INSPECTION AND STRUCTURAL EVALUATION OF THE WILLIAM POWELL BRIDGE IN MIAMI-DADE COUNTY, FLORIDA

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ABSTRACT

The William Powell Bridge has been an integral component of the Rickenbacker Causeway since 1985, providing a direct link from mainland Miami to Virginia Key and the resort island of Key Biscayne. Though the structure is considered a young bridge and recent routine inspections revealed mostly minor deficiencies consistent with construction of this era, the presence of unanticipated cracks in several of the pier caps has prompted Miami-Dade County to investigate the cause of these abnormalities and the effect they have on the intended performance of the bridge.

Phase I of the project consisted of a condition inspection of the pier caps to document crack patterns and an analytical component to determine the cause of the fractures and the repercussions of such on the serviceability of the bridge. The magnitude of cracking observed was not typical and indicative of member overstress, signifying flexural and flexural-shear deficiencies. Structural analyses determined the probable failure to consider deep beam behavior and inadequate reinforcement detailing led to the cracks seen today.

Preparations are currently underway for Phase II, which will involve the installation of crack gauges on a sample of the pier caps to determine if the cracks are propagating or if they have arrested. Only by knowing the status of the cracks can it be determined if internal equilibrium has been achieved, and proper rehabilitation schemes identified and advanced.

Keywords: Bridge, Cracking, Deep Beam, Inspection, Pier

INTRODUCTION

Cracking of reinforced concrete members is a reality structural engineers have acknowledged since the inception of the material. Fine cracking is a common occurrence that is usually not a sign of structural failure, but rather is a necessity to transfer internal forces. Excessive cracking of a member, however, is not typical and is a condition that needs to be investigated. Such a condition usually signifies component overstress and can provide a means for moisture, chlorides, and other harmful substances to access the reinforcing steel, leading to corrosion and section loss, and eventually resulting in reduced member capacity.

The concrete pier caps of the William Powell Bridge contain excessive cracking with abnormal patterns and magnitude that do not reflect what is accepted as typical behavior. The majority of the cracking is concentrated over the exterior columns, particularly on the south side of the structure, exemplifying flexural and flexural-shear deficiencies. This condition has prompted Miami-Dade County to conduct a study to investigate the cause of the cracks and the effect they have on the integrity and serviceability of the bridge. The findings of that study are presented in this paper.

BRIDGE DESCRIPTION AND STRUCTURAL CHARACTERISTICS

The William Powell Bridge (Photo 1) is a high-level crossing of the Intracoastal Waterway and a dominating feature of the Rickenbacker Causeway, which provides a link from the Miami mainland to Virginia Key and the resort island of Key Biscayne. Rising to a height of over 90’, the thirty-five span structure carries three westbound traffic lanes, two eastbound traffic lanes, two 8’ shoulders, two 4’ shoulders, and an 8’ sidewalk across the navigable portion of the Intracoastal Waterway. Constructed in 1985, the 3611’ long bridge replaced the outdated structure immediately to the south, which has since been restricted to pedestrian traffic and converted to fishing piers.

Photo 1. Elevation of William Powell Bridge

The causeway superstructure consists of 14 simple and continuous multi-span units comprised of a concrete deck slab supported by a multiple beam system. Thirty-four of the thirty-five spans are 102' in length and fashioned of AASHTO Type IV prestressed girders, while the section over the navigation channel is 143' and spanned by Type VI girders. Substructure units consist of reinforced concrete stub abutments and 4-column concrete pier bents supported by prestressed and steel piles. Bridge components were designed for the HS 20 live loading, as was appropriate at the time of design.

The structure serves a moderate volume of traffic predominantly consisting of light cars and trucks, though heavier vehicles such as garbage and tri-axle dump trucks frequent the bridge. The number of users is expected to dramatically increase over the years, from an average daily traffic value of 33,007 vehicles per day reported in 2004 to a predicted value of 53,091 vehicles per day in 2024. In addition to the motoring public, the causeway accommodates respectable pedestrian and bicycle traffic.

At the heart of the investigation are the reinforced concrete pier caps, which are 4' wide, and vary in depth from 4.5' between columns to 3' at the tapered cantilever end. Columns are spaced at a center-to-center distance of 24', and the cantilevered overhang extends 9'-9" from the centerline of the exterior columns. Primary longitudinal reinforcement consists of fourteen No. 11 bars in the negative moment region over the exterior column, and nine No. 11 bars in the positive moment region between columns. Additional reinforcement includes two sets of No. 5 band stirrups at 6" spacing in the cantilever, two sets of No. 5 band stirrups at 9" and 12" spacing between the columns, and No. 5 skin reinforcement. Figure 1 provides details of this reinforcement.

