Numerical Simulation of Partial-Depth Precast Concrete Bridge Deck Spalling

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Abstract: This paper describes the results of numerical simulations performed to investigate the spalling mechanism observed in several partial-depth precast prestressed concrete (PPC) bridge decks. Corrosion-induced cracking of prestressed steel reinforcement and panel butting were modeled using 2D finite element analysis to examine the nature of crack propagation that triggers the spalling effect observed. A parametric study was carried out on the basis of field observations from several bridges. FEM results showed that spalling is sensitive to side and bottom cover, and spacing of reinforcement attributable to bridging cracks. Findings indicate that the spalling mechanism is triggered by the presence of a critical bridging crack. DOI: 10.1061/(ASCE)BE.1943-5592.0000254. © 2012 American Society of Civil Engineers.

CE Database subject headings: Bridge Decks; Precast concrete; Corrosion; Finite element method; Simulation; Spalling.

Author keywords: Bridge decks; Concrete; Corrosion; Cracking; Finite element simulation; Spalling.

Introduction

This paper presents the results of numerical simulations conducted to examine the possible mechanisms of edge concrete spalling in partial-depth precast prestressed concrete (PPC) bridge deck panels. Some bridges with this deck system were recently observed to have experienced rusting of embedded steel reinforcement and edge concrete spalling issues in the PPC deck panels (Wenzlick 2008; Sneed et al. 2010; Wieberg 2010). From their works, corrosion of embedded steel reinforcement and degradation of concrete are believed to be the main sources of the spalling problem in the panel on the basis of explicit phenomenological evidence. However, the mechanism of the crack propagation that ultimately triggers the spalling could not be detected from the field investigation findings. Accordingly, numerical simulations using finite element (FE) techniques were conducted to investigate the causes and determine the mechanism of spalling observed in the panels.

Background

Bridge Deck Description

Partial-depth precast concrete deck panels are thin prestressed concrete panels that span between girders and serve as stay-in-place (SIP) forms for a cast-in-place (CIP) concrete bridge deck. Typical panel geometries are 75–90 mm (3.0–3.5 in) thick, 2.4 m (8 ft) long in the longitudinal direction of the bridge, and sufficiently wide to span between the girders in the bridge transverse direction. The panels are typically pretensioned with prestressing steel strands placed, and the CIP concrete portion of the deck [typically 125–140 mm (5.0–5.5 in) thick] is placed on top of the panels. In the service state, the CIP concrete and SIP panels act as a composite deck slab. Panels are typically constructed with a roughened top surface to transfer horizontal shear, and the prestressing strands in the panels serve as the bottom layer of reinforcement of the composite bridge deck.

The most common problem reported with the use of partial-depth deck panels is cracking in the CIP concrete portion of the deck at the transverse joints between panels and at the locations at which the panels are supported on the girders. The cause of the transverse reflective cracks can be attributed primarily to the concentration of shrinkage of cast-in-place concrete at the joints between the precast panels (Hieber et al. 2005). The transverse reflective cracks are not believed to affect significantly the structural performance of the deck slab (Goldberg 1987 and Sprinkel 1985). However, transverse reflective cracks generally raise a deterioration concern because they permit the ingress of moisture and corrosion agents to the reinforcement in the deck. Transverse reflective cracks usually extend only partway through the CIP portion of the deck slab, although in some instances they can extend through the entire thickness. Especially when full-thickness reflective cracks are present, the ingress of moisture and corrosion agents can be concentrated at the panel joint locations, evidenced by water staining and corrosion issues within the panels. This situation creates a concern in highly corrosive environments such as coastal areas or regions in which deicing salts are frequently used. Other adverse issues related to flexibility of supporting girders and/or improper panel bearing conditions may occur, including simple-span behavior of the bridge deck between girders instead of continuity over girders and transfer of all of the live load shear to CIP concrete topping. Delamination of the panels from the CIP concrete...
near the joints has also been reported (Fagundo et al. 1985; Ross Bryan Associates Inc. 1988; Abendroth 1991).

