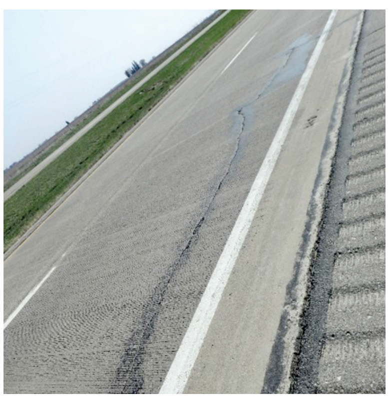
Prevention of Longitudinal Cracking in Iowa Widened Concrete Pavement

Final Report
June 2018

PROSPER



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16. Abstract

Iowa has adopted 14 ft widened concrete slabs (as opposed to the standard 12 ft concrete slabs) in jointed plain concrete pavement (JPCP) design and construction since the 1990s. The additional 2 ft slab paved beyond the normal traffic path is intended to reduce stresses and deflections at the critical concrete pavement edge location by effectively moving the normal traffic path well away from the edge. However, many widened concrete pavements are now approaching 20 years of service life, and some 14 ft widened concrete pavements are experiencing sudden and significant amounts of longitudinal cracking. To understand the causative factors contributing to longitudinal cracking in widened JPCP and to provide recommendations for preventing its occurrence, field investigations were performed at 12 sites in spring and summer 2017. These sites included widened JPCPs of various ages, shoulder types, mix design aspects, environmental conditions during construction, and traffic levels. The location and extent of existing longitudinal cracking, including transverse cracking, were well documented. The amount and severity of cracks were linked to traffic level and shoulder type. Concrete cores were also examined to better understand how the cracking had developed. It was found that the 14 ft widened slabs with tied PCC shoulders outperformed the others in terms of producing less cracking, even though they had experienced higher levels of truck traffic. Widened slabs with granular shoulders were the worst performers, producing higher cracking compared to others. ISLAB 2005 and EverFE 2.25 finite element analysis (FEA) programs were also utilized to demonstrate through numerical analysis the potential of top-down longitudinal cracking for widened JPCP. Transverse joints and wheel paths were found to be critical locations for longitudinal cracking. Widened slabs with skewed joints were also found to have higher potential for longitudinal cracking. Shoulder design alternatives used in Iowa were compared for their effect on longitudinal cracking when they were used adjacent to widened and regular-sized slabs.

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EXECUTIVE SUMMARY

Since 1992, Iowa has adopted 14 ft widened concrete slabs (as opposed to the standard 12 ft concrete slabs) in jointed plain concrete pavement (JPCP) design and construction. In this design, the traffic lane (on the right side) is constructed with 14 ft widened slabs, while the passing lane (on the left side) uses 12 ft concrete slabs. The lane width of the traffic lane composed of widened slabs is still established at 12 ft wide by drawing white strips 2 ft away from the slab edge in the direction of traffic. Adding 2 ft in widened slabs can help reduce stresses and deflections at the critical concrete pavement edges, thereby reducing shoulder maintenance costs and increasing the safety of maintenance crews by minimizing their exposure on high-volume roadways.

In Iowa, some 14 ft widened concrete pavements have experienced sudden and significant longitudinal cracking (LC). These pavements were typically constructed with skewed joints around the end of the 20th century and were approaching 20 years of service life. Since the unexpected longitudinal cracking is detrimental to the long-term performance of JPCP, it is of paramount importance to identify the causes of such longitudinal cracking in Iowa widened JPCP. Recommendations are needed for widened JPCP design features and construction practices to minimize and prevent longitudinal cracking.

This research study was to document and assess longitudinal cracking at Iowa's widened concrete pavement sites. A comprehensive literature review was conducted to explore current practices adopted by other states in widened concrete slab design and construction and to identify probable causes and mitigation strategies with respect to longitudinal cracking problems. Field surveys were performed in 2017 at 12 existing sites in Iowa, including 4 control sites and 8 other sites undergoing different levels of longitudinal cracking.

The four control sites were as follows:

- Control site/No.1: Polk County, MP 71.58 to MP 72.99, US 65
- Control site/No.2: Polk County, MP 72.99 to MP 74.11, US 65
- Control site/No.3: Tama County, MP 193.1 to MP 197.76, US 30
- Control site/No.4: Story County, MP 159.85 to MP 160.15, US 30

The eight LC sites were as follows:

- LC site/No. 1: Linn County, MP 30.12 to MP 33.48, US 151
- LC site/No. 2: Linn County, MP 259.82 to MP 263.3, US 30
- LC site/No. 3: Mahaska County, MP 44.99 to MP 55.6, IA 163
- LC site/No. 4: Henry County, MP 43.14 to MP 46.64, US 218
- LC site/No. 5: Jasper County, MP 21.44 to MP 24.87, IA 163
- LC site/No. 6: Polk County, MP 76.82 to MP 78.99, US 65
- LC site/No. 7: Lee County, MP 30.32 to MP 32.61, US 61
- LC site/No. 8: Washington County, MP 235.09 to MP 241.95, IA 92

These sites represent widened concrete pavements of various ages, mix design aspects, construction conditions, shoulder types, and traffic levels. Slab geometry was checked, and existing longitudinal and other types of cracking (e.g., transverse cracking) were also documented. The extent and pattern of longitudinal cracking were linked to pavement age, mix design aspects, construction conditions, and traffic level to determine potential factors contributing to the cracking. Concrete cores extracted from cracking locations were also examined. Key findings from the field investigations are summarized as follows:

- Longitudinal cracks occurred mainly in the traffic lane about 2 to 4 ft away from the slab edge, parallel to the traffic direction, and some arc-shaped longitudinal cracks were also observed.
- Most observed longitudinal cracks were initiated from slab joints.
- Longitudinal cracks were not observed at control site 1 and control site 2 in Polk County, sites with tied Portland cement concrete (PCC) shoulders, even though these sites experienced relatively higher traffic volumes.
- Based on field observations, sites with tied PCC shoulders performed better than sites with hot mix asphalt (HMA) shoulders and granular shoulders in terms of the observed level of longitudinal cracks.
- Concrete core samples (LC site/No. of 1, 2, 3, and 5) reflected top-down longitudinal cracking.

Numerical analysis using ISLAB2005 and EverFE 2.25 Finite Element Analysis (FEA) software was conducted to further understand the underlying mechanisms of longitudinal cracking failures in concrete pavements. Critical longitudinal cracking locations were also identified through numerical analysis. Numerical analysis validated that widened JPCP with skewed joints had higher potential for developing longitudinal cracking. Through numerical analysis, shoulder design alternatives used in Iowa were compared in terms of their effects on longitudinal cracking when they were used adjacent to both widened and regular-sized slabs. Potential contributing factors for longitudinal cracking based on the findings of this study are summarized. Recommendations are offered on how to minimize longitudinal cracking based on a literature review, field study, and related finite element analysis carried out in this research.

Most sites selected for field investigations in this study showed different levels of longitudinal cracking, were built with skewed joints before 2000, and were approaching 20 years of service life. The Iowa Department of Transportation (DOT) started using rectangular joints in widened JPCP only after 2005, and the only such site (US 30 in Tama County) considered for field investigation in this study demonstrated few longitudinal cracking issues.

In consideration of these findings, the project technical advisory committee (TAC) recommended a follow-up study (i.e., Phase II study) to do the following:

- Evaluate the performance of rectangular joints in widened JPCP at a greater number of sites (about 30 to 40 sites, or 3 to 4 times the number of sites considered in this study)
- Verify the effectiveness of those design features (or alternative design features) for preventing and minimizing longitudinal cracking

- Identify the best practices for repairing longitudinal cracking problems in Iowa JPCP

INTRODUCTION

Problem Statement

Over the past 10 to 15 years, Iowa has used widened concrete slabs rather than the standard 12 ft concrete slabs in jointed plain concrete pavement (JPCP) design and construction. It was theorized that as the distance between traffic loading location and slab edge increases, the critical tensile stresses for bottom-up cracking decrease (Sawan et al. 1982). The extended 2 to 3 ft slab width paved beyond the normal traffic path is intended to reduce stresses and deflections at critical concrete pavement edge locations by effectively moving the normal traffic path well away from the edge (ARA 2004). Other advantages of using widened slabs include reduced shoulder maintenance costs and increased safety of maintenance crews through minimizing their exposure on high-volume roadways.

It is known that portland cement concrete (PCC) slabs tend to crack when tensile stresses, which can vary considerably in early stages, exceed the slab tensile strength. In an attempt to minimize induced stresses and induce a plane of weakness, transverse and longitudinal saw cuts typically are created at locations where a crack is likely to initiate and propagate downward.

In Iowa, many widened concrete pavements are approaching 20 years of service life. Some 14 ft widened concrete pavements are experiencing sudden appearances of significant numbers of longitudinal cracks, but field observations suggest that these cracks are appearing at unexpected locations rather than at saw cut locations. Field observations by Iowa Department of Transportation (DOT) PCC materials engineers and the Iowa State University (ISU) research team indicated that, while random longitudinal cracks in Iowa widened JPCPs typically start as 4 to 6 in. long cracks from the joint within distances of 2 to 4 ft from the slab edge, they eventually grow in degree and severity. Iowa has not reported such distress patterns before, and an investigative study was warranted to better understand the causes of such distress patterns.

A number of studies dating back to 1935 (Janda 1935) have been conducted by different state highway agencies (SHAs) in the US to determine causes of the occurrence of longitudinal cracking in JPCP and preventive strategies to mitigate it. A general observation emerging from the existing literature on JPCP longitudinal cracking is that it results from several interrelated factors, including variations in temperature, moisture gradients between slab top and bottom, jointing practices, and type of base/subbase materials. Jointing practices include saw cut characteristics (timing, sawing process, saw cut depth, etc.), joint spacing, and alignment of dowels at joints. However, the widespread belief is that longitudinal cracking primarily results from poor construction practices and other non-load-related causes.

The Wisconsin Highway Research Program (WHRP) recently published a research project report (Owusu-Ababio and Schmitt 2013) that evaluates and statistically compares the performance of Wisconsin concrete pavements with wider panels (14 ft wide or greater) to that of pavements with standard panel widths (12 to 13 ft). However, there does not appear to be a great deal of research focusing on the design and construction of widened JPCP to minimize and prevent longitudinal cracking before it occurs.

Research Objectives

The objectives of this project were as follows:

- Conduct a field investigation to survey longitudinal cracking in Iowa’s widened concrete pavements
- Identify the causes of longitudinal cracking in widened JPCP in Iowa
- Perform finite element analysis (FEA) to simulate widened JPCP responses for axle loads under various temperature gradients
- Develop recommendations for widened JPCP design features and construction practices to minimize and prevent longitudinal cracking

LITERATURE REVIEW

Background on Current Widened Concrete Slab Design Concept

Most states currently use 12 ft wide concrete slabs as standard practice for lane construction of rigid pavements. In the *Mechanistic-Empirical Pavement Design Guide* (MEPDG), 12 ft is also the default input value for lane width design, where lane width usually refers to the distance between lane markings on either side of the lane, not necessarily equal to slab width. However, over the past 20 years, state DOTs have begun using 14 ft widened slabs for lane construction as a means of crack mitigation. These 14 ft widened slabs usually are constructed only for the traffic lane (on the right side) next to a passing lane composed of regular 12 ft wide slabs. The lane composed of 14 ft slabs is still identified as being 12 ft wide by marking a white line 2 ft away from slab edge (see Figure 1).

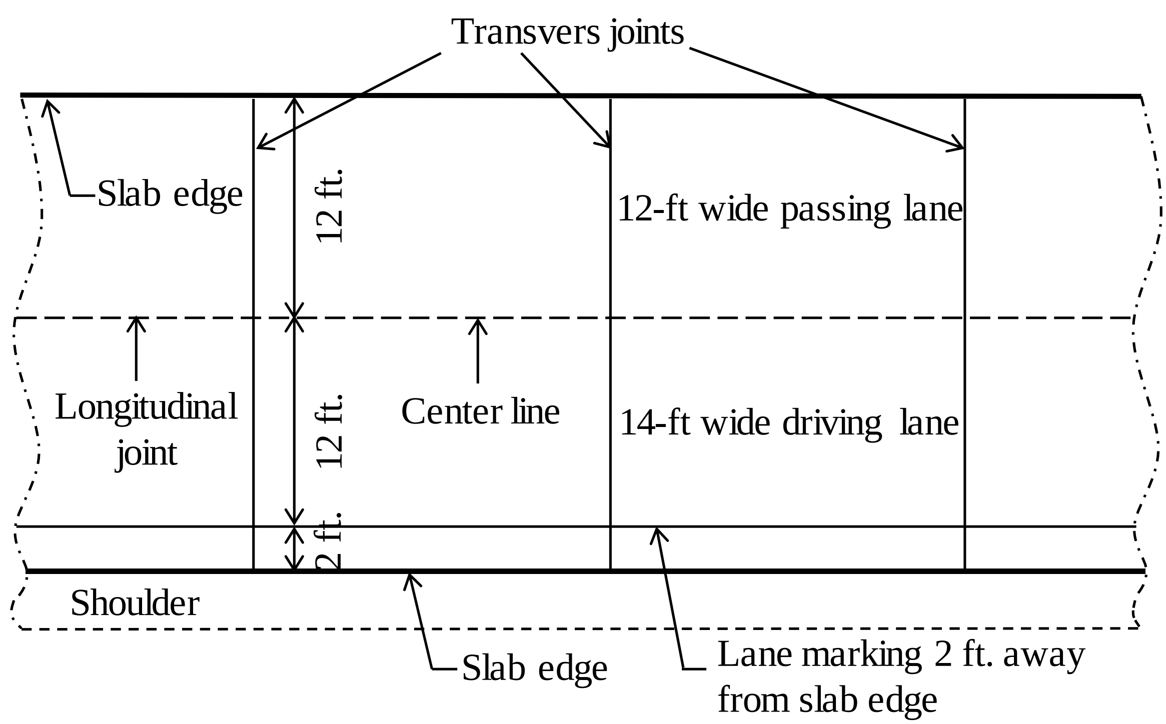


Figure 1. Typical widened slabs used in two-lane pavement

A slab edge is universally recognized as a critical location for stress analysis in rigid pavement systems. Westergaard (1926) investigated slab curling by looking into the effects of temperature differentials in PCC pavements. An infinite or semi-infinite PCC slab supported on a Winkler foundation was used to produce closed-form solutions for loading conditions at corners, edges, and interior of the slab (Westergaard 1926). The study revealed that stress due to edge loading was higher than stress due to interior loading. Furthermore, fatigue analysis of concrete pavement subjected to truckloads also indicated that midway between the slab edge and wheel path at slab transverse joints is the most critical location for structural response and crack initiation, and truckloads near a slab edge usually result in higher critical stress than at other locations (Huang 2004). The maximum bending stress due to traffic loads can decrease dramatically if the location of outside tire loads is moved slightly inward from the slab edge. Figure 2 illustrates the critical stresses for loading at the slab edge and transverse joint (Huang 2004).

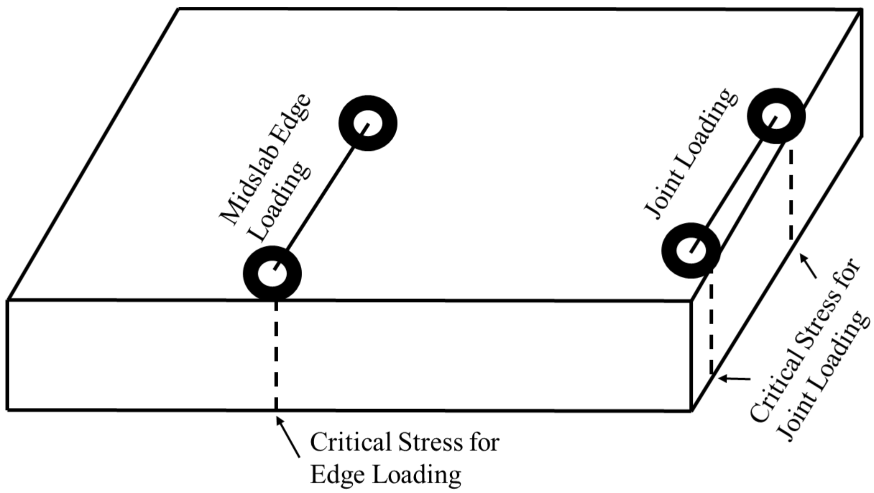
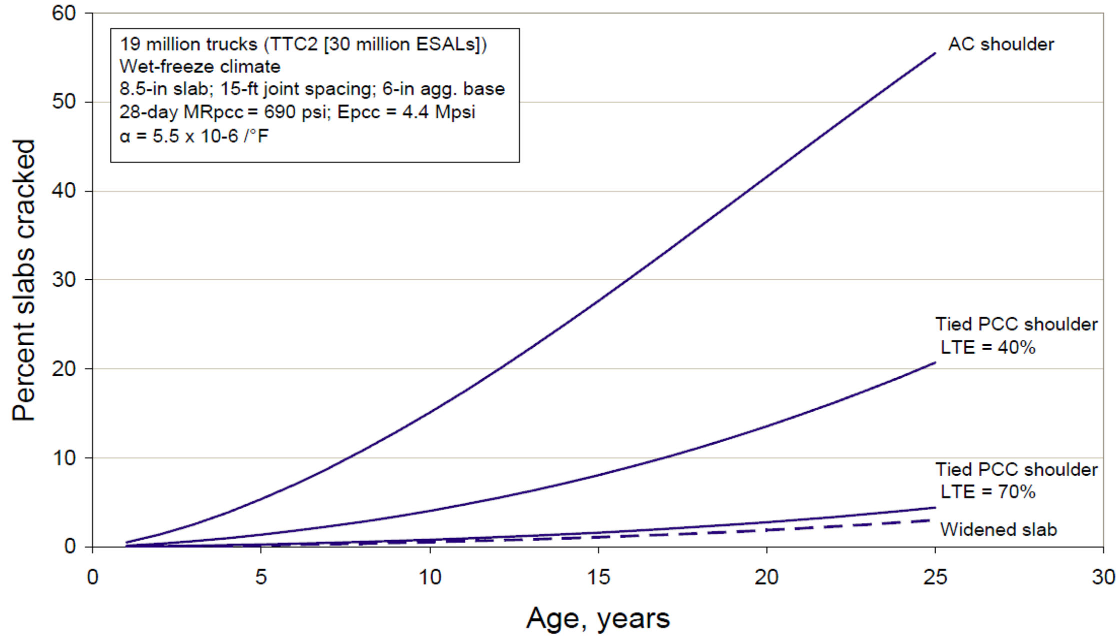


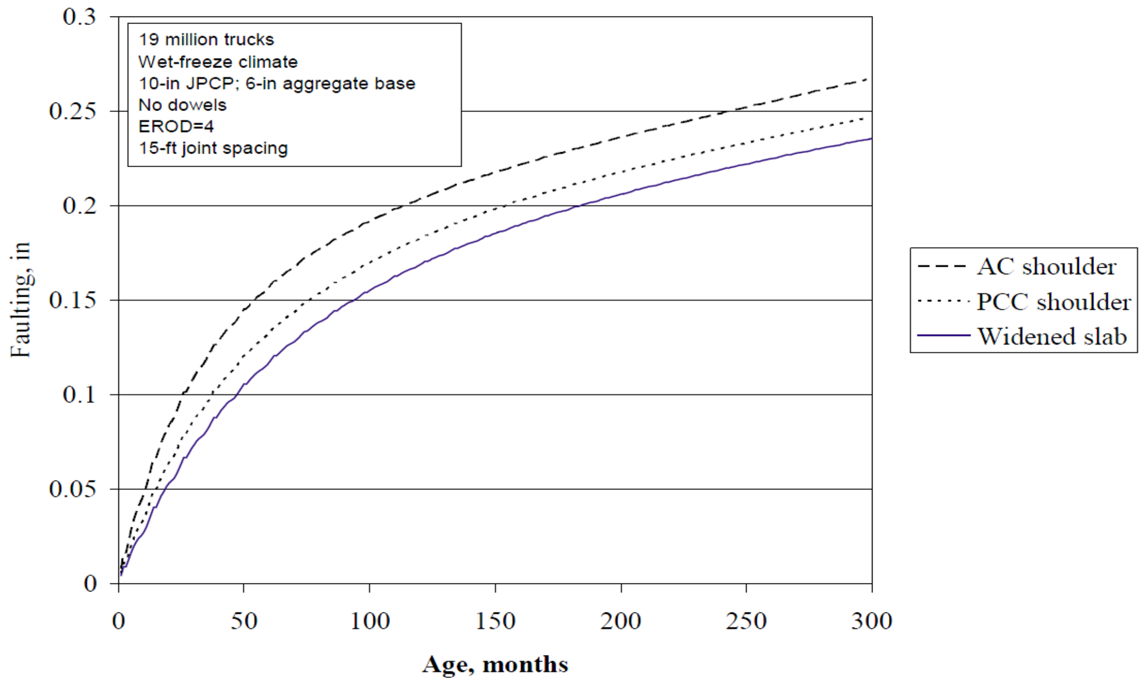
Figure 2. Critical stresses for loading at slab edge and transverse joint

Widened concrete slabs offer significant advantages by reducing critical stresses, strains, and deflections at slab edges, thus reducing structural damage (ARA 2004). Potential for bottom-up and top-down transverse cracking, erosion, pumping, and joint faulting can be effectively reduced (Figure 3 and Figure 4) with the use of widened JPCP. The additional 2 ft width helps to move truck axles further from free edges and corners to create interior-loading rather than edge-loading conditions. The mean wander will also be increased from 18 in. to 42 in. away from the slab edge in the longitudinal direction, helping to reduce potentially dangerous edge drop-offs (Lederle 2014). In summary, widened concrete slabs improve pavement performance, load carrying capacity, and safety and reduce pavement maintenance costs by moving traffic loads away from critical locations. It should be noted that 14 ft widened slabs for lane construction are effective only when the traffic lane is striped for a 12 ft lane width. Slabs wider than 14 ft are not recommended, since this creates the potential for longitudinal cracking. Rumble strips can be placed on the widened area to discourage vehicles from running into the area (ARA 2004, Lederle 2014).



ARA 2004

Figure 3. Effects of lateral edge support on transverse cracking in JPCP



ARA 2004

Figure 4. Effects of lateral edge support on joint faulting in JPCP

Widened slabs are a good alternative for improving overall pavement performance in JPCP design, especially for fatigue-related transverse crack control. However, an increased tendency for longitudinal cracking development along the wheel path has been observed in some widened slab sections, and this issue has not been clearly addressed in the MEPDG. More research is needed to fully understand the impact of widened slabs on concrete pavements (ARA 2004).

Current Practices of Widened Concrete Slab Design and Construction

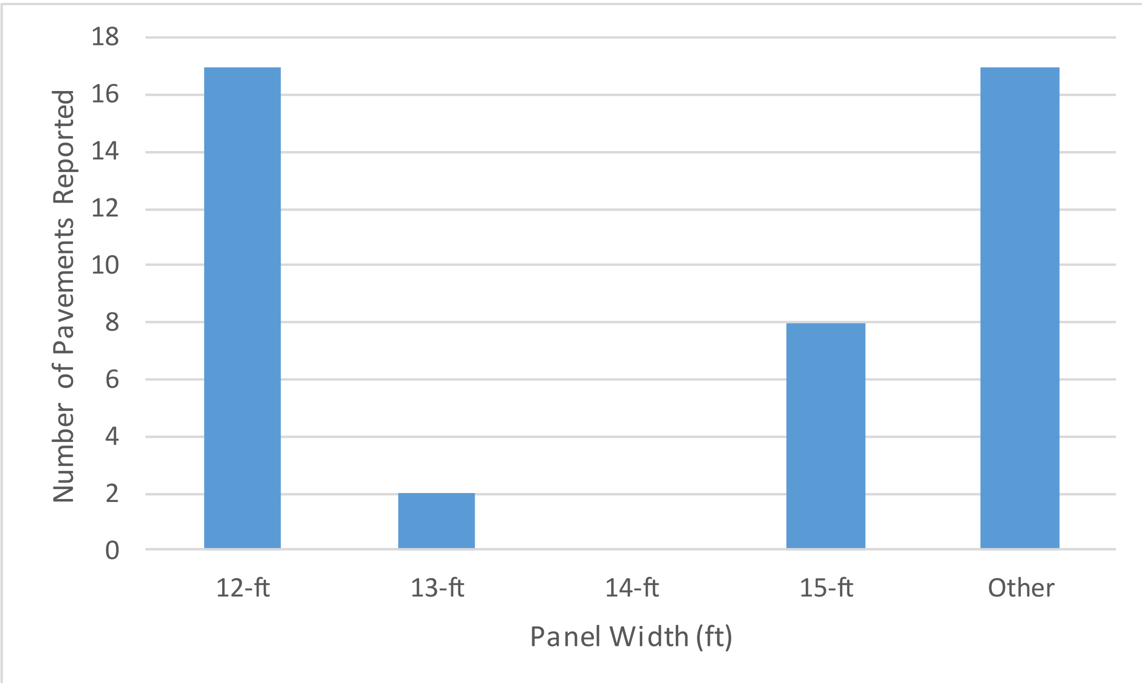
Wisconsin Department of Transportation (WisDOT) and Other Studies in Midwest States

WisDOT started building widened concrete pavements in the early 1990s. This new pavement structure included two general categories: 26 ft wide pavements for rural four-lane divided highways and 30 ft wide pavements for rural two-lane highways. For the four-lane divided highways, the pavement sections comprising the right lanes were paved at 14 ft wide. Owusu-Ababio and Schmitt (2013) reported that the reasoning behind the use of such widened sections on mainline paving was to reduce the amount of stress and deflection at the concrete slab pavement edges from tires running near the edge. From later in situ investigations, researchers noticed that the stress problem diminished when an additional 2 or 3 ft were paved beyond the normal traffic path (Owusu-Ababio and Schmitt 2013). Based on these field investigations, there appear to be three principal advantages of such widening: longer service life, reduced maintenance costs especially on shoulders, and enhanced safety due to elimination of the hazardous edge dropoff at the right edge of 12 ft lanes.

However, Owusu-Ababio and Schmitt (2013) stated that at the time of their research WisDOT had very little information to evaluate the performance of widened PCC pavements. Even though some previous studies had investigated design and construction practices for mitigating edge stresses and deflections and consequently reducing shoulder maintenance cost, there was still a need for “a broader perspective to allow the performance of concrete pavement width alternatives to be evaluated for cost-effectiveness” (Owusu-Ababio and Schmitt 2013). Owusu-Ababio and Schmitt (2013) thought that, in order to address this problem, it would be necessary to conduct a comprehensive investigation of concrete pavements with all kinds of panel widths, in Wisconsin and in other states. It was also recommended that the cost-effectiveness and applicability for Wisconsin be analyzed, because despite the success of recently constructed pavements in improving edge cracking and reducing shoulder maintenance costs, the susceptibility of pavements to other pavement distresses may increase (Owusu-Ababio and Schmitt 2013).

Owusu-Ababio and Schmitt (2013) conducted an online survey in Iowa, Ohio, Michigan, Wisconsin, Illinois, and Minnesota to collect data from 522 county engineers and pavement researchers on JPCP practices and how those practices affect longitudinal cracking development in JPCP. The information sought pertained to cross-section practices, including criteria for determining panel widths on rural highways and for commonly used panel widths. Unfortunately, a large percentage of the invitees said they were unable to participate in the survey because of a lack of JPCP in their respective jurisdictions (Owusu-Ababio and Schmitt 2013). After 100 of the invitees visited the online site, only 37 invitees ultimately replied to the survey questionnaires.

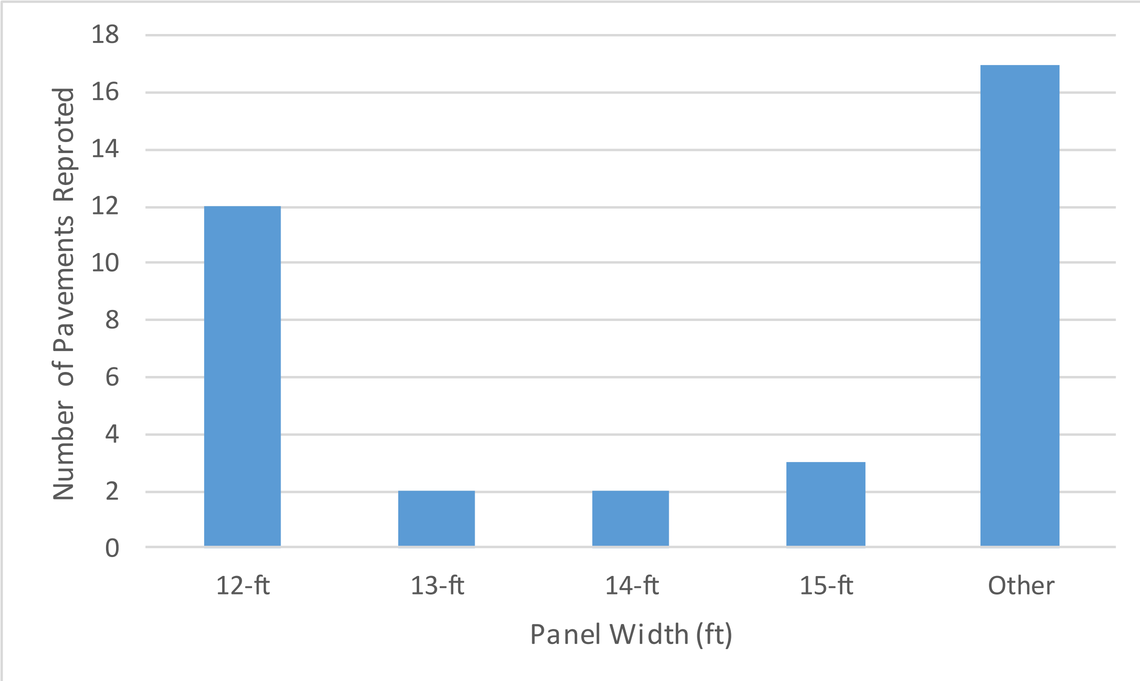
The first survey question related to the typical panel widths applied on two-lane, two-way rural JPCP (Owusu-Ababio and Schmitt 2013). Thirty-three participants responded to this question, and their answers appear in Figure 5. No 14 ft wide panel pavement was reported, and 12 ft wide panel pavements were reported as the most widely used (for 17 JPCPs), while only 2 JPCPs with 13 ft panels were reported. Eight applications were reported to use 15 ft panels for widened pavements.



Owusu-Ababio and Schmitt 2013

Figure 5. Responses from Midwest states based on panel width usage for two-lane, two-way JPCP

Answers in the “Other” category referred to applications of 11 ft wide panels by five respondents, and 13.5 ft wide panels were used by two of the responding counties. In addition to the two-lane, two-way rural JPCP, Owusu-Ababio and Schmitt (2013) investigated panel width usage on multilane rural JPCP highways. Survey results summarized in Figure 6 reflect that panel widths applied in practice ranged from 12 ft to 15 ft. Like the two-lane, two-way rural JPCP, 12 ft panels were most commonly used in these counties (according to 12 of 31 respondents, about 39%), although there were 17 total 11 ft and 13.5 ft wide panels, more than the number of 12 ft wide panels. Two or three applications of 13, 14, and 15 ft panels were reported (Figure 6) (Owusu-Ababio and Schmitt 2013).



Owusu-Ababio and Schmitt 2013

Figure 6. Responses from Midwest states based on panel width usage for multilane rural JPCP

To "ascertain whether the dominant pavement thickness criterion input is used in isolation or in combination with other factors in the selection of panel width," statistical analysis called "cross-tabulation" was performed. The results shown in Table 1 suggested that several additional factors may affect panel width selection, including traffic volume, truck percentage, ease of construction, and construction and maintenance costs. The results also indicated no critical influence of highway functional class (mentioned by only 1 of 15 respondents, 6.7%) (Owusu-Ababio and Schmitt 2013).

Table 1. Additional panel width selection criteria inputs besides pavement thickness in Midwest

Panel Width Criterion Input	Responses Based on Factors Considered in Conjunction with Pavement Thickness
Traffic volume	40% (n=6)
Percent truck traffic	40% (n=6)
Ease of construction	33.3% (n=5)
Highway functional class	6.7% (n=1)
Construction and maintenance cost	26.7% (n=4)

Source: Owusu-Ababio and Schmitt 2013

Another "cross-tabulation" analysis of panel widths and previously determined criteria inputs was conducted for two-lane, two-way rural JPCP, with the results presented in Table 2. For all 15 county survey respondents, pavement thickness was one factor in selecting panel width, with

67% (10 of 15) indicating use of 12 ft panels and 33% (5/15) reporting use of 15 ft panels (Owusu-Ababio and Schmitt 2013). In a manner similar to that used for two-lane, two-way rural JPCP, a "cross-tabulation" analysis using the same input factors was conducted for multilane rural JPCP (Table 3). For example, for the 10 respondents who indicated that traffic volume was a factor in selecting multi-lane panel width, 70% said 12 ft panels were used (Owusu-Ababio and Schmitt 2013).

Table 2. Responses from Midwest states based on panel width relationship to width criteria inputs for two-lane, two-way rural JPCP

	Criterion Input for Panel Width Selection for two-lane, two-way Rural JPCP					
Panel Width	Traffic volume n = 10	Percent truck traffic n = 8	Ease of construction n = 7	Highway functional class n = 6	Pavement thickness n = 15	Construction and maintenance cost n = 7
12 ft	10 (100%)	6 (75.0%)	5 (71.4%)	4 (66.7%)	10 (66.7%)	5 (71.4%)
13 ft	1 (10.0%)	0 (0.0%)	1 (14.3%)	0 (0.0%)	0 (0.0%)	1 (14.3%)
14 ft	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)	0 (0.0%)
15 ft	1 (10.0%)	1 (12.5%)	4 (57.1%)	2 (33.3%)	5 (33.3%)	4 (57.1%)
Other	3 (30.0%)	2 (25.0%)	2 (28.6%)	1 (16.7)	4 (26.4%)	4 (57.1%)

Source: Owusu-Ababio and Schmitt 2013 (n refers to the total number of respondents)

Table 3. Responses from Midwest states based on panel width relationship to width criteria inputs for multilane rural JPCP

	Criterion Input for Panel Width Selection for Multilane Rural JPCP					
Panel Width	Traffic volume n = 10	Percent truck traffic n = 8	Ease of construction n = 7	Highway functional class n = 6	Pavement thickness n = 15	Construction and maintenance cost n = 7
12 ft	7 (70.0%)	5 (62.5%)	5 (71.4%)	2 (33.3%)	8 (57.1%)	4 (57.1%)
13 ft	1 (10.0%)	0 (0.0%)	2 (28.6%)	0 (0.0%)	0 (0.0%)	2 (28.6%)
14 ft	1 (10.0%)	0 (0.0%)	2 (28.6%)	0 (0.0%)	1 (7.1%)	2 (28.6%)
15 ft	0 (0.0%)	0 (0.0%)	1 (14.3%)	1 (16.7%)	1 (7.1%)	2 (28.6%)
Other	3 (30.0%)	3 (37.5%)	2 (28.6%)	3 (50.0%)	5 (35.7%)	2 (28.6%)

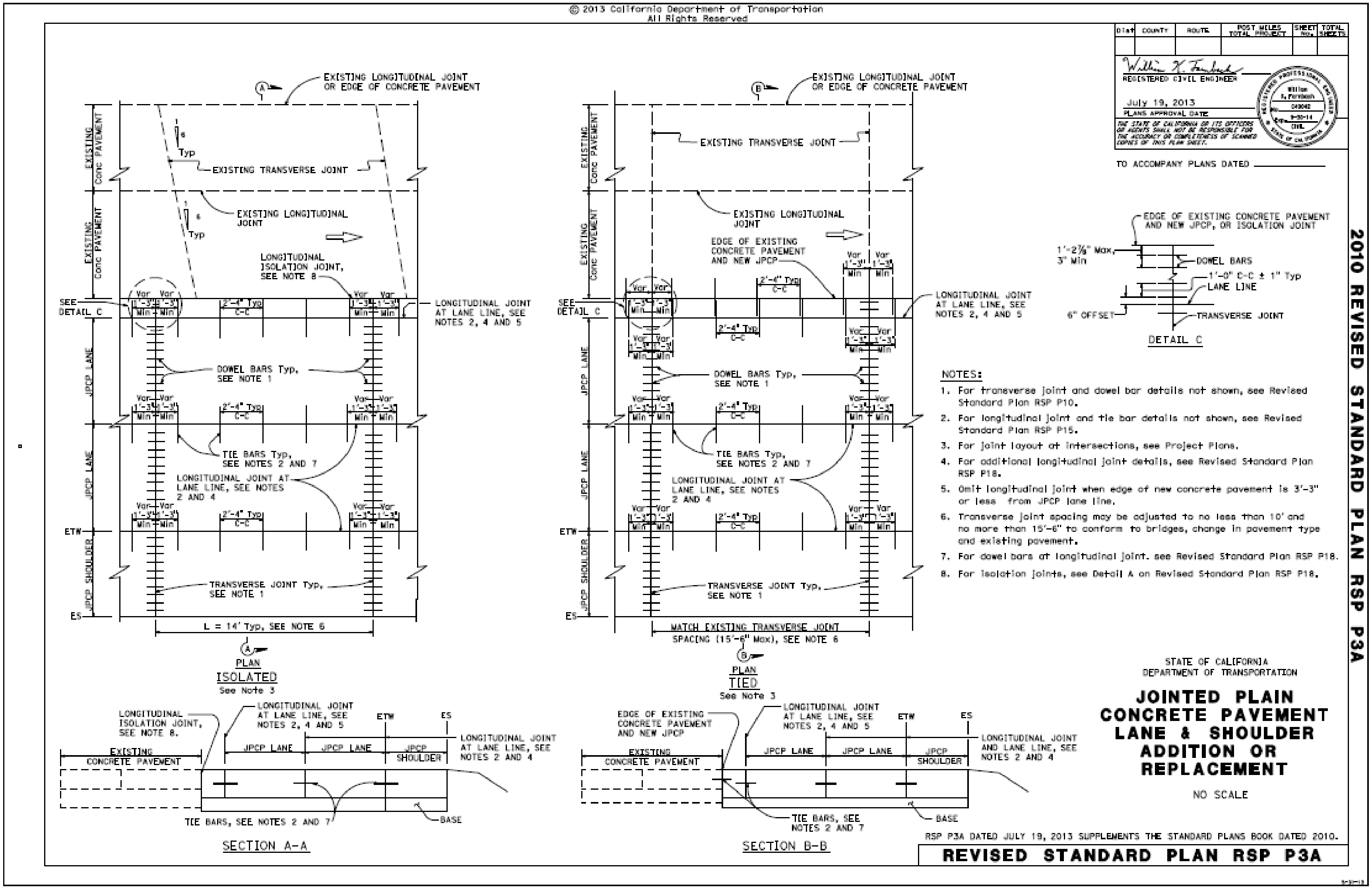
Source: Owusu-Ababio and Schmitt 2013 (n refers to the total number of respondents)

California Department of Transportation (Caltrans)

Caltrans (2010) provides a modified standard plan for lane and shoulder addition or replacement on standard and widened JPCP (Figure 7 and Figure 8), and Caltrans (2015) also documents several further considerations related to JPCP design, describing two conditions when longitudinal isolation joints are needed for JPCP construction. First, if a pavement with dowel bars must be constructed adjacent to a pavement without dowel bars, a longitudinal isolation joint should be considered to solve the problem of required repeated joint spacing for doweled pavements, because the joint spacing may differ between the doweled and non-doweled pavements. The second condition requiring isolation joints is similar, i.e., when the transverse joint spacing pattern needs to be changed from one JPCP section to another. The purpose of placing the isolation joints is to prevent cracks in the widened slabs from the old pavements where the transverse joint spacing is different (Caltrans 2015).

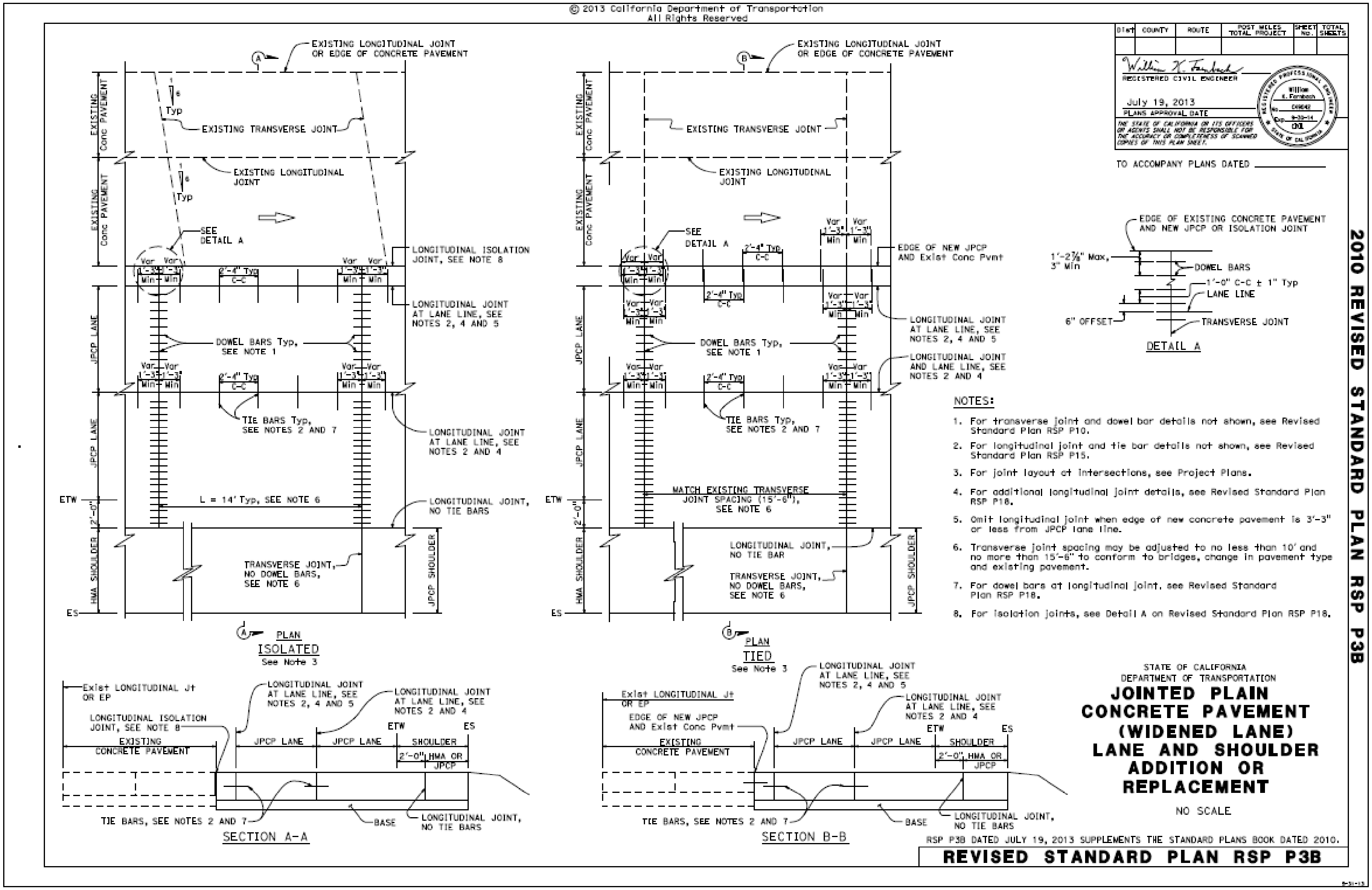
As specified in Caltrans (2010), while lane separation lines must be set in accordance with longitudinal joints, a new problem arising from the widened pavements is the possibility that the longitudinal isolation joints cannot match the lane lines. Caltrans (2015) addresses this problem and recommends using small tied strips of variable width to line up the dimensions. If the difference (width) between the lane line and the isolation joint is greater than 39 in., a longitudinal contraction joint should be constructed at the lane line locations; otherwise, no contraction joint is required. Caltrans (2015) also suggests that the existing adjacent lane should be ground smooth over its full width before starting new lane construction for widening purposes. This will help the paving machine achieve a leveled and smooth pavement surface.

Caltrans (2015) reports several other concerns for widened lanes with hot mix asphalt (HMA) shoulders. Recent construction practices have placed strips between the widened HMA shoulders and the previous 12 ft wide driving lanes, even though the lanes were not tied to the shoulders. These strips helped keep traffic flow away from the shoulders in order to reduce fatigue damage due to loading stresses on the pavement edges. From this viewpoint, and based on data collected from previous investigations, pavements widened through the addition of HMA shoulders provided performance as good as that of pavements with regular width pavements tied to shoulders. A potential concern relates to the thermal properties of different materials (PCC versus HMA) that may result in curling and warping during the service life (Caltrans 2015).



Caltrans 2010

Figure 7. Revised standard plan for JPCP lane and shoulder addition or replacement



Caltrans 2010

Figure 8. Revised standard plan for JPCP (widened lane) lane and shoulder addition or replacement

Florida Department of Transportation (FDOT)

Even though it has been reported in Florida (Nazef et al. 2011) that slab widths must be limited to 14 feet to minimize cracking, information provided by FDOT (2009) separates pavement widening into two aspects: strip-widening and lane addition. Fundamentally, strip-widening implies at least a 3 ft addition to the existing pavements, and lane addition means adding another lane of at least 12 ft width. Even though strip-widening and lane addition have been limited in practice, probably for multiple reasons (cost, right-of-way restrictions, age of existing pavement, vertical and horizontal controls, etc.), the objectives and purposes of such practices are explicit: they are always related to better safety considerations. When the practical width is less than that of the state transportation agency's design criteria, strip-widening might be an alternative to address this problem. Lane addition is an option to meet needs for expanded traffic capacity (FDOT 2009).

If a proposal is submitted to widen an existing pavement, FDOT (2009) raises two questions in evaluating the proposal. First, is the extensive widening necessary to increase service life of the existing pavements based on their condition? Second, are there any future rehabilitation, preservation, realignment, or reconstruction plans for the existing pavements?

FDOT (2009) requires investigation of existing pavements for widening projects. These investigations include researching pavement thicknesses, slab dimensions, embankment soils, and drainage conditions. Pavement thicknesses must be checked at both the road center and the road edge because in-service, relatively older pavements built with thicker edges need more attention. While the remaining life of the existing pavement and the desirable thickness for the designed lane should be estimated for lane addition projects by assessing the equivalent single axle load (ESAL) data, this is not required for strip-widening projects.

For a strip-widening project, no formal analysis of pavement thickness is needed, and the best solution is to match the existing pavement. There are three basic advantages of this approach (FDOT 2009). First, any flow of water between the existing slab and the subgrade will not be disrupted, pooled, or dammed. Second, trenching adjacent to the existing slab and below the slab bottom that may cause a weakening of subgrade support along the pavement edge may be avoided. Third, preservation of existing edge drains systems may be possible (FDOT 2009).

However, a detailed analysis of the remaining service life of the existing pavement is needed before designing the target pavement thickness for a lane addition project. If the calculated thickness is less than that of the existing pavement, the thickness of the new lane should match the existing thickness. On the other hand, if the calculated thickness of a lane addition project is greater than that of the existing pavement, the calculated thickness may be used if adequate drainage can be assured (FDOT 2009). The actual pavement performance still must be evaluated because of possible unresolved uncertainties between engineering judgement and the empirical American Association of State Highway and Transportation Officials (AASHTO) equation.

FDOT (2009) raises three considerations related to embankment influence on widening pavements. First, existing utility clearance relative to the depth of excavation could be a concern, especially in older urban areas. Second, there could be a loss of subgrade support condition along the pavement edge and settlement of adjacent pavement and structures due to excavation. Third, traffic control plans may be necessary in cases where the width of the existing pavement is less than 12 ft, and this could affect the selection of barricades (FDOT 2009).

For the transverse joint, FDOT (2009) specifies that the spacing should be the same as that of the existing pavement if the longitudinal joint spacing is equal to or smaller than 15 ft. It is recommended that, while the new widened pavement does not need to be tied with the existing pavement when the new constructed pavement width is greater than 6 ft, if it is necessary to tie the new and existing pavements, existing transverse joints must be the same and tie bars should offset from the transverse joints by 3 in. Another dowelled transverse joint should be added at the middle of the widened slabs that are equal to or less than 6 ft in width and greater than 10 ft in length (FDOT 2009).

FDOT (2009) also suggests not tying the lane and the shoulder to the existing pavements if concrete is used for widening in order to minimize the potential stresses building up at the shoulders and the road edges. In general, the shoulder should be appropriate for the facility when adding a lane.

New York State Department of Transportation (NYSDOT)

The NYSDOT (2002) recommends two typical values of concrete pavement slab width: 3.6 m (11.81 ft) and 4.2 m (13.78 ft). The widened 13.78 ft slab results from an approximately 2 ft wide extension to a standard 11.81 ft slab. These widened slabs should be applied under different conditions in accordance with different highway types. First, if the highway is two-lane, two-direction, it is required that both driving lanes be 13.78 ft wide. The right lanes should always have widened slabs under multi-lane conditions because, as the NYSDOT (2002) explains, the right lanes always experience greater traffic flows, especially with truck traffic, so the possibility of loading-stress-related distresses in the right lanes is potentially higher than in the left or center lanes. Because traffic sometimes passes across the joints, the lane-shoulder joints are always the most critical locations where such distresses occur. To reduce this type of deterioration, widened slabs are considered to better spread traffic load due to the outer movement of the lane-shoulder joint.

However, the NYSDOT (2002) reports that because widened slabs are not feasible in some cases (e.g., the transition pavements at exit or entrance ramps), regular 11.81 ft slabs must be used. Under this condition, a driving lane becomes one of the center lanes. The NYSDOT (2002) recommends no change in pavement thickness due to the lower ESALs on a ramp compared to those in the main driving lanes. Otherwise, for cases where widened slabs are not feasible, narrow slabs (11.81 ft) with greater thicknesses should be used instead of wide thin slabs to account for the stress issues at pavement edges. Pavement thickness-width ratio is a significant factor that must be considered for widened slab pavement design. Table 4 is a summary of pavement thickness values for all adjacent lanes depending on the amount of traffic (ESALs) and the driving-lane slab width suggested by the NYSDOT (2002).

Table 4. PCC pavement thickness for all adjacent lanes depending on the amount of traffic (ESALs) and the driving lane slab width

80-kN ESALs (millions)	PCC slab thickness for 4.2 m (13.78 ft) driving lane slab width	PCC slab thickness for 3.6 m (11.81 ft) driving lane slab width
ESALs ≤ 22	225 mm (8.86 in.)	225 mm (8.86 in.)
22 ≤ ESALs ≤ 36	225 mm (8.86 in.)	250 mm (9.84 n.)
36 ≤ ESALs ≤ 65	225 mm (8.86 in.)	275 mm (10.83 in.)
65 ≤ ESALs ≤ 100	250 mm (9.84 n.)	300 mm (11.81 in.)
100 ≤ ESALs ≤ 165	275 mm (10.83 in.)	325 mm (12.80 in.)
165 ≤ ESALs ≤ 250	300 mm (11.81 in.)	325 mm (12.80 in.)[1]
250 ≤ ESALs ≤ 400	325 mm (12.80 in.)	325 mm (12.80 in.)[1]

[1] For ESALs over 165 million, 3.6 m (12 ft) untied slabs may not be used for the right-hand driving lane. Use either 3.6 m (12 ft) tied slabs, 4.2 m (14 ft) untied slabs, or 4.2 m (14 ft) tied slabs.
Source: NYSDOT 2002

Similar to Caltrans (2015), an important concern in designing widened slabs for the NYSDOT relates to the tie conditions between lane slabs and shoulder slabs. While it typically is necessary to tie these two slabs for any slab widths, some special applications exist with HMA widened or other untied shoulders. Caltrans (2015) illustrated the proper conditions for constructing untied longitudinal joints (NYSDOT 2002, Chapter 8). For construction cases where driving lanes and shoulders are paved with a single concrete pour, while the slab length can range from 3.5 m (11.48 ft) to 5.5 m (18.04 ft), depending on the utilities and interruptions in pavements, the slab width-length must meet the criteria specified in NYSDOT 2002. This indicates that the maximum slab length and width should both be 5.5 m (18.04 ft).

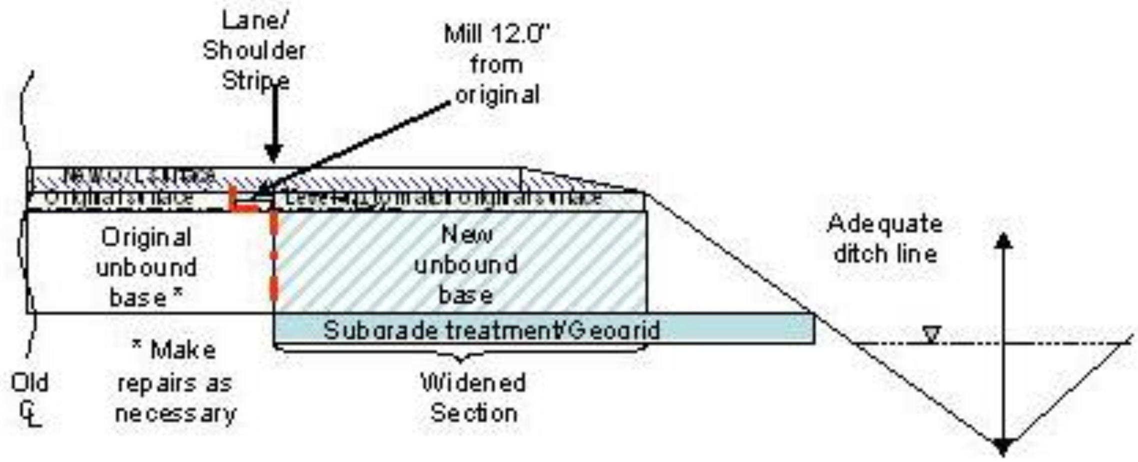
Texas Department of Transportation (TxDOT)

TxDOT (2011) is a documented pavement design guide for flexible pavement widening. Widening aims to improve the road’s traffic capacity with the addition of shoulders, turning bays, etc., and to reinforce driving safety. There are many benefits to keeping the original cross-section in the widened sections. It maintains uniformity in the section, facilitating future evaluation and rehabilitation options for the section as a whole, and preserves subsurface drainage patterns essential to preventing trapped moisture (TxDOT 2011).

However, other design considerations may be unsatisfactory performance of the existing pavements or a desire to quickly complete construction of the widened pavement to minimize traffic disturbance.

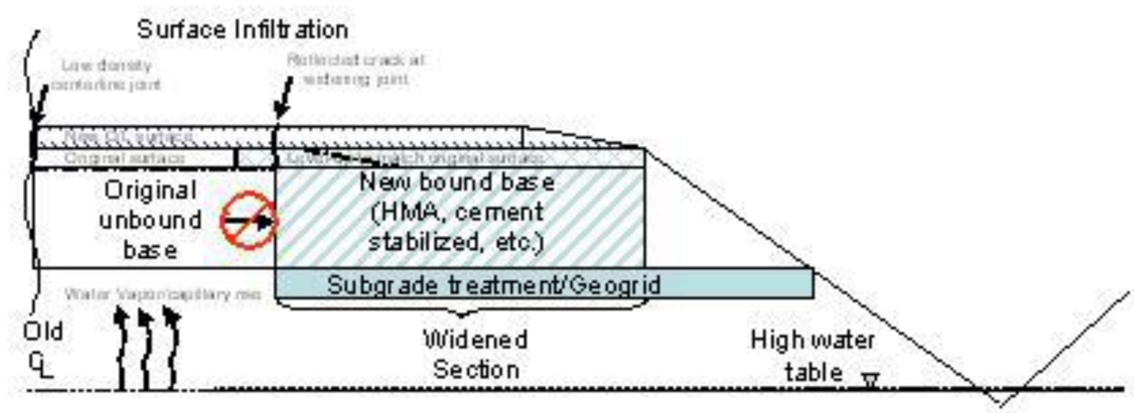
A full-depth joint, always the weakest portion in a pavement structure, is an inescapable result of widening pavements. TxDOT (2011) recommends leaving the distance between the wheel path and these joints as large as possible to improve service life and pavement performance. It also notes that compaction against the vertical plane of the existing structures will be more complex than with full-width construction (TxDOT 2011).

