Effect of Base Type on the Transverse Cracking of Jointed Plain Concrete Pavements- A Study of LTPP SPS-2 Sections

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Abstract

A previous sensitivity study indicated a tendency of Pavement ME to predict more transverse cracking for Jointed Plain Concrete Pavements (JPCP) with aggregate bases than those with stabilized bases. Such a tendency contradicts the experience of some engineers who have successfully used aggregate bases to achieve robust and enduring JPCPs. The controversy triggered an in-depth investigation of the JPCP models by comparing them to an independent 3-D finite element model, which showed that Pavement ME under predicted the environmental stress for JPCPs with stabilized bases and also neglected the benefit of aggregate bases in accommodating the deformation of the surface layer and thus reducing its stress. However, this investigation was solely based on numerical modeling and not validated by any field evidences. This study further examines the issue based on an analysis of SPS-2 JPCP field experiments of the long-term pavement performance (LTPP) database. While the SPS-2 sections were designed to investigate the effect of various factors such as base type, thickness, climate, and so forth on the structural performance of the JPCP, this study is focused on the comparison of different base types, namely dense graded aggregate base (DGAB), permeable asphalt treated base (PATB) and lean concrete base (LCB). The interplay between base type and the other factors is also discussed.

Introduction

Engineers use bases in both rigid and flexible pavement designs for many reasons. First of all, a base adds structure to the system by providing support to the pavement that is more uniform, durable and stronger than natural or treated subgrade. As a result, it reduces the stress in the pavement and increases its service life. Other benefits of using a base include a more stable working platform for construction, and better drainage when a drainable base is used. For jointed plain concrete pavements (JPCPs), a base also contributes to the load transfer at joints and cracks, mitigating the development of faulting and cracking.

The two major types of base are aggregate base and stabilized base. The latter is usually manufactured by treating the aggregate base with an "engineering adhesive" such as cement or asphalt, resulting in a layer of continuum that can sustain significantly larger tensile and shear forces than the aggregate base alone. Depending on the design philosophy, a stabilized base is either bonded or unbonded to the surface course, while an aggregate base is always unbonded. In general, the cost of constructing a stabilized base

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is notably higher than that for an aggregate base. However, the additional cost is often justified by the belief that a stabilized base leads to superior performance over an aggregate base. Such a proposition seems to be reinforced by the new AASHTO Pavement ME Design Guide, as Mu et al., (2012) demonstrated that Pavement ME almost always predicted less transverse cracking for JPCPs with stabilized bases versus those with aggregate bases, especially when the bond stayed intact between the stabilized base and the concrete layer.

This finding implies a preference of Pavement ME for stabilized bases over aggregate bases for JPCP designs because transverse cracking is usually the dominant distress. Intrigued by the finding, Mu and Vandenbossche (unpublished manuscript, 2015) further investigated the Pavement ME model by comparing its stress prediction to the results from their 3-D finite element model. They discovered practical scenarios where the critical stress was greater for JPCP with stabilized bases than those with aggregate bases in their 3-D model, but were opposite in Pavement ME. These scenarios shared a common feature in that stress due to environmental loading was larger than stress due to traffic loading. The reason was that Pavement ME failed to give enough credit to aggregate bases for imposing less restraint to the curling/warping of concrete slabs and thus reducing the environmental stresses caused to a JPCP. This raised the question "Do field observations contradict the Pavement ME prediction with respect to the efficacy of different bases?" If so, is the theory of Mu and Vandenbossche valid in explaining the discrepancy? It is the objective of this paper to exploit the field experiments in Long term pavement performance (LTPP) database in an effort to answer the questions above.

LTPP Specific Pavement Studies-2 (SPS-2)

Specific Pavement Studies-2 (SPS-2) refers to one set of the field experiments in the Long term pavement performance (LTPP) program. To date, it is the most systematic and comprehensive experiment in studying the effect of various factors on the performance of JPCP. In total, 168 test sections were constructed in 14 states, with 12 sections per state. The experimental variables include climate, subgrade soil (fine or coarse depending on percent particles passing No. 200 sieve), drainage (with and without edge drains), concrete thickness, concrete flexural strength, slab width and base type. In addition, all the sections have doweled transverse joints spaced at 4.5 m and tied longitudinal joints. Traffic loading varies among the states, but all of them have been in excess of 200,000 equivalent single axle loads (ESALs) annually in the travel lane.

The cross section of the test sections is presented in Figure 1, where for this paper they are classified into three main types in order to compare the bases: dense graded aggregate base (DGAB), lean concrete base (LCB) and permeable asphalt treated base (PATB). It is noteworthy that the PATB is actually a combination of a 100-mm asphalt stabilized base and a 100-mm aggregate base. Therefore, it cannot be categorized as a stabilized base as its name suggests. Instead, LCB epitomizes the characteristic of a stabilized base, and DGAB is a good representative of an aggregate base.