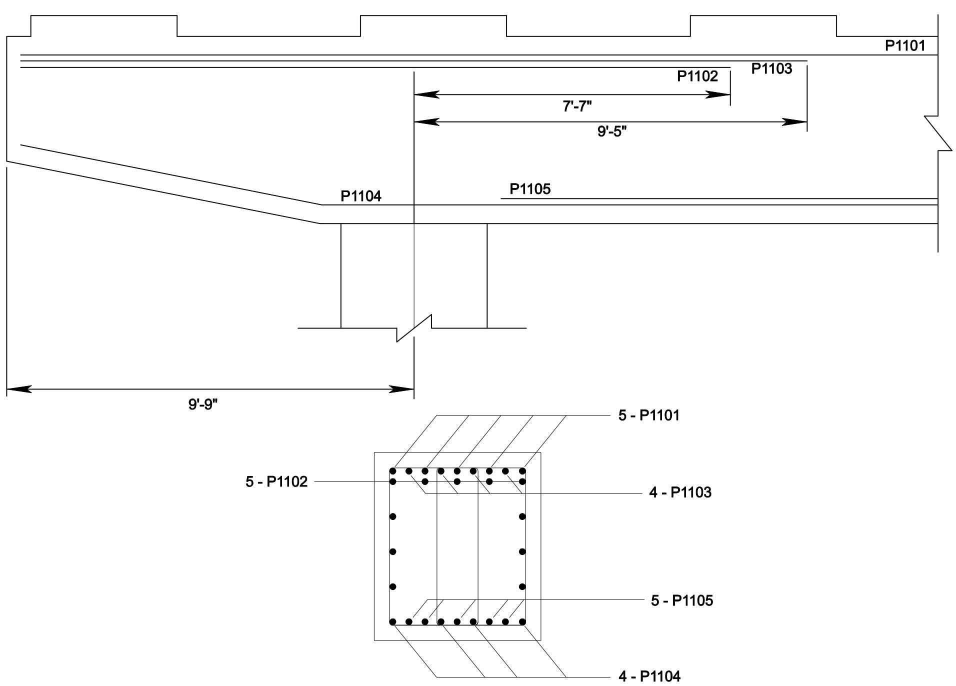

Figure 1. Typical Cap Reinforcement

EXISTING CONDITION

The report summarizing the findings of a routine inspection conducted on June 30, 2006 presented no significant defects that jeopardize the serviceability of the causeway. The bridge is neither listed as structurally deficient nor functionally obsolete, and no weight restrictions are currently in place. With exception of the cracks in the pier caps, only typical deterioration of elements from this era was noted. The inspection team noted the cracks in the pier caps, but did not elaborate on their presence or list them as a critical condition requiring immediate attention. The overall condition ratings assigned to the structure in 2006 are listed below:

Deck:	7 - Good
Superstructure:	6 - Satisfactory
Substructure:	6 - Satisfactory
Channel:	7 - Minor Damage
Sufficiency Rating:	80.0

No documentation exists that identifies when the cracks first appeared, though the earliest inspection report that could be acquired (dated June 24, 2004) refers to the cracks and contains a repair recommendation calling for their monitoring. Though excessive in density, the cracks do not seem to be propagating at an alarming rate since widths are listed as 1/64" in both the 2004 and 2006 reports.

FIELD INSPECTION OPERATIONS

A condition inspection of the William Powell Bridge was performed from September 24 - 28, 2007. This operation required the use of an Aspen Aerial UB-50 snooper under bridge inspection vehicle to access the caps (Photo 2) and the closure of one traffic lane, which was accommodated through an MOT plan based on the provisions presented in FDOT standards.

Photo 2. Access to Pier Caps with Aspen Aerial

The goals of the field work were threefold: 1) to document the extent and severity of cap cracking, 2) to determine the integrity of the pier cap concrete, and 3) to compare cap dimensions and reinforcement layout with that presented in the original contract drawings.