However, TxDOT (2011) reports a concern about possible reflective cracking from the underlying vertical interface through to the pavement surface at the widened section. Moisture (precipitation, melting snow, etc.) entering pavement structures can easily become trapped due to cutoff of a drainage path if sealing is later applied on cracks. TxDOT (2011) therefore recommends adopting geotextiles or a stress-absorbing membrane interlayer (SAMI) over the widening joint before applying the full-width overlay, because this approach could effectively delay reflective cracking. Figure 9 and Figure 10, respectively, show the cross-sections recommended in TxDOT (2011) for unbound and bound base layers in widened pavement sections.



TxDOT 2011

Figure 9. Cross-section of new unbound base in widened section



TxDOT 2011

Figure 10. Cross-section of new bound base in widened section

Significant considerations for similar or dissimilar cross-section widening strategies are as follows:

- To offset the longitudinal joint between original and widened lanes from the lower layer widened-original interface, it is recommended that a HMA overlay be placed 12 in. or more inside the joint next to the outside.
- Water infiltration into the subgrade can potentially cause longitudinal edge cracking, so stabilizing the subgrade layer or reinforcing the base-subgrade interface to control water movement is recommended.
- Moisture susceptibility is a key factor influencing the selection of the new base material, and the target moisture susceptibility of the new base material should be similar to that of the existing base. A significant difference in moisture susceptibilities between the existing and the new bases would probably result in either sending moisture from the existing to the new base or in the opposite direction.
- A sufficient number of ditch lines should be properly placed to avoid moisture flow from outside the pavement structure (e.g., backflow of drained water).
- Construction of the original pavement surface sufficient to level up the entire pavement elevation is recommended (TxDOT 2011).

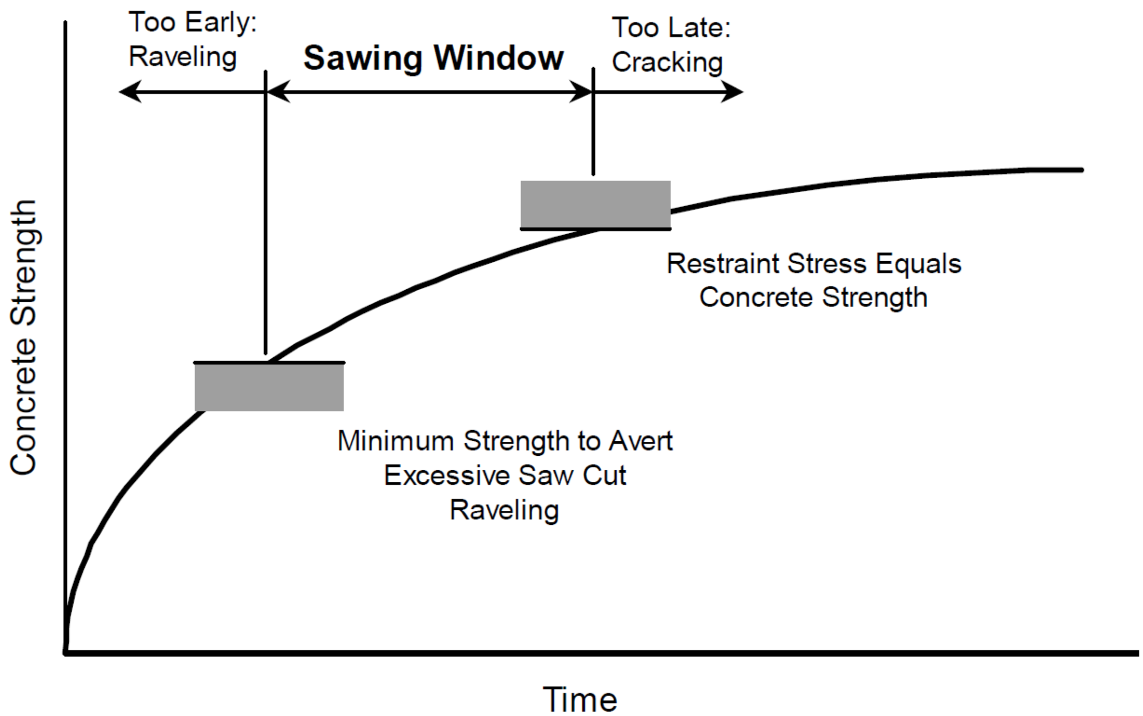
Concrete Slab Longitudinal Cracking—Causes and Mitigations

Voigt (2002)

Joint pavement systems rely on contraction joints to control cracking that usually occurs at regular intervals from environmental effects and concrete shrinkage. However, several interrelated factors can cause unexpected cracks to develop during early pavement age despite proper construction practices. Voigt (2002) summarized the causes of and recommendations for minimizing the occurrence of slab cracking (i.e., transverse cracking and longitudinal cracking). He discussed a series of interrelated factors that lead to crack initiation, including joint technique, local weather conditions, subbase conditions, concrete mixture, curing conditions, slab geometry, dowels, concrete shrinkage, and job site adjustments.

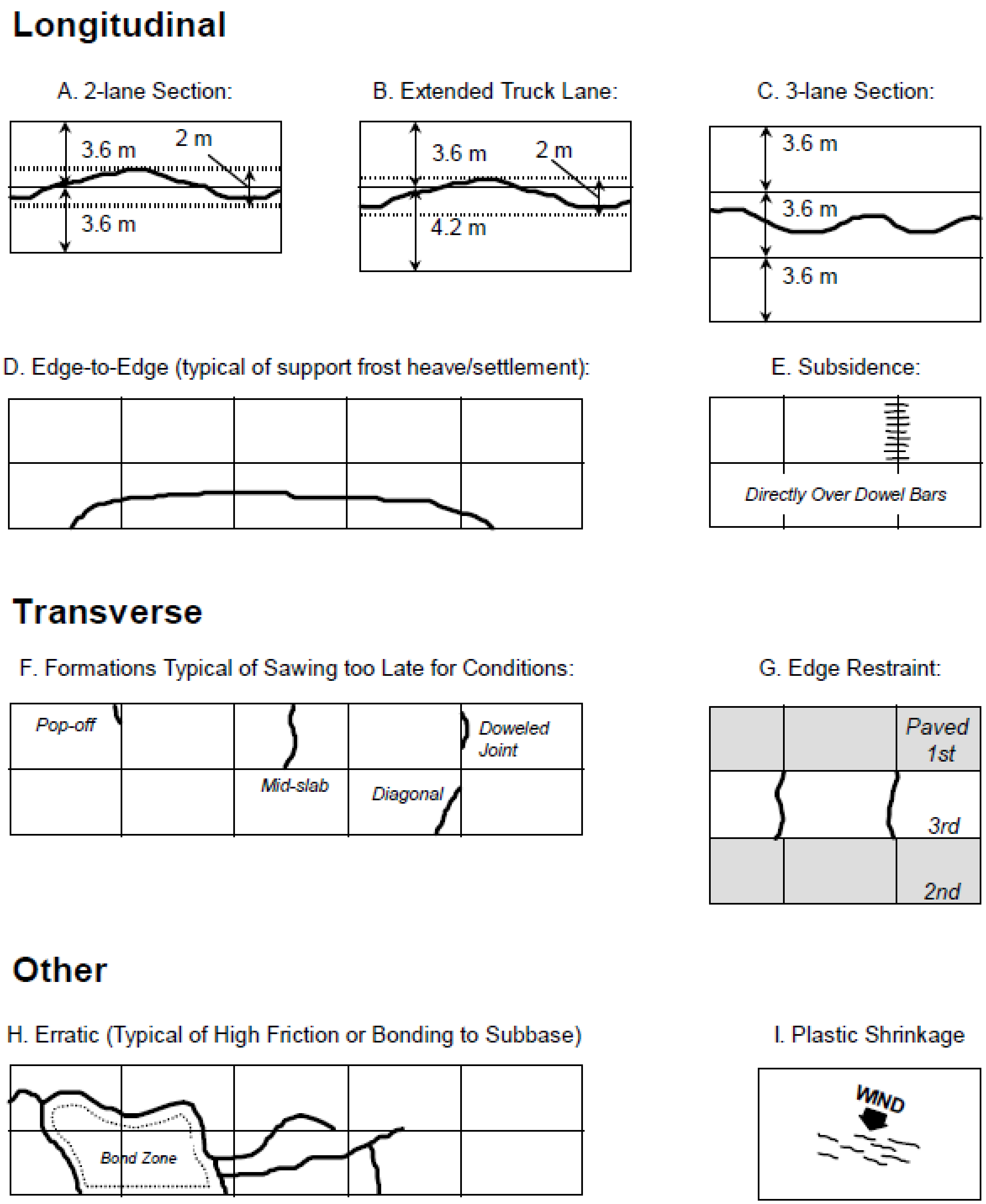
Proper joint technique is a significant factor in crack control. Concrete undergoes an early-age volume change through temperature-related contraction and expansion and moisture-related shrinkage, and this volume change and slab bending could result in tensile stress due to surrounding restraints. Once the tensile stress exerted exceeds the concrete's tensile strength, cracking begins and propagates further by a combination of environmental and traffic loads. While joint sawing is beneficial in providing limited movement to relieve shrinkage-related stress, saw cuts at transverse and longitudinal joints also induce weak points that allow initiation of cracks that will further propagate to the slab bottom.

To achieve successful sawing, the timing and depth of saw cuts are critical, and there is usually an optimal sawing window for a short period after concrete paving (Figure 11). During this window, the concrete should gain sufficient strength to allow sawing without raveling issues. Sawing too early can cause aggregate particles to break free due to the saw blade. Sawing too late may result in an inability to control cracking over time because shrinkage and temperature contraction may cause tensile stress to develop rapidly at the end of window. The tensile stress developed may exceed tensile strength, in which case uncontrolled cracks can be initiated across the entire slab width or between functioning joints. The locations of longitudinal cracking due to late sawing are predictable while locations of transverse cracking are not, and cracks may also develop during the sawing process; they typically occur within about 3 ft of the free edge of PCC slabs (Figure 12). The operator should pay close attention to orient the sawing with the wind direction because wind can accelerate shrinkage along the slab edge, possibly inducing pop-up cracks (Voigt 2002).



Voigt 2002

Figure 11. Sawing window



Voigt 2002

Figure 12. Typical slab cracking formation map

In addition to saw cut timing, sawing depth can also greatly affect the quality of joints in plain concrete, with an impact on early-age cracking formation that primarily depends upon saw cut timing. Sawing depth is usually specified with respect to slab depth (D). Conventionally, deeper saw cuts are preferred, and field experience indicates that a sawing depth between 1/4 D and 1/3 D usually leads to good cracking control in the majority of circumstances. However, limited studies have been performed to investigate crack initiation when the sawing depth is beyond the desired range (1/4 D and 1/3 D). Zollinger et al. (1994) observed better crack control from early-age sawing at a sawing depth less than 1/4 D compared to that obtained between 1/4 D and 1/3D. More studies related to saw cut depth on various pavement designs and mix designs are needed to verify its impact on cracking control because there are many factors that can contribute to longitudinal cracking. Table 5 summarizes typical sawing depth. Furthermore, extra attention should be paid to avoid use of worn blades that may result in inconsistent sawing depth, thus inducing cracks (Voigt 2002).

Table 5. Typical sawing depth for conventional sawing method

	Transverse[1]	**Longitudinal**
Granular Subbases (low friction)	D/4	D/3
Stabilized[2] Subbases (high friction)	D/3	D/3

[1]Early-entry dry saws that permit early sawing do not require these sawing depths.
[2]Stabilized subbase types include asphalt-treated, cement-treated, concrete, lean concrete, asphalt-treated open graded, and cement-treated open graded.
Source: Voigt 2002

Weather conditions can also influence crack formation. Ambient temperature and moisture, solar radiation, and wind speed all may have direct impact on early pavement performance through cement hydration and shrinkage; this can also affect the sawing window, as discussed earlier. Concrete paved in the morning will usually experience higher maximum temperature, more solar radiation, and a shorter sawing window than will afternoon paving (Voigt 2002).

Friction from the underlying base/subbase can exert tensile stress. Such friction is typically affected by bonding conditions between the top PCC layer and base/subbase layer. A stabilized base generally has higher friction and so has a greater possibility of higher tensile stress due to bond restraints compared to a granular base. Moreover, a stabilized base also may suffer from inadequate effective saw cut depth for crack-control purposes (Voigt 2002).

Concrete mix design is another important factor in crack initiation. Improperly designed mix is more susceptible to temperature-related expansion and contraction and moisture-related shrinkage; less cement, less water, and a certain amount of supplementary cementitious material (SCM) are recommended for mixing. Sand with well-graded gradation, lower bulking volume, and proper fineness modulus may also serve to lessen the potential of uncontrolled cracking. Also, based on the field study, crushed limestone exhibits a lesser propensity for crack initiation from concrete sawing (Voigt 2002).

In addition to the factors mentioned above, proper curing condition, joint spacing, dowel alignment, and the use of reinforcement at transverse joints can help reduce the chance for slab cracking. According to Voigt (2002), the following concrete sawing adjustments should be performed at the job site:

- Skip the saw cut when a crack has already formed at the specified location for jointing.
- Do not saw the joint of concrete pavement if a pop-off crack is observed that may lead to spall during sawing.
- Saw only every third or fourth joint if there is a chance for imminent uncontrolled cracks due to upcoming weather changes.
- Use early-age sawing equipment only if a conventional saw cut method for crack control purposes is impracticable.

Ardani et al. (2003)

Similar to Iowa, Colorado adopted widened (14 ft) concrete slabs for traffic lanes, and passing lane widths remained at 12 ft. While the saw cut along the 14 ft slab edge and shoulder usually requires a depth of 0.4 D, the Colorado Department of Transportation (CDOT) observed premature longitudinal cracks in several PCC pavements, so field surveys were conducted to identify possible causes of these unexpected cracks (Ardani et al. 2003).

Generally, longitudinal cracks are primarily initiated due to improper construction practices (Ardani et al. 2003), although heavy load repetition and/or loss of foundation support resulting from heave (i.e., such as frost action or swelling soils) can also accelerate crack propagation. Common improper construction practices include the following:

- Time and depth of saw cutting of the longitudinal joints
- Vibrator trails caused by malfunctioning vibrators operating at excessively high frequency
- Improper treatment of swelling soils with high plasticity index and low R-value
- Inadequate compaction of foundation soil
- Misaligned dowel bars (Ardani et al. 2003)

To discover the reasons for unexpected premature longitudinal cracking, CDOT carried out field surveys at three different locations that had experienced relatively severe premature longitudinal cracking (Ardani et al. 2003). Concrete cores and undisturbed subgrade soil samples were collected for further analysis to identify possible reasons for the longitudinal cracks. Tensile split tests were performed on the concrete cores taken from vibrator trail paths and between the trails. Swelling potential, such as profile index (PI) value and the fraction passing through a No. 200 sieve, was also checked, along with the saw cut depth. The shallow depth of the longitudinal saw cuts along the shoulder joint was found to be the major contributing factor leading to premature longitudinal cracking formation, while malfunctioning paver vibrators and soils with high swelling potential also contributed to the cracks (Ardani et al. 2003).

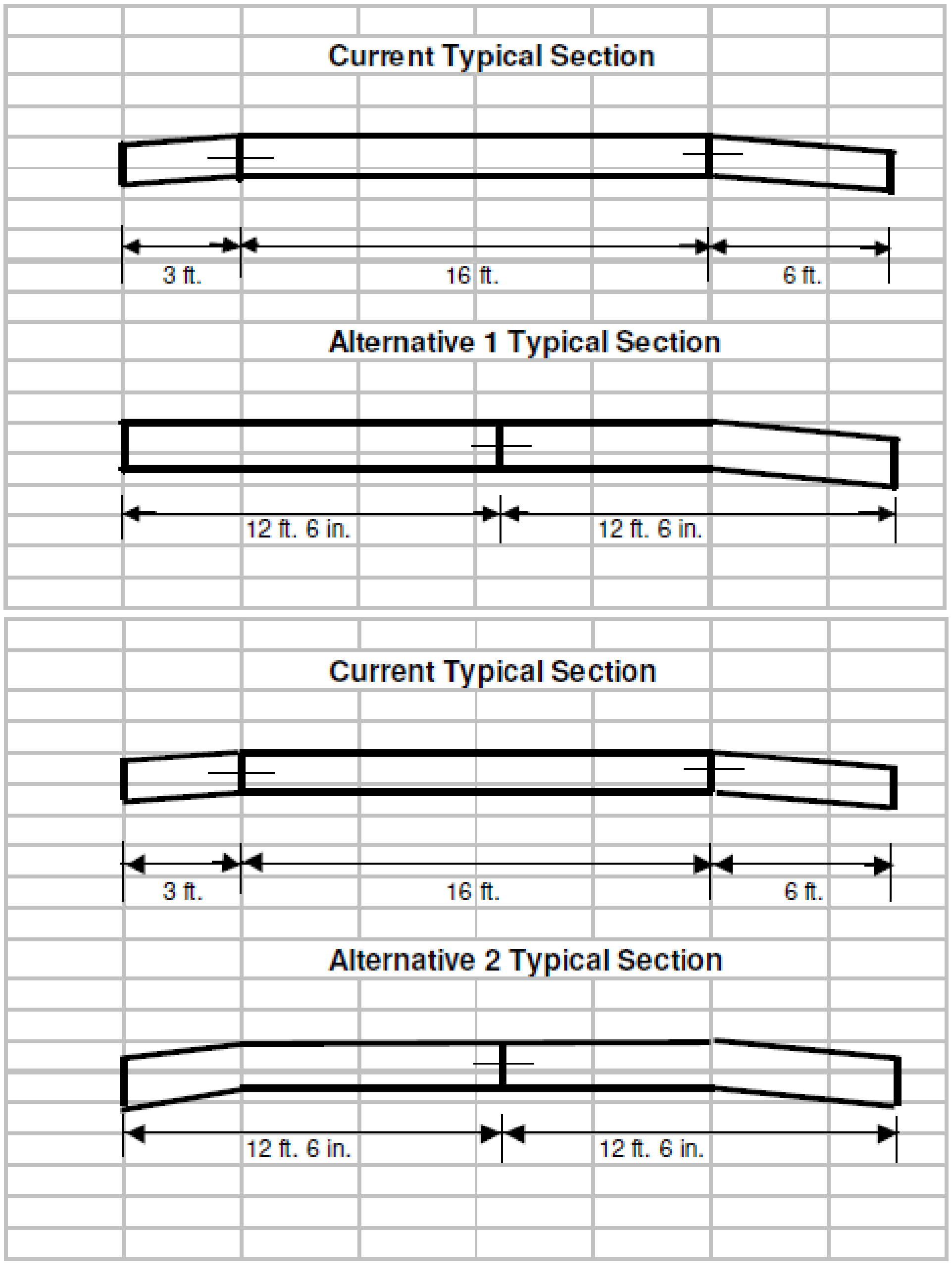
In addition to improper construction practices, design features such as slab geometry, base/subgrade properties, and drainage design can significantly influence premature longitudinal cracking. Other aspects such as material properties and concrete mix design may also affect longitudinal cracking. That study proved that malfunctioning or improperly adjusted paver vibrators could promote premature longitudinal cracking and found that the widened slab design was not a contributing factor with respect to premature longitudinal cracking (Ardani et al. 2003).

To minimize longitudinal crack propagation, Ardani et al. (2003) recommended setting up a related quality assurance/quality control (QA/QC) process for joint saw cutting to make sure that saw cut depth can reach D/3 for 12 ft slabs and 0.4 D for 14 ft slabs. CDOT recommends checking saw cut depth at intervals of 1/10 of a mile (528 ft). Compaction quality should also be monitored by vibrator-monitoring devices. Extra attention should be paid to potential volume change of expansive soils in the subgrade. Proper treatment is required prior to construction to provide better long-term performance and extend pavement service life (Ardani et al. 2003).

Ohio Department of Transportation (2003)

The Ohio Department of Transportation (ODOT) usually adopts a 12 ft wide lane for concrete pavement, although widened lanes (14 ft or larger) are also used in current ODOT practice to provide edge support, with longitudinal joints required for longitudinal cracking control (ODOT 2016).

ODOT in 2003 conducted a survey to investigate cracks on ramps. During that time, ODOT adopted a non-reinforced concrete-based pavement design with 16 ft slab width and 15 ft transverse joint spacing for ramp construction. Right and left side shoulders with 6 ft and 3 ft widths, respectively, and cross-slope breaks were usually added as well. However, longitudinal cracks (mainly located within 2 ft of sawed longitudinal joints and slab centers) were observed in several ramps, and these cracks occurred shortly (from several weeks to six months) after concrete placement. In the proposed ODOT recommendations, a longitudinal joint in the center of the ramp was recommended (ODOT 2003). Two sample alternatives based on the recommendations from the Office of Pavement Engineering in ODOT are shown in Figure 13 (ODOT 2003). Furthermore, in the 2016 pavement design manual from ODOT, ramp design was revised to require a tied longitudinal joint down the middle of the 16 ft wide lane to prevent longitudinal cracks and allow easier repair work on a half-ramp while traffic passed through on the other half-ramp and shoulder (ODOT 2016).



ODOT 2003

Figure 13. Alternatives concrete pavement sections recommended by ODOT

Smiley and Hansen (2007)

The use of non-reinforced JPCP in Michigan can be traced back to World War II. A typical design was used with 20 ft long joint spacing and 8 to 9 in. thick slabs on a dense-graded aggregate base. During the 20th century, jointed reinforced concrete pavement (JRCP) was the predominant pavement type in Michigan. It had excellent ride quality and a smaller number of joints requiring replacement due to material-related distress (i.e., D-cracking), even though the frequent occurrence of transverse cracking and joint spalling (i.e., joint blowups) was observed in the JRCP during its later service life. In the 1990s, JPCP was endorsed nationally by the concrete pavement industry and other states, so Michigan revised its JRCP design and started shifting emphasis to JPCP for all new pavement construction. The current standard recommends using 12 ft width for all highways of two or more lanes, although in some situations an 11 ft wide lane is also acceptable (MDOT Road Design Manual 1999, Smiley and Hansen 2007).

The Michigan Department of Transportation (MDOT) resumed new construction and reconstruction projects using JPCP in the middle of the1990s because of lower life-cycle costs compared to those of JRCP. Table 6 illustrates the pavement design parameter and condition data of the JPCP projects in the late 1990s. However, mid-slab transverse cracking and a limited amount of longitudinal cracking of highly variable extent and severity were observed in five of these projects shortly after pavement construction. Smiley and Hansen (2007) investigated the causes of this crack initiation and propagation within these projects. Results appear in Table 6.

1. Project I-96/I-275 Connecting Ramp (CS 63191)
2. Project US 12 (CS 82061)
3. Project I-75 (CS 82194)
4. Project I-96 (CS 47065)
5. Project I-94 near Watervliet (CS 11017)

Table 6. JPCP design parameter and pavement conditions for projects constructed in the later 1990s

Route	CS	ADT 1999	L (ft)	T (in.)	Widen Y or N	Shoulder Type	Description	Slabs Cracked (%)
I-96 Ramp	63191	3,500	16	12	N	Tied Conc.	13 slabs are cracked - in sequence	1.0
I-94	11017	4,000	Var[1]	12	Y	Bit.	Most TCs are spalled[2]. Some slabs with 2 cracks.	EB 60 WB 10
I-96	47065	3,500	16	11	N	Conc.	Almost all TCs are on EB in clusters	EB 6.3 WB <1
US 27	19033	1,050	14	9	N	Bit.	No visible cracking	0
EB US 12	82061	500	13 to 19	8.5	N	C&G	Isolated cracking from loading and jointing flaws	< 1
I-75	82194	6,000	16	12	N	Tied Conc.	Both TC and LC - mostly clustered in isolated areas	NB 0.39 SB 0.14
EB I-94	11018 80023	5,300	15	12	Y	Bit.	No visible cracking[3]	0
US 23	81076	3,700	15	11	Y	Tied Conc.	Not surveyed for distress	unknown
I-75	82195	6,000	15	12	N	Tied Conc.	No visible cracking	new
I-275/ I-96	82125 63191	7,500	15	12	N	Tied Conc.	No visible cracking	new
EB I-94	80024	5,300	14	12	Y	Ex. Conc. Rt New Conc. Lt.	No visible cracking	new
I-94	80023	5,300	14	12	Y	Ex. Conc. Rt. New Conc. Lt.	No visible cracking	new
I-69[4]	12033	2,750	15	10	Y	Bit.	No visible cracking. "Rocking" ride on NB.	new
M 14	81105	2,000	15	11	Y	Bit.	No visible cracking	new

[1] Three joints are spaced at approximately equal length on EB: random @ 15-16-17 ft, "Illinois" hinge, and 16 ft uniform (location from west to east). WB is entirely uniform 16 ft lengths. I-94 was a trial project approved by EOC (January 1995).
[2]Approximately 1,000 ft (portion with LC) at east end of EB was replaced in 1998 with project no. 38094. Spalling is entirely on approach (after) side of crack requirements.
[3]In the summer of 2002, Southwest Region reported project had developed mid-slab cracking. Project investigated in parallel UM study.
[4]SB I-69 has test location with unsealed contraction joints between MP 3.6 (STA 315+800) and MP 4.8 (STA317+800). Signs mark location. Normal expansion joints were used.
Source: Smiley and Hansen 2007

Among the five projects investigated, the main cause for cracking development in project I-96/I-275, project US 12, and project I-75 was poor construction practice such as improper base/subbase and subgrade course treatment and a high degree of built-in curling that could cause loss of support from underlying layers. Insufficient support resulted from slab settlement due to underlying frost heaving, and high deflection due to slab curling at transverse joints could exert very high tensile stress under traffic loading, possibly initiating longitudinal cracking. Inadequate pavement joint layout and drainage structure placed without joint isolation could also contribute to slab cracking (Smiley and Hansen 2007).

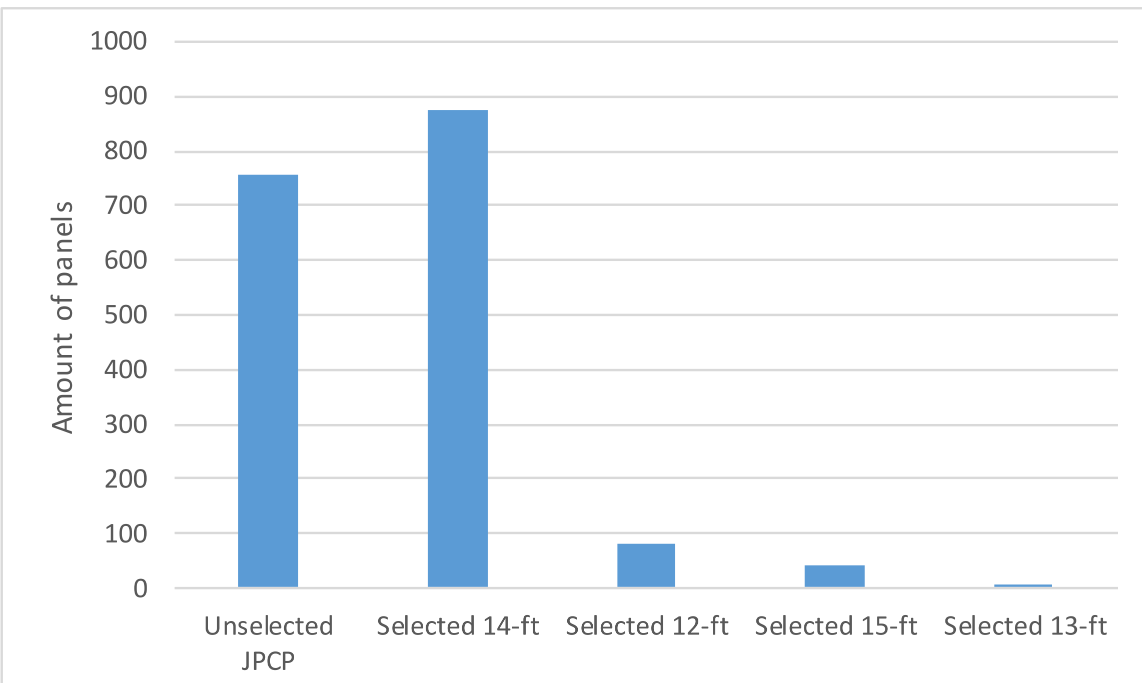
According to the investigation by Smiley and Hansen (2007), I-94 and I-96 were the most significant and revealing projects. In these two projects, cracking development was due to a complex combination of factors rather than solely to poor construction. Transverse cracking was first observed for project I-94 during the summer just one year after construction. The eastbound segment of route I-94 was built during the late summer of 1995 while the westbound direction was built during early summer of 1996, and longitudinal cracking was observed during paving, while pronounced longitudinal cracking had developed by the end of the project. According to the field survey on the eastbound direction conducted in December 1996, 30% of slabs were cracked among three paved sections, and most of the cracked slabs had been paved during the morning hours. After three years, another survey carried out during December 1999 showed that the percentage of cracked slabs increased to about 75% for two sections and 40% for another section. Similar to project I-94, project I-96 also exhibited a distinct contrast in terms of the extent and severity of slab cracking, with the two projects having similar pavement design and concrete mixture, as well as the same construction company. Based on field investigation, both of the projects were found to exhibit a very high deflection and loss of support at the transverse joints, mainly due to a high degree of slab curling and localized consolidation and settlement of the base/subbase course, resulting in high tensile stress developed under traffic loading applied to uplift at the slab corners. In addition, poor load transfer efficiency at the transverse joints and simultaneous joint loading resulting from fully loaded multi-axle truck traffic contributed to crack initiation, based on field and laboratory test and finite element modeling results. Notably, this study also indicated that JPCP seems to be sensitive to top-down transverse cracking initiation and rapid crack propagation when loss of support and simultaneous joint loading exist (Smiley and Hansen 2007).

To improve pavement performance by minimizing slab cracking, construction practice should avoid freezing and non-uniform thickness in the base/subbase course. Proper base/subbase treatment, placement of open-graded base, concrete curing, and aggregate moisture control are keys to guaranteeing good JPCP performance. To minimize built-in curling effects, construction time and concrete mixture should be adjusted to avoid extreme hot weather for concrete mixing and paving (Smiley and Hansen 2007).

Owusu-Ababio and Schmitt (2013)

Owusu-Ababio and Schmitt (2013) performed a literature review on the topic of longitudinal cracking on concrete panels with different panel widths. They concluded that several interrelated factors, including temperature and moisture gradients between slab top and bottom, jointing practices, and base material type, may result in longitudinal cracking in JPCP. Owusu-Ababio and Schmitt (2013) reported that theoretically concrete panels crack when subjected to tensile stress that is greater than their tensile strength. Causes of this phenomenon might include initial shrinkage from moisture loss, restraint by base or subbase friction from expansion and contraction caused by temperature changes, and thermal and moisture gradients between the top and bottom of the panels. Based on the literature, Owusu-Ababio and Schmitt (2013) showed that some new pavements may crack during the first two months immediately after construction. While these cracks initially appear at large intervals (30 to 150 ft) and form at closer intervals as service life increases, most longitudinal cracking can be summarized as random and uncontrolled in nature. Slab restraint or movement that results in high tensile stress development within the slabs might result in initial random cracking after paving and sawing. Non-uniform settlement and frost heave with restraint from the stabilized foundation layers may lead to the movement.

Owusu-Ababio and Schmitt (2013) conducted a study comparing the performance of 14 ft concrete panels to that of regular 12 and 13 ft standard sized panels. This study formed the basis of a technical online survey in the Midwest and broad field investigations in Wisconsin. Another significant objective of this study was to document the relationship between maximum allowable pavement widths and optimal pavement thickness. For the field investigation, a total of 1,008 of 1,767 concrete segments in Wisconsin, with an average segment length of 1.14 mi., were selected (Figure 14). These segments included 12, 14, and 15 ft wide concrete panels. Longitudinal cracks existed in 60% of these pavement segments, while the 14 ft panels had the lowest rate of longitudinal cracking, 56%. The corresponding percentages for the 12 and 15 ft panels were 81% and 84%, respectively. Most longitudinal cracking was observed between wheel paths or between the right wheel paths and the right edge. A minor finding was that all eight 13 ft wide panels showed longitudinal cracks (Owusu-Ababio and Schmitt 2013). Table 7 summarizes the results of this field investigation.



Owusu-Ababio and Schmitt 2013

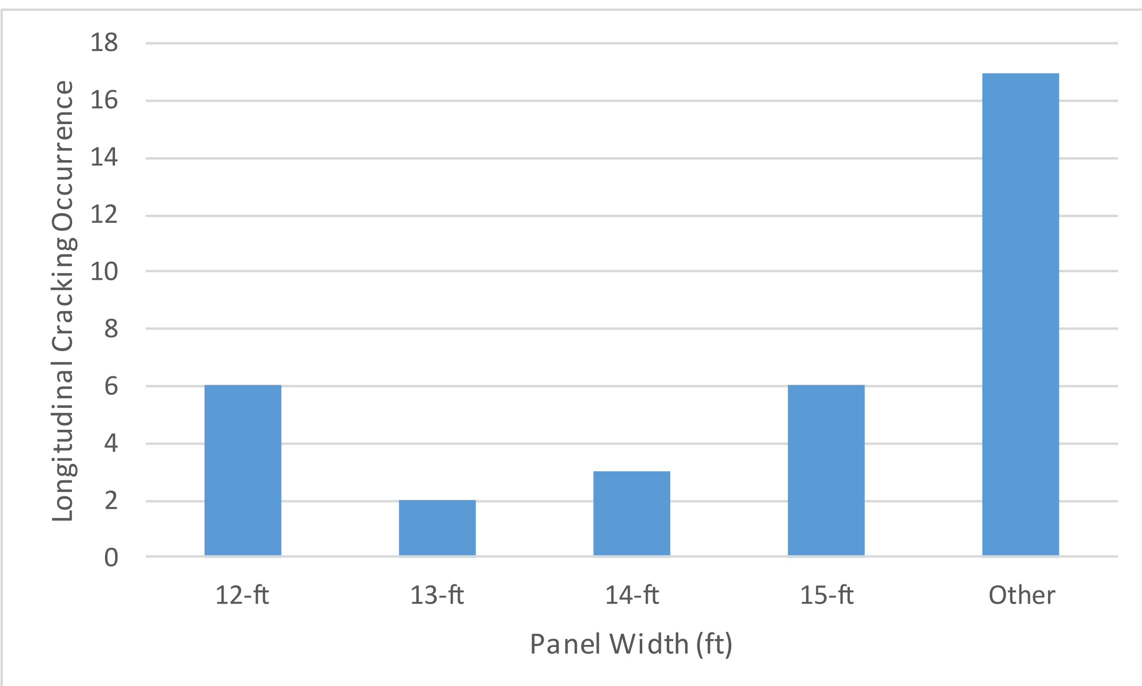
Figure 14. Distribution of widths of JPCP panels in Wisconsin

Table 7. Summary of longitudinal cracking on JPCP in Wisconsin

Panel Width	Number of selected panels	Number of panels with longitudinal cracking	Rate of cracked panels
12 ft	81	66	81%
13 ft	8	8	100%
14 ft	877	491	56%
15 ft	42	35	84%

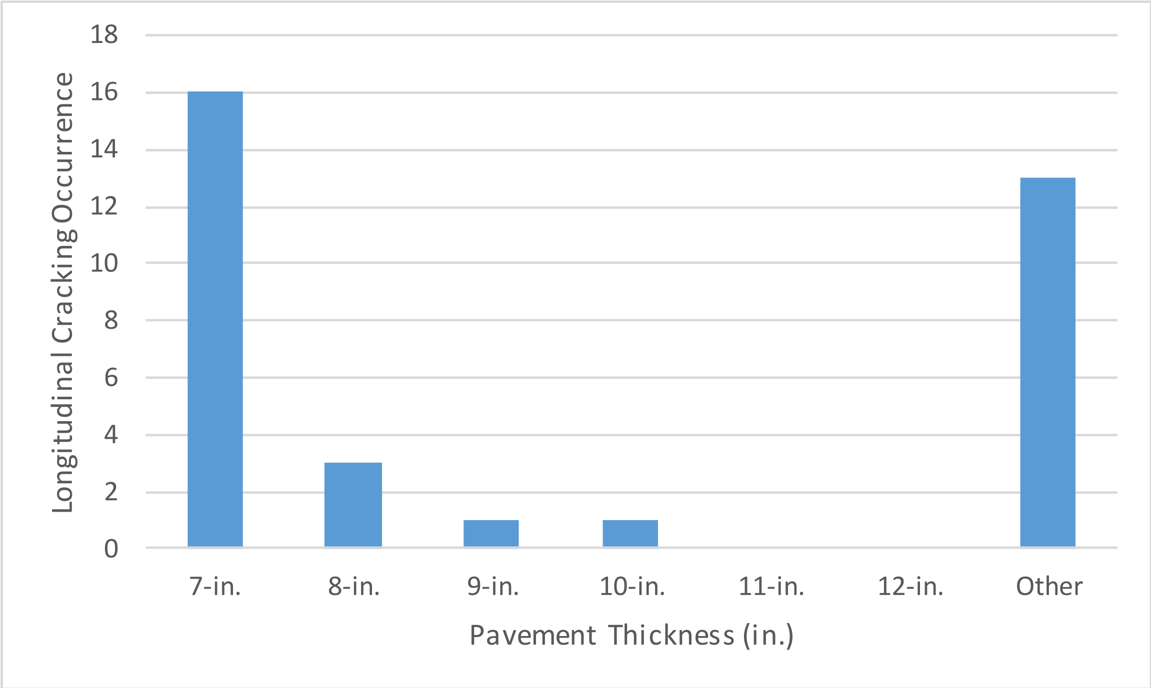
Source: Owusu-Ababio and Schmitt (2013)

Owusu-Ababio and Schmitt (2013) carried out an online survey of JPCP practices (mainly on rural highways) in six Midwest states and asked how those practices affected the development of longitudinal cracking. Although 522 county engineers and pavement professionals in those six Midwest states were invited to participate, only 37 invitees ultimately completed the survey. One major reason for the low response rate was that many of the county engineers invited to participate in the survey quickly expressed their inability to do so because they experienced no JPCP failure within their jurisdiction. Based on the survey results, panel width (Figure 15) and slab thickness were significant influences on the occurrence of longitudinal cracking, and no longitudinal cracking was observed on pavements with thicknesses greater than 11 in. (Figure 16).



Owusu-Ababio and Schmitt 2013

Figure 15. Responses from Midwest states based on panel widths exhibiting longitudinal cracking occurrence



Owusu-Ababio and Schmitt 2013

Figure 16. Responses from Midwest states based on thicknesses exhibiting longitudinal cracking occurrence

The survey responses from 24 participants indicated that premature longitudinal cracking initiation time varied from less than 1 month to as high as 60 months, with an average initiation time of about 24 months. All highlighted findings from the JPCP longitudinal cracking survey (two- and multi-lane rural highways) conducted by Owusu-Ababio and Schmitt (2013) are summarized in Table 8.

Table 8. Summary of findings from the survey of JPCP panel width practices and longitudinal cracking in six Midwest states

Factor	Findings
Criteria for selecting JPCP panel widths	Pavement thickness is the dominant factor. Other factors: traffic volume, percentage of trucks, ease of construction, and construction and maintenance costs.
Standard JPCP panel widths used	12 ft (followed by the 15 ft) wide panels on two-lane, two-way rural pavements.
Effect of JPCP panel widths on frequency of longitudinal cracking	The 12 ft and 15 ft panel widths experience higher cracking frequencies than 13 ft and 14 ft wide panels.
Effect of JPCP slab thickness on frequency of longitudinal cracking	A JPCP thickness of ≥ 11 in. seems to reduce the chances of longitudinal cracking.
Effect of JPCP transverse joint spacing(s) on frequency of longitudinal cracking	Shorter joint spacing contributes to more longitudinal cracking.
Possible construction practices that contribute to premature longitudinal cracking	Inadequate subbase compaction, poor joint saw cut timing, misaligned dowel bars, and faulty vibrators, as well as the use of 12 ft panels contribute to construction-related premature longitudinal cracking.
Common locations for appearance of longitudinal cracking	More near panel edge and at mid-panel locations and fewer near the sawn longitudinal joints.
Time elapsed before the first appearance of premature longitudinal cracks	Average initiation time is 24 months (range: 1 to 60 months).
Main methods for repairing premature or normal longitudinal cracking	Rout and seal, cross-stitching, and partial or full panel replacement.

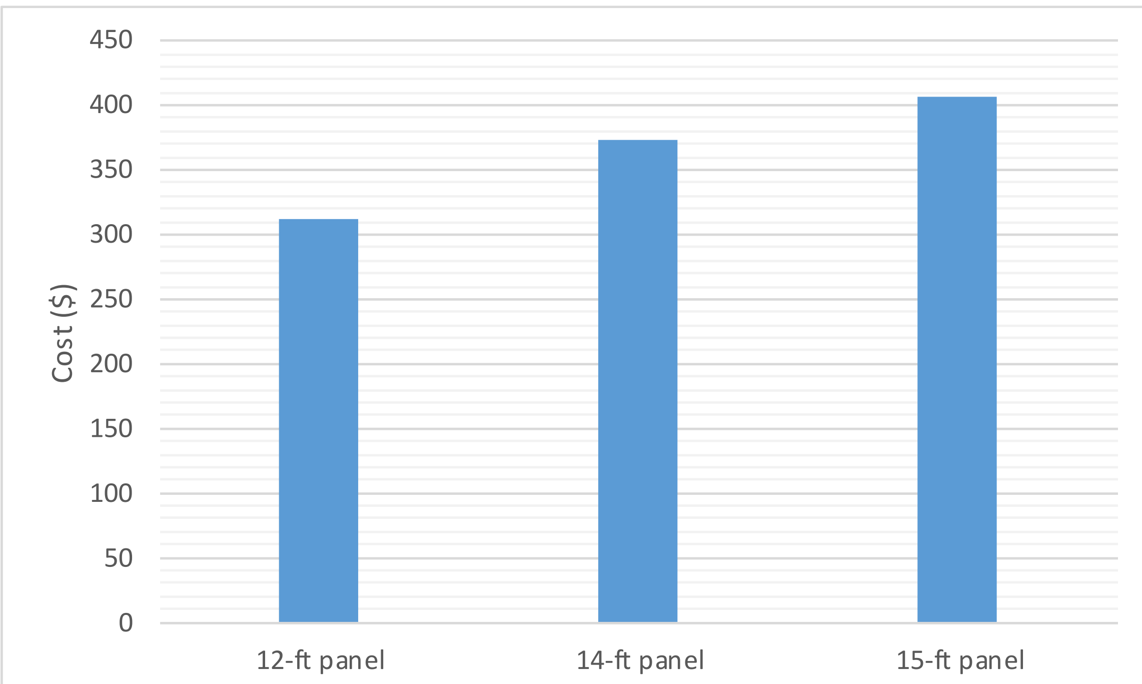
Source: Owusu-Ababio and Schmitt (2013)

In addition to the related factors summarized by the survey, elements influencing the presence of longitudinal cracking and the severity/extent of the existing longitudinal cracks were also reported by Owusu-Ababio and Schmitt (2013). “Width-to-thickness ratio, joint spacing, longitudinal jointing method, tining orientation, dowel bar installation, traffic level, age, and region” are factors affecting both the presence and severity of longitudinal cracking, and “rumble strips [and] base gradation (dense or open)” also influence its severity and extent (Owusu-Ababio and Schmitt 2013). When these statements from the survey and the field investigation are combined, it can be concluded that the JPCP width-thickness ratio is the most significant consideration with respect to longitudinal cracking.

Owusu-Ababio and Schmitt (2013) therefore explored the relationship between the rates of longitudinal cracking on widened concrete panels and pavement thickness variations. For the 14 ft panels, reducing the width-thickness ratio meant increasing pavement thickness from 9 in. to

10 in., resulting in a 25% decrease in the quantity of longitudinal cracking. The length of longitudinal cracking increased by 45% for 14 ft panels and by 18% for 15 ft panels when the width-thickness ratio increased from 1.4 to 1.6 and 1.5 to 1.7, respectively (both thickness increases are from 9 to 10 in.) (Owusu-Ababio and Schmitt 2013).

Based on a study focusing on rural highways, Owusu-Ababio and Schmitt (2013) also provided some insight into various concrete pavement width alternatives and the relationships between their performances and cost-effectiveness based on life-cycle cost analysis. For those selected highways, the overall increment in cost per 1.14 mi. from increasing the pavement thickness from 9 in. to 10 in. ranged from $25,000 to $28,500, while the corresponding length of longitudinal cracking due to the 1 in. increase in pavement thickness was about 70 to 80 ft. The authors reported that for a unit (in.) increment in pavement thickness, the 12 ft panel produced the minimum overall incremental cost of $312 per foot reduction of crack length. Corresponding costs for the 14 ft and 15 ft panels were 1.2 and 1.3 times those of the 12 ft panel, respectively, those for the 12 ft panels (Owusu-Ababio and Schmitt 2013) (Figure 17).

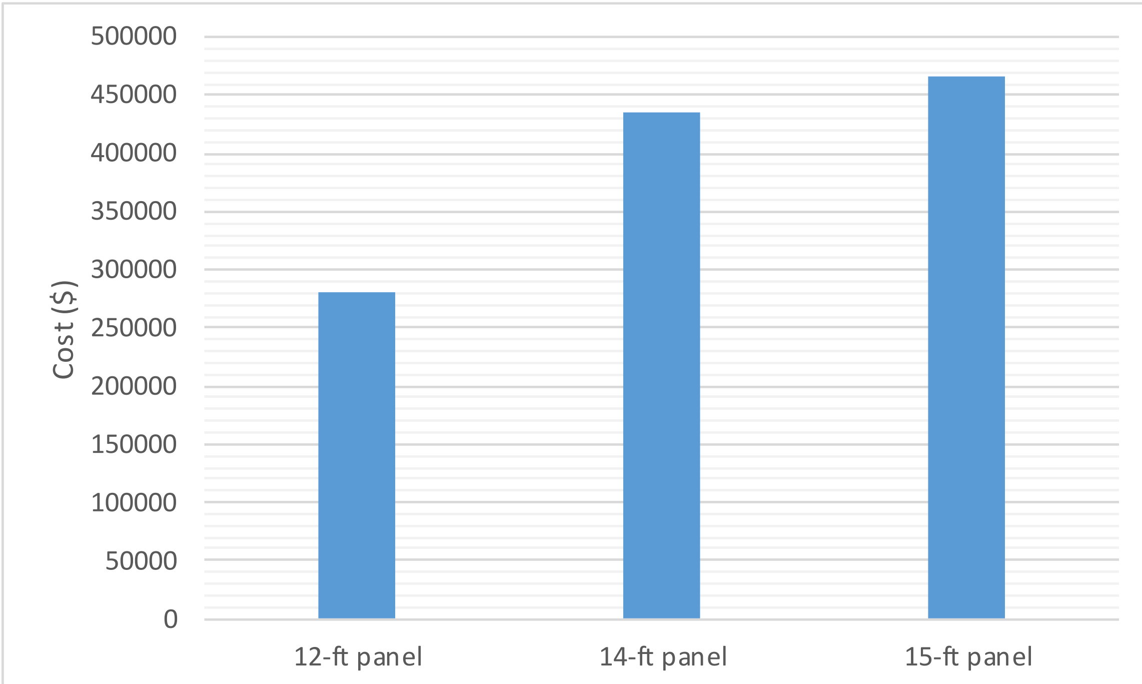


Owusu-Ababio and Schmitt 2013

Figure 17. Comparison of cost increments per foot reduction of crack length for 1 in. increment in pavement thickness

Considering the rehabilitation/maintenance costs, concrete panels with different widths presented various cost values. For the 12 ft panels, pavements with 9 and 10 in. thicknesses exhibited the highest maintenance costs; however, the overall maintenance cost for all 12 ft panels was the lowest in comparison with pavements with other panel widths. The average maintenance cost of the 15 ft panels was the highest, 1.7 times that of the 12 ft panels and 1.1 times that of the 14 ft panels. The detailed cost value for these panels appears in Figure 18 (Owusu-Ababio and

Schmitt 2013). Table 9 summarizes the average segment crack length and cost relationships for panel widths.



Owusu-Ababio and Schmitt 2013

Figure 18. Rehabilitation/maintenance costs of JPCP with different panel widths per 1.14 mi. in Wisconsin

Table 9. Average segment crack length and cost relationship for panel widths

Panel width	Pavement thickness	NPW[1]/1.14 mi. ($)	Ave. crack length/1.14 mi.	Cost increment ($)	Crack length reduction	Cost/unit crack length reduction ($)
12 ft	9 in.	638342	100 ft	24,924	80	312
	10 in.	663266	20 ft			
14 ft	9 in.	784075	238 ft	27,295	74	369
	10 in.	811370	164 ft			
15 ft	9 in.	821459	430 ft	28,482	68	419
	10 in.	849941	362 ft			

[1] Net present worth
Source: Owusu-Ababio and Schmitt (2013)

To minimize the effect of induced stresses in concrete panels, Owusu-Ababio and Schmitt (2013), based on their literature review, concluded that proper jointing techniques should be applied in creating transverse and longitudinal saw cut joints. They also made several recommendations based on the field study with respect to JPCP panel width selection. They suggested using 14 ft wide panels with width-thickness ratios ranging from 1.2 to 1.5, resulting in pavement thicknesses between 9.5 in. and 12 in. Furthermore, normal joint orientation, longitudinal tining, dowel basket installation, and untreated aggregate base, together with the 14 ft panels, were recommended. While 15 ft of transverse joint spacing provided the best performance in conjunction with those panels, Owusu-Ababio and Schmitt (2013) also stated that longitudinal cracking can potentially occur at any location on the pavement, including wheel paths, edges, and between wheel paths. For 14 ft panels, the extent of longitudinal cracking at all locations (except for those near the left edge) can be reduced through the use of a PCC rumble strip installation and open graded base. Otherwise, the width-to-thickness ratio is an important factor to consider for mid-panel cracking.

Johnston (2014)

Johnston (2014) presented issues and solutions related to longitudinal cracking of pavements in South Dakota (Figure 19). Several key features related to longitudinal cracking on new pavement projects since 1986 were summarized as follows:

- Length of cracking extends for miles.
- Cracking is typically confined to lane center and outer wheel path.
- Cracks continue to appear over time.
- Cracking predominates in wider 28 to 30 ft pavements compared to conventional 24 ft pavements.
- Cracking shows statewide distribution (Johnston 2014).

Johnston 2014

Figure 19. Longitudinal cracking noticed in South Dakota

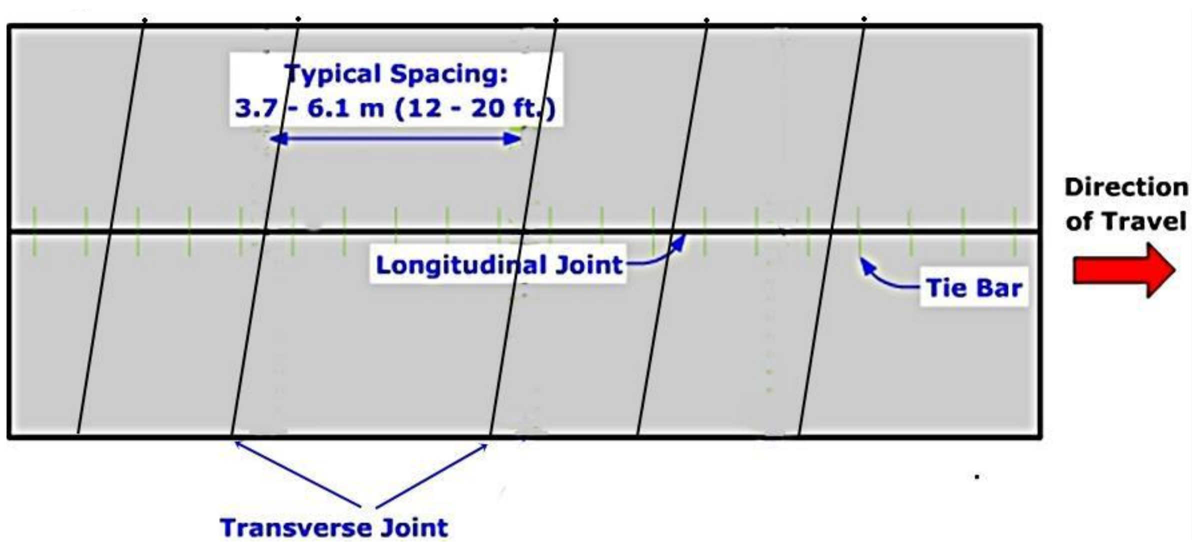
Several traditional explanations were provided and accepted as causes of longitudinal cracking in South Dakota. First, the consequences of cut depth for saw cutting at the centerline was considered a primary reason. Before noticing this problem, saw cut depth was controlled at 1/3 of the pavement thickness (Johnston 2014). To address the inadequacy, more aggressive sawing to ensure relief was performed by changing the saw cut depth to 1/4 of the pavement thickness, and related provisions or specifications were immediately planned and implemented. Unfortunately, longitudinal cracking still occurred. Johnston (2014) then summarized the major changes in current pavement projects that might be possible causes for longitudinal cracking (Table 10).

Table 10. Changes in pavement projects in South Dakota

Factor	Changes
Concrete materials	Aggregates are interlocked properly
Costs and timing	Savings on dowel elimination and faster construction
Joint construction	Randomly spaced skewed joints
Pavement thickness	No increase in slab thickness
Shoulder construction	Partial concrete shoulders (about 2 to 3 in.)

Source: Johnston 2014

Johnston (2014) conducted a statewide survey on concrete pavements to collect longitudinal cracking data and evaluate the correlation of longitudinal cracking with design features. In addition to the two primary factors, i.e., pavement width and thickness and base/subgrade thickness and strength, skewed joints were also evaluated based on mechanistic theories to assess longitudinal cracking (Figure 20). Skewed joints with typical joint spacings of 12 to 20 ft were widely constructed in South Dakota to reduce pumping and faulting due to lower shear from axial loading, and to improve harmonic ride effects (Johnston 2014).