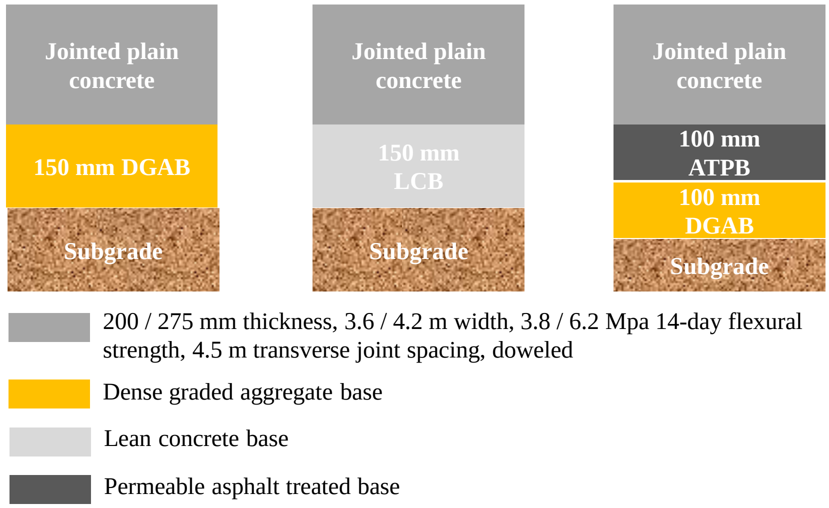

Figure 1. Cross section of the SPS-2 test sections

Table 1 summarizes the test sections in each state. All the sections were constructed in the 1990s, so ideally there should have been 15-25 years of distress data available. However, as shown at the bottom row of Table 1, data was screened in some states resulting in only a few years of data. The screening was done to address the following issues. First, some sections have been decommissioned or replaced. Since all the sections were constructed on in-service highways, excessive distresses were intolerable. Second, major interventions such as patching/grinding altered the trajectory of crack occurrence and growth. Therefore, the comparison might be biased if the sections that received major repairs were not distinguished from the rest. Finally, some of the interventions also contaminated the cracking data because afterwards, the cracks either disappeared or became less discernible. It is not uncommon to find a sudden drop of crack counts in the database that was related to rehabilitation events.

The percentages in Table 1 are "slabs cracked" normalized to traffic in terms of million equivalent single axle loads (ESALs). In other words, if a section shows 1% in Table 1, it would have 10% of its slabs cracked at 10 million ESALs. Such normalization had to be done to allow for comparison between states that experienced different traffic levels. Assuming 15% at 20 million ESALs as the failure threshold for cracking, the cells of less than 0.75% in Table 1 are considered "lightly cracked" and colored green in Table 1. In the same manner, the cells of 1%-2% in Table 1 are defined as "medium cracked" and the ones of more than 2% are defined as "heavily cracked".

While the authors believe such a parameter effectively quantifies the degree of damage received by the whole test section, it has its limitations. First, it distributes the cracks evenly among the slabs, even though it is possible that some slabs possess multiple cracks while others have none. The other limitation is that cracks of all severities were included in the table. Generally speaking, cracks of low severity do not impose as much of a serviceability loss to the pavement as cracks of high and medium severity do. Therefore, it should be noted that low severity cracks dominated six of the LCB sections and four of the DGAB sections, while only one section of the PATB sections.

Table 1. Rate of transverse cracking relative to traffic for all the SPS-2 sections