Researchers reported that butting of adjacent deck panels is another source of spalling-related problems in panels (Goldberg 1987; Hieber 2005). Inherent discontinuity of the partial depth precast panel deck system can result in external effect on the panel at the panel joints by butting of adjacent panels. Butting problems at the panel joints are mostly attributed to construction errors such as an insufficient gap between or misalignment of adjacent panels.

**Problem Description**

Recently, several bridges with partial-depth PPC panel bridge decks were observed to have experienced rusting of embedded steel reinforcement in deck panels and spalling of concrete at the bottom of panel joints (Wenzlick 2008; Sneed et al. 2010; Wieberg 2010). Bridges investigated as part of this study were located in the Midwest region of the United States and were constructed between 1985 and 1990. In each case, the panels were 75–90 mm (3.0–3.5 in) thick, 2.4 m (8 ft) wide, and supported on continuous steel plate girders spanning two to four spans.

Fig. 1 shows examples of the spalling observed during field investigations. Embedded steel reinforcement corrosion was found in many of the PPC panels at the panel edges. Cracking was observed on the bottom surface of panel in the direction of the steel tendons [Fig. 1(b)]. In panels that had not yet experienced spalling, these cracks appeared to be initiated at the tendon locations and to have propagated in the vertical direction to the bottom edge of panel. For panels that had experienced spalling, vertical cracks and an additional crack that extended in the horizontal direction to the side face of panel were present. This bidirectional cracking resulted in the spalling of a mass of concrete at the panel corner along the span length of the panel [Fig. 1(a)].

As shown Fig. 1, the spalling condition resulted in exposed prestressing tendons and mild temperature reinforcement at the panels edges. The tendons exhibit a large degree of corrosion, some to the extent of rupture. Field investigation study revealed the source of the corrosion to be the penetration of water and chlorides through the reflective cracking in the CIP topping, to the interface between the CIP topping and the PPC panels, then through the PPC panels to the prestressing tendons located near the panel joints. Other findings of note included full-thickness reflective cracking of the CIP topping slab in some locations and evidence of delamination at the CIP topping–PPC panel interface. Nondestructive evaluation results showed that deterioration of concrete and reinforcing steel was found to be concentrated near the areas of reflective cracking (corresponding to the panel joint locations). Also, physical gaps between adjacent PPC panels were noted to be variable in width and, in some cases, panels were in contact with one another (Fig. 2). This situation raised questions regarding potential effects of butting of adjacent panels.

**Numerical Simulation of Spalling Causes**

**Objective and Methodology**

The objective of the numerical simulations performed in this study was to determine the critical parameters that trigger the spalling mechanism observed in several partial-depth precast prestressed concrete (PPC) bridge decks. On the basis of findings from the field investigations discussed previously, two major sources of spalling were considered: spalling induced by corrosion of embedded reinforcing steel alone and spalling in combination with butting of adjacent panels.

Numerical models can provide valuable information to gain a better understanding of the mechanisms of damage propagation attributable to corrosion of reinforcement, delamination, and spalling of concrete structures. Thus, during the past decades, countless
research efforts have been devoted to estimating deterioration caused by reinforcement corrosion and predicting the service life of reinforced concrete structures using finite element (FE) methods of numerical modeling. However, most studies on this area focused on characterizing the mechanism of damage by utilizing the effective material properties and geometry of a member, and consequent element behavior. This situation is inevitable in FE modeling. Therefore, optimizing the modeling options to fit the purpose of this particular study is necessary.

