

Prediction of Top-Down Transverse Cracking in Jointed Plain Concrete Pavement

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Abstract

This paper describes the findings of a recently completed investigation by the University of Michigan to identify the causes for premature, mid-slab cracking of jointed plain concrete pavement (JPCP) in Michigan. The study found several factors are working interactively to induce slab cracking. The major factor is the support characteristics of the unbound open-graded aggregate base. Deflection testing showed a permanent loss of slab contact with the base (void) exists near the doweled joint. Time history analysis of the deflection data showed that the dowels are very likely loose. Loss of slab-base contact without pumping suggests that post-compaction under heavy vehicle loading or particle degradation is occurring. In addition, the study found that the load transfer efficiency (LTE) across joints has declined since construction. Finite element analysis (ABAQUS) was used to evaluate the extent and significance of these factors on tensile stress and fatigue cracking. Fatigue failure culminates from excessive tensile stress at mid-slab at the slab's topside outer edge. The slab stress predictions correlate well with the deflection-based fatigue model by Wu et al. (1993) suggesting that a threshold corner deflection criteria may be used to determine when loss of slab support is critical.

Introduction

This paper represents a synopsis of continuing research conducted by the Michigan DOT to find the causes for mid-slab cracking of several projects constructed with jointed plain concrete pavement (JPCP). In 1994 the Michigan DOT began constructing JPCP as a means to reduce troublesome transverse cracking in jointed reinforced concrete pavements (JRCP) and the associated high maintenance costs. At that time JRCP was Michigan's long-standing standard pavement. The previous performance of JRCP ranged from poor to excellent with many examples of projects showing only occasional slab cracking after decades of service. During the next five years fifteen JPCP projects were constructed, totaling 282 lane-miles. Five of these projects quickly developed mid-slab transverse cracking three to five years after construction. Three of the cracked projects involve Michigan interstate highways on I-94 and I-96. The rapid development of cracking is unacceptable because JPCP is designed not to crack, and the maintenance savings are lost. Michigan is not the only state to report mid-slab cracking of their JPCP designs. The Pennsylvania Department of Transportation (PennDOT) has probably received

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the most national attention regarding mid-slab cracking on their portion of I-80 near the Ohio border (Beckemeyer et al., 2002).

A preliminary investigation by the Michigan DOT revealed that the mid-slab, transverse cracking is occurring top-down from high tensile stresses at the slab's outer edge. The stress reaches maximum when the slab is subjected to a negative temperature gradient and simultaneous loading of both joints from a fully loaded truck. In addition, a built-in negative gradient from construction increases slab curvature and a subsequent loss of contact with the base that can extend 1.5m (5 ft) from the joint (Hansen et al., 2002). The investigation found the cracked projects to have high joint deflections from the loss of base contact with the slab. These factors have been identified by others (Khazanovich et al., 2000, Beckemeyer et al., 2002, Hiller and Roessler, 2002). The fatigue cracking process begins as micro-cracking at the top of the slab at the outer edge, which coalesce and propagate across and downward through the slab.

Research Significance

The Michigan research has identified a link between the slab's base support condition and the initiation of mid-slab transverse cracking. Without the benefit of steel mesh to maintain aggregate interlock for adequate load transfer, JPCP cracks soon fault and spall. They are difficult to effectively repair, as the "window of opportunity" quickly shuts. The findings can significantly improve JPCP design methods and construction procedures.

Field Study

Field testing was conducted in 2002 on two JPCP (I-94 and I-96) projects that have developed transverse cracking. The testing included distress surveys, joints deflection measurements using MDOT's Falling Weight Deflectometer (FWD), and joint fault measurements using the Georgia Faultmeter. In addition to the two JPCP projects, two JRCP projects (on I-94 and I-96) with excellent performance were also investigated to compare with the troublesome JPCP projects.

The two JPCP projects with transverse cracking share some similar features. The project I-94 was paved during the summer of 1998 with a slab length of 4.57m (15ft). Transverse cracking first became known during the summer of 2002, and the numerous cracked slabs would soon require some type of maintenance. The base type was unbounded open-graded drainage course (OGDC) limestone with MDOT's 4G gradation. This project was plagued with base and subbase construction problems, with the construction record showing that the base and subbase failed to meet gradation limits for part of the project, which subsequently has developed rapid mid-slab cracking. The project I-96 was constructed in 1997 with a slab length of 4.88m (16ft). Ninety slabs with mid-panel transverse cracks were detected as of August 2000. It contains two MDOT test sections of different aggregate for base course, slag and limestone, both unbound 4G (350AA) gradation, and the rest of the projects have 3G slag base and 3G limestone base.

Cores obtained for the EB I-96 JPCP project showed that the slab failure mode was top-down cracking, with fatigue failure starting along the outer edge near midslab and propagating inward and downward towards the base. This suggests that the maximum tensile stresses are located on the top of the slab along the outer edge near midslab, and that they can exceed 45% of the in-place flexural strength leading to rapid fatigue failure. The design flexure strength for the concrete is 4.5 MPa (650 psi). The results of this investigation have been presented by Hansen et al. (2002).

The pavement mix design was a Michigan standard 335 kg/m³ (5.5 sack) type IA cement with 20% class C fly ash replacement, with 1420 kg/m³ (1497 lb/yd³) Michigan 6 series coarse aggregate (air-cooled, blast furnace slag), and 1481 kg/m³ (1561 lb/yd³) class 2NS fine aggregate.

A unique feature of the FWD testing in this study is that time history results were obtained for selected drop load locations. Time history testing records the simultaneous load-deflection history for a given test, to determine the deflection versus load results up to peak load. It provides detailed information of the joint response to loading and can detect if slip between the two joint faces and dowel looseness has developed. These results were obtained between 0 and 40 kN (9000 lb).