JPCP				Climate, Subgrade															
				Wet									Dry						
				Freeze						NonFreeze			Freeze				NonFreeze		
				Fine					Coarse	Fine		Coarse	Fine			Coarse	Fine	Coarse	
Thickness, mm (in)	**Flexural, MPa (psi)**	**Lane width, m (ft)**	**Base Type**	**IA**	**KS**	**MI**	**ND**	**OH**	**WI**	**AR**	**NC**	**DE**	**CO**	**NV**	**WA**			**AZ**	**CA**
200 (8)	**3.8 (550)**	**3.6 (12)**	**PATB**		**0%**			**1%**			**0%**	**0%**			**0%**				**0%**
			DGAB		**2%**			**8%**			**0%**	**0%**			**1%**				**3%**
			LCB		**0%**			**22%**			**5%**	**4%**			**2%**				**3%**
		4.2 (14)	**PATB**	**0%**		**0%**	**0%**		**0%**	**0%**			**0%**	**18%**				**0%**	
			DGAB	**0%**		**0%**	**0%**		**0%**	**1%**			**0%**	**25%**				**0%**	
			LCB	**0%**		**0%**	**2%**		**0%**	**0%**			**1%**	**48%**				**4%**	
	6.2 (900)	**3.6 (12)**	**PATB**	**0%**		**0%**	**0%**		**0%**	**0%**			**0%**	**0%**				**1%**	
			DGAB	**0.7%**		**0%**	**0%**		**0%**	**0%**			**0%**	**15%**				**0%**	
			LCB	**1%**		**9%**	**0%**		**0%**	**3%**			**0%**	**108%**				**2%**	
		4.2 (14)	**PATB**		**0%**			**8%**			**0%**	**0%**			**0%**				**0%**
			DGAB		**2%**			**13%**			**0%**	**0%**			**3%**				**3%**
			LCB		**0%**			**15%**			**0%**	**0%**			**3%**				**2%**
275 (11)	**3.8 (550)**	**3.6 (12)**	**PATB**	**0%**		**0%**	**0%**		**0%**	**0%**			**0.6%**	**25%**				**0%**	
			DGAB	**0%**		**0%**	**0%**		**0%**	**0%**			**0%**	*****				**0%**	
			LCB	**0%**		**0%**	**0%**		**0%**	**0%**			**0%**	**25%**				**3%**	
		4.2 (14)	**PATB**		**0%**			**0%**			**0%**	**0%**			**0%**				**0%**
			DGAB		**0%**			**0%**			**0%**	**0%**			**0%**				**0%**
			LCB		**0%**			**0%**			**0%**	**0%**			**4%**				**0%**
	6.2 (900)	**3.6 (12)**	**PATB**		**0%**			**5%**			**0%**	**0%**			**0%**				**0%**
			DGAB		**0%**			**12%**			**0%**	**0%**			**0%**				**0%**
			LCB		**0%**			**1%**			**0%**	**0%**			**0%**				**0.6%**
		4.2 (14)	**PATB**	**0%**		**0%**	**0%**		**0%**	**0%**			**0%**	**2%**				**0%**	
			DGAB	**0%**		**0%**	**0%**		**0%**	**0%**			**1%**	**82%**				**0%**	
			LCB	**0%**		**0%**	**0%**		**0%**	**0%**			**0%**	**10%**				**0%**	
Age, year				20	20	2	18	10	15	12	9	16	21	8	19			21	14
ESALs, million				9	16	4	9	6	6	41	13	7	30	6	7			26	36

Note 1.* Severe cracking occurred soon after paving and thus the section was removed from the experiment

2. Green cells: Lightly cracked; Yellow cells: Medium cracked; Pale Red cells: Heavily cracked.
3. The underlined cells are mainly dominated by cracks of low severity

Deviation of Construction from Experimental Design

The construction of SPS-2 test sections was required to meet the LTPP experimental design specifications / tolerances. Unfortunately, some test sections still deviated from the experimental design due to various reasons. In this section, some of the key deviations that might be responsible for the cracking pattern in Table 1 are summarized, after synthesizing the construction details reported by Chatti et al. (2005).

The as-built thickness of the sections was determined based on coring data, whose results are presented in Figure 2. Among the sections with a target thickness of 200 mm, the PATB sections were built on average 12 mm (1/2 inch) thicker than targeted, while the DGAB and LCB sections were on average 5-8 mm (1/4 inch) thicker than targeted. On the other hand, among the sections with a target thickness of 275 mm, most of them were constructed within the thickness tolerance. However, there were a few 200-mm sections constructed significantly thinner (12 mm or 1/2 in) than the target thickness, which made them more prone to cracking. These sections include:

- 200mm-6.2MPa-LCB section in MI – constructed 178 mm thick,
- 200mm-6.2MPa-DGAB section in KS – constructed 185 mm thick, and
- 200mm-3.8MPa-DGAB section in AR – constructed 185 mm thick.