In general, deterioration problems have been investigated using two-or three-dimensional (2D or 3D) FE analysis with respect to local areas considering a variety of configurations (Dagher and Kulendran 1992; Chen and Mahadevan 2008; Richard et al. 2010), whereas serviceability of deteriorated concrete structures has been simulated by 2D FE models for global area (Lee et al. 1999; Coronelli and Gamarova 2004; Saetta 2005). In 2D FE analysis of local sections, plane stress or strain elements were mostly used, and corrosion-induced forces were modeled by a uniform pressure of dilatation. The same concept of corrosion-induced force was also applied in 3D FE analysis for local sections with different finite elements. Regardless of the geometry of the FE model, adoption of proper material models is an important factor in developing an accurate model. Many established concrete models utilize strength and fracture mechanics criteria with respect to tensile and compressive behavior of concrete. Identifying the most appropriate for all cases is difficult because material models can be varied according to the purpose of the analysis. Presently, most commercial FE programs include material models as a built-in option.

The most critical feature in modeling reinforced concrete structures is the cracking model of FE elements. In FE analysis, reinforced concrete cracking has long been modeled using either the smeared crack model or the discrete crack model. The smeared crack model, initially proposed by Rashid (1968), characterizes cracking as systems of parallel cracks continuously distributed over the finite elements. Since its conception, many researchers have successfully utilized the smeared crack model, which has undergone a number of refinements. Important contributions to the classical smeared crack model were offered by Bazant (1979), leading to what is now referred to as the cracked band model. In contrast to the smeared model, the discrete crack model proposed by Ngo and Scordelis (1967) uses discrete cracks that form between the elements. Each concept has its own advantages and disadvantages. Although the discrete crack model has been successfully implemented in large FE programs, the smeared crack model offers a clear computational advantage, because only the material properties of the cracked elements are modified to account for the crack. In the discrete crack model, a node must be separated into two nodes to allow the crack to form, thereby increasing the number of nodes, changing the topological connectivity of the mesh, and destroying the band structure of the structural stiffness matrix. In addition, the discrete crack model cannot easily incorporate an unknown crack direction, because doing so may necessitate repositioning the nodes when the new crack direction does not closely coincide with existing element lines. Besides being more convenient to implement in a computer code, the crack band model was found to be essentially equivalent to the inter element crack model endowed with a softening cohesion zone near the crack tip. For this reason, most FE analyses have adopted the smeared crack model.

In this study, spalling simulations were carried out by developing a 2D FE model, which has a more flexible applicability with respect to local and global region simulation than a 3D FE model. Simulations were performed using the commercial FE program, DIANA, for a typical PPC panel having 75 mm (3.0 in) thickness and 2.4 m (8 ft) width. Panels of 75 mm (3.0 in) thickness were simulated because they are currently specified in new construction in parts of the Midwest. Because the simulation was conducted on the cross-section, longitudinal geometry and actions caused by prestressing forces were not considered. Compressive strength of concrete was assumed as 41.4 MPa (6 ksi). The FE model developed in this study adopted the Thorenfeldt et al. (1987) uniaxial model for the concrete compressive behavior and an exponential model for the concrete tensile behavior on the basis of fracture energy related to a crack bandwidth as commonly adopted in smeared crack models. Accordingly, the FE results represent the converged solution when the concrete tensile strain reaches the ultimate crack strain

\[ \epsilon_{ult} = 4.226 \frac{G_f}{f_t h} \]

where \( G_f \) = the fracture energy (N/mm); and \( f_t \) = concrete tensile strength (MPa); and \( h \) = crack bandwidth (mm). In the FE model, the value of crack bandwidth was computed as a function of the area of the element in DIANA.

**Corrosion-Induced Spalling Simulation**

Corrosion of reinforcement embedded in concrete is well known to develop internal pressure along the reinforcement, resulting in cracking of the surrounding concrete when the tensile strength is exceeded. Boundary and geometrical conditions at the spalled region of the panel have been found to be important factors that influence crack development and propagation. Accordingly, a 2D FE parametric analysis was carried out to simulate the effects of panel geometry on the nature of corrosion-induced crack development and propagation. Parameters varied included section position, side cover to reinforcement, and reinforcement spacing. Given model complexity and the number of parameters involved, the simulations were performed in two stages: preliminary and final. The preliminary FE simulation was conducted to examine parameters including boundary conditions and section geometry before developing the final FE model. After determining the appropriate boundary and section geometry, the final simulation was conducted to examine the influence of reinforcement spacing and side cover on the cracking behavior. Preliminary and final simulations are described in the sections that follow.