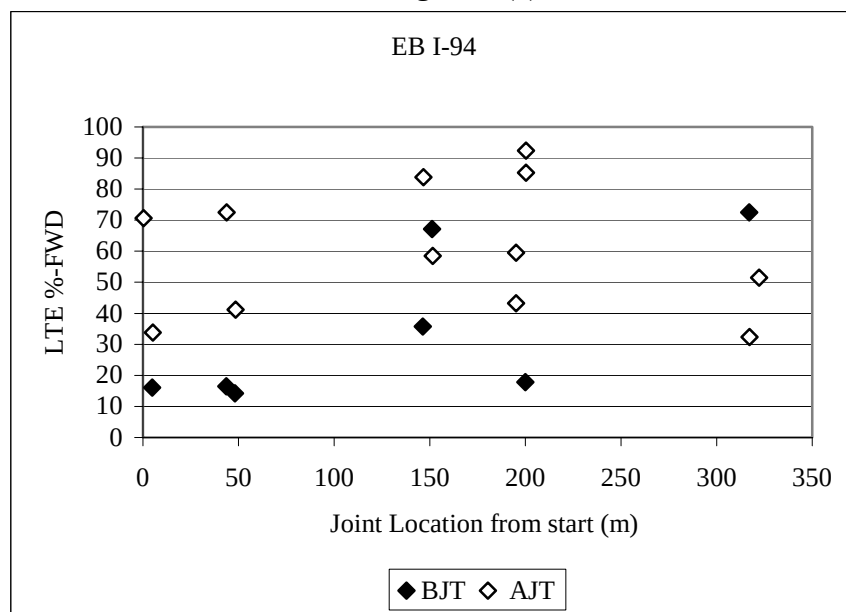
Load transfer efficiency (LTE) was determined before the joint (BJT) and after the joint (AJT) respectively to evaluate the joint performance. BJT means that the testing load is applied before the joint in the traffic direction, with the deflection measured underneath the load and 0.3 m (12in) across the joint. The same is for AJT except the load is applied after the joint.

Severe joint damage has developed for the 320m (1050 ft) JPCP I-94 test section with an average LTE (BJT) of 34% and 60% (AJT) and is highly variable ranging between 16% and 73%, see Figure 1(a). The results suggest that there is a permanent damage of joint load transfer, which increases joint deflections and promotes erosion (Wu et al., 1993). Load transfer was also measured at the crack face, with an average LTE of 18% before the crack (BCK) and 28% after the crack (ACK), see Figure 1(b). These results suggest that there is no aggregate interlock and remaining load transfer is through elastic deflection of the foundation.

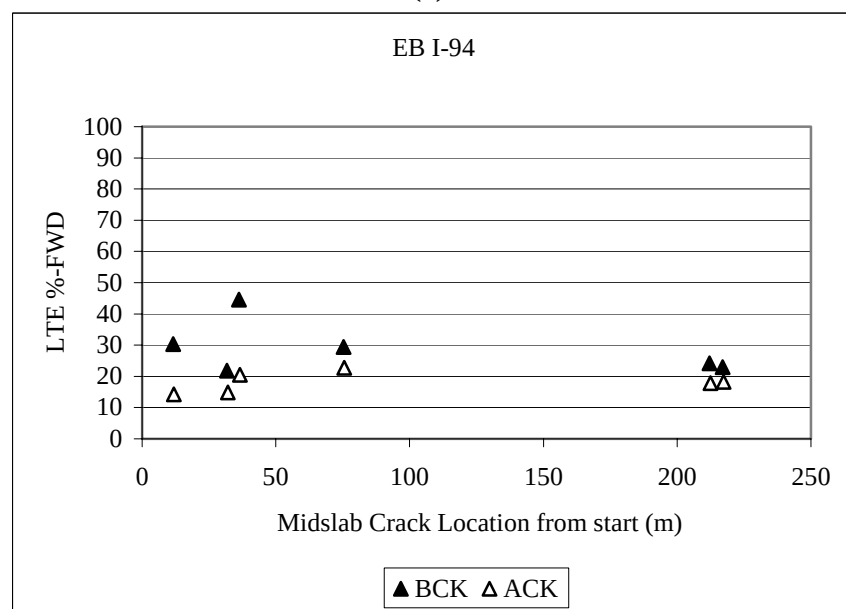
The D0 deflections underneath the loading plate were determined for the joint corner and mid slab, as shown in Figure 2. This information is used to evaluate the support condition underneath the slab. The mid-slab deflections averaged slightly below 100 microns, while the joint corner deflections average 400 microns. The temperature was almost constant throughout the slab depth at the time of testing, as shown in Figure 3, with the maximum negative gradient being -2°C. This indicates that the high joint corner deflection is unlikely due to upward curling. It has been found that joint corner deflection is 2 to 3 times of mid-slab deflection for slabs in full contact with the base. Higher corner to midslab deflection ratios indicate loss of slab support (Crovetti, 2002).

The time-history curves at the joint under FWD loading are shown in Figure 4, to provide detailed information on joint performance under loading. It is clear from a typical time-history curve that the three joint-load transfer components (dowel action, aggregate interlock and base deflection contribution) are all ineffective as seen (Figure 4 (b)) of development of D1 (deflection across the joint 0.3 m/12

inches from the load-plate) versus D0 (slab deflection underneath the load-plate) for testing BJT and AJT at the slab outer corner (i.e. 0.3 m/12 ft line). As the loaded slab joint side deflects under increasing load (i.e. increasing D0) the unloaded side does not follow initially due to a slip of 150 to 200 microns (6 to 8 mils) before there is some deflection, although not nearly as much as the loaded joint side indicating that the dowel is loosened more than 200 microns (8 mils). On the contrary, the simultaneous deflection response on both sides of a joint for the JRCP is nearly linear elastic from start with little or no joint slip and full dowel action and base contribution as seen from the results in Figure 4 (a).



(a)



(b)