Johnston 2014

Figure 20. Top view of skewed joint

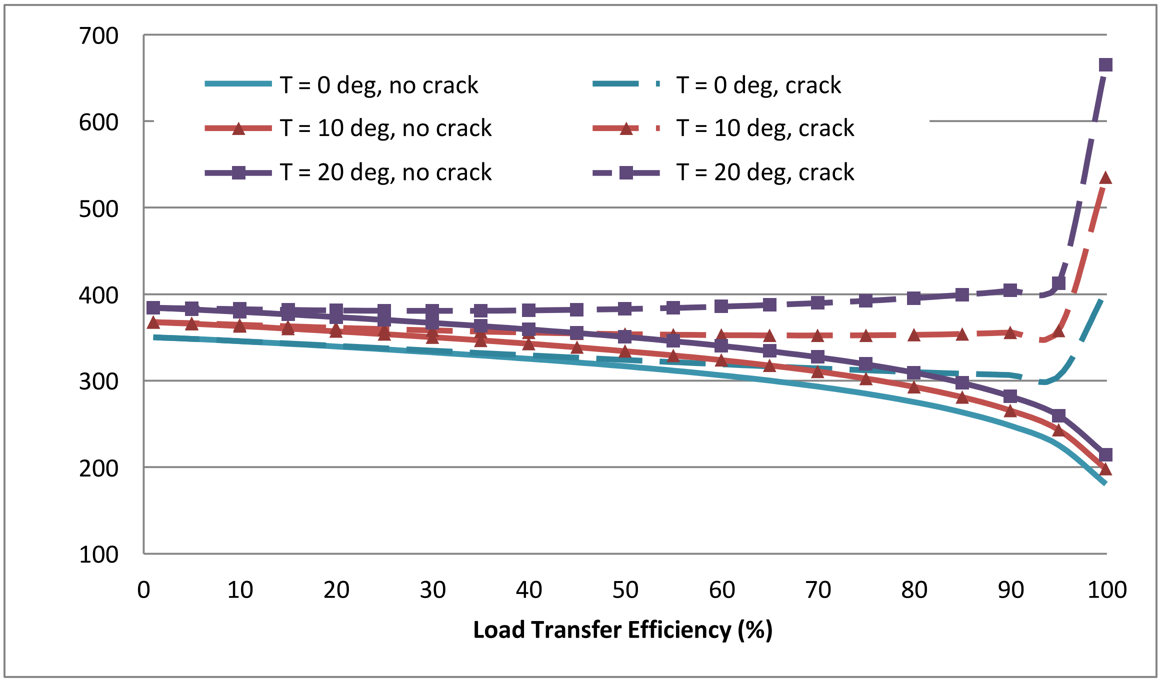
Furthermore, while additional width was supposed to provide more safety considerations for rumble strips on concrete shoulders, Johnston (2014) reported that a load cantilever from skewed joints imparts excessive tensile strain in a slab. With respect to the pavement width/thickness factor, it was noted that the extent of cracking on 15 ft panels was worse than for other panels. If pavement thickness remained constant, increasing pavement width resulted in more warping and curling, and cracking seemed to occur immediately after opening to traffic. With respect to base/subgrade, the extent of cracking was found to be worse when foundation layers were considered weak. At locations where there were larger amounts of gravel in the base layers, less cracking was observed. Non-uniform pavement thicknesses also had the potential to accelerate cracking. Based on these findings, the general conclusion was that cracking on concrete panels occurred because of the combined causes of wider slabs, skewed joints, and traffic loads, and the following recommendations were made:

- Keep using dowel bars for load transfer
- Eliminate skewed joint applications
- Increase gravel base to 5 in. after trimming
- Confine slab width up to 14 in.
- Use minimum slab thickness of 8 in. (Johnston 2014)

Lederle (2014)

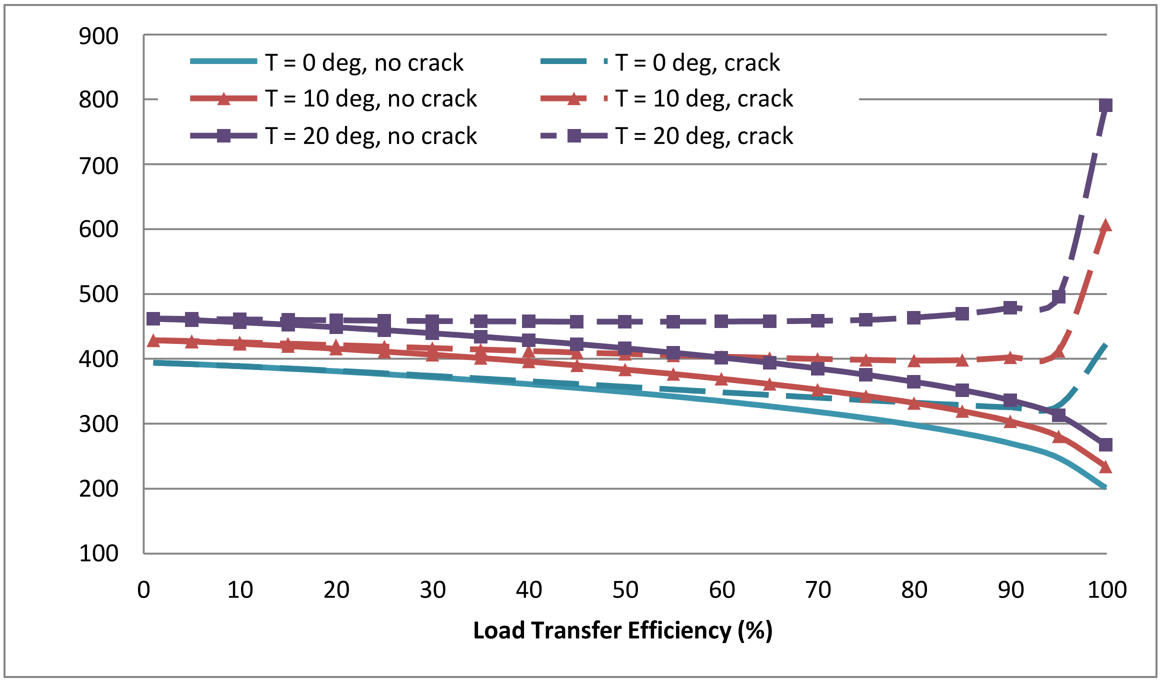
Lederle (2014) initiated a study to incorporate a longitudinal cracking prediction model in the MEPDG, one that was not included in the original MEPDG-based mechanistic-empirical (M-E) pavement design. A model compatible with the MEPDG framework for predicting and analyzing incremental damages from longitudinal cracking was developed, and stresses exerted from axle loading and environmental loading at critical locations related to longitudinal cracking were computed. Neural networks, trained by running a large finite element factorial from ISLAB2000, were also employed to accelerate the stress estimation. Furthermore, because of the large number of cases loaded, an equivalent structure concept (also known as similarity) was adopted for predicting stress and deflection in a slab based on a similar slab, significantly saving both computational resources and time.

Lederle (2014) also investigated the stress level when transverse cracking or longitudinal cracking is presented in an adjacent slab with 10 in. slab thickness, an elastic modulus of 4×10^6 psi, and a modulus of subgrade reaction of 100psi/in. under a combination of various environmental and traffic loads. Cases were considered for both a standard 12 ft wide slab and a widened slab (14 ft) with 15 ft in slab length. It was found that transverse cracking occurring in an uncracked slab does not affect stress distribution in adjacent cracked and uncracked slabs, although this was not the case for longitudinal cracking. For longitudinal cracking, the most critical loading case for initiation is when the load is added at the mid-slab edge. In this case, the closest loads toward the slab edge are 36 in. from the edge of standard 12 ft wide slabs and 60 in. from the edge of 14 ft widened slabs, respectively. For this “mid-slab” loading case, the results indicated both the standard wide slabs and widened slabs in general show similar behavior. While the widened slabs have a higher stress level than the standard wide slabs when the loads are placed close to mid-slab, the widened slabs also are sensitive to environmental loads due to larger slab size (Figure 21 and Figure 22). However, when loads are placed in the wheel path, widened slabs exhibit a more pronounced loss of benefit of load transfer efficiency for adjacent cracked and uncracked slabs. It could be that standard wide slabs behave like Westergaard's corner-cracking loading case, while widened slabs still behave like an edge loading case due to the extra 2 ft of width away from the slab edge. Furthermore, widened slabs still show higher stresses than standard wide slabs (Figure 23 and Figure 24), and, as a result, the widened slabs are more prone to longitudinal cracking because of the larger loss of benefit of load transfer efficiency coupled with higher stresses induced. It can therefore be seen that slab width has an impact on pavement system behavior if the loads are applied along the wheel path (Lederle 2014).



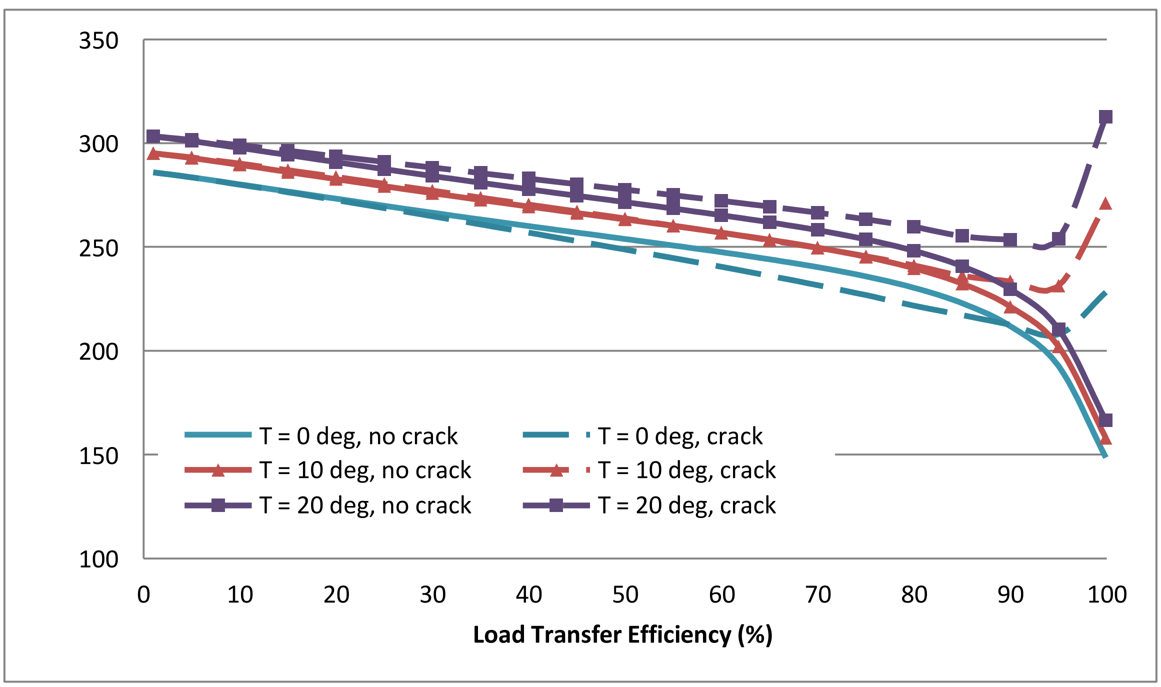
Lederle 2014

Figure 21. Stresses at the bottom surface of the standard wide slab for the "mid-slab" load case



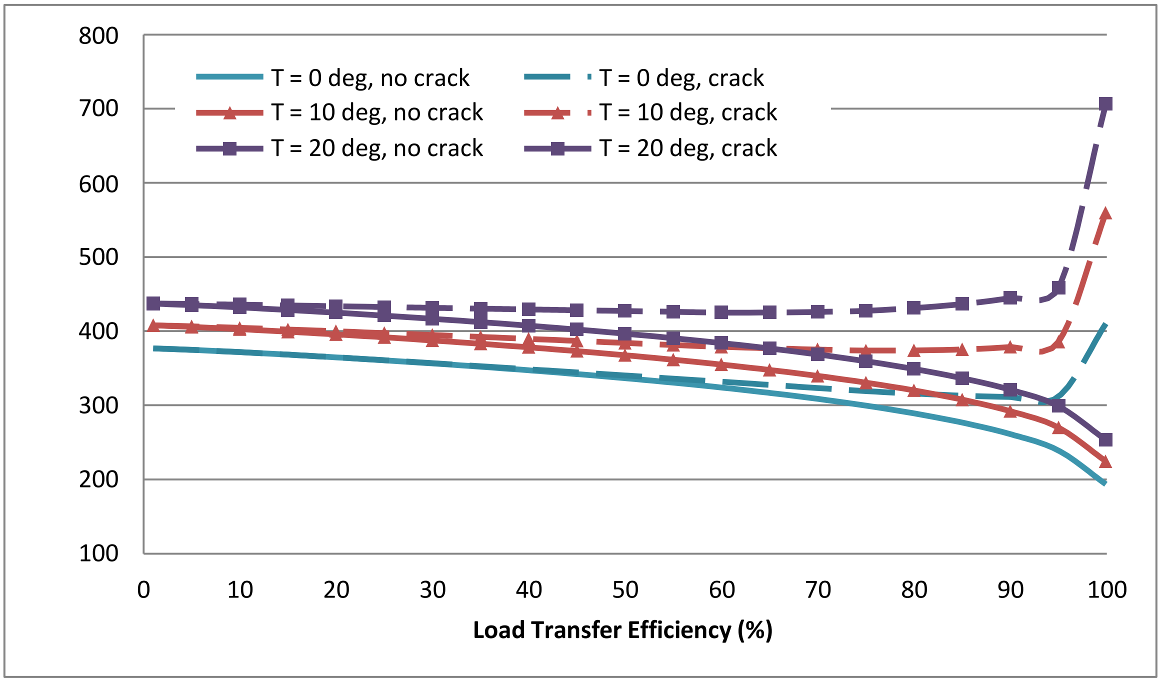
Lederle 2014

Figure 22. Stresses at the bottom surface of the widened slab for the “mid-slab” load case



Lederle 2014

Figure 23. Stresses at the bottom surface of the standard wide slab for the “wheel path” load case



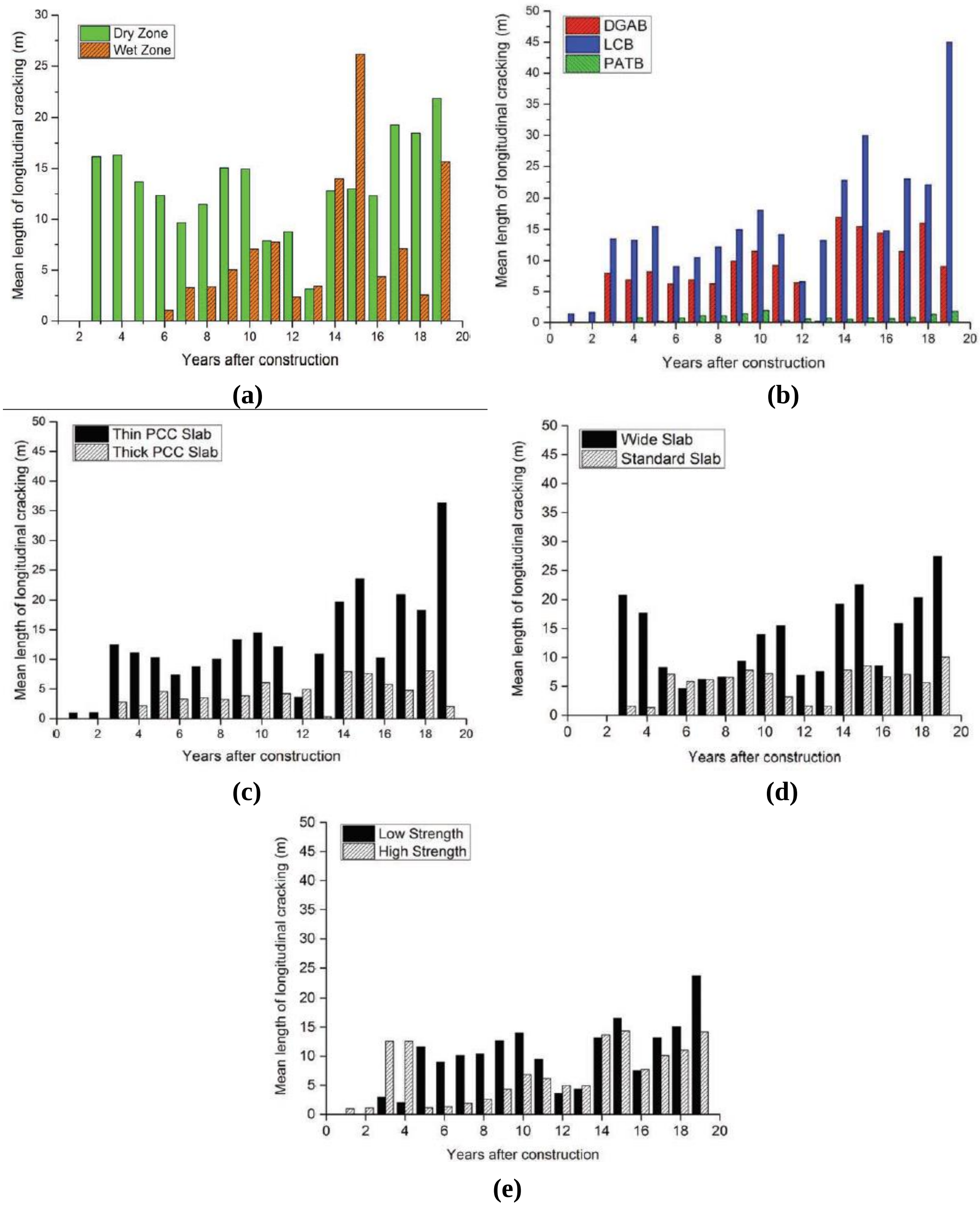
Lederle 2014

Figure 24. Stresses at the bottom surface of the widened slab for the "wheel path" load case

A similar approach was employed to estimate deflection at the joint to account for the effects of subgrade erosion on longitudinal cracking for both standard wide slabs and widened slabs. The results indicated a high potential for pumping and erosion for both slab widths when longitudinal cracking and load are present, resulting in large increases in joint deflection. Although the erosion potential is similar for both slab widths, widened slabs have higher stresses, as observed from the finite element modelling, and thus a higher potential for crack propagation. Additionally, by constructing the prediction model of longitudinal cracking damage, Lederle (2014) learned that some standard wide slabs with tied PCC shoulders are similar to widened slabs in terms of susceptibility to longitudinal cracking, while standard wide slabs with asphalt shoulders are much less susceptible with respect to longitudinal cracking. Furthermore, dowel bars can significantly reduce the damage of longitudinal cracking, even though they may also encourage crack propagation once a crack occurs (Lederle 2014).

Xu and Cebon (2017)

Xu and Cebon (2017) conducted a study to investigate the trends of longitudinal cracking in JPCP based on the Long-Term Pavement Performance (LTPP) Program Strategic Study of the Structural Factors for Rigid Pavements (SPS-2) database. The LTPP SPS-2 pavement sections, built between 1992 and 1999, had more than 20 years of service life in 2017. In this study, the length of longitudinal cracking per section was considered for pavement performance evaluation. A number of factors, including slab geometry, base type, concrete strength, and environmental factors, were assessed with respect to the occurrence and extent of longitudinal cracking (Figure 25).



Xu and Cebon 2017

Figure 25. Factors affecting the extent of longitudinal cracking: (a) weather conditions, (b) types of base course, (c) slab thickness, (d) slab width, and (e) concrete strength

Figure 25 shows that pavement sections in a dry zone cracked sooner than those in a wet zone (Figure 25a) and pavement sections in wet zones exhibited a more gradual longitudinal cracking pattern. Pavement sections lying on a lean concrete base (LCB) also exhibited more severe longitudinal cracking compared to those on a dense-graded aggregate base (DGAB). Pavement sections with a permeable asphalt-treated base (PATB) had the lowest level of longitudinal cracking (Figure 25b). It was also found that thinner (8 in.) and wider (14 ft) slabs exhibited more longitudinal cracking than thicker (11 in.) and standard width (12 ft) slabs (Figure 25c and Figure 25d). Pavement sections with higher 14-day slab strength (900 psi) also exhibited earlier longitudinal cracking than pavement sections with lower slab strength (550 psi) (Figure 25e), and weaker pavement sections also exhibited more severe longitudinal cracking, especially during the period from 5 to 11 years after pavement construction.

Summary of Literature Review Results

Longitudinal cracking can occur under many conditions. A primary factor contributing to longitudinal cracking is poor construction practices, including soil heaving and swelling, improper compaction of underlying layers, misaligned dowels, late saw cuts for joints, and inadequate vibration during paving (Voigt 2002, Ardani et al. 2003, Owusu-Ababio and Schmitt 2013, Lederle 2014). Other factors, including pavement and shoulder types, slab geometry, joint types, mix design, and curling and warping, can also contribute to longitudinal cracking. Table 11 summarizes the factors contributing to longitudinal cracking, and Table 12 summarizes the strategies that can be applied to mitigate longitudinal cracking, according to the literature review.

Table 11. Factors contributing to longitudinal cracking

Causes	Effect on development of longitudinal cracking	Reference
Joint sawing	Joint sawing performed too early will cause raveling, while a saw cut performed too late will cause cracks between joints. Shallow sawing depth can contribute to longitudinal cracking.	Voigt 2002, Ardani et al. 2003, Lim and Tayabji 2005, Johnston 2014
Compaction	Inadequate compaction (i.e., vibrator trails) on underlying layers can lead to premature longitudinal cracking due to settlement.	Voigt 2002, Ardani et al. 2003, Owusu-Ababio and Schmitt 2013
Shoulder Type	PCC tied shoulder provides better crack control.	Lederle 2014, ARA 2004
Underlying layer type	Stabilized base has higher friction, which results in higher tensile stress, thus increasing the chance for cracking development compared to granular base.	Voigt 2002, Lim and Tayabji 2005, Owusu-Ababio and Schmitt 2013
Underlying layer treatment	High swell potential (high PI/low R-value) soil and frost heave lead to settlement, which causes cracks.	Ardani et al. 2003, Smiley and Hansen 2007
Concrete curing	Inadequate/poor curing can result in slab warping due to excessive concrete surface moisture loss and can affect early-age concrete strength development, which may lead to higher stress and lower strength, respectively.	Voigt 2002, Lim and Tayabji 2005
Paving time	Paving in the morning and/or in summer will result in a higher maximum temperature within the paving day and increase the chance for crack initiation.	Voigt 2002
Ambient weather condition	High ambient temperature, moisture, solar radiation, and strong wind during paving can increase shrinkage and stress, increasing potential for cracking. Dry weather will produce more potential for cracking in later stages.	Voigt 2002, Lim and Tayabji 2005, Xu and Cebon 2017
Dowel bar	Dowel bar reduces cracking potential, but misaligned dowels can lead to uncontrolled cracks near sawed joints, even when a crack exists below the saw cut.	Voigt 2002
Slab geometry	Thicker pavements (≥ 11 in) show less cracking compared to thinner pavements. More longitudinal cracking tends to occur with shorter joint spacing, i.e., 20 ft spacing. Widening slabs may influence longitudinal cracking because it may induce higher stresses.	Lim and Tayabji 2005, Owusu-Ababio and Schmitt 2013, Lederle 2014, Xu and Cebon 2017
Coarse aggregate (CA)	The CA with lower water demand and lower CTE can result in a lower degree of curling and warping, and thus less potential for longitudinal cracking. Crushed limestone exhibits less possibility of crack initiation from concrete sawing.	Voigt 2002

Fine aggregate (FA)	Sand with coarser particles, well-graded gradation, lower bulking volume, and proper fineness modulus can result in less potential for uncontrolled cracking.	Voigt 2002
Cement	High paste content with a given w/c ratio is more susceptible to temperature-related expansion, thus providing more potential for cracking.	Voigt 2002
w/cm ratio	High water use creates more susceptibility to moisture-related shrinkage, thus creating more potential for cracking.	Voigt 2002
Supplementary cementitious material (SCMs)	SCMs potentially decrease chances for cracking by delaying concrete setting, resulting in concrete sawing without excessive raveling.	Voigt 2002
Concrete properties	Slower rate of strength development can increase the chance for slab cracking.	Lim and Tayabji 2005
Curling and warping	Curling and warping can cause stresses, and traffic loads can magnify the effect, possibly creating critical conditions for slab cracking, especially in early-age pavement.	Lim and Tayabji 2005, Johnston 2014, Ceylan et al. 2016

Table 12. Strategies that can be applied to mitigate longitudinal cracking, based on literature review

Categories	Factors	Specific factors	Impact on longitudinal cracking
Material Properties	Coarse Aggregate (CA)	Type	The CA with lower water demand and lower CTE can result in a lesser degree of curling and warping and thus less potential for longitudinal cracking. Furthermore, crushed limestone exhibits decreased possibility for crack initiation from concrete sawing.
		Gradation	Increase of CA content can result in decreased shrinkage, helping reduce potential for cracking.
		Fines	Lower amounts of fine materials (i.e., clay) result in decreased shrinkage, thus reducing potential for cracking.
	Fine Aggregate (FA)	Type	The FA with lower water demand can result in less warping and has less potential for cracking.
		Gradation	Sand with coarser particles, well-graded gradation, lower bulking volume, and proper fineness modulus can result in lower potential for uncontrolled cracking.
	Cement	Chemistry	Cement with lower C_3A and alkali reduces autogenous shrinkage, thus decreasing potential for cracking.
		Fineness	Coarser cement may decrease potential for cracking because of less warping.
	SCMs	Fly ash and slag	SCMs potentially decrease chances for cracking by delaying concrete setting, resulting in concrete sawing without excessive raveling.
Mix Design	w/c ratio		Moderate w/c ratio is preferred because a higher w/c ratio can lead to higher drying shrinkage, while lower w/c ratio can lead to higher autogenous shrinkage, with high potential for cracking.
	Paste content		Lower paste content with given w/c ratio can reduce potential for cracking.
	Aggregate content		High content of CA and moderate content of FA are desirable because less CA and excessive FA will increase water demand to maintain slump, thus increasing warping and potential for cracking.
Pavement Design	Slab geometry	Slab thickness	Thicker pavements (≥ 11 in) have less cracking compared to thinner pavements.
		Slab length	More longitudinal cracking tends to occur with shorter (20 ft) joint spacing.
		Slab Width	Widening slabs may influence longitudinal cracking because they may induce higher stresses.
	Dowel bar		Dowel bars can restrain vertical displacement at joints to reduce potential for cracking.
	Underlying layer	Type	When compared to granular layers, stabilized layers can induce more tensile stresses due to higher friction. This may lead to higher potential for cracking if the tensile stresses are greater than tensile strength.
Construction Practice	Ambient weather conditions	Temperature	High temperature during paving can result in a short sawing window, increasing risk of crack initiation.
		RH	High RH during paving can reduce moisture loss from surface, reducing potential for cracking.

Categories	Factors	Specific factors	Impact on longitudinal cracking
		Solar radiation	Strong solar radiation during paving results in a short sawing window, increasing risk of crack initiation.
		Wind	Lower wind speed during paving can induce less shrinkage, resulting in lower potential for cracking.
	Paving time		Paving done in late fall and paving at night can reduce potential for slab cracking.
	Curing		Proper curing is significant in reducing dry shrinkage and reducing potential for cracking.
	Saw cut	Timing and depth	Joint sawing done too early will cause raveling, while saw cuts made too late will cause cracks. Shallow sawing depth can contribute to longitudinal cracking.
	Dowel bar	Alignment	Misaligned dowels can lead to uncontrolled cracks near a sawed joint even when a crack exists below the saw cut.
	Soil treatment		High swell potential (high PI/low R-value) soil and frost heave can lead to settlement, resulting in cracks.
	Compaction		Inadequate compaction (i.e., vibrator trails) on foundation can lead to premature longitudinal cracking.

FIELD INVESTIGATION SUMMARY

With the intent of investigating longitudinal cracking in in-service widened PCC pavements, several field surveys were conducted at 12 identified sites in Iowa, including four control sites and eight sites suffering from different extents of longitudinal cracking. Most of the surveys were carried out between spring and late summer of 2017. The sites surveyed had used different concrete mixtures and construction designs. Table 13 and Table 14 summarize basic information about the control sites and longitudinal cracking sites, respectively (Iowa DOT 2015, 2016). Photos taken at the visited sites appear in Appendix A. A detailed summary of field investigation activities is presented in the following sections.

Table 13. Basic information on the control sites for field investigation

No.	Site	Route	Dir[1]	MP[2]	CY[3]	AADTT[4]	Project
1	Polk County	US 65	1	71.58	1997	1,517	NHS-500-1(96)--19-77
2	Polk County	US 65	1	72.99	1997	1,735	NHS-500-1(79)--19-77
3	Tama County	US 30	1	194.6	2005	1,079	NHSX-030-5(146) --3H-64
4	Story County	US 30	2	159.85	1995	784	NHS-30-5(71) --19-85

[1]Dir is direction and "1" means eastbound or northbound while "2" means westbound or southbound.
[2]MP is the milepost number.
[3]CY is the construction year.
[4]AADTT is average annual daily truck traffic in 2015.

Table 14. Basic information on the longitudinal cracking sites for field investigation

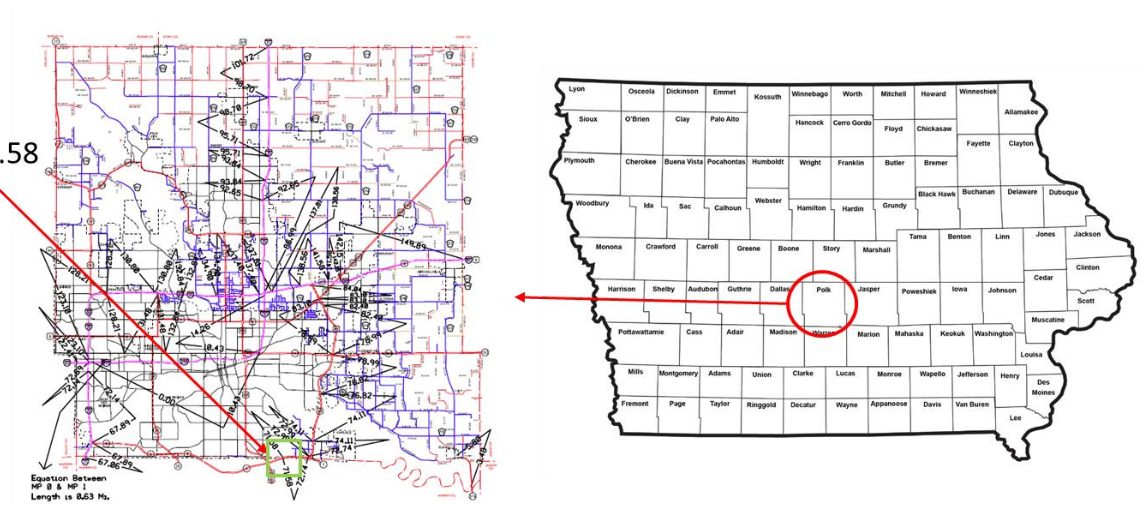
No.	Site	Route	Dir[1]	MP[2]	CY[3]	AADTT[4]	Project
1	Linn County	US 151	2	33.15	1999	2,927	NHS-151-3(87) --19-57
2	Linn County	US 30	2	261.2	1999	1,191	NHSX-30-7(94) --3H-57
3	Mahaska County	IA 163	1	44.99	1998	1,585	NHSN-163-4(22)--2R-62
4	Henry County	US 218	1	44.05	2001	1,699	NHSX-218-2(57)--3H-44
5	Jasper County	IA 163	1	21.44	1996	1,368	NHS-163-2(30)--19-50
6	Polk County	US 65	2	78.70	1997	1,178	NHS-500-1(9)--19-77
7	Lee County	US 61	1	30.32	1995	1,153	NHS-61-1(67)--19-56
8	Washington County	IA 92	1	235.09	2001	402	STP-92-9(74)--2C-92

[1]Dir is direction and "1" means eastbound or northbound while "2" means westbound or southbound.
[2]MP is the milepost number.
[3]CY is the construction year.
[4]AADTT is average annual daily truck traffic in 2015.

Control Site 1 – US 65 in Polk County, STA 650 (MP 71.58)

Pavement Design and Construction

At control site 1 (Figure 26), the PCC pavement was constructed starting at 6:00 a.m. on August 1, 1997. A 12 in. thick PCC layer was paved over a 10 in. granular subbase layer. During paving, the recorded weather was sunny with a maximum air temperature of 85°F. Additionally, the PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and the travel lane. Skewed joints were adopted for this site, with the transverse joint spacing set at 20 ft. Dowel bars 18 in. in length and 1.5 in. in diameter were also used at the joints. The shoulder consists of 8 ft wide PCC slabs.



I-35, I-80, I-235, and US 65 Bypass Polk County

Figure 26. Control site 1 on US 65 near Polk County in Iowa

Pavement Materials

Table 15 summarizes the materials used in the concrete mixture for this site.

Table 15. Mix design of US 65 in Polk County, STA 650 (MP 71.58)

Component	Description	Batch Weight
Portland cement	Buffalo, IA - Type I	457 lbs/yd^3
GGBFS		
Fly ash	Portage, WI. - Type C (SG = 2.70)	114 lbs/yd^3
Coarse aggregate	AMES MINE (SG = 2.51)	1,593 lbs/yd^3
Fine aggregate	AMES MINE (SG = 2.68)	1,429 lbs/yd^3
Water		227 lbs/yd^3
Admixture 1	Conchem Air-Entraining Admixture	2.6 oz/yd^3
Admixture 2	Conchem Water Reducer	2.0 oz/yd^3
w/cm ratio		0.398
Air content		8.5

Field Investigation Description

Field investigations at control site 1 on US 65 in Polk County were conducted on the morning of April 13, 2017. Photos and measurements were taken to document site conditions, including slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. As expected, no cracking was observed at this site. Figure 27 illustrates site conditions based on the field investigation (red lines indicate cracks in the figure, same in the figures in the rest of this report).

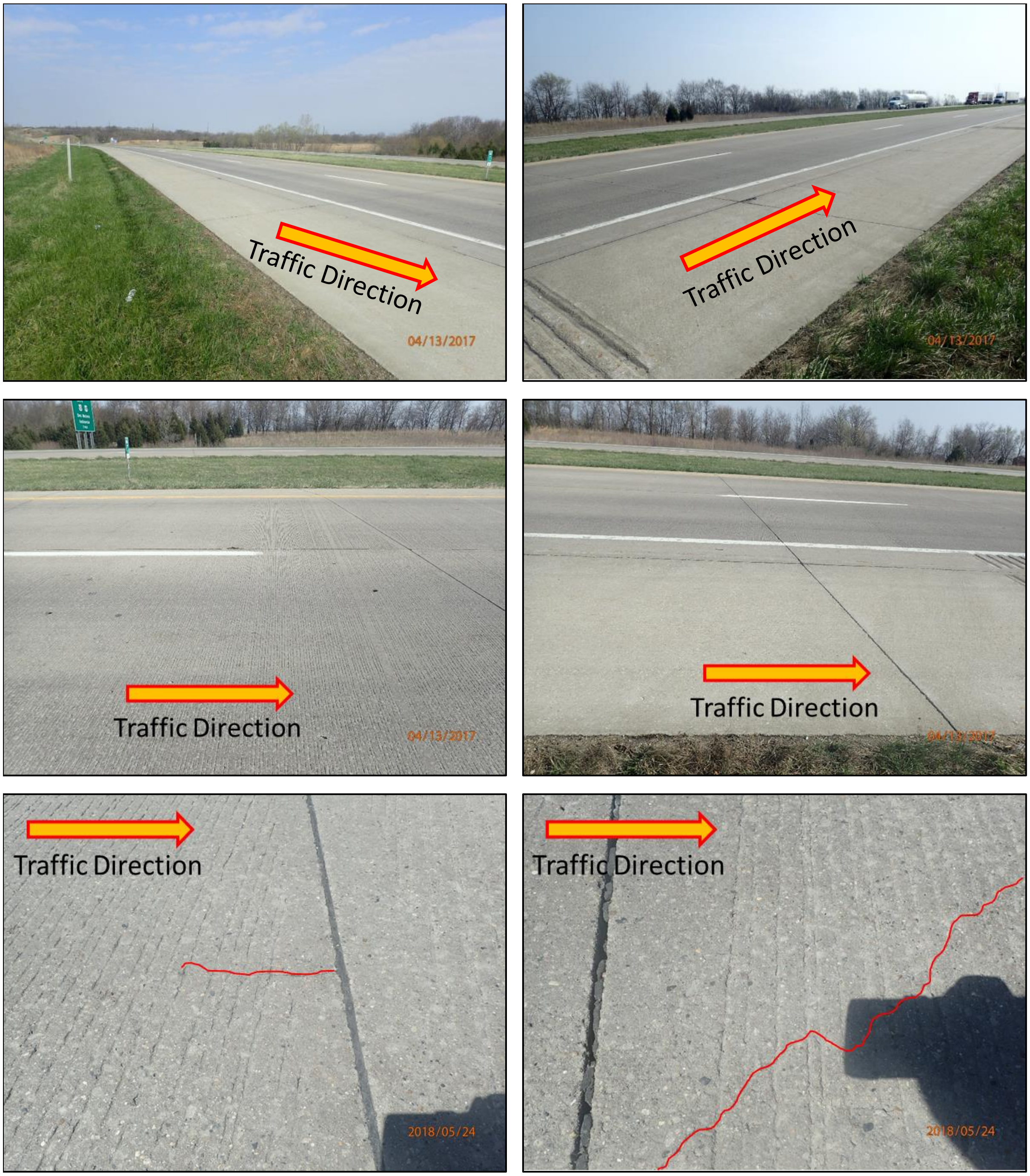


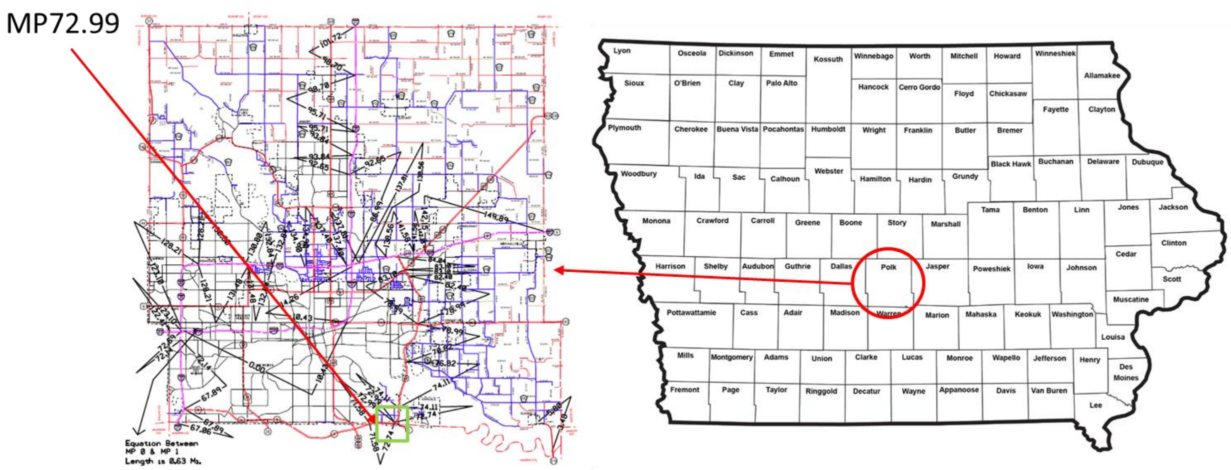
Figure 27. Field investigation on control site 1 on US 65 near Polk County, Iowa

As expected, no cracking was observed at this site during the field investigation in 2017. However, one year later a few longitudinal cracks with very short lengths (less than 3 ft) and low severity levels were observed in spring 2018 (see bottom two photographs in Figure 27).

Control Site 2 – US 65 in Polk County, STA 705 (MP 72.99)

Pavement Design and Construction

At control site 2 (Figure 28), PCC pavement was constructed starting at 16:25 p.m. on August 25, 1997, and an 11 in. thick PCC layer was paved over a 10 in. granular subbase layer. During paving, the weather was sunny with maximum air temperature of about 68°F. Additionally, the PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 20 ft. Dowel bars 18 in. in length and 1.5 in. in diameter were also used at the joints. The shoulder consists of 8 ft wide PCC slabs.



I-35, I-80, I-235, and US 65 Bypass Polk County

Figure 28. Control site 2 on US 65 near Polk County, Iowa

Pavement Materials

Table 16 summarizes the materials used in the concrete mixture for this site.

Table 16. Mix design of US 65 in Polk County, STA 705 (MP 71.58)

Component	Description	Batch Weight
Portland cement	Ash Grove - Type I	457 lbs/yd^3
GGBFS		
Fly ash	Midwest Ottumwa- Type C (SG = 2.60)	114 lbs/yd^3
Coarse aggregate	DURHAM MINE (SG = 2.51, moisture content = 2.0%)	1,439 lbs/yd^3
Fine aggregate	EDM #2 Vandalla (SG = 2.67, moisture content = 3.4%)	1,555 lbs/yd^3
Water		238 lbs/yd^3
Admixture 1	W.R. Grace Air-Entraining Admixture	
Admixture 2	W.R Grace (WRDA-82) Water Reducer	
w/cm ratio		0.417
Air content		6.82

Field Investigation Description

Field investigations at control site 2 on US 65 in Polk County were conducted on the morning of April 13, 2017. Photos and measurements taken to document site conditions include slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. As expected, no cracking was observed at this site. Figure 29 illustrates site conditions based on the field investigation.

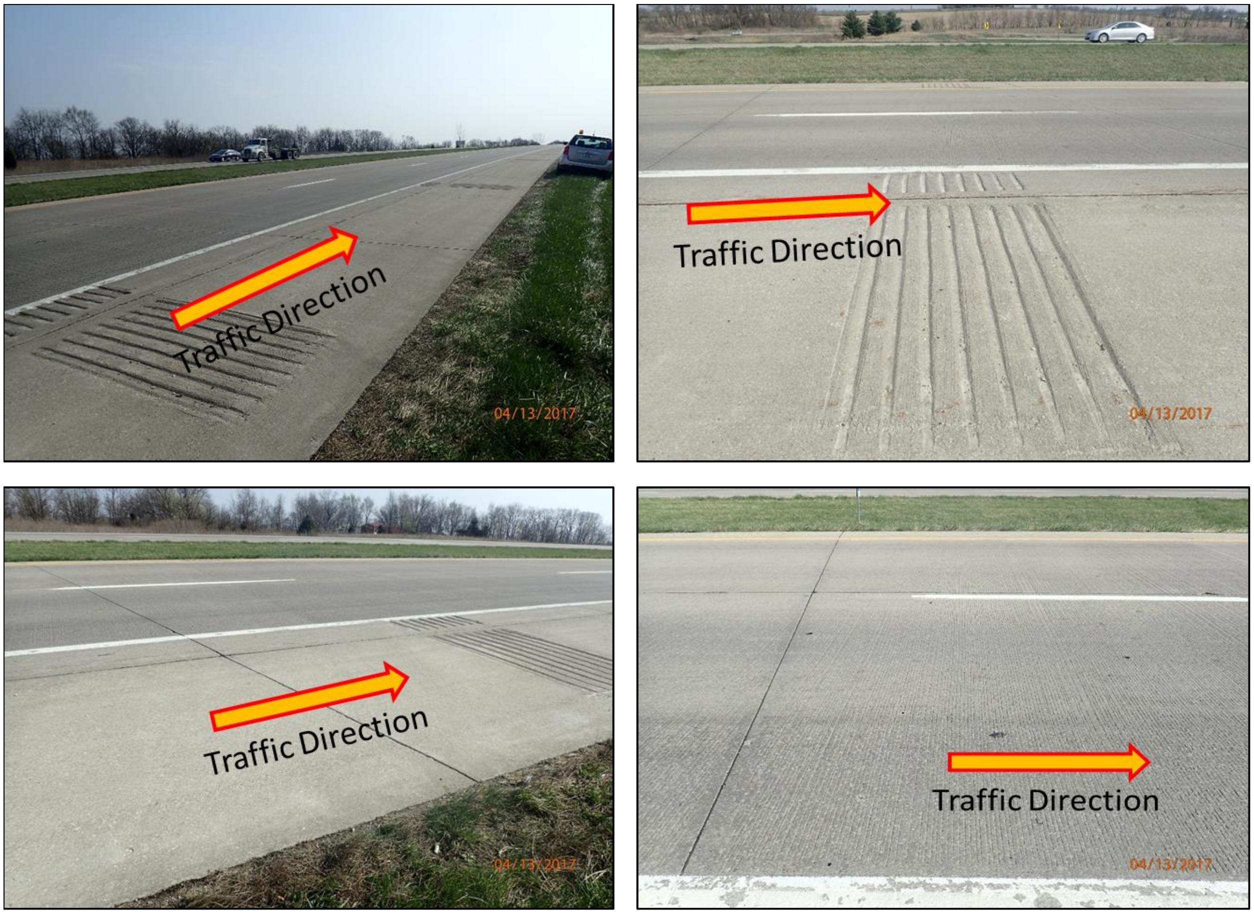
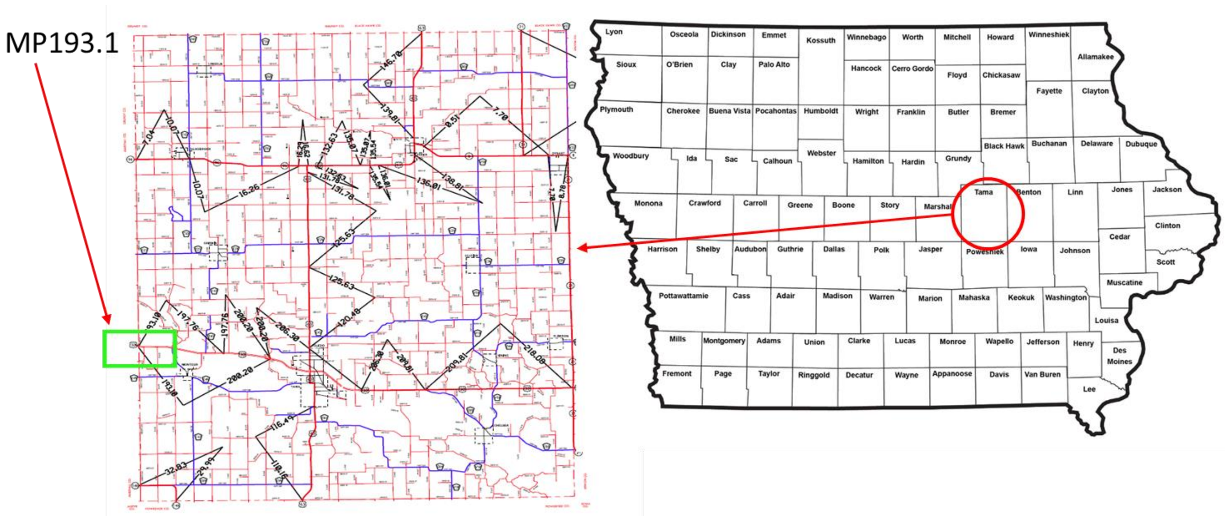


Figure 29. Field investigation on control site 2 on US 65 near Polk County, Iowa

Control Site 3 – US 30 near Toledo, Tama County, STA113 (MP 194.6)

Pavement Design and Construction

This site is located at the border of Tama County. The pavement at this section was paved starting at 3:00 p.m. on July 13, 2005. Figure 30 illustrates the location of this site.



Tama County

Figure 30. Control site 3 on US 30 near Tama County, Iowa

At this site, a 10 in. thick JPCP was constructed on top of a 6 to 10 in. open-graded granular base layer. The concrete was paved by a concrete spreader and paver. The concrete was transported to the construction site by haul trucks, graded, and fed into the spreader. The paver was then applied to level off the fresh concrete. Dowel baskets were placed at transverse joints prior to concrete paving. The baskets held smooth dowel bars, 18 in. long and 1.5 in. in diameter, that were placed approximately every 12 in. in the baskets. A transverse joint was cut approximately every 20.1 ft during the early age of the concrete, using an early-age saw cut 1.25 in. deep. For the longitudinal joints, size #5 tie bars, 35 in. length and 0.63 in. diameter, were inserted every 2.5 ft. The longitudinal joints were cut 3.4 in. deep by conventional saws. The passing lane and travel lane were approximately 12 ft and 14 ft wide, respectively. It should be noted that the travel lane is about 2 ft wider than the passing lane because general practice in Iowa is to construct the travel lane to be 2 ft wider than the passing lane. A HMA shoulder was also constructed next to the traffic lane.

Pavement Materials

Table 17 summarizes the materials used in the concrete mixture for this site.

Table 17. Mix design of STA113 on US 30 near Toledo, Tama County

Component	Description	Batch Weight
Portland cement	Ash Grove (Louisville, NE) - Type I/II	448 lbs/yd^3
GGBFS		
Fly ash	Ottumwa Generating Station - Type C (SG = 2.61)	112 lbs/yd^3
Coarse aggregate	Wendling - Montour #86002 (SG = 2.61, Absorption = 2.4%)	1,539 lbs/yd^3
Fine aggregate	Manatt - Flint #86502 (SG = 2.66)	1,272 lbs/yd^3
Water		224 lbs/yd^3
Admixture 1	Daravair 1,400	3.92 oz/yd^3
Admixture 2	WRDA 82	19.6 oz/yd^3
w/cm ratio		0.40
Air content		6.0

The coefficient of thermal expansion (CTE) measured from the field-fabricated cylinders was 5.35×10-6 Ɛ/°F. Class C fly ash was added to constitute 20% of the total weight of the cementitious materials to improve concrete workability and durability. Water reducer was used to produce an average w/cm ratio of 0.4. Air-entraining agent was added to the concrete mixture, resulting in an average air content of 6%.

Field Investigation Description

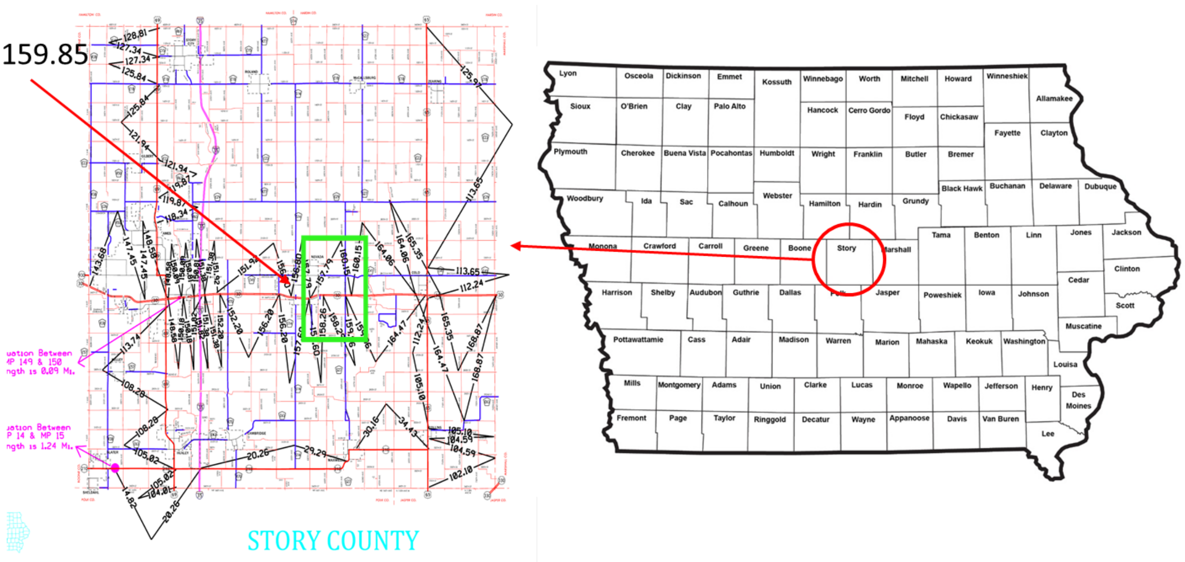
Field investigations at control site 3 on US 30 in Tama County were conducted on the afternoon of June 15, 2017. Photos and measurements taken to document site conditions include slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. As expected, no cracking was observed on the pavement surface at this site. Few cracks were seen on the HMA shoulder. Figure 31 illustrates site conditions based on the field investigation.

Figure 31. Field investigation on control site 3 on US 30 near Tama County, Iowa

Control Site 4 – US 30 near Nevada, Story County, STA2207 (MP 159.85)

Pavement Design and Construction

The PCC pavement at this site was built in June 1995. Figure 32 illustrates the location of this site.



Story County

Figure 32. Control site 4 on US 30 near Story County

Prior to pavement construction at this site, the moist subgrade was covered. The construction started around 7:00 a.m., and transit mixing was used for concrete mixing. On the day of construction, the weather was sunny with a maximum daytime ambient temperature of 78°F. At night, the ambient temperature decreased to 48°F. After concrete placement using a slip form paver, white-pigmented curing compound was used for surface curing, and broom grooving was used to create surface texture. At the end, a 10 in. thick PCC layer was paved over a 10 in. granular subbase layer. The PCC pavement was crowned with a 2.0% transverse slope and had designed widths of 12 ft and 14 ft, respectively, for the passing lane and travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 20.2 ft. In addition to the driving lanes, an approximately 9.5 ft wide granular shoulder was built.

Pavement Materials

Table 18 summarizes the materials used in the concrete mixture for the pavement at this site.

Table 18. Mix design of STA2207 on US 30 near Nevada, Story County

Component	Description	Batch Weight
Portland cement	Ash Grove Type I (SG = 3.14)	457 lbs/yd^3
GGBFS		
Fly ash	Kenosha Type C (SG = 2.67)	114 lbs/yd^3
Coarse aggregate	Martin Marietta, Ames Mine Gradation No.5 (SG = 2.59, Absorption = 2.6%)	1,653 lbs/yd^3
Fine aggregate	Halletts (SG = 2.68)	1,446 lbs/yd^3
Water		282 lbs/yd^3
Admixture 1	Daravair	Not available
Admixture 2	Plastocrete-161	Not available
Water/cementitious ratio		0.493
Air content		7.8%

It can be seen that 20% Class C fly ash was added to improve concrete properties such as workability and durability. Furthermore, water reducer was used to produce an average w/cm ratio of 0.44. The maximum w/cm ratio was 0.49 among the different mixes. An air-entraining agent was added to the concrete mixture, and the resulting average air content was 7.8%.

Field Investigation Description

Field investigations at control site 4 on US 30 in Story County were conducted on the afternoon of June 15, 2017. Photos and measurements taken to document site conditions include slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. As expected, no cracking was observed on the pavement surface at this site. Figure 33 illustrates site conditions based on the field investigation.

