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Recycling and Overlay of Portland Cement Concrete Pavement—a Case Study

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A broken and seated portland cement concrete pavement (PCCP) was analyzed using destructive and nondestructive testing (NDT). The paper focuses on the results of a series of NDT methodologies by which the effective structural capacity of the overlaid pavement was estimated. The NDT included Falling Weight Deflectometer (FWD) and Road Rater (RR), followed by back-calculation and verification routines. A comparison was performed between the FWD and RR deflection bowls, which demonstrated a close correspondence between the two NDT devices. The effective modulus of the broken and seated PCCP was estimated and it showed close agreement with previous work conducted by other investigators. A large-stone hot mix asphalt (HMA) overlay was placed on top of the broken and seated PCCP; this decision was made based upon rutting resistance of this type of mix as reported in previous studies.

Keywords: Pavements; Pavement recycling; Asphalt; Concrete overlay; NDT

INTRODUCTION

Kentucky is well known for its mechanistic–empirical approach to pavement design. In recent years, the Kentucky Transportation Cabinet (1998) has extended its pavement thickness design methodologies to pavement rehabilitation and overlay design situation. One such example is the rehabilitation and in-place recycling of an old portland cement concrete pavements (PCCP), which usually involves some type of hot mix asphalt (HMA) overlay. At the same time, asphalt reflective cracking in such overlay situations is a very common problem. Breaking and seating of PCCP prior to asphalt overlay has proven to be an effective means for: (a) reducing the effective slab size and thereby minimizing the effect of mechanisms

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that lead to reflective cracking (i.e. horizontal differential movement caused by thermal contraction/expansion, and vertical differential movement at cracks and joints caused by traffic loads), and (b) providing an opportunity for filling any cavities and weak spots underneath the deteriorated PCCP. National Asphalt Pavement Association (NAPA) published a historical perspective as well as a state-of-the-art on such applications (Eckrose 1983, Crawford 1985).

The focus of this paper is on a particular breaking, seating, and overlay project within the coal region of Kentucky. Special features of this project included an old PCCP which was broken, seated, and overlaid with a large-stone HMA layer. Typically, asphalt overlays on PCCP are subjected to extremely high shear stresses, as reported by Carpenter and Freeman (1986), and Ameri-Gaznon and Little (1990), which often lead to severe rutting in the asphalt layer. Large-stone mixes have desirable rut resistant properties which make them suitable for PCCP overlay application. Additionally, destructive and nondestructive tests were conducted on this pavement.

MOUNTAIN PARKWAY PROJECT

The Mountain Parkway is a four-lane, limited-access highway in eastern Kentucky. The rehabilitation included breaking, seating, and overlaying an old 230 mm PCCP over a distance of 27 km.

Breaking, Seating, and Overlay

The breaking of the 230 mm PCCP was done with a hydraulic breaker. One pass was made with the nominal PCC block size being 300 mm. The broken pavement was seated with a 100 ton pneumatic-tire roller. The pavement was then overlaid with 190 mm of HMA. The asphalt overlay included two 75 mm lifts of a large-stone mix (CK Base, Kentucky 1998), and a 40 mm surface wearing course.

Large-stone Hot Mix Asphalt Overlay

Kentucky has been using large-stone asphalt mixes as a means for reducing rutting on heavy duty new pavements as well as rehabilitation of heavy haul roads (Mahboub & Allen, 1990; Mahboub & Williams, 1990; Anderson *et al.*, 1991; Mahboub *et al.*, 1992). Recent work at Texas Transportation Institute, Texas A&M University (1997) summarized the state-of-the-art related to large-stone mixes and recommended guidelines for design and construction of such mixes. It is important to note that results presented in this paper were completed prior to the full implementation of Superpave™ technology in Kentucky. Therefore, all mixtures were designed based upon the Marshall procedure. A summary of the mix design parameters is provided in Fig. 1.

Large-stone asphalt mixes have a number of advantages over conventional mixes, they include: (a) higher creep resistance, which translates into more rutting resistance, (b) higher resilient modulus, which contributes to higher structural capacity and load carrying capability, and exhibit lower asphalt demand due to lower surface area in large-stone gradation blends. The major disadvantage of large-stone mixes lies in their susceptibility to segregation, which can be brought out as a result of poor construction practices. It is important to note that high strength and rutting resistance of large-stone mixes are caused by the stone-to-stone contact phenomenon.