Inspection efforts were limited to a portion of each pier cap extending from the cap end to +/- 8' beyond the centerline of the first interior column (Figure 2). A total of eight caps were studied, and all surfaces (top, bottom, and sides) were examined. Tasks included hammer sounding the concrete for abnormalities, crack measurement, and the logging of observed deficiencies on field inspection forms. Additionally, cap dimensions were obtained and ground penetrating radar scans conducted to map the steel reinforcement. This information was used to establish conformance with the as-built drawings.

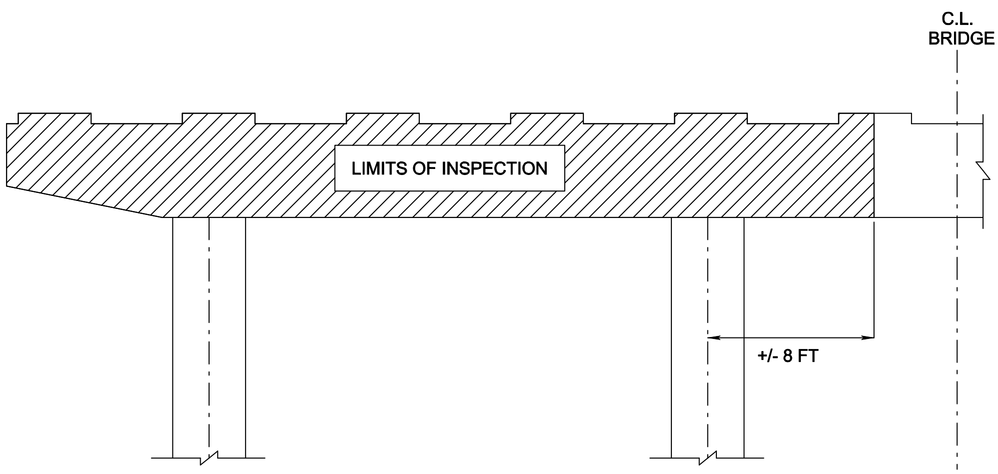

Figure 2. Limits of Inspection

Initially it was believed cracking would be most severe on the north side of the bridge at a cap of a tall interior bent within a typical three span continuous unit. The north side of the bridge was originally chosen for two reasons. First, three west bound lanes are on the bridge. Closing a lane for accessibility would maintain two travel lanes, whereas traffic would be restricted to one lane on the south side. Second, there is not a sidewalk present on the north side, allowing traffic to directly load the fascia girder and, therefore, the cap cantilever. Hence, it was anticipated the north side of the bridge would be more heavily loaded than the south, and that cracking would be more significant. It was desired to investigate a tall bent since such piers are exposed to increased wind pressures, resulting in increased lateral load and induced moments in the caps due to frame action with the columns. An interior pier was chosen due to the increased superimposed dead and live load girder reactions attributable to continuity.

Field work initiated with the inspection of the north side of Pier 22, which is one of the taller interior piers. Though the north side of the first two piers inspected exhibit unusual crack patterns and densities, crack widths were generally fine and occurrences somewhat sporadic. While under the bridge, cracks were clearly visible on the south side of the caps and appeared much more abundant. At the end of the first day, a coarse visual inspection of the south side of the pier caps was conducted from the adjacent fishing pier and it was discovered these areas indeed demonstrate greater signs of distress than those to the north. Hence, it was decided to modify the inspection plan to more closely examine the condition of the elements at these locations. As field operations progressed, it was confirmed the south side contains more pronounced cracking, and it was determined to focus on the south side for the remaining cap inspections and evaluation. A list of the piers and the location examined is provided in Table 1 below.

Table 1. Inspected Elements

Element	**Location**
Pier 18	North Side
Pier 22	North Side
Pier 23	South Side
Pier 24	South Side
Pier 25	South Side
Pier 29	South Side
Pier 30	South Side
Pier 32	South Side

*Piers are numbered from west to east

A possible cause for the formation of the observed cracks is failure to provide the quantity of reinforcement called for in the design documents. To verify the reinforcement layout placed in the pier caps, non-destructive technology known as ground penetrating radar (GPR) was used. GPR is a proven method of detecting voids, cracks, and changes of material within a given medium by sending pulses of electromagnetic radiation through and recording the signals reflected from subsurface abnormalities. The patterns of these recorded signals can be interpreted to map the cross section of an element, revealing information unattainable by the naked eye. Though this process is not an exact science in terms of determining bar diameters and locations, reasonable confidence in the amount and size of reinforcement contained within a concrete element can be acquired if performed correctly. An occurrence as major as the omission or misplacement of steel would be obvious when reviewing the data.