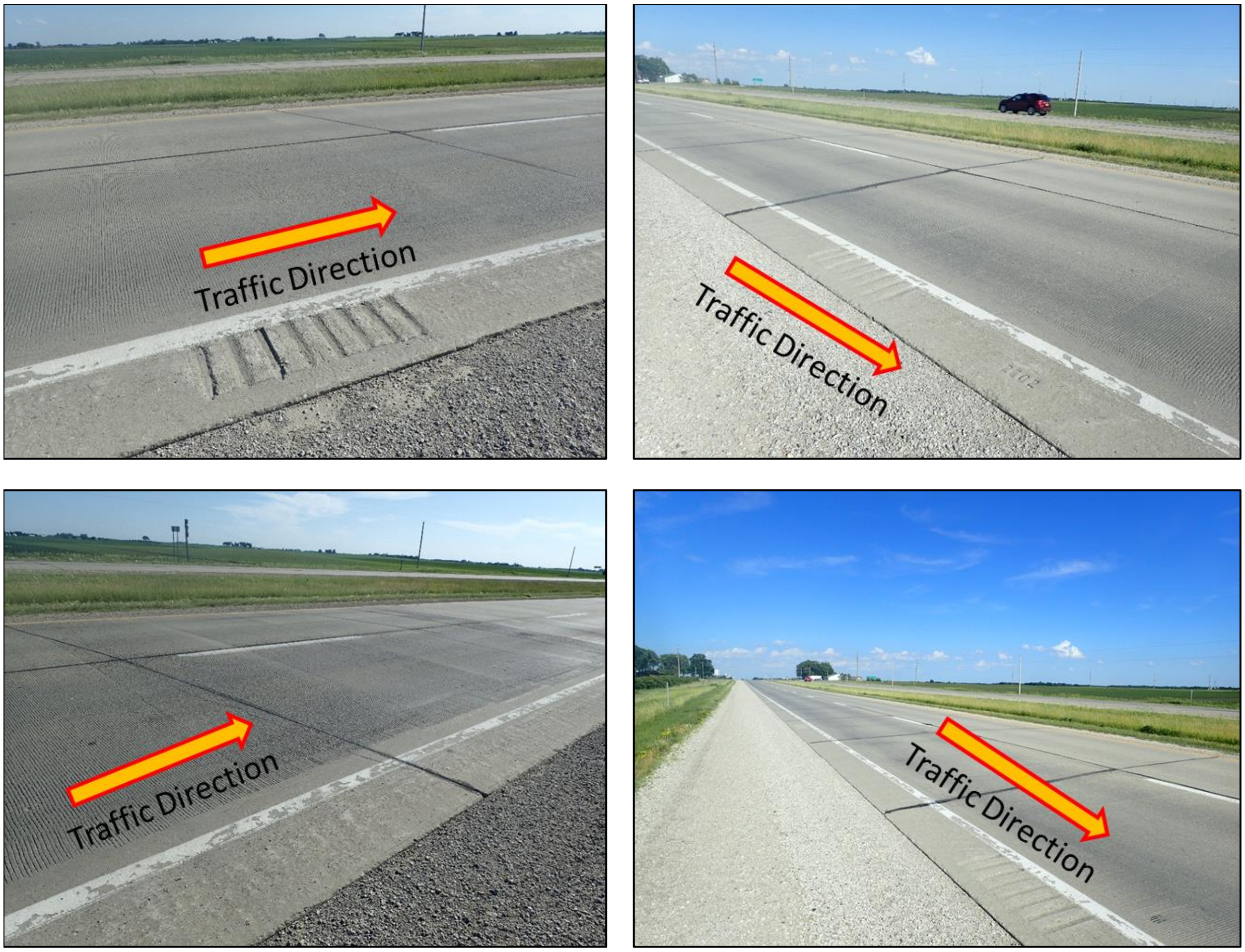
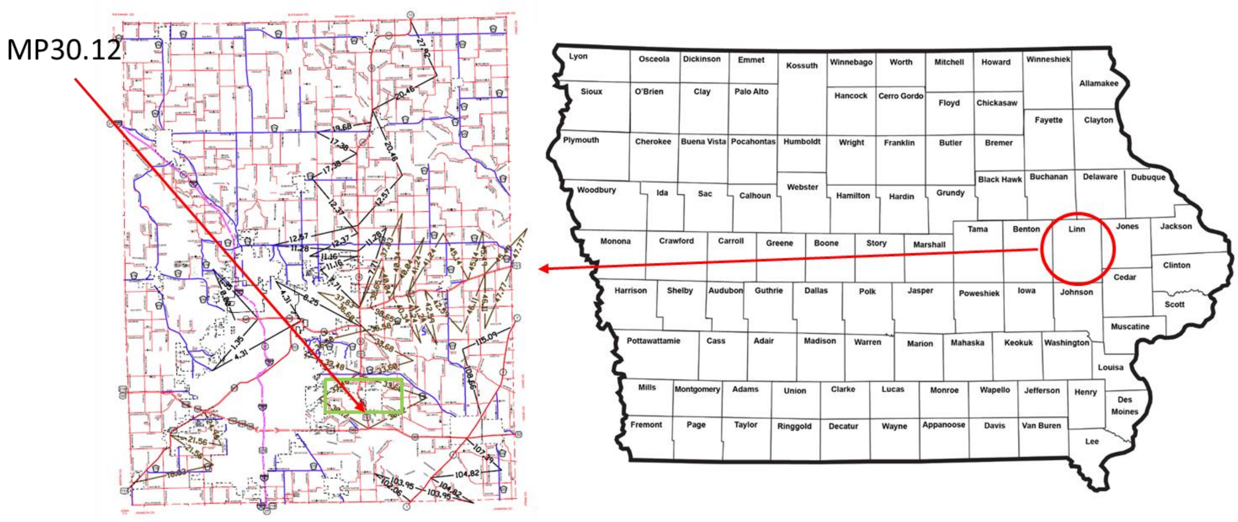


Figure 33. Field investigation on control site 4 on US 30 near Story County, Iowa

LC Site 1 – US 151 near Cedar Rapids, Linn County, STA162 (MP 32.75)

Pavement Design and Construction

The PCC pavement at this site was constructed on November 1, 1999. Figure 34 illustrates the location of this site.



Linn County

Figure 34. LC site 1 at US 151 near Cedar Rapids, Linn County

The PCC layer was 10 in. thick and built on top of a 10.5 in. thick granular subbase. The travel lane was 14 ft wide. Some portions of the lanes were cut to have skewed joints, with 21.2 ft wide joint spacing. The joints were sealed by an asphalt sealant. It should be noted that a right-turn lane had recently been added alongside part of the traffic lane. The granular shoulder was 9 ft wide.

Construction at this site started at 8:00 a.m. on November 1, 1999. A slip form paver was used, and cold weather protection was adopted. A vibrator was used for consolidation after slip form paving. The day of construction was sunny but cold, with a maximum ambient temperature of 67°F and a minimum ambient temperature of 45°F. During concrete mixing, the maximum mix temperature was 67°F, the same as the ambient temperature at that time.

Pavement Materials

Table 19 summarizes the materials used in the concrete mixture for this site.

Table 19. Mix design of STA162 on US 151 near Cedar Rapids, Linn County

Component	Description	Batch Weight
Portland cement	Lafarge Cement Type SM (SG = 3.10)	479 lbs/yd^3
GGBFS		
Fly ash	ISG RES-Louisa Type C (SG = 2.68)	85 lbs/yd^3
Coarse aggregate	SO. Cedar Rapids Crushed Limestone #4 (SG = 2.63, Absorption = 2.6%)	1,675 lbs/yd^3
Fine aggregate	T-203 A-57506 Grad #1 (SG = 2.64)	1,415 lbs/yd^3
Water		222 lbs/yd^3
Admixture 1	Brett AEA 92	2.0 fl. oz/cwt
Admixture 2	Brett WR-91	2.0 fl. oz/cwt
Water/cementitious ratio		0.393
Air content		7.8%

Note: fl. oz/cwt = fluid ounces per hundred pounds of cement

The 15% Class C fly ash was added to improve concrete properties such as workability and durability. Water reducer was also used to produce an average w/cm ratio of 0.4. The maximum w/cm ratio was 0.49 among the different mixes. An air-entraining agent was added to the concrete mixture, and the resulting average air content was 7.8%.

Field Investigation Description

Field investigations at LC site 1 on US 151 in Linn County were conducted on the afternoon of June 15, 2017. Photos and measurements taken to document site conditions include slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the field investigation, a large amount of longitudinal cracking was observed on the pavement surface, and transverse cracking was seen at some slabs. Figure 35 illustrates the longitudinal cracking observed during field survey.

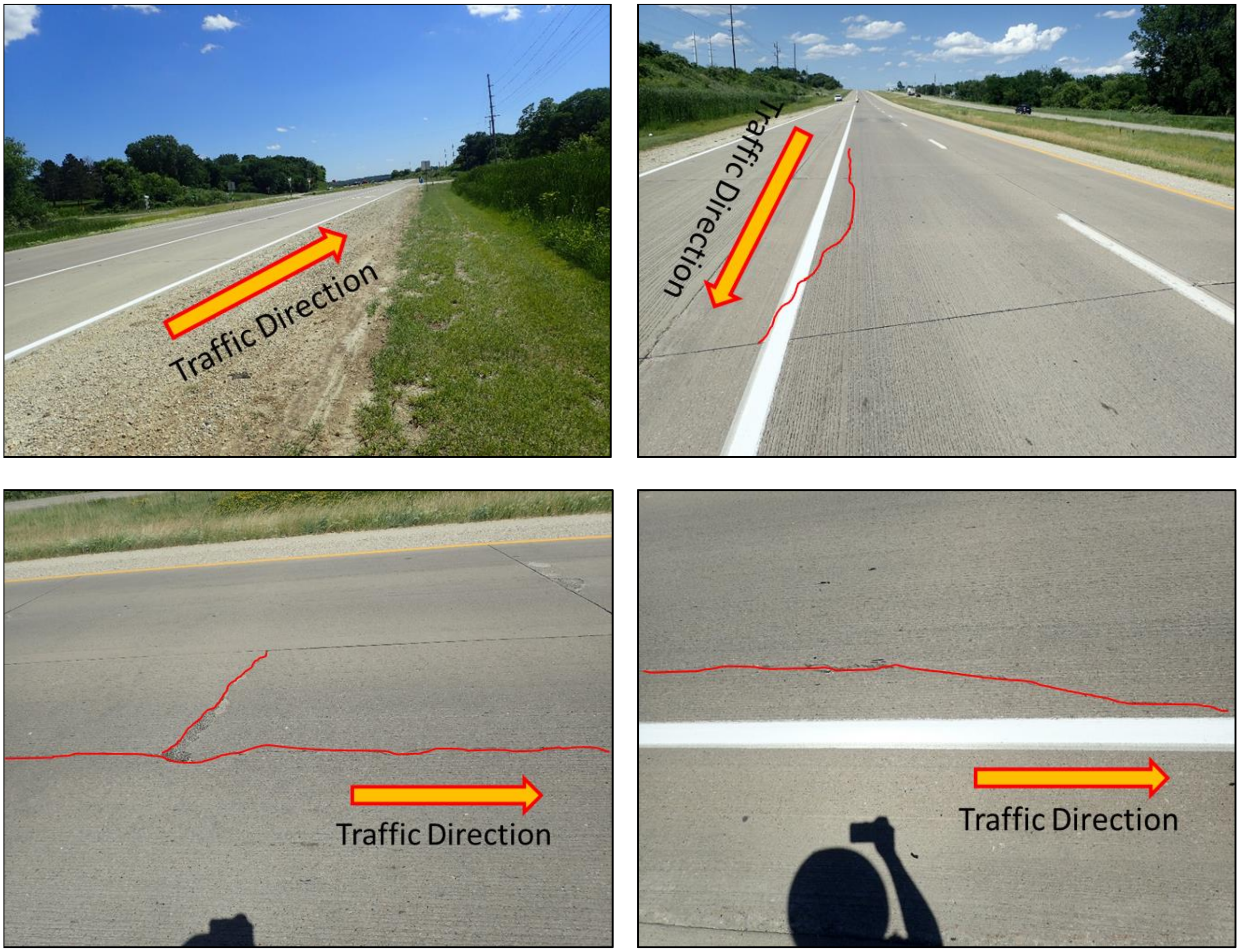
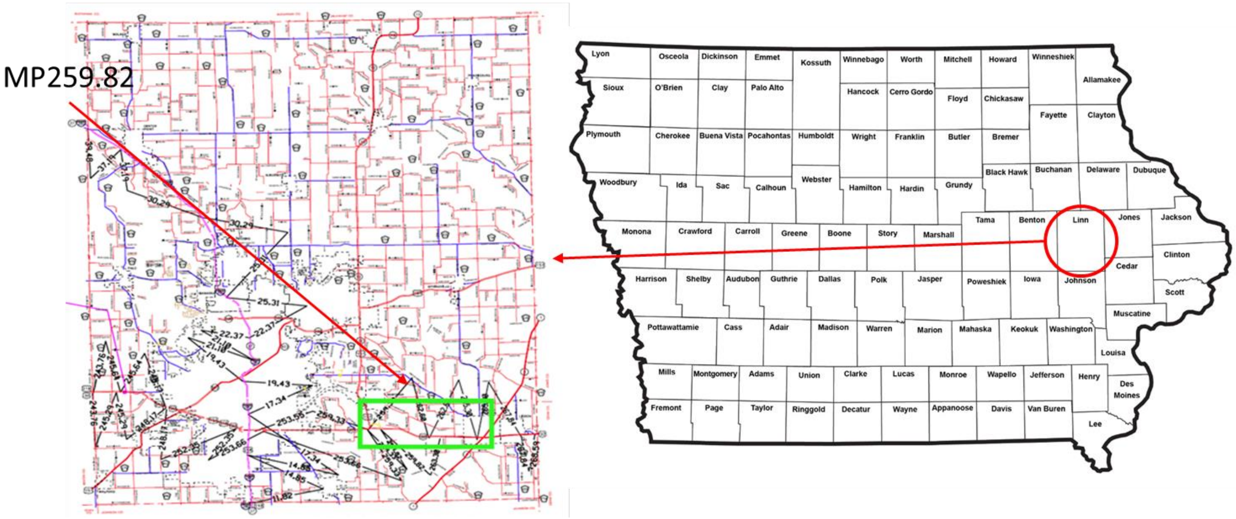


Figure 35. Field investigation on LC site 1 on US 151 near Linn County, Iowa

LC Site 2 – US 30 near Cedar Rapids, Linn County, STA463 (MP 261.2)

Pavement Design and Construction

The PCC pavement at this site was constructed on about May 15, 1999. Figure 36 illustrates the site location. Construction started about 8:00 a.m. A slip form paver was used for the concrete paving. The day of construction was sunny with a maximum ambient temperature of 77°F and a minimum ambient temperature of 42.8°F.



US 30 and I-380 Linn County

Figure 36. LC site 2 at US 30 near Cedar Rapids, Linn County

The PCC layer was 10 in. thick and built on top of a 10.5 in. thick granular subbase. The travel lane was 14 ft wide. The joint spacing was cut to be approximately 20.7 ft wide and was sealed by an asphalt sealant. Skewed joints were adopted for this site, and the granular shoulder was 10 ft wide. Longitudinal cracks were found on roughly every other measured slab.

Pavement Materials

Table 20 summarizes the materials used in the concrete mixture for this site. It can be seen that 20% Class C fly ash was added to improve concrete properties such as workability and durability. Water reducer was also used, producing an average w/cm ratio of 0.43.

Table 20. Mix design of STA463 on US 30 near Cedar Rapids, Linn County

Component	Description	Batch Weight
Portland cement	Lafarge Type 1SM (SG = 3.10)	451 lbs/yd^3
GGBFS		
Fly ash	ISG Resource, Inc. (SG = 2.68)	113 lbs/yd^3
Coarse aggregate	SO. Cedar Rapids Crushed Limestone Grad#3 (SG = 2.66, Absorption = 2.6%)	1,702 lbs/yd^3
Fine aggregate	Wendling A57520 Grad#1 (SG = 2.65)	1,447 lbs/yd^3
Water		240 lbs/yd^3
Admixture 1	Daravair 1,000	4.0 fl. oz/cwt
Admixture 2	WRDA-82	3.5 fl. oz/cwt
Water/cementitious ratio		0.43
Air content		Not available

Note: fl. oz/cwt = fluid ounces per hundred pounds of cement

Field Investigation Description

Field investigations at LC site 2 on US 30 in Linn County were conducted on the afternoon of June 15, 2017. Photos and measurements taken to document site conditions included slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the field investigation, a large amount of longitudinal cracking was seen on the pavement surface. Figure 37 illustrates the longitudinal cracking observed during the field survey.

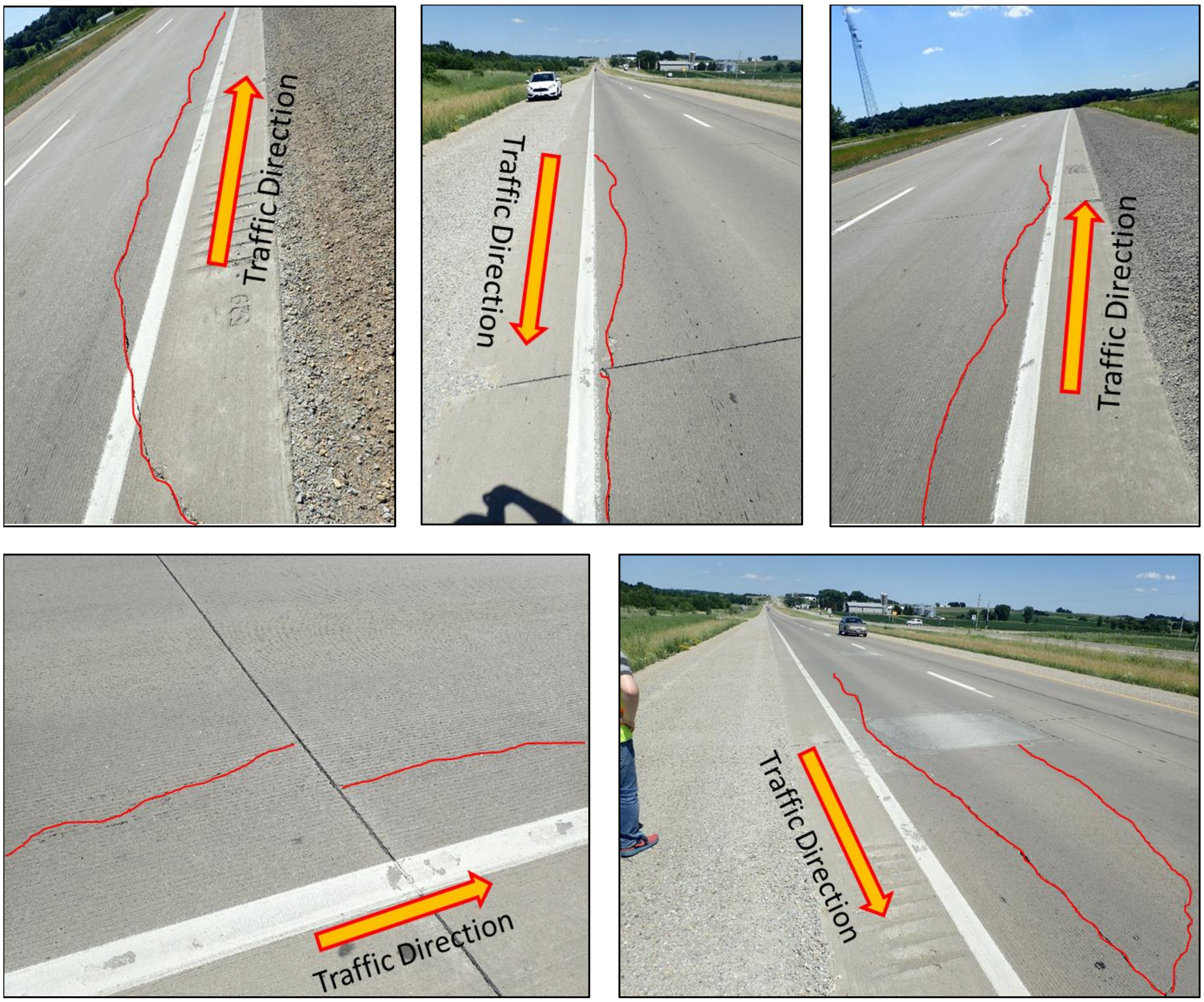


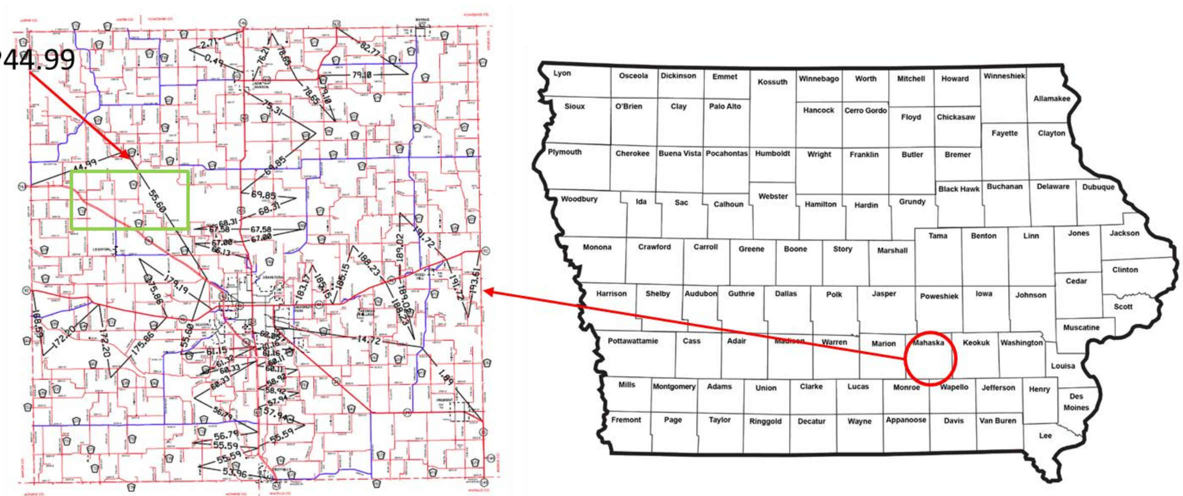
Figure 37. Field investigation on LC site 2 on US 30 near Linn County, Iowa

LC Site 3 – IA 163 in Mahaska County, STA 39 (MP 44.99)

Pavement Design and Construction

At LC site 3 (Figure 38), the PCC pavement was constructed starting at 8:55 a.m. on September 30, 1997. A 10 in. thick PCC layer was paved over a 10 in. granular subbase layer.

During paving, the weather was sunny with a maximum temperature around 80°F. The PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 19.7 ft. Dowel bars, 18 in. in length and 1.5 in. in diameter, were also used at the joints. The shoulder consisted of 13 ft wide granular slabs.



Mahaska County

Figure 38. LC site 3 at IA 163 near Mahaska County

Pavement Materials

Table 21 summarizes the materials used in the concrete mixture for this site.

Table 21. Mix design of IA 163 in Mahaska County, STA 939 (MP 44.99)

Component	Description	Batch Weight
Portland cement	Ash Grove - Type I	457 lbs/yd^3
GGBFS		
Fly ash	Council Bluffs - Type C (SG = 2.60)	114 lbs/yd^3
Coarse aggregate	SULLY MINE (SG = 2.56, moisture content = 0.5 %)	1,625 lbs/yd^3
Fine aggregate	DURHAM MINE (SG = 2.67, moisture content = 2.4%)	1,415 lbs/yd^3
Water		220.9 lbs/yd^3
Admixture 1	SIKA-AEA 15 Air-Entraining Admixture	4.9 oz/yd^3
Admixture 2	PLASTOCRETE 161 Water Reducer	16.0 oz/yd^3
w/cm ratio		0.387
Air content		7.5

Field Investigation Description

Field investigations at LC site 3 on IA 163 in Mahaska County were conducted on the morning of June 20, 2017. Photos and measurements taken to document site conditions include slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the field investigation, a large amount of longitudinal cracking was observed on the pavement surface, and transverse cracking was seen on some slabs. Figure 39 illustrates the longitudinal cracking observed during the field survey.

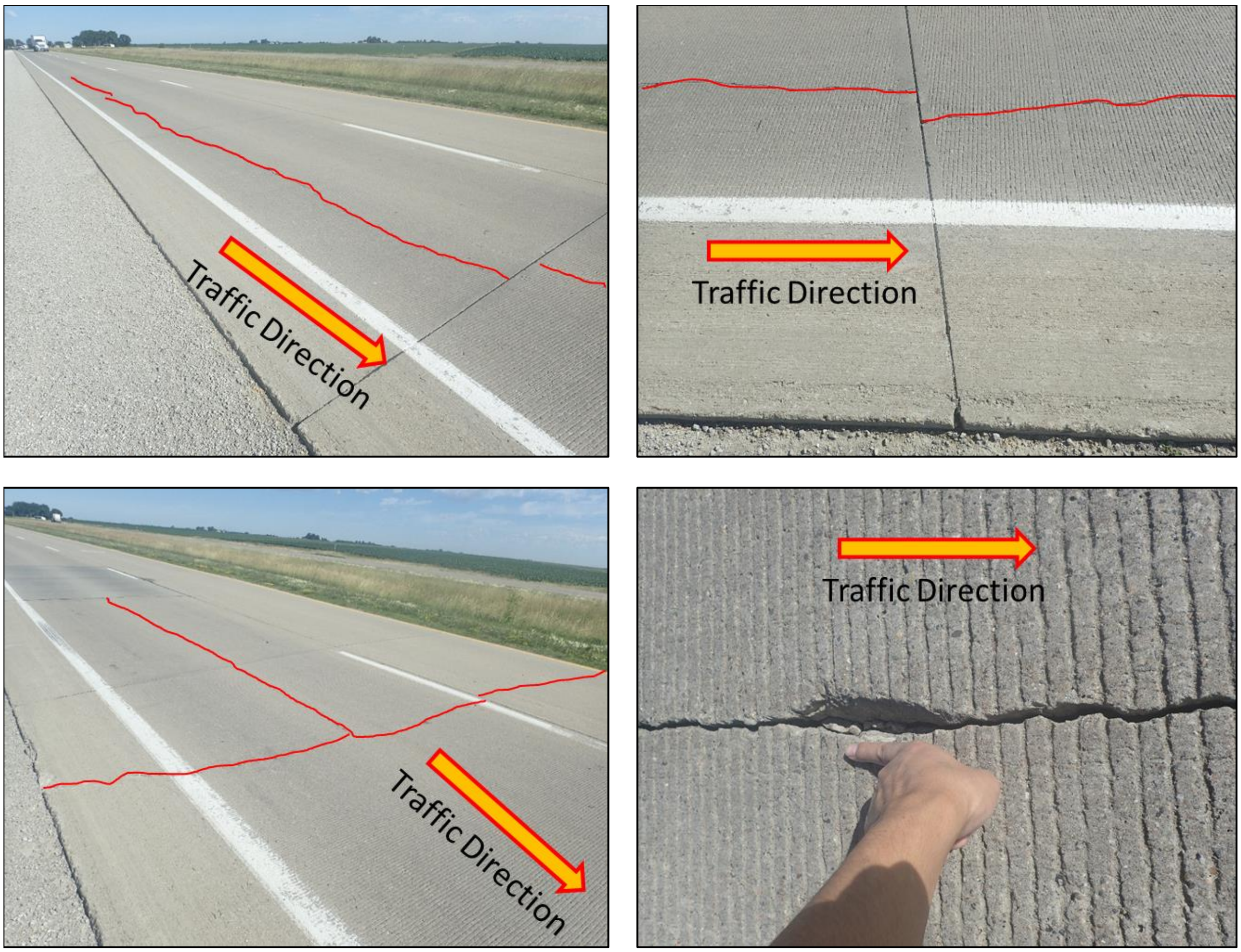
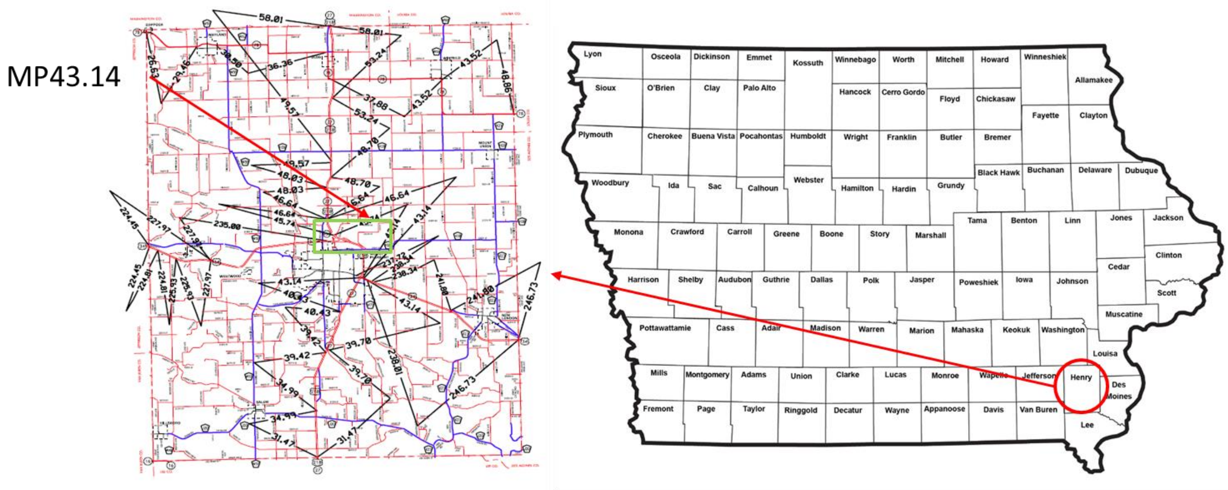


Figure 39. Field investigation on LC site 3 on IA 163 near Mahaska County in Iowa

LC Site 4 – US 218 in Henry County, STA 883 (MP 44.05)

Pavement Design and Construction

At LC site 4 (Figure 40), the PCC pavement was constructed on July 23, 2001. A 10.5 in. thick PCC layer was paved over a 10 in. granular subbase layer. During paving, the weather was windy with a maximum temperature around 90°F. The PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 19.8 ft. Dowel bars, 18 in. in length and 1.5 in. in diameter, were also used at the joints. The shoulder consisted of a 4 ft wide HMA shoulder.



Henry County

Figure 40. LC site 4 at US 218 near Henry County

Pavement Materials

Table 22 summarizes the materials used in the concrete mixture for this site.

Table 22. Mix design of US 218 in Henry County, STA 883 (MP 44.05)

Component	Description	Batch Weight
Portland cement	Lafarge Corp. - Type ISM	470 lbs/yd^3
GGBFS		
Fly ash	ISG-Ottumwa - Type C (SG = 2.61)	83 lbs/yd^3
Coarse aggregate	Conklin (SG = 2.60, moisture content = 0.3 %)	1,621 lbs/yd^3
Intermediate Aggregate	Conklin (SG = 2.60, moisture content = 1.6 %)	290 lbs/yd^3
Fine aggregate	Ensminger-Rome (SG = 2.67, moisture content = 3.5%)	1,244 lbs/yd^3
Water		227 lbs/yd^3
Admixture 1	W.R. Grace-Daravair 1400 Air-Entraining Admixture	2.28 ml/kg
Admixture 2	W.R. Grace-WRDA 82 Water Reducer	131.8 ml/CM
w/cm ratio		0.410
Air content		9.8

Field Investigation Description

Field investigations at the LC site 4 on US 218 in Henry County were conducted on the morning of June 20, 2017. Photos and measurements taken to document site conditions include slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the investigation, considerable longitudinal cracking was observed on the pavement surface, and transverse cracking was seen on some slabs. Figure 41 illustrates the longitudinal cracking observed during the field survey.

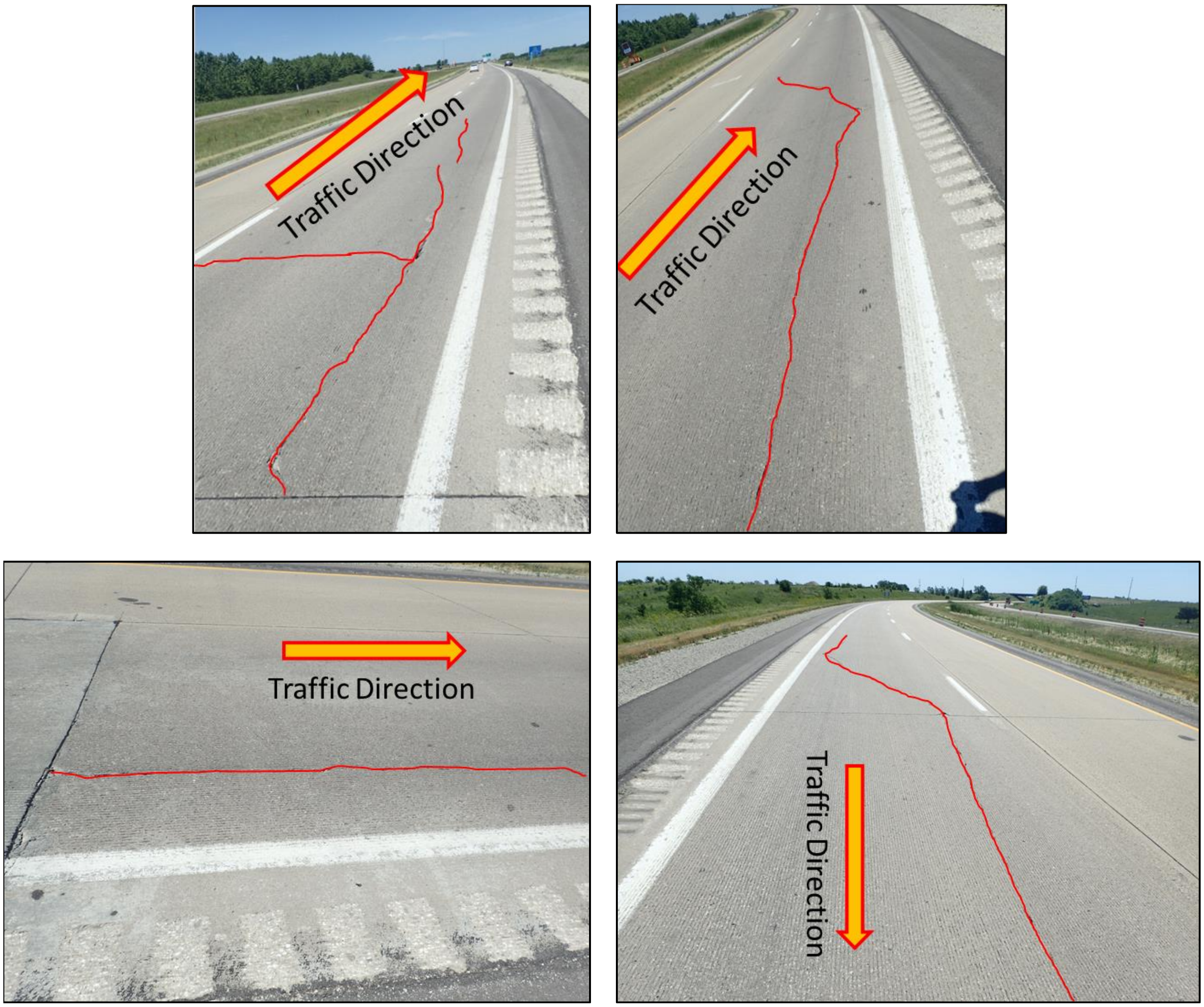


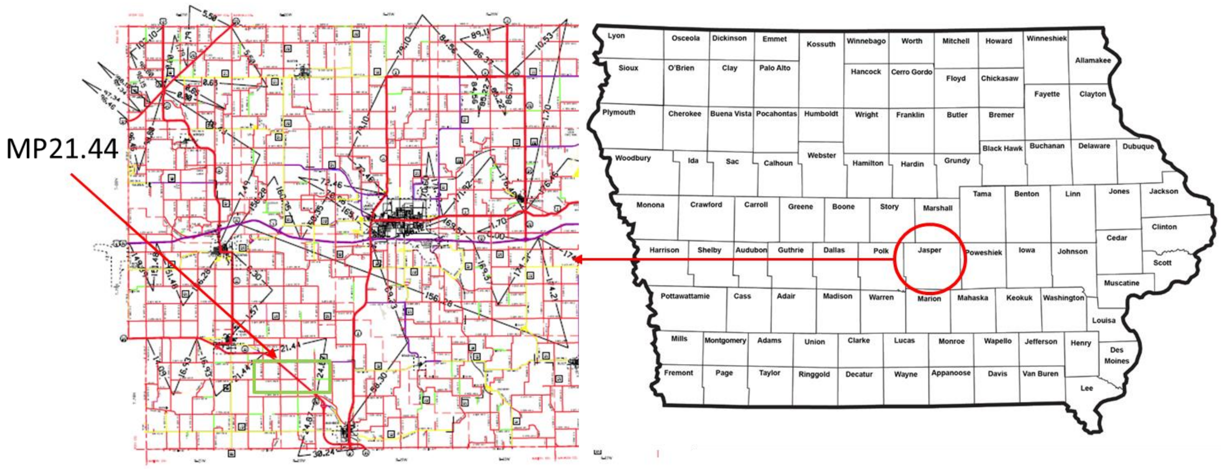
Figure 41. Field investigation on LC site 4 on US 218 near Henry County, Iowa

LC Site 5 – IA 163 in Jasper County, STA446 (MP 21.44)

Pavement Design and Construction

At LC site 5 (Figure 42), the PCC pavement was constructed starting at 7:00 a.m. on May 3, 1996. A 10 in. thick PCC layer was paved over a 10 in. granular subbase layer. During paving, the weather was cloudy with a maximum temperature around 52°F. The PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and the travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 208 ft. Dowel bars, 18 in.

in length and 1.5 in. in diameter, were used at the joints. The shoulder consisted of a 4 ft wide HMA shoulder.



Jasper County

Figure 42. LC site 5 at IA 163 near Jasper County

Pavement Materials

Table 23 summarizes the materials used in the concrete mixture for this site.

Table 23. Mix design of IA 163 in Jasper County, STA446 (MP 21.44)

Component	Description	Batch Weight
Portland cement	Ash Grove - Type I	457 lbs/yd^3
GGBFS		
Fly ash	Midwest Ottumwa- Type C (SG = 2.67)	114 lbs/yd^3
Coarse aggregate	SULLY MINE (SG = 2.56, moisture content = 0.4 %)	1,629 lbs/yd^3
Fine aggregate	Van Dasscldorp (SG = 2.67, moisture content = 3.0%)	1,424 lbs/yd^3
Water		206 lbs/yd^3
Admixture 1	Daravair 1000 Air-Entraining Admixture	N/A
Admixture 2	WRDA 82 Water Reducer	N/A
w/cm ratio		0.361
Air content		6.1

Field Investigation Description

Field investigations at LC site 5 on IA 163 in Jasper County were conducted on the afternoon of April 13, 2017. Photos and measurements taken to document site conditions included slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During field investigation, a few slabs showed longitudinal cracking on the pavement surface, and transverse cracking was observed at some slabs. Figure 43 illustrates the longitudinal cracking observed during the field survey.

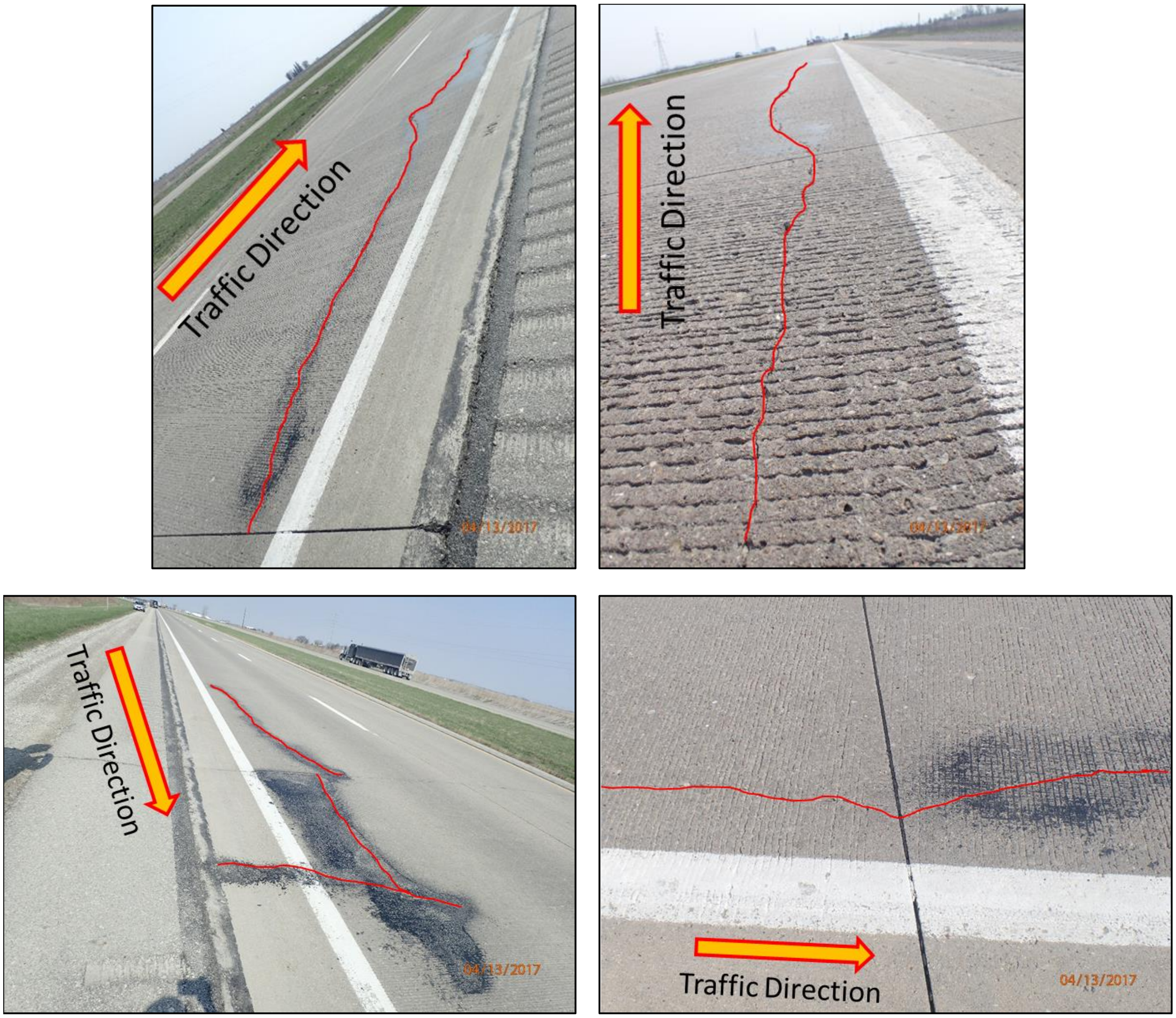
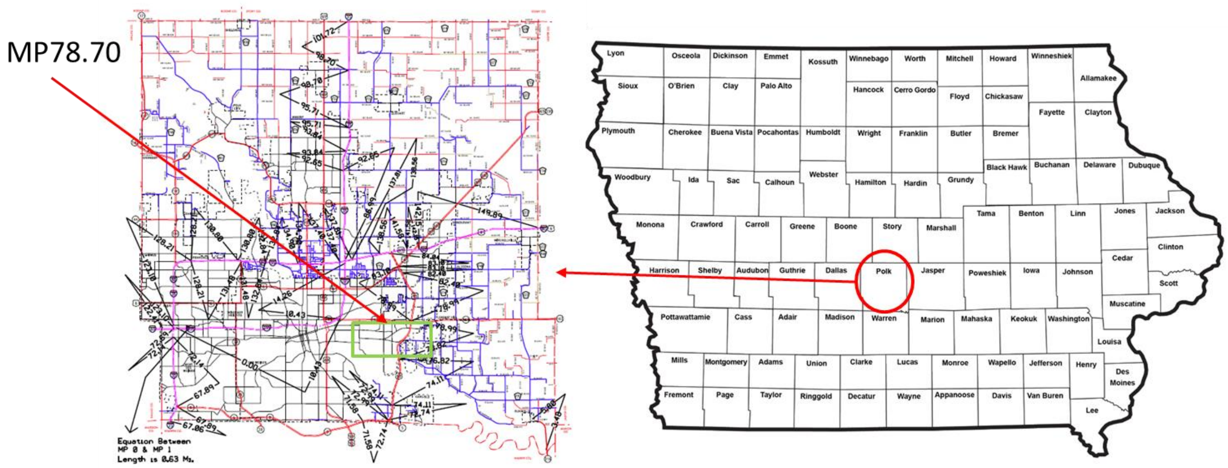


Figure 43. Field investigation on LC site 5 on IA 163 near Jasper County, Iowa

LC Site 6 – US 65 in Polk County, STA399 (MP 78.70)

Pavement Design and Construction

At LC site 6 (Figure 44), the PCC pavement was constructed starting at 11:40 a.m. on August 15, 1997. A 12 in. thick PCC layer was paved over a 10 in. granular subbase layer. During paving, the weather was cloudy with maximum temperature around 89°F. The PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and the travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 20.2 ft. Dowel bars, 18 in. in length and 1.5 in. in diameter, were used at the joints. The shoulder consisted of a 9 ft wide HMA shoulder.



I-35, I-80, I-235, and US 65 Bypass Polk County

Figure 44. LC site 6 at US 65 near Polk County

Pavement Materials

Table 24 summarizes the materials used in the concrete mixture for this site.

Table 24. Mix design of US 65 in Polk County, STA399 (MP 78.70)

Component	Description	Batch Weight
Portland cement	Lafarge Corp. - Type I	474 lbs/yd^3
GGBFS		
Fly ash	Portage, WI- Type C (SG = 2.75)	119 lbs/yd^3
Coarse aggregate	Ames MINE (SG = 2.59, moisture content = 0.8 %)	1,478 lbs/yd^3
Fine aggregate	Ames MINE (SG = 2.68, moisture content = 4.5%)	1,584 lbs/yd^3
Water		251 lbs/yd^3
Admixture 1	MBAE 90 Air-Entraining Admixture	7.5 oz/yd^3
Admixture 2	Masterpave N Water Reducer	11.9 oz/yd^3
w/cm ratio		0.423
Air content		9.4

Field Investigation Description

Field investigations at LC site 6 on US 65 in Polk County were conducted on the afternoon of April 13, 2017. Photos and measurements taken to document site conditions included slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the field investigation, a few slabs showed longitudinal cracking on the pavement surface. Figure 45 illustrates the longitudinal cracking observed during the field survey.

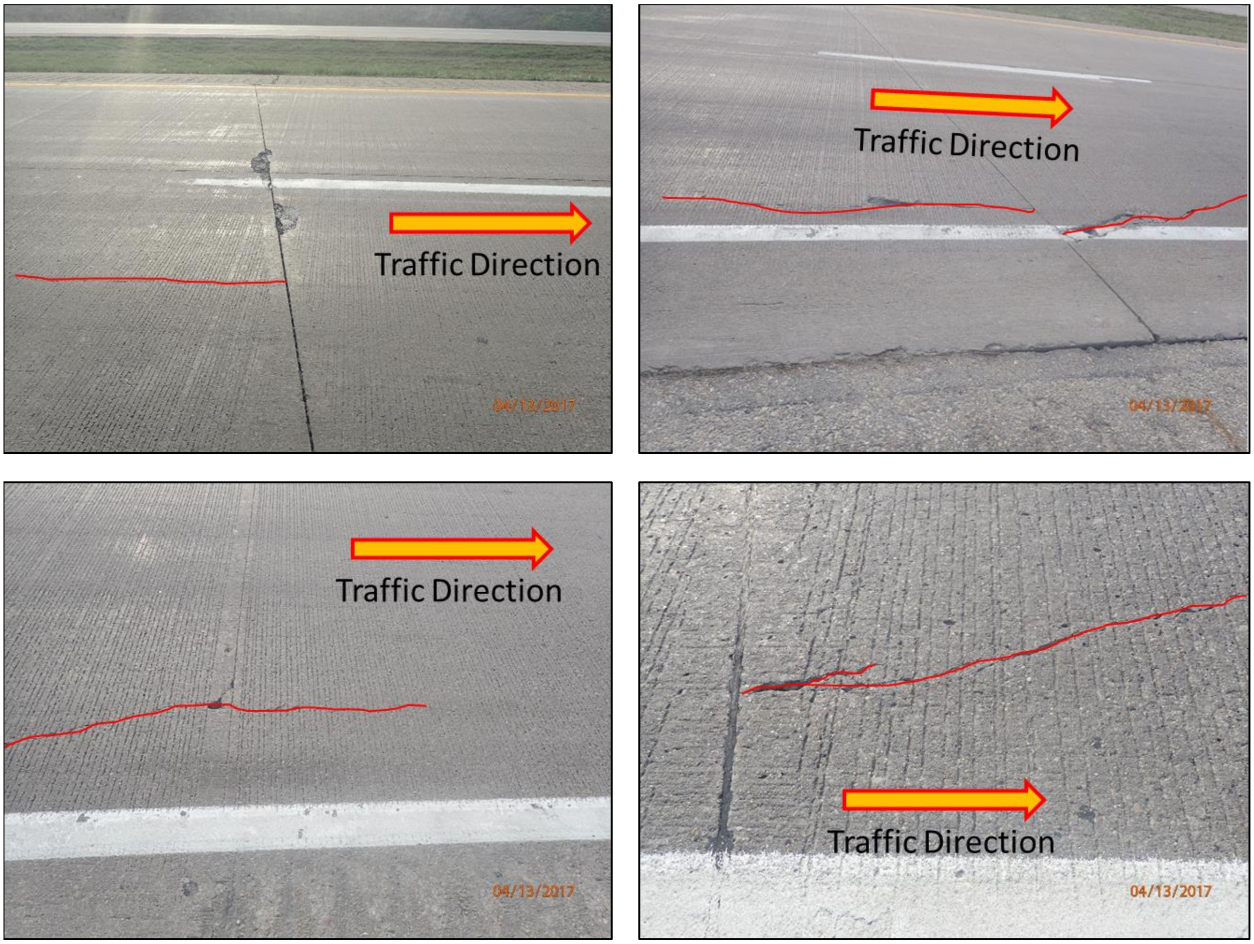
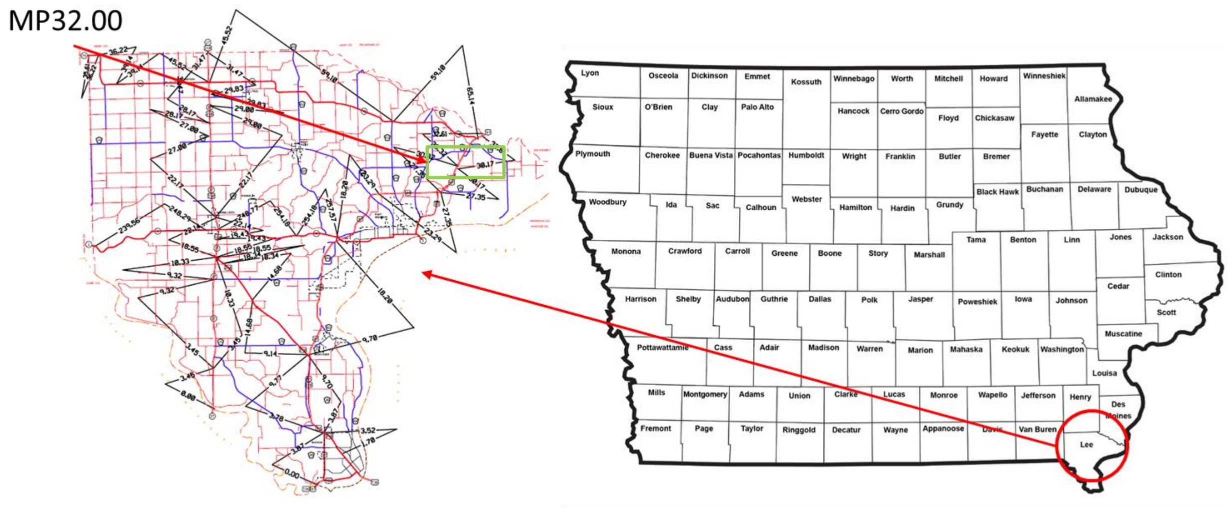


Figure 45. Field investigation on LC site 6 on US 65 near Polk County, Iowa

LC Site 7 – US 61 in Lee County, STA405 (MP 32.00)

Pavement Design and Construction

At LC site 7 (Figure 46), the PCC pavement was constructed starting at 7:00 a.m. on November 20, 1995. A 10 in. thick PCC layer was paved over a 10 in. granular subbase layer. During paving, the weather was sunny with a maximum temperature around 45°F. The PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and the travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 19.8 ft. Dowel bars, 18 in. in length and 1.5 in. in diameter, were used at the joints. The shoulder consisted of a 4 ft wide HMA shoulder.



Lee County

Figure 46. LC site 7 at US 61 near Lee County, Iowa

Pavement Materials

Table 25 summarizes the materials used in the concrete mixture for this site.

Table 25. Mix design of US 61 in Lee County, STA405 (MP 32.00)

Component	Description	Batch Weight
Portland cement	Lafarge Corp. - Type I	593 lbs/yd^3
GGBFS		
Coarse aggregate	Ames MINE (SG = 2.67, moisture content = 0.6 %)	1,529 lbs/yd^3
Fine aggregate	Ames MINE (SG = 2.66, moisture content = 4.0%)	1,582 lbs/yd^3
Water		242 lbs/yd^3
Admixture 1	Daravair Air-Entraining Admixture	oz/yd^3
Admixture 2	WRDA-82 Water Reducer	oz/yd^3
w/cm ratio		0.408
Air content		7.4

Field Investigation Description

Field investigations at LC site 7 on US 61 in Lee County were conducted on the afternoon of June 20, 2017. Photos and measurements taken to document site conditions included slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the field investigation, a few slabs showed longitudinal cracking on the pavement surface, and a large number of slabs showed transverse cracking. Figure 47 illustrates the cracking observed during the field survey.

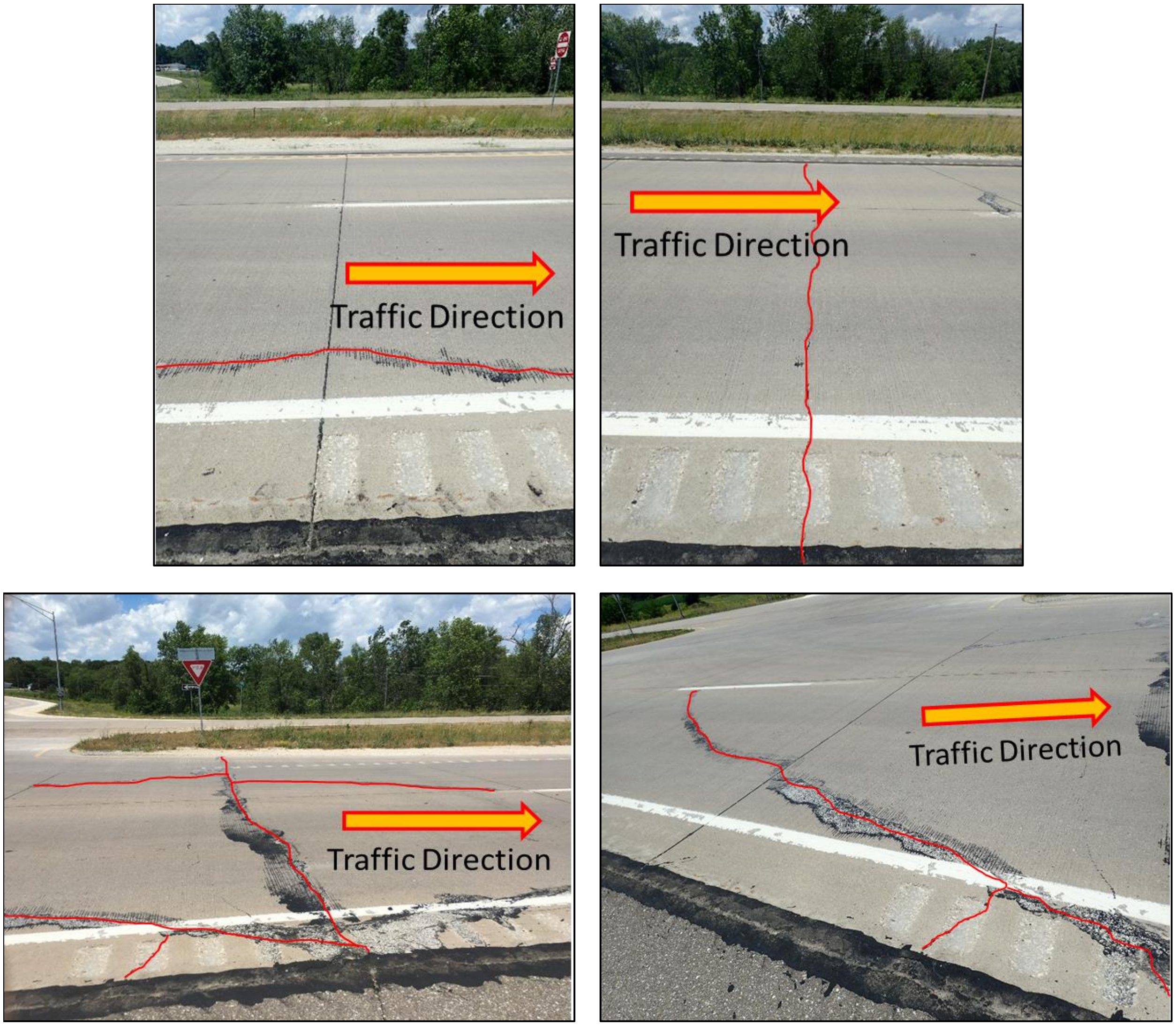
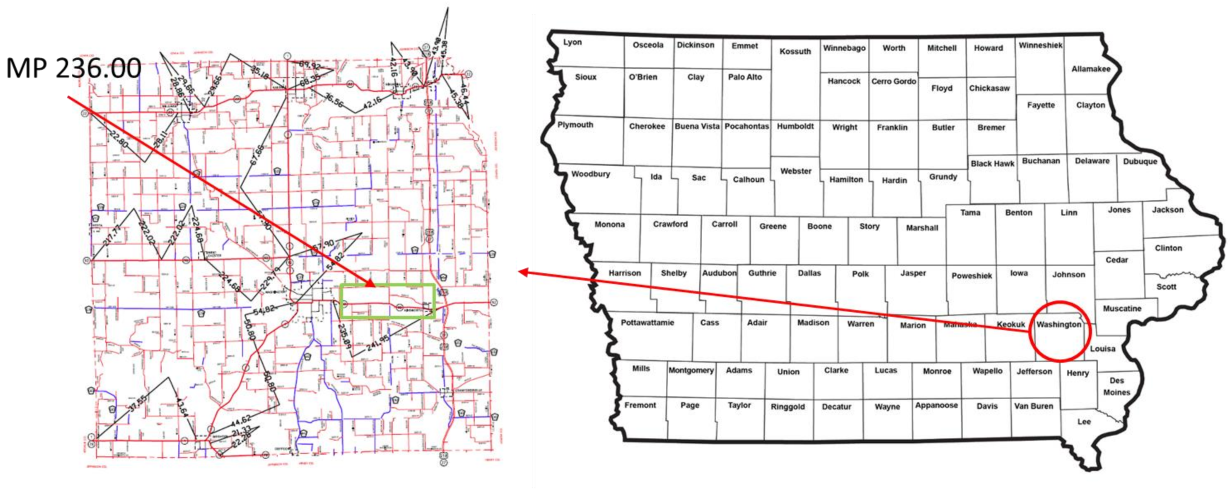


Figure 47. Field investigation on LC site 7 on US 61 near Lee County, Iowa

LC Site 8 – IA 92 in Washington County, STA364 (MP 236)

Pavement Design and Construction

At LC site 8 (Figure 48), the PCC pavement was constructed on September 24, 2001. A 9.5 in. thick PCC layer was paved over an 11 in. granular subbase layer. During paving, the weather was sunny with a maximum temperature around 60°F. The PCC pavement had designed widths of 12 ft and 14 ft, respectively, for the passing lane and travel lane. Skewed joints were adopted for this site. The transverse joint spacing was set at 20.4 ft. Dowel bars, 18 in. in length and 1.5 in. in diameter, were used at the joints. The shoulder consisted of a 14.5 ft wide granular shoulder.



