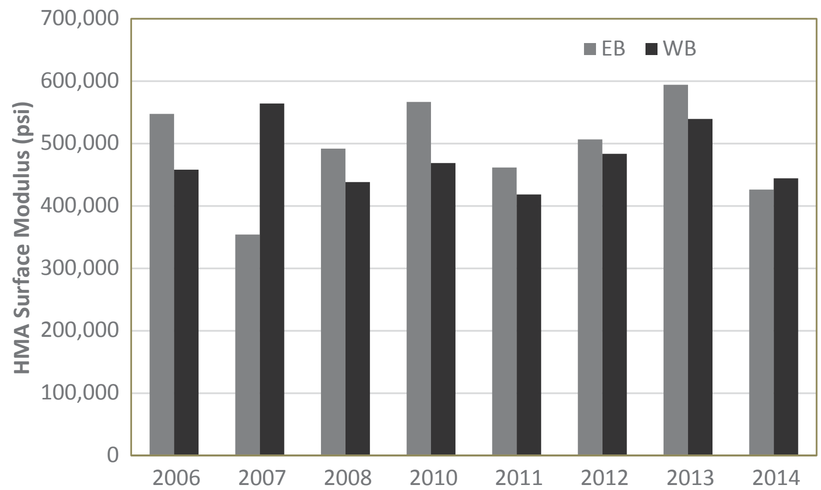

FIGURE 2 Backcalculated modulus values of HMA.

surface, including transverse cracking (resembling reflective cracking) and fatigue cracking, as well as coarse aggregate loss (raveling). In addition, a consistent longitudinal crack had developed in the center of the passing lane. To investigate the source (or instigation location of the cracking), many pavement cores were cut through locations of various types of distresses. In all cases, distresses observed on the pavement surface were found to have originated at the surface and were migrating downward through the HMA mat (*8*). None of the distresses were found to have propagated through the full depth of HMA. The top-down cracking and surface distress phenomenon are expected in a perpetual pavement, especially one in this location, where each year the asphalt surface is subjected to significant temperature variation, significant amounts of sunshine, and many freeze–thaw cycles. These environmental stresses take their toll on liquid asphalt binders and asphalt surface mixtures. It was recommended to the tollway to perform a mill-and-overlay treatment to extend the service life of the pavement to the planned time of the second phase of construction. Staff elected to place a double-layer microsurface on these pavements in 2012 and 2013.

CONCLUSION

The Illinois Tollway staff has successfully validated the staged rubblization pavement design and construction strategy that it deployed in 2005. Through annual monitoring and pavement preservation, the staff has verified that the horizontal strain levels in the bottom of the HMA mat are below the threshold values assumed to instigate bottom-up fatigue cracking. The pavement is behaving as a perpetual pavement and should serve as a long-lasting foundation for the Stage 2 construction and future resurfacing projects. The tollway staff has chosen to further increase the chances for this pavement's success by completely reconstructing the shoulders and installing longitudinal edge drains at both lane-to-shoulder joints for the entire length of the project. Stage 2 construction for this project is scheduled for completion in 2016.

TABLE 3 Predicted Microstrain Levels Using WinJULEA

Year	Eastbound Microstrains ($\mu\varepsilon$, 10^{-6} in.^{2})	Westbound Microstrains ($\mu\varepsilon$, 10^{-6} in.^{2})
2006	71.8	74.0
2007	40.3	47.7
2008	54.9	47.2
2010	43.7	45.2
2011	33.7	28.4
2012	20.4	35.6
2013	37.4	30.0
2014	37.1	22.6

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