**Preliminary Simulation—Section Geometry and Boundary Conditions**

The purpose of the preliminary simulation was to investigate the effects of boundary conditions and section geometry (i.e., local and global behavior and discontinuous edge effects) on panel cracking and spalling behavior through a parametric study. The preliminary model utilized a typical FE model adopted by other researchers (Dagher and Kulendran 1992; Zhou et al. 2005; Ahmed et al. 2007; Chen and Mahadevan 2008) as discussed in the previous section. As shown in Fig. 3, local panel behavior was considered by modeling panel sections of different widths that include the corresponding number of steel tendons at a given spacing. Section thickness [75 mm (3.0 in)] and spacing of reinforcement [100–115 mm (4–4.5 in.)] were consistent with specifications for the panels investigated. Two different boundary conditions were adopted to represent the restraint conditions of the panel within the overall bridge deck. Cases P1 and P3 represent an interior section of the panel having three-side fixed boundaries along both side and top surfaces, whereas cases P2 and P4 represent an edge section of the panel having two-side fixed boundaries along one side and the top surface. (Although the spalling problem was observed only at the panel edges, the corrosion process within the interior...
sections was also of interest to study the degradation level in the middle of the panel and to compare crack propagation behavior between interior and edge sections.) In the preliminary model, incremental internal pressure representing the increasing internal pressure induced by rust expansion of the reinforcement was applied at each tendon location (concrete void in the model). The final stage of the corrosion process was determined when the FE analysis terminated because of the convergence problems caused by excessive tensile strains in the concrete.

The final stage of the corrosion process is summarized in Fig. 3 with respect to Von Mises stress variations and crack patterns. Panel sections of different widths in Fig. 3 are termed one section containing one tendon, two sections containing two tendons, and full panel representing the full panel width with all tendons. As shown in Fig. 3, no significant differences are observed between one and two sections in terms of stress variation and crack propagation. Cracks in all sections tend to propagate in the vertical direction, but not in the horizontal direction to adjacent reinforcement. These findings indicate that a large enough distance between adjacent reinforcement [115 mm (4.5 in) spacing in this case] can prohibit lateral crack propagation to adjacent reinforcement. Results also revealed that the cracking process is sensitive to the boundary conditions. Interior sections having three-sided fixed boundary conditions on top and both side surfaces (cases P1 and P3) show that crack propagation to the bottom surface is less severe than that of edge sections having two-sided fixed boundary conditions on top and right surfaces (cases P2 and P4). This behavior shows that constraint effects in the internal section tend to restrain the crack propagation in the vertical direction. Thus, clearly, edge sections are more vulnerable to corrosion-induced cracking effects than interior sections.

Whereas the preliminary FE analysis was able to simulate distinct differences in severity of crack propagation to the bottom surface in edge and interior sections, lateral crack propagation to the side surface was not triggered in any of the sections. Thus, other factors that likely contribute to the spalling problem were investigated and are discussed next.

<table>
<thead>
<tr>
<th>Case</th>
<th>Boundary Conditions</th>
<th>Von Mises Stress Variations</th>
<th>Crack Propagation</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1 One Section</td>
<td><img src="image" alt="P1 One Section" /></td>
<td><img src="image" alt="Von Mises Stress Variations" /></td>
<td><img src="image" alt="Crack Propagation" /></td>
</tr>
<tr>
<td>P2 One Section</td>
<td><img src="image" alt="P2 One Section" /></td>
<td><img src="image" alt="Von Mises Stress Variations" /></td>
<td><img src="image" alt="Crack Propagation" /></td>
</tr>
<tr>
<td>P3 Two Sections</td>
<td><img src="image" alt="P3 Two Sections" /></td>
<td><img src="image" alt="Von Mises Stress Variations" /></td>
<td><img src="image" alt="Crack Propagation" /></td>
</tr>
<tr>
<td>P4 Two Sections</td>
<td><img src="image" alt="P4 Two Sections" /></td>
<td><img src="image" alt="Von Mises Stress Variations" /></td>
<td><img src="image" alt="Crack Propagation" /></td>
</tr>
<tr>
<td>P5 Full Panel; All Sections</td>
<td><img src="image" alt="P5 Full Panel; All Sections" /></td>
<td><img src="image" alt="Von Mises Stress Variations" /></td>
<td><img src="image" alt="Crack Propagation" /></td>
</tr>
</tbody>
</table>