GPR surveys were conducted on the south half of pier caps 25 and 32 (Photo 3). These caps exhibit extensive cracking and appeared to be good candidates for such investigations. Vertical and horizontal scans were made on the side and bottom faces of the caps to determine the number and approximate size of longitudinal and transverse reinforcement, as well as to assure the soundness of the concrete. The GPR study confirmed the steel provided in the pier caps is in conformance with the design documents.

Photo 3. GPR Scan of Representative Pier Cap

Field Inspection Findings

Substantial cracking of the pier caps over the exterior columns, in the cantilever, and extending into the first bay between columns was observed in the field, with cracking on the southern side more prevalent than that to the north. Flexural and flexural-shear cracks were the dominant patterns documented, although isolated locations of shear, shrinkage, and surface map cracks were noted.

Crack types in concrete members are identifiable by the path of the crack growth. Flexural-shear cracks intersect the tension side of a member and propagate along the flow of stress until forces have dropped below the tensile resistance of the concrete, mimicking the shape of an arc. Flexural cracks also intersect the tension side of a member, but grow vertically, not diagonally as do flexural-shear cracks. Shear cracks are characterized by their 45° angle and occurrence at approximately the member's depth from the support. Orientations of these common cracks found are shown in Figure 3.

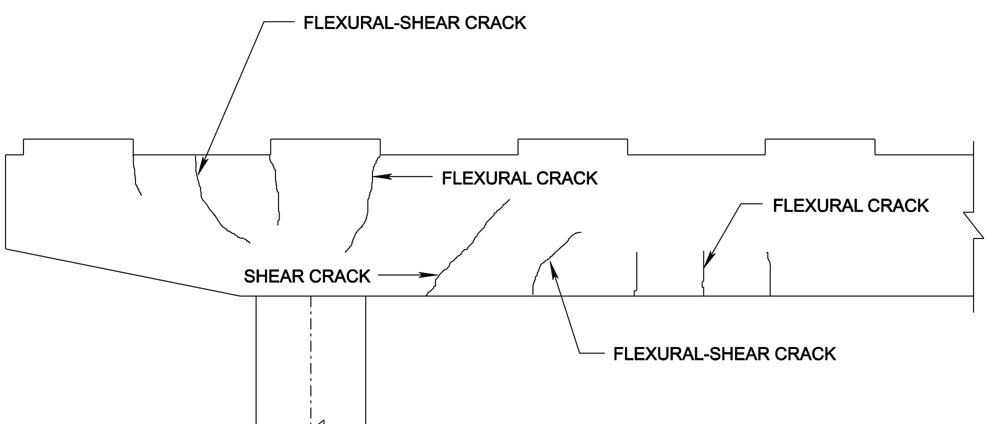

Figure 3. Typical Observed Cracks

The flexural cracks present in the bottom of the caps between columns (Photo 4) are not a major concern in terms of the integrity of the structure. At these locations the stress trajectories are relatively flat given that shear stresses are low, and longitudinal reinforcement is provided in the same plane to resist tension the cracked concrete is no longer able to carry. In fact it is necessary for concrete to crack in order to transmit significant force to the reinforcing steel, and these cracks are evidence of that. The behavior at these locations is similar to that of shallow beams, which was well defined in the codes used to design the caps. Cracks in this region were less than 0.010”, which is in the expected range of normal behavior and fine enough to limit the penetration of detrimental substances that can corrode the steel.

Photo 4. Typical Cracks in Positive Moment Region Between Columns

The cracks present in the cantilever and over the columns, specifically over the exterior columns on the south side, are a little more significant (Photo 5). The magnitude and pattern of these cracks are indicative of reinforcement overstress and/or inadequate development, for what was observed is not typical behavior. Most of the cracks are fine, with widths less than 0.010” common, though a limited number of widths up to 0.060” were measured. The current edition of the *AASHTO LRFD Bridge Design Specifications* limits crack width to 0.017”, a value that is significantly exceeded in these localized areas. Many of the cracks contain leakage with white and brown efflorescence, especially those located at piers supporting an expansion joint. The concrete was sound around these cracks, inferring that excessive corrosion of the reinforcement has not yet developed. It does, however, signify breakdown of the epoxy coating and deterioration of the reinforcement is occurring.