OVERVIEW OF PAVEMENT PERFORMANCE EVALUATION

Pavement deflection measurements were obtained using both a Road Rater (RR) and a JILS-20 Falling Weight Deflectometer (FWD). Deflections were taken at various locations throughout the test section. However, the only locations which will be discussed in this paper are the 30 and 120 m locations. These were the locations where, in addition to the nondestructive tests, destructive tests were also conducted; these tests included: in-place CBR,

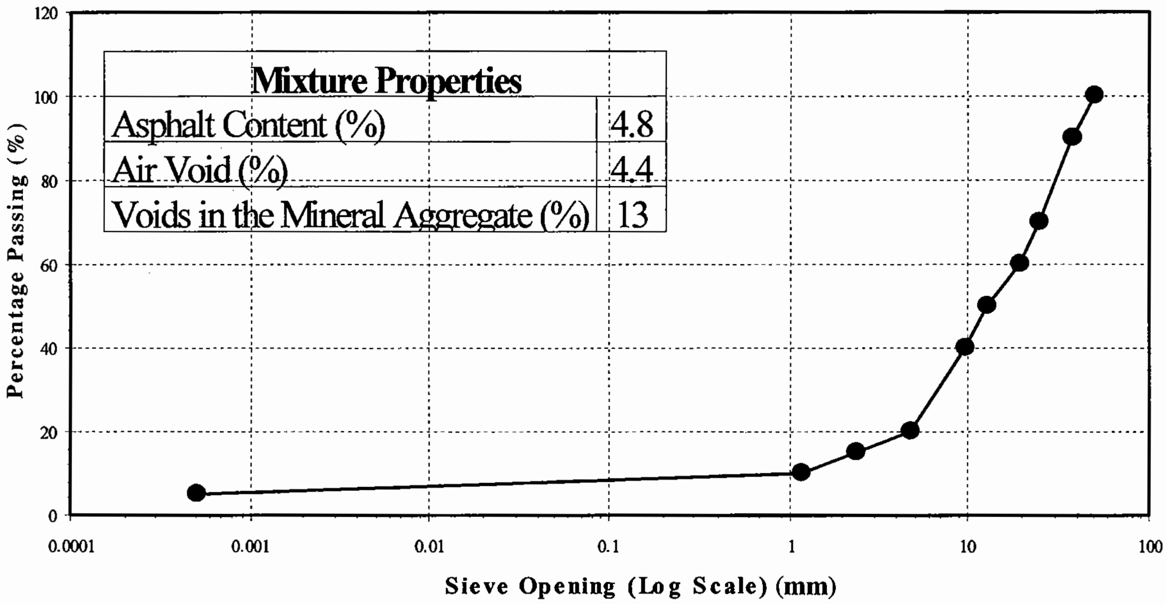

FIGURE 1 Large stone asphalt mixture properties, Mountain Parkway project.

resilient modulus on cored specimens, and soil index/classification tests.

Two sets of nondestructive tests (NDT) were conducted: one week after construction, and one year later. The FWD deflection bowls are shown in Fig. 2 for each location soon after the construction and one year later (each point on this plot represents an average of three data points). It may be seen from this figure that the bowls from the fresh pavement data have steeper slopes indicating a weaker asphalt layer as compared to the pavement after one year of service. This was also observed in the deflection bowls generated by the RR. This phenomenon is probably due to the stiffening of the asphalt layer caused as a result of asphalt aging (see Table I).

The RR data were analyzed by a modulus back-calculation routine, which was based upon deflections calculated with the Chevron N-Layer computer program (Michelow, 1963; Southgate *et al.*, 1987). Based upon a large number of projects in Kentucky, a data base of deflections was created using the Chevron program and then a Lagrangian interpolation technique was used to study the deflection bowls. The pavement layer moduli in the data base for the NDT model ranged from 345 to 13,790 MPa for the asphalt layer, and 14 to 690 MPa for the subgrade. The base layer modulus was calculated based upon the modulus of the confining layers above and below. A least squares fit of the bowl was then determined by varying the asphalt and subgrade moduli; a summary is given in Table I (each point on this table represents an average of three data points). The data which were analyzed had been adjusted to a reference temperature of 21°C based upon a methodology proposed by Dean *et al.* (1984). It may be seen from this table that the asphalt modulus average is approximately 7000 MPa, which corresponds to 3310 MPa at 0.5 Hz; Kentucky's flexible pavement thickness design methodology assumes an asphalt modulus of 3310 MPa at 0.5 Hz (Havens *et al.*, 1981; Southgate and Dean, 1988).