Fig. 3. Preliminary FE analysis of corrosion process

Fig. 4. Configuration of panel edge reinforcement
Panel Edge Distance and Reinforcement Spacing
Specifications for the panels investigated indicated that the side edge distance to the center of prestressed reinforcement ranged to accommodate the construction of panels with different widths; thus, concrete side cover also ranged accordingly. Specifications also showed that the lifting hooks, fabricated from mild reinforcing steel bars of length approximately the length of panel, were present within the panel and could vary in location with respect to the panel width. Lifting hook bars were aligned in the direction of the tendons, and the location was specified to be halfway between adjacent tendons at panel mid-depth. Accordingly, a parametric study using 2D FE simulations was carried out with respect to different combinations of side cover and spacing of steel reinforcement, including location of the lifting hook bar. Section thickness was maintained at 75 mm (3.0 in). Fig. 4 illustrates the configuration of the panel simulated in the FE models. As illustrated in the figure, only the local area of the panel was considered because of symmetry of the panel. Assumptions were made with respect to boundary conditions, loading conditions, and element type as explained in the following discussion.

Fig. 5 shows FE models depicting the parameters considered. A three-node triangular isoparametric plane stress element (T6MEM in DIANA) was used as illustrated in Fig. 5. Each void in the model represents the location of embedded reinforcing steel of the same size as the corresponding reinforcement. Incremental internal pressures of the same magnitude were applied to the reinforcement locations assuming the same corrosion level of all reinforcement. Note that although this assumption is not consistent

![Fig. 5. 2D FE models for corrosion-induced cracking (1 in = 25.4 mm)](image)

<table>
<thead>
<tr>
<th>Applied Pressure</th>
<th>Crack Propagation</th>
<th>Applied Pressure</th>
<th>Crack Propagation</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.8 MPa 400 psi</td>
<td><img src="image" alt="Image" /></td>
<td>13.8 MPa 2000 psi</td>
<td><img src="image" alt="Image" /></td>
</tr>
<tr>
<td>8.3 MPa 1200 psi</td>
<td><img src="image" alt="Image" /></td>
<td>15.2 MPa 2200 psi</td>
<td><img src="image" alt="Image" /></td>
</tr>
<tr>
<td>12.4 MPa 1800 psi</td>
<td><img src="image" alt="Image" /></td>
<td>16.5 MPa 2400 psi</td>
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</tr>
</tbody>
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![Fig. 6. Crack propagation of case I FE model [edge distance = 40 mm (1.5 in)]](image)
with the deterioration observed from the field investigations (Sneed et al. 2010; Wieberg 2010), it represents a critical situation to stimulate the spalling problem in the panel. The right surface of the model in Fig. 5 was a fixed boundary condition to simulate continuity of the section. The top surface of the model was also modeled as a fixed boundary condition assuming full composite action between the SIP panel and CIP concrete deck. Location of the lifting hook bar (labeled Hook Bar) is the distinction between the two FE models shown in Fig. 5. The case I figure illustrates the FE model considering the lifting hook bar located between the first and second tendons from the panel side edge, with a tendon spacing of 100 mm (4 in). With the lifting hook bar in the center of the tendons, the resulting spacing between reinforcements is 50 mm (2 in). In the case II figure, the lifting hook bar is located between the second and third tendons, and the tendons are spaced at 115 mm (4.5 in) on center. Thus, with the lifting hook bar in the center of the tendons, the spacing between reinforcement is 57 mm (2.25 in). Case I represents a critical configuration of reinforcement arrangement that can occur, whereas case II represents a typical reinforcement arrangement for the panels considered. Side cover was varied from 40 to 100 mm (1.5 to 4 in) in both case I and case II models.