Figure 1. Load Transfer at (a) Joints and(b) Cracks

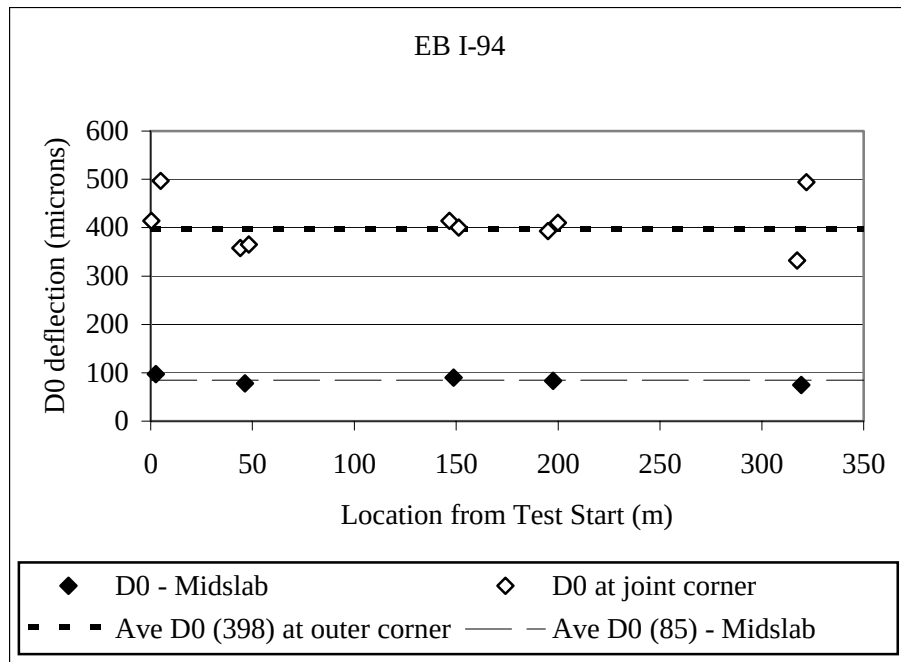


Figure 2. Joint corner deflection in JPCP versus Midslab deflection

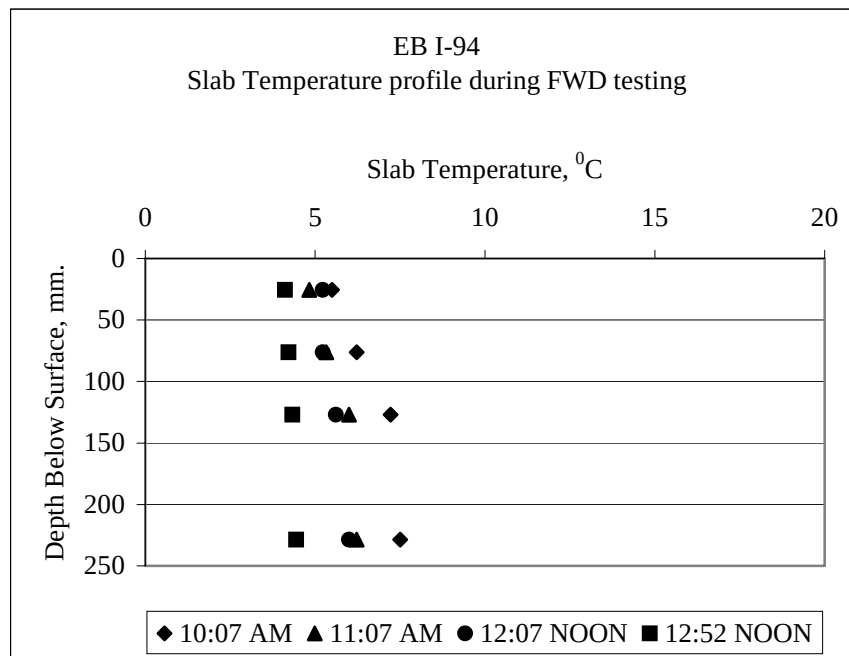


Figure 3. Temperature gradient in JPCP during test period

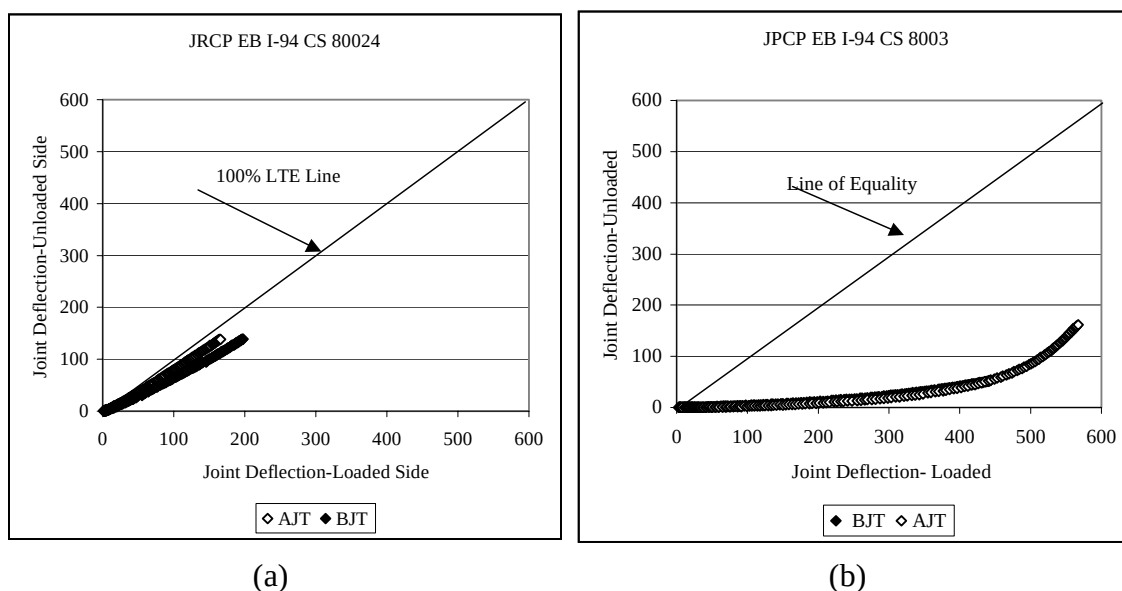


Figure 4. Deflections of loaded and unloaded slabs during time history tests (a) JRCP (b) JPCP

Another manifestation of joint damage from loss of slab support is the development of joint faulting in the JPCP as shown in Table 1. The average faulting is -1550 micron (-61 mils) at the joints, which is substantial for a doweled joint with a dowel diameter of 31.75 mm (1.25 inches) (Wu et al., 1993). This supports the FWD results that the joint damage due to loss of slab support has rendered the dowels ineffective.