In-place CBR testing was also performed at the 120 m location. Results from these tests indicate an in-place subgrade CBR = 15, which would correspond to an estimated subgrade modulus of 155 MPa. Additionally, laboratory measurements of the DGA resilient modulus yielded 357 MPa. A composite modulus of the combined broken and seated concrete and the crushed stone dense graded aggregate (DGA) was calculated as a function of the confining layers of asphalt and subgrade. A modular ratio between the

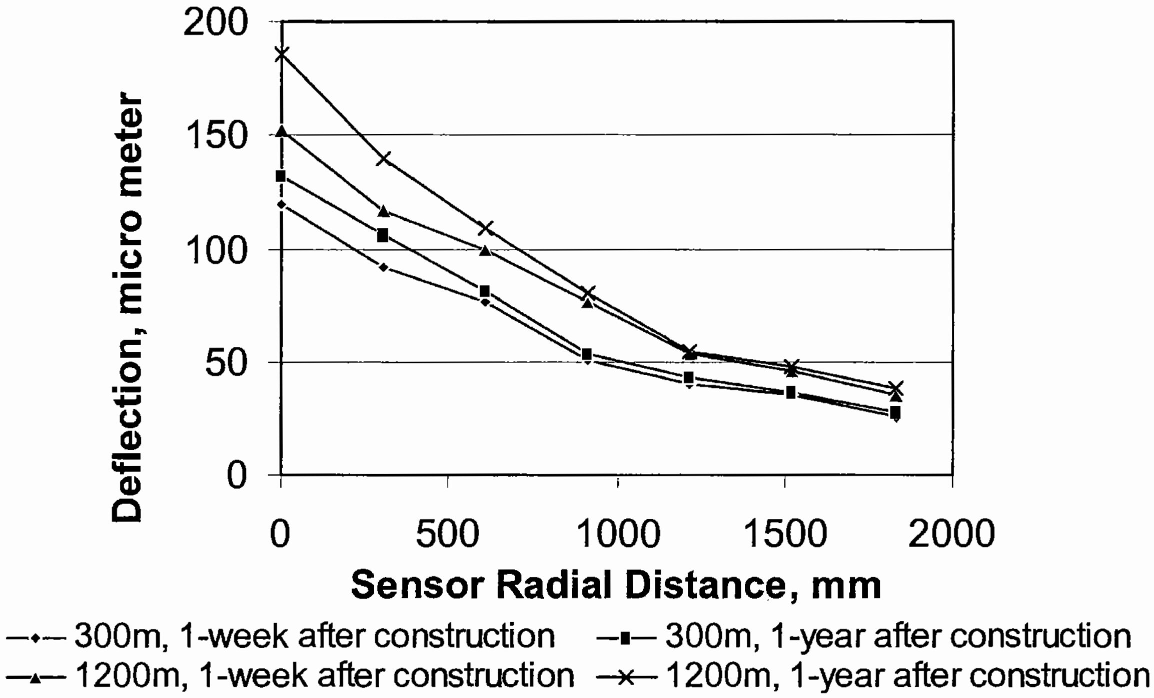

FIGURE 2 FWD deflection bowls.

subgrade and the DGA moduli was used to calculate the DGA modulus (Dean *et al.*, 1984). The ratio for the 30 and 120 m locations were calculated to be 2.3. Multiplying the subgrade modulus given in Table I by this ratio gives the corresponding DGA+PCC composite modulus in that table. The back calculated modulus values were then input into the elastic layer program ELSYM5 (Kopperman *et al.*, 1986) in order