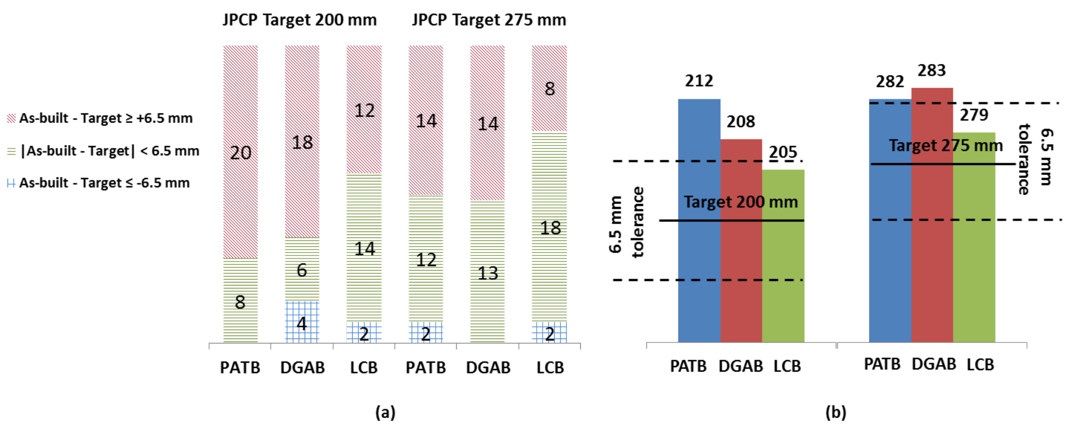

Figure 2. (a) number of sections deviated from target thickness (b) average as-built thickness for each base type

Similarly, the paving mix was significantly altered from the design mix in some states. In DE, the mix with a target flexural strength of 3.8 MPa was replaced with the DOT's 4.5 MPa mix due to the concern of extensive cracking. In NV, the 3.8 MPa and 6.2 MPa mixes were substituted by 3.3 MPa and 5.2 MPa mixes due to the difficulty of achieving the specified design strength with local materials. Furthermore, the 14-day flexural strength was measured at 12 of the 14 states, which was compared with the target strength in Table 2. On average, the actual flexural strength is higher than the target value for the 3.8 MPa mixes, while it is lower than the target for the 6.2 MPa mixes. Using 15% deviation (approximately one standard deviation) as the threshold, the gray cells in Table 2 indicate significant deviation from the average of the states.

In addition, drying shrinkage cracks were found soon after paving at two of the NV sections (275mm thick-6.2MPa strength-4.2m wide-DGAB base and 275mm-6.2MPa-3.6m-LCB). Interestingly, these two sections also show excessive cracking in Table 1. In WA, the 200mm-6.2MPa-4.2m-LCB section also had shrinkage cracks throughout the section right following paving. Finally, the CA sections used 3.9 m (13 ft) instead of 4.2 (14ft) for their widened lanes.

Table 2. Deviation of actual flexural strength from target value.

	Target, MPa	Actual, MPa		Actual/ Target	Target, MPa	Actual, MPa		Actual/ Target
		Ave.	Stdev.			Ave.	Stdev.	
IA	3.8	3.2	0.2	85%	6.2	5.2	0.3	84%
KS	3.8	4.2	0.3	111%	6.2	5.8	0.3	94%
MI	3.8	4.3	0.2	112%	6.2	6.7	0.0	108%
OH	3.8	4.7	0.4	124%	6.2	4.2	1.1	68%
WI	3.8	4.4	0.2	115%	6.2	6.1	0.4	98%
AR	3.8	3.8	0.2	99%	6.2	4.6	1.6	74%
NC	3.8	4.8	0.4	126%	6.2	5.9	0.0	95%
DE	3.8	4.5	0.7	119%	6.2	5.2	1.0	84%
CO	3.8	3.6	0.3	95%	6.2	6.2	0.4	100%
NV	3.8	3.4	0.2	89%	6.2	5.0	0.6	81%
WA	3.8	3.3	0.4	88%	6.2	5.7	0.2	92%
AZ	3.8	3.9	0.1	104%	6.2	5.8	0.4	93%
Mean among States	3.8	4.0	-	106%	6.2	5.5	-	89%

Note: Gray cells indicate outliers that deviates 15% or more from group average

Statistical Comparison of Stabilized and Aggregate Bases

Because the transverse cracking of JPCP is sensitive to pavement thickness and concrete strength; any deviation of them could easily overshadow the effect of base type in the experiment. Therefore, the sections in Table 1 need to be screened so that only those with small deviation from each other (or from the experimental design specification) are kept. Sections with a target thickness of 200 mm were removed if their thickness deviated from the target by more than 12.5 mm (1/2 inch). A higher deviation of 25 mm (1 inch) was allowed for sections with a target thickness of 275 mm, because they were thick enough to be unaffected by thickness variance less than this. Sections with anomalous flexural strength (the gray cells in Table 2) were also removed from the analysis, together with the NV and DE sections that did not meet to the LTPP construction specifications. In addition, the sections with excessive early-age shrinkage cracks were excluded because the cracking was due to material or construction problems – not fatigue damage.