Table 1. Fault Measurements at Slab Edge at Transverse Joints and Cracks of EB I-94, in Van Buren County. (*- Indicate Drop From Leave to Approach Slab)

Measures	Joints (microns/mils)	Cracks (microns/mils)
Average	-1550 / -61	-152 / -6
Maximum	1854 / 73	889 / 35
Minimum	-3886 / -153	-2134 / -84
Standard Deviation	914 / 36	1016 / 40

Stress Analysis

FWD field testing and analysis show that loss of slab support associated with base and subbase compaction under the transverse joints is the key factor affecting premature cracking in JPCP projects. The compaction, generally identified as erosion without pumping, initiates a sequence of events which culminate in slab failure. Initially, the combined effects of truck loading and water infiltration at the joints instigate the compaction and erosion process. This creates a permanent and increasing void directly under the slab. The void magnitude is greatest at the outer

slab corners adjoining the shoulder and diminishes towards the slab interior. As this erosion process progresses, the joint deflections increase under truck loading, until the ability to transfer stress across the joint is completely lost. The mechanism controlling the ability to transfer stress across the joints are the dowels, aggregate interlock, and the foundation layers. If the base or subbase has compacted or eroded, the load transfer contribution from the foundation is lost. Truck loading, at the unsupported doweled joints, results in large bearing stresses on the concrete supporting the dowels. Overtime, the dowels become loose from concrete crushing. Aggregate interlock is assumed non-existent under this condition.

The erodibility of the base material depends on the hydrodynamic forces created by the dynamic action of moving wheel loads (Van Wijk et al., 1989; and Titus Glover, 1999). Any untreated granular material, including some weakly cement stabilized materials is erodible. Construction records for the EB I-94 JPCP project showed, that the cracked slab sections correlated with base sections where the open-graded base aggregate was replaced just before paving due to non-specification material.

Finite Element Modeling

In this section, a detailed finite element analysis is presented to estimate the total slab stresses by combining the input from four plausible factors that can contribute to transverse cracking. They are: voids underneath the joint, low load transfer, axle loading, and negative temperature curling. The focus of the analysis is the voids directly under the concrete slab at the corners.

Figure 5 shows a sketch of voids under the slab's corners due to base erosion, with the void radiating toward the center. Loss of support due to base erosion can be characterized by two parameters: the void's area, and maximum depth.

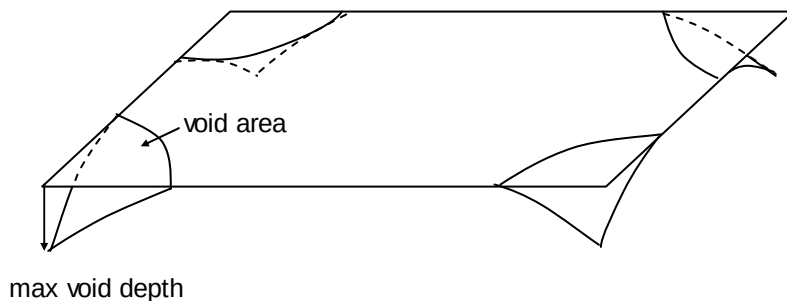


Figure 5. Sketch of voids underneath pavement slabs

ISLAB2000 was first used to model voids underneath the slab. In ISLAB2000, voids can be modeled as a rectangular area, and a void depth can be specified at each corner. However, preliminary analysis showed that when truck loading is applied, the maximum tensile stress remained almost constant, even with increasing void depth. ISLAB2000 appears inadequate to accurately model voids underneath the slab. Therefore, a more powerful and flexible finite element

program, ABAQUS, was used for stress analysis.

ABAQUS is a powerful and reliable general-purpose finite element code capable of solving various structural problems (ABAQUS/Standard Manual, 1997). The incorporation of voids is easily realized by using nonlinear spring elements for the foundation, as shown in Figure 6(a). In the areas with voids, the foundation springs were defined to exert no reaction to the slab until the slab deflection is greater than the magnitude of voids. See Figure 6 (b) for the constitutive law of the nonlinear spring. Two areas of voids 1×1m, and 1.5×1.5 m (3×3ft, and 5×5 ft) were used to simulate moderate and severe base erosion, as established from field investigations (see preceding section). The maximum void depth was varied from 0 to 2540 micron (0 to 100 mils). The decrease in void depth from corner to slab interior was assumed to be linear, see Figure 6(a). The full-contact portion of foundation was modeled using a tensionless spring.

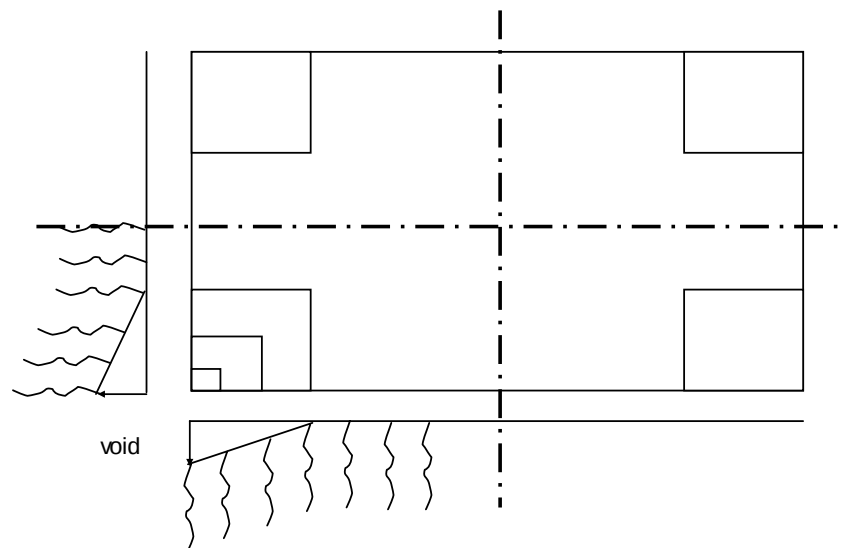
3-D shell element S9R5 of element size 0.3×0.3m (12×12in) was used to model the pavement slab. It is a thin shell element that can be used when the shell normal direction remains orthogonal to the shell reference surface. For homogenous shells, this occurs when the thickness is less than about 1/15 of a characteristic length on the surface of the shell, such as the distance between supports. For the case in point with slab lengths of 4.57 to 4.88 m (15 to 16 ft), the shell element can be used for slab thickness less than 330 mm (13 in).

In ABAQUS, only one slab was modeled for simplicity. The modeling parameters were determined from the field testing and are listed as follows. The slab dimension is 3.66×4.57 m (12×15 ft) and 279mm (11in) in thickness. Young's modulus, E , is 24,138 MPa (3.5×10^6 psi), and the coefficient of thermal expansion, CTE , is $9.9 \times 10^{-6}/^{\circ}\text{C}$ ($5.5 \times 10^{-6}/^{\circ}\text{F}$). One needs to be aware that CTE depends on the type of aggregate used, and can vary between $6\text{--}12 \times 10^{-6}/^{\circ}\text{C}$ (www.cement.ca/cement.nsf). Since the focus of this paper is on the permanent voids due to base erosion, an average CTE value was selected and the effect of CTE is not included. A -11.1°C (-20°F) slab temperature differential, typical for Michigan's climate, was used to incorporate early morning (nighttime) temperature effects. The subgrade reaction k is 68 kPa/mm (250 pci).