TABLE I Back-calculated pavement moduli

Location	Modulus (MPa)		
	Asphalt	DGA+broken PCC	Subgrade
	One year after construction (30°C)		
30 m	5300, 6457, 8100 Mean: 6619 Std Dev: 1407	253, 331, 368 Mean: 317 Std Dev: 58.7	110, 144, 160 Mean: 138 Std Dev: 25.5
120 m	7900, 10680, 11000 Mean: 9860 Std Dev: 1705	207, 230, 322 Mean: 253 Std Dev: 60.9	90, 100, 140 Mean: 110 Std Dev: 26.5
	One week after construction (28°C)		
30 m	3502, 4430, 5100 Mean: 4344 Std Dev: 802	230, 299, 327 Mean: 285 Std Dev: 49.9	100, 130, 142 Mean: 124 Std Dev: 21.6
120 m	5600, 7100, 8399 Mean: 7033 Std Dev: 1400	345, 368, 473 Mean: 396 Std Dev: 68.2	150, 160, 206 Mean: 172 Std Dev: 29.9

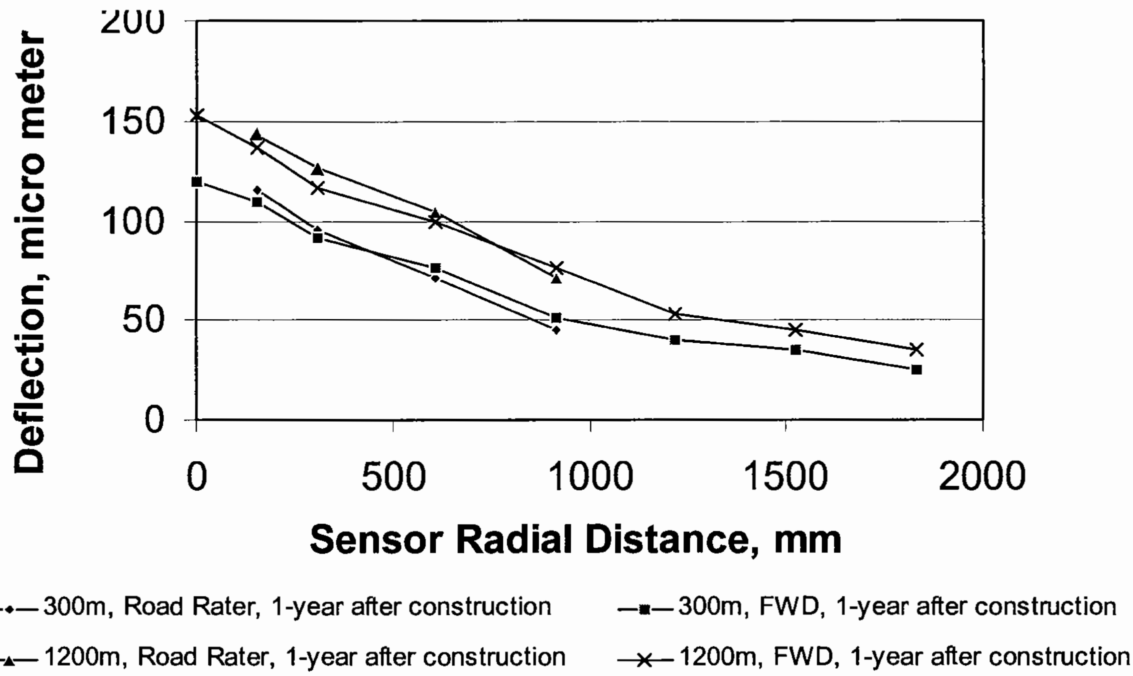

FIGURE 3 Comparing FWD and RR deflection bowls.

to simulate deflection bowls measured in the field; these calculated deflections were in agreement with the RR data.

Several factors contribute to variability in pavement NDT data. These include variability in material properties, subgrade conditions, depth to rock, moisture gradient, temperature gradient, and equipment/operator error. The variability of the data in Table I was analyzed using an analysis of variance technique. It was discovered that the location of testing did not contribute to any significant effects. However, the passage of time (approximately one year) resulted in changes in pavement layer moduli. In particular, the asphalt overlay exhibited a higher stiffness after one year of service. This phenomenon is due to aging/oxidation of asphalt. All statistical comparison were made at alpha = 5%.