Fig. 6 shows a representative crack propagation of the case I (critical configuration) model with 40 mm (1.5 in) side edge distance to tendon at increasing levels of internal pressure. First, cracking is observed when the internal pressure reaches 2.8 MPa (400 psi). As the internal pressure increases, cracks propagate around the reinforcement locations similarly to the preliminary FE analysis in the previous section. A very interesting phenomenon happens when the internal pressure exceeds 13.8 MPa (2,000 psi). Cracks at the lifting hook bar location propagate laterally to both adjacent reinforcement locations and finally join with cracks from the adjacent reinforcement. This kind of crack propagation is usually called a bridging crack. After the formation of this bridging crack at 15.2 MPa (2,200 psi), cracks propagate in random directions at 16.5 MPa (2,400 psi). Cracks finally reach the bottom surface and nearly reach the side surface. This final stage of crack propagation is bidirectional in nature and can be considered to represent spalling of the section. These results show that the formation of the bridging crack plays a role in triggering spalling in the panel.

The same phenomenon is observed in other case I models at the final stage as shown in Fig. 7. Excessive crack propagation is observed in each FE model after formation of the bridging crack. However, propagation of cracks from the edge tendons to the side surface is not observed because the edge distance (and corresponding side cover) is greater than 50 mm (2.0 in) in the models. In addition, cracks directed to the bottom surface are significantly diminished with increasing side cover.

Fig. 8 shows crack patterns of the case II (typical configuration) model at the final stage corresponding to 17.9 MPa (2,600 psi). The crack patterns are very different from the crack patterns of case I and are similar with those of the preliminary FE analysis shown in

<table>
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<th>Applied Pressure</th>
<th>Crack Propagation</th>
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<td>15.2 MPa 2200 psi</td>
<td><img src="image1.png" alt="Image" /></td>
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<td><img src="image2.png" alt="Image" /></td>
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<tr>
<td>16.5 MPa 2400 psi</td>
<td><img src="image3.png" alt="Image" /></td>
<td>16.5 MPa 2400 psi</td>
<td><img src="image4.png" alt="Image" /></td>
</tr>
<tr>
<td>17.9 MPa 2600 psi</td>
<td><img src="image5.png" alt="Image" /></td>
<td>17.9 MPa 2600 psi</td>
<td><img src="image6.png" alt="Image" /></td>
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</table>

**Fig. 7.** Crack patterns of case I FE model [edge distance = 50 and 75 mm (2.0 and 3.0 in)]

<table>
<thead>
<tr>
<th>Applied Pressure</th>
<th>Crack Propagation</th>
<th>Applied Pressure</th>
<th>Crack Propagation</th>
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<tbody>
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<td>16.5 MPa 2400 psi</td>
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<td>16.5 MPa 2400 psi</td>
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</tr>
<tr>
<td>17.9 MPa 2600 psi</td>
<td><img src="image9.png" alt="Image" /></td>
<td>17.9 MPa 2600 psi</td>
<td><img src="image10.png" alt="Image" /></td>
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<td>17.9 MPa 2600 psi</td>
<td><img src="image11.png" alt="Image" /></td>
<td>17.9 MPa 2600 psi</td>
<td><img src="image12.png" alt="Image" /></td>
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</table>

**Fig. 8.** Crack patterns of case II FE model [edge distance = 40, 50, and 75 mm (1.5, 2.0, and 3.0 in)]
Fig. 3. Increased distance between adjacent reinforcement near the side edge surface is shown to prohibit the crack propagation between adjacent reinforcement (bridging cracks). Additionally, effects of side cover are found to be insignificant in this case.