Photo 5. Typical Crack Pattern in Cantilever and Over Exterior Column

Though documentation presenting the history of these cracks does not exist, some assumptions can be made as to their origin. Many of the cracks propagating from the top of the cap near the bearing seats could have occurred during construction, being associated with early shrinkage due to the abrupt change in section and/or early form removal. The cracks would have initially been small, and grew due to the application of additional loads and the creep effect of concrete. Map cracking may be the result of insufficient consolidation of the concrete, leaving an imperfect finish susceptible to such deficiencies. The fact that cracking is more prevalent on the south side can be attributed to meteorological conditions. The south side is exposed to direct sunlight, resulting in the potential for extreme volume changes due to thermal effects not experienced on the shaded north side. With time, this cyclic load could exacerbate cracks already in existence.

Findings in the field led the team to believe the effects of frame action due to the interaction of lateral loads with the columns are insignificant since the cracking issue does not discriminate between taller and shorter bents. Both tall and short piers possess caps that contain cracking, and from what was seen in the field, one grouping does not appear to have more significant issues than the other. This assumption was verified through the analysis, as it was shown a load condition without the contribution of wind produced maximum cap forces. Along the same lines, the effect of live load seems to be irrelevant. The north side of the structure experiences a greater live load reaction than that to the south, whereas the south side exhibits more severe cracking. These findings seem to justify live loads are not a major player in the emergence or growth of these cracks.

While in the field it was also interesting to note cracks have formed adjacent to the exterior pedestal as well as at mid length of the cantilever. This phenomenon is of particular interest since these regions are not areas of extreme moment, indicating that inadequate reinforcing development may be the driving force behind the presence of the cracks. Pier cap cantilevers perform differently than typical beam members due to their dimensional ratios, though it was common practice in the past to ignore this in design. Misunderstanding of this behavior leads to a misrepresentation of the internal forces and possibly an under reinforced section in areas believed to be under little stress. The appearance of these cracks prompted the team to closely examine the cantilever reinforcing details to assure they are adequate to resist this unique force flow. The specifics of proper reinforcement development are further discussed in the analysis section of this report.

PIER CAP ANALYSES

General Behavior Principles

The behavior of members subjected to bending and shear can be grouped into two main categories: shallow beam and deep beam behavior. Shallow beams represent the typical case when one thinks of beam behavior. They possess large span-to-depth ratios and abide by flexural provisions founded in conventional beam theory, which rely on an internal couple consisting of a compressive force from the concrete and a tensile force from the reinforcing steel to resist applied moment. Deep beams are governed by a different set of rules and do not carry load in the same manner as their shallow beam counterparts, displaying a nonlinear stress distribution. These elements have much smaller span-to-depth ratios and resist a significant amount of load by an internal compression thrust joining the load to the reaction. This force flow, known as arching, is really the disruption of horizontal shear flow from the longitudinal steel to the compression zone that alters behavior from normal beam action. When this action is achieved, an internal truss mechanism is formed, with the compression strut representing the arch action accompanied by a tension tie. Hence, modeling a deep beam as a truss via strut-and-tie principles is an effective way to acquire internal forces.

The geometrics of pier cap cantilevers are typically such that they are representative of those associated with the deep beam classification. Uniformly loaded elements are considered deep beams if they possess a span-to-depth ratio less than 4, and concentrated loaded members fall into this category if the load is applied within 2D of the support. For the cap cantilever, the span-to-depth ratio is 1.8 and the beam reaction acts at a distance of 5.7’ from the face of the column, which is less than 2D, or 9’. Following these criteria, it is clear the cap cantilever should be evaluated using deep beam provisions.

Imagining the internal truss analogy, the distribution of flexural stresses suggests the force in the longitudinal tension tie is constant along the length of the cantilever. Here the tension reinforcement serves as the tie, and must be able to resist a uniform tensile force from the point of load application (fascia girder reaction) to the assumed point of fixity (centerline of exterior column). The most common mode of failure for a deep beam is an anchorage failure at the end of the tension tie. The final failure of such beams is caused by bond failure, splitting failure, dowel failure along the tensile reinforcement, or by crushing of the compression zone over the crack. Adequate reinforcement development must be provided to assure the proper resistance of force.