Finally, a comparison was made between the FWD and the RR deflection bowls on the same pavement section under identical environmental conditions. This was done in order to verify the validity of our assumption that pavement response to different NDT devices can be correlated in some fashion, when proper adjustment factors are incorporated in the data analysis routine. The RR data were obtained under 544 kg at 25 Hz dynamic loading, while the FWD data were obtained under 4082 kg impact load. Figure 3 demonstrates that the RR and FWD deflection bowls are very similar; of course, the RR data were scaled linearly to represent the 4082 kg FWD loading.

STRUCTURAL ANALYSIS

The large-stone asphalt overlay of the Mountain Parkway's broken and seated PCCP was designed by the Kentucky Transportation Cabinet (1998). The overlay thickness design was based upon structural worth (modulus) of the broken and seated concrete over stone base over subgrade soil (Sharpe *et al.*, 1987). For the purpose of this paper, a generic layered-elastic structural analysis of the Mountain Parkway overlay pavement is presented. Simple Layered elastic tools

TABLE II Broken and seated PCCP structural coefficients.

PCCP fragment size (m)	Layer structural coefficient
0.3	0.25*
0.6	0.35†
0.9	0.45†

*Extrapolated by NAPA (Lukanen, 1987) based upon the 1986 AASHTO design guide.
†Reported by the AASHTO (1986).

were employed in this study so that our results would be compatible with the KY-DOT data bases, which are by-and-large based upon layered elastic computations. In this regard, AASHTO (1968) and NAPA (Lukanen, 1987) recommendations were followed.

The existing overlay structure of the Mountain Parkway project includes the following pavement layers:

surface wearing course: 40 mm
large-stone asphalt overlay: 150 mm (two 75 mm lifts)
broken and seated PCCP: 230 mm
DGA: 100 mm
silty clay subgrade: CBR = 15.

The layer modulus back-calculation routines yielded the following average pavement layer moduli (see Table I for more details):

broken and seated+DGA: 317 Mpa (composite of two layers)
Subgrade: 138 MPa

The NAPA has published a methodology (Lukanen, 1987) for thickness design of asphalt overlay on broken and seated PCCP; the methodology is based upon the AASHTO Design Guide (AASHTO, 1986). It includes a procedure by which modulus of the broken and seated PCCP can be estimated. This was developed based upon a methodology which was originally proposed by Odemark (1949). Using Odemark's concept of layer equivalency for the broken and seated PCCP and the underlaying DGA one can derive the following relationship:

$$a_{bs} = a_2(E_{bs}/E_2)^{1/3} \tag{1}$$

where a_{bs} is the structural layer coefficient of broken and seated PCCP, a_2 is the structural layer coefficient of DGA (approximately 0.14), E_{bs} is the modulus of broken and seated PCCP and E_2 is the modulus of DGA (357 MPa).

According to the AASHTO Design Guide (AASHTO, 1986) and NAPA (Lukanen, 1987), the numerical value of the structural coefficient of broken and seated PCCP is a function of the size of broken and seated concrete fragments, see Table II.

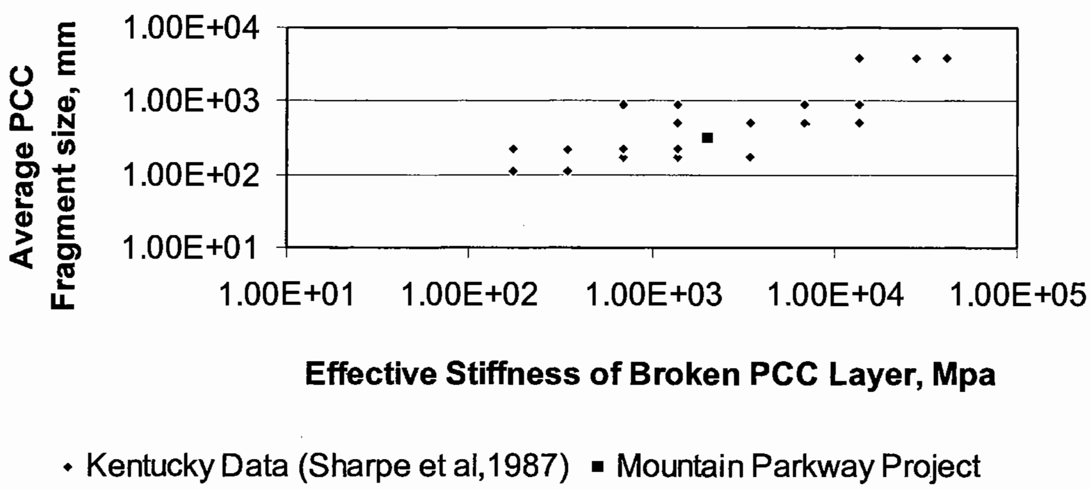

FIGURE 4 Comparing Kentucky data to the Mountain Parkway Project.