The revised new matrix contains 28 section sets, instead of the 56 sets in Table 1. Each set consists of three sections that share all the features but the base type. Table 3 compares the different bases in terms of their effect on transverse cracking, which was done by first separating the 28 sets into high and low value groups for each feature and then computing the mean rate of transverse cracks for each base type within each group. Additionally, a paired t-test was conducted within each group with a null hypothesis that the two types of base in question result in the same level of transverse cracking. The confidence level for the tests is also presented in Table 3. Considering a confidence level of 5%, there seems to be no significant difference between the three types of base, except for sections that were thin or had high concrete strength. In these exceptions, the PATB sections have less cracking than the LCB or DGAB sections. However, one should be reminded that PATB as shown in Figure 1 is actually a hybrid of asphalt and aggregate base, and the benefits might arise from the pavement having both stabilize and aggregate bases. Another important discovery is that although the DGAB sections nearly always have less cracking than the LCB sections, the significance of the difference varies between the two groups of the same feature. One can observe that the difference is more notable for sections with 3.8 MPa concrete, wider (4.2 m) slabs, Dry-NonFreeze climate, coarse subgrade, and light traffic.

Table 3. Statistical comparison of the effect of different bases on transverse cracking.

Feature		Average cracks/million EASLs			p-value		Number of observations
		DGAB	PATB	LCB	PATB vs. DGAB	LCB vs. DGAB	
Thickness	200 mm	0.9%	0.0%	1.3%	0.030	0.205	9
	275 mm	0.1%	0.0%	0.3%	0.218	0.146	19
Strength	3.8 MPa	0.3%	0.0%	0.9%	0.124	0.076	14
	6.2 MPa	0.4%	0.0%	0.3%	0.049	0.425	14
Width	3.6 m	0.4%	0.0%	0.6%	0.082	0.201	15
	4.2 m	0.3%	0.0%	0.7%	0.082	0.167	13
Climate	Dry	0.7%	0.1%	1.3%	0.073	0.101	10
	Wet	0.2%	0.0%	0.2%	0.067	0.358	18
	Freeze	0.2%	0.0%	0.2%	0.068	0.484	18
	Nonfreeze	0.5%	0.0%	1.3%	0.084	0.054	10
Subgrade	Coarse	0.5%	0.0%	1.1%	0.083	0.054	12
	Fine	0.3%	0.0%	0.3%	0.068	0.485	16
Traffic	Heavy (> 2 M/yr)	0.8%	0.0%	0.9%	0.086	0.253	7
	Light	0.2%	0.0%	0.5%	0.068	0.118	21
Overall		0.3%	0.0%	0.6%	0.022	0.094	28

Note: Gray cells indicate difference between the two bases at 5% confidence level

Recalling the hypothesis of Mu and Vandenbossche (unpublished manuscript, 2015) that smaller stresses will develop in JPCP with aggregate base when the environmental loading is more predominant than traffic loading, the above discovery seems to confer their hypothesis for the following reasons. First, the slab warping due to differential drying shrinkage is potentially larger for the 3.8 MPa concrete mix than the 6.2 MPa concrete mix. The ultimate drying shrinkage for the two concrete mixes can be predicted using the model implemented by the Pavement ME guide as a function of cement type, cement content, water-cement ratio, compressive strength and curing methods. Table 5 shows the calculated ultimate drying shrinkage based on the average inputs from the Table 4. The transient shrinkage at the top of the concrete slabs was assumed to be 50% of the ultimate shrinkage recognizing the rewetting process. On the other hand, research has shown that concrete slabs tend to remain saturated at the bottom. Therefore, negligible shrinkage could be assumed at the bottom of the slabs. As different drying shrinkages developed between the top and bottom of the slabs, the slabs would warp upward and the magnitude should be proportional to transient shrinkage values in Table 4. As a result, a greater warping can be expected for slabs with the 3.8 MPa concrete mix than those with the 6.2 MPa concrete mix. Other factors will also influence the degree of slab warping. For example, warping will be aggravated in a dry climatic region where higher drying shrinkage can be expected at the top of the slabs. Large slabs will also warp more than smaller slabs.

Table 4. Estimated drying shrinkage for the concrete mixes used in SPS-2 sections.

Concrete mix	Cement, kg/m3	w/c ratio	28-day fc', MPa	Curing	50% of ultimate shrinkage, microstrain	
					Type I cement	Type II cement
3.8 MPa	272	0.65	28	Compound	390	330
6.2 MPa	444	0.32	53	Compound	280	240

Finally, freezing weather and subgrade type might influence cracking through the deformability of the base as well as the subgrade. Granular base as well as subgrade can readily conform to the deformation of pavements resulting in less environmental stress. However, when frozen, this benefit may be significantly curtailed.