**Butting Effects**

The case II model in the previous section (representing the typical reinforcement arrangement) showed that spalling may not occur at the concrete edges with minor crack propagation unless another mechanism exists to form the critical bridging crack. Thus, external effects on panel edges were investigated to determine whether butting effects contribute to concrete spalling in combination with internal cracks induced by corrosion of embedded reinforcement.

To encompass various conditions, several parameters were taken into account including side cover, boundary conditions, and butting pressure magnitude and distribution as shown in Fig. 9. The case II model described previously was used for this simulation. On the basis of the results of the case II corrosion cracking analysis, 50% of the ultimate internal pressure 9.0 MPa (1,300 psi) was applied at each reinforcement location to simulate a given amount of corrosion at each reinforcement location. The butting load was modeled by incremental pressure on the joint side edge of the panel.

<table>
<thead>
<tr>
<th>Top Surface Boundary Condition</th>
<th>Side Pressure Distribution</th>
<th>Applied Pressure</th>
<th>Crack Propagation</th>
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<td>Restrained Vert. and Horiz.</td>
<td>Full-thickness</td>
<td>11.0 MPa 1600 psi</td>
<td></td>
</tr>
<tr>
<td>Restrained Vert. and Horiz.</td>
<td>Bottom half-thickness</td>
<td>13.8 MPa 2000 psi</td>
<td></td>
</tr>
<tr>
<td>Restrained Vert.</td>
<td>Full-thickness</td>
<td>20.7 MPa 3000 psi</td>
<td></td>
</tr>
<tr>
<td>Restrained Vert.</td>
<td>Bottom half-thickness</td>
<td>27.6 MPa 4000 psi</td>
<td></td>
</tr>
<tr>
<td>Restrained Vert.</td>
<td>Bottom edge</td>
<td>35.9 MPa 5200 psi</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 10. Final crack pattern of 2D FE butting analysis
Varying pressure distributions (in terms of height) of constant magnitude were used to account for varying misalignment of the panels. Delamination of the interface region between the SIP panel and CIP deck was also considered by releasing the horizontal restraint at the upper surface of the model as discussed in the previous section. Note that although the assumption of free horizontal movement is extreme in representing delamination in this region, it serves to achieve the goal of maximizing the effects of butting on spalling.

On the basis of the model described above, final crack patterns are shown in Fig. 10. As shown in the figure, simulations that included a fixed boundary condition at the top surface (corresponding to full composite action) show only local failure at the top edge surface regardless of the applied pressure distribution. In contrast, simulations that included a released boundary condition at the top surface (corresponding to complete delamination at the SIP-CIP interface) are strongly affected by applied pressure resulting in a variation of failure modes.

**Discussion**

The preliminary simulation conducted showed distinct differences in crack propagation to the bottom surface at the panel edges in exterior and interior sections, with exterior sections being more severe. However, bidirectional corrosion-induced crack propagation was not simulated for the typical panel geometry specified for the panels under consideration. Thus, other factors must be present to trigger spalling.

A difference in cracking behavior is observed when reinforcement spacing decreases attributable to miscellaneous reinforcement present in the panel, such as the lifting hook bar. The formation of bridging cracks was shown to play a role in triggering bidirectional cracking at the panel edge (representing spalling of the panel), on the basis of a comparison of the preliminary and case I models. Recall that case I represents a critical configuration of reinforcement arrangement that can occur. After formation of the bridging crack, excessive crack propagation is observed in each case I FE model, with final stages that are bidirectional in nature extending to both the vertical and horizontal edges of panel.

Without a doubt, increasing concrete cover increases the corrosion protection of embedded reinforcement. The case I (critical configuration) model showed that propagation of bridging cracks to the side surface was precluded when the edge distance (and corresponding side cover) exceeds a limiting value [50 mm (2.0 in) in this simulation]. Thus, concrete side cover plays a critical role in arresting bridging crack propagation to the side surface of the panel, even in situations with closely-spaced reinforcement.