One can use the following parameters in Eq. (1), and solving it for the modulus of the broken and seated PCCP layer (E_{bs}) will yield:

$$a_{bs} = 0.25 \quad \text{(from Table II)}$$
$$a_2 = 0.14 \quad \text{(AASHTO, 1986)}$$
$$E_2 = 357\,\text{MPa}$$
$$0.25 = 0.14(E_{bs}/253)^{1/3}$$

therefore,

$$E_{bs} = 2033\,\text{MPa}.$$

Sharpe *et al.* (1987) reported a relatively large data base, which was generated from RR NDT studies and back-calculation of broken and seated PCCP moduli in Kentucky. Similar to NAPA (Lukanen, 1987), they reported that the modulus of broken and seated PCCP is a function of the size of broken PCCP fragments. In fact, the Mountain Parkway project with a nominal PCCP fragment size of 0.3 m, and estimated broken and seated PCCP modulus of 2033 MPa (determined from Eq. (1)) falls well within the Kentucky data base as previously reported by Sharpe *et al.* (1987). This is a very important observation about the estimated modulus of broken and seated PCCP at the Mountain Parkway project, which was determined from two independent sources, and the results are in close agreement (see Fig. 4).

To further investigate structural issues related to the overlay, recommended procedures by AASHTO (1986) and NAPA (Lukanen, 1987) were followed as shown below:

$$SN_{ol} = SN_y - 0.7(a_{bs}D_0 + SN_{rp}) \qquad (2)$$

where SN_{ol} is the structural number of asphalt overlay, SN_y is the required structural number for the entire pavement, a_{bs} is the layer coefficient for the broken and seated PCCP, D_0 is the thickness of broken and seated layer and SN_{rp} is the structural number of all remaining pavement layers above the subgrade, except for the existing PCCP.

The contribution of individual pavement layers to the entire overlay structure can be described as the following:

DGA layer: $SN_{rp} = 0.14 \times 100\,\text{mm} = 14$
broken and seated PCCP: $a_{bs}D_0 = 0.25 \times 230\,\text{mm} = 57.50$
asphalt overlay: $SN_{ol} = 0.44 \times 190\,\text{mm} = 83.60$
solving for SN_y in Eq. (2) will yield: $SN_y = 134$.

Finally, an estimate of the Mountain Parkway overlay pavement service life was determined in accordance with the information presented above and the AASHTO Design Guide (AASHTO, 1986). The following input parameters were incorporated in the analysis:

Standard Deviation $= 0.45$

Reliability $= 95\%$

$E_{subgrade} = 155\,\text{MPa}$

Delta psi $= (\text{psi})_{initial} - (\text{psi})_{terminal} = 4.5 - 2.5 = 2.0$

The estimated service life of this pavement, determined by using the AASHTO thickness design system, is approximately 18 million equivalent single axle load (ESAL) repetitions. Under normal situations, this represents a conservative design; however, the heavy truck traffic on the Mountain Parkway is such that it may use up the entire predicted life (18 million ESALs) in less than ten years.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the information presented in this paper the following conclusions are made:

1. Breaking and seating is very effective means of PCCP recycling. A simple structural design/evaluation case was presented.
2. FWD and RR devices can be effective tools for characterization of the modulus of the broken and seated PCCP.

The following recommendations are made for future work to be conducted in this area:

1. Standard NDT and back-calculation protocols, hardware tools (same NDT model and type versus different manufacturers and devices) as well as software tools, need to be developed so that data can be easily cross-referenced among different researchers and practitioners. A similar recommendation was made by Lytton *et al.* (1990)
2. Having accomplished the first recommendation, a large data set is needed for the development of mechanistically-based overlay thickness design methodologies.
3. Long-term performance evaluation of rehabilitated pavements should be monitored for the purpose of refinement of existing, and development of future design/analysis methodologies.

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