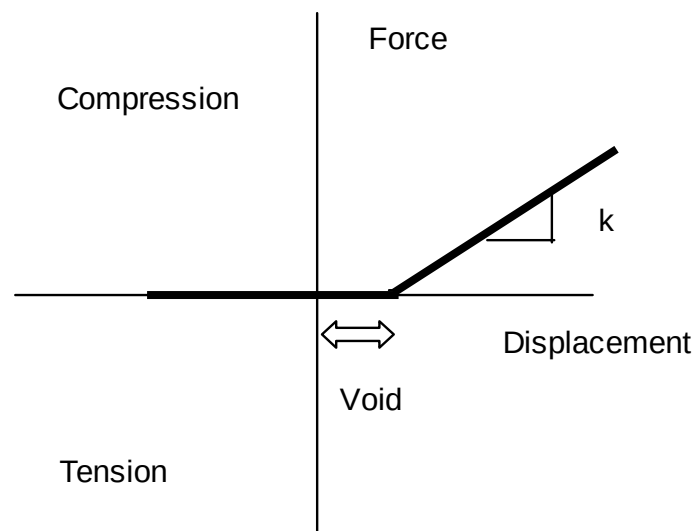
The most critical loading position is directly above the unsupported portion of slab near the joint corners. Thus, simultaneous joint loading is the most critical loading condition. This agrees with published work (Khazanovich et al., 2000; Hansen, et al., 2002). Figure 7 demonstrates the load configuration used in this study taken from Michigan Weight and Axle Load Limits and considered to be the most critical loading case. The rear tandem axle is 142.3 kN (32kip) with axle spacing of 1.07m (3.5 ft), and the front axle is 80.0 kN (18kip). The spacing between the single axle and the front wheel of the tandem is 2.74 m (9ft). Two loading cases were simulated in ABAQUS: truck loading only, combined truck loading with a -11.1°C (-20°F) temperature differential.

The effect of load transfer efficiency (LTE) was included for two extreme cases of LTE equal to 0% and 100% for both the longitudinal and transverse joints. The actual situation in the field is within that range. The two extreme cases of LTE can be modeled in ABAQUS even though only one slab was included. For LTE = 0%, all loading is carried by the single slab; therefore one slab is sufficient for

modeling. For $LTE = 100\%$, it was assumed, that 50% of the load at the joints are transferred to the adjacent slabs, leaving only 50% of the load on the slab. Therefore, the tire loads right over the joints are affected by a reduction, whereas the adjacent axle loads are not reduced. The reduced load method in ABAQUS has been compared with results from ISLAB2000 for the cases of zero voids, and an adequate agreement was found.



(a)



(b)

Figure 6. (a) Nonlinear spring in ABAQUS to model voids (b) constitutive law for nonlinear spring

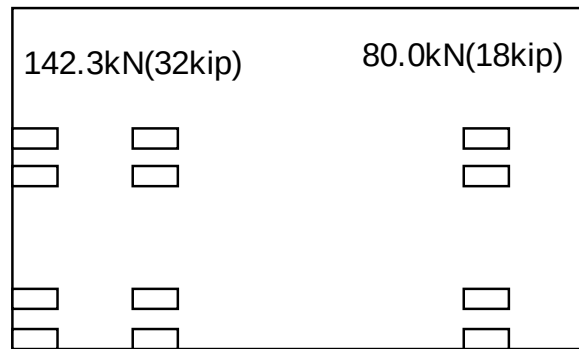


Figure 7. Simultaneous joint loading, tandem axle loading of 142.3 kN (32kip), and single axle of 80.0 kN (18kip)

Figure 8 (a) shows the 3-D deflection profiles for truck loading only for the case of full contact with $LTE = 100\%$ for both direction, and the case of severe base erosion and joint damage (erosion area is $1.5 \times 1.5\text{m}/5 \times 5\text{ ft}$, the maximum void depth is 2540 microns/100 mils, and $LTE=0\%$ for both directions). Again this range covers the expected behavior. The x-axis is the traffic direction, and the y-axis is the transverse direction. For the z-axis deflection, zero is the original position of the slab without loading or voids; “+” means upward movement, and “-” means downward movement. The deflection profiles demonstrate that there is a significant increase in slab deflection and curvature when base erosion exists. The increase in curvature results in substantially increased tensile stresses (from 0.5 MPa to around 2 MPa) at the top of the slab. In Figure 8(b) the flexural strength and fatigue limit (45% of the flexural strength) are shown as bold lines. The result indicates that for truck loading only, the maximum tensile stress reaches the fatigue limit only when severe base erosion has developed. Similar profiles are shown in Figure 9, for combined truck loading and a negative temperature difference of -11.1°C (-20°F). The maximum tensile stress increases from 1.5 MPa to 2.7 MPa, as a result of severe base erosion. These stress levels exceeds the fatigue limit and rapid fatigue failure will result. For both cases, the maximum tensile stresses are located at the slab’s edge near mid-slab. This agrees with the observations of the top-down cracking process.

For both loading cases the maximum slab deflection is still less than the maximum void depth, meaning that the gap due to base erosion does not close under loading. When the maximum void depth is 2540 microns (100mils), the maximum slab deflection is around 2200 microns for truck loading only, and much less for truck loading and curling. The reason is that base erosion is caused by the hydrodynamic forces from the moving wheel load. Therefore it is not necessary for the slab to deflect the same as the magnitude of void.