The real concern in deep beams is diagonal tension stress, which is a combination of shear stress and longitudinal flexural stress. In regions of high moment and shear, flexural cracks typically are the first to form and are controlled by the provided longitudinal reinforcement. This reinforcement, however, does not reinforce the tensionally weak concrete against diagonal stresses that occur elsewhere in the member caused by shear or the combined effect of shear and flexure. Additionally, vertical reinforcement does not prevent inclined cracks from forming – they come into play only after cracks have formed. Because of the orientation of the principal stresses in deep beams, when diagonal cracking occurs, it will usually be at a slope steeper than 45 degrees. Consequently, while it is important to include vertical stirrups, they are apt to be less effective than in elements dominated by shallow beam behavior. Once the diagonal tension stress at the tip of one of these cracks exceeds the tensile strength of the concrete, the crack bends in the direction perpendicular to local stresses and continues to propagate. The cracks observed in the field reveal this behavior, as can be seen in Figure 4 below. Notice how the cracks are perpendicular to the tension stress trajectories.

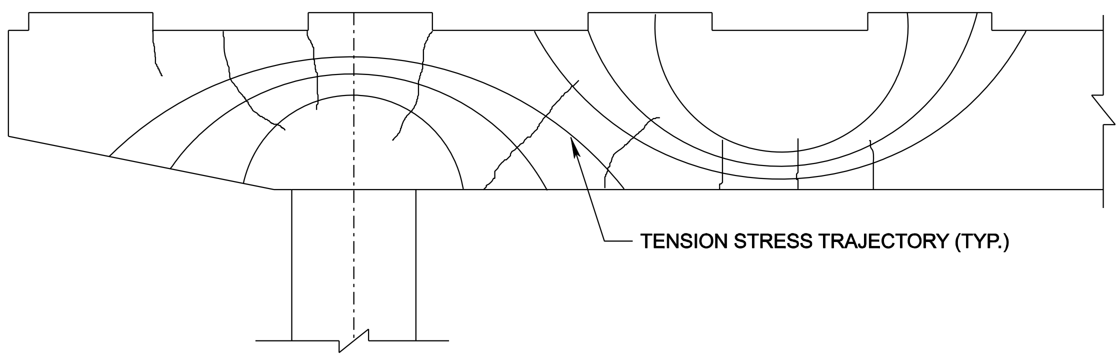

Figure 4. Principal Tension Stress Trajectories and Observed Cracks

Diagonal cracks affect the load carrying capacity of a member in several ways. First, since the inclined crack generally extends higher into the beam than a flexural crack, failure occurs at less than the flexural moment capacity. In fact, cracking will occur at 1/3 to 1/2 the ultimate load in a deep beam. Second, the presence of cracks reduces the area of uncracked concrete available to resist shear, increasing the need for shear reinforcement. Additionally, it has been experimentally shown that large bending moments can reduce the shear force at which diagonal cracks form to roughly half the value they would form if no bending was present, again increasing the need for shear reinforcement. Though a few shear cracks were identified in the caps, no signs of premature shear failure were evident in the field, such as splitting failures at the level of the flexural tensile reinforcement or crushing of the concrete due to combined shear and compression, signifying flexural issues appear to dominate.

Limited guidance provided by past design specifications has led to inadequately designed and detailed members. Assumptions regarding behavior of certain elements believed to be legitimate are now found to be invalid. Before deep beam behavior was better understood, older design codes required all beams be designed using shallow beam theory.

Acknowledging this weakness, current design specifications (ACI and AASHTO) now require that beams with large depth-to-span ratios be designed using deep beam provisions.

Analytical Models

A series of 2-D and 3-D computer models were developed to simulate the behavior of the pier caps under several load conditions in an effort to identify the mechanisms leading to the observed cracking and to determine their adequacy to perform as required. The most important aspect of this project is to assure the structural integrity of the bridge has not been compromised, and with that in mind, the first analyses performed were those regarding the ultimate strength of the caps. Models were created to generate and distribute loads to the piers, as well as to obtain internal force effects, particularly moments and shears, in the caps themselves. Dead loads accounting for all permanent structural components were identified, and the HL-93 live load as presented in the AASHTO LRFD Specifications was used to acquire live load effects. Wind load as defined in AASHTO and supplemented by FDOT's *Structures Design Guidelines* was applied to capture the forces induced due to frame behavior between columns and caps. An interior pier of a typical three span unit was the subject of the investigations, warranting higher superimposed dead and live load reactions due to continuity. The model was built to the geometry of Pier 7 (Figure 5), which is the tallest pier without an intermediate strut bracing the columns. Therefore, it should produce the largest bending moments transferred into the pier cap. The results of this model were used to determine the flexural and shear adequacy of the section of the caps between the columns as outlined in the current AASHTO LRFD Specifications.