Curvature and Stress Analysis for Arizona Sections

In this section, the hypothesis by Mu and Vandenbossche was further examined by introducing other evidence, namely the curvature of the slabs. If the benefit of an aggregate base is that it has less restraint to the warping of slabs than a stabilized base, we can naturally infer that more curvature should be observed for concrete slabs on an aggregate base than those on a stabilized base. Figure 3 presents the curvature for the SPS-2 sections in AZ. The data was based on the research of Karamihas and Senn (2012) who calculated the curvature of the AZ sections from the measured profiles of the sections. The curvature was defined in terms of pseudo strain gradient (PSG) that is the gross strain gradient required to deform a slab into the measured shape from a flat baseline.

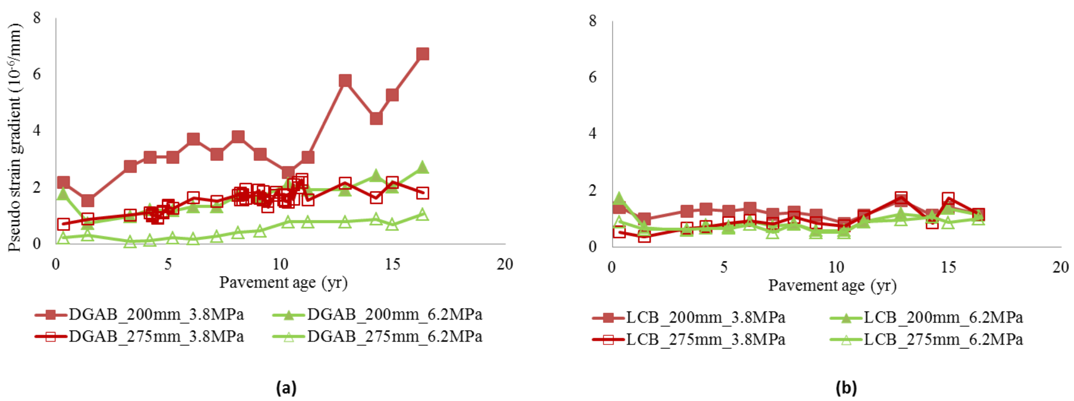

Figure 3. Curvature of AZ sections based on measured profiles (a) DGAB and (b) LCB.

In Figure 3, each data point corresponds to an incidence of profile measurement and a positive PSG denotes an upward curvature of the slab. Despite the local fluctuation that is a function of the transient temperature gradients at the time of profile measurement, there is an overall trend of the gradual growth of PSG especially for the slabs with DGAB. This trend is a powerful indication that the long-term development of differential drying shrinkage is exerting a significant influence on the shape of the slabs. Furthermore, it is also evident that a larger PSG relates to greater shrinkage (3.8 MPa mix) and thinner slabs. For slabs with LCB, the impact of slab thickness on curvature is almost negligible. Nevertheless, it is still possible to see that greater shrinkage occurs with a larger PSG.

In Figure 4, the curvature of the AZ slabs was calculated in a 3-D finite element model developed using EverFE (Davids, 2009). The model was of six slabs with doweled transverse joints and tied longitudinal joints. No traffic load was applied, but a temperature difference (slab top - bottom) of -6.7 °C and -13.4 °C was applied for the 6.2 MPa and 3.8 MPa concrete mixes, respectively. The temperature difference was created to imitate the effect of differential drying shrinkage. The modulus of elasticity for the concrete was let to vary with the flexural strength, while the modulus of the DGAB and LCB was assumed to be constant, 0.3 and 13.8 GPa, respectively. Modulus of subgrade reaction was 28 kPa/mm. Note that the strain gradient in Figure 4 is not strictly identical to PSG. Rather, it was calculated by dividing the differential deflection between slab corner and center with half of the diagonal length of the slab as well as its thickness. Nevertheless, both parameters are meant to evaluate slab curvature and of same units and thus they are comparable especially in the context of discovering trends.

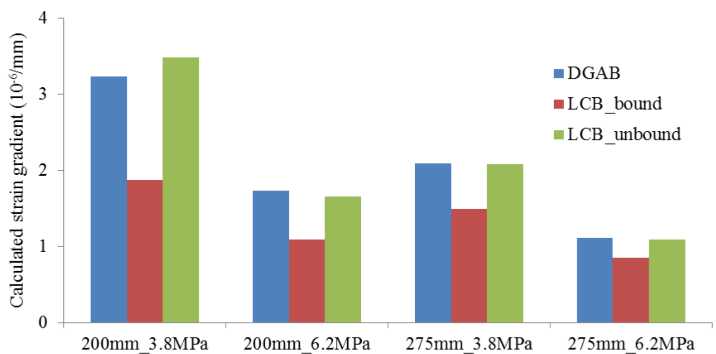

Figure 4. Estimated curvature of AZ sections based on finite element model

Figure 5 summarizes the mean of each series of PSG in Figure 3. Again, greater curvature is associated with DGAB than with LCB, except for the 275mm_6.2MPa sections for which PSG is essentially the same between the two. Unsurprisingly, the curvature for PATB is in between the other two bases, considering that it is, at least in the context of SPS-2 experiment, a hybrid of stabilized and aggregate bases.