Washington County

Figure 48. LC site 8 at IA 92 near Washington County

Pavement Materials

Table 26 summarizes the materials used in the concrete mixture for this site.

Table 26. Mix design of IA 92 in Washington County, STA364 (MP 236)

Component	Description	Batch Weight
Portland cement	Lafarge Corp. - Type ISM	470 lbs/yd^3
GGBFS		
Fly ash	ISG-Louisa Type C (SG = 2.68)	83 lbs/yd^3
Coarse aggregate	RIVER PRODUCT (SG = 2.66, moisture content = 0.8 %)	1,680 lbs/yd^3
Intermediate Aggregate	RIVER PRODUCT (SG = 2.68, moisture content = 1.3%)	413 lbs/yd^3
Fine aggregate	RIVER PRODUCT (SG = 2.68, moisture content = 3.4%)	1,043 lbs/yd^3
Water		225.6 lbs/yd^3
Admixture 1	GRACE/DARAVAIR 1400 Air-Entraining Admixture	2.92 oz/yd^3
Admixture 2	GRACE/WRDA Water Reducer	3.6 oz/cwt
w/cm ratio		0.408
Air content		N/A

Field Investigation Description

Field investigations at LC site 8 on IA 92 in Washington County were conducted on the afternoon of June 20, 2017. Photos and measurements taken to document site conditions included slab geometry, shoulder type and geometry, joint type, and amount and location of cracks. During the field investigation, a large number of slabs showed transverse cracking. Figure 49 illustrates the cracking observed during the field survey.

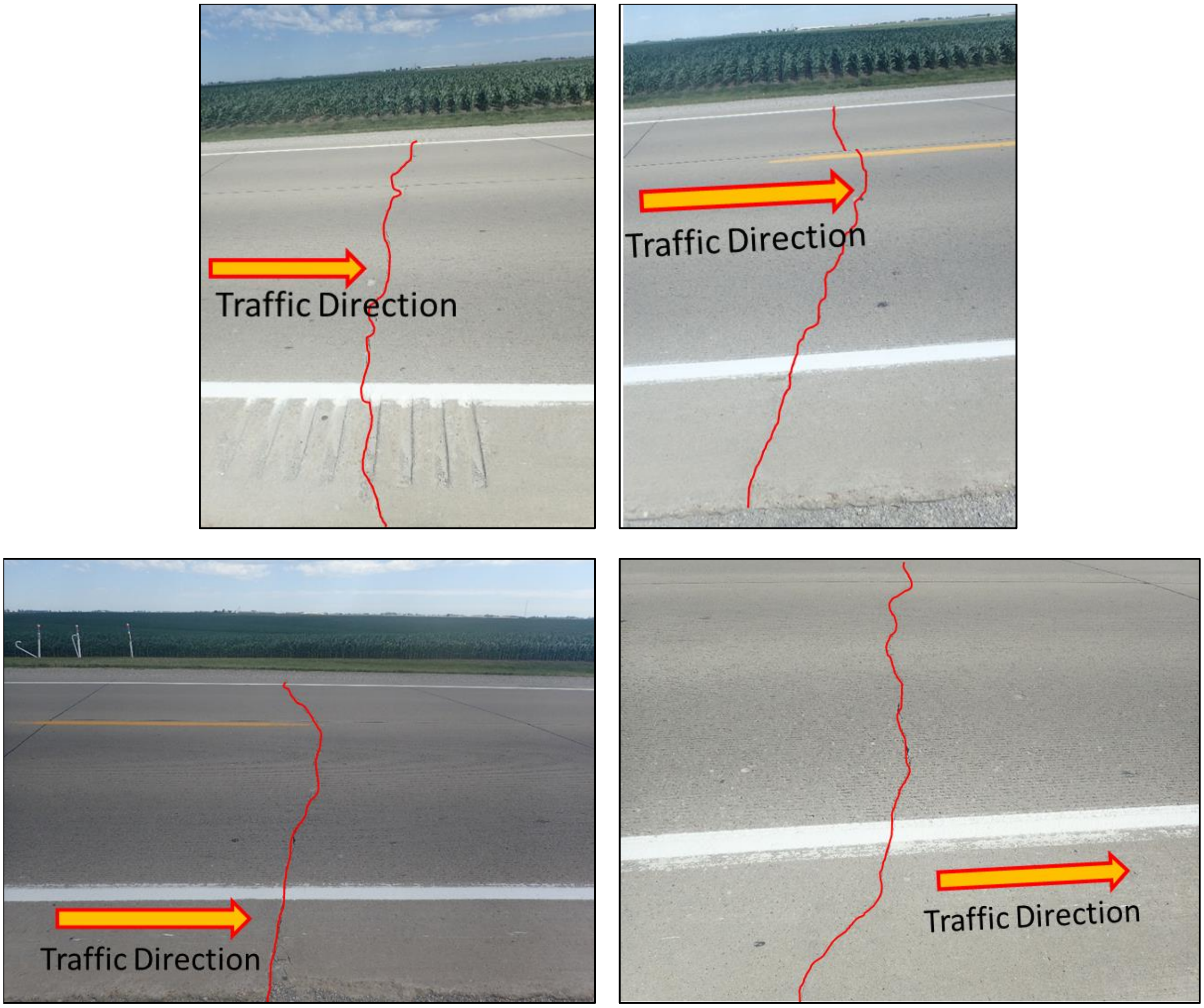


Figure 49. Field investigation on LC site 8 on IA 92 near Washington County, Iowa

Discussion of Field Investigation Results

The previous sections documented the field investigations of cracking on widened concrete slabs in Iowa. Information that was summarized related to pavement design, concrete mix design, construction-related parameters, and pavement performance for the visited sites. Based on this field survey, the researchers found that the observed longitudinal cracking occurred mainly on the widened traffic lane about 2 to 4 ft away from slab edges. These cracks were parallel to the traffic direction and usually extended across the entire length of two to three slabs. There were also some longitudinal cracks exhibiting arc shapes that started from a transverse joint and ended at the slab edge; these could be due to foundation issues such as improper subbase/subgrade compaction or frost heave (Voigt 2002). Very little longitudinal cracking was observed on the passing lanes, and the observed longitudinal cracking usually started from transverse joints. Additionally, some longitudinal cracks were found in a concrete patching area (Figure 37, Figure 39, and Figure 41). These were usually located at the middle of the edges of the square patching area perpendicular to the traffic direction. Transverse cracks were also found at a few sites. However, there is a possibility that these near-patching cracks may already have existed before the patching.

Effects of JPCP Design-Related Features on Longitudinal Cracking

Table 27 and Table 28 summarize pavement design-related parameters at control sites and LC sites, respectively. These two tables show that while all the sites had applied skewed joints (except for control site 3 in Tama County), only limited study was performed to investigate the effect of skewed joints on longitudinal cracking. Through numerical analysis, Rasmussen et al. (2007) demonstrated that skewed joints effectively increase slab dimensions and consequently result in higher curling and warping stresses and deflections at the corners than slabs with a rectangular shape. Owusu-Ababio and Schmitt (2013) and Johnston (2014) indicated that pavements using skewed joints have higher potential for longitudinal cracking. Because the sites visited all had similar slab geometry, no clear trend was observed to correlate the slab geometry with the amount of longitudinal cracking documented.

Table 27. Summary of pavement design-related parameters for control sites

Control Site No.	County	Route	Slab Length	Slab Width	Slab Thick.	Shoulder Width and Type	Base Thick.	Joint Type
1	Polk	US 65	20.0 ft	14.2 ft	12.0 in.	8.0 ft PCC	10.0 in.	Skewed
2	Polk	US 65	20.0 ft	14.2 ft	11.0 in.	8.0 ft PCC	10.0 in.	Skewed
3	Tama	US 30	20.1 ft	14.1 ft	10.0 in.	4.0 ft HMA	10.0 in.	Rectangular
4	Story	US 30	20.2 ft	14.0 ft	10.0 in.	9.5 ft Granular	10.0 in.	Skewed

Table 28. Summary of pavement design-related parameters for LC sites

LC Site No.	County	Route	Slab Length	Slab Width	Slab Thick.	Shoulder Width and Type	Base Thick.	Joint type
1	Linn	US 151	21.2 ft	14.0 ft	10.5 in.	9.0 ft Granular	10.5 in.	Skewed
2	Linn	US 30	20.7 ft	14.1 ft	10.0 in.	10.0 ft Granular	10.5 in.	Skewed
3	Mahaska	IA 163	19.7 ft	13.9 ft	10.0 in.	13.0 ft Granular	10.0 in.	Skewed
4	Henry	US 218	19.8 ft	13.8 ft	10.5 in.	4.0 ft HMA	10.0 in.	Skewed
5	Jasper	IA 163	20.0 ft	14.0 ft	10.0 in.	4.0 ft HMA	10.0 in.	Skewed
6	Polk	US 65	20.2 ft	14.1 ft	12.0 in.	9.0 ft HMA	10.0 in.	Skewed
7	Lee	US 61	19.8 ft	14.1 ft	10.0 in.	4.0 ft HMA	10.0 in.	Skewed
8	Washington	IA 92	20.4 ft	14.0 ft	9.5 in.	14.5 ft Granular	11.0 in.	Skewed

Table 29 and Table 30 summarize pavement performance for the control sites and LC sites, respectively. The International Roughness Index (IRI), pavement condition index (PCI), and average annual daily truck traffic (AADTT) data were obtained from 2015 Pavement Management Information Systems (PMIS) (Iowa DOT 2015). The number of slabs having longitudinal and transverse cracks per mile at the investigated sites was also recorded for pavement performance assessment (at least 2 miles were surveyed for each site). Based on Table 29 and Table 30, the key findings are as follows:

- There was no cracking observed at control sites 2, 3, and 4, while a few longitudinal cracks (lengths less than 3 ft) with low severity levels were found at control site 1.
- LC site 1 at US 151 in Linn County, LC site 2 at US 30 in Linn County, and LC site 3 at IA 163 in Mahaska County exhibited the highest number of slabs with longitudinal cracking.
- LC site 4 at US 218 in Henry County had fewer slabs with longitudinal cracking than LC sites 1, 2, and 3 had but more slabs with such cracking than other sites.
- LC site 5 at IA 163 in Jasper County, LC site 6 at US 65 in Polk County, and LC site 7 at US 61 in Lee County had the lowest amounts of slabs exhibiting longitudinal cracking.
- No longitudinal cracking was observed at LC site 8 at IA 92 in Washington County.
- LC site 7 at US 61 in Lee County and LC site 8 at IA 92 in Washington County had many slabs where transverse cracking was seen.
- The sites with higher IRI and lower PCI (LC sites 1, 2, and 8) also had higher numbers of cracks (both longitudinal and transverse).
- The sites with higher truck traffic volumes also had more longitudinal cracks observed.

Table 29. Summary of pavement performance for control sites

Control Site No.	County	Route	IRI (in./mile)	PCI	AADTT in 2015	Shoulder Width and Type	L- Crack (per mile)	T-Crack (per mile)
1	Polk	US 65	97	88	1,517	8.0 ft PCC	2.6	0
2	Polk	US 65	108	86	1,735	8.0 ft PCC	0	0
3	Tama	US 30	88	81	1,079	4.0 ft HMA	0	0
4	Story	US 30	104	86	784	9.5 ft Granular	0	0

Table 30. Summary of pavement performance for LC sites

LC Site No.	County	Route	IRI (in./mile)	PCI	AADTT in 2015	Shoulder Width and Type	L- Crack (per mile)	T-Crack (per mile)
1	Linn	US 151	146	57	2,927	9.0 ft Granular	10.5	2
2	Linn	US 30	148	66	1,191	10.0 ft Granular	6.5	0
3	Mahaska	IA 163	93	85	1,585	13.0 ft Granular	7	1
4	Henry	US 218	109	82	1,699	4.0 ft HMA	5	1.5
5	Jasper	IA 163	80	87	1,368	4.0 ft HMA	2.5	0.5
6	Polk	US 65	105	84	1,178	9.0 ft HMA	2	0
7	Lee	US 61	91	80	1,153	4.0 ft HMA	2.5	6
8	Washington	IA 92	141	74	402	14.5 ft Granular	0	11

Figure 50 illustrates the number of slabs exhibiting longitudinal cracking correlated to truck traffic volume and shoulder type. Each bar indicates a site, and the height of the bar is associated with the truck traffic volume. A color was assigned to each site to indicate the predefined range of the number of slabs exhibiting longitudinal cracking (frequency) and the associated severity levels of the cracks based on the distress identification manual for the LTPP program. Low frequency was defined as three or fewer slabs per mile having longitudinal cracking. Moderate frequency was defined as more than three but fewer than or equal to six slabs per mile having longitudinal cracking. High frequency was defined as more than six slabs per mile having longitudinal cracking. All sites were sorted into three groups based on their shoulder types: PCC, HMA, and granular.

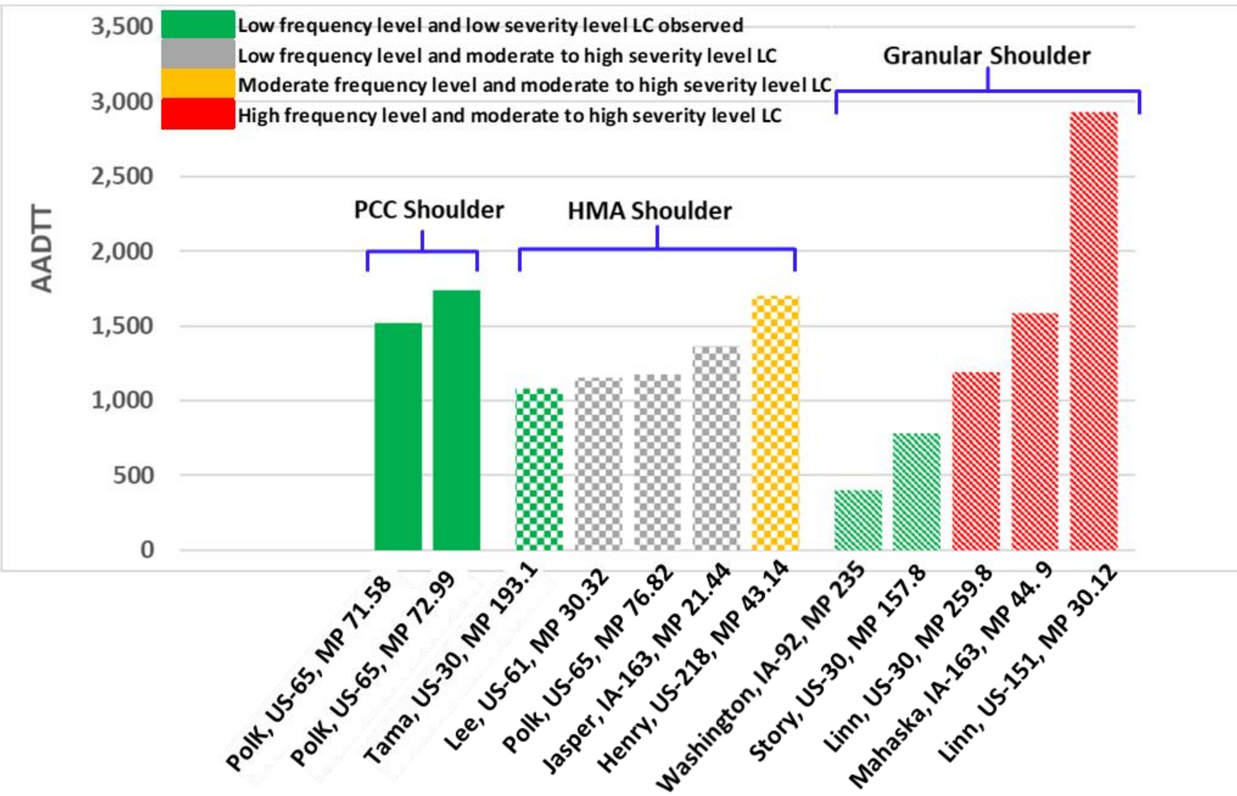


Figure 50. Frequency and severity level of slabs having longitudinal cracking versus truck traffic volume and shoulder type

According to this figure, the sites with granular shoulders are more prone to a high frequency of longitudinal cracking with moderate- to high-severity cracking, while the sites with HMA shoulders provide fair performance with less longitudinal cracking and moderate- to high-severity cracking. The sites with PCC shoulders perform best, with less longitudinal cracking (crack lengths less than 3 ft) and low-severity cracking (with no cracks observed at control site 2 in Polk County). It should be noted that there are also two sites (control site 4 in Story County and LC site 8 in Washington County) with granular shoulders that have no longitudinal cracking observed. This can be explained by taking into account the extremely low truck traffic volumes (< 800) at these two sites compared to the others. Additionally, although the sites with PCC shoulder (control sites 1 and 2) have higher truck traffic volume than other sites, lower frequencies of cracking were observed at these two sites, indicating that PCC shoulders perform better than HMA shoulders and granular shoulders in terms of controlling longitudinal cracking. The sites with HMA shoulders have fewer longitudinal cracks in comparison to the sites with granular shoulders.

Effects of Mix Design-Related Features on Longitudinal Cracking

Table 31 and Table 32 summarize the mix design for the sites visited. While the use of lower water-demand limestone, coarser sand, and less cement can help to lower the likelihood of uncontrolled cracks, no clear relationship was observed between the amount of longitudinal cracking and the concrete mixture ingredients.

Table 31. Summary of mix design-related parameters for control sites

Control Site No	County	Route	Cement lbs/cy	Fly Ash lbs/cy	Coarse Agg. lbs/cy	Fine Agg. lbs/cy	Water lbs/cy	w/cm
1	Polk	US 65	457	114	1,590	1,429	227	0.4
2	Polk	US 65	457	114	1,439	1,555	238	0.42
3	Tama	US 30	448	112	1,539	1,272	224	0.4
4	Story	US 30	457	114	1,653	1,446	282	0.49

Table 32. Summary of mix design-related parameters for LC sites

LC Site No.	County	Route	Cement lbs/cy	Fly Ash lbs/cy	Coarse Agg. lbs/cy	Fine Agg. lbs/cy	Water lbs/cy	w/cm
1	Linn	US 151	479	85	1,675	1,415	222	0.39
2	Linn	US 30	451	113	1,702	1,447	240	0.43
3	Mahaska	IA 163	457	114	1,625	1,415	221	0.39
4	Henry[1]	US 218	470	83	1,621	1,244	227	0.41
5	Jasper	IA 163	457	114	1,629	1,424	206	0.36
6	Polk	US 65	474	119	1,478	1,584	251	0.42
7	Lee	US 61	593	N/A	1,529	1,528	242	0.41
8	Washington[2]	IA 92	470	83	1,680	1,043	226	0.41

[1]Henry County site used 290 lbs/cy intermediate aggregate and [2]Washington County site used 413 lbs/cy intermediate aggregate in the mix design.

Table 33 and Table 34 summarize the construction-related parameters for all the control sites and LC sites, respectively. In general, late afternoon or nighttime paving can lead to less built-in curling and warping and therefore less potential for longitudinal cracking. However, based on these field study results, no clear relationship was observed between the amount of longitudinal cracking and construction-related parameters.

Table 33. Summary of construction-related parameters for control sites

Control Site No	County	Route	Month	Time	Weather	Max Ambient Temperature during Paving
1	Polk	US 65	August	6:00 a.m.	Sunny	85°F
2	Polk	US 65	April	16:25 p.m.	Sunny	68°F
3	Tama	US 30	July	3:00 p.m.	Sunny	90°F
4	Story	US 30	May	8:00 p.m.	Sunny	85°F

Table 34. Summary of construction-related parameters for LC sites

LC Site No.	County	Route	Month	Time	Weather	Max Ambient Temp. during Paving
1	Linn	US 151	November	8:00 a.m.	Sunny	67°F
2	Linn	US 30	May	8:00 a.m.	Sunny	78°F
3	Mahaska	IA 163	September	8:55 a.m.	Sunny	80°F
4	Henry	US 218	July	N/A	Cloudy	90°F
5	Jasper	IA 163	May	7:00 a.m.	Cloudy	52°F
6	Polk	US 65	August	11:40 a.m.	Cloudy	89°F
7	Lee	US 61	November	7:00 a.m.	Sunny	45°F
8	Washington	IA 92	September	N/A	Sunny	60°F

Results from Concrete Cores from LC Sites

Concrete cores were taken by Iowa DOT crews at these longitudinal cracking locations (Appendix B):

- LC site 1 on US 30 in Linn County
- LC site 2 on US 151 in Linn County
- LC site 3 on IA 163 in Mahaska County
- LC site 5 on IA 163 in Jasper County
- LC site 6 on US 65 in Polk County

Most cores exhibited top-down cracking, and the other cores exhibited cracking across the whole cylinder depth. The cores with full-depth cracks split open after being extracted (Figure 115).

Based on visual inspections, all cores revealed that the aggregates in the mixes were uniformly distributed. It was difficult to observe aggregate segregation or vibrator trails. There also were no excessive entrapped air voids observed in these cores. The cracks on the cores were found to cross the aggregate, indicating no issue associated with concrete early strength, in agreement with the construction report results that all the concrete satisfied strength test requirements.

NUMERICAL MODELING OF LONGITUDINAL CRACKING

Widened slabs have been used to mitigate transverse cracking in JPCP because it has been known since the development of Westergard's (1926) theory that edge load compared to interior and corner loads usually produces the highest stress on JPCP. By using widened slabs, load is not applied to slab edges, so the transverse cracking potential is significantly diminished. As the literature review section of this report documented, it has been well-known that widened slabs increase longitudinal cracking potential in JPCP.

Two finite element analysis software packages, ISLAB 2005 and EverFE 2.25, were used as the main structural modeling tools for generating rigid pavement responses to simulate Iowa widened slab JPCP under mechanical and temperature loading.

The objectives of this chapter are as follows:

- Case 1 study: single axle load simulations using ISLAB 2005 to seek understanding of critical loading cases, including both mechanical and temperature loading that increases longitudinal cracking potential in JPCP
- Case 2 study: truck load simulations using ISLAB 2005 to simulate longitudinal crack initiation on transverse joints
- Case 3 study: skewed joint simulations using EverFE 2.25 to compare effects of rectangular and skewed joints on longitudinal cracking potential
- Case 4 study: examining shoulder design alternatives using ISLAB 2005 to compare different shoulder alternatives (paved shoulder practices such as tied PCC and HMA shoulders, and granular shoulder) in terms of their contributions to mitigation of longitudinal cracking potential.

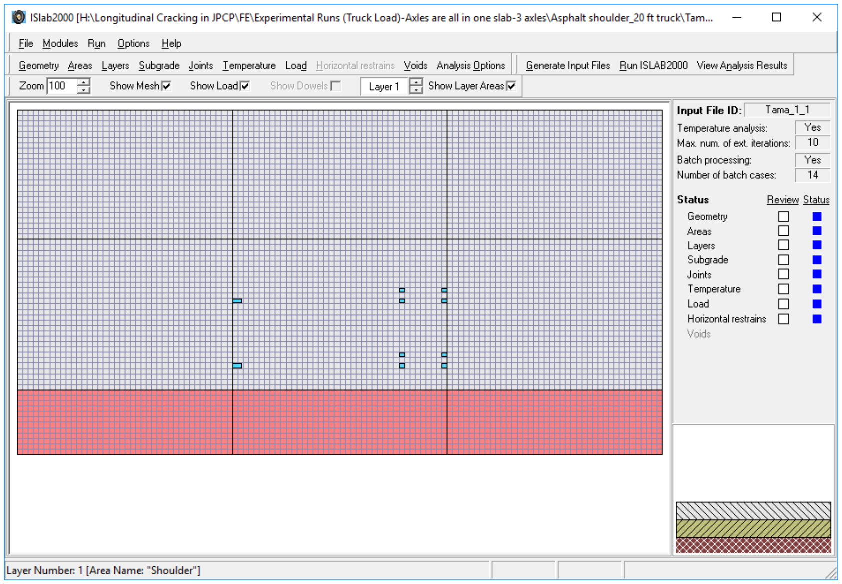
ISLAB 2005 was specifically developed for rigid pavement analysis. It has evolved historically, and previous versions have had other names: ILSL2, ILLI-SLAB, and ISLAB2000. The earliest version of ISLAB 2005 was ILSL2 (Khazanovich 1994), developed through collaboration of many partners: ERES Consultants in cooperation with the Michigan and Minnesota Departments of Transportation, Michigan Technological University, University of Michigan, Michigan State University, and University of Minnesota.

ISLAB 2005 has some advanced features that significantly assist in modeling rigid pavement systems as realistically as possible (ARA 2004, Mu and Vandenbossche 2016, Kim et al. 2014). Among these features are the following abilities:

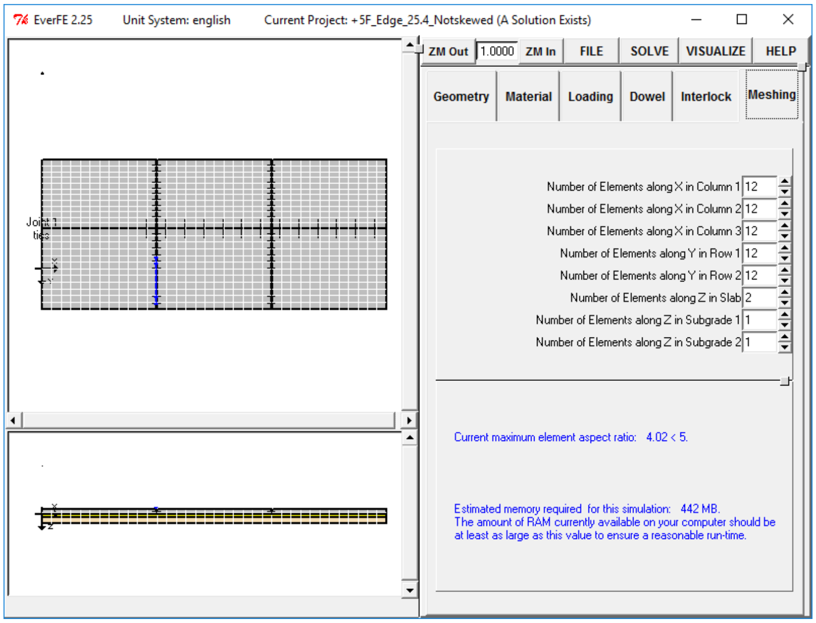
- Select various subgrade models such as Winkler, elastic solid, Pasternak, Kerr-Vlasov, and Zhemochkin-Sinitsyn-Shtaerman
- Analyze the effects of linear and nonlinear temperature distribution throughout the pavement thickness
- Model interaction between a slab and its base using three models: bonded, unbonded, and Totsky

- Model a portion of a pavement system with different properties and features than the other parts of the pavement system

EverFE 2.25 is a three-dimensional, finite element (3D-FE) package developed at the University of Washington and the University of Maine. EverFE 2.25 is the latest version of EverFE software, whose previous version was EverFE 2.24 (Davids 2018). EverFE 2.25 has several advanced modeling features, including modeling skewed joints, aggregate interlock joint shear transfer, dowel joint shear transfer, and temperature nonlinearities. Figure 51 shows screenshots from ISLAB 2005 and EverFE 2.25.



(a)



(b)

Figure 51. Screenshots from (a) ISLAB 2005 and (b) EverFE 2.25

Numerical Modeling Approach

A typical Iowa widened JPCP slab was modeled using a six-slab setup (three widened slabs in the traffic direction and three regular slabs adjacent to the widened slabs). Model definitions used throughout this report are shown in Figure 52, where it can be seen that widened slabs have a width of 14 ft while regular slabs have a width of 12 ft. The lane edge shows where the lane marking is located, typically 2 ft away from the widened slab edge.

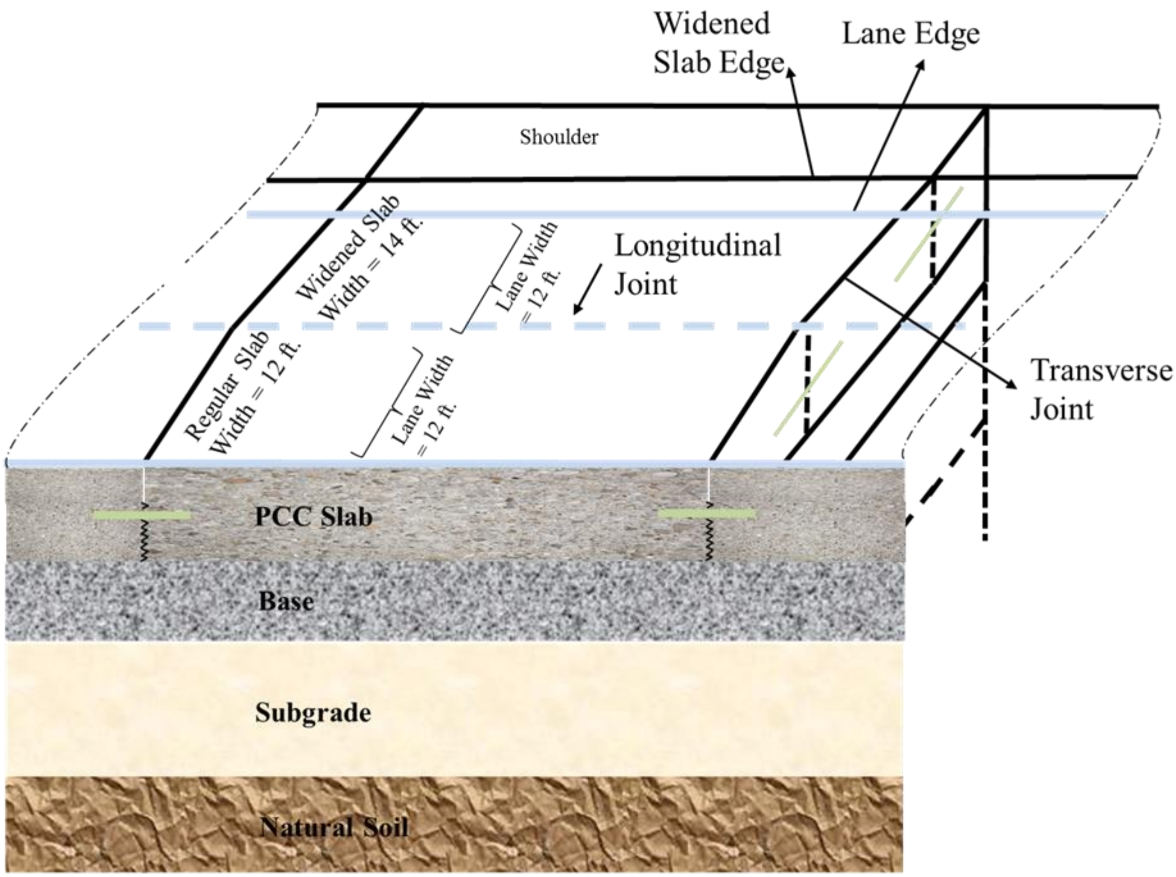


Figure 52. FEA model definitions

A pavement configuration with a 10 in. PCC thickness, 10 in. granular base, and typical Iowa subgrade was used. Table 35 shows details of the inputs used in the FEA model.

Table 35. FEA model inputs

Slab Size and Properties	
Slab Size in Traffic Direction (ft)	20
Slab Size in Transverse Direction (ft) - Regular Slab	12
Slab Size in Transverse Direction (ft) - Widened Slab	14
Element Size for Mesh (in.)	6
Slab Thickness (in.)	10
Elastic Modulus (psi)	4,000,000
Poisson Ratio	0.2
CTE (in./ºF)	4.90E-06
Unit weight (ksi/in^3)	8.68E-05
Granular Base Size and Properties	
Base Thickness (in.)	10
Elastic Modulus (psi)	35,000
Poisson Ratio	0.35
CTE (in./ºF)	5.00E-06
Unit Weight (ksi/in^3)	7.36E-05
Subgrade Properties	
k (psi/in.)	163.4
Mechanical and Temperature Loading Properties	
Load level (lbs.)	20,000 (single axle), 34,000 (tandem axle)
Tire Pressure (psi)	120
Load Location in Traffic Direction	Every 2 ft for single axle load cases
Wander Pattern	0, 1 and 2 ft away from lane edge (for single axle load cases) 0. 0.5, 1, 1.5, and 2 ft away from lane edge (for truck load)
Long term LTE (%)	70
Temperature Gradient (in./◦F)	-2 to 2 with an increment of 0.2

ISLAB 2005 discretizes modeled slabs into meshes and nodes. FEA uses a fine mesh size (nominal element size of 6 in.). At the completion of FEA, ISLAB produces an output file in “txt” format for each FEA scenario (630 txt files in total for the single axle load cases introduced later in this report) that has stress (in x direction, y direction, principal stress, and von Mises stress) and deflection results for each nodal value. These output files require post-processing so that critical pavement responses for each FEA scenario can be calculated and extracted.

A post-processing scheme was developed using Microsoft Excel VBA (Visual Basic for Applications) and MATLAB (version 9.3.0.713579 [R2017b]). It combines all output files, calculates and summarizes critical pavement responses for each FEA scenario, and presents them in a summary worksheet.

Figure 53 demonstrates the post-processing scheme for FEA results. Initially, output files are transferred into a master Excel spreadsheet using Microsoft Excel VBA. Then, using MATLAB, critical pavement responses are calculated, extracted, and written into a summary Excel spreadsheet (Figure 53). Critical stresses are summarized as follows: maximum top and bottom (top and bottom of slab) tensile stresses in x and y directions, maximum top and bottom principal and von Mises stresses, and maximum deflection.

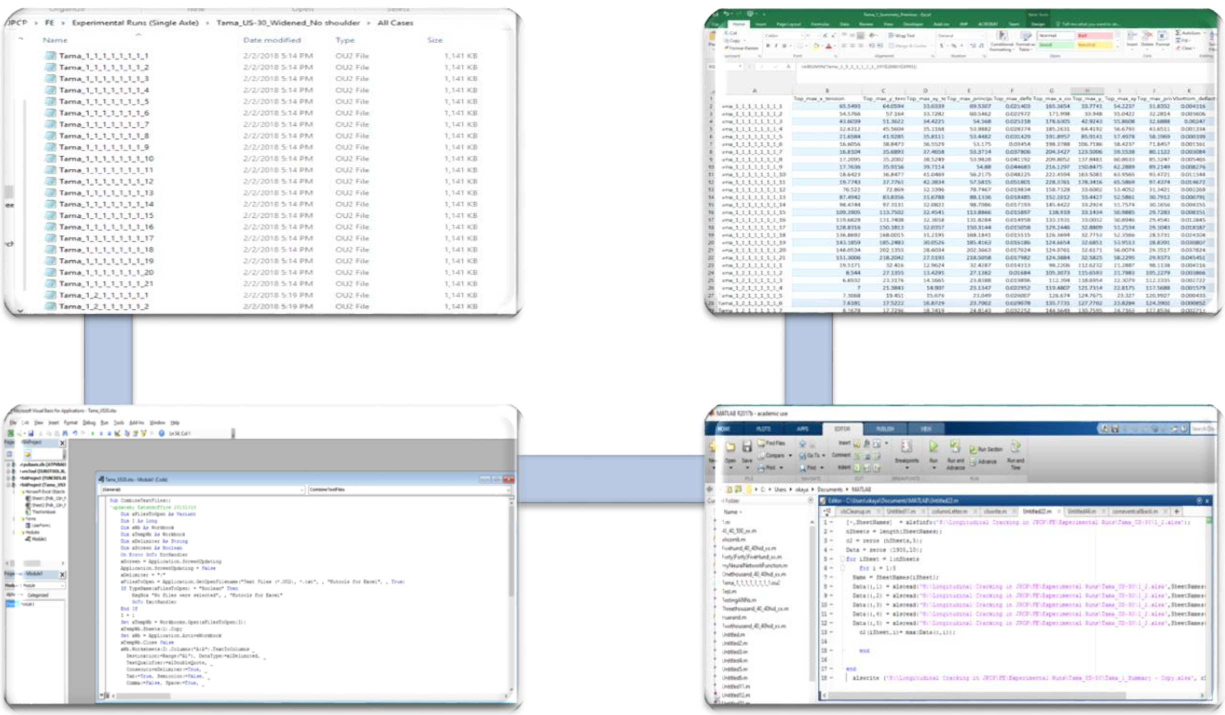
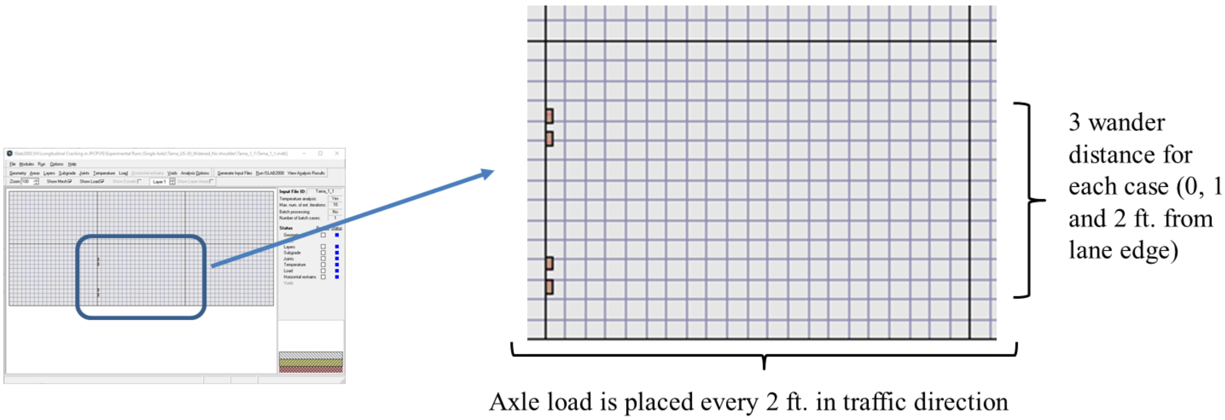


Figure 53. Post-processing scheme for FEA results

Numerical Modeling Case 1 Study: Single Axle Load Simulations

Several FEA models were developed for (1) mechanical load only cases and (2) combined temperature and mechanical load cases. To simulate mechanical load, a single axle with dual wheels carrying a total load of 20,000 lbs was used. To simulate temperature loads, 21 different cases were used, with a temperature gradient between -2 to 2 (in./°F) in increments of 0.2 (in./°F) (Table 35). A single axle load was placed every 2 ft in the traffic direction and three wander distances (0, 1, and 2 ft from lane edges). For the Case 1 study, a total of 630 scenarios were modeled in ISLAB 2005 (Figure 54).



For each load and wander cases, 21 different temperature loading scenarios = 10×3×21= **630** scenarios

Figure 54. Single axle load cases

Critical pavement responses for each load case were analyzed. Detailed tensile stress and deflection distributions for both transverse-edge loading and mid-slab loading will be presented in the following section, along with a summary of tensile stress distribution for all cases investigated.

Transverse Edge Loading

Transverse edge is one of the critical load locations that was investigated. In this loading scenario, a single axle load is placed on a transverse joint (Figure 55).

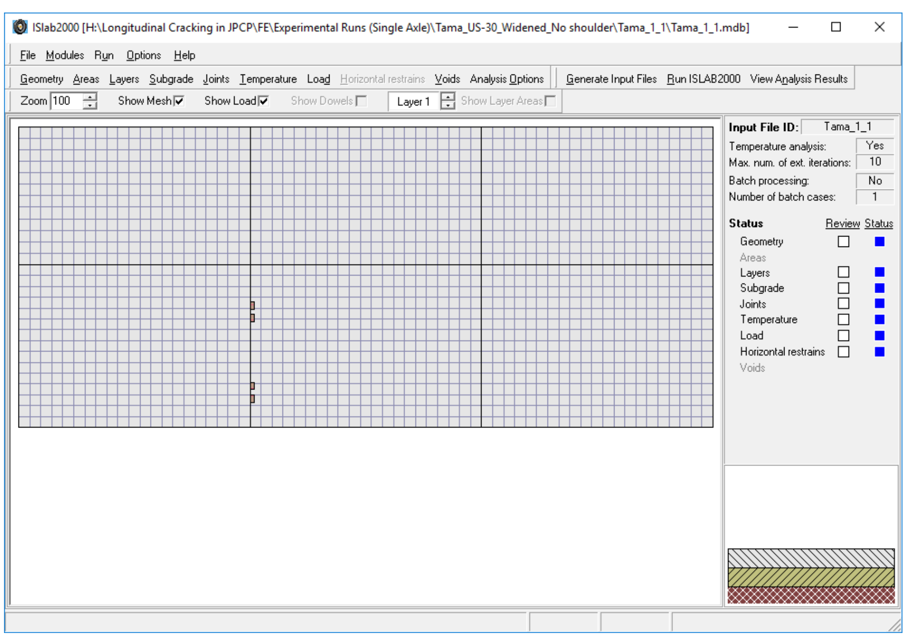
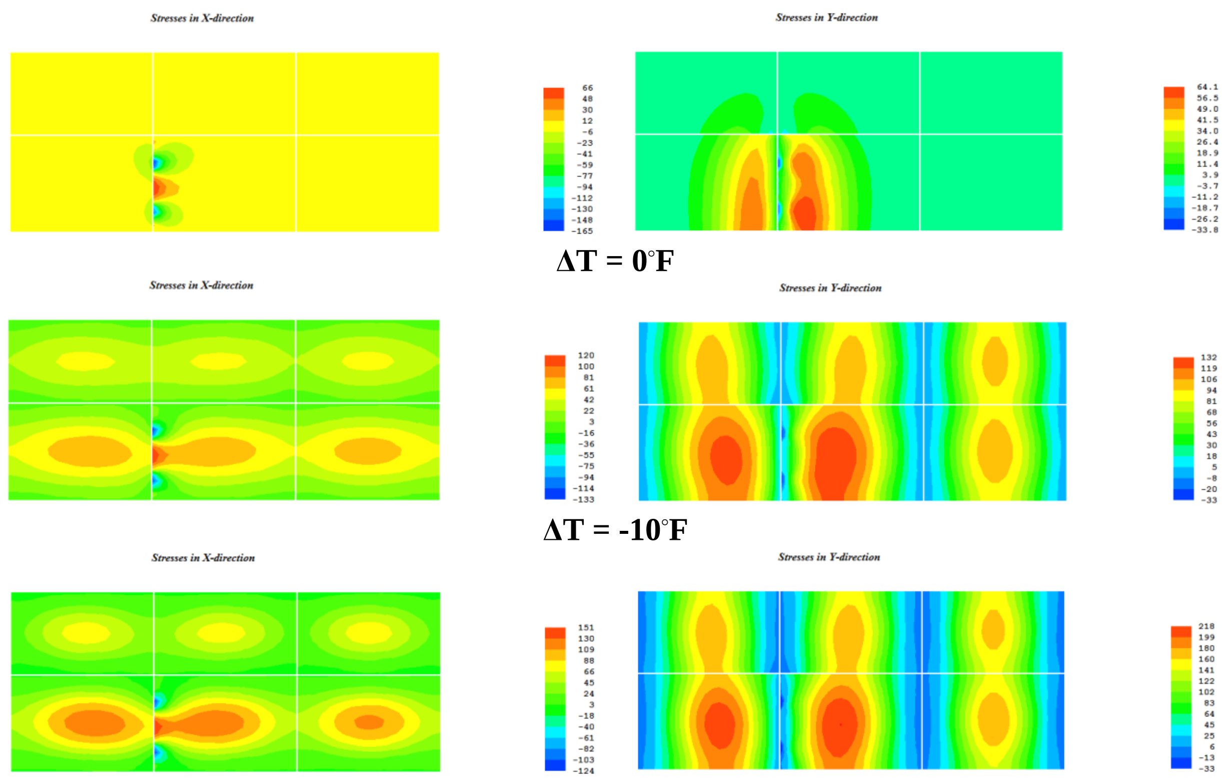


Figure 55. Case 1 study: Transverse edge loading

ISLAB 2005 produces tensile stress results in the x and y directions (x direction is perpendicular to the traffic direction, y direction is the traffic direction). Initially, tensile stress results on the slab surface (top) in the x and y directions as well as deflection results were analyzed to determine which of the tensile stress types (in the x or y direction) is the critical tensile stress type producing longitudinal cracking.

Figure 56 shows tensile stress distribution in the x and y directions for three different temperature-loading scenarios:

1. No temperature load (ΔT = 0°F)
2. Temperature difference between bottom and top of slab is 10°F (ΔT = top-bottom = -10°F)
3. Temperature load with ΔT = -20°F



Tensile stresses in x direction **Tensile stresses in y direction**

Figure 56. Case 1 study: Transverse edge loading - top tensile stress distribution

The graphs on the left side show top tensile stress results in the x direction (i.e., perpendicular to the traffic direction), while the graphs on the right side show top tensile stresses in the y direction (i.e., traffic direction) for three different temperature-loading cases. The red color in the graphs represents tensile stresses, while the blue color represents compressive stresses. As can be seen in Figure 57, tensile stresses in the x direction are the critical stresses for longitudinal cracking because they are tensile stresses perpendicular to the traffic direction. The tensile stresses in the y direction would be critical for transverse cracking because they are tensile stresses parallel to the traffic direction. As can be seen in Figure 56, as the temperature gradient increases, the top tensile stresses in both directions also increase, and the high top tensile stresses in the x direction occur around the transverse joint.

Figure 57 shows the deflection distribution when three different temperature-loading scenarios are used:

1. No temperature load (ΔT = 0°F)
2. Temperature load with ΔT = -10°F
3. Temperature load with ΔT = -20°F

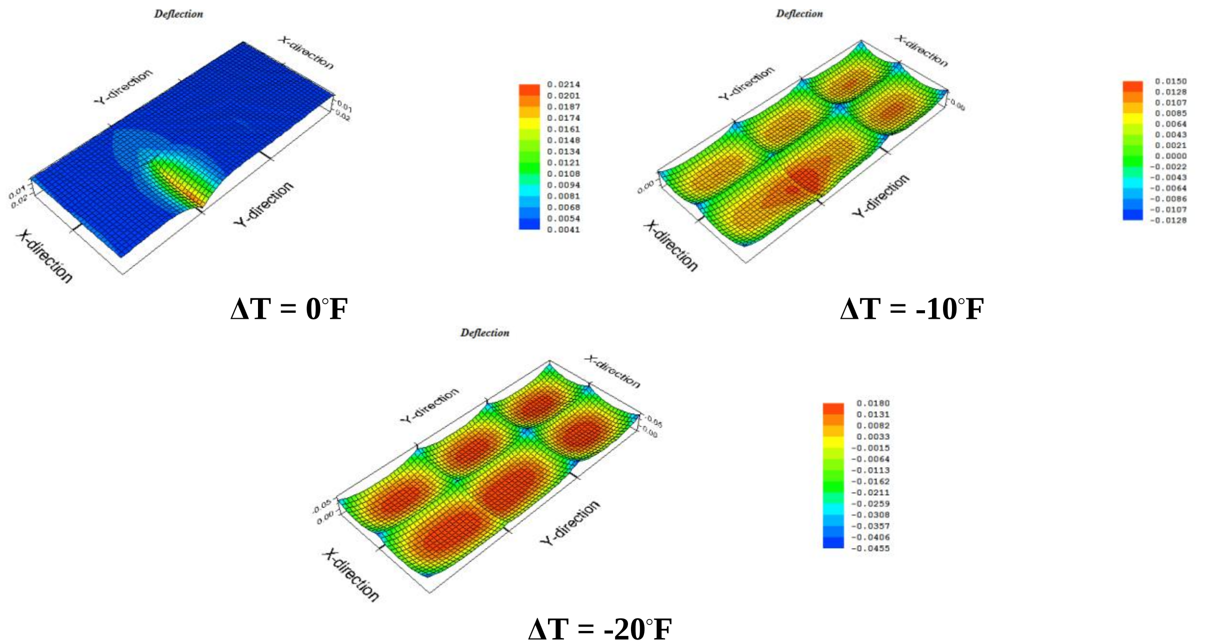


Figure 57. Case 1 study: Transverse edge loading - deflection distribution

Mid-Slab Loading

Mid-slab is one of the critical load locations that also was investigated. In this loading scenario, a single axle load is placed at the middle of the slab in the traffic direction (Figure 58).

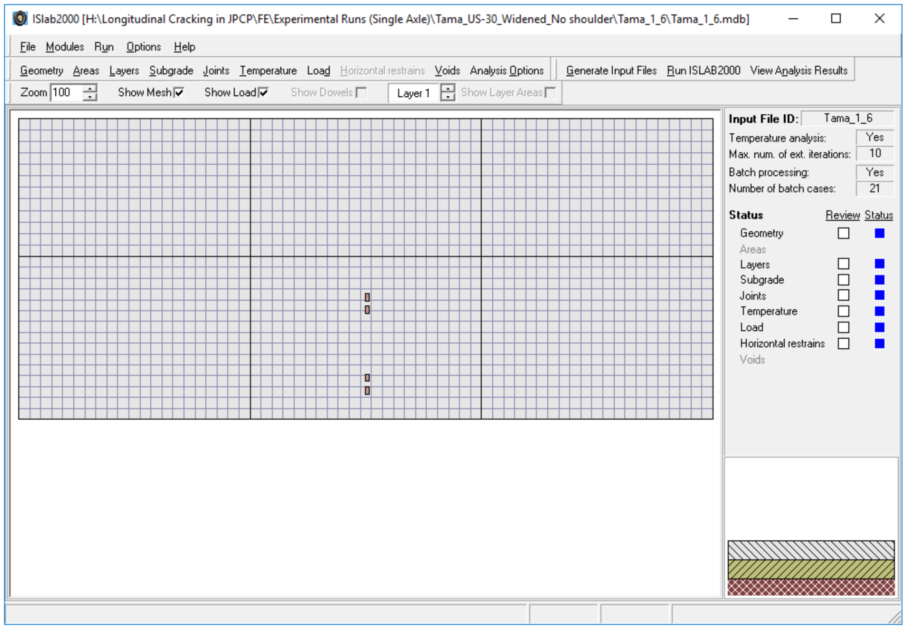
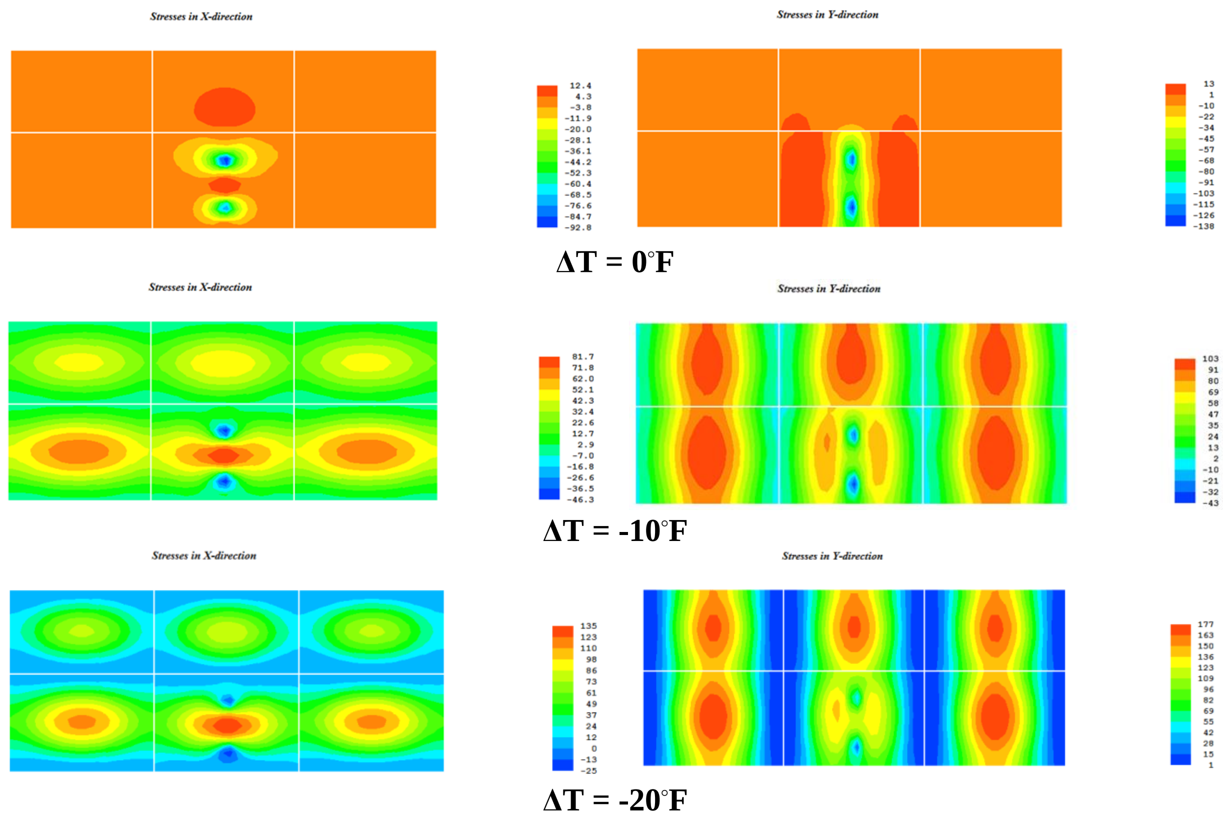


Figure 58. Case 1 study: Mid-slab loading

Figure 59 shows the tensile stress distribution in x and y directions under three different temperature loading scenarios:

1. No temperature load (ΔT = 0°F)
2. Temperature load with ΔT = -10°F
3. Temperature load with ΔT = -20°F

The graphs on the left side show top tensile stress results in the x direction (i.e., perpendicular to the traffic direction), while the graphs on the right side show top tensile stresses in the y direction (i.e., the traffic direction) for the three different temperature loading cases. The red color represents tensile stresses while the blue color represents compressive stresses. As seen in Figure 59, as temperature gradient increases, the top tensile stresses also increase.