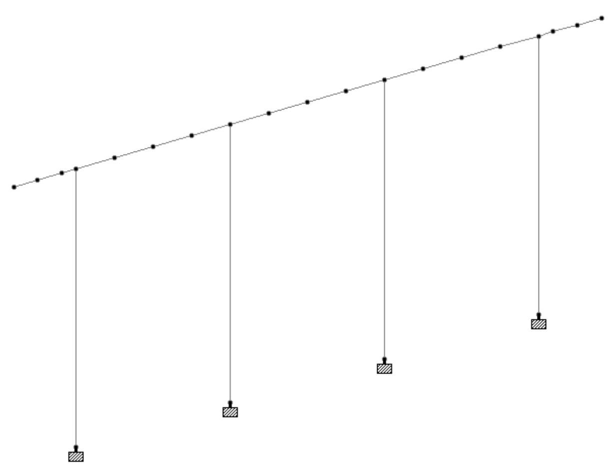

Figure 5. Ultimate Strength Model

The above procedure does not apply to the cantilevered portion of the pier caps, for provisions pertaining to deep beams need to be followed. Elastic analyses of deep beams are helpful in predicting where cracks will form, but are only meaningful prior to cracking. Once cracking has occurred, the internal elastic stress field is disrupted, resulting in a major reorientation of forces, and a different approach must be taken. Arch action has developed, and the most effective way to capture this behavior is through the use of a strut-and-tie model. With that said, such an analysis was performed using the controlling loadings determined through previous models (Figure 6). Tension tie forces were computed and used to determine the stress in the cantilever reinforcement and related to crack width parameters.

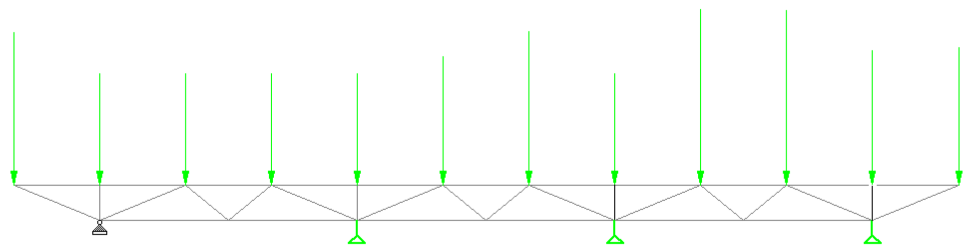

Figure 6. Strut-and-Tie Model

In addition to the capacity of the section, the cause of the cracks must be determined. As can be inferred from the previous discussions, flexural and shear stresses are critical in determining the location, orientation, and propagation of cracks in concrete members. To better understand what is causing the excessive cracking of the pier caps, it would be beneficial to determine where cracks are expected to form. An effective way to predict where cracks will occur in an uncracked concrete section is through the use of a 3-D finite element model, whose results show the flow and distribution of internal stresses that lead to cracking and, hence, give guidance as to the direction of cracking and the flow of forces after cracking. Such a model was created to develop this information and validate modeling techniques employed.

Analysis Findings

Stress contour diagrams were generated from the results of the finite element model to understand the flow of forces in the uncracked section and locate areas where cracks should theoretically occur. These pictorial representations serve as a pseudo map of internal force flow, depicting areas of varying stress concentration. As can be seen in the stress contour diagrams represented in Figure 7, the model accurately predicted the location and crack patterns noted in the field. The modulus of rupture of the cap concrete was calculated to be 443 psi, and the areas of red, orange, and yellow in the flexural stress diagram and those of green and blue for the shear stress diagram exceed this value, distinguishing areas where cracks will form.

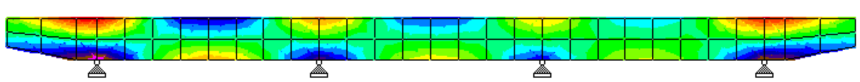

Figure 7. Flexural (Top) and Shear (Bottom) Stresses in Cap

As previously stated, the pier caps were analyzed to determine their adequacy to perform as intended, and through these studies it was found the positive moment reinforcement satisfies both strength and serviceability requirements. The provided area of steel is sufficient to resist the flexural and shear demands resulting from the controlling load case, which is the Strength I limit state, and service steel stresses and bar spacing are in conformance with that required to limit crack width. These analyses were executed via the semi-empirical shallow beam equations presented in AASHTO LRFD Specifications, as the span-to-depth ratio of this portion of the cap is such that this approach is valid.