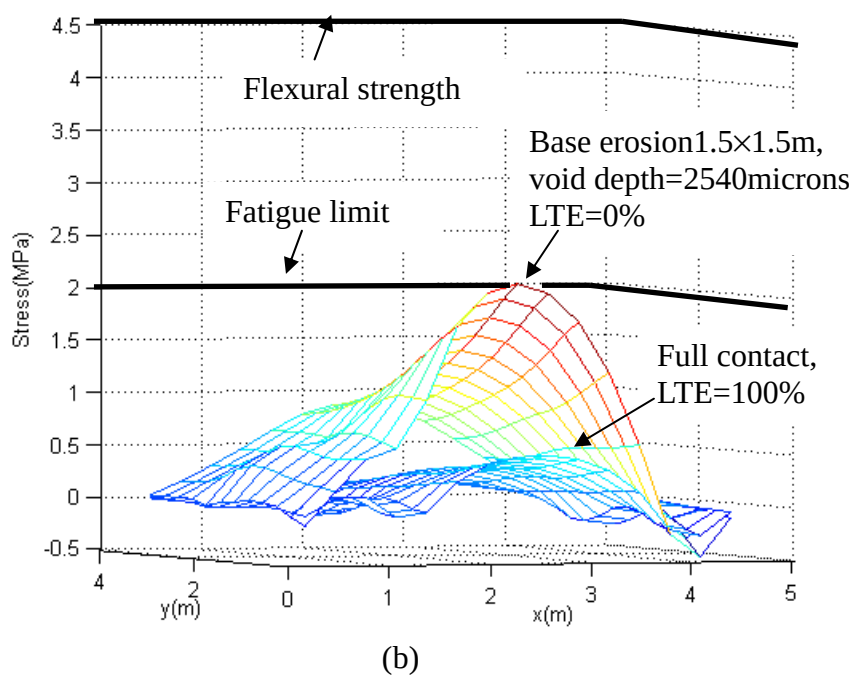
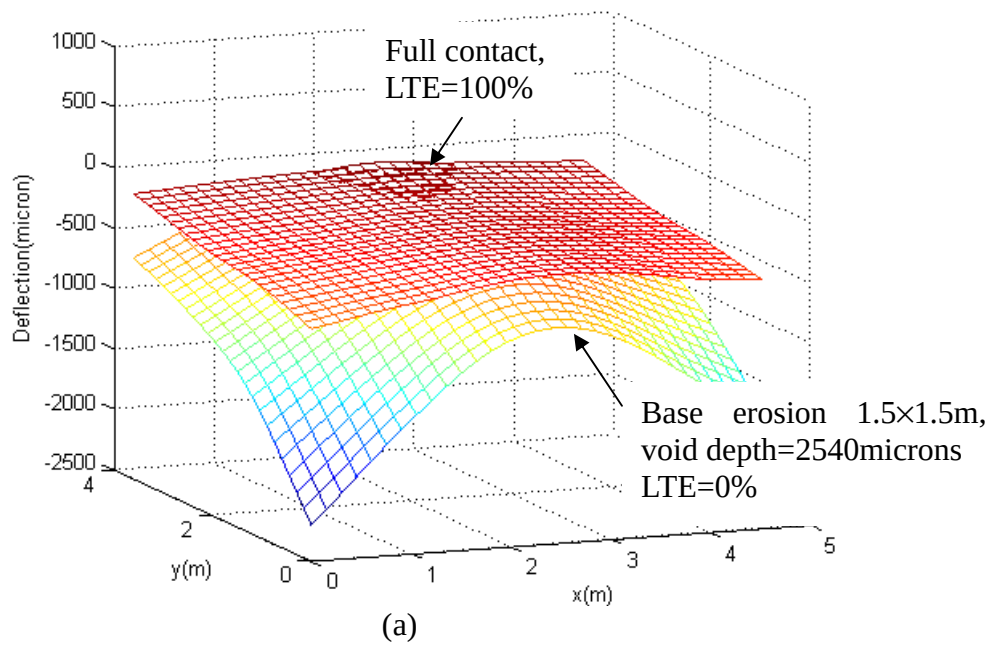


Figure 8. (a) 3D deflection, and (b) 3D stress profiles, truck loading only, for full contact case, LTE=100%, for base erosion case, LTE=0%, this shows the possible range of stress when there is no curling