Tensile stresses in x direction **Tensile stresses in y direction**

Figure 59. Case 1 study: Mid-slab loading - top tensile stress distribution

Figure 60 shows the deflection distribution when three different temperature-loading scenarios are used:

1. No temperature load (ΔT = 0°F)
2. Temperature load with ΔT = -10°F
3. Temperature load with ΔT = -20°F

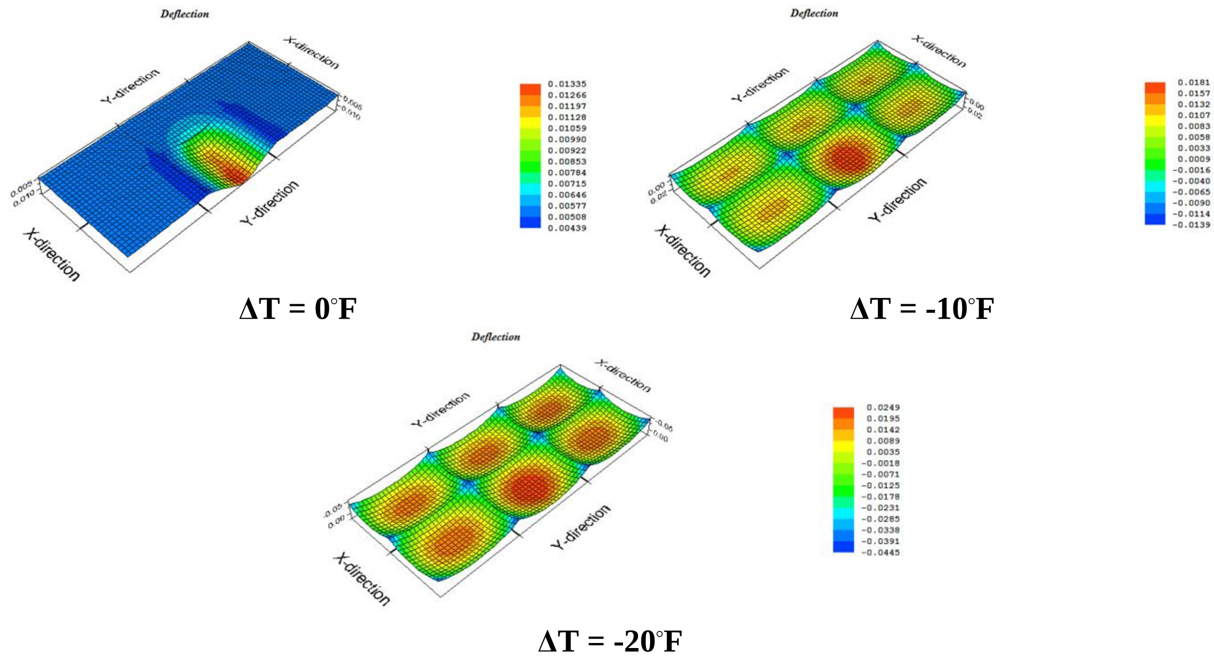


Figure 60. Case 1 study: Mid-slab loading - deflection distribution

Single Axle Load Simulation Results

Figure 61 shows the top-to-bottom tensile stress ratio distribution when a single axle mechanical load is applied at various locations in both traffic and wander directions. As seen in Figure 61, higher top-to-bottom tensile stress ratio values were observed when single axle mechanical load was applied on transverse joints. There was no significant difference in top-to-bottom tensile stress ratio results for different wander distances. However, a slightly higher top-to-bottom stress ratio was observed when the outer wheel of the single axle was placed 1 ft away from the lane edge compared to cases when the outer wheel of the single axle was placed on the lane edge and 2 ft away from the lane edge.

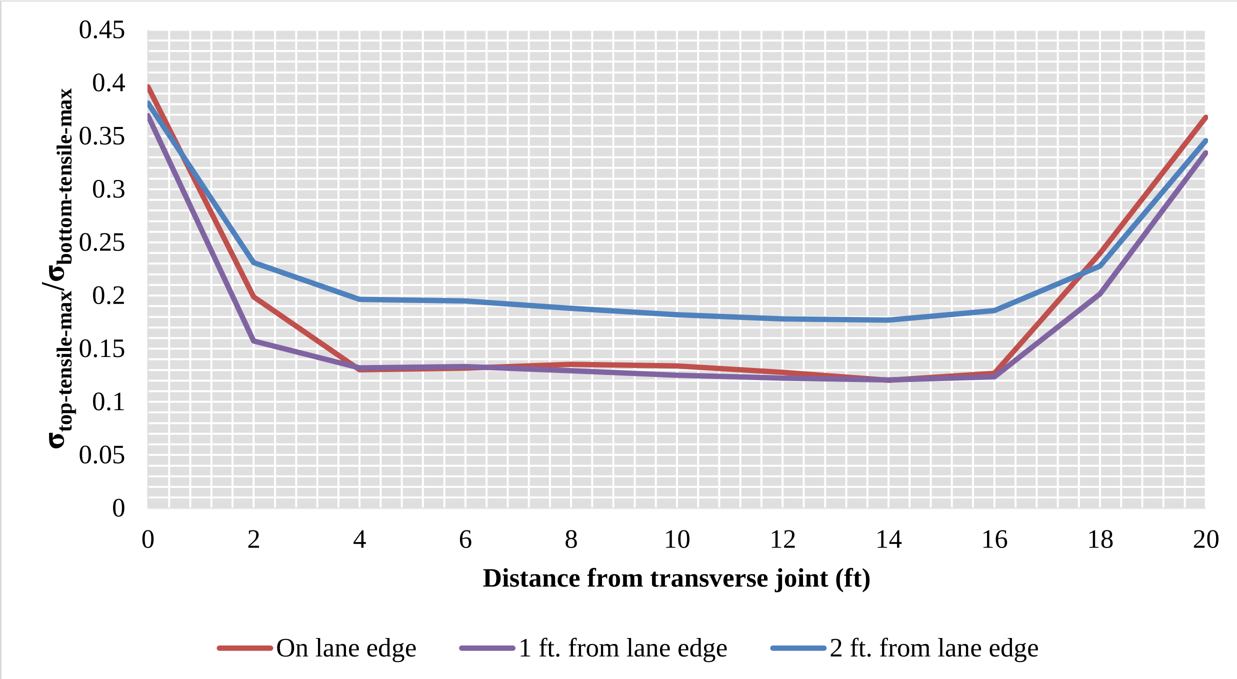


Figure 61. Case 1 study: Top-to-bottom tensile stress ratio – mechanical load only ($\Delta T = 0°F$)

Figure 62 shows the top-to-bottom tensile stress ratio distribution when a combined mechanical and temperature load ($\Delta T = -10°F$) is applied at various locations in both the traffic and wander directions. As seen in Figure 62, very high (as high as 1.8) top-to-bottom tensile stress ratio values were observed when the combined mechanical and temperature load ($\Delta T = -10°F$) was applied around mid-slab. There was no significant difference in top-to-bottom tensile stress ratio results for different wander distances. But when the outer wheel of the single axle was placed on the lane edge, a slightly higher top-to-bottom stress ratio was observed compared to when the outer wheel of the single axle was placed 1 ft and 2 ft away from the lane edge.

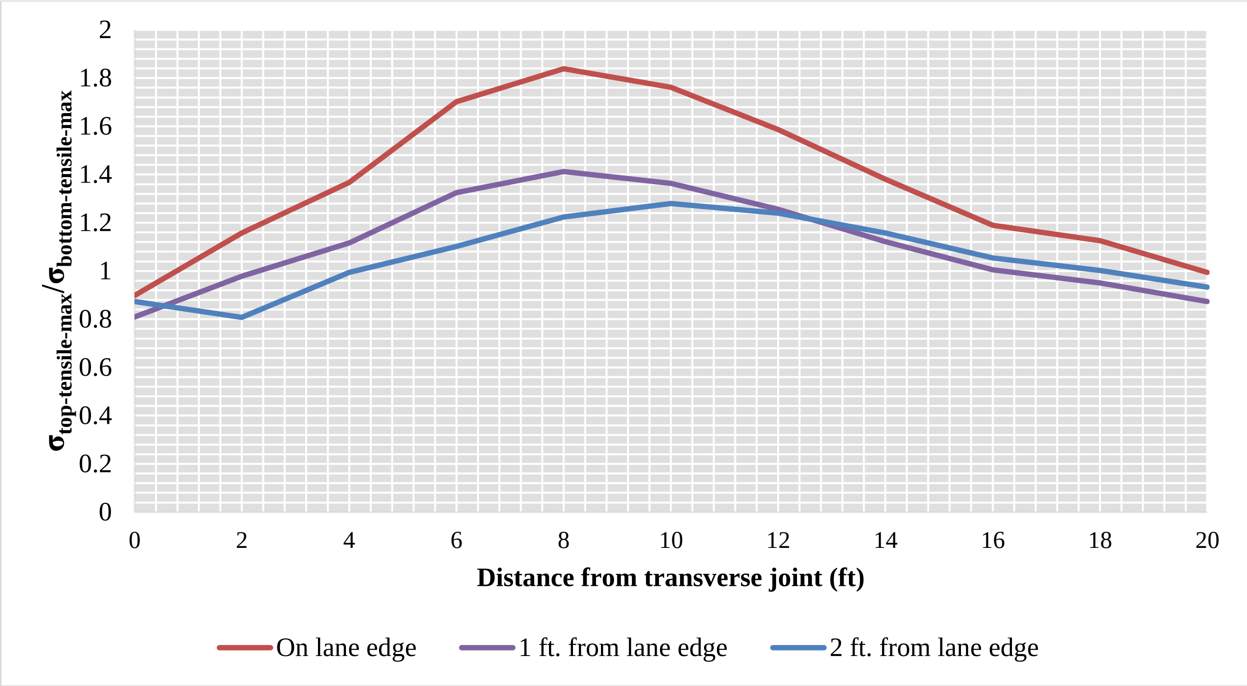


Figure 62. Case 1 study: Top-to-bottom tensile stress ratio – combined mechanical and temperature load cases (ΔT = -10°F)

Figure 63 shows the top-to-bottom tensile stress ratio distribution when a combined mechanical and temperature load (ΔT = -20°F) is applied at various locations in both traffic and wander directions. As seen in Figure 63, very high (as high as 5.8) top-to-bottom tensile stress ratio values were observed when the combined mechanical and temperature load (ΔT = -20°F) was applied around mid-slab. Although there was no significant difference in top-to-bottom tensile stress results for different wander distances, when the outer tire of the single axle was placed on the lane edge, a slightly higher top-to-bottom stress ratio was observed compared to cases 1 ft and 2 ft away from the lane edge.

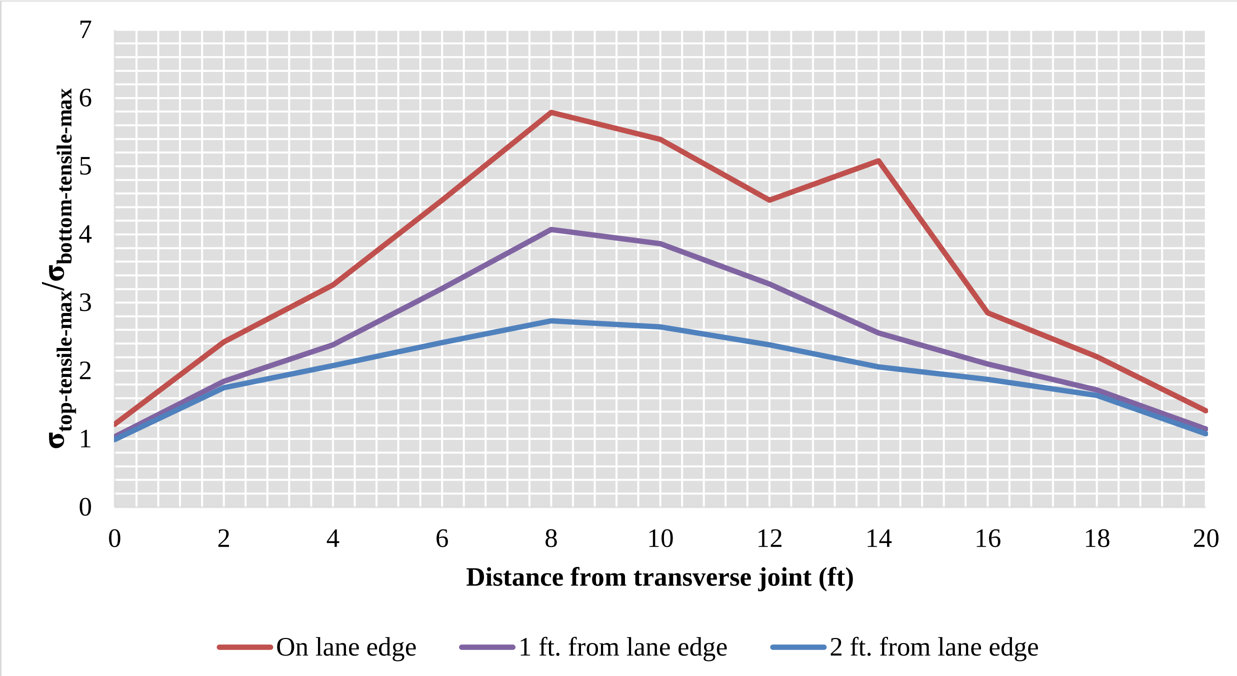


Figure 63. Case 1 study: Top-to-bottom tensile stress ratio – combined mechanical and temperature load cases (ΔT = -20°F)

Figure 64 shows the top-to-bottom tensile stress ratio distribution when various combined mechanical and temperature load scenarios are applied at lane edge and various locations in the traffic direction. As seen in Figure 64, as the negative temperature gradient increases, higher top-to-bottom tensile stress ratio values are observed around mid-slab.

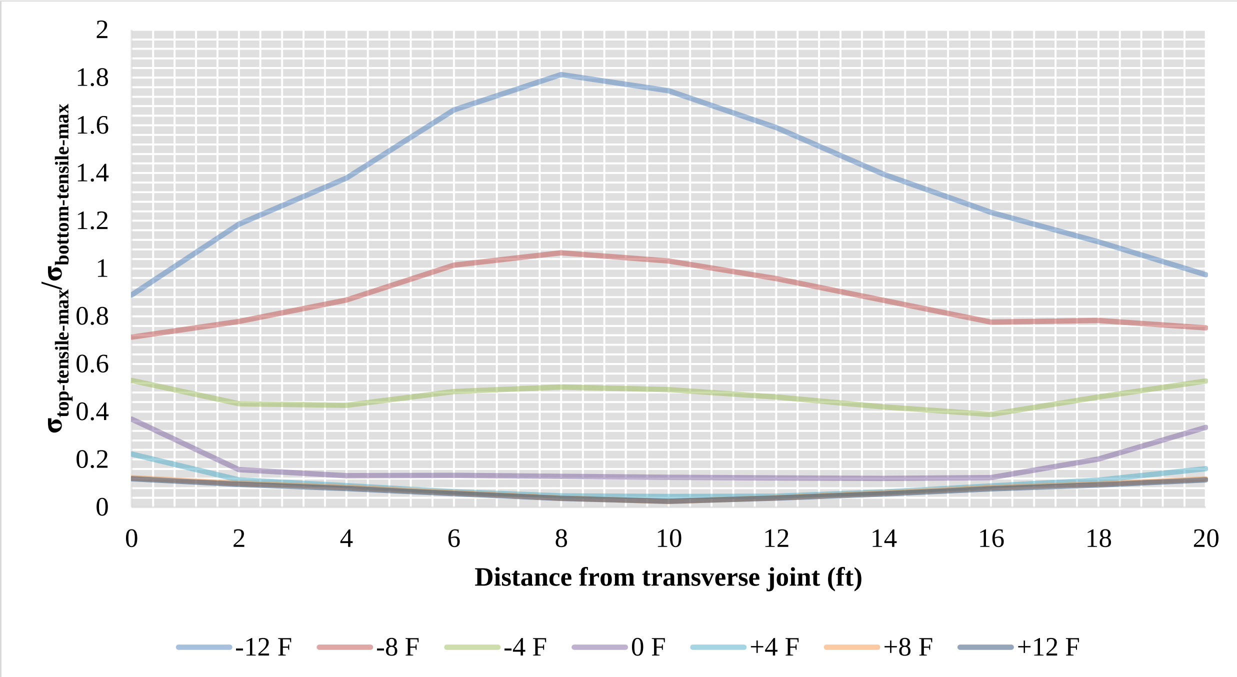


Figure 64. Case 1 study: Top-to-bottom tensile stress ratio – various combined mechanical and temperature load cases

Single Axle Load Simulations - Summary of Findings

As part of the Case 1 study, various FEA cases using single axle loads were examined, and the effects of combined mechanical and temperature loads on tensile stress development on slab surfaces were investigated. Effects of load and wander patterns on tensile stress development on slab surfaces also became better understood. It was determined that the critical tensile stress locations are as follows:

- Close to the transverse joint for mechanical load only
- Close to the mid-slab surface as temperature gradient increases

In combined mechanical and temperature loading cases, as the negative temperature gradient increased, higher top-to-bottom tensile stress ratio values were observed around mid-slab. Further analysis was conducted for applied truckloads (i.e., Case 2 study).

Numerical Modeling Case 2 Study: Truck Load Simulations

Based on the field investigations described in the previous chapters of this report, the failure mechanisms of Iowa JPCP widened slabs with respect to longitudinal cracking include longitudinal cracks starting from transverse joints as top-down cracking, mainly on the widened traffic lane and about 2 to 4 ft away from the slab edge (Figure 65).

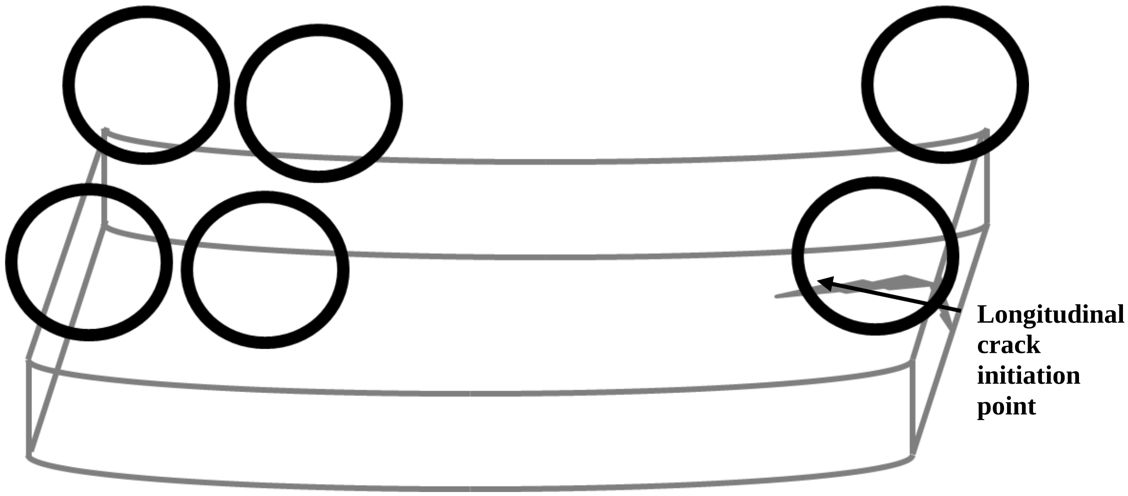


Figure 65. Failure mechanism for longitudinal cracking from field investigation

In this section, several truck axle load and spacing configurations are investigated to evaluate the effects of axle load and spacing configurations on longitudinal cracking, and the critical axle load and spacing configuration with the highest longitudinal cracking potential is also identified.

Mechanical loads for single axle and tandem axles were applied as 20 kips and 34 kips, respectively, based on Federal Highway Administration (FHWA) (FHWA 2015) and Iowa DOT guidelines (Iowa DOT 2017).

Several what-if scenarios that included various axle load and spacing configurations were investigated:

- Three-axle truck with 20 ft axle spacing placed on a single slab
- Three-axle truck with 22 ft axle spacing with the center of the tandem axle placed on a transverse joint
- Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs
- Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs – truck on slab edge
- Four-axle truck with 23 ft axle spacing with both axle groups partially placed on adjacent slabs

Three-Axle Truck with 20 ft Axle Spacing Placed on a Single Slab

In this loading scenario, a truck with a single axle and a tandem axle is used as a truckload (Class 6 based on FHWA [2015] truck classification) (Figure 66). Single axle and tandem axles apply mechanical loads of 20 kips and 34 kips, respectively, on a pavement system (Figure 66). The 20 ft figure was selected as the axle spacing, i.e., the distance between the center of the rear axle of the tandem axle and the single axle, so that both single and tandem axle loads are placed on two transverse joints of the widened slabs (JPCP has a joint spacing of 20 ft) (Figure 67). Five different wander distances were tested (0, 0.5, 1, 1.5, and 2 ft away from lane edge) (Table 35).

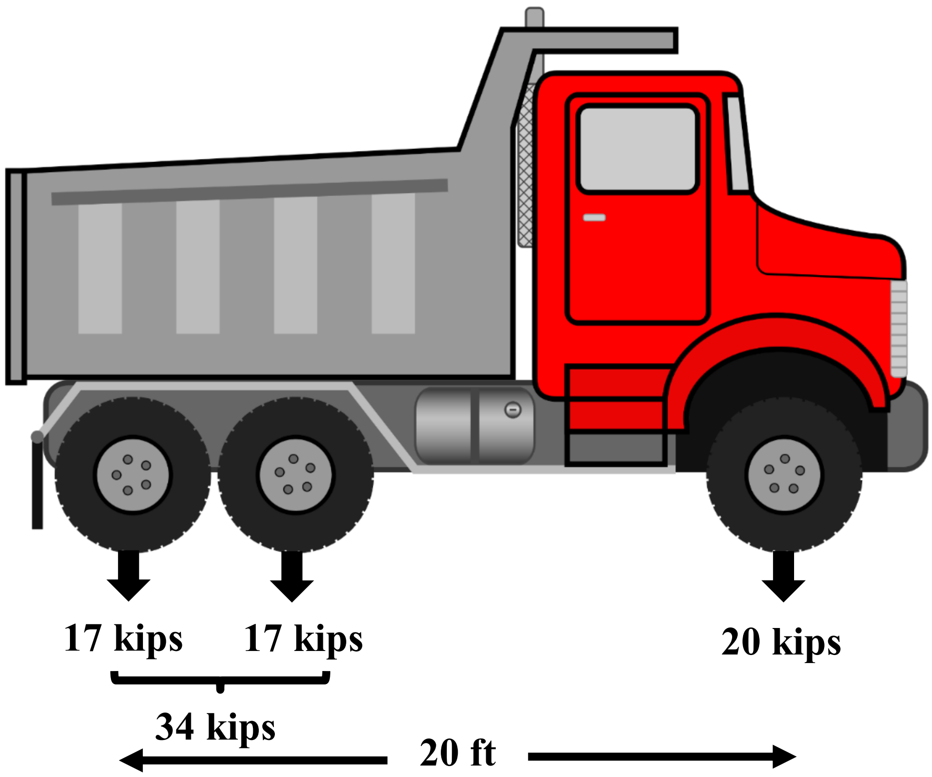


Figure 66. Three-axle truck with 20 ft axle spacing

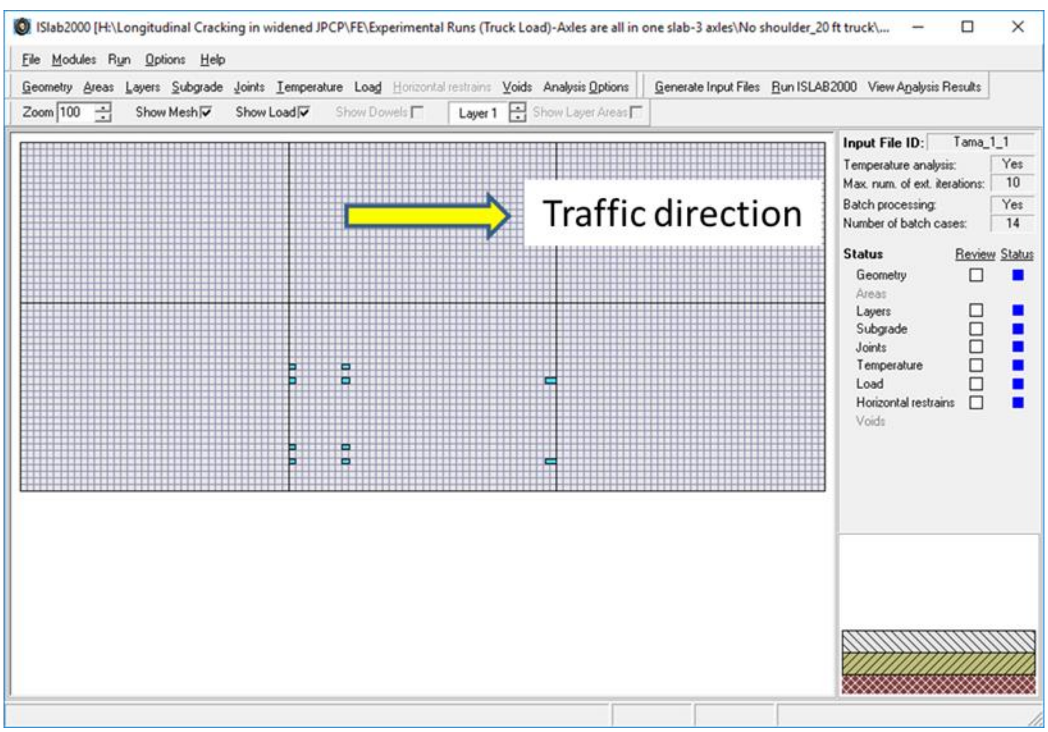
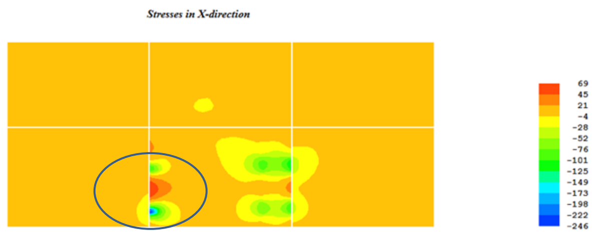
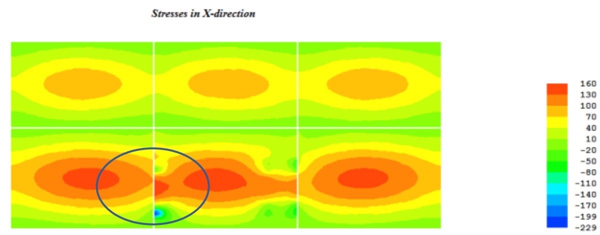


Figure 67. Three-axle truck with 20 ft axle spacing - discretized truck load

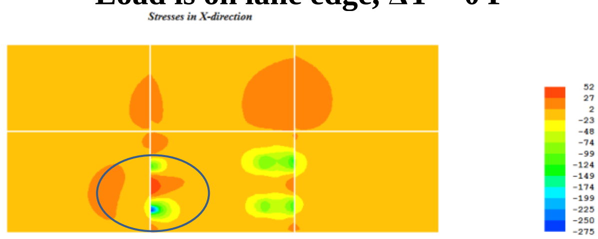
Figure 68 shows the top tensile stress distribution when a truck load is applied at three wander distances (on lane edge and 1 ft and 2 ft away from lane edge) for two temperature load cases (only mechanical load and combined mechanical and temperature load [ΔT = -20°F]). As seen in Figure 68, very high top tensile stresses can be observed starting from the transverse joints, producing greater potential for longitudinal crack initiation starting from the transverse joint of the slab surface.



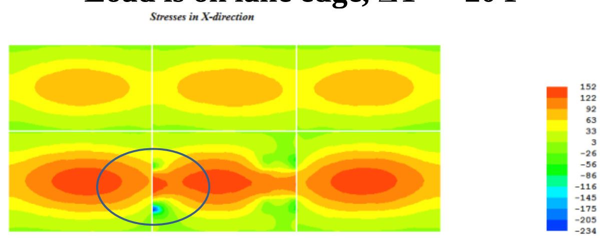
Load is on lane edge, ΔT = 0°F



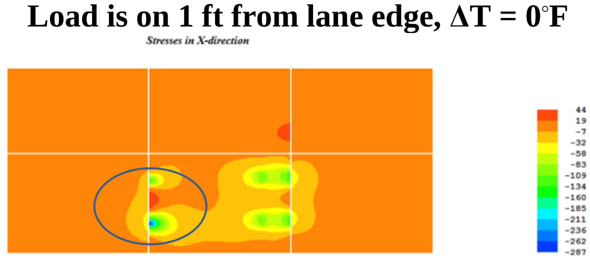
Load is on lane edge, ΔT = -20°F



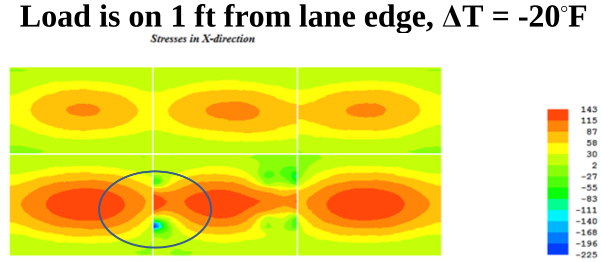
Load is on 1 ft from lane edge, ΔT = 0°F



Load is on 1 ft from lane edge, ΔT = -20°F



Load is on 2 ft from lane edge, ΔT = 0°F



Load is on 2 ft from lane edge, ΔT = -20°F

Figure 68. Three-axle truck with 20 ft axle spacing – top tensile stress distribution for 3 wander distances and 2 temperature load cases

Figure 69 shows the top-to-bottom tensile stress ratio distribution when various combined mechanical and temperature load scenarios are applied at various wander distances (0 to 2 ft). As seen in Figure 69, as the negative temperature gradient increases, the top-to-bottom tensile stress ratios increase. Moreover, as the truckload is placed closer to the lane edge (wander distance decreases), the top-to-bottom tensile stress ratios increase.

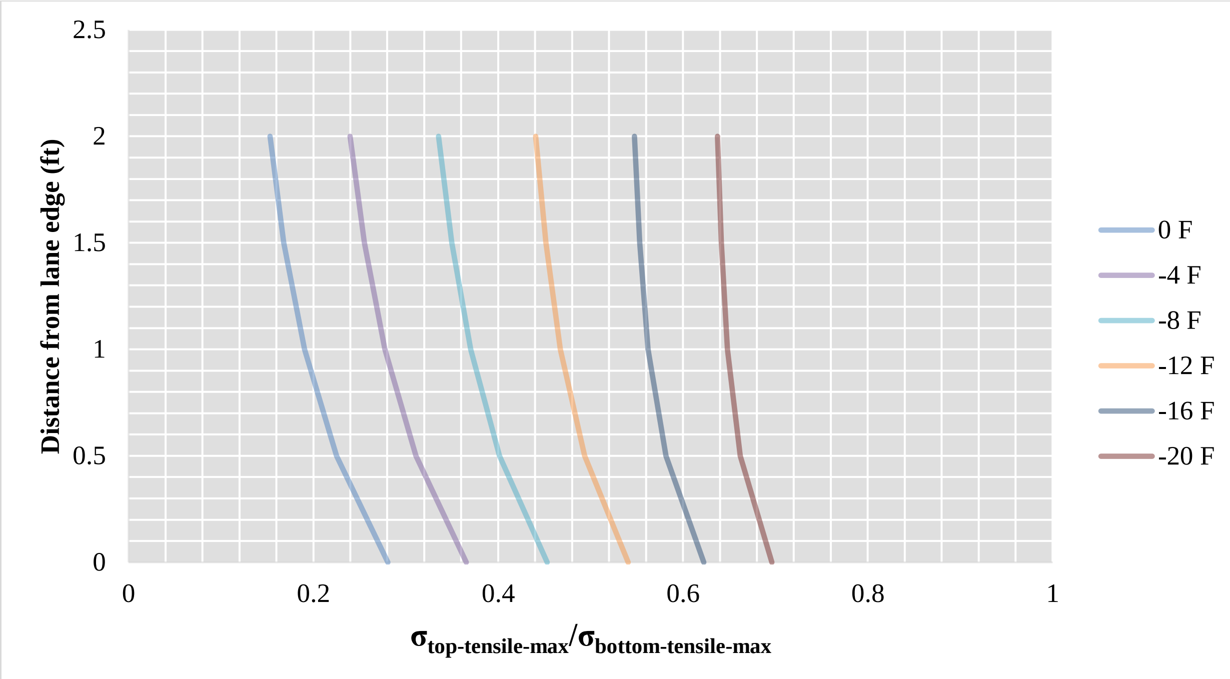


Figure 69. Three-axle truck with 20 ft axle spacing – top-to-bottom tensile stress ratio distribution

Figure 70 shows the top tensile stress distribution when various combined mechanical and temperature load scenarios are applied at various wander distances (0 to 2 ft). The top tensile stress distribution exhibits a similar trend as the top-to-bottom tensile stress ratios; as the negative temperature gradient increases, the top tensile stresses also increase; and as the truckload is placed closer to the lane edge, the top tensile stresses also increase.

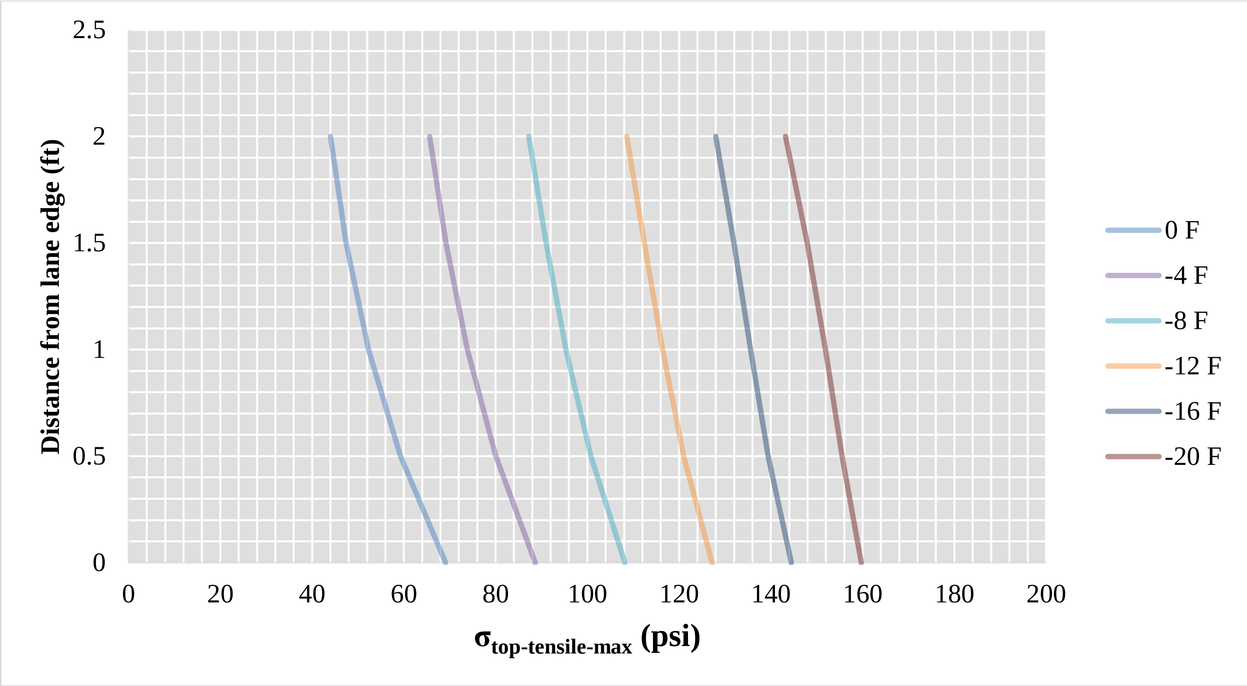


Figure 70. Three-axle truck with 20 ft axle spacing – top tensile stress distribution

Results based on this loading scenario are summarized as follows:

- A higher temperature gradient produced higher top-to-bottom tensile stress ratios.
- Close to the lane edge, higher top-to-bottom tensile stress ratio values were observed (highest on the lane edge).
- For high temperature load cases, the critical tensile stress location was identified as the transverse slab joint.
- However, further analysis was required when some of the axle load was placed on an adjacent slab, causing tensile stresses to be transferred between slabs.

Three-Axle Truck with 22 ft Axle Spacing with the Center of the Tandem Axle Placed on a Transverse Joint

In this loading scenario, the same axle configuration as in the previous case was used. A 22 ft axle spacing was applied so that the center of the tandem axle was placed on a transverse joint (Figure 71) and tensile stress transfer between slabs could be analyzed (Figure 72).

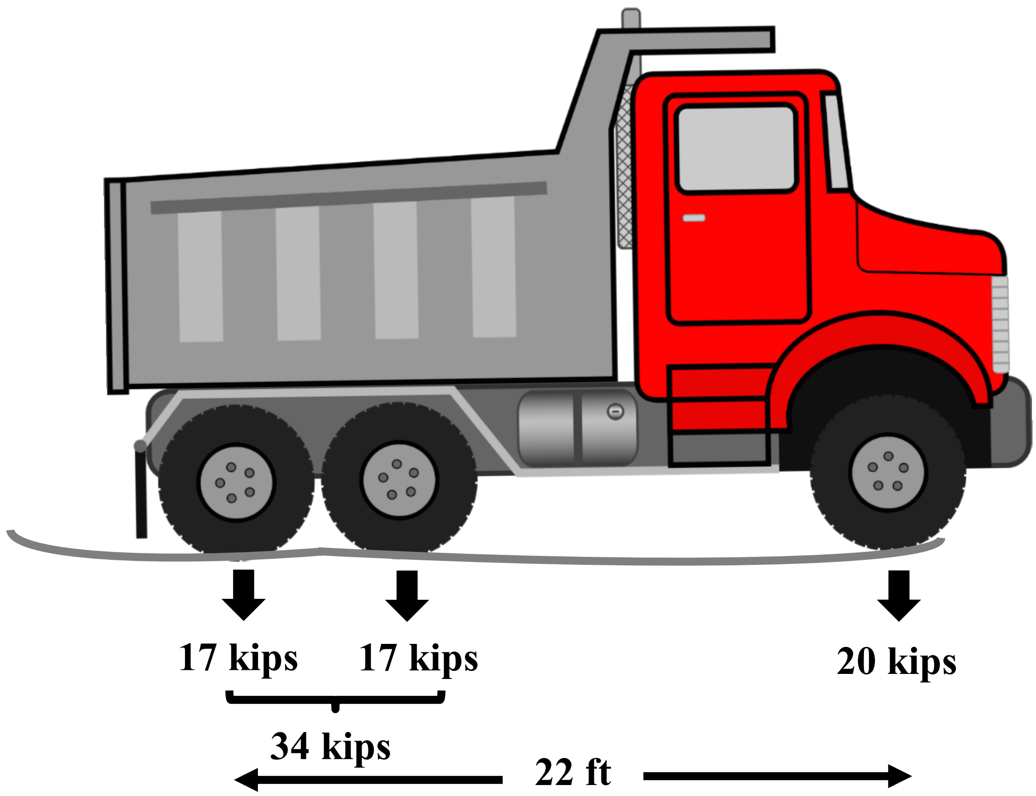


Figure 71. Three-axle truck with 22 ft axle spacing, with the center of the tandem axle placed on a transverse joint

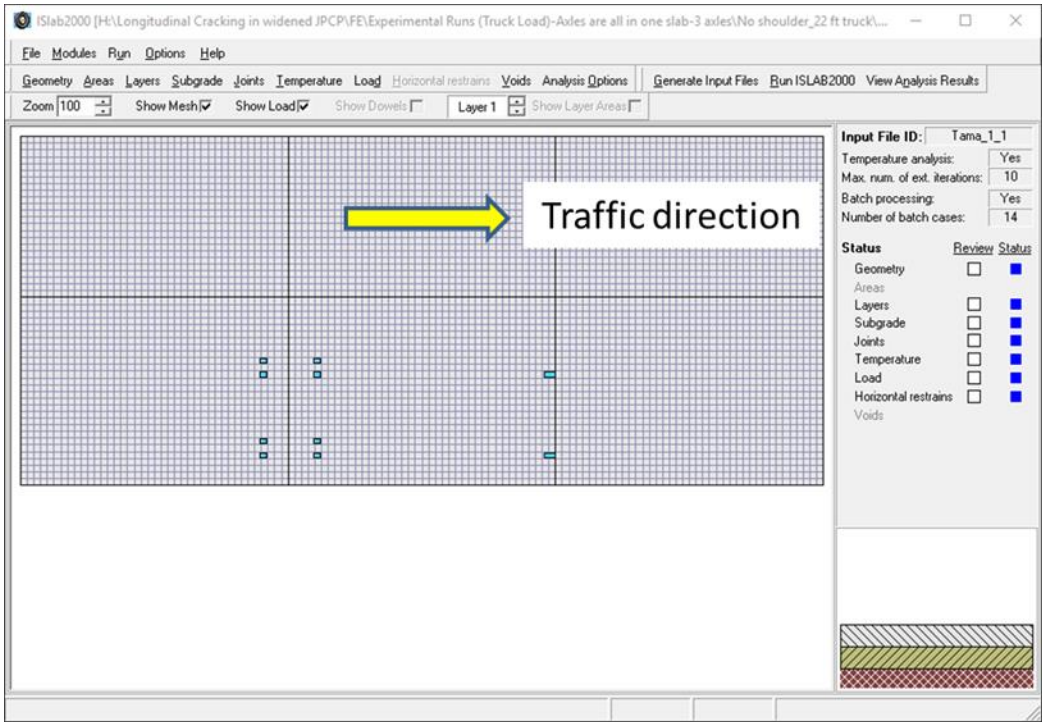
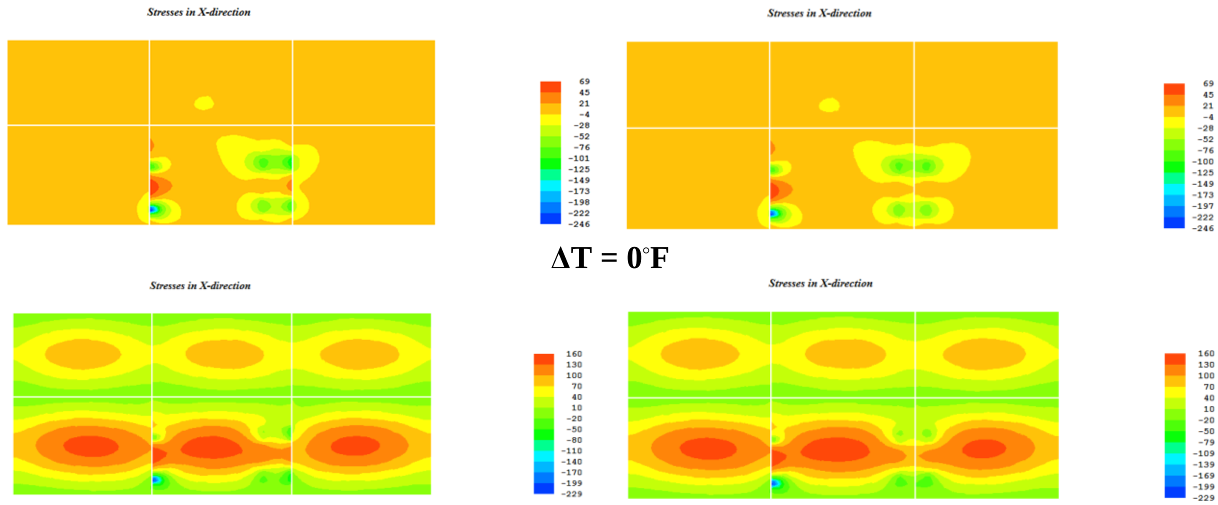


Figure 72. Three-axle truck with 22 ft axle spacing with the center of the tandem axle placed on a transverse joint - discretized truck load

Figure 73 shows comparisons of tensile stress distributions between a three-axle truck with 20 ft axle spacing and a three-axle truck with 22 ft axle spacing for two loading scenarios: mechanical load only and combined mechanical and temperature load (ΔT = -20°F). As seen in Figure 73, the tensile stress transfer between adjacent slabs was well-modeled, and no significant difference in overall tensile stress results is observed between the two axle spacing cases.



ΔT = -20°F

20 ft axle spacing **22 ft axle spacing**

Figure 73. Comparisons of tensile stress distributions between three-axle truck with 20 ft axle spacing and three-axle truck with 22 ft axle spacing

Three-Axle Truck with 23 ft Axle Spacing with Both Axles Groups Placed on Adjacent Slabs

In this loading scenario, both the single axle and one axle of the tandem axle group are placed on adjacent slabs, with the single axle and tandem axles applying 20 kips and 34 kips mechanical load, respectively, on the pavement system. The axle spacing chosen was 23 ft so that tensile stress transfer between slabs could be further analyzed (Figure 74).

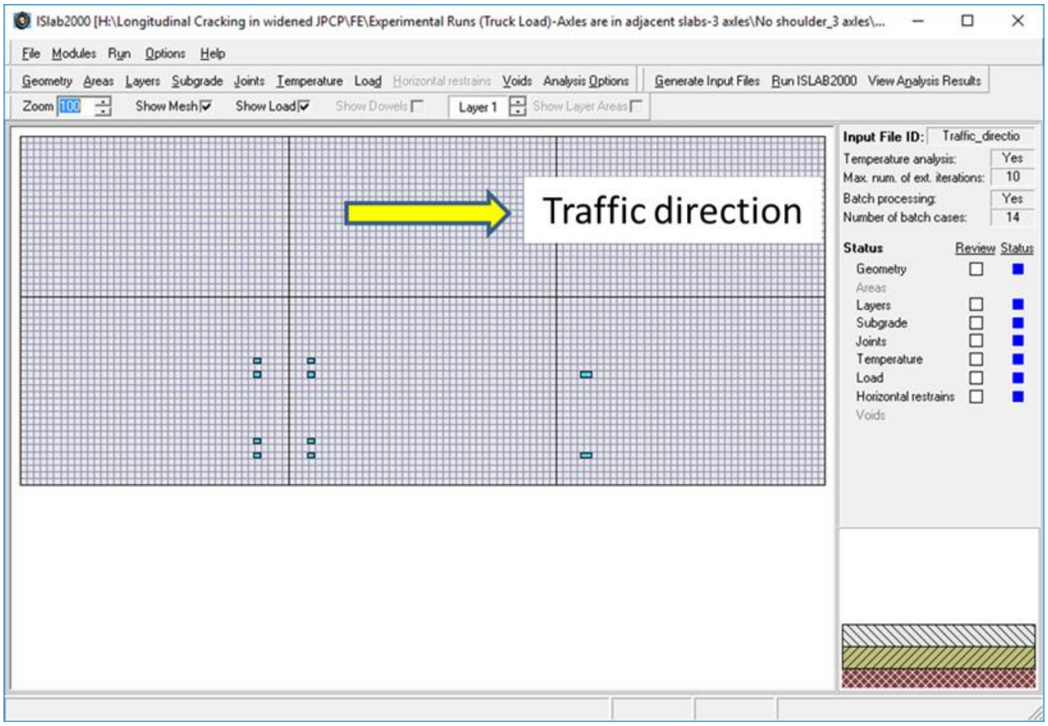
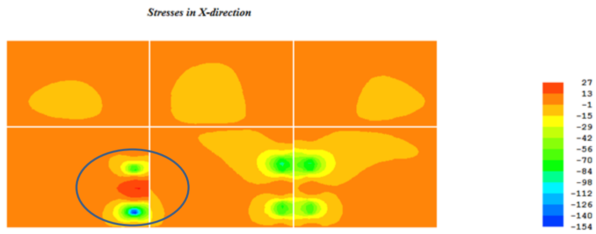
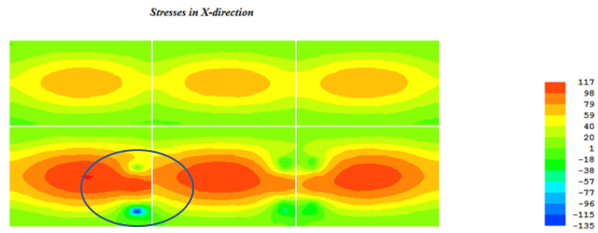


Figure 74. Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs - discretized truck load

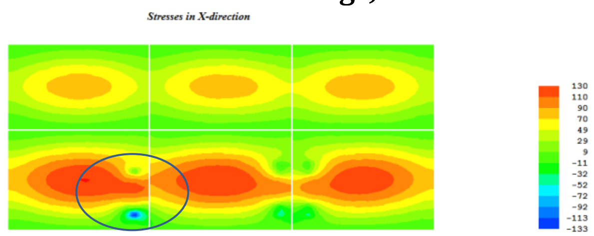
Figure 75 shows the top tensile stress distribution when the truckload is applied on the lane edge for four temperature load cases: mechanical load only and combined mechanical and temperature load cases (ΔT = -16 °F, -18°F, and -20°F). As seen in Figure 75, very high top tensile stresses starting from the transverse joints are observed, creating greater potential for longitudinal crack initiation starting from the transverse joint of the slab surface as a higher negative temperature gradient is applied.



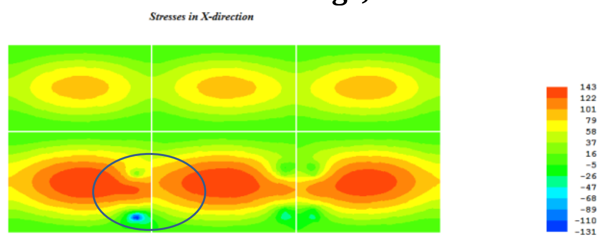
Load is on lane edge, ΔT = 0°F



Load is on lane edge, ΔT = -16°F



Load is on lane edge, ΔT = -18°F



Load is on lane edge, ΔT = -20°F

Figure 75. Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs – top tensile stress distribution for four temperature load cases

Figure 76 shows the top-to-bottom tensile stress ratio distributions when various combined mechanical and temperature load scenarios are applied at various wander distances (0 to 2 ft). As seen in Figure 76, as the temperature difference between the top and bottom of the slab increases, the top-to-bottom tensile stress ratios also increase, exceeding a value of 1 when the temperature difference is around 18°F (ΔT = -18°F).

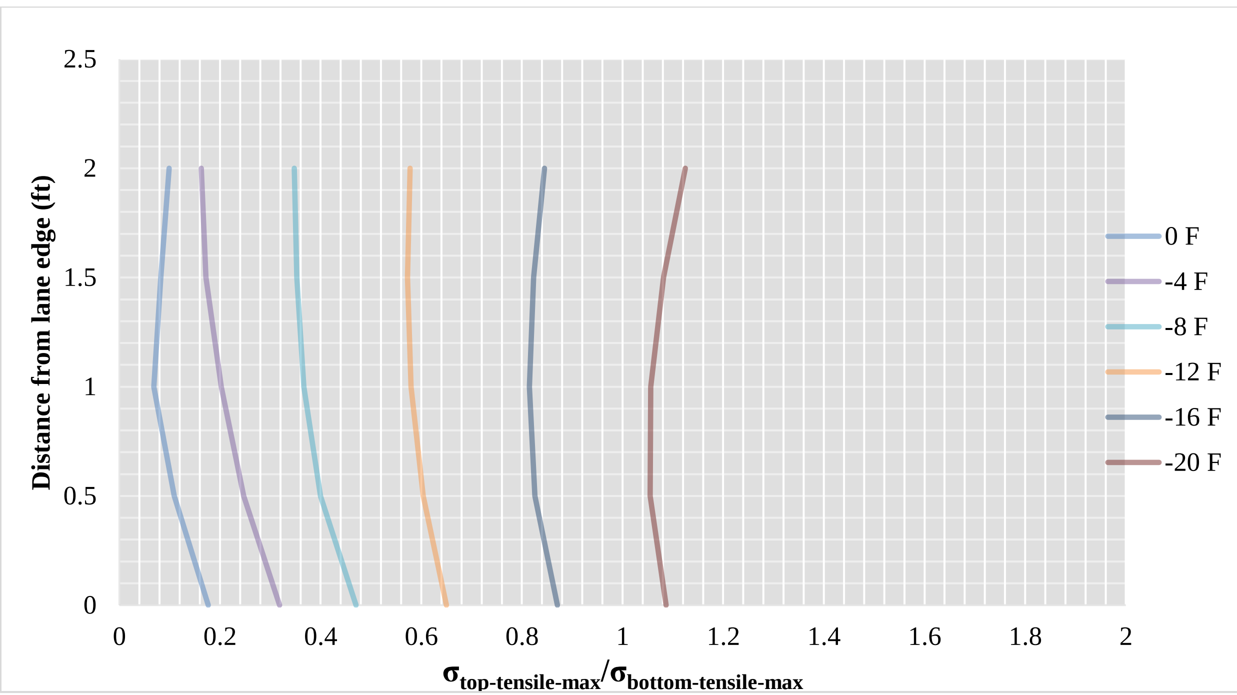


Figure 76. Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs – top-to-bottom tensile stress ratio distribution

Figure 77 shows the top tensile stress distribution when various combined mechanical and temperature load scenarios are applied on a lane edge. The top tensile stress distribution shows a similar trend for the top-to-bottom tensile stress ratios: as the negative temperature gradient increases, top tensile stresses also increase.