Note that the case I situation is an atypical situation, and the case II situations are more common. Although conditions in the case II model did not trigger the spalling phenomenon, the significance of the bridging crack in the case I model is a key finding. Note that in this analysis, the bridging crack was formed from crack propagation between closely spaced reinforcement induced by corrosion. However, another mechanism exists to aid in the formation of the bridging crack. As illustrated in Fig. 11, corrosion of temperature reinforcement oriented perpendicular to the prestressing tendon direction can induce cracks along the length of the temperature reinforcement playing a similar role as the bridging crack. Although this phenomenon was not simulated in this 2D analysis, convincing physical evidence exists of a link between corrosion of temperature reinforcement and the bridging crack. Most of the spalled panels observed in the field survey exposed heavily corroded temperature reinforcement in addition to corroded edge tendons [see Fig. 1(b)]. Thus, both an analytical approach and field investigation conclude that the bridging crack is a primary agency for the corrosion-induced spalling problem in the panel.

Finally, results from the case II model showed butting of adjacent panels, particularly in combination with delamination at the SIP-CIP interface, were shown to contribute to the spalling problem. Determining the most critical case among the models with a released boundary condition is difficult because ultimate pressures differ in magnitude with respect to the different distributions considered. Clearly, however, delamination of the SIP-CIP interface plays a key role in the contribution of butting to the spalling problem. Evidence of delamination at the SIP-CIP interface of spalled panels was found during field investigations discussed previously. In addition, misaligned panels were also observed during field investigations shown in Fig. 2.

**Conclusions**

Many states in the United States have used PPC panels in bridges. Design specifications and details differ from state to state with respect to geometry, material properties, and reinforcement details. Critical combinations of these factors can lead to long-term serviceability problems. This study aimed to investigate the causes and mechanisms of concrete spalling observed in the PPC panels investigated through parametric study using FE simulations. Panels investigated in this study were 75 mm (3.0 in) thick and 2.4 m (8 ft) wide with a tendon spacing of 100–115 mm (4–4.5 in). On the basis of this work, design specifications and details of PPC panels with other combinations of factors can be examined in a similar manner to determine the susceptibility to these problems. The results of this evaluation led to a subsequent study to develop corrosion-resistant deck panels utilizing corrosion-free materials such as FRP bars or epoxy-coated tendons. On the basis of the findings, the following conclusions are made.

1. Preliminary FEM simulations in this study indicated that the typical arrangement of prestressing tendons in a deck panel produced only localized cracking around the tendons and bottom surface. Lateral crack propagation to side surfaces or adjacent tendons was not observed. This phenomenon was attributable to the restraint effects caused by geometrical conditions governing the stress release observed in Von Mises stress variations.
2. Further simulations showed that initiation of a bridging crack between adjacent tendons is a primary agency to trigger corrosion-induced spalling of the panel. The bridging crack is triggered by closely-spaced reinforcement, which can be the result of the presence of miscellaneous reinforcement in the panel, such as the lifting hook bar. The significance of the bridging crack is a key finding of this analysis because corrosion of temperature reinforcement oriented perpendicular to the tendon direction can also induce cracks along the temperature reinforcement, playing a similar role to the bridging crack.

3. For panels of a specified thickness, increasing the bottom cover is not a solution to mitigate spalling. However, for a given thickness increasing amounts of side cover showed beneficial effects on mitigating the spalling problem by detaining crack propagation to the side surface of the panel regardless of reinforcement arrangement.

4. The results showed butting of adjacent panels in combination with delamination of the SIP-CIP interface, also shown to contribute to the spalling problem.

Publisher’s Note. The reviewers of this paper requested additional changes that were not included in the original preview manuscript. The authors responded to the comments, and their updates appear in this version of the paper.

Acknowledgments

This project is funded by the Missouri Department of Transportation (MoDOT) and the National University Transportation Center (NUTC) at Missouri University of Science and Technology (Missouri S&T). The funding is gratefully acknowledged.

References