The comparison of Figure 4 to Figure 5 suggests that the finite element model yielded curvature predictions in good agreement with those based on measured profiles. However, if the interface bond for LCB becomes ineffective, the slab curvature will get larger, even larger than that for DGAB. This is a limitation of the finite element model and is unexpected in reality, because a warped slab should relax more when it rests on a stiffer base. In the model, the LCB base was designated approximately 50 times stiffer than the DGAB. When assuming it was not bonded to the pavement, we observed a gap between the two layers, which was approximately 10% larger in area than its counterpart for DGAB. However, the finite element model does not consider the effect of relaxation.

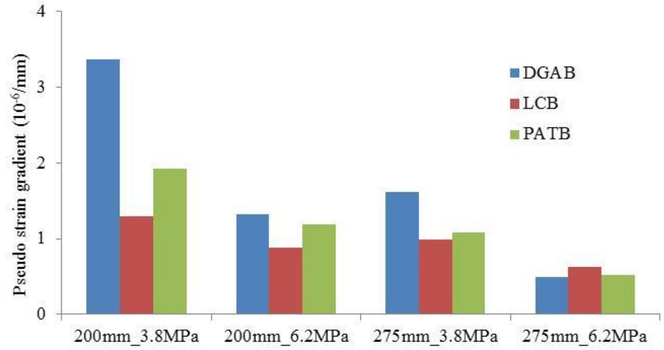

Figure 5. Mean curvature of AZ sections based on measured profiles

Next, the finite element model that was validated by its reasonable deformation prediction was used to estimate the critical stress of the slabs under various scenarios. In Figure 6, it summarizes the critical stress when there is no axle loading. The stress for bonded LCB almost doubles that for unbonded LCB or DGAB. When two single axles were applied each acting at one end of the slab, the critical stress increased significantly for slabs with unbonded LCB or DGAB, but not much for those with bonded LCB as shown in Figure 7. In other words, the slabs with a bonded LCB were constantly exposed to a high stress level, while those with unbonded LCB or DGAB were only exposed (experiencing high stresses) when there was heavy vehicles passing. Therefore, the accumulated damage should be lower for DGAB and unbonded LCB than bonded LCB.

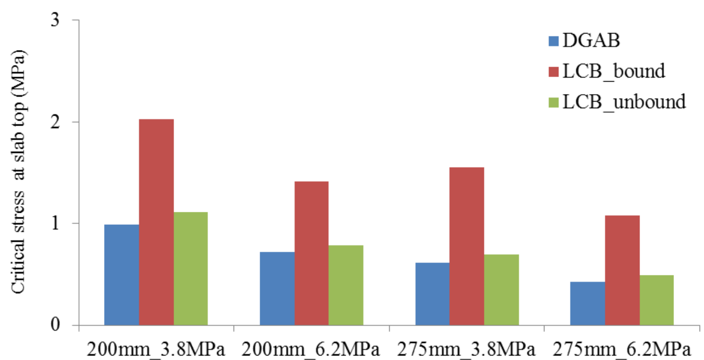

Figure 6. Estimated critical stress for AZ sections subjected to no loads

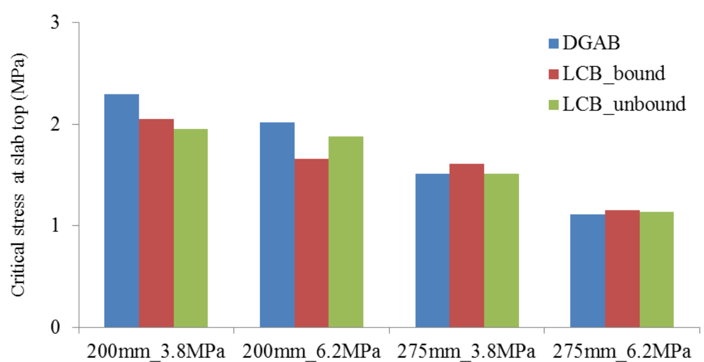

Figure 7. Estimated critical stress for AZ sections subjected to axle loads

Pavement ME 2.2 was then employed to predict the transverse cracking for the AZ sections. The nominal slab thickness and width were used because they did not deviate significantly form the as-built values for the AZ sections, while the actual concrete properties in Table 5 were used. For the sections with LCB, two cases were run one assuming full friction (bonded) at the concrete-base interface over the analysis period while the other assuming no friction at all (unbonded). Transverse cracking was estimated at Year 21 as summarized in Table 5 so that it could be compared with the observations from Table 1. While Pavement ME was able to provide fair prediction of cracking to most of the sections with DGAB and PATB, it significantly underestimated the cracking for sections with LCB. This echoes the finding of Mu et al. (2012) that Pavement ME tends to prefer stabilized bases over granular base especially when a stabilized base remains bonded to JPCP.