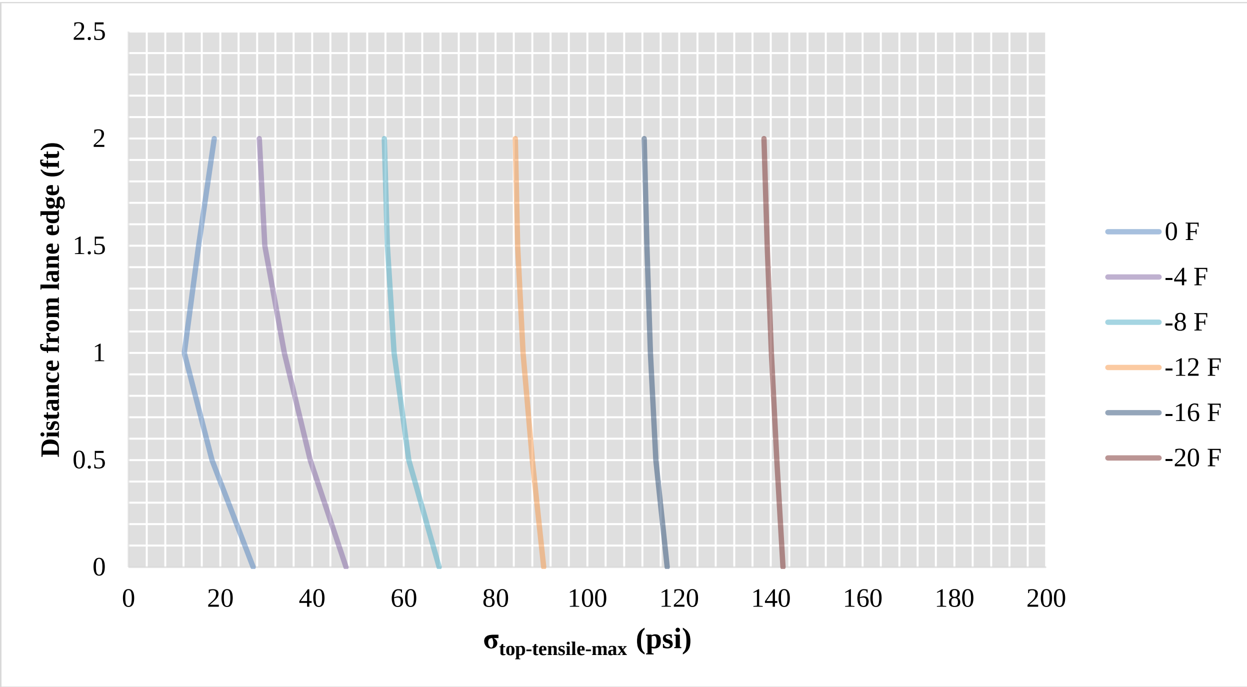


Figure 77. Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs – top tensile stress distribution

Results based on this loading scenario can be summarized as follows:

- Very high top-to-bottom tensile stress ratio values were observed.
- Tensile stress accumulation close to a transverse joint was identified.
- A four-axle truck configuration should also be investigated.

Three-Axle Truck with 23 ft Axle Spacing with Both Axle Groups Placed on Adjacent Slabs – Truck on Slab Edge

In this loading scenario, the same axle configuration as in the previous case was used for a 23 ft axle spacing with both the single axle and one axle of the tandem axle group placed on adjacent slabs (see Figure 78). The objective of this analysis was to understand the tensile stress distribution in a scenario where a truck is driving or standing on a widened slab edge (see Figure 78).

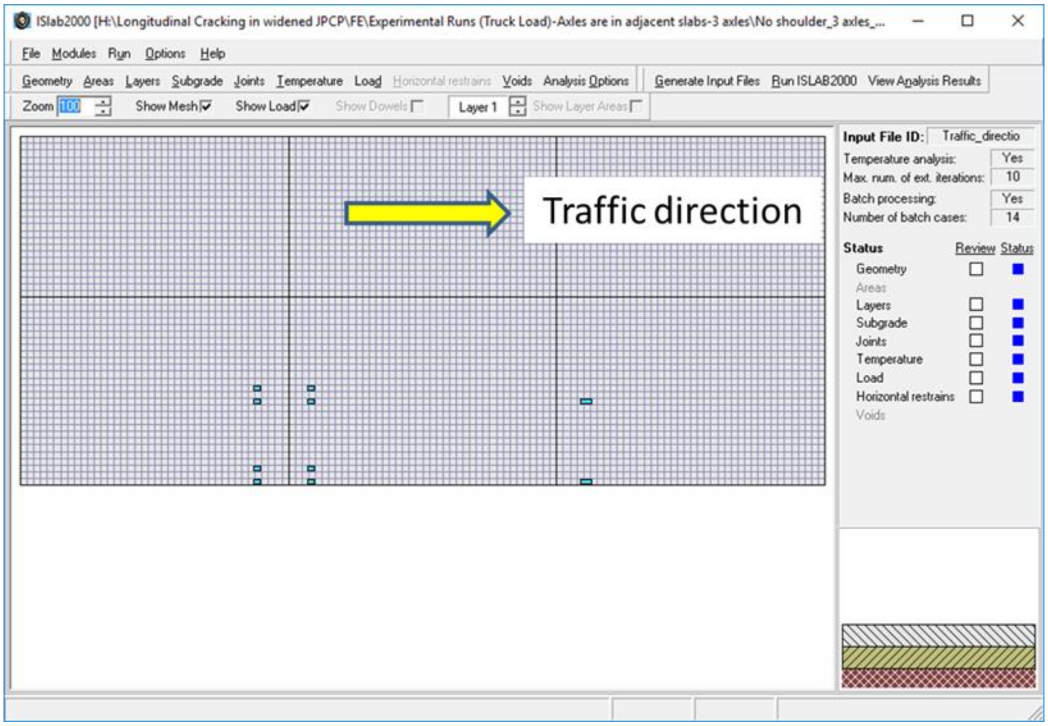


Figure 78. Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs (truck on slab edge) – discretized truck load

Figure 79 shows the top tensile stress distribution when a truckload is applied on a widened slab edge for four temperature/load cases: only mechanical load and combined mechanical and temperature load cases (ΔT = -16°F, -18°F, and -20°F). As seen in Figure 79, very high top-to-bottom tensile stress ratios can be observed between the lane edge and the wheel path; top-to-bottom tensile stress ratios are as high as 4.

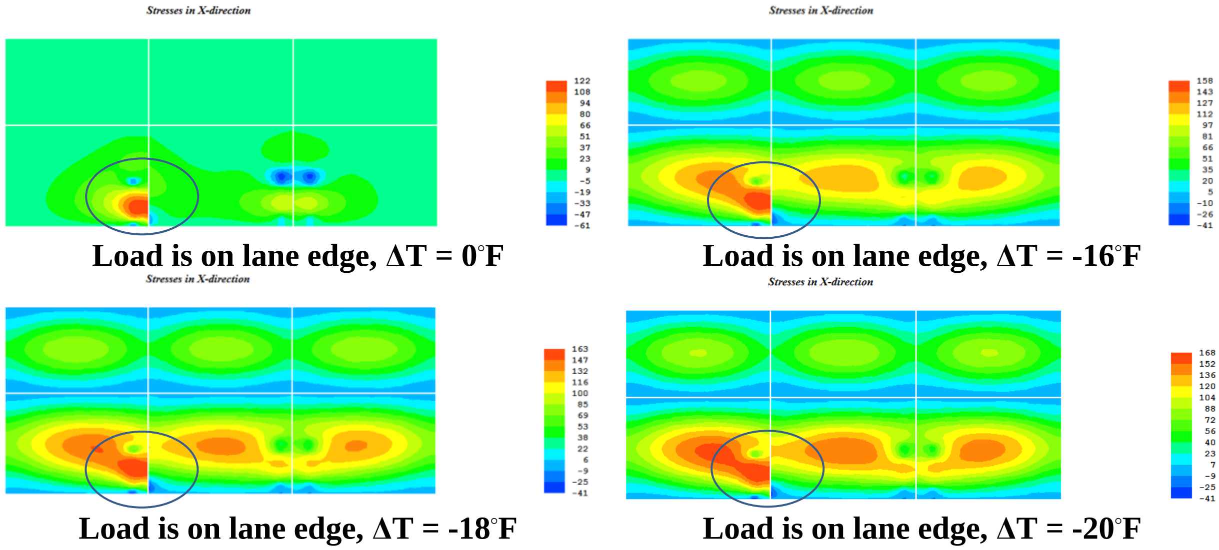


Figure 79. Three-axle truck with 23 ft axle spacing with both axle groups placed on adjacent slabs (truck on slab edge) – top tensile stress distribution for four temperature load cases

Four-Axle Truck with 23 ft Axle Spacing with Both Axle Groups Partially Placed on Adjacent Slabs

It was concluded from the three-axle truck cases that when axle loads are placed on adjacent slabs, tensile stresses are transferred to a critical slab (the slab between adjacent slabs), causing very high tensile stress to accumulate around the top surface of critical slab surface close to the transverse edge. This is especially true for high negative temperature gradient cases (when slabs curl up) where the center of the axle loads is placed close to the transverse edges of an adjacent slab on that slab's side (Figure 80). In that case, the top tensile stresses on the transverse edges of the adjacent slabs are transferred to the critical slabs and very high top tensile stresses are observed around the transverse joints of the critical slabs (Figure 80).

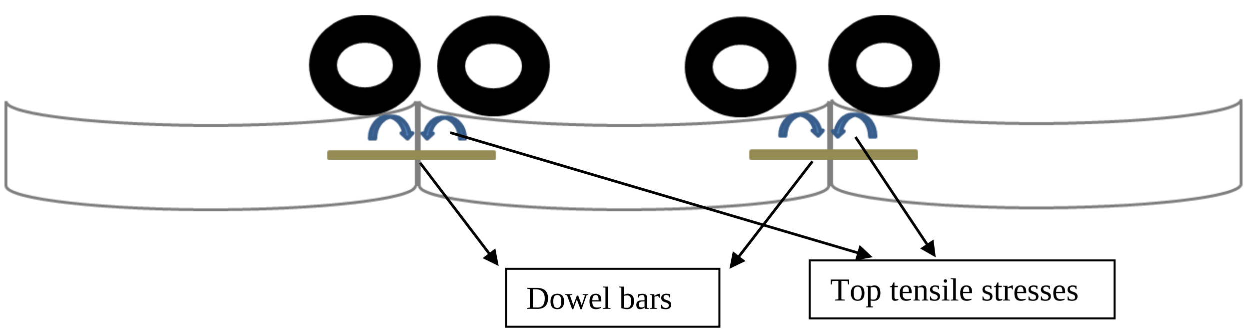


Figure 80. Top tensile stress transfer mechanism in four-axle truck

In this loading scenario, a two-tandem axle (four-axle) configuration with a 23 ft axle spacing is used, and the centers of the axle loads are placed close to the transverse edges of an adjacent slab on the adjacent slab's side. Each tandem axle applies a total mechanical load of 34 kips (Figure

81). Use of two tandem axles as the mechanical load simulates the two axles of a Class 9 18-wheeler truck (FHWA 2015), the most commonly used truck type (ARA 2004). The objective of this analysis is to determine the critical loading scenario producing the highest top-to-bottom tensile stress ratios.

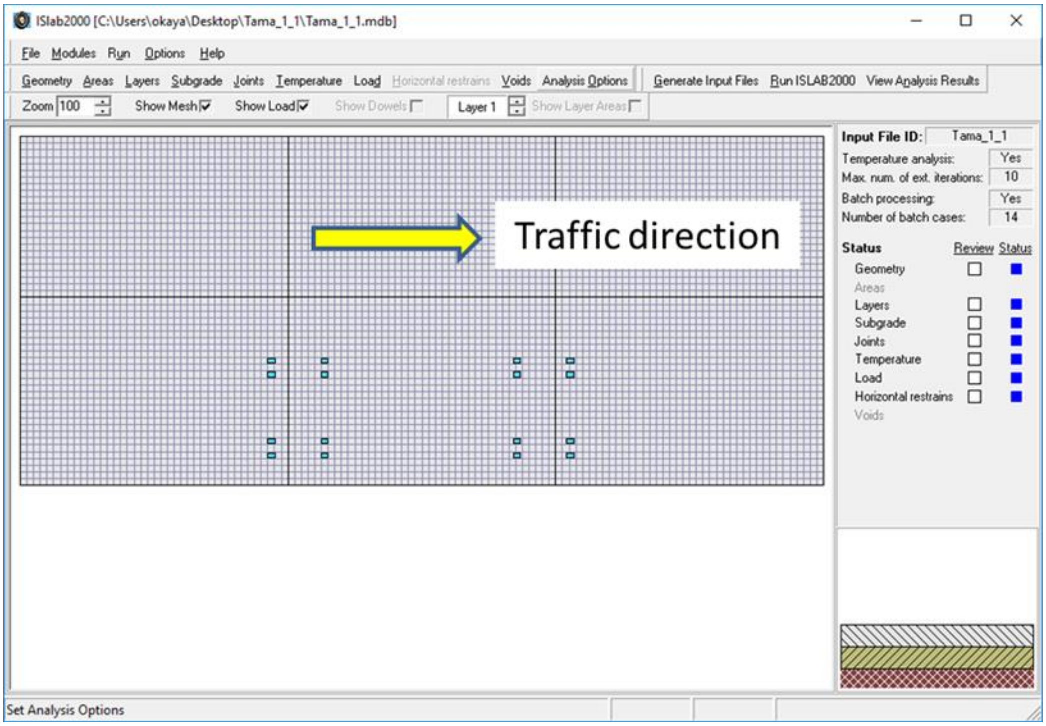
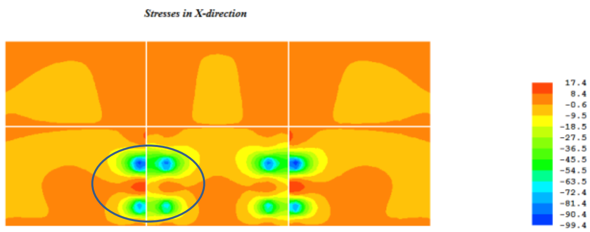
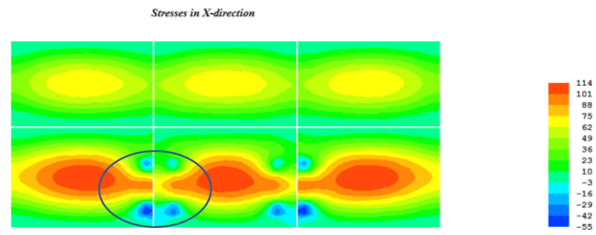


Figure 81. Four-axle truck - discretized truck load

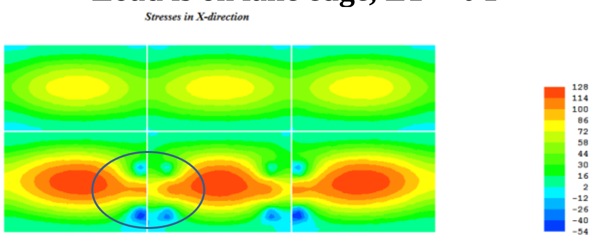
Figure 82 shows the top tensile stress distribution when the truckload is applied on the lane edge for four temperature load cases, including mechanical load only and combined mechanical and temperature load (ΔT = -16°F, -18°F, and -20°F). As can be seen in Figure 82, very high top-to-bottom tensile stress ratios, as high as 3.2, are observed close to the transverse edge.



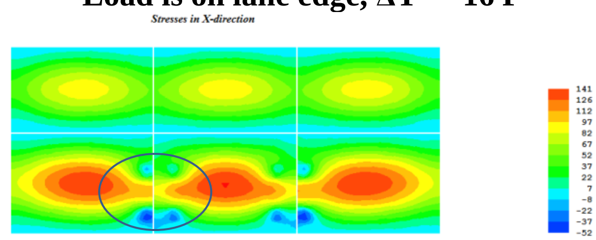
Load is on lane edge, ΔT = 0°F



Load is on lane edge, ΔT = -16°F



Load is on lane edge, ΔT = -18°F



Load is on lane edge, ΔT = -20°F

Figure 82. Four-axle truck – top tensile stress distribution for four temperature load cases

Figure 83 shows the top-to-bottom tensile stress ratio distributions when various combined mechanical and temperature load scenarios are applied at various wander distances (0 to 2 ft). As seen in Figure 83, as the temperature difference between the top and bottom of the slab increases, the top–to-bottom tensile stress ratios also increase to as much as 3.2.

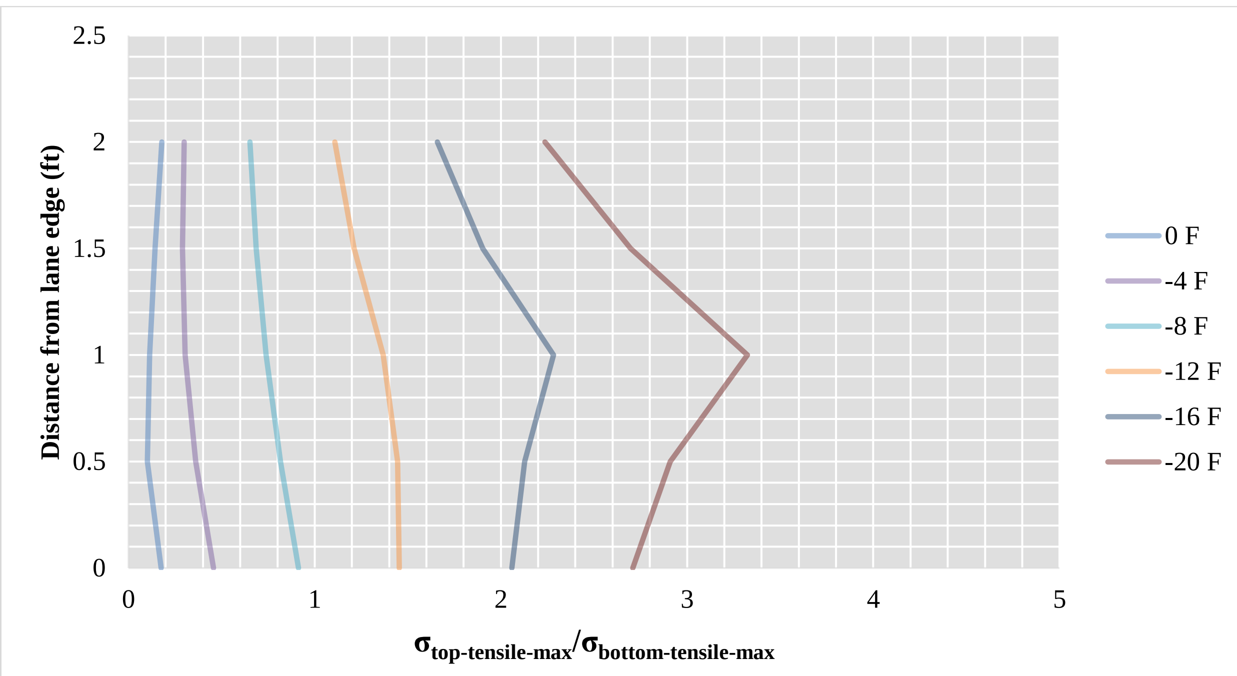


Figure 83. Four-axle truck – top-to-bottom tensile stress ratio distributions

Figure 84 shows top tensile stress distributions when various combined mechanical and temperature load scenarios are applied at the lane edge. The top tensile stress distribution shows a similar trend as the top-to-bottom tensile stress ratios: as the negative temperature gradient increases, the top tensile stresses also increase.

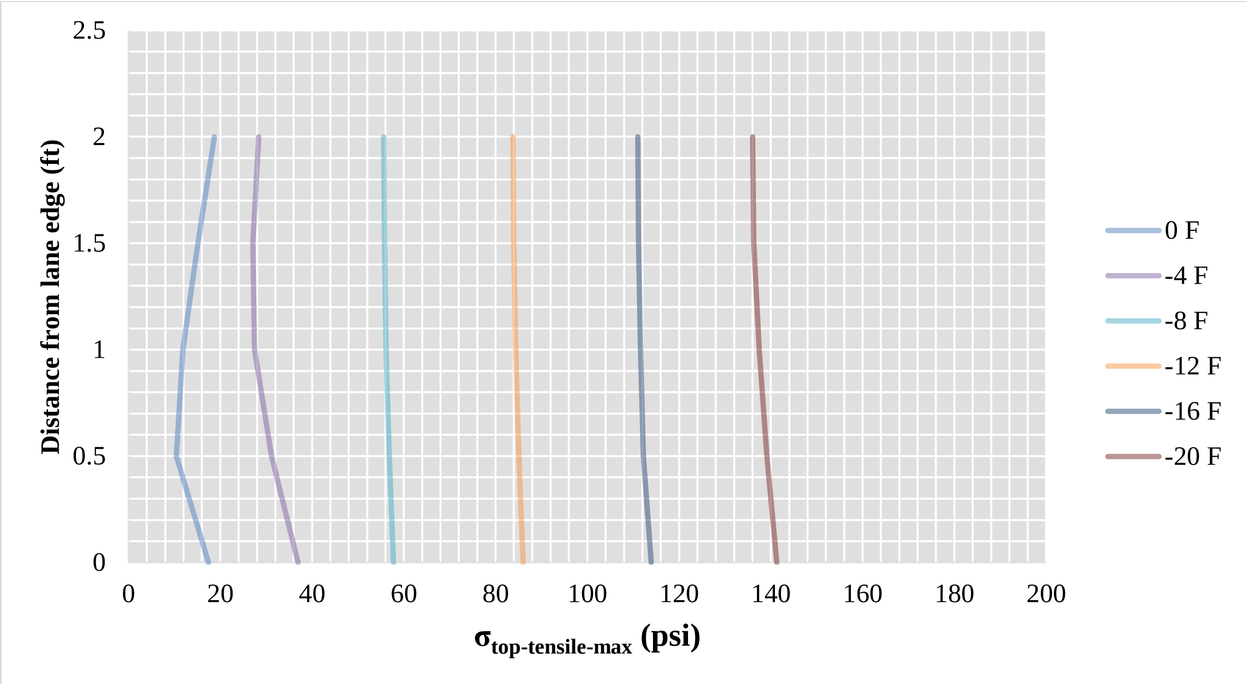
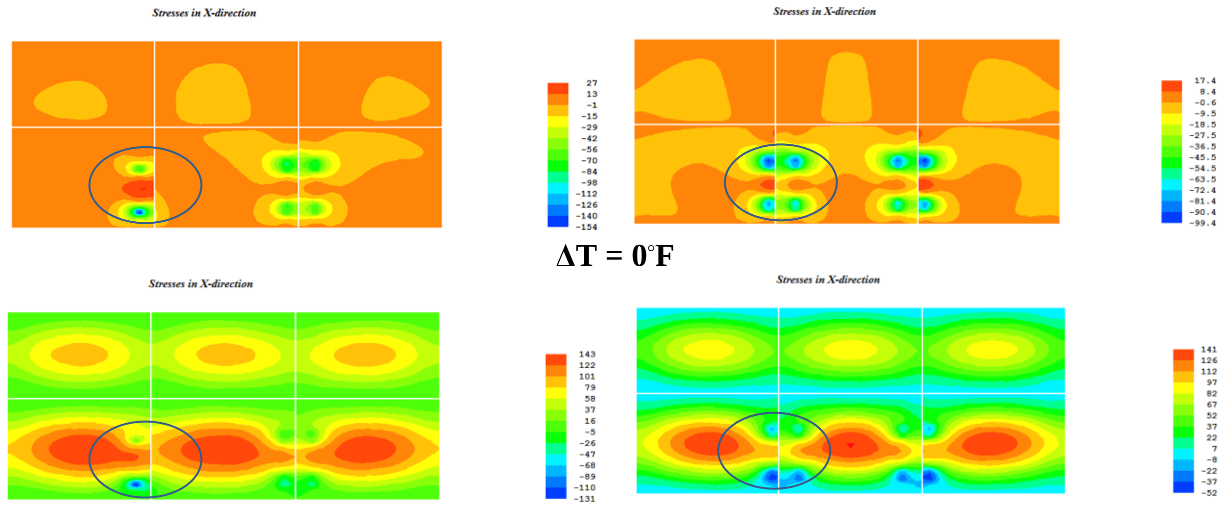


Figure 84. Four-axle truck – top tensile stress distribution

Figure 85 shows comparisons of tensile stress distributions between a three-axle truck with 23 ft axle spacing and a four-axle truck for two loading scenarios: mechanical load only and combined mechanical and temperature load (ΔT = -20°F). As seen in Figure 85, similar top tensile stress results were observed in both cases, except the truck with a four-axle transfer case produced a significantly higher (as high as 2.7) top-to-bottom tensile stress ratio.



ΔT = -20°F

Three-axle truck **Four-axle truck**

Figure 85. Comparisons of tensile stress distributions between three-axle truck with 23 ft axle spacing and four-axle truck

Results based on these loading scenarios can be summarized as follows:

- Although both three- and four-axle configurations produced similarly high tensile stresses, a truck with a four-axle transfer case produced significantly higher (as high as 2.7) top-to-bottom tensile stress ratios.
- A truck with a four-axle configuration where the center of the axle loads is placed close to the transverse edges of the adjacent slabs on the adjacent slab's side was identified as the critical loading scenario.
- High tensile stress accumulation on a slab surface close to the transverse joints offers a good explanation for top-down longitudinal cracking initiation at the transverse joints of the top slab surface.

Truck Load Simulations - Summary of Findings

The top-down longitudinal cracking potential for JPCP with widened slabs was satisfactorily demonstrated using several truckload configurations. The key findings were as follows:

- Longitudinal cracking initiates from the transverse joints between the lane edge and wheel path.
- Longitudinal cracking potential increases with a higher negative temperature gradient.
- A truck with a four-axle configuration with the center of its axle loads placed close to the transverse edges of the adjacent slabs on the adjacent slab's side was identified as the critical loading scenario.

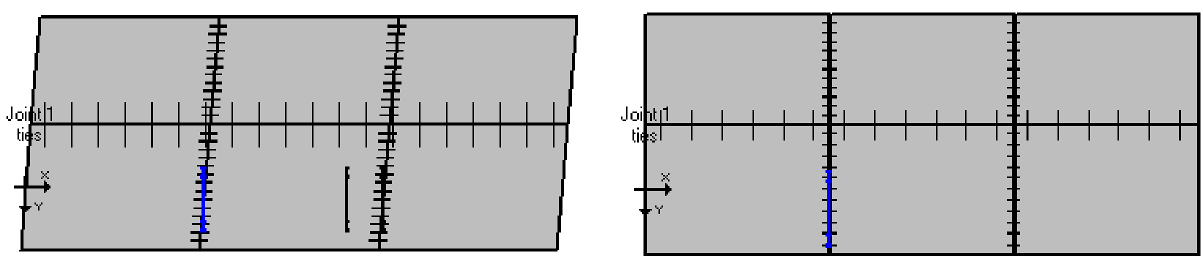
- A higher negative temperature gradient between the top and bottom of the slab produced higher top-to-bottom tensile stress ratios and, in turn, led to greater longitudinal cracking potential.

Numerical Modeling Case 3 Study: Skewed Joint Simulations

As mentioned in the literature review chapter of this report, some studies have shown that pavements with skewed joints have higher potential for longitudinal cracking (Owusu-Ababio and Schmitt 2013, Johnston 2014).

In this section, the longitudinal cracking potential of skewed jointed Iowa JPCP with widened slabs will be evaluated through numerical analysis. Comparisons will be made between rectangular and skewed jointed JPCP with widened slabs in terms of their effect on longitudinal cracking potential.

As stated earlier in this chapter, EverFE 2.25 software was used to model skewed and rectangular jointed JPCP for both widened and regular slab sizes (Figure 86). Widths of widened slabs and regular slabs were taken as 14 ft and 12 ft, respectively, and slabs had a joint spacing of 20 ft. A three-axle truckload with a single wheel and a tandem wheel was used, with the single wheel and tandem wheel applying 20 kips and 34 kips of mechanical load, respectively. Axles were located on the transverse edges of the widened slabs with half of the single and the back axles of the tandem axle on the critical slab and the other half on the adjacent slabs (Figure 86).



Skewed joint Rectangular joint

Figure 86. Skewed and rectangular jointed JPCP with widened and regular slab sizes

Figure 87 shows comparisons of top tensile-stress distributions between skewed and rectangular jointed widened slab cases for three loading scenarios: mechanical load only ($\Delta T = 0°F$) and combined mechanical and temperature loads ($\Delta T = -5$ and $-10°F$). Note that the red color in the contour represents tensile stresses, while the blue color represents compressive stresses. As seen in Figure 87, in all cases investigated, skewed jointed widened slabs produced higher top tensile stresses compared to rectangular jointed widened slabs.

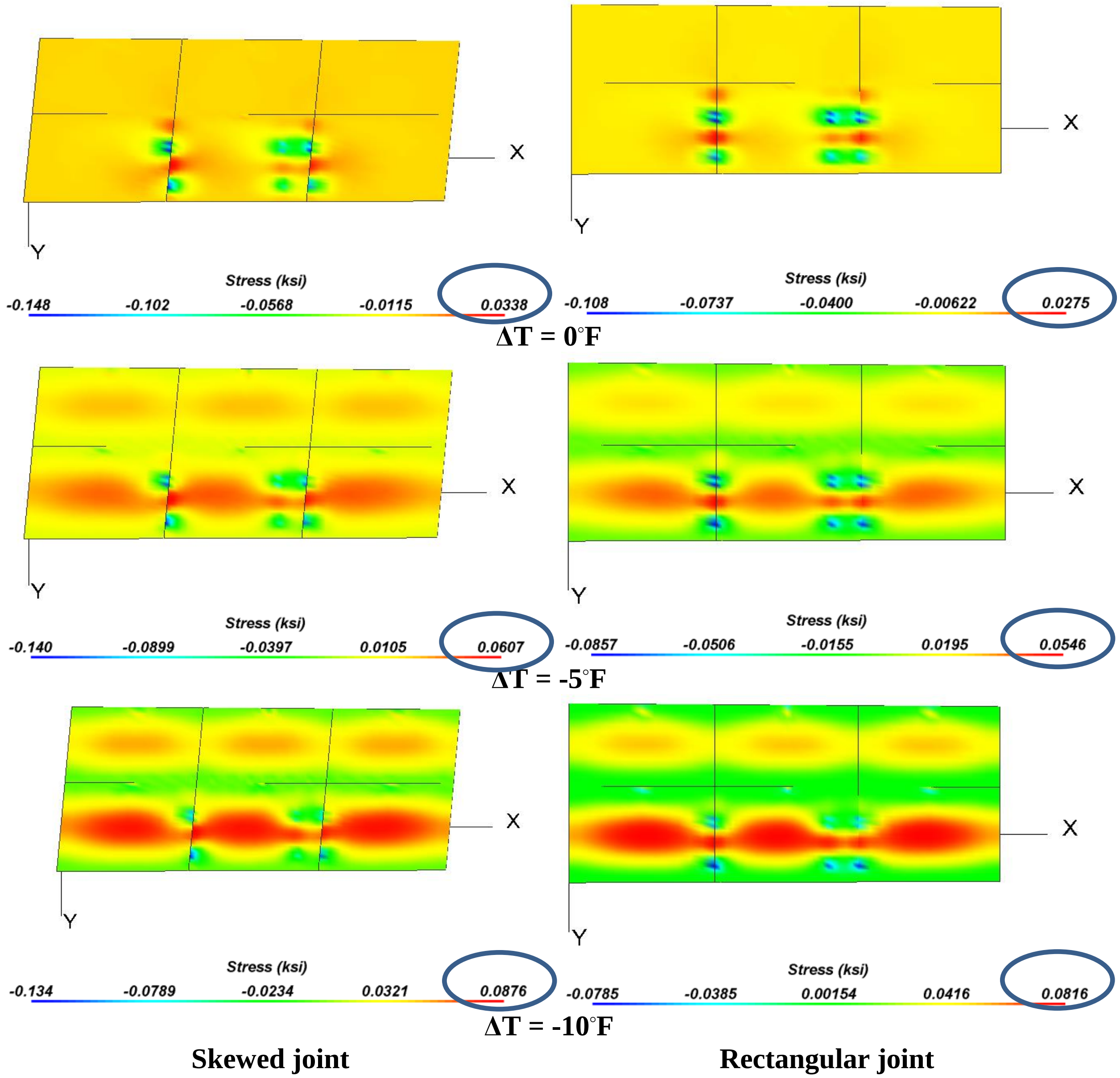


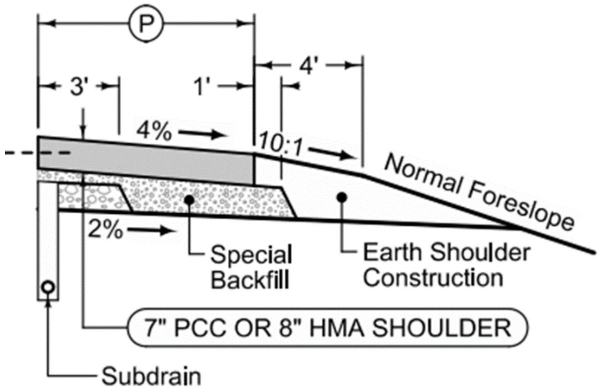
Figure 87. Comparisons of top tensile stress distributions between skewed and rectangular jointed widened slabs for three temperature load scenarios

The results of this case study are summarized as follows:

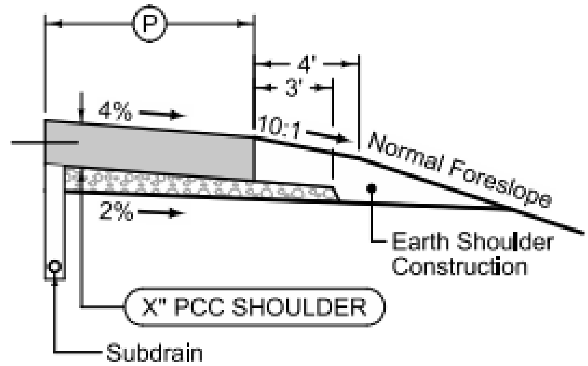
- A skewed joint causes higher top tensile stresses on widened slab transverse joints, so it increases top-down longitudinal cracking potential compared to a rectangular joint.
- Skewed joints are not recommended for Iowa widened JPCP.

Numerical Modeling Case 4 Study: Shoulder Design Alternatives

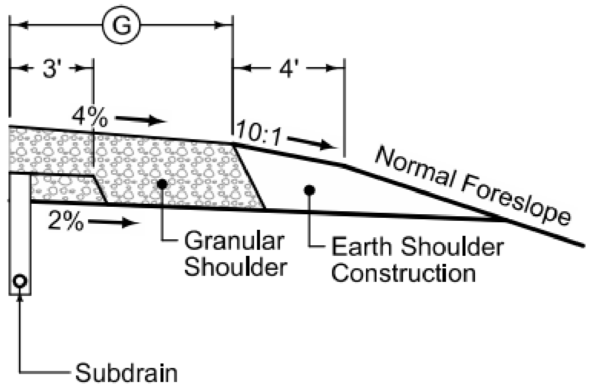
In this case study, four shoulder design alternatives were compared for both widened (14 ft wide) and regular size (12 ft wide) slabs: PCC, HMA (paved shoulder alternates), full-depth PCC shoulder, and granular shoulder (Figure 88). These shoulder types were modeled based on the Iowa DOT’s typical design details (Iowa DOT 2018) (Figure 88).



PCC or HMA Shoulder (Paved Shoulder Alternates)



Full-Depth PCC Shoulder



Granular Shoulder

Figure 88. Shoulder design alternatives

Shoulder design alternatives were compared for the following cases:

Tied PCC shoulder using

- Regular slabs (12 ft) with full-depth tied PCC shoulder alternative; shoulder thickness is the same as regular slab thickness (i.e., 10 in.)
- Widened slabs (14 ft) with tied PCC shoulder alternative; shoulder thickness is less than regular slab thickness (i.e., 7 in.)

HMA shoulder using

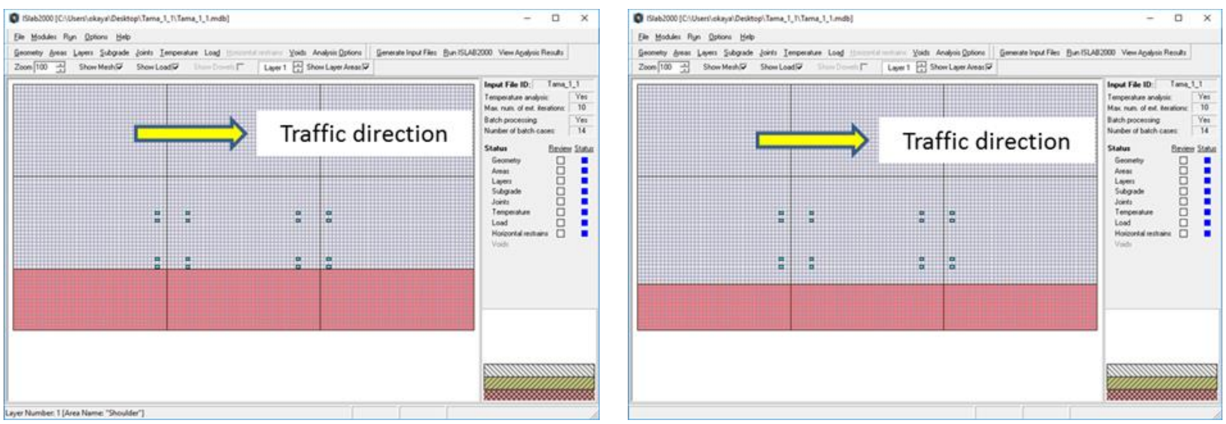
- Regular slabs (12 ft) with HMA shoulder alternative; shoulder thickness is less than regular slab thickness (i.e., 8 in.)
- Widened slabs (14 ft) with HMA shoulder alternative; shoulder thickness is less than regular slab thickness (i.e., 8 in.)

Granular shoulder using

- Regular slabs (12 ft) with granular shoulder
- Widened slabs (14 ft) with granular shoulder

The critical load configurations found in the truckload simulations were used for mechanical load configurations. Five different wander distances were investigated: 0, 0.5, 1, 1.5, and 2 ft, respectively, away from the lane edge for widened slabs, and at the slab edge itself for regular slab sizes. Other model inputs were the same as in the truckload simulations (Table 35).

Figure 89 shows the discretized models for the shoulder design alternatives. The widened slab (14 ft wide) had a 2 ft extended width compared to a regular slab size (12 ft wide). An alternative shoulder width was selected to ensure that the total width, including both slab and shoulder, would constitute a 20 ft widened slab with a 6 ft shoulder and a regular slab width with an 8 ft shoulder (Figure 89).



Regular slab sizes (12 ft) with shoulder design alternatives

Widened slab sizes (14 ft) with shoulder design alternatives

Figure 89. Widened and regular slab sizes with shoulder design alternatives

Tied PCC Shoulder

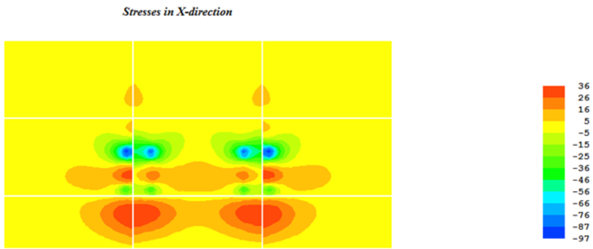
In this alternative shoulder scenario, two cases were compared:

- A regular slab (12 ft wide) with a full-depth tied PCC shoulder in which the shoulder thickness is the same as a regular slab thickness (i.e., 10 in.)

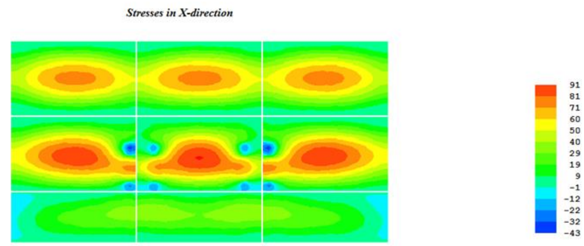
- A widened slab (14 ft wide) with a tied PCC shoulder alternative in which the shoulder thickness is less than a regular slab thickness (i.e., 7 in.)

A full-depth tied PCC shoulder has the same PCC thickness as its adjacent slab (10 in. in this case), while the tied PCC shoulder has a thickness of 7 in., as shown in the typical design cross-section in Figure 88.

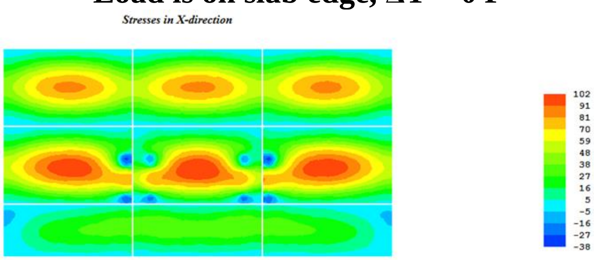
Figure 90 shows the top tensile stress distribution when a full-depth tied PCC shoulder along with a regular size slab (12 ft wide) was used with a truck load applied at the slab edge for four temperature load cases: mechanical load only and combined mechanical and temperature load cases (ΔT = -16°F, -18°F, and -20°F). As seen in Figure 90, a high top tensile stress accumulation is observed at about mid-slab as the negative temperature gradient increases.



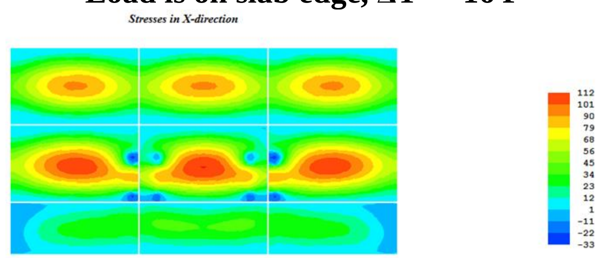
Load is on slab edge, ΔT = 0°F



Load is on slab edge, ΔT = -16°F



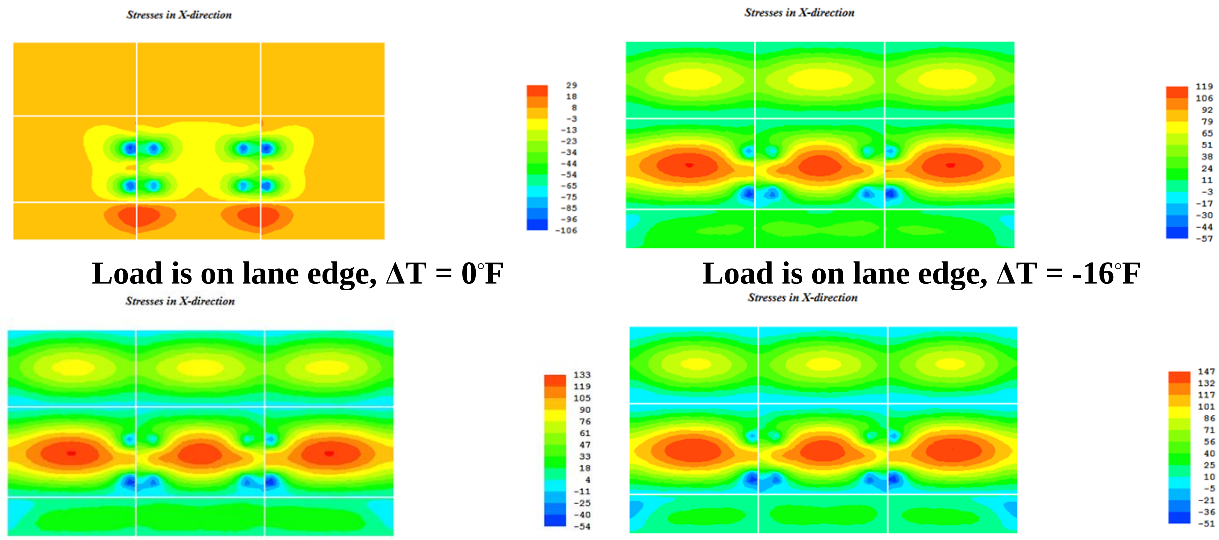
Load is on slab edge, ΔT = -18°F



Load is on slab edge, ΔT = -20°F

Figure 90. Full-depth tied PCC shoulder with regular size slab – top tensile stress distribution for four temperature load cases

Figure 91 shows the top tensile stress distribution when a tied PCC shoulder along with a widened slab (14 ft wide) was used with a truckload applied at the lane edge (2 ft from the slab edge) for four temperature load cases: only mechanical load and combined mechanical and temperature load cases (ΔT = -16°F, -18°F, and -20°F).

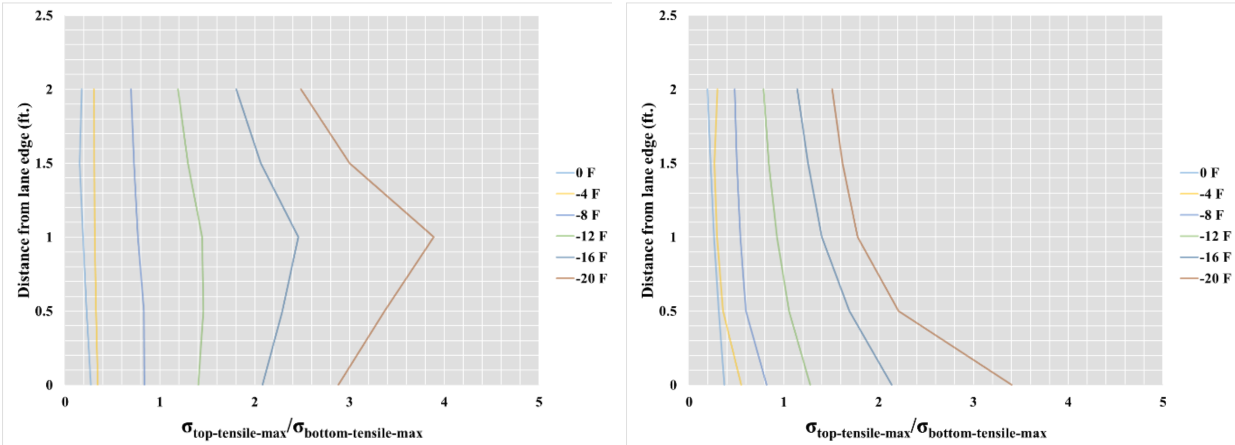


Load is on lane edge, ΔT = 0°F **Load is on lane edge, ΔT = -16°F**

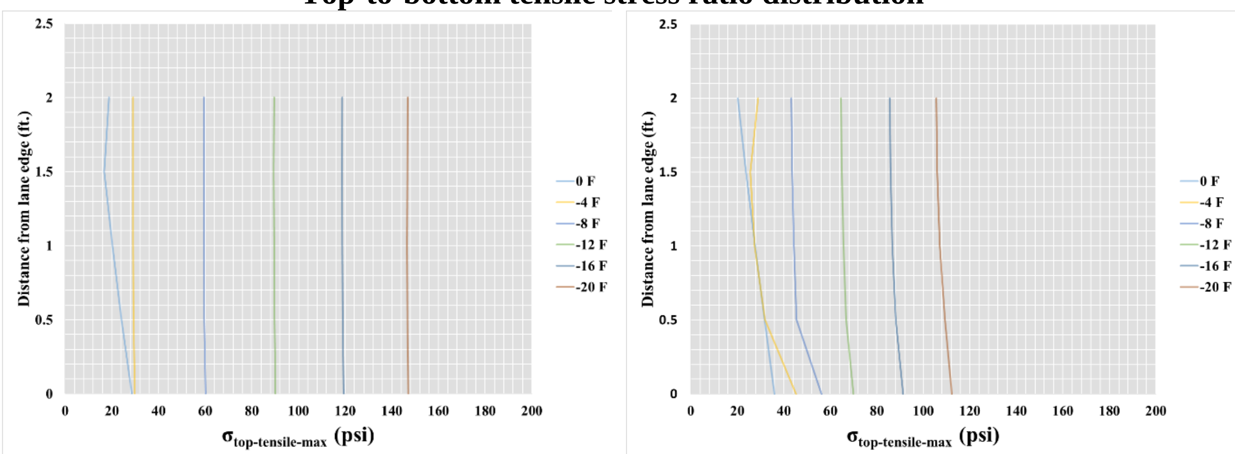
Load is on lane edge, ΔT = -18°F **Load is on lane edge, ΔT = -20°F**

Figure 91. Tied PCC shoulder with widened slab – top tensile stress distribution for four temperature load cases

Figure 92 compares the top-to-bottom tensile stress ratios and top tensile stress distributions between a widened slab with a tied PCC shoulder and a regular slab with a full-depth tied PCC shoulder. As shown in Figure 92, higher top-to-bottom tensile stress ratios and top tensile stresses were observed for a widened slab with a tied PCC shoulder alternative compared to a regular slab with a full-depth tied PCC shoulder.



Top-to-bottom tensile stress ratio distribution



Top tensile stress distribution

Widened Slab with Tied PCC Shoulder Alternate | **Regular Slab with Full-depth Tied PCC Shoulder**

Figure 92. Top-to-bottom tensile stress ratio and top tensile stress comparisons between a widened slab with tied PCC shoulder and a regular slab with a full-depth tied PCC shoulder

Results based on the tied PCC shoulder scenarios are summarized as follows:

- Higher top-to-bottom tensile stress ratio and top tensile stresses were observed for a widened slab with a tied PCC shoulder alternative compared to a regular slab with a full-depth tied PCC shoulder.
- In terms of longitudinal cracking potential, the following was observed:
  - The mid-slab edge was found to be critical when regular slabs were used.
  - The transverse joint edge was found to be critical when widened slabs were used.

What-If Scenario: Truck on Tied PCC shoulder

In this scenario, a truckload is placed on a tied PCC shoulder with a widened slab, with the truck having the same axle configuration as in the previous case. The tied PCC shoulder has a 7 in. thickness, the same pavement configuration as in the previous case. The objective of this analysis is to determine tensile stress distribution in a scenario where a truck is driving or standing on a tied PCC shoulder edge (Figure 93).

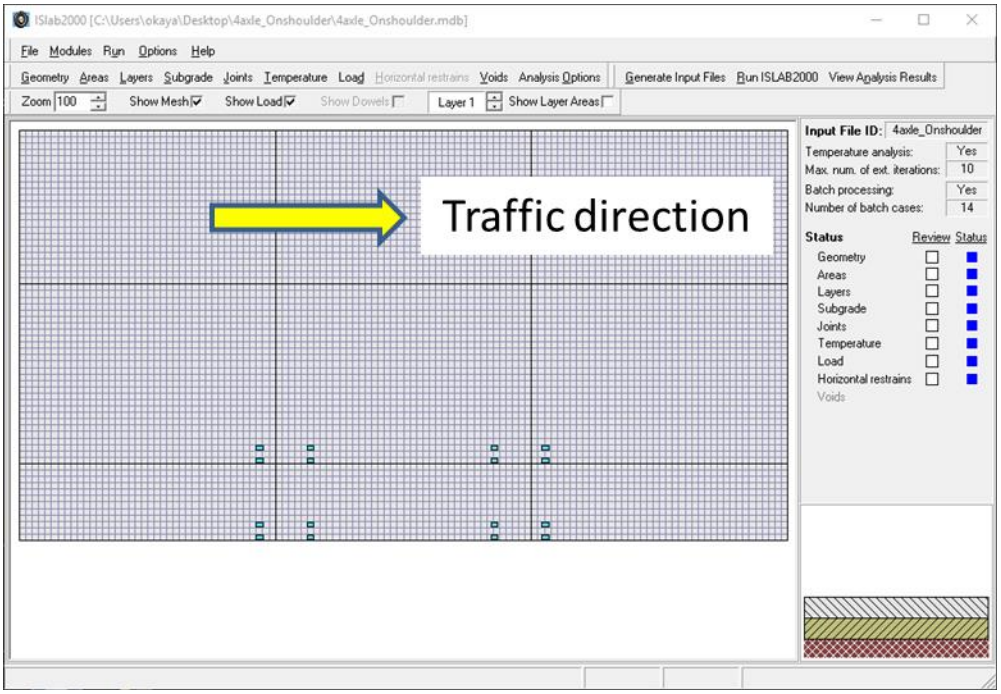
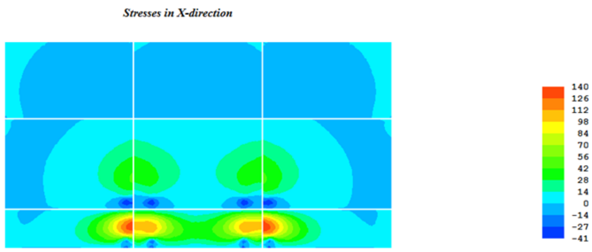
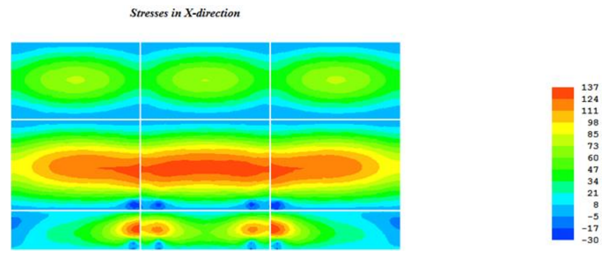


Figure 93. Truck on tied PCC shoulder

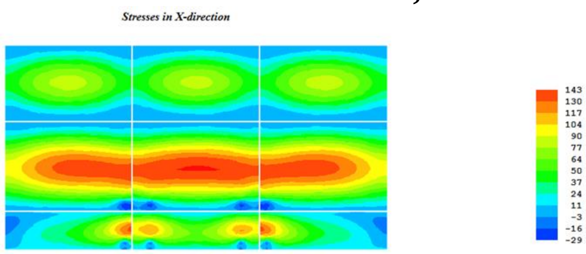
Figure 94 shows the top tensile stress distribution when a truckload is applied on a tied PCC shoulder edge for four temperature load cases: mechanical load only and combined mechanical and temperature load (ΔT = -16°F, -18°F, and -20°F). As seen in Figure 94, very high top-to-bottom tensile stress ratios can be observed between the lane edge and wheel path, and top-to-bottom tensile stress ratios are as high as 6.



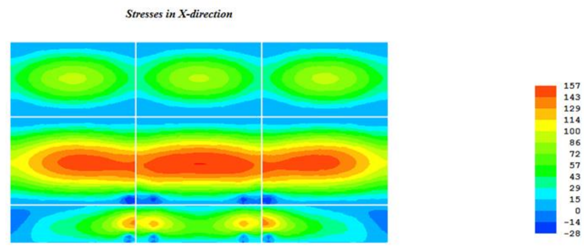
Load on tied PCC shoulder, ΔT = 0°F



Load on tied PCC shoulder, ΔT = -16°F



Load on tied PCC shoulder, ΔT = -18°F



Load on tied PCC shoulder, ΔT = -20°F

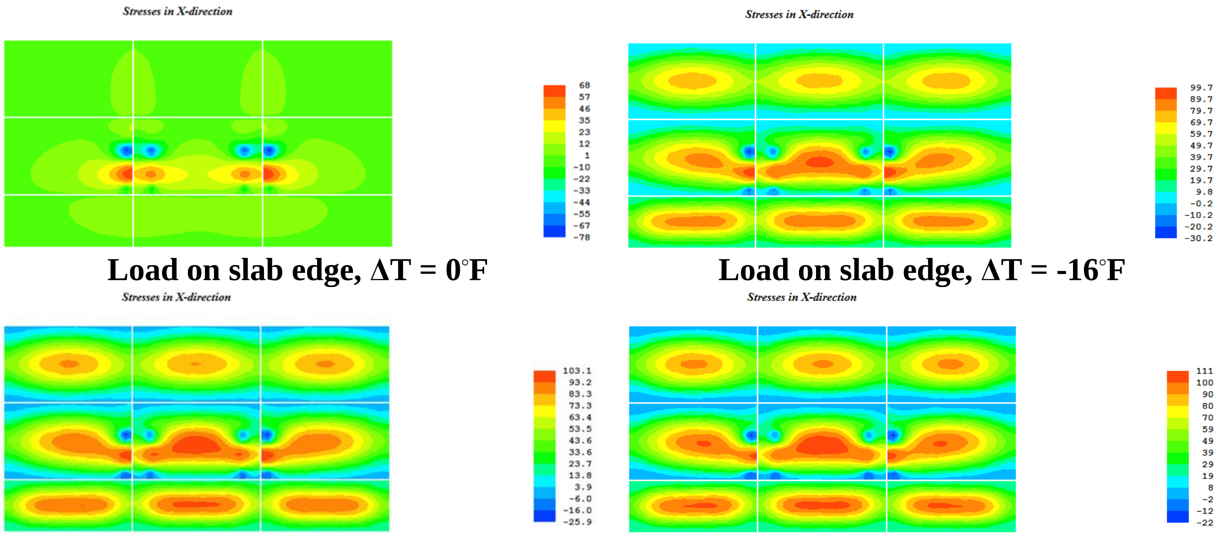
Figure 94. Truck on tied PCC shoulder edge – top tensile stress distribution for four temperature load cases

HMA Shoulder

In this alternative shoulder scenario, two cases were compared.