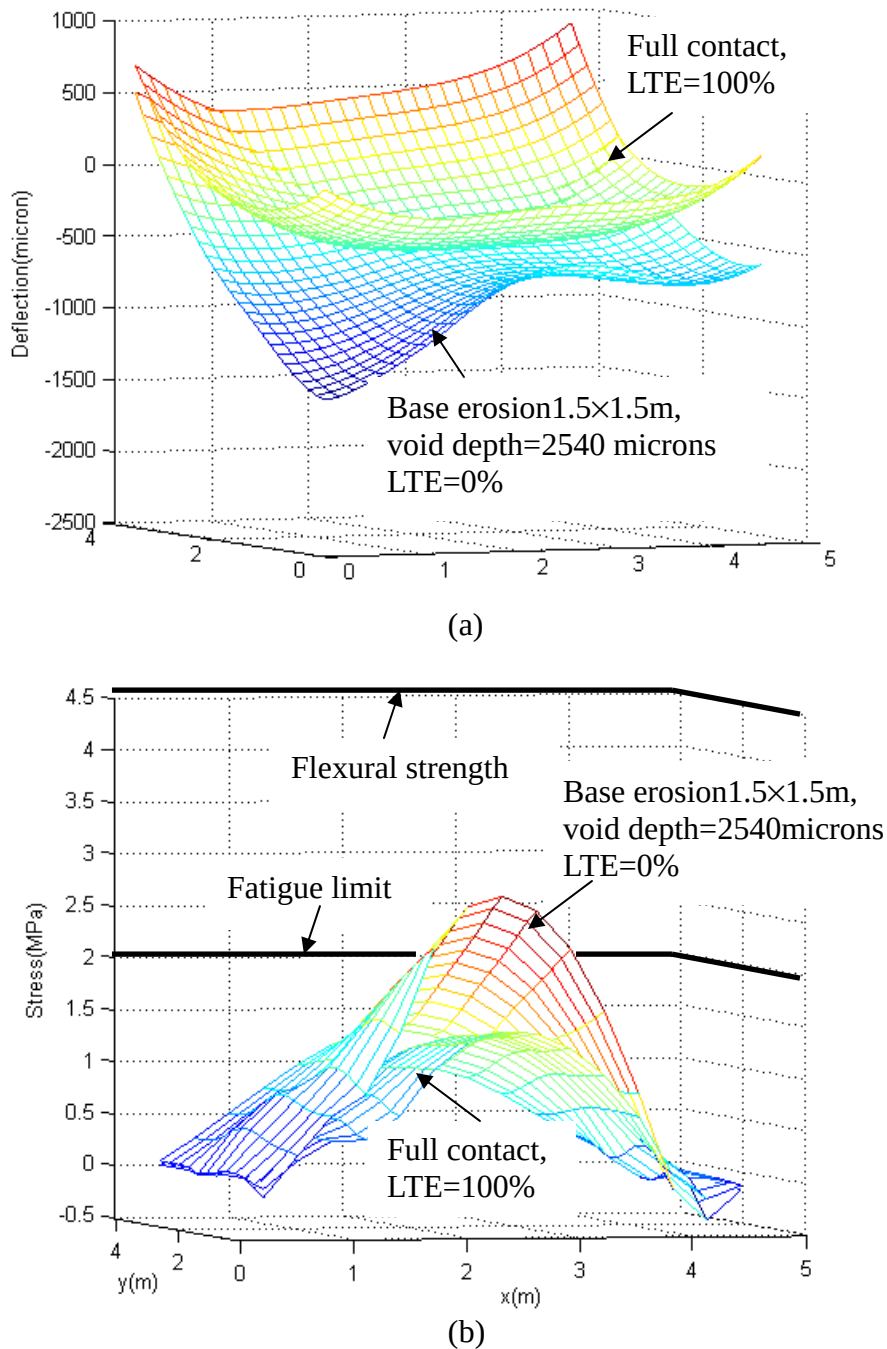


Figure 9. (a) 3D deflection, and (b) 3D stress profiles, truck loading and negative temperature gradient, for full contact case, LTE=100%, for base erosion case, LTE=0%, this shows the possible range of stress when there is curling

The maximum tensile stresses, as a function of axle loading, curling, void area and depth, and LTE, are shown in Figure 10 for the two evaluated load cases. The maximum tensile stress increases with an increase in void area and depth. An increase in maximum stress with a decrease in LTE is also observed. Truck loading

alone is unlikely to cause fatigue cracking because the void needs to be very severe for the maximum tensile stress to be higher than the fatigue limit. However, when a temperature difference is also present, there are several combinations that will result in tensile stresses greater than the fatigue limit. The critical load case includes combined truck loading and a -11°C (-20°F) temperature differential:

- The maximum stress will exceed the fatigue limit when the erosion area is 1.5×1.5m, a maximum void depth greater than 500 microns, and LTE=0%.
- The maximum stress will exceed the fatigue limit when the erosion area is 1.0×1.0m, maximum void depth greater than 1500 microns, and LTE = 0%.
- The maximum stress will exceed the limit when the erosion area is 1.5×1.5m, maximum void depth greater than 1500 microns, and LTE=100%.

Recall that these three combinations are all idealized cases, as it is unrealistic to actually have 0% load transfer, or 100% LTE when the void depth is 1500 microns. These combinations rather represent the boundary condition of stress greater than the fatigue limit for certain field conditions.

The allowable load repetition related to base erosion can also be calculated using the methodology proposed by the Portland Cement Association (PCA), see Equation (3). It is based on the work by Wu et al. (1993), who found that erosion can be related to the rate of work defined as the product of corner deflection, w , and pressure, p , at the slab-foundation interface divided by the length of deflection basin, which is a function of the radius of relative stiffness l .

$$\log N = 14.524 - 6.777(C_1 P - 9.0)^{0.103} \quad (3)$$

where N = allowable number of load repetitions base on a present serviceability index of 3.0

C_1 = an adjustment factor with a value of 1 for untreated subbases and 0.9 for stabilized subbases

$$P = 268.8 \frac{p^2}{hk^{0.73}}, \text{ rate of work or power}$$

$p = kw$, pressure on the foundation under the slab corner

The predicted allowable load repetitions are shown in Figure 11 for k ranging from 13 to 136 kPa/mm (50 to 500 pci). It is obvious that the allowable load repetitions decrease rapidly as the corner deflection increases, because the rate of work is proportional to w^2 . For the condition of this study ($k=68$ kPa/mm or 250 pci), the allowable load repetitions is less than 10^7 , when the corner deflections are greater than 510 microns (20mils). This correlates well with the results from finite element analysis, see Figure 10 (b). Assume that the void is caused by base compaction, and that the corner deflection is the same as the maximum void depth. From Figure 10(b), for the two areas of base erosion, the maximum tensile stress increases beyond the fatigue limit when there is 510 to 700 micron maximum void depth, and LTE = 0%. This further validates the finite element results in this study. The threshold corner deflection value for the slabs with an erosion problem increases with a decrease in k . But this does not mean that slabs on a soft foundation are unlikely to fail by erosion, because the deflection will be much higher for soft foundation under the same load.