Negative bending over the exterior columns is governed by the behavior of the cantilever, which is defined by deep beam action. The strut-and-tie model yielded a maximum service tension tie force of 825 kips, which is constant from the point of load application (bearing) to the support (centerline of column). Assuming all the steel is effective, this force corresponds to a steel stress of 38 ksi, a value resulting in significant steel strain, and in turn, substantial concrete cracking of moderate widths. Research has shown that a service load stress limit of 24 to 26 ksi in the tensile flexural reinforcement be used to ensure the durability of elements by limiting crack widths. The Florida Department of Transportation seems to have acknowledged this fact in section 3.10 of the *Structures Design Guidelines*, where it states "limit service tension stresses in longitudinal reinforcing steel for all mildly reinforced pier columns, pier caps, and bent caps under construction loading and Service III Loading to 24 ksi for Grade 60 reinforcing." This explains the excessive crack widths, but does not fully clarify the location of much of the cracking.

The above stress was calculated assuming the full area of steel is engaged, though it is evident the area of the steel provided is not effective along the entire length of the tension tie. Only 2' of development is provided prior to reaching the centerline of the fascia girder, which is the location of the theoretical node point for the tie. According to the current edition of the AASHTO LRFD Specifications, 9'-0" is required to develop the full capacity of the primary reinforcement. This number is based on several parameters. First, the reinforcing consists of #11 bars, which are capable of carrying considerable force that must be transferred thru the concrete. As the force to be transferred increases, so does the required development length. Second, the reinforcing bars are epoxy coated, reducing the bond strength between the concrete-bar interface and again leading to an increased development length. Lastly, hooks are not provided at the reinforcement cut-off locations. The use of a hook decreases development length over that of a straight bar since it induces a second mechanism, bearing of the bar against the concrete, in addition to bond to transfer force between materials. With all of this taken into account, it was determined the area of steel provided is not fully developed at any location in the negative moment region over the columns. Referring back to Figure 1, it can be seen that the length of the P1102 bars is less than two development lengths, or 18'-0", meaning that full bar development cannot be provided on both sides of a design section at any point along those bars. Since the reinforcement is not fully developed, it becomes less likely that insufficient steel area, which would result in large steel strains and hence cracking in the concrete, is the major player in this situation. Rather it is the failure to provide adequate means to transfer force from the concrete to the steel before the tensile strength of the concrete is exceeded – the mechanism at work here is inadequate bond between the steel and concrete.

CONCLUSIONS AND RECOMMENDATIONS

The cause of the cracks present in the pier caps can be traced back to limited understanding of the behavior of pier cap overhangs at the time of design, which resulted in inadequate reinforcement detailing. The actual stress distribution that occurs in pier cap cantilevers is different than what was assumed in common practices of the early 1980's. These elements tend to carry force through arch action, which mimics a truss with compression struts and tension ties. The tension tie, represented by longitudinal reinforcement, is subjected to a constant stress from the point of load application to the support, unlike the gradual increase of stresses demonstrated by shallow beam behavior. The primary reinforcement, consisting of #11 bars, is not developed along the full length of this tie, resulting in insufficient bond between the concrete and reinforcing steel and the observed cracking of the concrete in the pier cap overhangs and over the exterior columns.

Calculations do not indicate there is danger of catastrophic collapse, though the caps currently exhibit flexural and flexural-shear deficiencies. The behavior of such elements in this condition is difficult to predict, and corrective procedures can vary from minimal operations to the most extreme of retrofits. As a minimum, the cracks need to be monitored on a regular basis. Phase II of the project, which is currently underway, involves the installation of crack gauges on several of the pier caps so consistent measurements can be taken and maintained, allowing it to be identified if the cracks are continuing to propagate or if their growth has arrested. If the cracks have arrested, no further action is required with the exception of sealing the cracks with an FDOT approved penetrating sealant. If the cracks are continuing to grow, however, additional steps may have to be taken. Such action could include externally post-tensioning or carbon fiber wrapping the caps to increase their load carrying capacity. Concrete core samples can also be taken to determine the actual compressive strength of the concrete and, in turn, get a batter grasp of the ultimate load carrying capacity of the member.

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