Table 5. Comparison of observed cracking to Pavement ME predictions for AZ sections .

Thickness, mm	Base	Width, m	Flexural strength, MPa	Trans cracking @ 21 yrs		
				Pavement ME, 50% reliab.		Observed
				Full Friction	No Friction	
200	DGAB	4.2	3.8	2.49%	0.00%	0%
	LCB	4.2	3.8	0.00%	1.36%	94%
	PATB	4.2	3.8	0.24%	2.49%	0%
200	DGAB	3.6	6.2	0.06%	0.00%	3%
	LCB	3.6	6.2	0.00%	0.10%	65%
	PATB	3.6	6.2	0.02%	0.05%	32%
275	DGAB	3.6	3.8	0.03%	0.00%	0%
	LCB	3.6	3.8	0.00%	0.01%	68%
	PATB	3.6	3.8	0.01%	0.04%	0%
275	DGAB	4.2	6.2	0.00%	0.00%	0%
	LCB	4.2	6.2	0.00%	0.00%	0%
	PATB	4.2	6.2	0.00%	0.00%	0%

Conclusion

This study investigated the effect of base type on the transverse cracking of JPCP using data from the LTPP SPS-2 experiment. The test sections were first screened in order to eliminate those with suspect cracking data, significant deviations from the proposed experimental designs, or contaminated by material related cracks. Overall, the censored database showed no statistically significant difference at 5% significance level between the three base types, namely dense graded aggregate base, lean concrete base, and permeable asphalt treated base. Under certain scenarios, it was found that slabs with aggregate base exhibited less cracking than those with lean concrete base, which is contrary to the Pavement ME's supposition that stabilized bases tend to result in less cracking than aggregate bases especially when bonded to JPCP.

The cracking data further indicated that the favorable scenarios for aggregate bases were when the environmental loading was predominant relative to traffic loading. To validate this finding, the SPS-2 test sections in Arizona were studied in terms of the development of their curvatures. As a result, the effect of significant slab warping due to differential drying shrinkage was identified. A 3-D finite element model that considered the warping mechanism was then developed, which yielded reasonable estimation of slab curvatures in good agreement with the observed values. Next, this finite element model was used to estimate the critical stress of the slabs. The results showed that slabs with a bonded lean concrete base were exposed to high stress level even when there was no axle load. On the other hand, slabs with an aggregate base were only subjected to high stress level at the passing of heavy axles. Therefore, the slabs with a bonded lean concrete base would accumulate more fatigue damage. This has the potential to explain the more observed cracking for JPCPs with stabilized bases in the LTPP database and vividly contrasts with the Pavement ME's prediction where the sections with a bonded stabilized base would always have less cracking than those with an aggregate base.

The readers should note that the intent of this paper is to discuss the validity of the modeling of different base types and its conclusion should be interpreted within the context of transverse cracking. The selection of base type in pavement design is a complex process compromising various considerations. Even within the context of transverse cracking, there is no absolute superiority for any base type. Rather, it depends on the relative dominance between traffic loading and environmental loading on the designed pavement.

References

ARA, Inc., ERES Division (2004). *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structure*, Final Report NCHRP 1-37A, Transportation Research Board of the National Academies, Washington, D.C.

Chatti, K., Buch, N. Haider, S. W., Pulipaka, A. S., Lyles, R. W., Gilliland, D., and Desaraju, P.(2005). *LTPP Data Analysis: Influence of Design and Construction Features on the Response and Performance of New Flexible and Rigid Pavements*. NCHRP Web-Only Document 74 (Project 20-50 [10/16]).

Davids, B. *Software for the 3D Finite Element Analysis of Jointed Plain Concrete Pavements,* Civil Engineering Department, University of Maine, http://www.civil.umaine.edu/EverFE/. (Apr. 27, 2009)

Karamihas, S. M. and Senn, K. (2012). *Curl and Warp Analysis of the LTPP SPS-2 Site in Arizona*. FHWA Report, FHWA-HRT-12-068.

Mu, F., Vandenbossche J. M., Gatti K. A., and Sherwood J. (2012). "An Evaluation of JPCP Faulting and Transverse Cracking Models of the Mechanistic-Empirical Pavement Design Guide". *Road Materials and Pavement Design*, 13(1), 128-141.