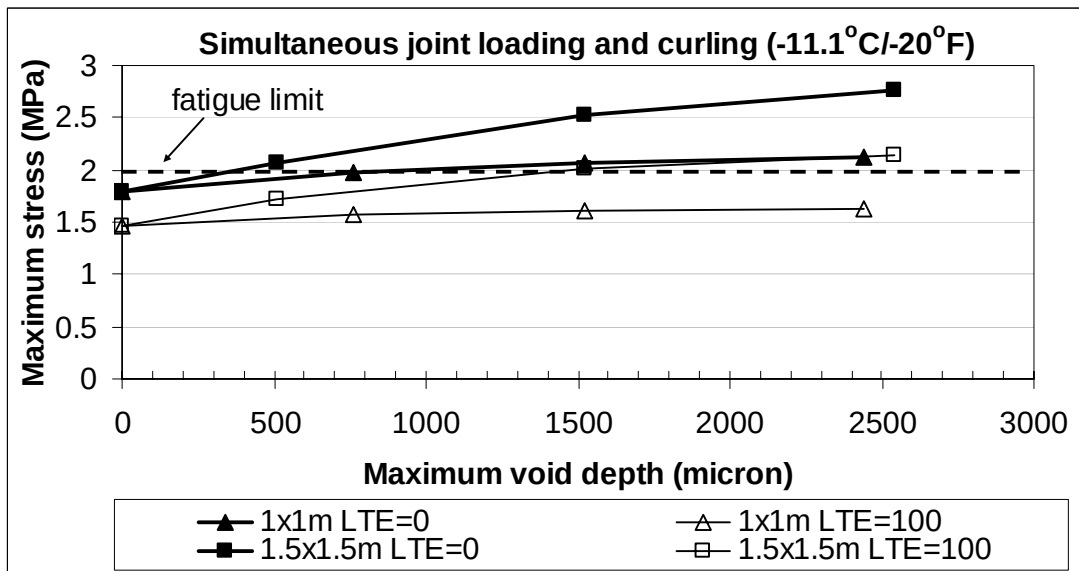
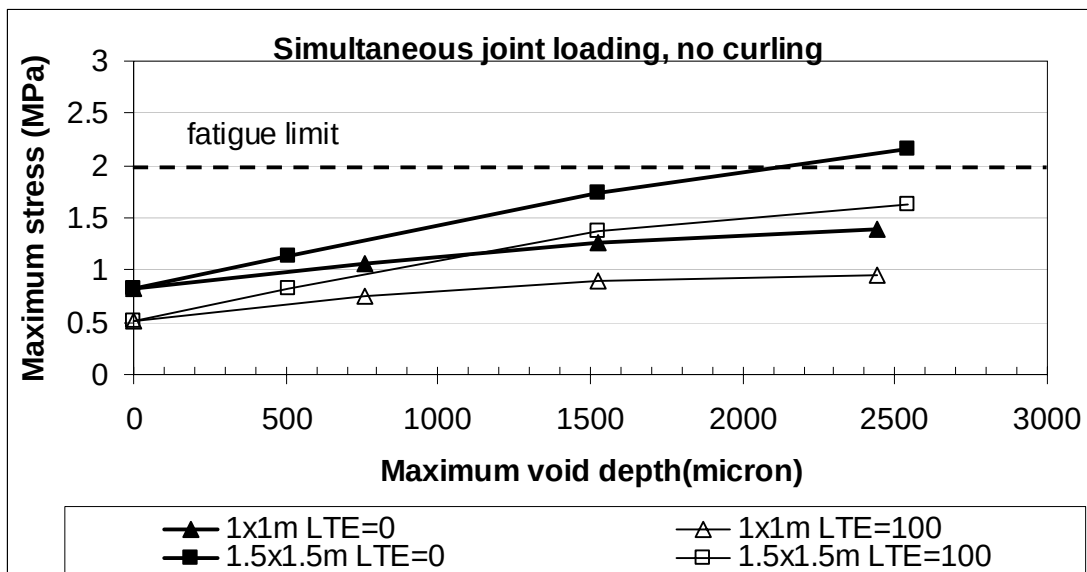


Figure 10. Maximum tensile stresses (a) truck loading only (b) truck loading and negative temperature difference

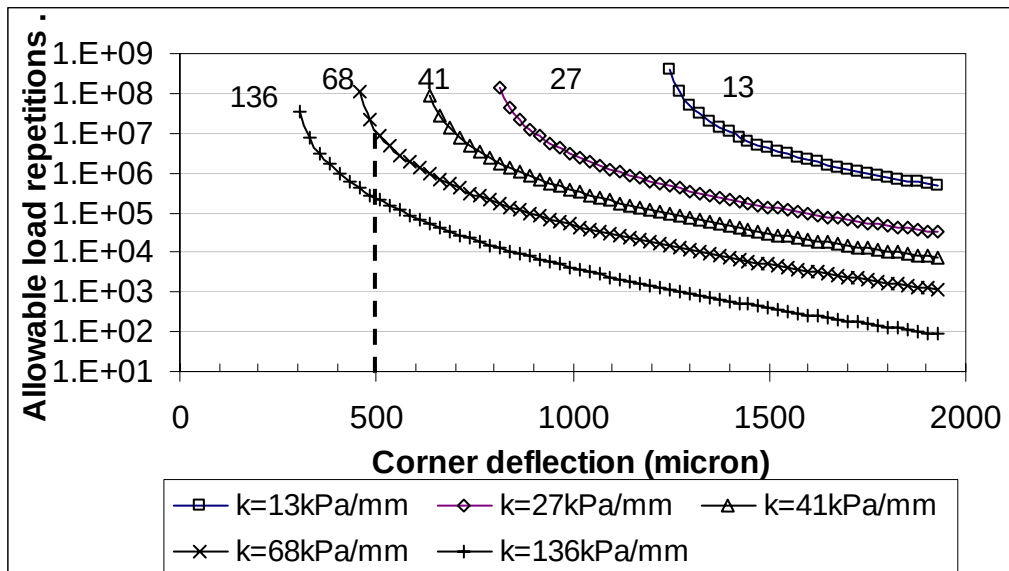


Figure 11 Allowable load repetitions for base erosion calculated per PCA model

Conclusions

A permanent loss of support near the joint has developed for Michigan JPCP with cracked slabs. The specific reason for the loss is still unknown, but the OGDC base used by the Michigan DOT for the cracked projects appears to be susceptible to post-compaction or particle degradation from water infiltration under fully-loaded, axle loading. Construction records showed the I-94 JPCP project in particular had difficulties in meeting the OGDC in-place gradation specification. The gap condition under the slab has prompted serious joint damage, consisting of loose dowels, a loss in aggregate interlock, and the onset of faulting. Over time sufficient load transfer is lost and the slab becomes susceptible to mid-slab cracking under simultaneous axle loading at both ends of the slab. Top-down cracking is the result, initiating at the outer edge. Field observations show that fatigue failure develops rapidly (1-2 years) after construction, once joint load transfer is exhausted.

A 3-D finite element based stress analysis was implemented in ABAQUS to understand the combined sensitivity of axle loading, negative temperature curling, and the gap magnitude, which is maximum at the slab's four corners and decreases toward mid-slab. The analysis predicted that top-down, mid-slab fatigue cracking will initiate (stress level exceeds 45% of flexural strength) for the following conditions: slab length is 4.57 m (15 ft) and 279 mm (11 in.) thick; modulus of subgrade reaction is 0.068 MPa/mm (250 psi/in.); the joints along the outer edge are subjected to a tandem axle loading of 142.3 kN (32 kip) at one end and a single axle load of 80 kN (18 kip) near the other joint; for a curling gradient of -11°C (-20°F); when a permanent void of about 510 microns (20 mils) is present under the outer slab corners.

This prediction is confirmed by the M-E model developed by Wu et al.

(1993) at PCA to study erosion induced pavement damage. It can be used to predict mid-slab transverse cracking for a JPCP on an open-graded base if corner deflection without curling exceeds about 510 microns. Truck and curling stress from negative curling gradient of about -11°C will result in midslab cracking. Transverse cracking without curling would require a considerably larger permanent void (2000 microns) at the joint.

Acknowledgments

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