- Regular slab (12 ft wide) with a HMA shoulder alternative in which the shoulder thickness is less than a regular slab thickness (i.e., 8 in.)
- Widened slab (14 ft wide) with HMA shoulder alternative in which the shoulder thickness is less than a regular slab thickness (i.e., 8 in.)

Figure 95 shows the top tensile stress distribution when a HMA shoulder and regular size slab (12 ft wide) is used with a truckload applied on the slab edge for four temperature-load cases: mechanical load only and combined mechanical and temperature load (ΔT = -16°F, -18°F, and -20°F). As can be seen in Figure 95, high top tensile stress accumulation can be observed near the mid-slab as the negative temperature gradient increases.

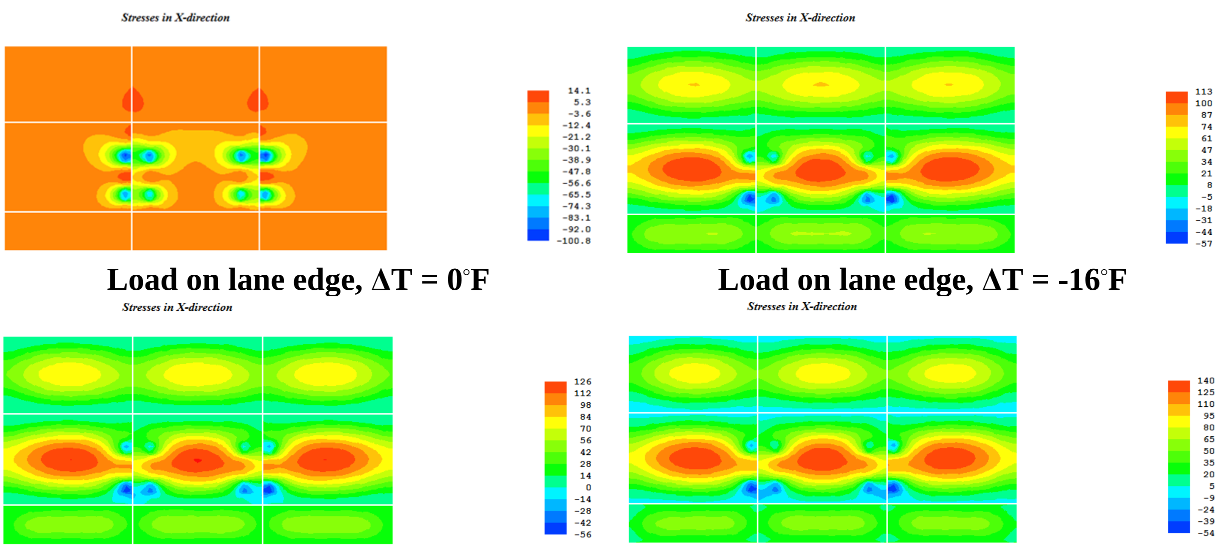


Load on slab edge, ΔT = 0°F **Load on slab edge, ΔT = -16°F**

Load on slab edge, ΔT = -18°F **Load on slab edge, ΔT = -20°F**

Figure 95. HMA shoulder with regular size slab – top tensile stress distribution for four temperature load cases

Figure 96 shows the top tensile stress distribution when a HMA shoulder along with a widened slab (14 ft wide) is used with a truckload applied at the lane edge (2 ft from the slab edge) for four temperature load cases: mechanical load only and combined mechanical and temperature load (ΔT= -16 °F, -18 °F, and -20 °F).

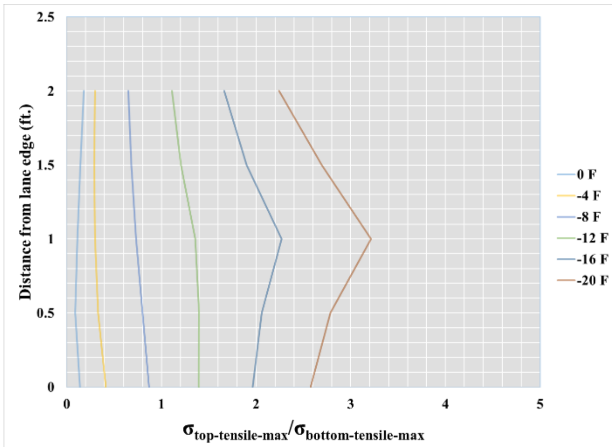


Load on lane edge, ΔT = 0°F **Load on lane edge, ΔT = -16°F**

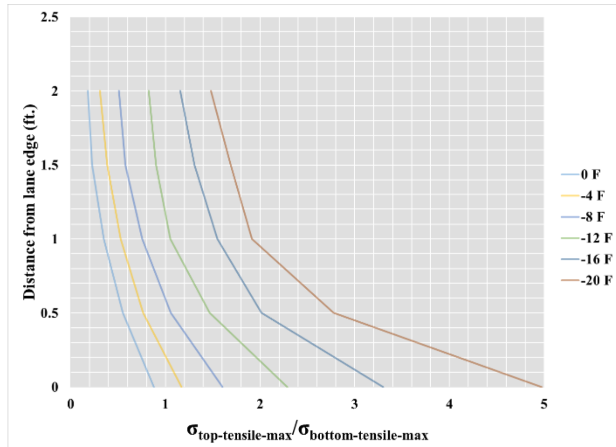
Load on lane edge, ΔT = -18°F **Load on lane edge, ΔT = -20°F**

Figure 96. HMA shoulder with widened slab – top tensile stress distribution for four temperature load cases

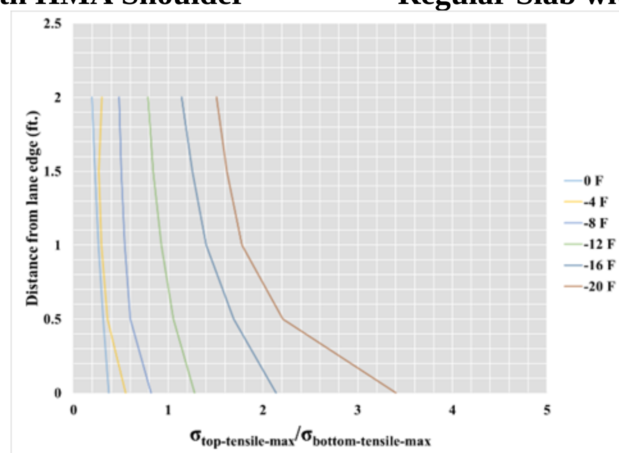
Figure 97 compares the top-to-bottom tensile stress ratios between a widened slab with a HMA shoulder, a regular slab with a HMA shoulder, and a regular slab with a full-depth tied PCC shoulder. As can be seen in Figure 97, the highest top-to-bottom tensile stress ratio was observed for the regular slab with a HMA shoulder.



Widened Slab with HMA Shoulder



Regular Slab with HMA Shoulder



Regular Slab with Full-Depth Tied Concrete Shoulder

Figure 97. Top-to-bottom tensile stress ratio comparisons between widened slab with HMA shoulder, regular slab HMA shoulder, and regular slab with full-depth tied PCC shoulder

Results based on the HMA shoulder scenarios are summarized as follows:

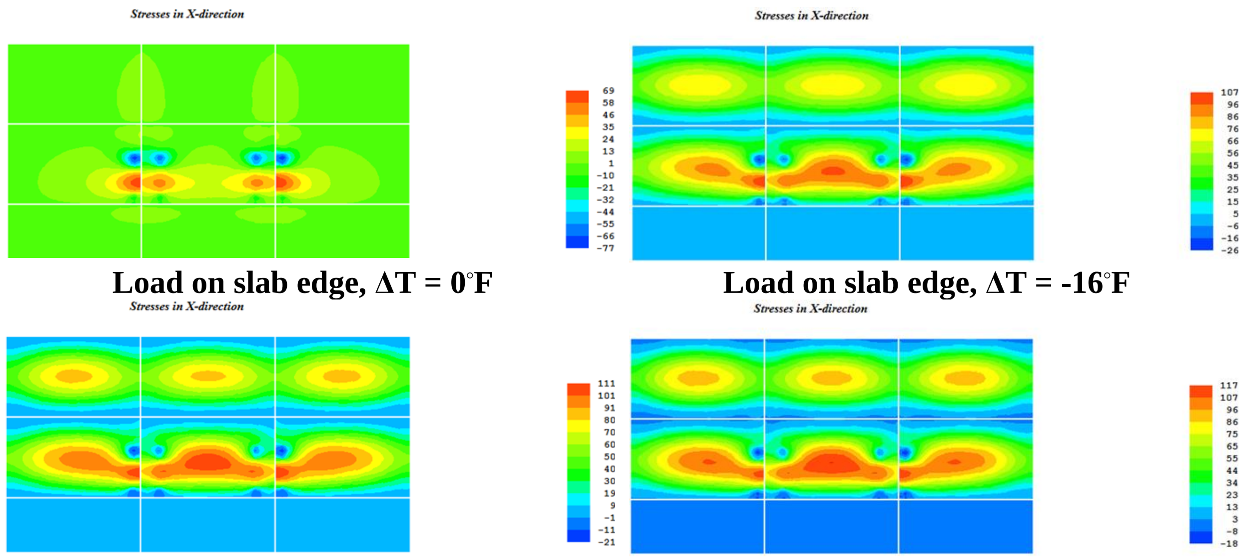
- A higher top-to-bottom tensile stress ratio was observed for a regular size slab (12 ft) with a HMA shoulder alternative compared to a widened slab (14 ft) with a HMA shoulder alternative.

Granular Shoulder

In this alternative shoulder scenario, two cases were compared:

- Regular slab (12 ft wide) with a granular shoulder alternative
- Widened slab (14 ft wide) with a granular shoulder alternative

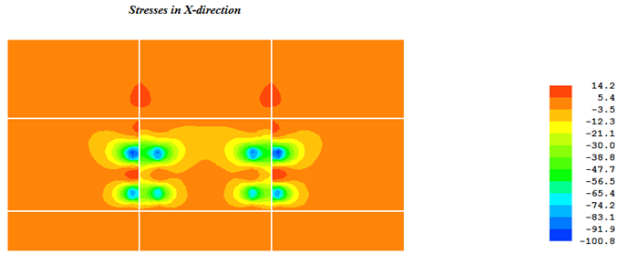
Figure 98 shows the top tensile stress distribution when a granular shoulder along with a regular size slab (12 ft wide) is used with a truckload applied on the slab edge for four temperature load cases: mechanical load only and combined mechanical and temperature load cases (ΔT = -16°F, -18°F, and -20°F). As seen in Figure 98, a higher top tensile stress accumulation is observed around mid-slab as the negative temperature gradient increases.



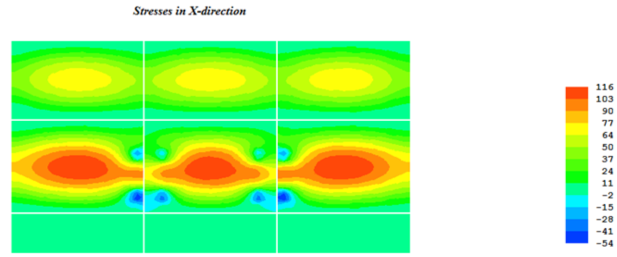
Load on slab edge, ΔT = 0°F
Load on slab edge, ΔT = -16°F
Load on slab edge, ΔT = -18°F
Load on slab edge, ΔT = -20°F

Figure 98. Granular shoulder with regular size slab – top tensile stress distribution for four temperature load cases

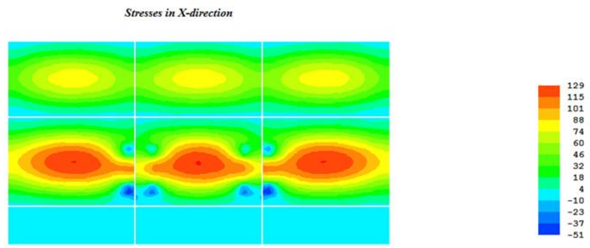
Figure 99 shows the top tensile stress distribution when a granular shoulder along with a widened slab (14 ft wide) is used with a truckload applied on the lane edge (2 ft from the slab edge) for four temperature load cases: mechanical load only and combined mechanical and temperature load (ΔT = -16°F, -18°F, and -20°F).



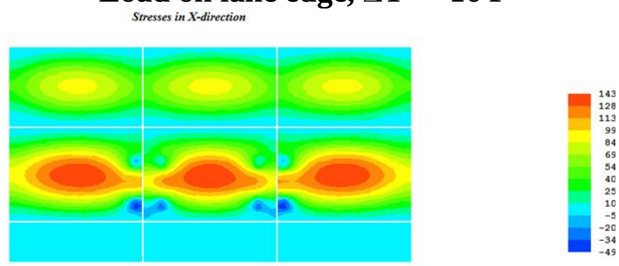
Load on lane edge, ΔT = 0°F



Load on lane edge, ΔT = -16°F



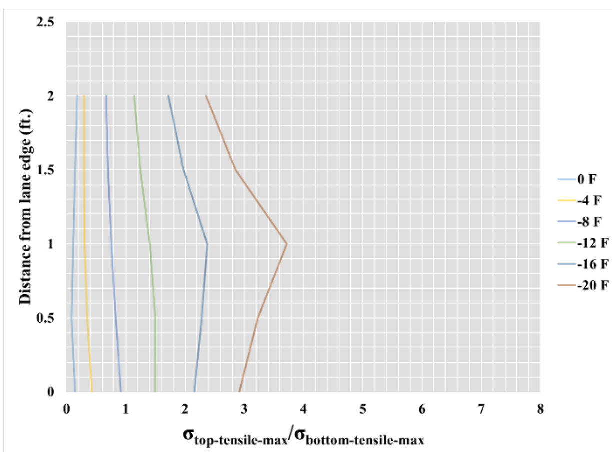
Load on lane edge, ΔT = -18°F



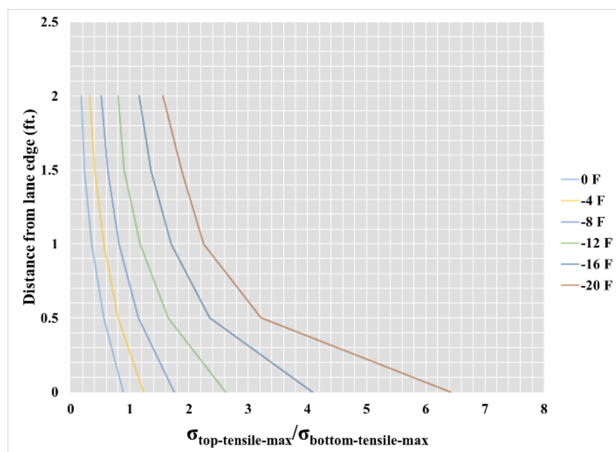
Load on lane edge, ΔT = -20°F

Figure 99. Granular shoulder with widened slab – top tensile stress distribution for four temperature load cases

Figure 100 compares the top-to-bottom tensile stress ratios between a widened slab and a regular slab for the granular shoulder cases. As seen in Figure 100, a higher top-to-bottom tensile stress ratio was observed for the regular slab with a granular shoulder compared to the widened slab with a granular shoulder.



Widened Slab with Granular Shoulder



Regular Slab with Granular Shoulder

Figure 100. Top-to-bottom tensile stress ratio comparisons between widened slab with granular shoulder and regular slab with granular shoulder

Results based on granular shoulder scenarios are summarized as follows:

- A higher top-to-bottom tensile stress ratio was observed for a regular size slab (12 ft wide) with a granular shoulder alternative compared to a widened slab (14 ft) with a granular shoulder alternative.

Shoulder Design Alternatives - Summary of Findings

A higher top-to-bottom tensile stress ratio and top tensile stress were observed when a widened slab (14 ft wide) with a tied PCC shoulder alternative was used compared to a regular size slab (12 ft wide) with a full-depth tied PCC shoulder. A higher top-to-bottom tensile stress ratio was also observed when a regular size slab (12 ft wide) with a HMA or granular shoulder was used compared to a widened slab (14 ft wide) with a HMA or granular shoulder. Compared to the use of a widened slab, the use of a regular size slab was found to be beneficial in mitigating longitudinal cracking at the expense of increasing transverse cracking potential.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The primary objectives of this study were to identify the causes of longitudinal cracking in 14 ft widened JPCP slabs in Iowa and to provide recommendations for minimizing and preventing such cracking. An extensive literature review was conducted to investigate the factors contributing to unexpected longitudinal cracking. Field surveys were performed on 12 selected widened JPCP sites with various ages, slab shapes, shoulder types, mix designs, environmental conditions during construction, and traffic levels, and the location and extent of existing longitudinal cracking were well documented. Concrete cores were also examined to better understand the development of such cracking. Finite element analyses were conducted using ISLAB 2005 and EverFE 2.25 to simulate the pavement response of widened JPCP under different loading cases and temperature gradients to identify the critical locations for longitudinal cracking. The project conclusions are summarized in this chapter.

Conclusions from Field Investigations

- Longitudinal cracks observed in this study were mainly in the traffic lane about 2 to 4 ft away from slab edges and parallel to the traffic direction.
- Some arc-shaped longitudinal cracks were also observed, and these can generally be attributed to poor foundation construction (i.e., improper subbase/subgrade construction or frost heave) (Voigt 2002).
- Most observed longitudinal cracks started from slab joints, and cracks are more prone to initiation at joints.
- The longitudinal cracks observed were all on pavement with skewed joints, and similar observations were reported in a WisDOT study (Owusu-Ababio and Schmitt 2013).
- Few longitudinal cracks with low severity levels were found at control site 1 (less than 3 ft), and longitudinal cracks were not observed at control site 2 in Polk County, which used tied PCC shoulders, even though these two control sites experienced relatively higher traffic volumes.
- Based on field observations, sites with tied PCC shoulders performed better than sites with either HMA or granular shoulders in terms of the level of longitudinal cracks observed.
- Based on field observations, sites with HMA shoulders had fewer longitudinal cracks observed in comparison to sites with granular shoulders.
- Sites with higher truck traffic volumes also had more observed longitudinal cracks.
- Concrete core samples indicate the existence of top-down longitudinal cracking.

Conclusions from the Numerical Analysis

- Top-down longitudinal cracking potential for JPCP with widened slabs was demonstrated repeatedly, and longitudinal cracks were initiated from the transverse joints between the lane edge and wheel path. Longitudinal crack potential increased with a higher negative temperature gradient.

- Skewed jointed widened slabs have higher potential for developing longitudinal cracking compared to rectangular joints.
- Higher (1) top-to-bottom tensile stress ratios and (2) top tensile stresses were observed when a widened slab (14 ft wide) with a tied PCC shoulder alternative was used compared to a regular size slab (12 ft wide) with a full-depth tied PCC shoulder.
- Higher top-to-bottom tensile stress ratios were observed when a regular size slab (12 ft wide) with a HMA or granular shoulder was used compared to a widened slab (14 ft wide) with a HMA or granular shoulder.

Recommendations

Based on a comprehensive literature review, detailed forensic investigations, and numerical analyses, the following recommendations for widened JPCP design features and construction practices to prevent and minimize longitudinal cracking have been developed:

- Longitudinal cracks occur mainly in the traffic lane about 2 to 4 ft away from slab edges, so shorter joint spacing can result in lower curling and warping and possibly offer less of a chance for longitudinal cracking as well.
- Dowel bars can restrain vertical deflection at joints, so proper dowel bar installation will help mitigate longitudinal cracking because most longitudinal cracks were observed to start from slab transverse joints.
- Use of a rectangular joint design option instead of a skewed jointed design option is recommended. (Note that the Iowa DOT began using rectangular joints in widened JPCP after 2005.)
- A tied PCC shoulder design option can perform better than other shoulder design options.
- A regular slab (12 ft wide) with a full-depth tied PCC shoulder is recommended as a design option.
- Proper subbase compaction, joint sawing, and base/subbase treatment are crucial because longitudinal cracking is believed to be more closely related to poor construction practices than to traffic loads.
- For the specified 14 ft panel, a width-to-thickness ratio of 1.2 (12 in. thickness) to 1.5 (9.5 in. thickness) should be used to minimize the severity and extent of cracking.
- Better performance is expected with a 14 ft panel when it is used in conjunction with a normal joint orientation, untreated aggregate base, and transverse joint spacing of 15 ft.
- Internal curing for concrete can help reduce the degree of curling, stress development, and cracking potential of concrete pavement.
- Increased coarse aggregate content can lead to decreased shrinkage, possibly resulting in decreased warping that can lessen the potential for longitudinal cracking.
- Use of aggregates with a low coefficient of thermal expansion (i.e., limestone) can help mitigate longitudinal cracking.
- A moderate w/c ratio is preferred because a higher w/c ratio can lead to higher drying shrinkage while a lower w/c ratio can lead to higher autogenous shrinkage, possibly resulting in higher potential for warping and longitudinal cracking.

- High ambient temperatures during paving can result in a higher degree of built-in curling and warping, so paving at night or in late fall could help reduce the potential for longitudinal cracking.
- Paving during cloudy weather is preferred because less solar radiation results in less built-in curling.
- Proper curing should be applied to mitigate curling and warping and to lower the potential for longitudinal cracking.

Also worth noting, most of the selected sites exhibiting different levels of longitudinal cracking in the field investigations described in this study were built with skewed joints before 2000, so they were approaching 20 years of service life. Since the Iowa DOT began using rectangular joints in widened JPCP after 2005, only one such site (US 30 in Tama County) was chosen for field investigation in this study, and it exhibited few longitudinal cracking issues.

The project technical advisory committee (TAC) recommended a follow-up study (i.e., Phase II study) to do the following:

- Identify and evaluate the effectiveness of various JPCP and shoulder design options, including current Iowa highway and county road practices for preventing longitudinal cracking through three concurrent research studies:
    - Numerical investigations employing finite element (FE) modeling
    - Field implementation, instrumentation, and monitoring for a set of test sections, including 12 ft regular slabs with a full-depth tied PCC shoulder design alternative and current Iowa widened JPCP slabs
    - Forensic evaluations of Iowa highway and county concrete pavements having longitudinal cracking problems
- Understand the mechanisms of longitudinal cracking failures and quantify longitudinal cracking potential for various JPCP design options, including current Iowa practices
- Evaluate the effectiveness of different JPCP design features and shoulder design options through real field implementation projects
- Identify the causes of longitudinal cracking in Iowa county concrete pavements
- Evaluate the in situ performance of rectangular slabs utilized in Iowa widened JPCP construction after 2005
- Develop implementation recommendations for the Iowa DOT, Iowa counties, and contractors to prevent and repair longitudinal cracking on Iowa concrete pavements
- Identify rehabilitation options for PCC slabs suffering from longitudinal cracking
- Conduct economic analyses for various JPCP and shoulder design options utilized to prevent longitudinal cracking

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APPENDIX A. IMAGE LOG OF VISITED SITES

(a)

(b)

(c)

(d)

Figure 101. Pavement at Control Site 1 – US 65 in Polk County

(a)


(b)

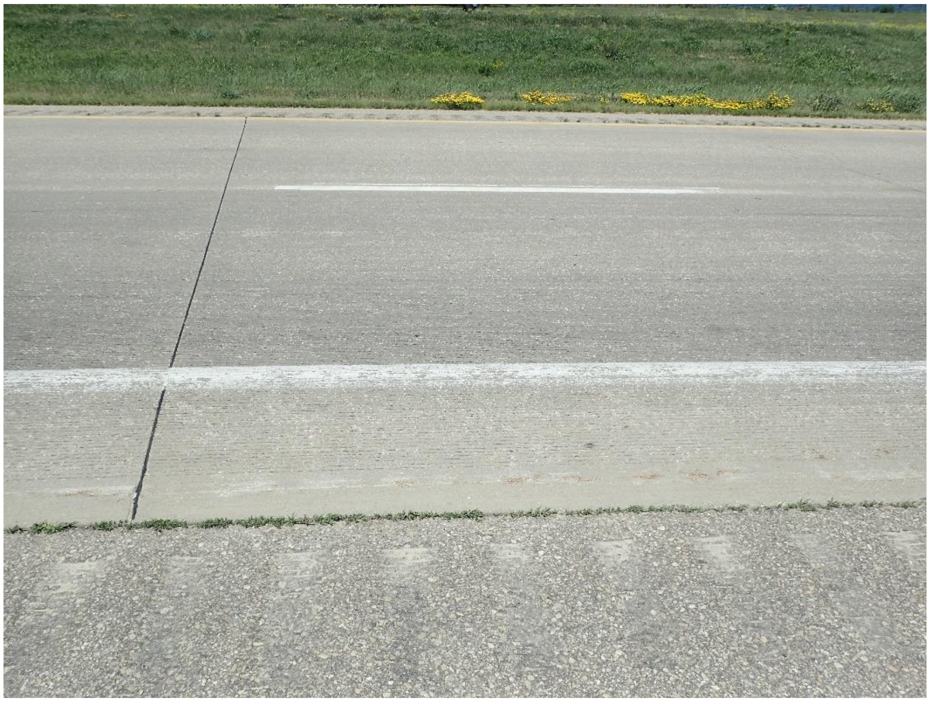
(c)

(d)

Figure 102. Pavement at Control Site 2 – US 65 in Polk County

(a)

(b)



(c)

(d)

Figure 103. Pavement at Control Site 3 – US 30 in Tama County

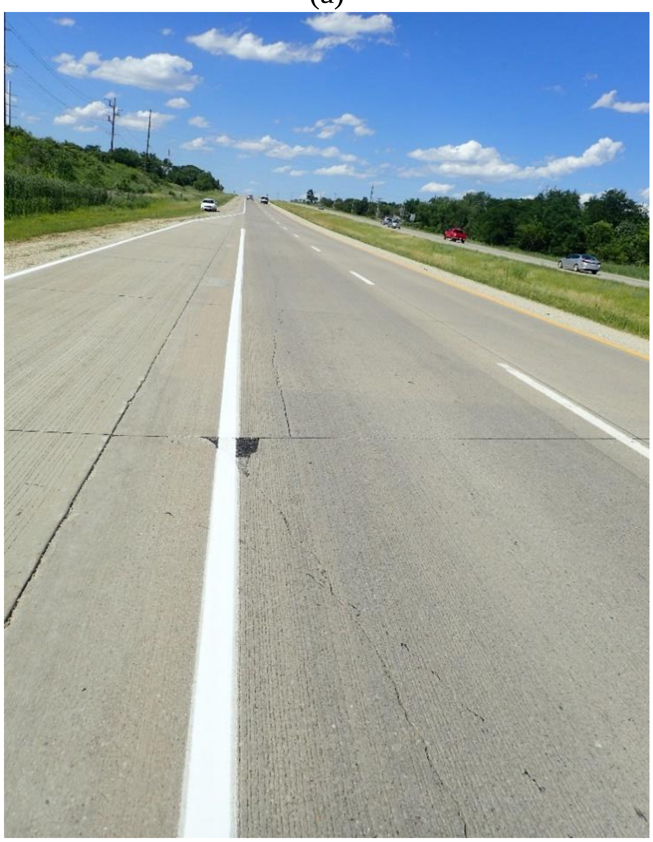
(a)

(b)

(c)

(d)

Figure 104. Pavement at Control Site 4 – US 30 in Story County


(a)

(b)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(f)

Figure 105. Pavement at LC Site 1 – US 151 in Linn County

(a)

(b)

(c)

(d)

(e)

(f)

Figure 106. Pavement at LC Site 2 – US 30 in Linn County


(a)

(b)


(c)

(d)

(e)

(f)

Figure 107. Pavement at LC Site 3 - IA 163 in Mahaska County


(a)

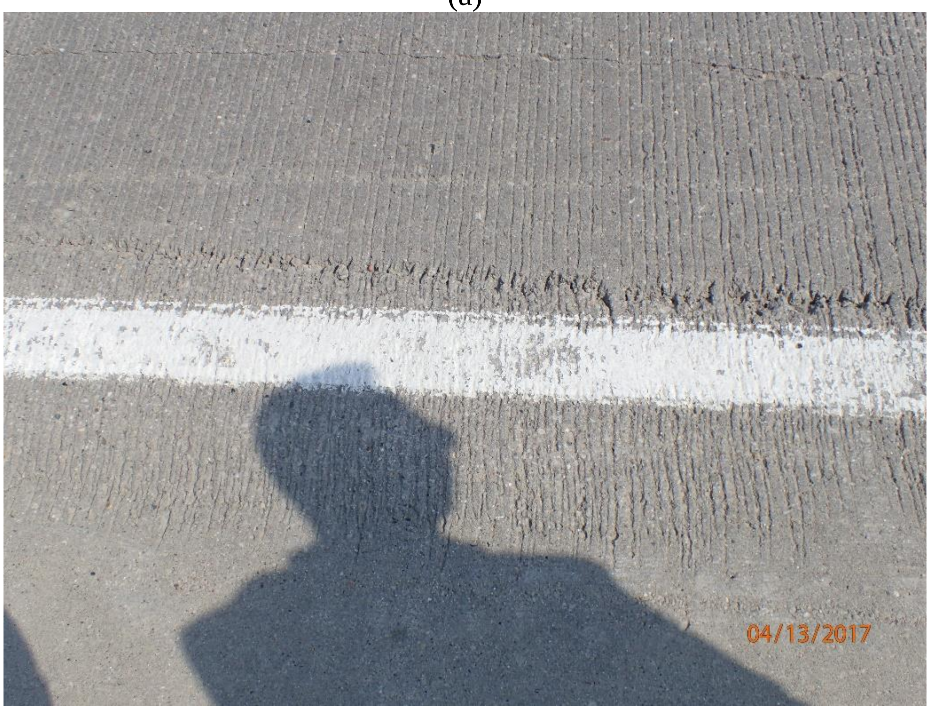
(b)

(c)

(d)

Figure 108. Pavement at LC Site 4 – US 218 in Henry County

(a)



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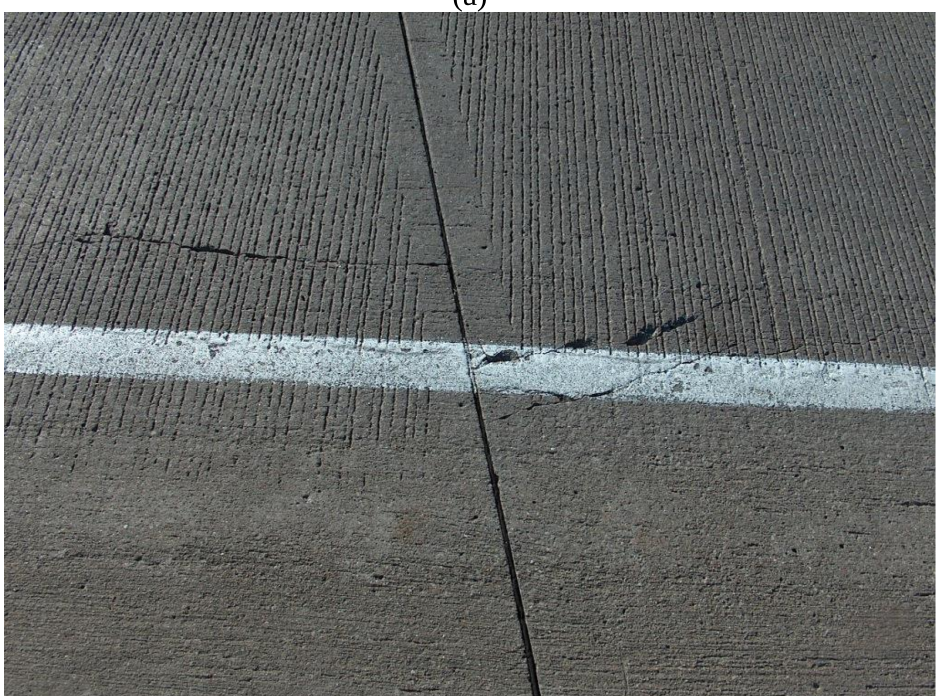
(d)

Figure 109. Pavement at LC Site 5 - IA 163 in Jasper County

(a)



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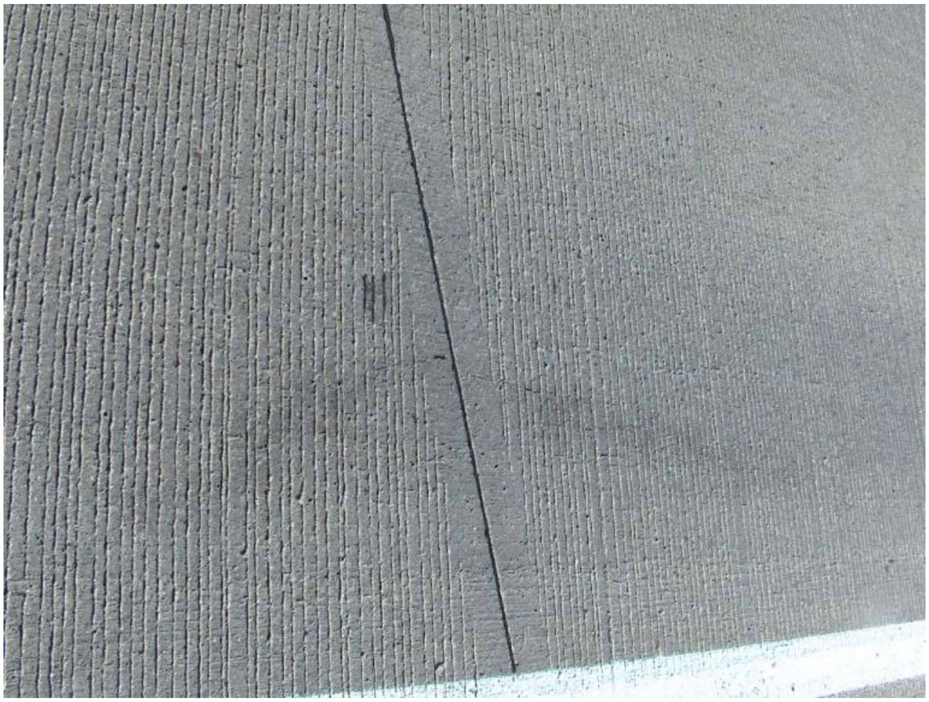

(c)

(d)

Figure 110. Pavement at LC Site 6 – US 65 in Polk County

(a)

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(d)

Figure 111. Pavement at LC Site 7 – US 61 in Lee County


(a)

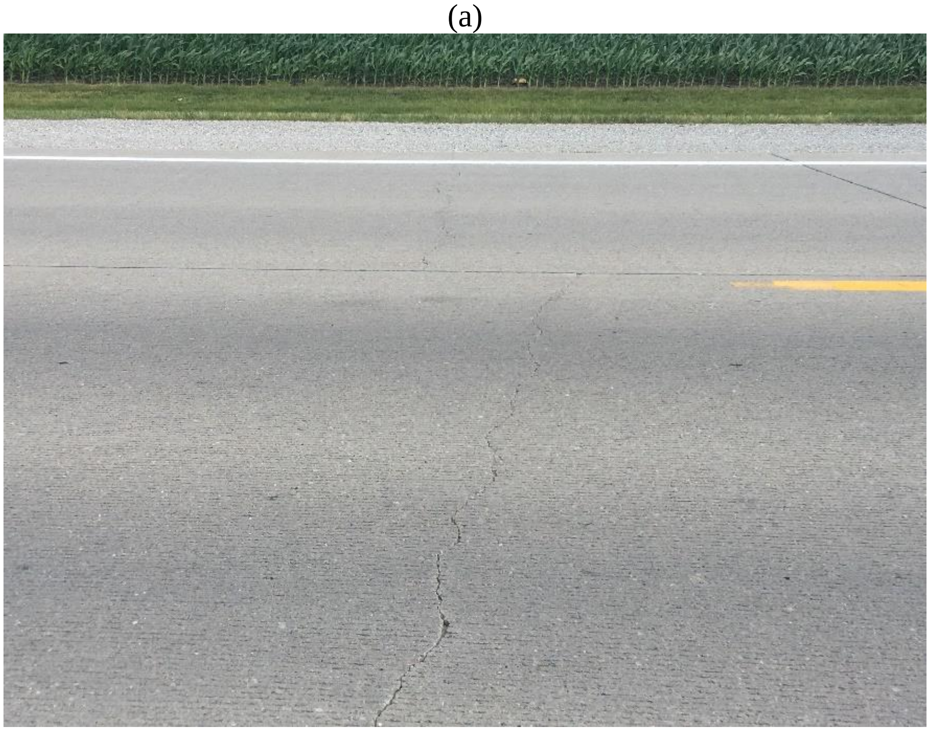
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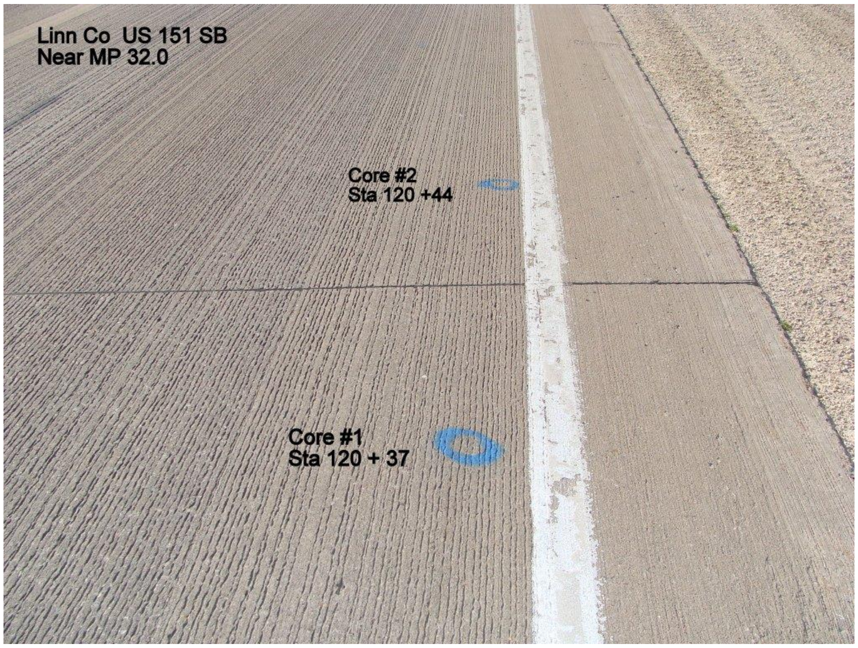
Figure 112. Pavement at LC Site 8 - IA 92 in Washington County

APPENDIX B. CONCRETE CORING IMAGES



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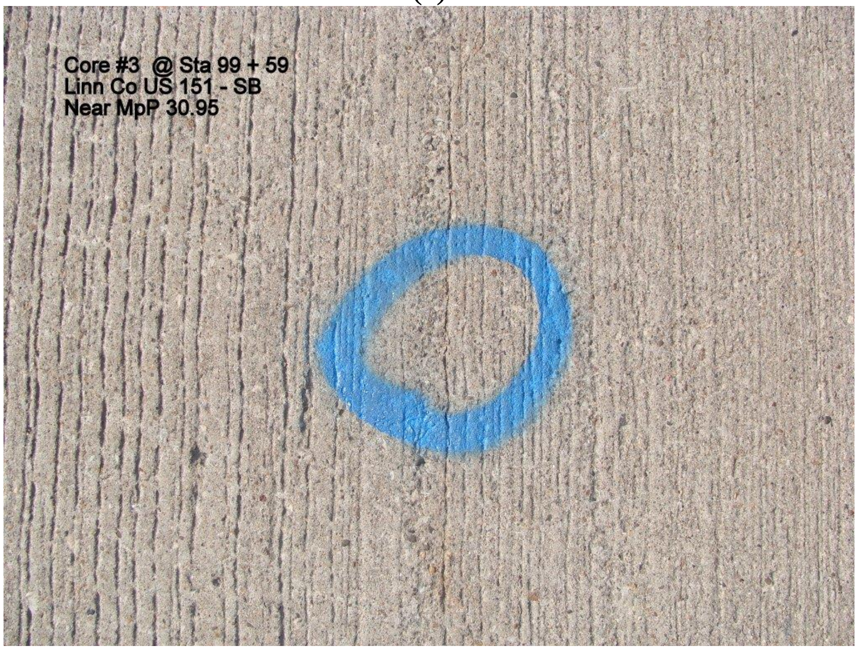

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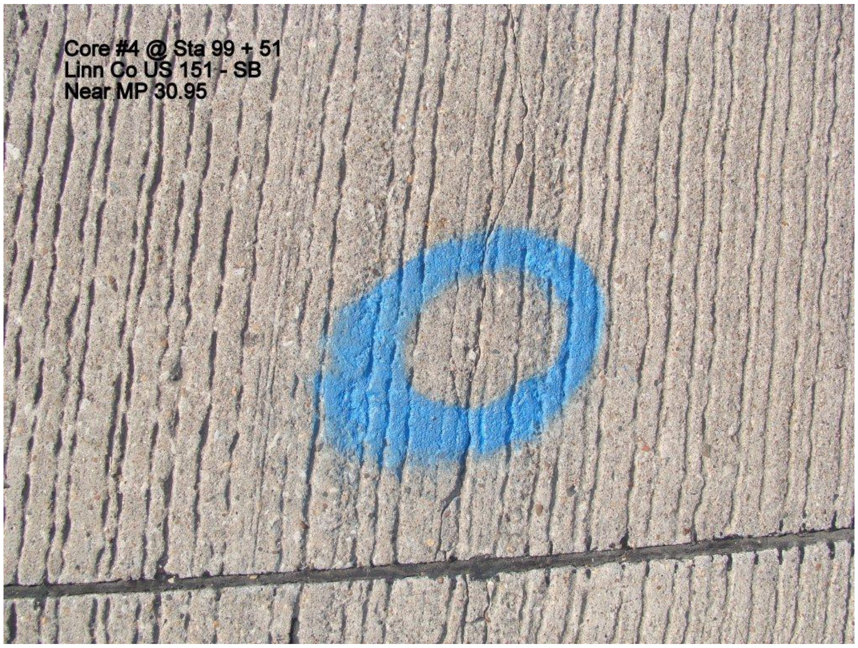
Figure 113. Coring location #1 to #3 at LC Site 1 – US 151 in Linn County



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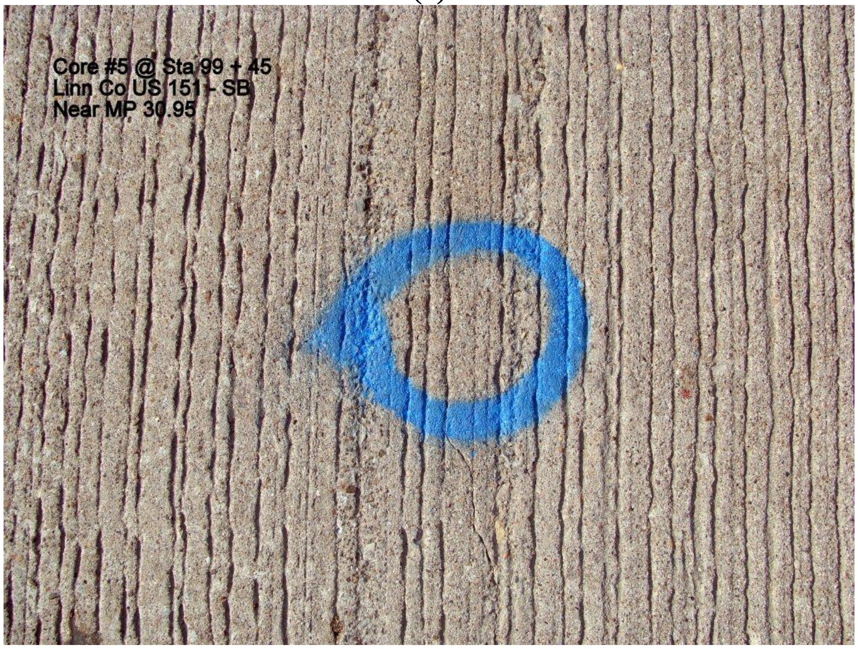

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Figure 114. Coring location #4 and #5 at LC Site 1 – US 151 in Linn County

(a)

(b)

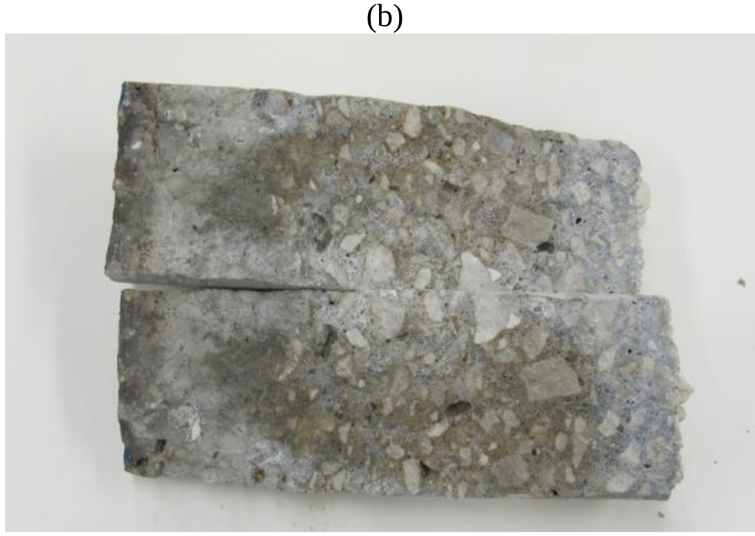

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Figure 115. Concrete cores #1 and #2 from LC Site 1 – US 151 in Linn County

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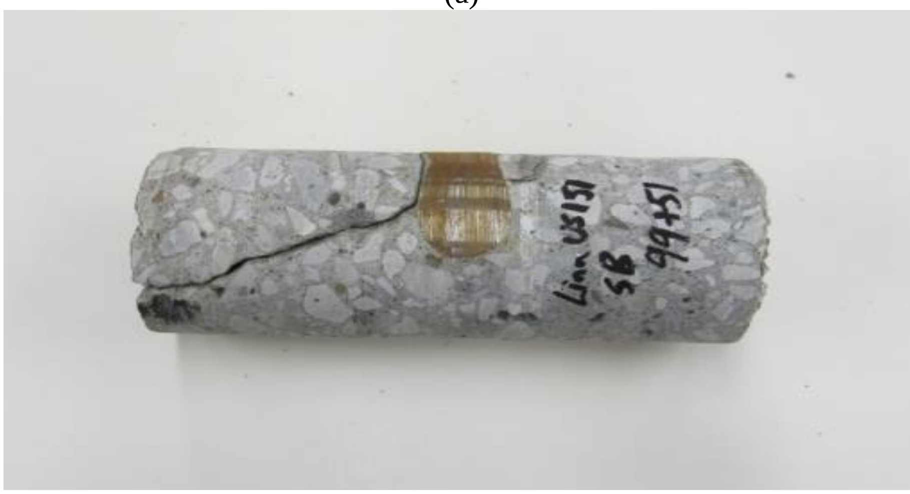

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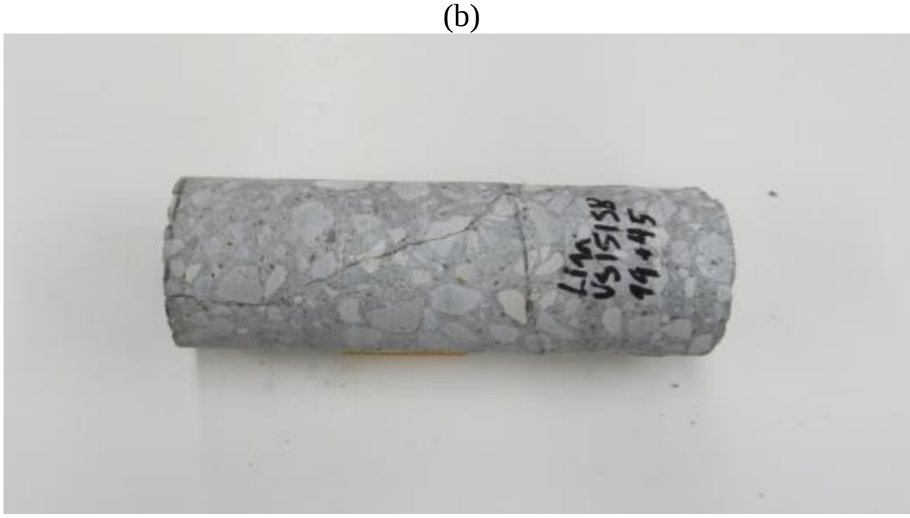

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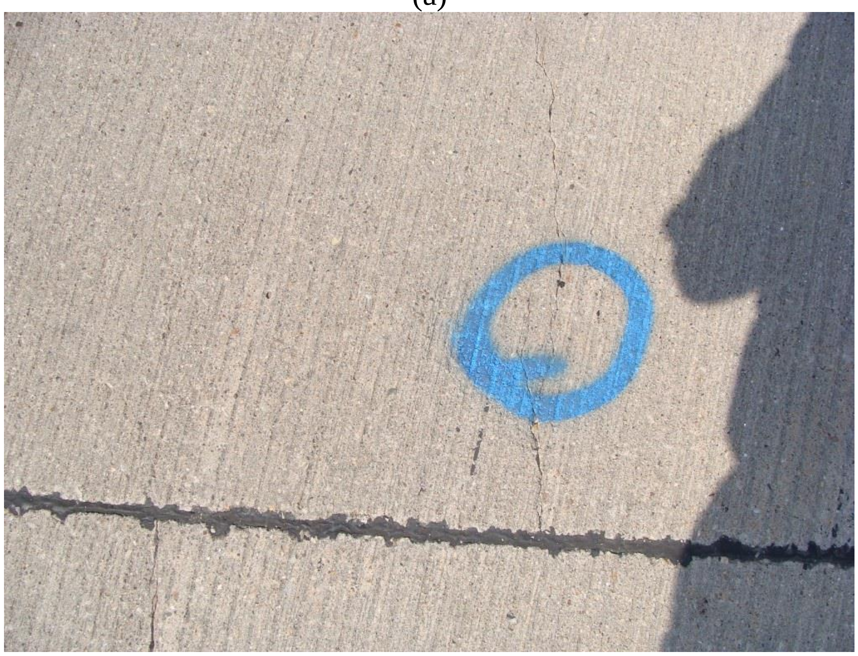
Figure 116. Concrete cores #3 to #5 from LC Site 1 – US 151 in Linn County

(a)



(b)

Figure 117. Coring locations #1 and #2 at LC Site 2 – US 30 in Linn County

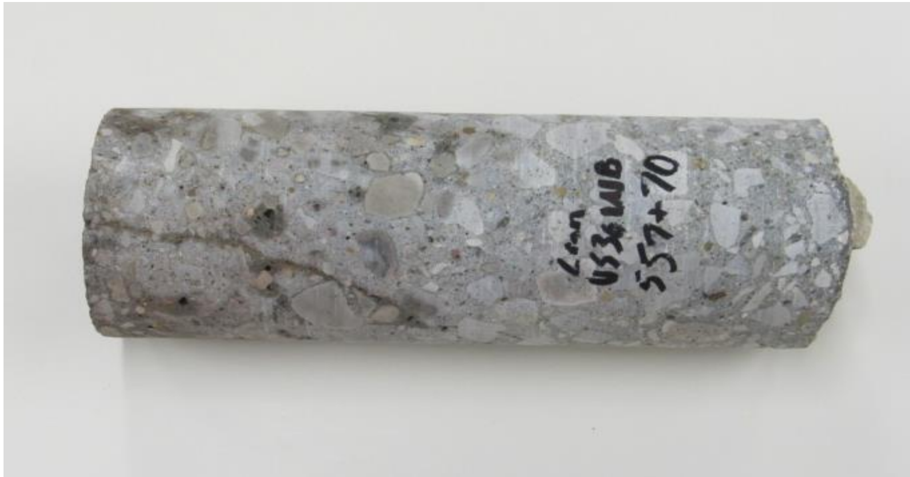
Figure 118. Coring locations #3 at LC Site 2 – US 30 in Linn County

(a)

(b)

(c)

Figure 119. Concrete core #1 from LC Site 2 – US 30 in Linn County



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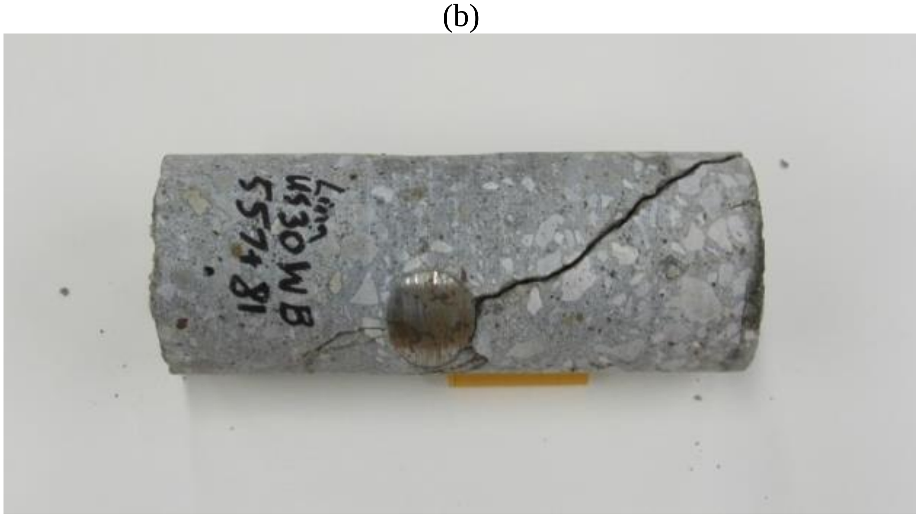

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Figure 120. Concrete cores #2 and #3 from LC Site 2 – US 30 in Linn County

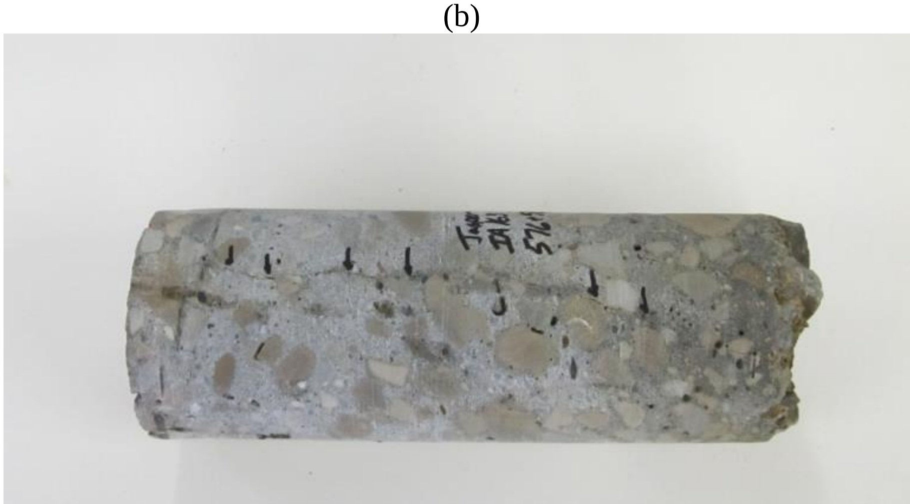
(a)

(b)

Figure 121. Concrete core from LC Site 3 – IA 163 in Mahaska County

(a)

(b)



(c)

Figure 122. Concrete cores from LC Site 5 – IA 163 in Jasper County

Figure 123. Concrete cores from LC Site 6 – US 65 in Polk County

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